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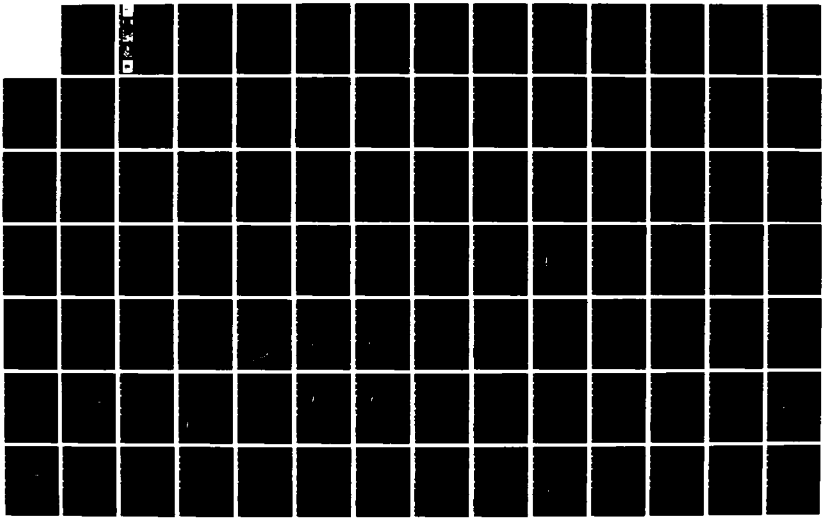

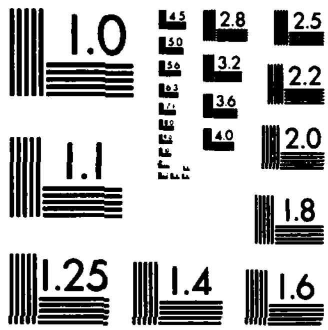
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US Army Corps of Engineers

TECHNICAL REPORT K-84-2

LATERALLY LOADED PILES AND COMPUTER PROGRAM COM624G

by

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When the soil immediately below the base of a structure will not provide adequate bearing capacity, piles can be used to transfer load from the structure to soil strata which can support the applied load. This report deals with analysis of the lateral interaction of pile shaft and soil. Examples of such problems encountered by the Corps of Engineers are single-pile dolphins and baffles for grade control structures.

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20. ABSTRACT (Continued).

A computer program called COM624, along with documentation, was developed at the University of Texas (UT) at Austin, to analyze laterally loaded pile problems. Analysis performed by Program COM624 is dependent upon soil parameters input to the program. These soil parameters take the form of curves which simulate the nonlinear interaction of the pile and the surrounding soil. The UT Report also presented criteria for developing these soil response curves in various types of soils.

This report consolidates the information available on laterally loaded pile analysis and provides supplementary data on Program COM624 (redesignated as COM624G). It describes modifications made in the input procedures and the addition of graphics options. Several examples of laterally loaded pile problems encountered in the Corps are added. Also included is a procedure for nondimensional analysis of laterally loaded piles which can be used to perform companion hand calculations to verify the results of the computer solutions.

PREFACE

This report reviews soil-structure interaction analyses of laterally loaded piles and provides supplementary documentation on a computer program COM624 developed by Prof. Lymon C. Reese, Nasser Al Rashid Professor, Civil Engineering Department, University of Texas (UT) at Austin, and Mr. W. R. Sullivan who was a graduate student at UT. Liberal use is made herein of material previously published by Prof. Reese and his graduate students.

Mr. Reed L. Mosher and Mr. Michael E. Pace of the Computer-Aided Design Group, Automatic Data Processing (ADP) Center, U. S. Army Engineer Waterways Experiment Station (WES), modified the original program to run in the time-sharing mode, added graphics options, and also restructured the input to the program. The modified program has been designated as COM624G. Messrs. Mosher and Pace prepared Appendix C which contains the input to the modified program. Mr. A. E. Templeton, Vicksburg District (VXD), ran all of the computer and hand-derived examples contained in this report. Contributions of all of the above are gratefully acknowledged.

Funds for this work were authorized by the U. S. Army Engineer Division, Lower Mississippi Valley (LMVD), as part of the analysis support provided by the WES ADP Center. Mr. James A. Young, Geology, Soils, and Materials Branch, LMVD, was the technical point of contact.

The work was accomplished during the period July 1981 through April 1983. This report was written by Prof. Reese, Mr. Larry A. Cooley, Chief, Foundation and Materials Branch, VXD, and Dr. N. Radhakrishnan, Special Technical Assistant, ADP Center, WES.

COL Nelson P. Conover, CE, and COL Tilford C. Creel, CE, were Commanders and Directors of WES during the course of the work and the preparation of this report. Mr. F. R. Brown was Technical Director.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic inches	16.3871	cubic micrometers
feet	0.3048	meters
feet per second	0.3048	meters per second
feet per second squared	0.3048	meters per second squared
foot-kips (force)	4.448222	kilonewtons
foot-pounds (force)	1.355818	joules
inches	2.54	centimeters
inches per pound	0.1129848	newton meters
inches to the fourth power	0.4162	micrometers to the fourth power
kips	4.4482	kilonewtons
kips per square inch	6.8497	megapascals
pounds per inch	175.1268	newtons per meter
pounds per cubic inch	27,679.9000	kilograms per cubic meter
pounds per square inch	6.8948	millipascals
pounds per cubic foot	16.0185	kilograms per cubic meter
pounds per square foot	4.8824	kilograms per square meter
tons (force)	8.8964	kilonewtons
tons (mass) per square foot	9,764.856	kilograms per square meter

LATERALLY LOADED PILES AND COMPUTER PROGRAM COM624G

PART I: INTRODUCTION

Need for Soil-Structure Interaction Analyses in Design of Pile Foundations

1. Pile foundations are frequently used to support structures when the soil immediately below the base will not provide adequate bearing capacity. Piles transfer load from the structure to soil strata which can support the applied load. The behavior of such a system depends on the interaction of the piles with both the structure and the soil. Rational analysis of a problem involving pile design must take into consideration the effects of these interactions. Equilibrium of forces and compatibility of displacements throughout the total system must be achieved in the analysis. This report deals with analysis of the lateral interaction of the pile shaft and the soil. The problem of satisfying equilibrium between the pile shaft and superstructure is outside the scope of this report. A number of references are available on this topic for the interested reader (CASE Task Group on Pile Foundations 1980; Martin, Jones, and Radhakrishnan 1980; Awoshika and Reese 1971; Radhakrishnan and Parker 1975; Haliburton 1971; and Dawkins 1982).

Acknowledgments

2. A major portion of the material presented herein is excerpted or summarized from reports published by Prof. Lymon C. Reese and his students/associates at The University of Texas at Austin (UT). The computer program presented herein (COM624G) was developed under the direction of Prof. Reese and modified by the Automatic Data Processing (ADP) Center at the U. S. Army Engineer Waterways Experiment Station (WES) to provide interactive capability and graphics.

3. Excellent summaries of the methods used in analysis of laterally loaded piles are available (Reese and Sullivan 1980, Reese and Allen 1977). It is suggested that the user study these references before becoming deeply involved in pile design using the method of analysis presented herein. Excerpts from these two references appear throughout this report and are acknowledged where included.

Example Applications

4. If a structure is supported on vertical piles and if all loads from the structure are also vertical, then the loads transmitted to the piles will all be axial. If some horizontal component of load is present, a lateral force will also be transmitted to the piles. If some of the piles are battered, an axial and lateral force will be transmitted to the piles regardless of the direction of the applied load. For most structures, particularly hydraulic structures, both horizontal and vertical components of load are present. The theory and the computer program presented in this report consider the response of individual piles to lateral loads. The program is not directly applicable to problems where group effects must be considered, such as pile-supported retaining structures where the piles are closely spaced. Several methods to analyze such problems are available (O'Neill, Hawkins, and Mahar 1980; Reese 1980; and Davisson 1970) but will not be addressed herein. Axially loaded pile behavior and a computer program for analyzing such behavior will be the subject of another report.

5. The method of analysis presented in this report is directly applicable to problems in which the lateral response of single-pile foundation elements is analyzed. Examples of such problems encountered by the Corps are single-pile dolphins (Figure 1) and baffles for grade control structures (Figure 2). The method can also be extended and used in multiple-pile foundation elements such as in the continuous frame pile-supported pumping station shown in Figure 3. To solve problems of this type, the user must ensure in the analysis that the predicted behavior of the structural frame is compatible with the predicted behavior of each of the foundation elements. Thus, the problem is analyzed in two parts: (a) a frame analysis using methods which may vary from a finite element analysis to a moment distribution analysis depending on the level of sophistication desired by the user, and (b) a laterally loaded pile analysis. The analysis is performed on an idealized frame resting on piles which are subjected to horizontal and vertical loads. The frame is separated from the piles at the groundline as shown by the insert in Figure 3. Final results of the analysis must show the lateral deflection, rotation, shear, axial load, and moment to have the same values at the points where the piles connect to the frame.

6. Because analysis of this problem must be performed in two parts, the

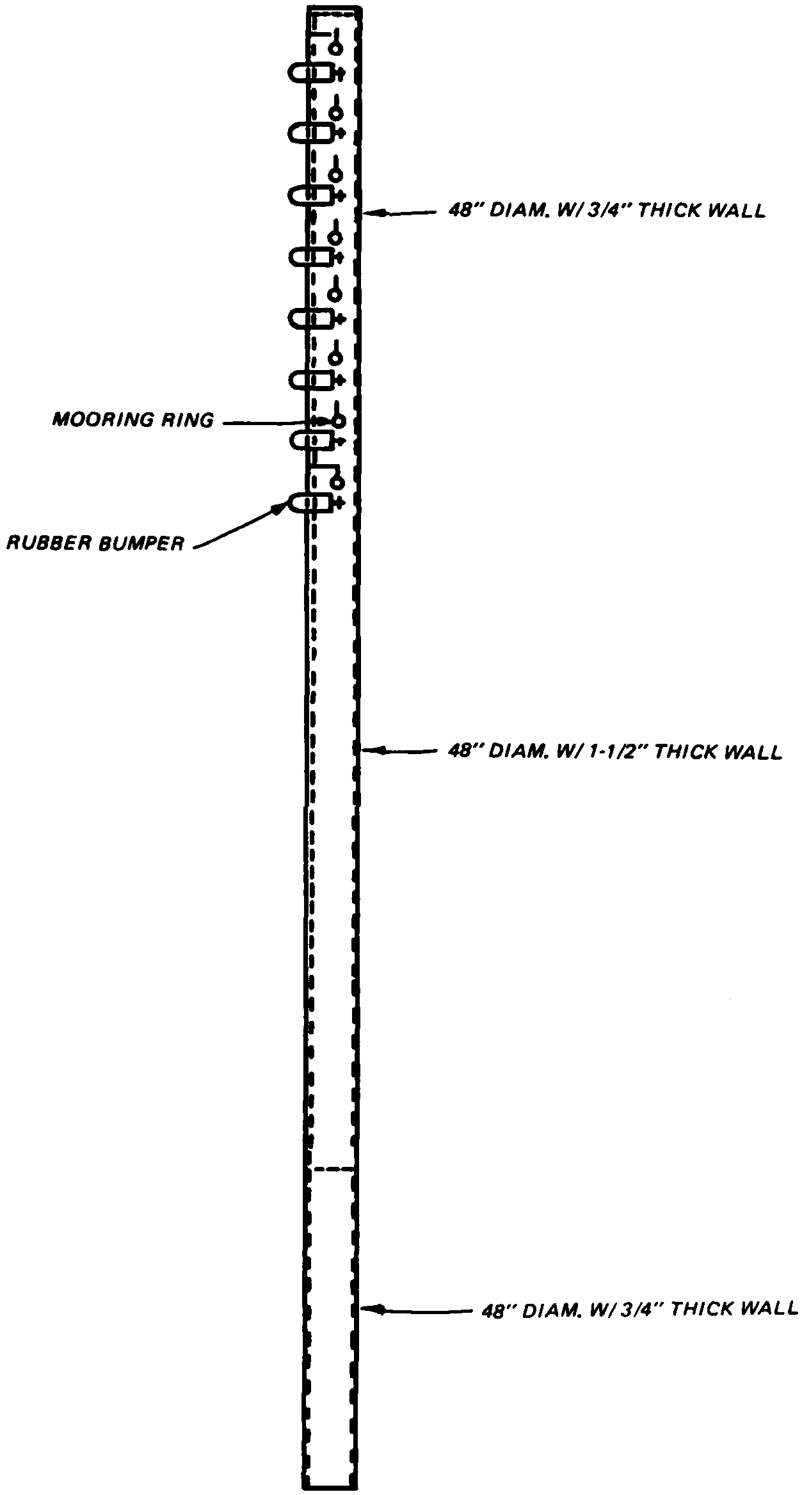

Figure 1. Single pile mooring dolphin

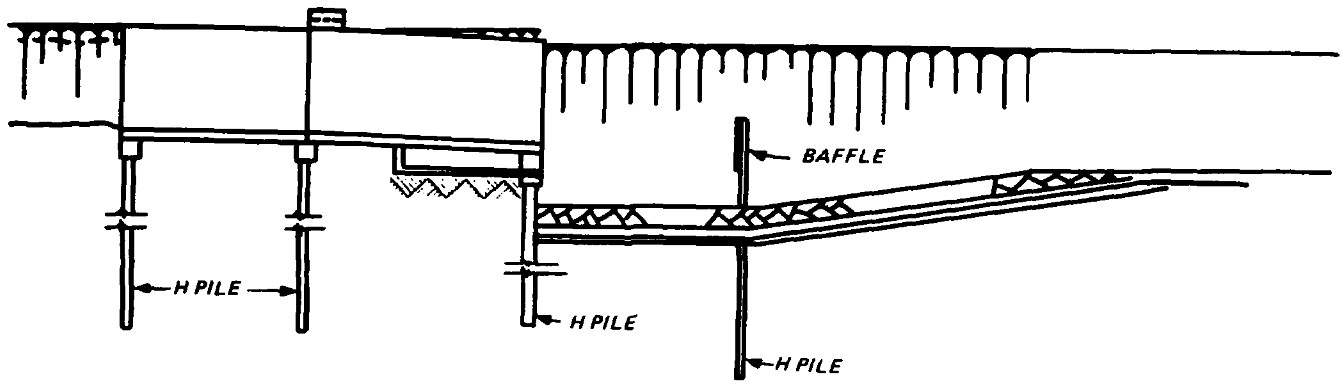

Figure 2. Grade control structure

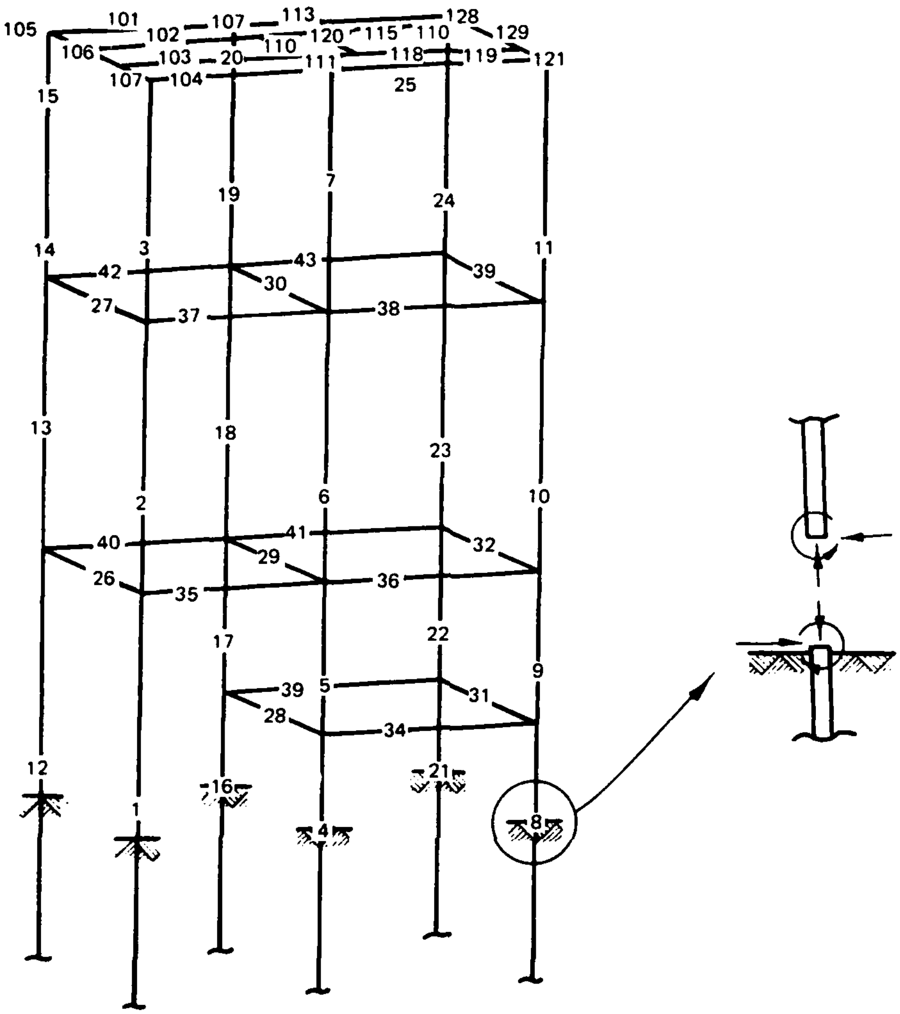

Figure 3. Idealized continuous frame pile-supported pumping station

analysis is iterative. One approach is to assume the reactions of each pile on the frame, apply these reactions to the frame, and analyze. Results of this analysis are then applied to the piles. Then the results of the pile analysis are compared to the assumptions made for the frame analysis, the inputs for the frame analysis are revised, and the process is repeated until compatible forces, moments, and deflections result from both analyses. This approach is discussed in more detail by Reese and Allen (1977).

Methods of Analysis

7. Many different methods have been used in analysis of laterally loaded piles, where the analysis in general consists of computing pile deflection,

bending moment, and shear as a function of depth below the top of the pile. Figure 4 presents the results of a laterally loaded pile analysis. Several of the methods of analysis are based on the theory of subgrade reaction in which the soil around the pile shaft is replaced by a series of discrete springs. Solution of the problem involves solution of a fourth-order differential equation. Most researchers utilizing this approach solve the equation using either a closed-form or a power series solution which requires numerous simplifying assumptions. The more critical of these assumptions are: (a) a constant or linear variation of subgrade modulus with depth, (b) linearly elastic soil behavior, and (c) constant flexural stiffness of the pile. Examples of these methods of analysis are given in Davisson (1970), Terzaghi (1955), Winkler (1967), Broms (1964a), and Broms (1964b).

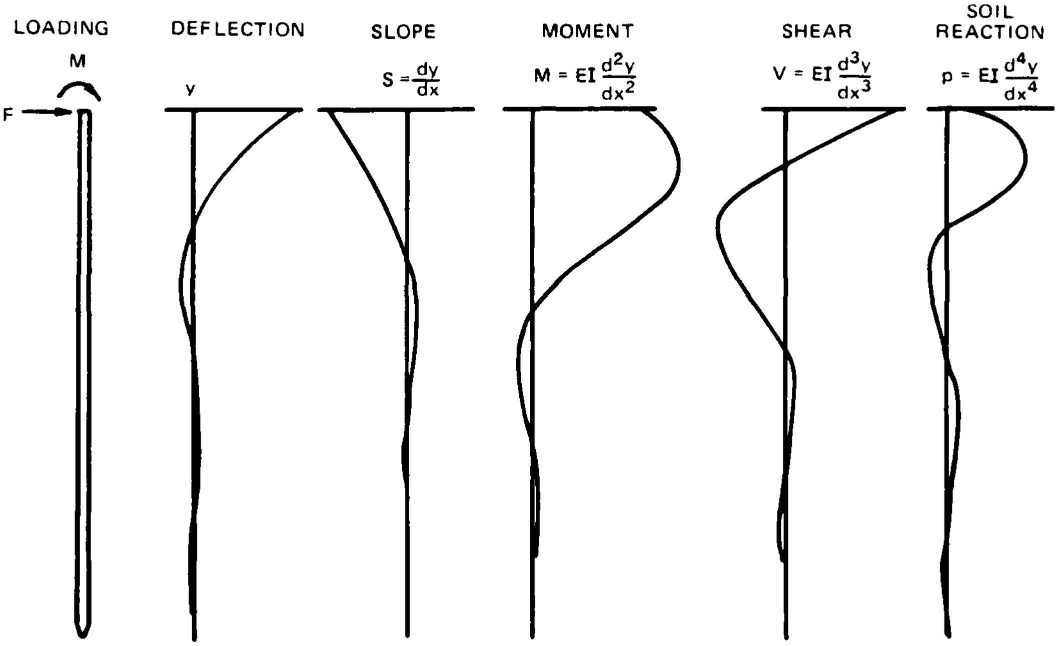

Figure 4. Form of the results obtained from a laterally loaded pile (Reese and Cox 1968)

8. An entirely different approach (Poulos 1971) assumes the soil to be an elastic, homogeneous, isotropic half-space with a constant Young's modulus and Poisson's ratio. The pile is modeled as a thin, rectangular, vertical strip with soil pressures constant across the pile width. This method suffers from the critical limitation of the other methods previously discussed; i.e., the soil response is assumed to be linear.

9. The method utilized in the laterally loaded pile program, COM624G, is

based on the theory of subgrade reaction discussed above. However, the method used for solution of the fourth-order differential equation is the finite difference technique. This solution method, which is presented in Part II, offers several advantages over the conventional methods: (a) the soil modulus can be varied both with depth and pile deflection, (b) stratified soil deposits can be analyzed, (c) the pile stiffness with depth can be considered, (d) the flexural stiffness of the pile can be varied, and (e) several types of boundary conditions can be employed.

Nonlinear Interaction Curves

10. Program COM624G presents mathematical solutions of physical models which are capable of describing the actions and reactions of the pile shaft-soil systems. However, as with most geotechnical engineering applications, the analysis is only as reliable as the soil parameters input to the problem. In this case, the soil parameters take the form of curves which simulate the nonlinear interaction of the pile and the surrounding soil.

11. A family of curves describes the behavior of the soil around a laterally loaded pile in terms of lateral soil reaction versus lateral pile movement for a number of locations along the pile. Each curve represents lateral force (per unit length) transferred to the soil by a given lateral movement at a given location.

12. Criteria used in developing these nonlinear pile shaft-soil interaction curves are presented in Part III. These criteria are thought to yield conservative estimates of soil response; however, the user must always bear in mind that the criteria are based on limited data and there are many inevitable uncertainties in estimating soil response. Nevertheless, the criteria presented here represent the current state of the art. In Part IV of an earlier report by Radhakrishnan and Parker (1975), soil criteria are provided for laterally and axially loaded piles. The material presented herein updates these criteria for laterally loaded piles. Soil criteria for axially loaded piles presented in Radhakrishnan and Parker (1975) will be updated in a separate report.

Purpose and Scope

13. The primary purpose of this report is to present background

information on laterally loaded pile analaysis and to provide supplementary documentation of computer program COM624G. The subject area covered is rich in technical literature, and no attempt is made herein to discuss the methods of analysis in detail. However, enough theory and background are presented to explain the basis of the method used in the computer program. Examples of problems encountered by the Corps of Engineers are used where appropriate for illustrative purposes.

14. Background and theory for laterally loaded pile analysis (the basis for program COM624G) are presented in Part II. Part III presents criteria for developing soil response curves. Appendix A presents a procedure for nondimensional analysis of laterally loaded piles which can be used to perform companion hand calculations to verify the results of the computer solutions. Appendix B presents a design example which illustrates the importance of engineering judgment in analysis of laterally loaded piles. A user's guide for COM624G is presented in Appendix C. A complete and well-documented user's guide for COM624 is presented by Reese and Sullivan (1980). Appendix D presents examples of problems particularly applicable to Corps of Engineers projects. The notations used in the report are summarized in Appendix E.

PART II: BACKGROUND AND THEORY FOR LATERALLY LOADED PILE ANALYSIS

15. Two steps are involved in obtaining the response of a given pile to a lateral load: (a) the soil response must be determined as a function of depth, pile deflection, pile geometry, and nature of loading; and (b) the equations must be solved that yield pile deflection, slope, bending moment, and shear. In this part of the report, the theory involved in developing and solving the equations will be reviewed. The procedures for developing the nonlinear curves which predict the soil response will be presented in Part III.

Review of Basic Beam-Column Relations

16. The method of analysis used in COM624G is based on the theory of a beam on an elastic foundation. In this case, however, the beam is inserted vertically into the ground instead of being placed horizontally on the surface and is treated as a beam-column. The basic concepts of beam-column relations are covered in detail in numerous engineering mechanics texts (see Higdon et al. 1967); therefore, a review of them will not be presented here.

17. The basic relationships between deflection, slope, moment, shear, and load for a beam (Figure 5, without the axial load, P_x)* of constant flexural rigidity are

$$S = \frac{dy}{dx} \tag{1}$$

$$M = EI \frac{d^2y}{dx^2} \tag{2}$$

$$V = \frac{dM}{dx} = EI \frac{d^3y}{dx^3} \tag{3}$$

and

$$q = \frac{dV}{dx} = EI \frac{d^2M}{dx^2} = EI \frac{d^4y}{dx^4} \tag{4}$$

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* For convenience, symbols and unusual abbreviations are listed and defined in the Notation (Appendix E).

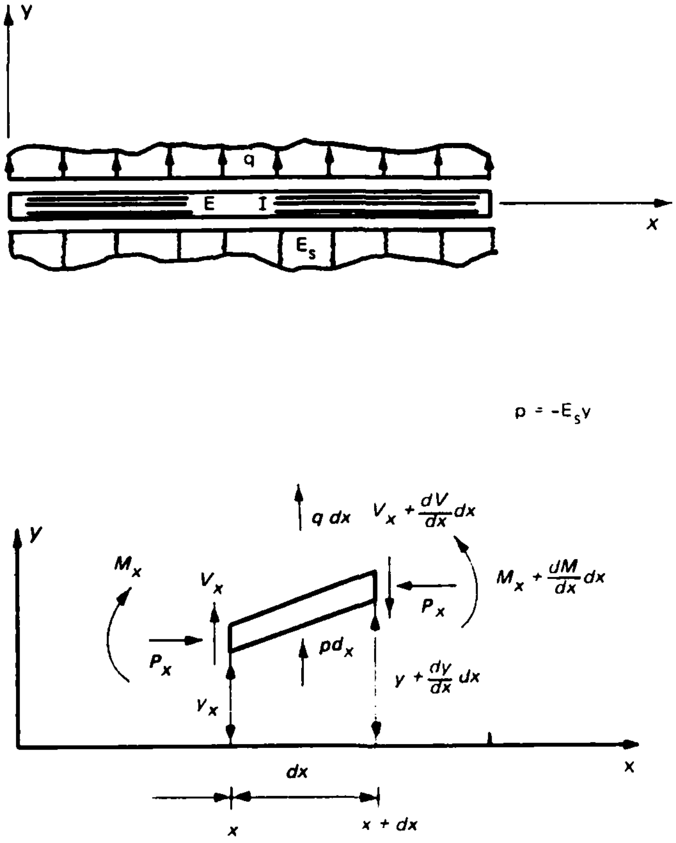


Figure 5. Relationships between deflection, shear, and load for a typical beam-column

where

S = slope

M = moment

EI = flexural rigidity

V = shear

q = uniformly distributed vertical load on beam

y = deflection at point x along the length of the column

Writing these equations in terms of load and deflection gives

$$q = \frac{d^2M}{dx^2} \qquad (5)$$

and

$$y = \frac{1}{EI}\iint M dx \qquad (6)$$

The differential equation for a beam-column subjected to loads only at its ends can be obtained by taking the equation for bending due to flexure and adding to it the bending due to a constant axial load P_x

$$EI \frac{d^4y}{dx^4} + P_x \frac{d^2y}{dx^2} = 0 \tag{7a}$$

If the beam-column is resting on or embedded in soil, a soil reaction p will be resisting the movement of the system and Equation 7a will be transformed to

$$EI \frac{d^4y}{dx^4} + P_x \frac{d^2y}{dx^2} = q + p \tag{7b}$$

where p is the soil resisting pressure applied to the beam.

p-y Concepts of Lateral Load Transfer

18. When the basic beam-column is inserted vertically as a pile shaft, the method of analysis in COM624G considers the soil surrounding the shaft as a set of nonlinear elastic springs as depicted in Figure 6. This assumption is attributed to Winkler (1967), and it states that each spring acts independently; i.e., the behavior of one spring has no effect on any of the adjacent springs. Intuitively, this assumption does not seem correct for describing the nonlinear response of soils. Consequently, this approach has been criticized by some. However, available experimental data (Matlock 1970; Reese, Cox, and Koop 1975) suggest that, for the range of boundary conditions a pile is normally subjected to, the soil response at a point is affected only marginally by the changes in deflected shape.

19. In the analysis, the response of the springs can be taken as either linear or nonlinear. The approach in program COM624G is to treat the springs as nonlinear with their response represented by curves which relate soil resistance p to pile deflection y . In general, these curves are nonlinear and depend on several parameters including depth, pile geometry, shear strength of the soil, and type of loading (static or cyclic). The response of a pile to sustained or dynamic loading is not treated in this report.

20. The concept of a p-y curve can be defined graphically by considering a thin slice of a pile and surrounding soil, as shown in Figure 7a. The earth pressures which act on the surface of the pile prior to lateral loading

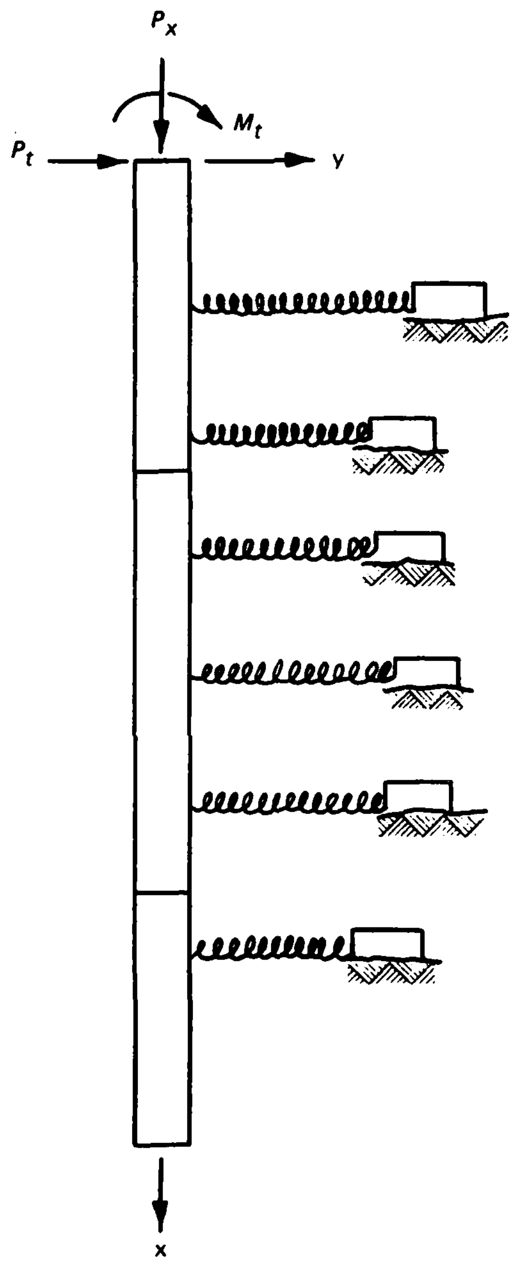
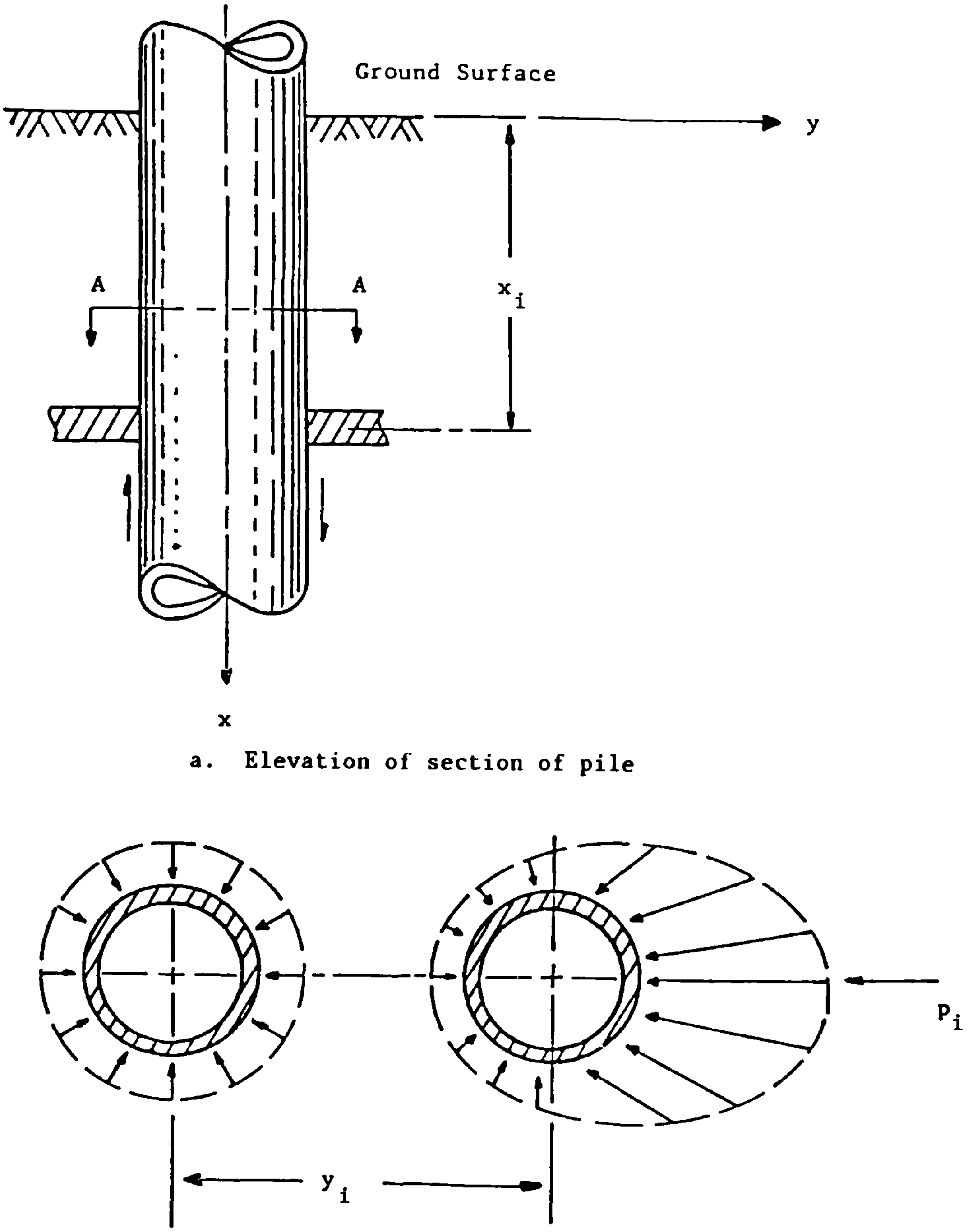


Figure 6. Model of pile-soil system with soil represented as a set of nonlinear elastic springs (Reese 1978)



a. Elevation of section of pile

b. Section A-A. Earth pressure distribution prior to lateral loading

c. Section A-A. Earth pressure distribution after lateral loading

Figure 7. Graphical definition of p and y (Reese and Sullivan 1980)

are assumed to be uniform (Figure 7b). For this condition, the resultant force, obtained by integrating the pressures, is zero. If the pile is given a lateral deflection y_i, as shown in Figure 7c, a net soil reaction p_i will be obtained upon integrating the pressures. This process can be repeated in concept for a series of deflections y, resulting in a series of forces per unit length of pile p, which can be combined to define a p-y curve. In a similar manner, p-y curves may be generated for a number of depths. A family of p-y curves for different depths is shown in Figure 8. The curves are plotted in the second and fourth quadrants to indicate that the soil resistance p is opposite in sign to the deflection y. The user should note that p stands for a force per unit length of pile and is expressed in units

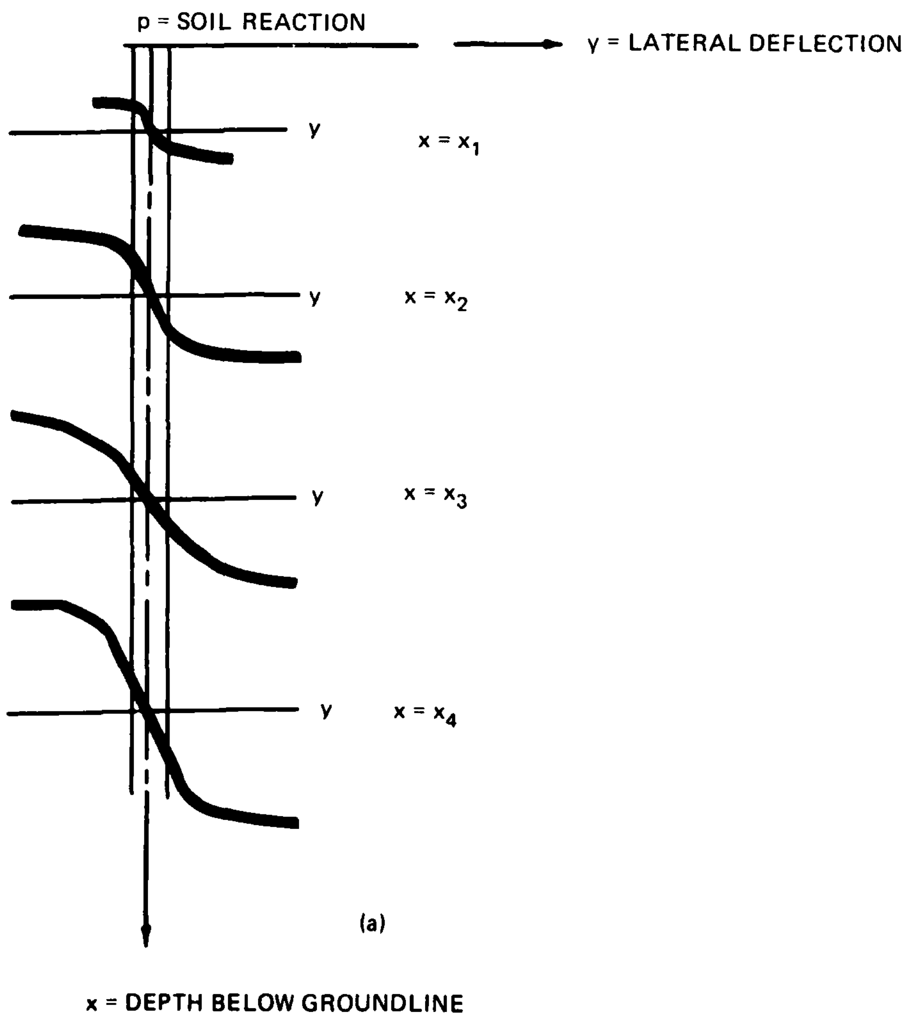


Figure 8. Possible family of p-y curves (Reese and Sullivan 1980)

of pounds per linear inch or pounds per linear foot. It is not a soil pressure which is stated in units of pounds per square inch or pounds per square foot.

21. A typical p-y curve is shown in Figure 9. The curve is plotted in the first quadrant for convenience. The soil modulus E_s is defined as $-p/y$ and is taken as the secant modulus to a point on the p-y curve as shown in Figure 8. Because the curve is strongly nonlinear, the soil modulus changes from an initial stiffness E_{s_1} to an ultimate stiffness p_u/y_u. As can be seen, the soil modulus E_s is not a constant except for a small range of deflections. The soil modulus has units of force per length squared, which is the force per unit length of the pile per unit of movement of the pile into the soil. The soil modulus should not be confused with Young's modulus which has the same units but a different meaning.

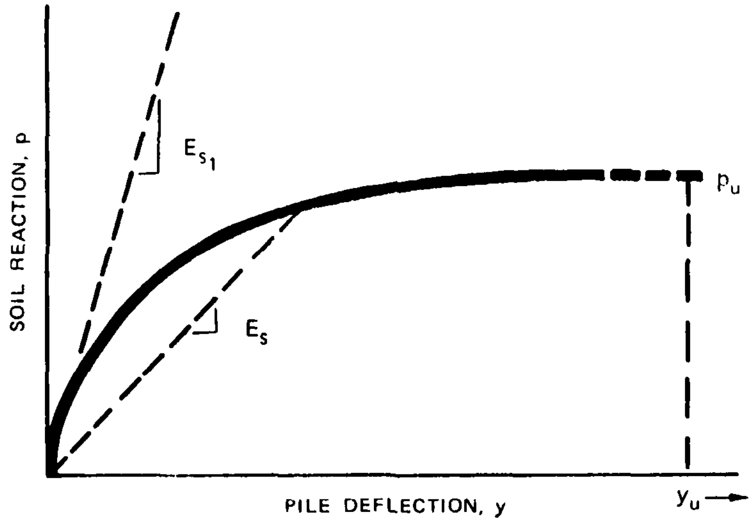


Figure 9. Characteristic shape of p-y curve (Reese and Sullivan 1980)

22. The soil modulus is introduced into the analysis with the relationship:

$$p = -E_s y \tag{8}$$

By substituting this relationship in Equation 7b, the basic equation for laterally loaded piles becomes

$$EI \frac{d^4y}{dx^4} + P_x \frac{d^2y}{dx^2} + E_s y = q \tag{9}$$

Also,

$$V = \frac{dM}{dx} + P_x \frac{dy}{dx} \qquad (10)$$

and

$$M = EI \frac{d^2y}{dx^2} \qquad (11)$$

Equation 9 is developed in the following paragraphs of this part of the report and its solution is presented.

Solution of Governing Differential Equation

23. Computer program COM624G utilizes central difference approximations to describe the load-deformation response of laterally loaded piles. In the following paragraphs, central difference approximations describing the elastic curve of a laterally loaded pile will be derived and used in formulating a set of simultaneous equations for describing the load-deformation response of a laterally loaded pile.

Formulation of finite difference approximations

24. The finite difference approach to the solution of laterally loaded piles was first suggested by Gleser (1953). The idea was extended by a number of investigators including Reese and Matlock (1956, 1960).

25. The first step in the formulation is the derivation of the central difference approximations for the elastic curve (Figure 10). It can be seen from this figure that the slope of the curve at station i may be approximated as a secant drawn through the points on the curve of the two adjacent stations. Mathematically, this step is expressed as

$$\left(\frac{dy}{dx}\right)_i \approx \frac{y_{i+1} - y_{i-1}}{2h} \qquad (12)$$

where h denotes the increment length. For higher derivatives, the process could be repeated by taking simple differences and dividing by 2h each time. However, to keep the system more compact, temporary stations j and k are considered and the slopes at these points computed on the basis of the deflection

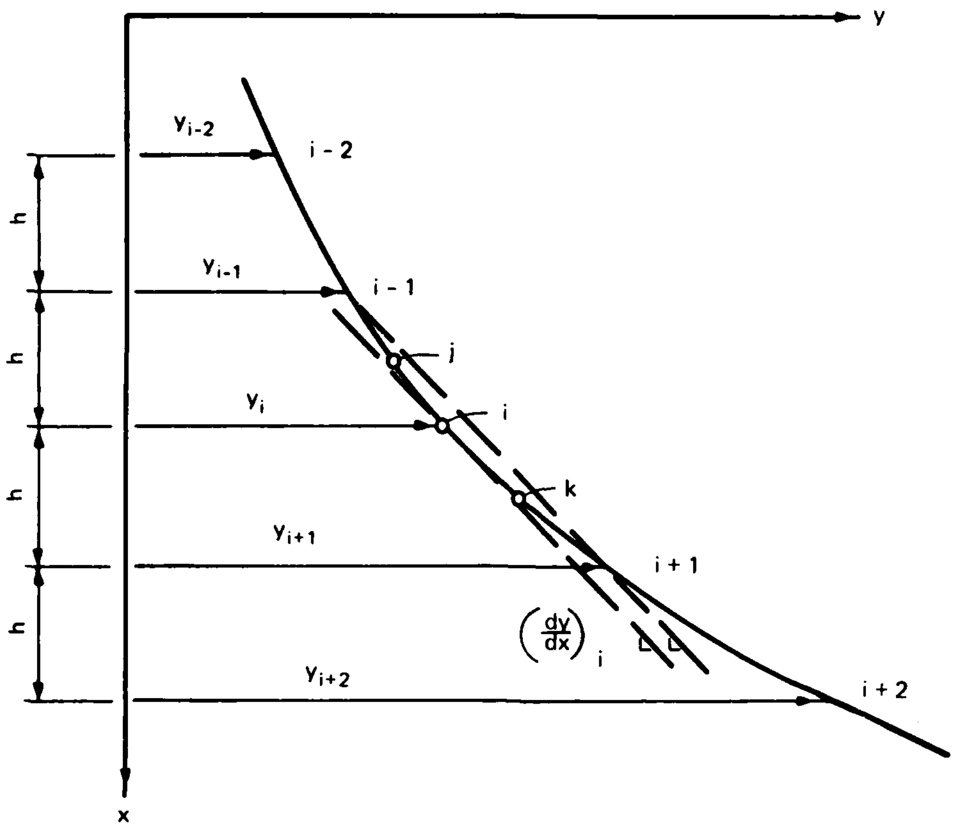


Figure 10. Geometric basis for central difference approximations (Reese and Sullivan 1980)

of the station on each side. The second derivative for each permanent station is then written as the difference between these slopes divided by one increment length in the following equation:

$$\left(\frac{d^2y}{dx^2}\right)_i = \frac{\left(\frac{dy}{dx}\right)_k - \left(\frac{dy}{dx}\right)_j}{h}$$

$$= \frac{y_{i+1} - 2y_i + y_{i-1}}{h^2} \qquad (13)$$

Similarly, the third derivative is expressed as

$$\left(\frac{d^3y}{dx^3}\right) = \frac{\left(\frac{d^2y}{dx^2}\right)_{i+1} - \left(\frac{d^2y}{dx^2}\right)_{i-1}}{2h}$$

$$= \frac{y_{i+2} - 2y_{i+1} + 2y_{i-1} - y_{i-2}}{2h^3} \qquad (14)$$

and the fourth derivative as

$$\left(\frac{d^4y}{dx^4}\right)_i = \frac{\left(\frac{d^3y}{dx^3}\right)_k - \left(\frac{d^3y}{dx^3}\right)_j}{h}$$

$$= \frac{y_{i+2} - 4y_{i+1} + 6y_i - 4y_{i-1} - y_{i-2}}{h^4} \qquad (15)$$

Formulation of finite difference approximations for equations of bending of laterally loaded piles

26. In the development of the equations, consideration must be given to the assumptions regarding the variation in pile bending stiffness (EI = R). For the case of pure bending and constant bending stiffness, the second derivative of moment is usually written as

$$\frac{d^2M}{dx^2} = EI\,\frac{d^4y}{dx^4} \qquad (16)$$

For the case of pure bending and a variable bending stiffness, the second derivative of moment is expressed as

$$\frac{d^2M}{dx^2} = EI\,\frac{d^4y}{dx^4} + 2\,\frac{d}{dx}\,(EI)\,\frac{d^3y}{dx} + \frac{d^2}{dx^2}\,(EI)\,\frac{d^2y}{dx^2} \qquad (17)$$

However, in formulating the finite difference equations, the assumption was made that the moment was a smooth continuous function of x and that the second derivative of moment could be approximated by the expression

$$\frac{d^2M}{dx^2} \approx \frac{M_{i+1} - 2M_i + M_{i-1}}{h^2} \quad (18)$$

where M_{i+1} , M_i , and M_{i-1} are the moments at joints i+1 , i , and i-1 , respectively. For a variable stiffness, Equation 18 is a somewhat cruder approximation than Equation 20. However, it permits the bending stiffness to vary from station to station.

27. Equations 9, 10, and 11 may now be written in finite difference form by using the central difference approximations for the first and second of the elastic curves. The equations will be written for a general point referred to as station i. Station numbering increases from the bottom to the top of piles. The equations obtained for station i, formulated from Equation 11, are as follows:

$$M_i = R_i\left(\frac{y_{i+1} - 2y_i + y_{i-1}}{h^2}\right) \quad (19)$$

where R = flexural rigidity (EI). Equations 8, 13, 16, 18, and 19 can be employed and Equation 20 can be formulated from Equation 9.

$$\begin{aligned} & y_{i+2}(R_{i+1}) + y_{i+1}\left(-2R_{i+1} - 2R_i + P_x h^2\right) \\ & + y_i\left(R_{i+1} + 4R_i + R_{i-1} - 2P_x h^2 + E_{si} h^4\right) \\ & + y_{i-1}\left(-2R_i - 2R_{i-1} + P_x h^2\right) + y_{i-2}(R_{i-1}) - q = 0 \end{aligned} \quad (20)$$

Equation 21 can be formulated from Equation 10 in a similar manner.

$$\begin{aligned} V_i = \frac{1}{2h^3} & \left[y_{i+2}(R_{i+1}) + y_{i+1}\left(-2R_{i+1} + P_x h^2\right)\right] \\ & + y_i(R_{i+1} - R_{i-1}) + y_{i-1}\left(-P_x h^2\right) + y_{i-2}(-R_{i-1}) \end{aligned} \quad (21)$$

Solution of the finite difference equations (extracted from Reese and Sullivan 1980)

28. The final step is the formulation of a set of simultaneous equations which when solved yield the deflected shape of the pile. The solution

requires the application of four boundary conditions, since Equation 9 is actually a fourth-order differential equation in terms of the dependent variable y . If values of deflection are found, moment, shear, and soil reaction can be obtained for any location along the pile by back-substitution of appropriate values of deflection into appropriate equations.

29. The pile is divided into equal increments of length h (Figure 11). In addition, two fictitious increments are added to both the top and bottom of the pile. The four fictitious stations are used in formulating the set of equations, but they will not appear in the solution or influence the results. The coordinate system and numbering system used are also illustrated in Figure 11.

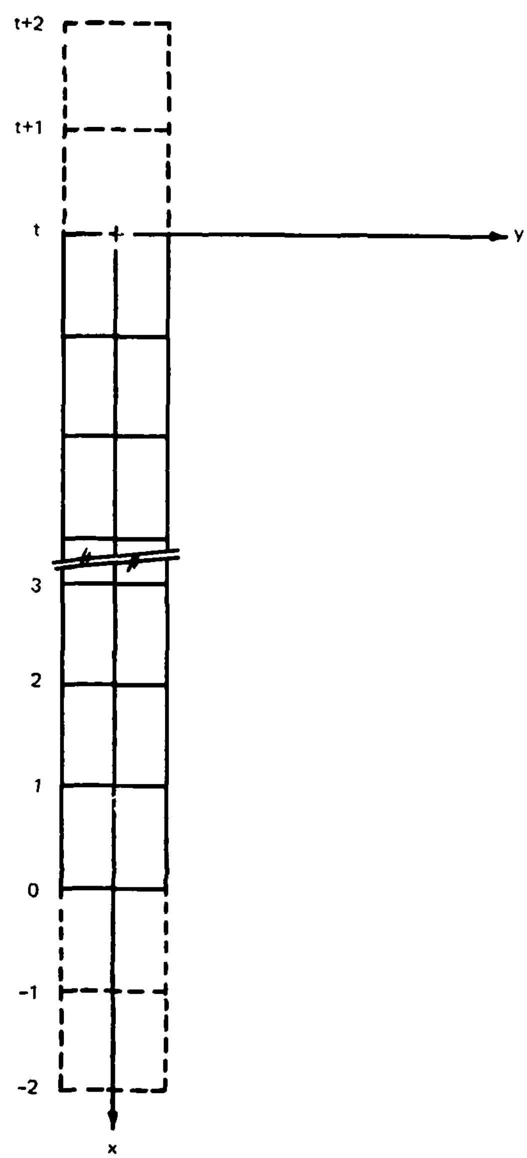


Figure 11. Finite difference representation of a pile (Reese and Sullivan 1980)

30. Using the notation shown in Figure 11, the two boundary conditions at the bottom of the pile (point 0) are zero bending moment,

$$R_0 \left(\frac{d^2y}{dx^2}\right)_0 = 0 \qquad (22a)$$

and zero shear,

$$R_0\left(\frac{d^3y}{dx^3}\right)_0 + P_x\left(\frac{dy}{dx}\right)_0 = 0 \qquad (22b)$$

For simplicity it is assumed that

$$R_{-1} = R_0 = R_1 \qquad (22c)$$

These boundary conditions are, in finite difference form,

$$y_{-1} - 2y_0 + y_1 = 0 \tag{23a}$$

$$y_{-2} = y_{-1}\left(2 - \frac{P_x h^2}{R_0}\right) - y_1\left(2 - \frac{P_x h^2}{R_0}\right) + y_2 \tag{23b}$$

respectively. Substituting these boundary conditions in finite difference form in Equation 20 where i is equal to zero, and rearranging terms, results in the following equations:

$$y_0 = a_0 y_1 - b_0 y_2 \tag{24a}$$

where

$$a_0 = \frac{2R_0 + 2R_1 - 2P_x h^2}{R_0 + R_1 + E_{so} h^4 - 2P_x h^2} \tag{24b}$$

$$b_0 = \frac{R_0 + R_1}{R_0 + R_1 + E_{so} h^4 - 2P_x h^2} \tag{24c}$$

$$d_0 = \frac{qh^4}{R_0 + R_1 + E_{so} h^4 - 2P_x h^4} \tag{24d}$$

31. Equation 20 can be expressed for all values of i other than 0 and the top of the pile by the following relationships:

$$y_i = a_i y_{i+1} - b_i y_{i+2} + d_i \tag{25a}$$

$$a_i = \frac{-2b_{i-1}R_{i-1} + a_{i-2}b_{i-1}R_{i-1} + 2R_i - 2b_{i-1}R_i + 2R_{i+1} - P_x h^2(1 - b_{i-1})}{c_i} \tag{25b}$$

$$b_i = \frac{R_{i+1}}{c_i} \tag{25c}$$

and

$$c_i = R_{i-1} - 2a_{i-1}R_{i-1} - b_{i-2}R_{i-1} + a_{i-2}a_{i-1}R_{i-1} + 4R_i$$

$$- 2a_{i-1}R_i + R_{i+1} + k_i h^4 - P_x h^2 (2 - a_{i-1}) \quad (25d)$$

$$d_i = \frac{q_i h^4 - d_{i-1}\left(a_{i-2}R_{i-1} - 2R_{i-1} - 2R_i + P_x h^2\right) - d_{i-2}R_{i-1}}{c_i} \quad (25e)$$

32. The top of the pile (i=t) is shown in Figure 11. Three sets of boundary conditions are considered.

a. The lateral load (P_t) and the moment (M_t) at the top of the piles are known.

b. The lateral load (P_t) and the slope of the elastic curve (S_t) at the top of the pile are known.

c. The lateral load (P_t) and the rotational-restraint constant (M_t/S_t) at the top of the pile are known.

33. For convenience in establishing expressions for these boundary conditions, the following constants are defined.

$$J_1 = 2hS_t \quad (26a)$$

$$J_2 = \frac{M_t h^2}{R_t} \quad (26b)$$

$$J_3 = \frac{2P_t h^3}{R_t} \quad (26c)$$

$$J_4 = \frac{h}{2R_t} \frac{M_t}{S_t} \quad (26d)$$

and

$$U = \frac{-P_x h^2}{R_t} \quad (26e)$$

34. The difference equations expressing the first of the boundary conditions for the top of the pile are:

$$\frac{R_t}{2h^3}(y_{t-2} - 2y_{t-1} + 2y_{t+1} - y_{t+2}) + \frac{P_x}{2h}(y_{t-1} - y_{t+1}) = P_t \tag{27a}$$

$$\frac{R_t}{h^2}(y_{t-1} - 2y_t + y_{t+1}) = M_t \tag{27b}$$

After some substitutions the difference equations for the deflection at the top of the pile and at the two imaginary points above the top of the pile are:

$$y_t = \frac{Q_2}{Q_1} \tag{28a}$$

$$y_{t+1} = \frac{J_2 + G_1 y_t - d_{t-1}}{G_2} \tag{28b}$$

$$y_{t+2} = \frac{a_t y_{t+1} - y_t + d_t}{b_t} \tag{28c}$$

where

$$Q_1 = H_1 + \frac{G_1 H_2}{G_2} + \left(1 - a_t \frac{G_1}{G_2}\right)\frac{1}{b_t} \tag{28d}$$

$$Q_2 = J_3 + \frac{a_t(J_2 - d_{t-1})}{b_t G_2} + \frac{H_2(d_{t-1} - J_2)}{G_2} + \frac{d_t}{b_t} + d_{t-1}(2 + U - a_{t-2}) - d_{t-2} \tag{28e}$$

$$G_1 = 2 - a_{t-1} \tag{28f}$$

$$G_2 = 1 - b_{t-1} \tag{28g}$$

$$H_1 = -2a_{t-1} - Ua_{t-1} - b_{t-2} + a_{t-1}a_{t-2} \tag{28h}$$

and

$$H_2 = -a_{t-2}b_{t-1} + 2b_{t-1} + 2 + U(1 + b_{t-1}) \tag{28i}$$

35. The difference equations for the second set of boundary conditions are Equations 27a and 29:

$$y_{t-1} - y_{t+1} = J_1 \tag{29}$$

36. The resulting difference equations for the deflections at the three points at the top of the pile are:

$$y_t = \frac{Q_4}{Q_3} \tag{30a}$$

$$y_{t+1} = \frac{a_{t-1}y_t - J_1 + d_{t-1}}{G_4} \tag{30b}$$

$$y_{t+2} = \frac{a_t y_{t+1} - y_t + d_t}{b_t} \tag{30c}$$

where

$$Q_3 = H_1 + \frac{H_2 a_{t-1}}{G_4} - \frac{a_t a_{t-1}}{b_t G_4} + \frac{1}{b_t} \tag{30d}$$

$$Q_4 = J_3 + \frac{J_1 H_2}{G_4} - \frac{J_1 a_t}{b_t G_4} \tag{30e}$$

and

$$G_4 = 1 + b_{t-1} \tag{30f}$$

and the other constants are as previously defined.

37. The difference equations for the third set of boundary conditions are Equations 27a and 31:

$$\frac{y_{t-1} - 2y_t + y_{t+1}}{y_{t-1} - y_{t+1}} = J_4 \tag{31}$$

38. The resulting difference equations for the deflections at the three points at the top of the pile are:

$$y_t = \frac{J_3 - \frac{a_t d_{t-1}(1 - J_4)}{b_t(G_2 + J_4G_4)} + \frac{d_t}{b_t} + d_{t-1}(2 + E - a_{t-2}) - d_{t-2} + \frac{d_{t-1}H_2(1 - J_4)}{G_2 + J_4G_4}}{H_1 + H_2H_3 - \frac{a_t}{b_t} H_3 + \frac{1}{b_t}} \qquad (32a)$$

$$y_{t+1} = \frac{y_t(G_1 + J_4a_{t-1}) - d_{t-1}(1 - J_4)}{G_2 + J_4G_4} = H_3y_t - \frac{d_{t-1}(1 - J_4)}{G_2 + J_4G_4} \qquad (32b)$$

$$y_{t+2} = \frac{1}{b_t}(a_ty_{t+1} - y_t + d_t) \qquad (32c)$$

where

$$H_3 = \frac{G_1 + J_4a_{t-1}}{G_2 + J_4G_4} \qquad (32d)$$

The other constants have been previously defined.

39. Using the above equations, the behavior of a pile under lateral load may be obtained by using COM624G.

PART III: CRITERIA FOR DEVELOPING SOIL RESPONSE CURVES FOR LATERALLY LOADED PILES

40. The methods of constructing p-y curves as presented in this report were developed at UT. The methods were derived largely from results obtained in field tests of piles under lateral loading. The approach was to take the experimental field curves and correlate them empirically with simple, basic soil mechanics theory and experience. By combining soil mechanics theory with experimental results, correlations could be made between soil properties, pile diameter, and depth. This gives generality to the methods used in construction of the p-y curves.

41. McClelland and Focht (1958) were the first to report p-y criteria which considered the nonlinearity of the soil. Since their work, numerous researchers have contributed to p-y curve development; however, most of the developmental work has been performed at UT. A history of the development will not be presented here; however, the interested reader can refer to Meyer and Reese (1979) for more detailed information.

42. The methods presented herein represent the current state of p-y curve development; however, it is expected that this development will continue as more field tests are performed and as more experience is gained. The user must remain abreast of these changes in order to ensure that the analyses reflect the state of the art at the particular time they are performed.

43. Recommended methods for computing p-y curves are based on field tests presented in five different references for four different types of soil conditions. These are:

- a. Soft clay below the water table (Matlock 1970)
- b. Stiff clay below the water table (Reese, Cox, and Koop 1975)
- c. Stiff clay above the water table (Reese and Welch 1975).
- d. Unified clay criteria developed for combined soft and stiff clays below the water table, (Sullivan, Reese, and Fenske 1979).
- e. Sands (Reese, Cox, and Koop 1974).

44. These references describe field experiments, the soil conditions in which they were performed, the rationale and considerations involved in evaluating the data, and conclusions from the experiments presented in the form of recommended p-y curve criteria. As can be seen from the descriptive names, the criteria were developed separately for clays above and below the

water table and for sands. Other soil types would be expected to exhibit characteristics falling between the extremes of the soils and conditions in these tests.

45. The criteria for the conditions listed in subparagraphs 43a, b, c, and e have been combined into summary form and are presented in Reese and Sullivan (1980) and Reese and Allen (1977). The material presented herein is extracted primarily from these two references. However, the user of COM624G is strongly encouraged to study the references cited in paragraph 42 before becoming deeply involved in the analysis of laterally loaded piles. Also, the user should bear in mind that any one set of p-y curves is strongly related to only one or two lateral load tests, and this fact should be considered when using the curves for design.

Factors Influencing p-y Curves

46. Factors that most influence p-y curves are soil properties, pile geometry, nature of loading, and pile spacing. The correlations that have been developed for predicting soil response have been based on best estimates of soil properties determined from borings, laboratory tests, and field in situ tests. Thus far, no investigations have been performed to determine the effect which the method of pile installation has on these soil properties. The logic supporting this approach is that the effects of pile installation on soil properties are principally confined to a zone of soil close to the pile wall, while a mass of soil several diameters from the pile is stressed as lateral deflection occurs. There are instances where the method of pile installation must be considered; e.g., if a pile is jetted into place, a considerable volume of soil could be removed with a considerable effect on the soil response. In such instances, the user must rely on experience in adjusting the p-y curves to account for the effect of pile installation.

47. The principal dimension of the pile which affects the soil response is its diameter. All recommendations for developing p-y curves include the term for the diameter of the pile: if the cross section of the pile is not circular, the width of the pile perpendicular to the direction of loading is usually taken as the diameter. Field tests have been performed on piles with a limited range of diameters. Experience indicates that, for the normal range of pile diameters encountered in practice, the criteria adequately represent

the effect of pile diameter. However, additional research is needed on large-diameter piles (30 in.* and larger) to determine the effect of pile diameter on large pile behavior (Meyer and Reese 1979). Stevens and Audibert (1979) have presented evidence that, for piles 50 in. and larger, the observed ground-line deflections are approximately half the predicted deflections.

48. p-y curves can be greatly affected by the type of loading. This report summarizes recommendations for short-term static loads and for cyclic (or repeated) loading. The curves do not consider any consolidation effects that would occur under sustained loading. Nor do they consider cases where the loadings are dynamic, as would occur during an earthquake.

49. Because the field tests were run on single piles, the p-y criteria do not consider group effects. Unfortunately, the designer is often faced with the problem of analyzing the lateral response of pile groups. Although several methods are available in the literature, there is no one established, widely used method which considers the group effect on soil response. Four available methods which address group effect are presented in O'Neill, Hawkins, and Mahar (1980), Davisson (1970), Focht and Koch (1973), and Poulos (1971a and b).

50. Another factor which can influence p-y criteria is the effect of pile batter. The criteria were derived from experiments on vertical piles. As the batter of a pile is increased, some point will eventually be reached where the criteria for vertical piles are no longer applicable. Information for specific recommendations on this problem is not available; however, some comparison studies performed by Meyer and Reese (1979) indicate that by applying adjustment factors recommended by Kubo (1967), reasonable estimates of pile deflection for laterally loaded batter piles can be obtained.

Analytical Basis for p-y Curves

51. As discussed previously, the methods of constructing p-y curves were derived from results obtained in field tests of piles under lateral loading. Results were then correlated with soil properties, pile diameter, and depth to give generality to the methods. Soil resistance-pile deflection

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* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

curves are generally considered to be composed of an initial elastic portion and an ultimate failure value. Principles of the theory of elasticity are generally applied for the definition of the initial portion. Several failure mechanisms are postulated and used to define the ultimate values. The following paragraphs briefly describe the analytical concepts which were correlated with the experimental curves.

52. The theory of elasticity is only applicable to linearly elastic materials; however, use has been made of the theory of elasticity and related approaches in describing certain concepts which have been incorporated into the nonlinear p-y curves.

Initial Portion of p-y Curve

Terzaghi

53. In his classic paper "Evaluation of Coefficients of Subgrade Reaction," Terzaghi (1955) proposed coefficients of lateral subgrade reaction which used a straight-line relationship between deflection of the pile y and resistance offered by the soil p. Terzaghi recognized the limitations of this approach and stated that the linear relationship between p and y was valid for values of p that were smaller than about half the ultimate bearing capacity of the clay.

54. For stiff clays, Terzaghi gave the relationship

$$k_h = \frac{\bar{k}_{s1}}{1.5b} (1 \text{ ft}) \qquad (33)$$

where

k_h = coefficient of horizontal subgrade reaction

$\bar{k}_{s1}$ = coefficient of vertical subgrade reaction for a 1-ft-wide beam

b = width of the pile, ft

Adapting the coefficient of lateral subgrade reaction to fit the soil modulus E_s yields

$$E_s = k_h b \qquad (34)$$

55. Terzaghi proposed that the coefficient of horizontal subgrade reaction for piles in stiff clay was constant with depth and recommended the values of $\bar{k}_{s1}$ given in Table 1.

Table 1

Terzaghi's Recommendations for Soil Modulus $\bar{k}_{s1}$ for Laterally Loaded Piles in Stiff Clay

	Consistency of Clay		
	Stiff	Very Stiff	Hard
Value of q_u , tsf	1-2	2-4	4-7
Range for $\bar{k}_{s1}$, pci	58-116	116-232	232-464
Proposed values for $\bar{k}_{s1}$, pci	87	174	348*

* Higher values should be used only if estimated on the basis of adequate test results.

56. For sands, Terzaghi recognized that the stiffness increases with depth (or confining pressure). Thus, the family of p-y curves recommended for sand consisted of a series of straight lines with slopes horizontal at the ground surface and increasing linearly with depth. The linear relationship between p and y can be expressed in terms of E_s as:

$$E_s = kx \tag{35}$$

where

k = constant giving variation of soil modulus with depth

x = depth below ground surface

Table 2 gives Terzaghi's recommendations for k . Terzaghi also recognized that, as for clay, the assumed linear relationship between p and y was valid only for values of p smaller than about one-half the ultimate bearing capacity of the sand.

Table 2

Terzaghi's Recommendations for Values of k for Laterally Loaded Piles in Sand

	Relative Density of Sand		
	Loose	Medium	Dense
Dry or moist k , pci	3.5-10.4	13-40	51-102
Submerged sand k , pci	2.1-6.4	8-27	32-64

57. Even though Terzaghi's work assumed a linear relationship between pile deflection and soil resistance, it provided a useful concept for defining the initial soil reactions for the portions of certain p-y curves where the soil reaction is less than half the ultimate soil reaction. This concept was utilized in defining the p-y curves for stiff clay below the water table (Reese, Cox, and Koop 1975), for the unified soil criteria (Sullivan, Reese, and Fenske 1979), and for sands (Reese, Cox, and Koop 1974), except that the values were adjusted slightly to reflect the results from the individual field tests.

Skempton

58. Skempton (1951) suggested a relationship between load and settlement for various footing shapes bearing on clay. By combining the theory of elasticity with field observations from full-scale foundations, Skempton related settlements of footings to strains obtained from unconsolidated, undrained (Q) triaxial tests with the equation

$$\rho_1 = 2\varepsilon b \tag{36}$$

where

ρ_1 = mean settlement of the foundation for the particular case

ε = strain in laboratory triaxial test for the deviator stress corresponding to the mean foundation pressure under the footing

b = footing width

Equation 36 involves numerous approximations; nevertheless, because of the experimental evidence presented by Skempton, the method is frequently used in predicting foundation settlements. However, further assumptions are necessary before the equation can be used in predicting p-y curves. The concept is extended to the p-y curve for a laterally loaded pile by assuming that the depth is such that the behavior is not affected by the free surface of the soil.

59. As an example of the use of Skempton's concept, Equation 36 was extended to define the deflection of the pile, y_{50}, at one-half the ultimate soil resistance (Matlock 1970; Reese, Cox, and Koop 1975; Reese and Welch 1975; and Sullivan, Reese, and Fenske 1979). The equation is

$$y_{50} = A\varepsilon_{50}b \tag{37}$$

where

A = factor varying from 0.35 to 2.5 based on experimental results from the pile tests for the different soil conditions

ε_{50} = strain from an undrained soil test corresponding to half the maximum principal stress difference

McClelland and Focht

60. McClelland and Focht (1958) presented work which paralleled the work of Skempton (1951), although their work was not as strongly based on the theory of elasticity as his. Their paper represented the first report of experimental p-y curves from a full-scale load test. They attempted to relate soil resistance and pile deflection directly to stress-strain curves from consolidated undrained (R) triaxial tests with confining pressure equal to overburden pressure. To obtain values of soil resistance p from the laboratory tests, they recommended the following equation

$$p = 5.5b\sigma_{\Delta} \tag{38}$$

where

b = pile diameter

σ_{Δ} = deviator stress $(\sigma_1 - \sigma_3)$

To obtain values of pile deflection y from stress-strain curves, McClelland and Focht proposed

$$y = 0.5\varepsilon b \tag{39}$$

where the 0.5 corresponds to a value of 2 suggested by Skempton.

61. McClelland and Focht's work has been superseded by additional research on p-y curves because it has since been proven that the appropriate soil modulus cannot be determined directly from a shear test. Nevertheless, theirs was a very important step because it was the first effort to relate the nonlinearity of p-y curves to an analytical approach utilizing soil shear strength and stress-strain properties.

Soil Models for Predicting Ultimate Soil Resistance

62. This section reviews the concepts involved in determining the ultimate resistance p_u that can be developed against a pile near the ground

surface and at some depth below the surface. This review was extracted from Reese and Sullivan (1980) and Reese and Allen (1977).

Saturated clay

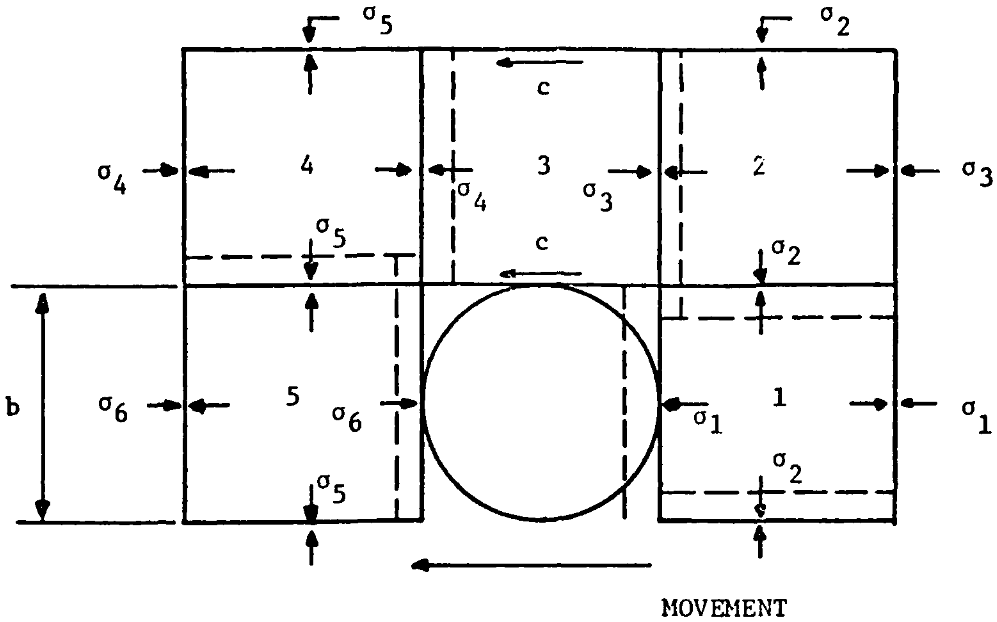
63. Theoretical values for ultimate resistance against piles in saturated clay employ the use of two models which assume that the clay around the pile shaft fails as either a group of sliding blocks or a wedge, depending on the depth below the surface. The soil is assumed to be saturated and to fail under undrained conditions so the shear strength is represented by cohesion c with the angle of internal friction ϕ equal to zero.

64. The failure of the clay as the pile shaft moves laterally into the soil is considered in two parts. At some depth in the ground, failure will occur by flow of the soil around the pile without vertical displacement; i.e., plane strain conditions. This type of failure is depicted in Figure 12. Near the surface, a wedge-shaped block of soil is assumed to form which is moved upward and outward by the force of the pile. Figure 13 illustrates this theoretical wedge of soil.

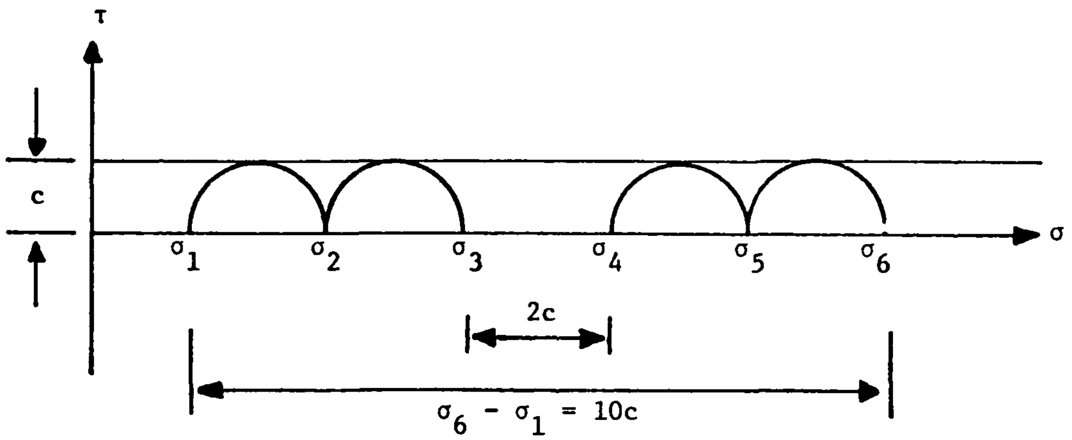
65. The blocks in Figure 12 can be considered to be samples of unit height which fail under plane strain conditions. If it is assumed that blocks 1, 2, 4, and 5 fail by shear and that block 3 develops resistance by sliding, the stress conditions are represented by Figure 12b. If σ_1 is taken to be some small stress equal to the active pressure, then block 1 must move in the direction of pile movement. σ_2 must be approximately $2c$ in order to cause failure of block 1. If σ_2 is considered to be the confining stress on block 2, then σ_3 must be approximately $4c$. If block 3 slides due to the stress σ_3, then block 3 must have a resistance to sliding of $2c$. By assuming that blocks 4 and 5 fail by the same line of reasoning as blocks 1 and 2 (i.e., $\sigma_4 = 6c$), it can be found that $\sigma_6 = 10c$. By examining a free body of a section of the pile (Figure 12c), it can be concluded that the total force exerted by the pile segment on the soil during failure is

$$p_u = 11cb \tag{40}$$

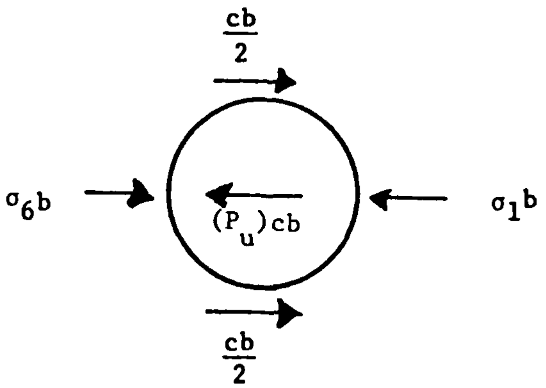
66. The wedge in Figure 13 offers resistance to lateral movement of the pile by means of cohesion along the sides and bottom and its weight. Summing components of the forces in the horizontal direction, the resultant force F_p is



a. Section through pile

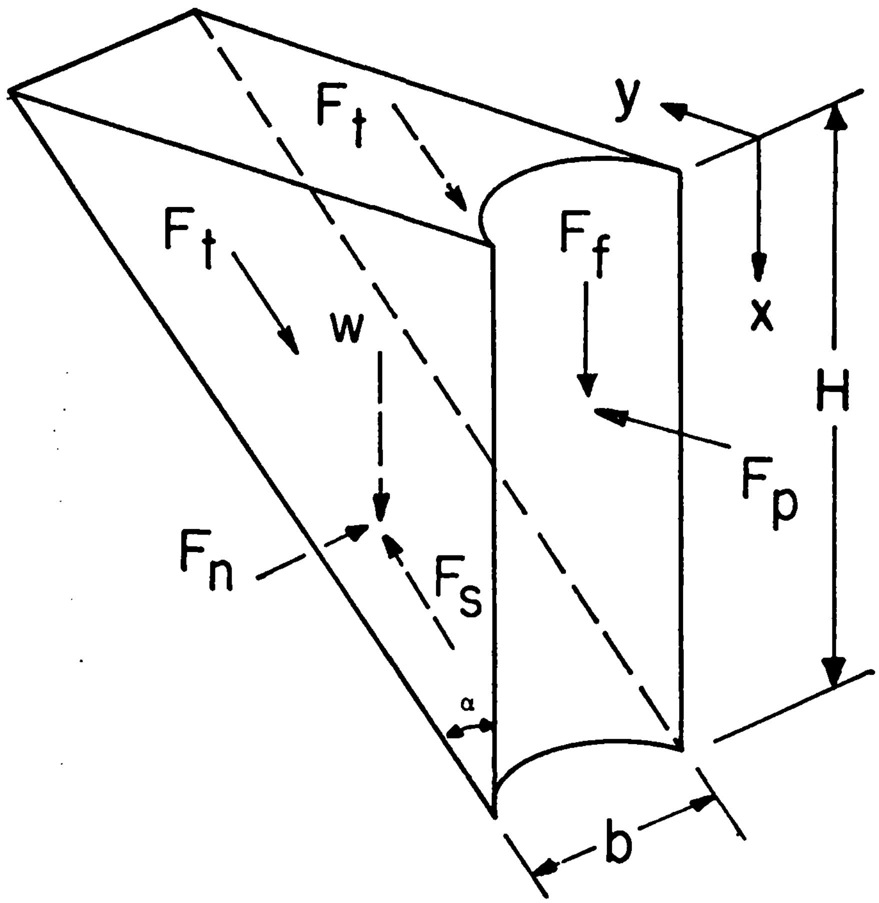


b. Mohr-Coulomb diagram

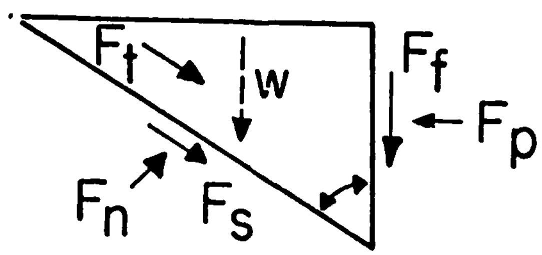


c. Forces acting on pile

Figure 12. Model of lateral flow-around type of failure for clay (Reese and Sullivan 1980)



a. Shape of wedge



b. Forces acting on wedge

Figure 13. Assumed passive wedge type of failure for clay (Reese and Sullivan 1980)

$$F_p = c_a bH \quad \tan \alpha + (1 + m) \cot \alpha \quad + \frac{1}{2} \gamma bH^2 + c_a H^2 \sec \alpha \qquad (41)$$

where

- c_a = average undrained shear strength
- H = depth to the point under consideration
- m = reduction factor to be multiplied by c_a to yield the average sliding stress between the pile and the stiff clay
- γ = average unit weight of the soil (submerged unit weight if the soil is below the water table)

The remaining terms are defined in Figure 13. It is possible to take the partial derivatives of Equation 41 with respect to the angle α and set the equation equal to zero to find the angle at which the equation is minimized. However, as an approximation, the angle α can be taken as 45° and m can be assumed equal to zero. Differentiation of the resulting expression with respect to H yields an expression for the ultimate resistance per unit length of pile as follows:

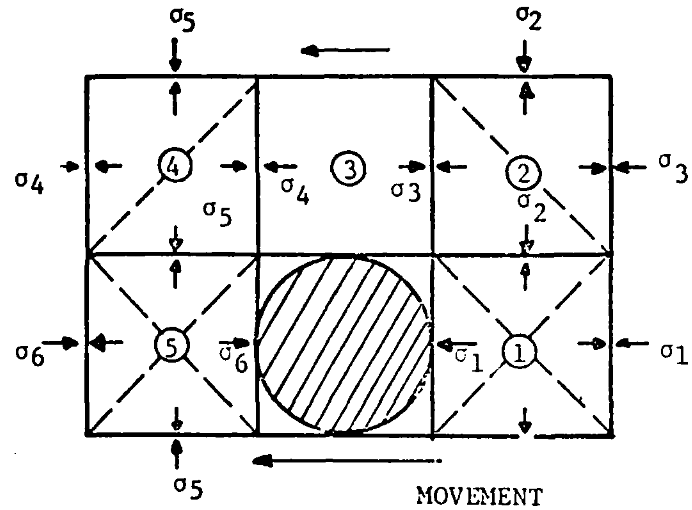
$$p_u = 2c_a b + \gamma bH + 2.83c_a H \qquad (42)$$

67. Equations 40 and 42 are approximate in that the two models give a greatly simplified picture of how saturated clay behaves in resistance to lateral loading. However, the theoretical expressions give a point of departure for using the results of experiments to arrive at more realistic expressions. The two equations can be solved simultaneously to find the depth at which the failure would change from the wedge type to the flow-around type.

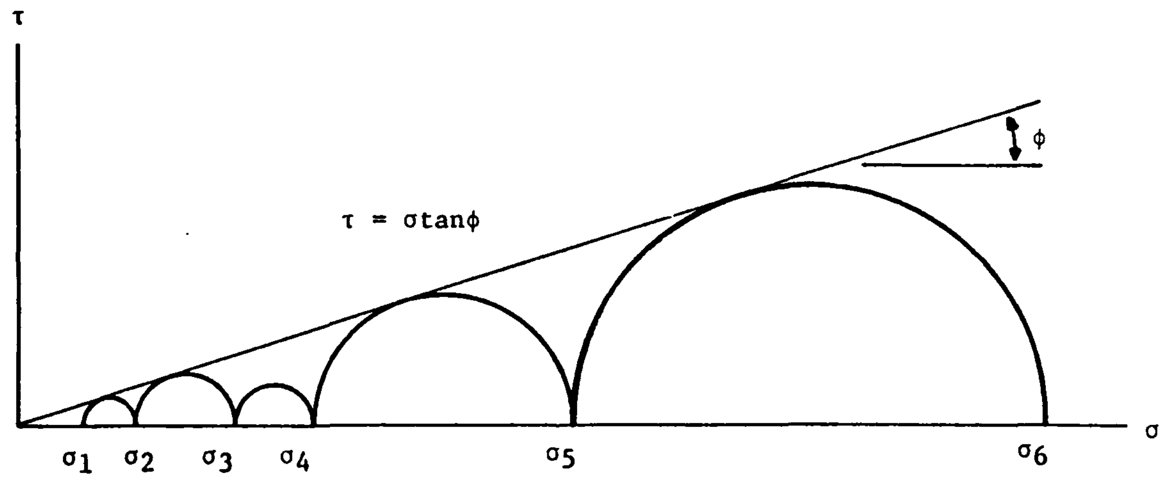
Sands

68. The expressions for determining the ultimate resistance of sand to the lateral movement of a pile can again be divided on the basis of two different failure mechanisms (group of sliding blocks or wedge).

69. The model for computing the ultimate soil resistance at a depth where the overburden is sufficient to enforce a plane strain condition is given in Figure 14. The stress σ_1 is obtained by assuming a Rankine active failure condition. This assumption is based on two-dimensional behavior and is subject to some uncertainty. However, the assumption should be adequate for present purposes because the developed equations will subsequently be adjusted to reflect observed conditions from field tests. If σ_1 is imposed as



a. Section through pile



b. Mohr-Coulomb diagram representing states of stress of soil flowing around a pile

Figure 14. Assumed mode of soil failure by lateral flow around the pile (Reese and Sullivan 1980)

the confining stress on block 1, the stress required to cause the failure of block 1 along the dashed lines would be approximately

$$\sigma_2 = \sigma_1 \tan^2 \left(45 + \frac{\phi}{2}\right) \tag{43}$$

where ϕ is the angle of internal friction of the sand. Assuming the states of stress shown in Figure 14b, block 2 would be required to fail along the dashed line because of the imposed stress of σ_3 . Block 3 could be assumed to move as a rigid unit. Continuing this line of reasoning leads to the establishment of the net force on the segment of pile as

$$p_u = b(\sigma_6 - \sigma_1)$$

$$p_u = K_a b\gamma H\ (\tan^8 \beta - 1) + K_o b\gamma H \tan \phi \tan^4 \beta \tag{44}$$

where

K_a = Rankine active earth pressure coefficient = $\tan^2 45 - (\phi/2)$

H = depth to the point under consideration

$\beta = 45 + (\phi/2)$

K_o = at-rest earth pressure coefficient

70. The ultimate soil resistance near the ground surface is computed using the free body shown in Figure 15. As can be seen in Figure 15c, the total ultimate lateral resistance F_{pt} on the pile is equal to the passive force F_p minus the active force F_a . The force F_a is computed from Rankine's theory using the minumum coefficient of active earth pressure. The passive force F_p is computed from the geometry of the wedge, assuming the Mohr-Coulomb failure theory to be valid for sand. The directions of the forces are shown in Figure 15b. By summing forces in the horizontal and vertical directions, the magnitudes of the forces F_a and F_p can be determined. No frictional force is assumed to be acting on the face of the pile. The equation for F_{pt} is

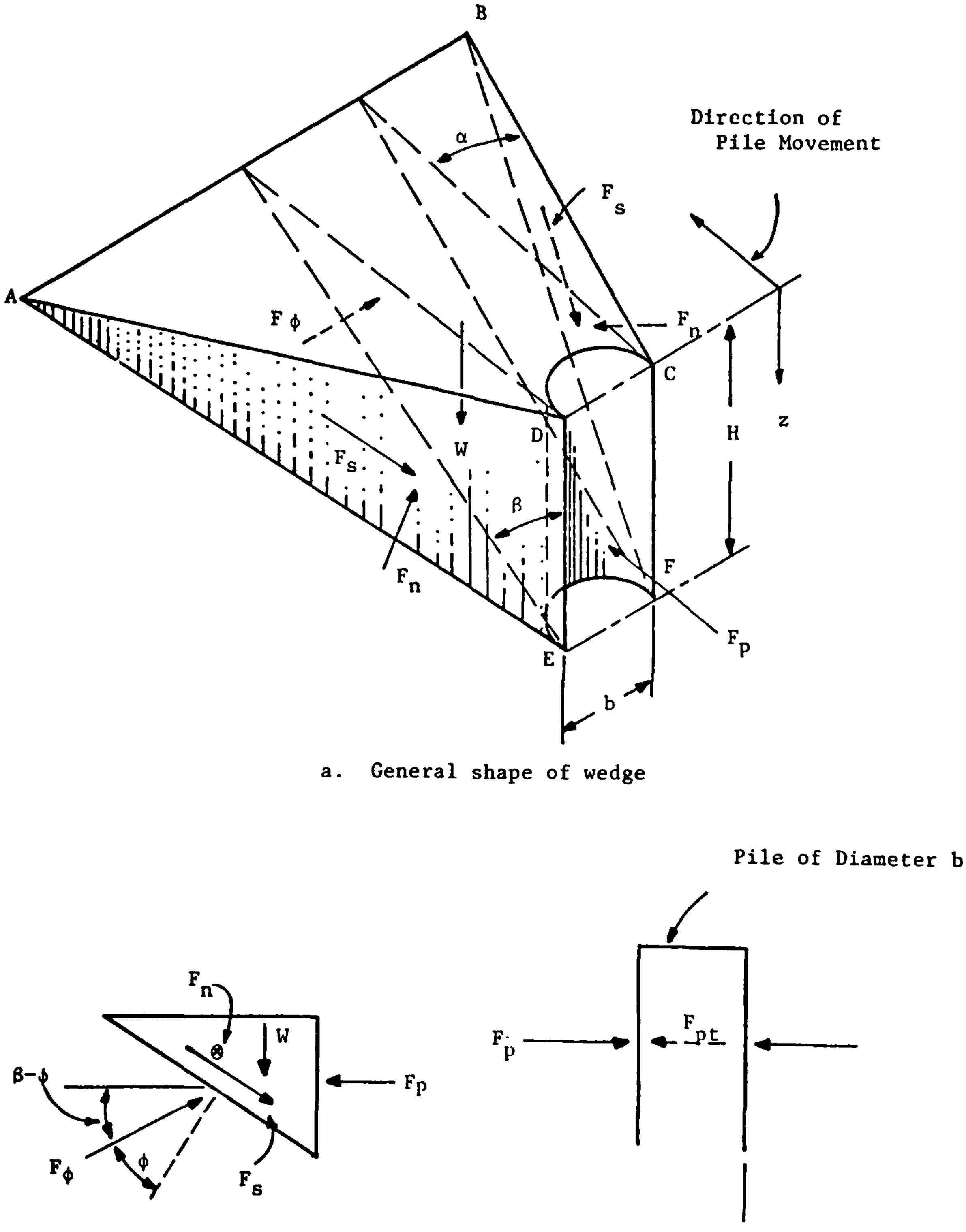
$$F_{pt} = \gamma H^2 \left[\frac{K_o H \tan \phi \sin \beta}{3 \tan (\beta - \phi) \cos \alpha} + \frac{\tan \beta}{\tan (\beta - \phi)} \left(\frac{b}{2} + \frac{H}{3} \tan \beta \tan \alpha \right) + K_o H \frac{\tan \beta}{3} (\tan \phi \sin \beta - \tan \alpha) - \frac{K_a b}{2} \right] \tag{45}$$

where

K_o = coefficient of earth pressure at rest

K_a = minimum coefficient of active earth pressure

71. The ultimate soil resistance per unit length of the pile at any depth can be obtained by differentiating the force F_{pt} with respect to the depth H . The result of that differentiation is given by



a. General shape of wedge

b. Forces acting on wedge

c. Forces acting on pile

Figure 15. Assumed passive wedge type of failure (Reese and Sullivan 1980)

$$p_u = \gamma H \left[\frac{K_o H \tan \phi \sin \beta}{\tan (\beta - \phi) \cos \alpha} + \frac{\tan \beta}{\tan (\beta - \phi)} \right.$$

$$\left. \times (b + H \tan \beta \tan \alpha) + K_o H \tan \beta (\tan \phi \sin \beta - \tan \alpha) - K_a b \right] \quad (46)$$

72. The values of the parameters in Equation 46 must be estimated using soil mechanics theory. Selection of the parameters will be discussed in the subsequent section on p-y curves.

73. Equations 44 and 46 can be solved simultaneously to find the approximate depth at which the soil changes from the wedge type to the flow-around type. Again, it should be emphasized that the equations are not expected to give perfect predictions of the ultimate soil resistance. However, correlating the equations with experimental results allows practical use of them and lends generality to the experimental results.

Experimental Techniques for Developing p-y Curves

74. The preceding paragraphs have described the basic theory utilized in correlating observed experimental p-y curves with theory. The following section describes several methods for obtaining experimental p-y curves.

Direct measurement

75. Direct measurement of p-y curves in the field would involve measuring the pile deflection at some predetermined points and then measuring the soil response corresponding with the measured deflection. Deflection can be measured by installing slope inclinometer casings either on the inside or on the surface of a pile and taking readings with a slope inclinometer. Alternatively, sighting down a hollow pile from a fixed position at scales that have been placed at intervals along the length of the pile has been used. This method is cumbersome in practice, however, and has not been very successful.

76. Measuring the soil response p is considerably more involved and difficult than measuring the deflection. The distribution of pressure acting on the pile must first be determined and then the pressure diagram integrated to determine soil response. Pressure meters of many different types are available and have been utilized in measuring pressures (Bierschwale, Coyle, and Bartoskewitz 1981). This approach requires measurement of the soil pressure at a few points around the exterior of a pile and estimation of soil

pressures between the pressure meters to obtain the pressure distribution. Whether or not this procedure yields accurate pressure distribution is a subject of debate (Reese and Sullivan 1980; Bierschwale, Coyle, and Bartoskewitz 1981).

Experimental moment curves

77. The method used most successfully at UT for determining p-y curves involves the placement of electrical resistance strain gages at points along the pile shaft. Before the field test is performed, strain readings are correlated with moment by placing the pile horizontally on simple supports and applying known moments. During the lateral load test, strain readings are taken at each point at each increment of load and converted to moment values by use of the moment calibration curves. Deflection values are obtained by use of Equation 47:

$$y = \iint \frac{M}{EI} \tag{47}$$

where

M = measured moment

EI = flexural stiffness of the pile

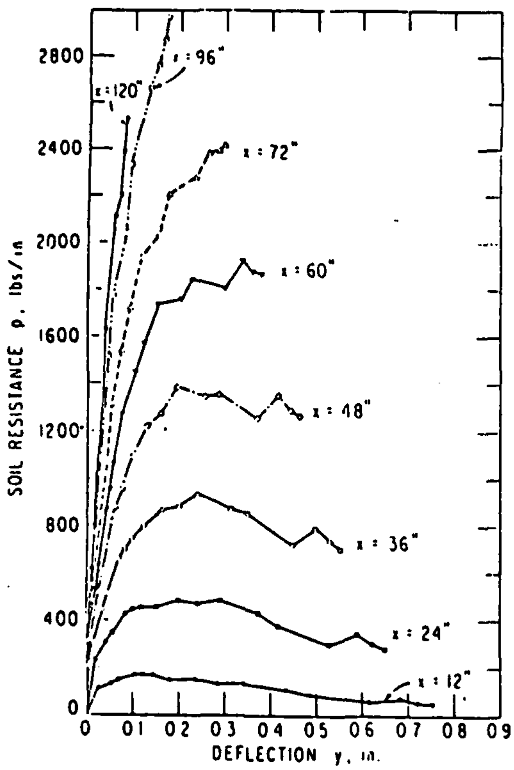
The deflection can be obtained with considerable accuracy using numerical procedures to doubly integrate the moment curves.

78. The computation of soil resistance is somewhat more difficult than determining deflections. It is obtained by double differentiation of the moment curves using Equation 48:

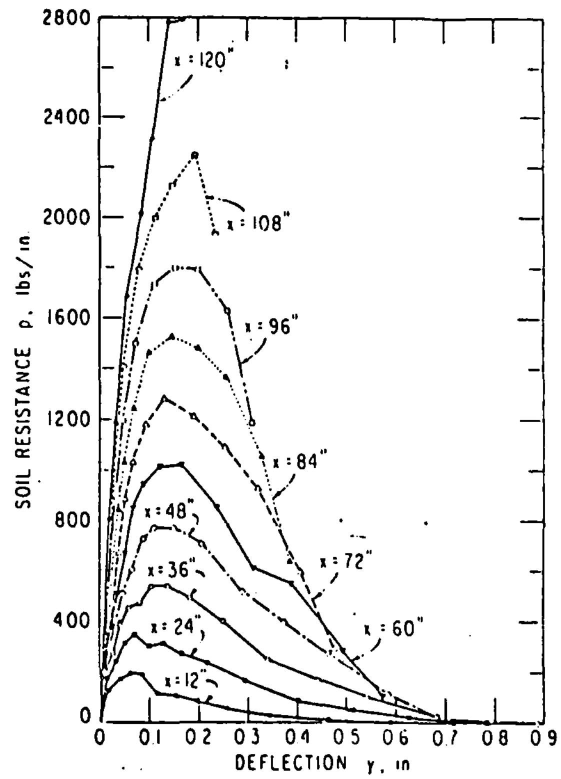
$$p = \frac{d^2M}{dx^2} \tag{48}$$

The difficulty in differentiating the moment curves lies in the fact that a curve fitted through data points is not necessarily accurate except at the data points and differentiation results can be erratic, particularly for double differentiation.

79. Taking the family of curves showing the distribution of deflection and soil resistance, p-y curves can be plotted as shown in Figure 16. The curves can be checked by performing an analysis using the field loads and comparing the results with the experimental moment curves as illustrated in Figure 17.



a. p-y curves developed from static-load test on 24-in. diameter pile



b. p-y curves developed from pile-load test on 24-in. diameter pile

Figure 16. Examples of experimental p-y curves from field test (Reese, Cox, and Koop 1975)

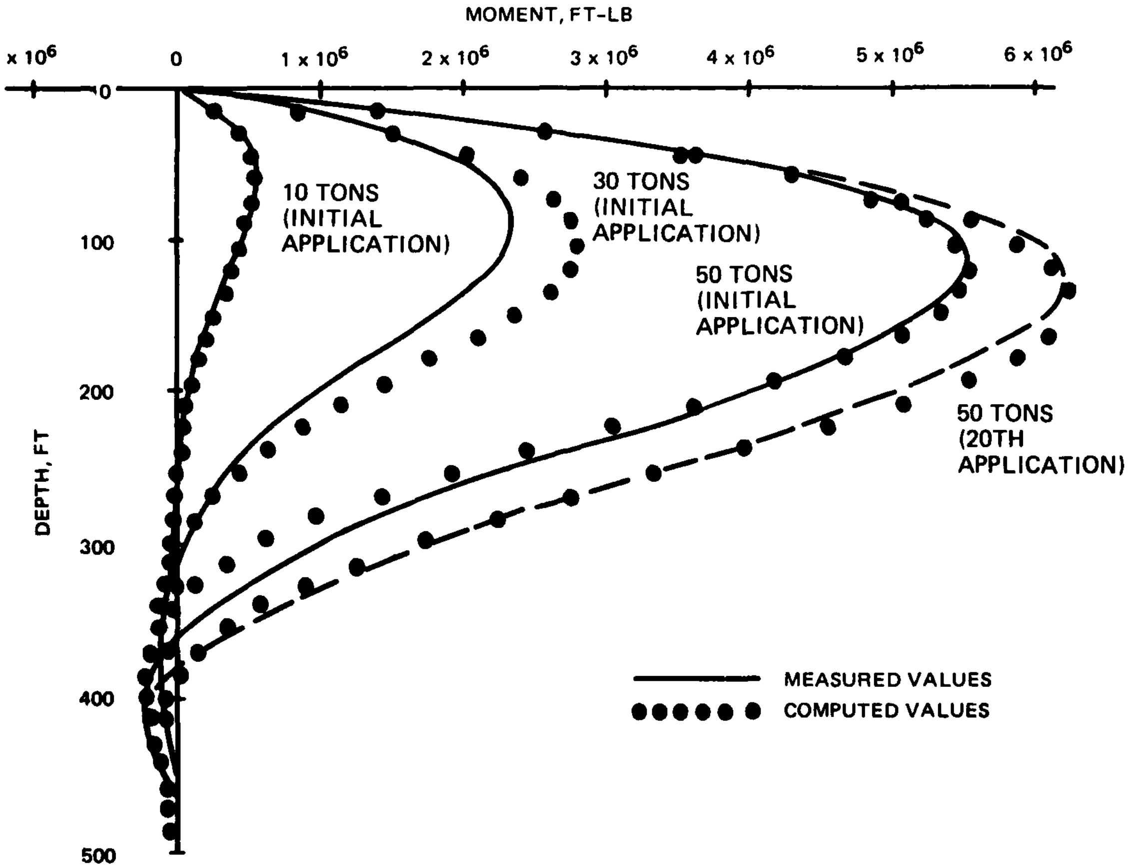


Figure 17. Computed and measured values of moment versus depth from a laterally loaded pile test (Welch and Reese 1972)

Nondimensional methods

80. Nondimensional methods have been used fairly successfully to obtain p-y curves from a lateral load test (Reese, Cox, and Koop 1974). The basis for this method is described in Appendix A. The procedure does not result in p-y curves which are as accurate as the curves obtained using strain gage data. The main advantage is that costly instrumentation is not required.

81. Deflection and slope are measured at the top of the pile after each increment of load is applied. The p-y curve is computed by first assuming a variation of soil modulus with depth for a particular load and then performing a nondimensional solution. This procedure is repeated until the assumed variation of soil modulus yields computed results which agree with the measured deflection and slope at the top of the pile. When the calculated slope and deflection agree with those measured, the assumed variation is taken to be correct. This "correct" modulus is used for the computer solution from which the deflection is obtained with depth. Given the soil modulus and the deflection, the value of resistance at desired depths can then be computed. One complete solution gives one point on the p-y curve at each depth being considered. The entire procedure is then repeated for each load to obtain additional points on the p-y curve.

Recommendations on Use of p-y Curves

82. Ideally, fully instrumented testing should be performed for each design involving laterally loaded piles. Unfortunately, the cost of load tests can often only be justified for large projects. On projects where fully instrumented lateral load tests can be justified, the tests should be performed at the specific site using the pile types and installation procedures to be utilized in construction. On intermediate-sized projects for which site-specific data are needed, but a fully instrumented lateral load test cannot be justified, the nondimensional methods for obtaining p-y curves presented by Reese and Cox (1968) are recommended. These methods are approximate; however, they require only pile head measurements which are relatively easy and economical to obtain and they provide project-specific data not available otherwise. In certain situations, the designer may also consider using a combination of instrumented pile testing and nondimensional methods. This can be accomplished by utilizing the slope inclinometer to obtain pile deflections while using

nondimensional methods to obtain soil resistance.

83. The p-y criteria presented in the remaining sections of this part of the report are provided for the purpose of assisting the designer in situations where laterally loaded pile tests cannot be justified. The designer must use the p-y criteria with extreme caution and a clear understanding of their limitations. Under no circumstances should a design be undertaken without a sufficient number of borings to define the subsurface profile and a sufficient number of soil tests to define the shear strength and the unit weight versus depth profile. Also, the designer should be ever mindful of the fact that any one set of p-y construction methods presented herein is strongly related to only one or two lateral load tests.

84. In performing analyses, the designer should, at a minimum, perform parametric studies to investigate the sensitivity of the results to the input parameters. For example, the load, boundary conditions, and parameters specific to developing the individual p-y curves should be varied to determine the parameters most critical to the design. The results of the parametric studies should then be considered in making design decisions. An example design problem is presented in Appendix B.

Curves for clays

85. The recommended p-y curves for clays were developed from three major test programs on three different types of clay soils: (a) soft clays below the water table, (b) stiff clays below the water table, and (c) stiff clays above the water table. In each test program, the piles were subjected to short-term static loads and to repeated (cyclic) loads. The test program is described briefly for each set of p-y criteria in the following paragraphs. In addition, step-by-step procedures are given for computing the p-y curves, recommendations are given for obtaining the necessary data on soil properties, and example curves are presented.

86. The final portion of this section on clays presents a method that has been developed for predicting p-y curves for clays below the water table of any shear strength. This "unified" method (Sullivan, Reese, and Fenske 1979) is based on all of the major experiments in clay below the water table.

Response of soft
clay below the water table

87. Field experiments. The research program leading to the development of p-y criteria for soft clay was carried out and reported by Matlock (1970).

The research involved extensive field testing with an instrumented pile, experiments with laboratory models, and parallel development of analytical methods and correlations.

88. There were two test sites: one at Lake Austin in Austin, Tex., and the other at the mouth of the Sabine River, which forms much of the Texas-Louisiana border. The soils at the Lake Austin site consisted of clays and silts, somewhat jointed and fissured due to desiccation during periods of low water with vane shear strengths averaging about 800 pcf. The Sabine clay appeared to be a more typical, slightly overconsolidated marine deposit with vane shear strengths averaging about 300 pcf in the significant upper zone.

89. A steel test pile 12.75 in. in diameter with an embedded length of 42 ft was used at both test sites. The pile contained 35 pairs of electrical resistance strain gages which were calibrated to provide extremely accurate determinations of bending moment. Gage spacings varied from 6 in. near the top to 4 ft in the lowest section. Tests were performed (a) with the pile head free to rotate and (b) with the pile head restrained against rotation to determine what difference there might be in the soil response due to different boundary conditions. The free-head tests were performed with only a lateral load applied at the mudline. The restrained head tests utilized a framework to simulate the effect of a jacket-type structure, as shown in Figure 18. Short-time static loading and cyclic loading were used in testing the pile. The moment curves obtained in the tests were differentiated to determine soil resistance and integrated to obtain pile deflection.

90. In addition to field experiments, some laboratory experiments were performed which were of value in explaining the nature of deterioration of soil resistance. These experiments were not utilized directly in constructing the p-y criteria, but were of use in explaining and interpreting the field data. Principal conclusions from the tests are listed below:

- a. The resistance-deflection characteristics of the soil were highly nonlinear and inelastic.
- b. Within practical ranges, the degree of pile head restraint appeared to have no effect on the p-y relationship.
- c. Cyclic loading produced a permanent physical displacement of the soil away from the pile in the direction of loading.
- d. The permanent displacement of the soil away from the pile produced a slack zone in the p-y relationship. Upon reloading

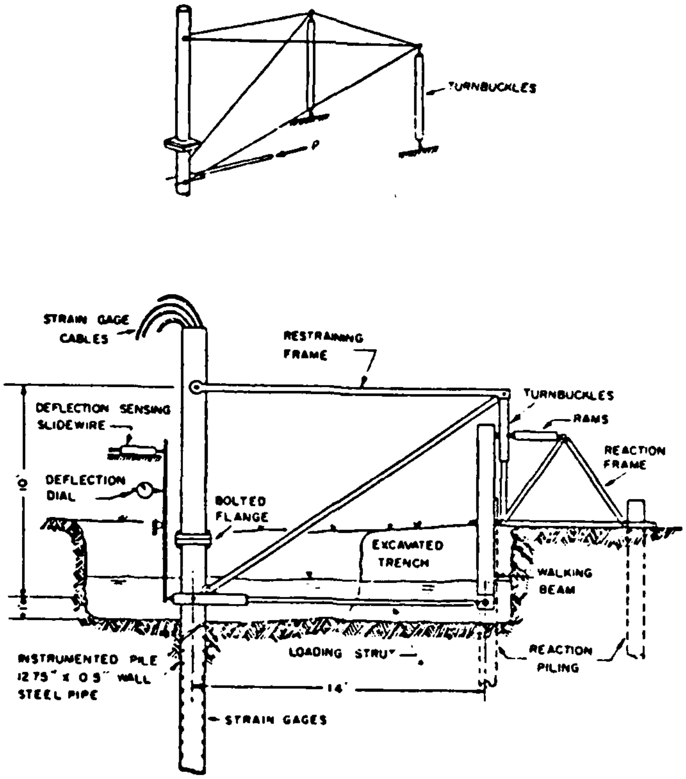


Figure 18. Arrangement for field tests at Sabine River site using restrained-head lateral loading (Matlock 1970)

the pile, this slack zone was reflected in bending moments which were much higher than those produced by equal loads during the initial cyclic series.

e. During cyclic loading with a constant load, the deflections and moments would gradually increase with each repetition, but the rate of increase diminished to the point where the soil-pile system practically stabilized and no further increases in deflections or moments occurred with continued repetitions of load. It can be intuitively seen that some upper limit of load must exist for any pile above which the system would not stabilize under cyclic loading, and this conclusion was borne out by the tests. Below this upper limit, stabilization generally occurred in less than 100 cycles.

f. The measured ultimate resistance near the surface was similar to the theoretical ultimate resistance as expressed in Equation 42.

g. If the p-y data resulting from the tests are plotted in non-dimensional form on log-log paper, a relatively smooth straight line can be fitted to the data up to the value of ultimate resistance. This result will be illustrated in the directions for constructing the p-y curves.

91. The details of the experiments for the soft-clay criteria are discussed more thoroughly here than will be the case for the remaining criteria. The discussion is primarily intended to provide the user with a clearer understanding of the experiments which provide the basis for the p-y criteria.

92. Recommendations for computing p-y curves. The following procedure is for short-term static loading and is illustrated by Figure 19a.

a. Obtain the best possible estimate of the variation of undrained shear strength c and submerged unit weight with depth x . Also, obtain the values of ε_{50} , the strain corresponding to half the maximum principal stress difference. If no stress-strain curves are available, typical values of ε_{50} given in Table 3 can be used.

Table 3

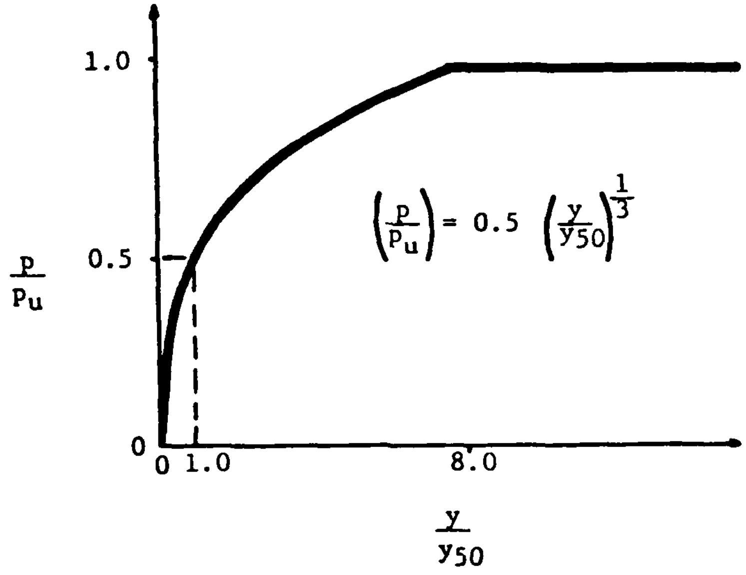
Representative Values of ε_{50}

Shear Strength c , psf	ε_{50} percent
250-500	2
500-1000	1
1000-2000	0.7
2000-4000	0.5
4000-8000	0.4

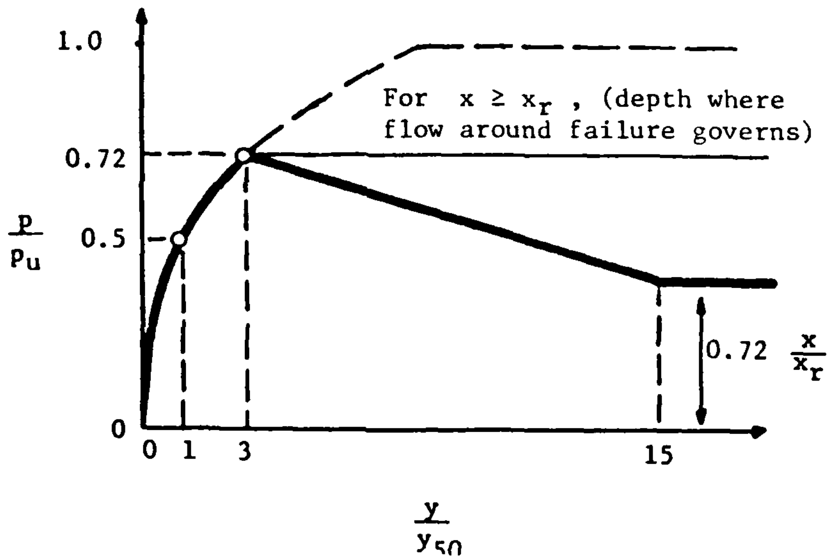
b. Compute the ultimate soil resistance per unit length of pile, using the smaller of the values given by the equations below:

$$p_u = \left(3 + \frac{\gamma'}{c} x + \frac{J}{b} x\right)(cb) \qquad (49)$$

$$p_u = 9cb \qquad (50)$$



a. Static loading



b. Cyclic loading

Figure 19. Characteristic shapes of the p-y curves for soft clay below the water surface (Matlock 1970)

where

γ' = average effective unit weight from the ground surface to the p-y curve

c = shear strength at depth x

x = depth from the ground surface to the p-y curve

b = width of the pile

Matlock (1970) states that the values of J were determined experimentally to be 0.5 for a soft clay and about 0.25 for a medium clay. A value of 0.5 is frequently used. The value of p_u is computed at each depth where a p-y curve is desired, based on shear strength at that depth.

c. Compute the deflection y_{50} at half the ultimate soil resistance from the following equation:

$$y_{50} = 2.5\varepsilon_{50}b \tag{51}$$

d. Points describing the p-y curve are now computed from the following relationship:

$$\frac{p}{p_u} = 0.5\left(\frac{y}{y_{50}}\right)^{1/3} \tag{52}$$

The value of p remains constant beyond $y = 8y_{50}$.

93. The following procedure is for cyclic loading and is illustrated in Figure 19b.

a. Construct the p-y curve in the same manner as for short-term static loading for values of p less than $0.72p_u$.

b. Solve Equations 49 and 50 simultaneously to find the depth x_r where the transition occurs. If the unit weight and shear strength are constant in the upper zone, then

$$x_r = \frac{6cb}{(\gamma b + Jc)} \tag{53}$$

If the unit weight and shear strength vary with depth, the value of x_r should be computed with the soil properties at the depth where the p-y curve is desired.

c. If the depth to the p-y curve is greater than or equal to x_r , then p is equal to $0.72p_u$ for all values of y greater than $3y_{50}$.

d. If the depth to the p-y curve is less than x_r , then the value of p decreases from $0.72p_u$ at $y = 3y_{50}$ to the value given by the following expression at $y = 15y_{50}$:

$$p = 0.72p_u\left(\frac{x}{x_r}\right) \tag{54}$$

The value of p remains constant beyond $y = 15y_{50}$.

94. Recommended soil tests. For determining the various shear strengths of the soil required in the p-y construction, Matlock (1970) recommended the following tests in order of preference.

a. In situ vane-shear tests with parallel sampling for soil identification.

b. Unconsolidated, undrained triaxial compression tests having a confining stress equal to the overburden pressure, with c being defined as half the total maximum principal stress difference.

c. Miniature vane tests of samples in tubes.

d. Unconfined compression tests.

Tests must also be performed to determine the unit weight of the soil.

95. Example curves. An example set of p-y curves was computed for soft clay for a pile with a diameter of 48 in. The soil profile that was used is shown in Figure 20. In the absence of a stress-strain curve for the soil, ε_{50} was taken as 0.01 for the full depth of the soil profile. The loading was assumed to be both static and cyclic.

96. p-y curves were computed for the following depths below the mudline: 0, 1, 2, 4, 8, 12, 20, 40, and 60 ft. The plotted curves are shown in Figure 21 for static loading and in Figure 22 for cyclic loading.

Response of stiff clay below the water table

97. Field experiments. Reese, Cox, and Koop (1975) performed lateral load tests employing steel pipe piles that were 24 in. in diameter and 50 ft long. The piles were driven into stiff clay at a site near Manor, Tex. The clay had an undrained shear strength ranging from about 1 tsf at the ground surface to about 3 tsf at a depth of 12 ft.

98. Recommendations for computing p-y curves. The following procedure is for short-term static loading and is illustrated by Figure 23.

a. Obtain values for undrained soil shear strength c , soil submerged unit weight γ' , and pile diameter b .

b. Compute the average undrained soil shear strength c over the depth x .

c. Compute the ultimate soil resistance per unit length of pile using the smaller of the values given by the equations

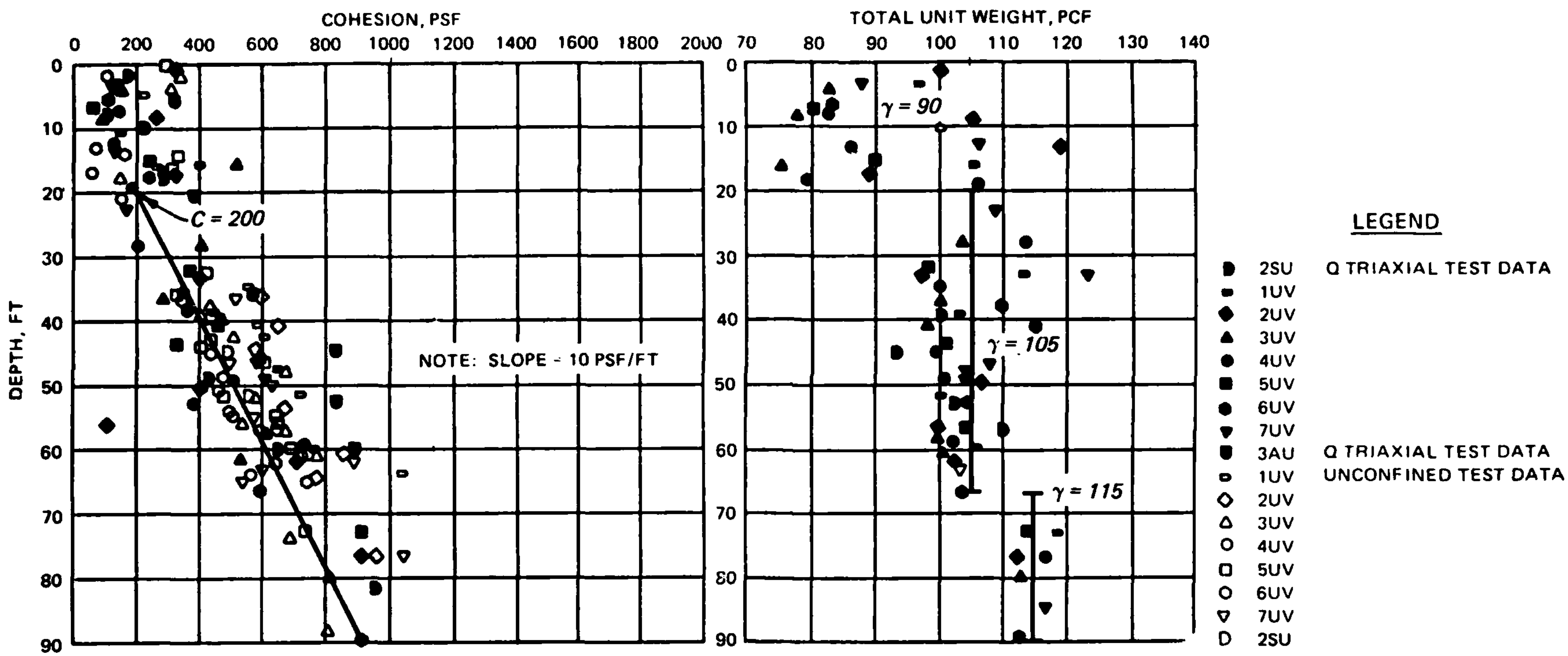


Figure 20. Soil profile used for example p-y curves for soft clay

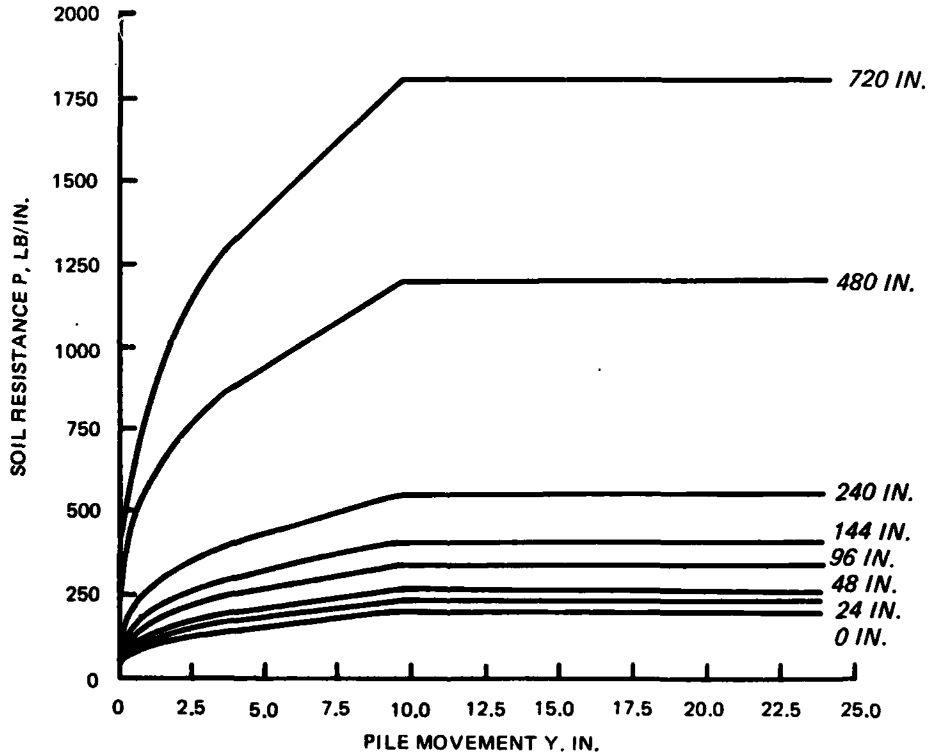


Figure 21. Example p-y curves for soft clay below the water table; Matlock criteria, static loading

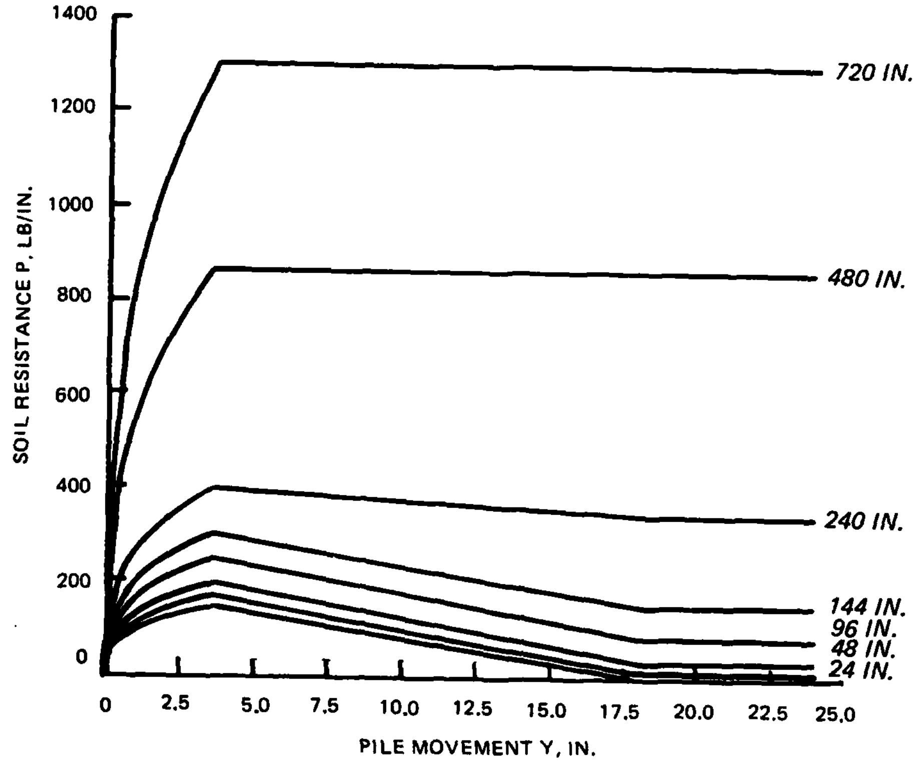


Figure 22. Example p-y curves for soft clay below the water table; Matlock criteria, cyclic loading

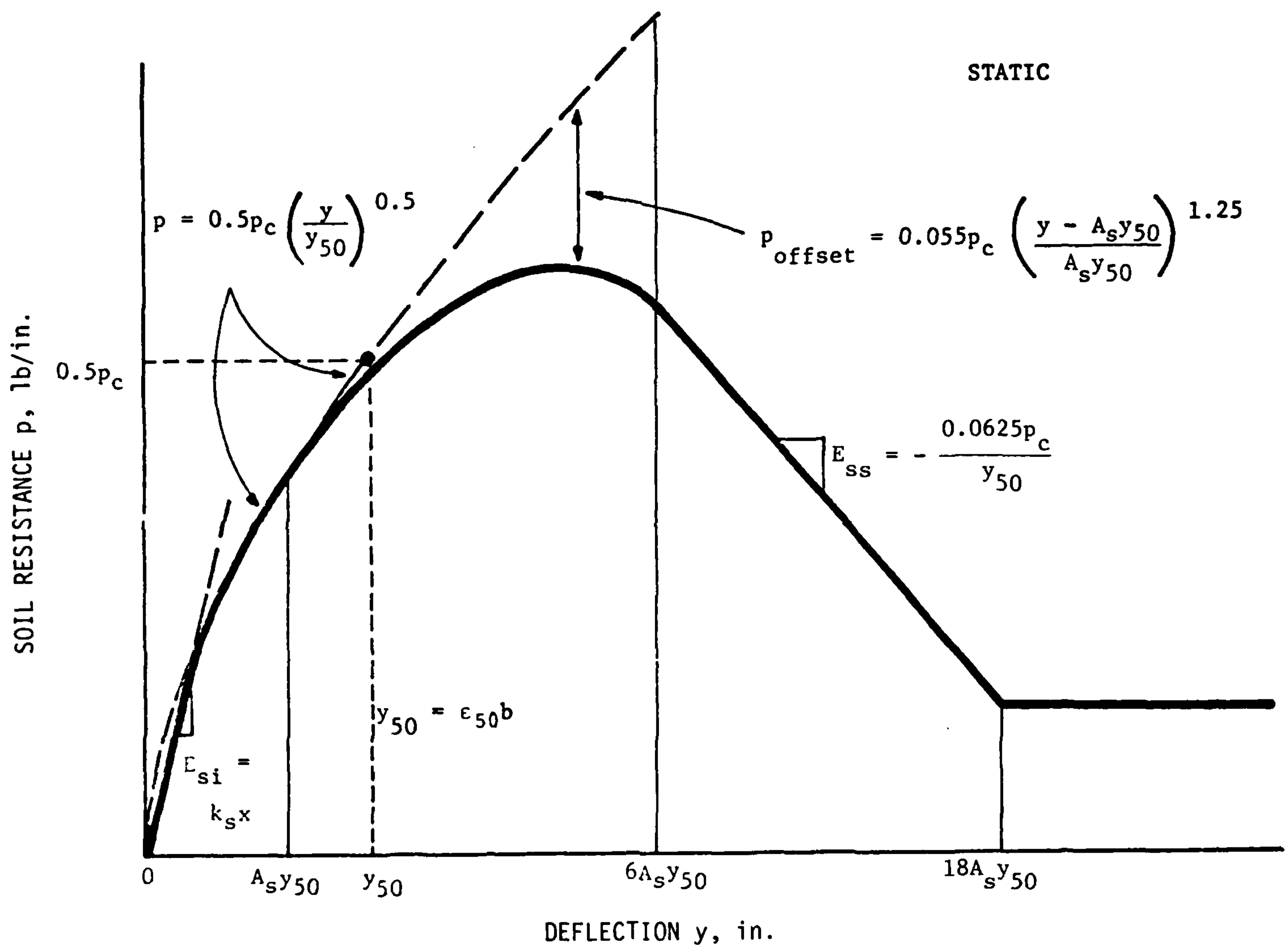


Figure 23. Characteristic shape of p-y curve for static loading in stiff clay below the water surface (Reese, Cox, and Koop 1975)

$$p_{ct} = 2cb + \gamma' bx + 2.83cx \quad (55)$$

$$p_{cd} = 11cb \quad (56)$$

d. Choose the approximate value of A_s from Figure 24 for the particular nondimensional depth.

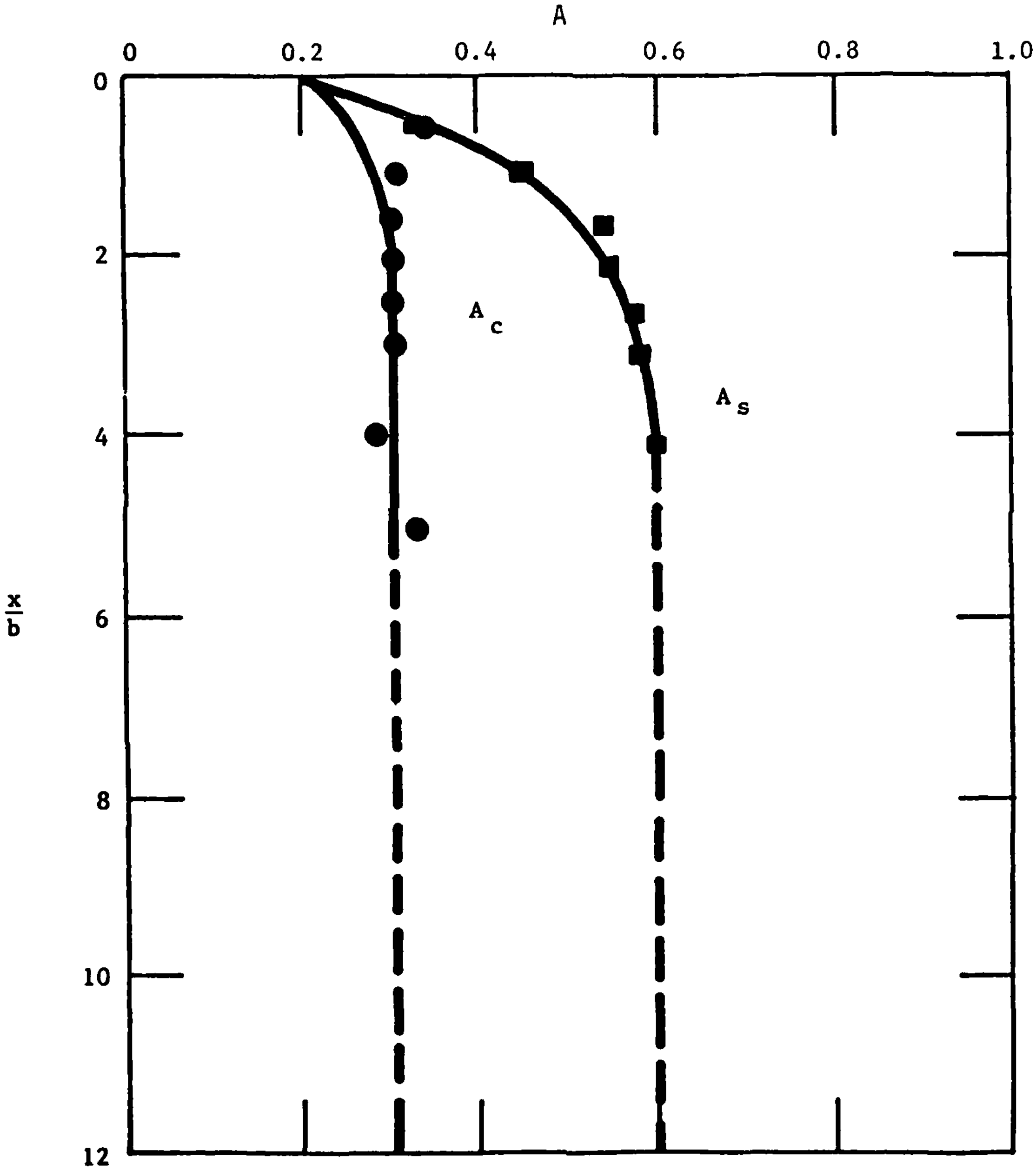


Figure 24. Values of the constants A_s and A_c (Reese, Cox, and Koop 1975)

e. Establish the initial straight-line portion of the p-y curve

$$p = (kx)y \tag{57}$$

Use the appropriate value of k_s or k_c from Table 4 for k.

Table 4

Representative Values of k for Stiff Clays

	Average Undrained Shear Strength,* tsf		
	0.5-1	1-2	2-4
k_s (static), pci	500	1000	2000
k_c (cyclic), pci	200	400	800

* The average shear strength should be computed from the shear strength of the soil to a depth of five pile diameters. It should be defined as half the total maximum principal stress difference in an unconsolidated undrained triaxial test. (Also see Table 6.)

f. Compute the following:

$$y_{50} = \varepsilon_{50} b \tag{58}$$

Use an appropriate value of ε_{50} from results of laboratory tests or, in the absence of laboratory tests, from Table 3.

g. Establish the first parabolic portion of the p-y curve using the following equation and obtaining p_c from Equation 55 or 56:

$$p = 0.5p_c\left(\frac{y}{y_{50}}\right)^{0.5} \tag{59}$$

Equation 59 could define the portion of the p-y curve from the point of the intersection with Equation 59 to a point where y is equal to $A_s y_{50}$ (see note after step j).

h. Establish the second parabolic portion of the p-y curve,

$$p = 0.5p_c\left(\frac{y}{y_{50}}\right)^{0.5} - 0.055p_c\left(\frac{y - A_s y_{50}}{A_s y_{50}}\right)^{1.25} \quad (60)$$

Equation 60 should define the portion of the p-y curve from the point where y is equal to $A_s y_{50}$ to a point where y is equal to $6A_s y_{50}$ (see note after step j).

i. Establish the next straight-line portion of the p-y curve,

$$p = 0.5p_c(6A_s)^{0.5} - 0.411p_c - \frac{0.0625}{y_{50}} p_c(y - 6A_s y_{50}) \quad (61)$$

Equation 61 should define the portion of the p-y curve from the point where y is equal to $6A_s y_{50}$ to a point where y is equal to $18A_s y_{50}$ (see note after step j).

j. Establish the final straight-line portion of the p-y curve,

$$p = 0.5p_c(6A_s)^{0.5} - 0.411p_c - 0.75p_c A_s \quad (62)$$

$$p = p_c(1.225\sqrt{A_s} - 0.75A_s - 0.411) \quad (63)$$

Equation 62 should define the portion of the p-y curve from the point where y is equal to $18A_s y_{50}$ and for all larger values of y (see following note).

(Note: The step-by-step procedure is outlined, and Figure 23 is drawn, as if there is an intersection between Equations 57 and 59. However, there may be no intersection of Equation 57 with any of the other equations defining the p-y curve. Equation 57 defines the p-y curve until it intersects with one of the other equations or, if no intersection occurs, Equation 57 defines the complete p-y curve.)

99. The following procedure is used for computing p-y curves in which loading is cyclic (see Figure 25).

a. Steps a, b, c, e, and f are the same as for the static case.

d. Choose the appropriate value of A_c from Figure 24 for the particular nondimensional depth.

$$y_p = 4.1A_c y_{50} \quad (64)$$

Compute the following.

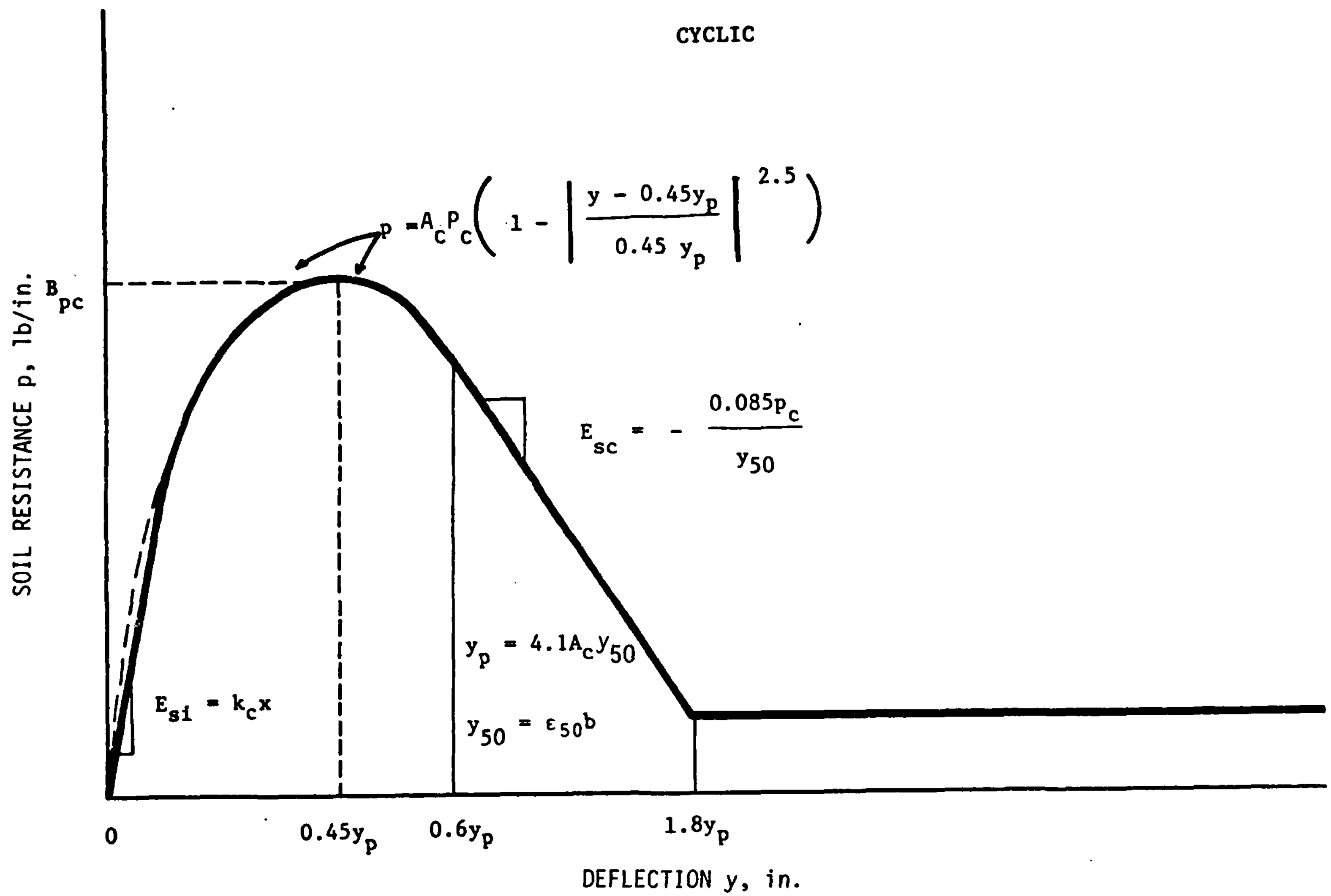


Figure 25. Characteristic shape of p-y curve for cyclic loading in stiff clay below the water surface (Reese, Cox, and Koop 1975)

g. Establish the parabolic portion of the p-y curve,

$$p = A_c p_c \left(1 - \left|\frac{y - 0.45y_p}{0.45y_p}\right|^{2.5}\right) \qquad (65)$$

Equation 65 should define the portion of the p-y curve from the point of the intersection with Equation 57 to the point where y is equal to $0.6y_p$ (see note after step i).

h. Establish the next straight-line portion of the p-y curve,

$$p = 0.936A_c p_c - \frac{0.085}{y_{50}} p_c(y - 0.6y_p) \qquad (66)$$

Equation 66 should define the portion of the p-y curve from the point where y is equal to $0.6y_p$ to the point where y is equal to $1.8y_p$ (see note after step i).

i. Establish the final straight-line portion of the p-y curve,

$$p = 0.936A_c p_c - \frac{0.102}{y_{50}} p_c y_p \qquad (67)$$

Equation 67 should define the portion of the p-y curve from the point where y is equal to $1.8y_p$ and for all larger values of y (see following note).

(Note: The step-by-step procedure is outlined, and Figure 25 is drawn, as if there is an intersection between Equations 57 and 65. However, there may be no intersection of those two equations, and there may be no intersection of Equation 57 with any of the other equations defining the p-y curve. If there is no intersection, the equation should be employed that gives the smallest value of p for any value of y.)

100. Recommended soil tests. Triaxial compression tests of the unconsolidated, undrained (Q) type with confining pressures conforming to in situ pressures are recommended for determining the shear strength of the soil. The value of ε_{50} should be taken as the strain during testing which corresponds to a stress equalling one-half the maximum total principal stress difference. The shear strength c should be interpreted as half of the maximum total stress difference. Values obtained from the triaxial tests might be somewhat conservative but would represent more realistic strength values than any from other tests. The unit weight of the soil must also be determined.

101. Example curves. Example sets of p-y curves were computed for stiff clay using a pile with a diameter of 48 in. The soil profile that was used is shown in Figure 26. The submerged unit weight of the soil was assumed to be 50 pcf for the entire depth. In the absence of a stress-strain curve, ε_{50} was taken as 0.005 for the full depth of the soil profile. The slope of the initial portion of the p-y curves was established by assuming a value of k_s of 1000 pci and a value of k_c of 400 pci. The loading was assumed to be both static and cyclic.

102. The p-y curves were computed for the following depths below the mudline: 0, 1, 2, 4, 8, 12, 20, 40, and 60 ft. The plotted curves are shown in Figure 27 for static loading and in Figure 28 for cyclic loading.

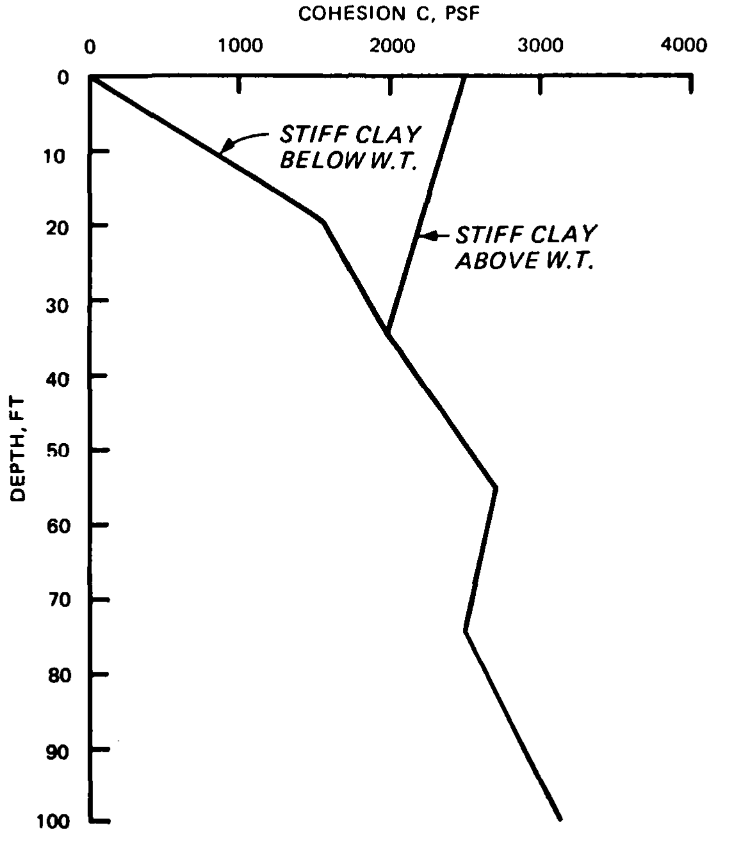


Figure 26. Soil profile used for example p-y curves for stiff clay

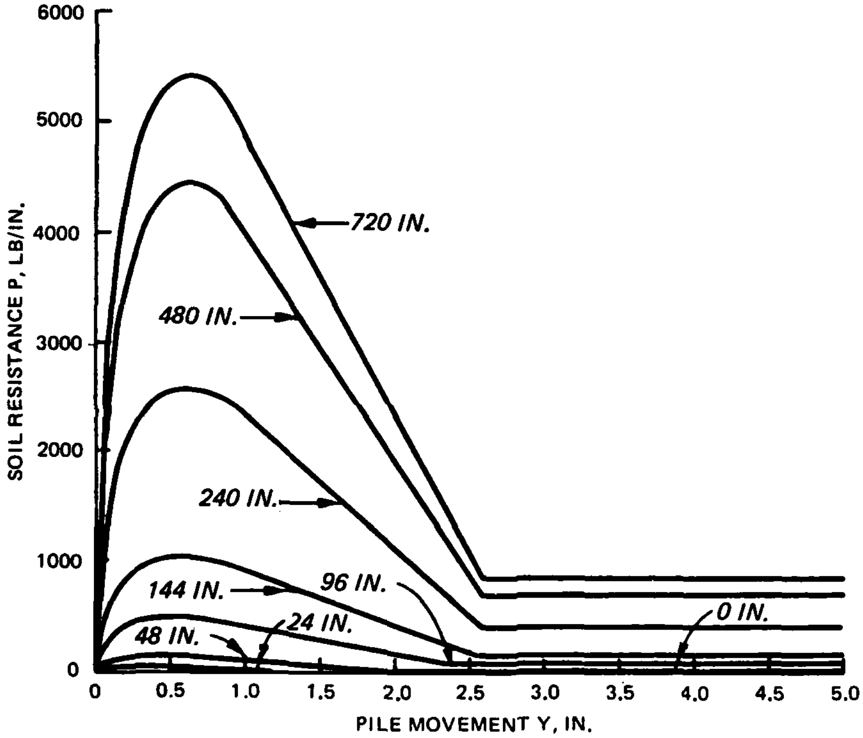


Figure 27. Example p-y curves for stiff clay below the water table; Reese criteria, static loading

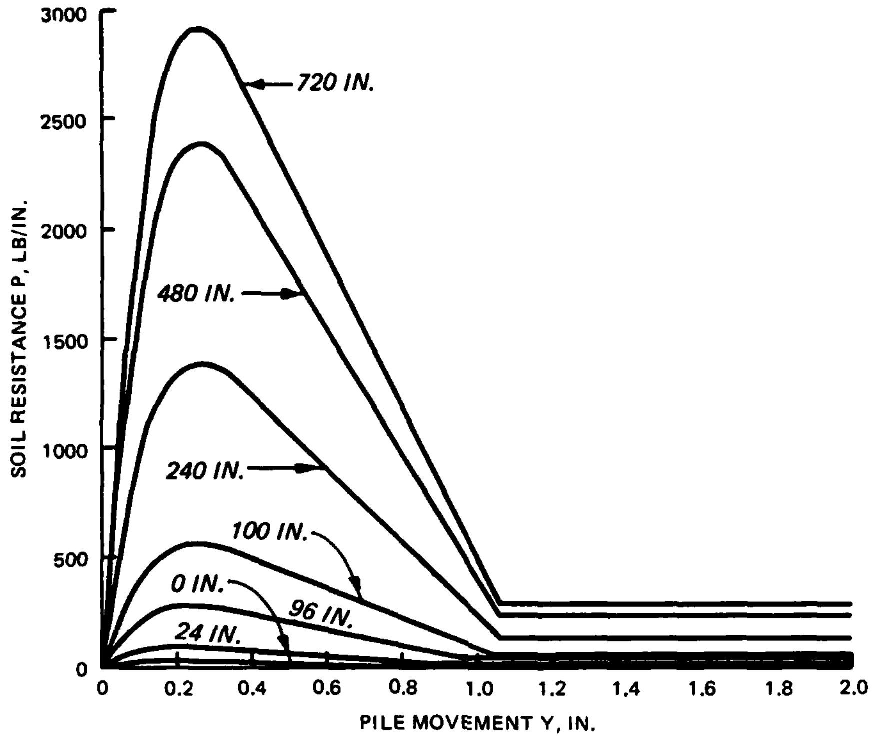


Figure 28. Example p-y curves for stiff clay below the water table; Reese criteria, cyclic loading

Response of stiff clay above the water table

103. Field experiments. A lateral load test was performed at a site in Houston, Tex., where the foundation was a drilled shaft, 36 in. in diameter. A 10-in.-diam pipe, instrumented at intervals along its length with electrical-resistance strain gages, was positioned along the axis of the shaft before concrete was placed. The embedded length of the shaft was 42 ft. The average undrained shear strength of the clay in the upper 20 ft was approximately 2200 psf. The experiments and their interpretation are discussed in detail by Welch and Reese (1972) and Reese and Welch (1975).

104. Recommendations for computing p-y curves. The following procedure is for short-term static loading and is illustrated in Figure 29:

a. Obtain values for undrained shear strength c, soil unit weight γ, and pile diameter b. Also obtain the values of ε_{50} from stress-strain curves. If no stress-strain curves are available, use a value of ε_{50} of 0.010 or 0.005 as given in Table 3, the larger value being more conservative.

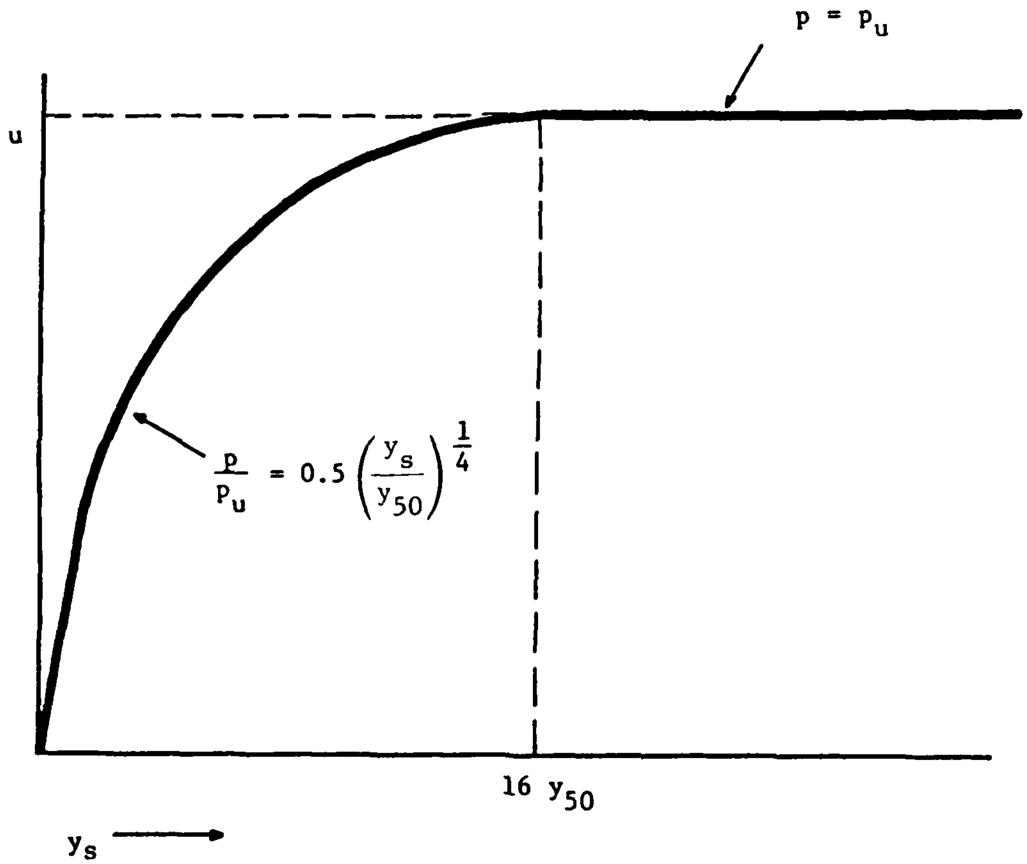


Figure 29. Characteristic shape of p-y curve for static loading in stiff clay above the water table (Reese and Sullivan 1980)

b. Compute the ultimate soil resistance per unit length of shaft p_u using the smaller of the values given by Equations 49 and 50. (In the use of Equation 49, the shear strength is taken as the average from the ground surface to the depth being considered, and J is taken as 0.5. The unit weight of the soil should reflect the position of the water table.)

c. Compute the deflection y_{50} at half the ultimate soil resistance from Equation 51.

d. Points describing the p-y curve may be computed from the relationship below.

$$\frac{p}{p_u} = 0.5\left(\frac{y}{y_{50}}\right)^{1/4} \quad (68)$$

$\underline{e}$. Beyond $y = 16y_{50}$, p is equal to p_u for all values of y .

105. The following procedure is for cyclic loading and is illustrated in Figure 30:

$\underline{a}$. Determine the p-y curve for short-term static loading by the procedure previously given.

$\underline{b}$. Determine the number of times the design lateral load will be applied to the pile.

$\underline{c}$. For several values of p/p_u , obtain the value of C , the parameter describing the effect of repeated loading on deformation, from a relationship developed through laboratory tests (Welch and Reese 1972) or, in the absence of tests, from the following equation:

$$C = 9.6 \left(\frac{p}{p_u}\right)^4 \tag{69}$$

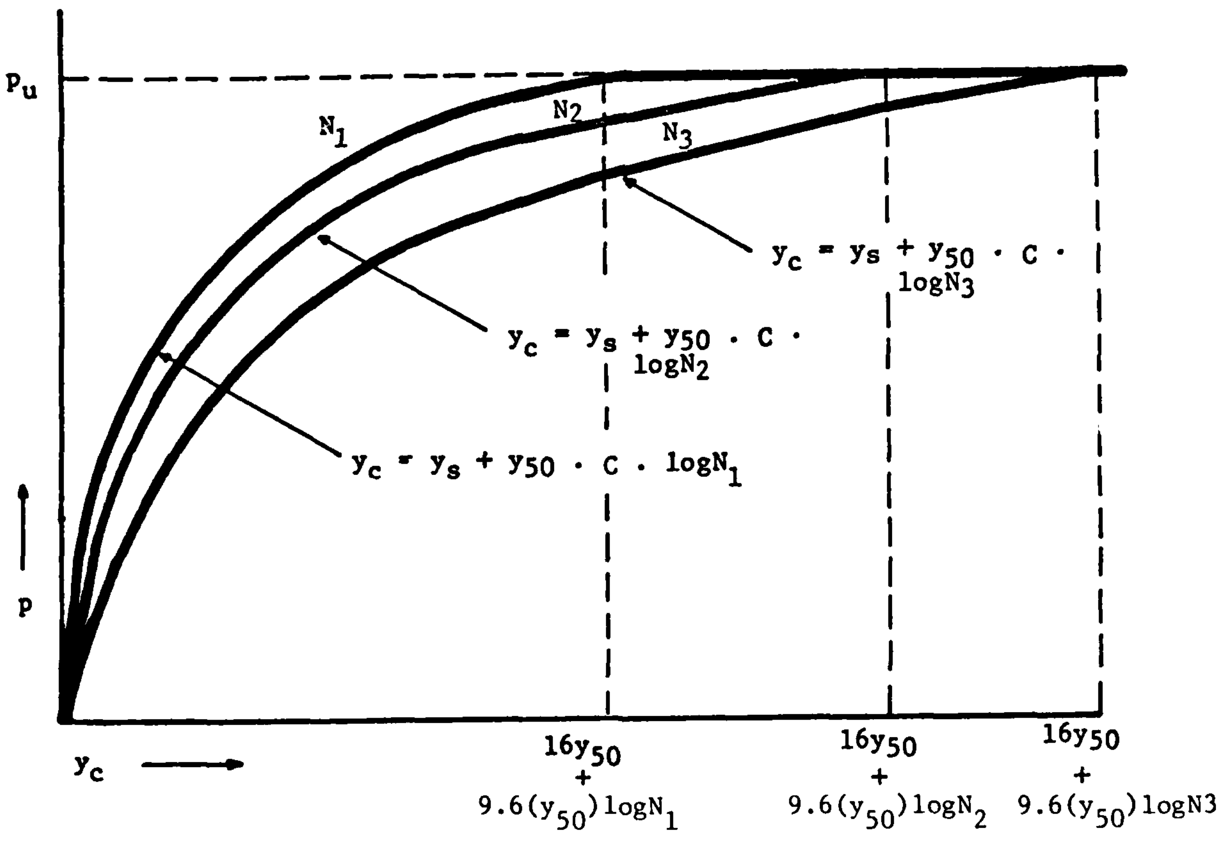


Figure 30. Characteristic shape of p-y curve for cyclic loading in stiff clay above the water table (Reese and Sullivan 1980)

d. At the value of p corresponding to the values of p/p_u selected in step c, compute new values of y for cyclic loading from

$$y_c = y_s + (y_{50})C \log N \quad (70)$$

where

y_c = deflection under N cycles of load

y_s = deflection under a short-term static load

y_{50} = deflection under a short-term static load at half the ultimate resistance

N = number of cycles of load application

e. The p-y curve defines the soil response after N cycles of load.

106. Recommended soil tests. Triaxial compression tests of the unconsolidated, undrained (Q) type with confining stresses equal to the overburden pressures at the elevations from which the samples were taken are recommended to determine the shear strength. The values of ε_{50} should be taken as the strain during the test corresponding to the stress equal to half the maximum total principal stress difference. The undrained shear strength c should be defined as half the maximum total principal stress difference. The unit weight of the soil must also be determined.

107. Example curves. An example set of p-y curves was computed for stiff clay above the water table for a pile with a diameter of 43 in. The soil profile that was used is shown in Figure 26. The unit weight of the soil was assumed to be 112 pcf for the entire depth. In the absence of a stress-strain curve, ε_{50} was taken as 0.005. The p-y curves were computed for both static and cyclic loadings. Equation 69 was used to compute values for the parameter C for cyclic loadings, and it was assumed that there are to be 100 cycles of load application.

108. p-y curves were computed for the following depths below the ground surface: 0, 1, 2, 4, 8, 12, 20, 40, and 60 ft. The plotted curves are shown in Figure 31 for static loading and in Figure 32 for cyclic loading.

Unified criteria for clays below the water table

109. Introduction. As was noted in the previous section, no recommendations were made for ascertaining the range of undrained shear strength in

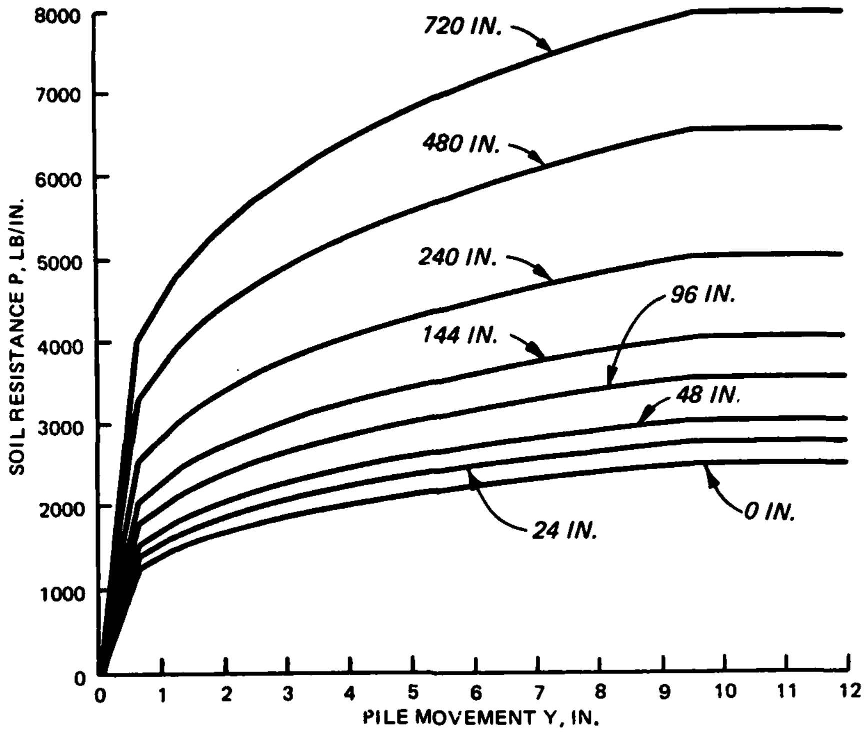


Figure 31. Example p-y curves for stiff clay above the water table; Reese and Welch criteria, static loading

which the criteria for soft clay versus those for stiff clay should be used. Sullivan (1977) and Sullivan, Reese, and Fenske (1979) examined the original experiments and developed a set of recommendations that yield computed behaviors in reasonably good agreement with the experimental results from the Sabine River tests reported by Matlock (1970) and with those from the Manor, Tex., tests reported by Reese, Cox, and Koop (1975). However, as will be seen from the following presentation, there is a need for the user to employ some judgment in selecting appropriate parameters for use in the prediction equations.

110. Recommendations for computing p-y curves. The following procedure is for short term static loading and is illustrated in Figure 33:

a. Obtain values for the undrained shear strength c, the submerged unit of weight γ', and the pile diameter b. Also, obtain values of ε_{50} from stress-strain curves. If no stress-strain curves are available, the values in Table 3 can be used as guidelines for selection of ε_{50}.

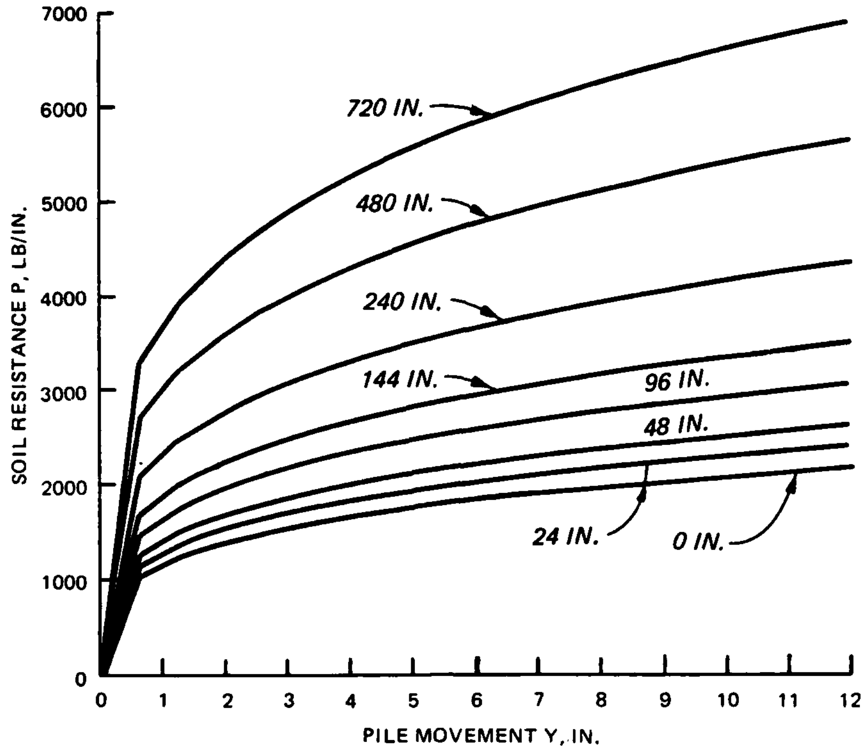


Figure 32. Example p-y curves for stiff clay above the water table; Reese and Welch criteria, cyclic loading

b. Compute c_a and $\bar{\sigma}_v$, for $x < 12b$, where

c_a = average undrained shear strength
$\bar{\sigma}_v$ = average effective stress
x = depth

c. Compute the variation of p_u with depth using the equation below:

(1) For $x < 12b$, p_u is the smaller of the values computed from

$$p_u = \left(2 + \frac{\bar{\sigma}_v}{c_a} + 0.833\,\frac{x}{b}\right) c_a b \qquad (71)$$

$$p_u = \left(3 + 0.5\,\frac{x}{b}\right) cb \qquad (72)$$

(2) For $x > 12b$,

$$p_u = 9cb \qquad (73)$$

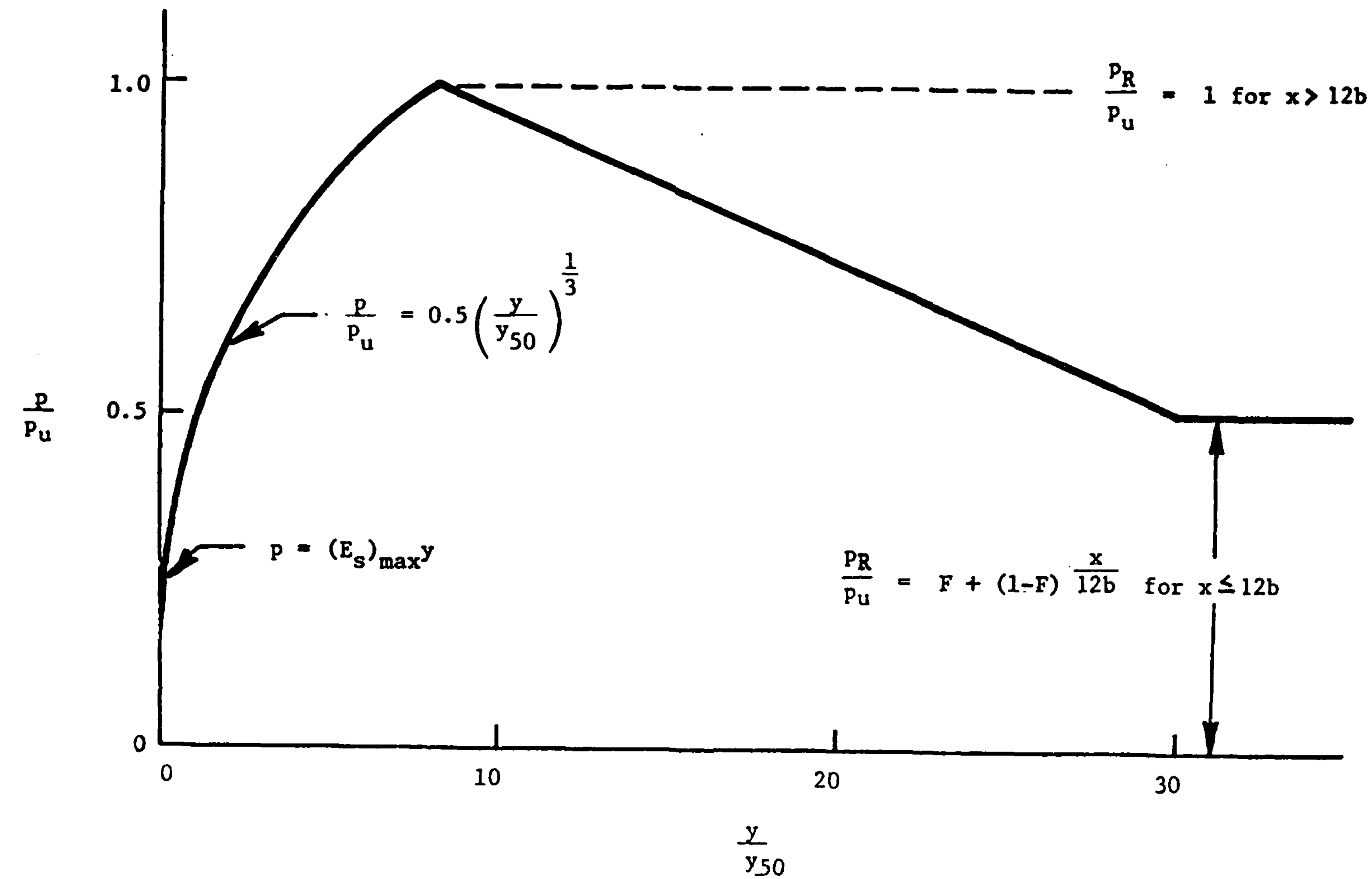


Figure 33. Characteristic shape of p-y curve for unified clay criteria, static loading (Reese and Sullivan 1980)

The steps below are for a particular depth x .

d. Select the coefficients A and F as indicated below. The coefficients A and F , determined empirically for the load tests at the Sabine River and Manor sites, are given in Table 5. The terms used in Table 5, not defined previously, are defined below:

W_L = liquid limit

PI = plasticity index

LI = liquidity index

O_R = overconsolidation ratio

S_t = sensitivity

The recommended procedure for estimating A and F for other clays is:

(1) Determine as many of the following properties of the clay as possible: c , ε_{50} , O_R , S_t , degree of fissuring, ratio of residual to peak undrained shear strength W_L , PI , and LI .

(2) Compare the properties of the soil in question to the properties of the Sabine and Manor clays listed in Table 5.

(3) If the properties are similar to those of either the Sabine or the Manor clay, use A and F for the similar clay.

(4) If the properties are not similar to either, the user should estimate A and F using his judgment and Table 5 as guides.

e. Compute

$$y_{50} = A\varepsilon_{50}b \tag{74}$$

f. Obtain $\left(E_s\right)_{max}$. When no other method is available, Equation 75 and Table 6 may be used as guidelines:

$$\left(E_s\right)_{max} = kx \tag{75}$$

Table 5

Curve Parameters for the Unified Criteria
(Reese and Sullivan 1980)

Clay Description	A	F
Sabine River site	2.5	1.0
Inorganic, intact		
$c = 300\ lb/ft^2$		
$\varepsilon_{50} = 0.7\%$		
$O_R = 1$		
$S_t \approx 2$		
$w_L = 92$		
$PI = 68$		
$LI = 1$		
Manor, Tex., site	0.35	0.5
Inorganic, very fissured		
$c \approx 2400\ lb/ft^2$		
$\varepsilon_{50} \approx 0.5\%$		
$O_R > 10$		
$S_t \approx 1$		
$w_L \approx 77$		
$PI \approx 60$		
$LI \approx 0.2$		

Table 6

Representative Values for k

Shear Strength c , psf	k , pci
250-500	30
500-1000	100
1000-2000	300
2000-4000	1000
4000-8000	3000

(Also see Table 4.)

g. Compute the deflection at the intersection between the initial linear portion and curved portion from the equation

$$y_k = \left[\frac{0.5p_u}{\left(E_s\right)_{max}}\right]^{3/2} \left(y_{50}\right)^{-1/2} \tag{76}$$

(y_k can be no larger than $8y_{50}$.)

h. (1) For $0 < y < y_k$

$$p = \left(E_s\right)_{max} y \tag{77}$$

(2) For $y_k < y < 8y_{50}$

$$p = 0.5p_u \left(\frac{y}{y_{50}}\right)^{1/3} \tag{78}$$

(3) For $8y_{50} < y < 30y_{50}$

$$p = p_u + \frac{p_R - p_u}{22y_{50}} \left(y - 8y_{50}\right) \tag{79}$$

where

$$p_R = p_u\left[F + (1 - F)\frac{x}{12b}\right] \tag{80}$$

(p_R will be equal to or less than p_u)

(4) For $y > 30y_{50}$

$$p = p_R \tag{81}$$

111. The following procedure is for cyclic loading and is illustrated in Figure 34:

a. Repeat steps a through h(1) for static loading.

b. Compute

$$p_{CR} = 0.5p_u \frac{x}{12b} \leq 0.5p_u \tag{82}$$

c. (1) For $y_g < y < y_{50}$

$$p = 0.5p_u \left(\frac{y}{y_{50}}\right)^{1/3} \tag{83}$$

(2) For $y_{50} < y < 20y_{50}$

$$p = 0.5p_u + \frac{p_{CR} - 0.5p_u}{19y_{50}} (y - y_{50}) \tag{84}$$

(3) For $y > 20y_{50}$,

$$p = p_{CR} \tag{85}$$

112. Comments. The procedures outlined above for both static and cyclic loading assume that an intersection of the curve defined by Equations 77 and 78 occurs. If that intersection does not occur, the p-y curve is defined by Equation 77 until it intersects a portion of the curve defined by Equations 79 and 81 for static loading and Equations 83 or 84 for cyclic loading.

113. Example curves. Example sets of p-y curves were computed using the unified criteria and the soil profiles in Figures 20 and 26. The soil profile in Figure 20 represents a soft clay, and the profile in Figure 26 represents a stiff clay, both below the water table. The p-y curves for both soil profiles were computed for static and cyclic loadings using a pile 48 in. in diameter and the following depths: 0, 1, 2, 4, 8, 12, 20, 40, and 60 ft.

114. For the soft clay profile in Figure 20, the value of ε_{50} was assumed to be 0.02 from the mudline to a depth of 20 ft and to decrease to 0.01

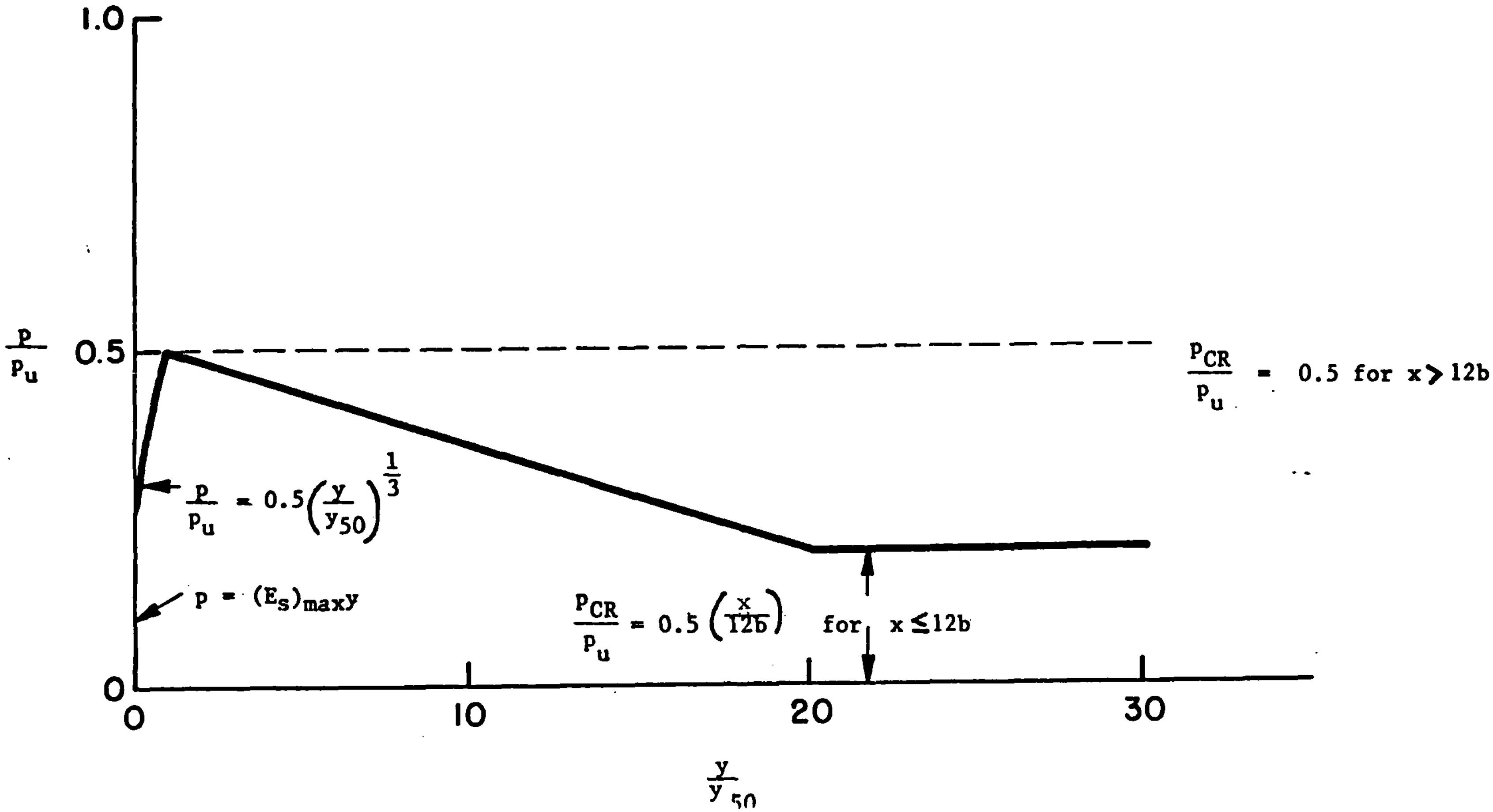


Figure 34. Characteristic shape of p-y curve for unified clay criteria, cyclic loading (Reese and Sullivan 1980)

at a depth of 90 ft. The value of A was assumed to be 2.5, and the value of F was assumed to be 1.0. The value of k for computing the maximum value of the soil modulus was assumed to be 200,000 pcf. Figure 35 shows the set of

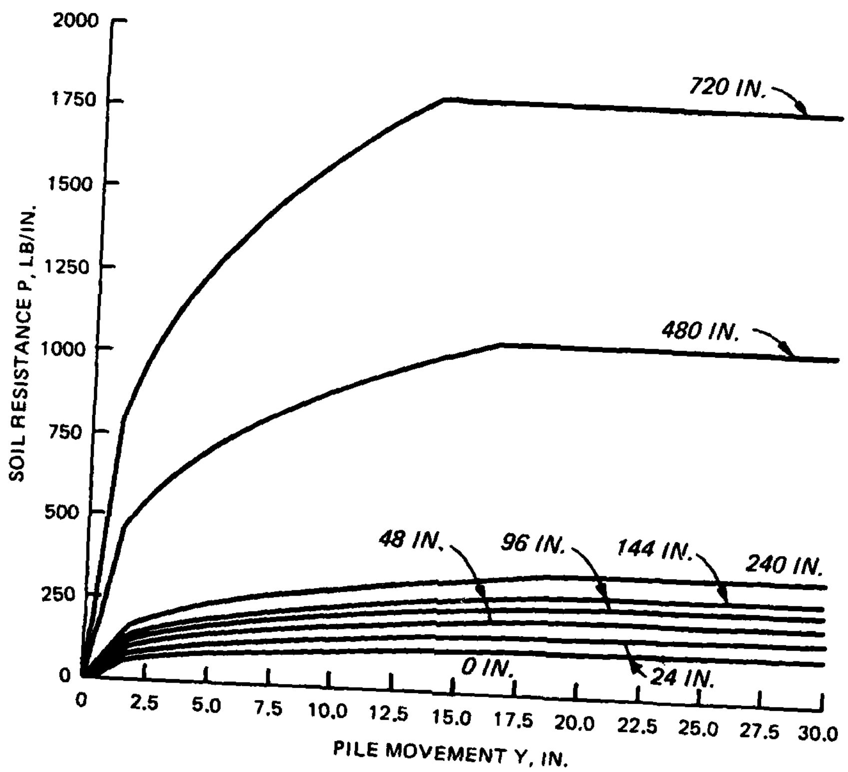


Figure 35. Example p-y curves for soft clay below the water table; unified criteria, static loading

p-y curves for static loading, and Figure 36 shows curves for cyclic loading.

115. For the stiff clay profile in Figure 26, the value of ε_{50} was assumed to be 0.005 and γ was taken as 50 pcf for the entire depth. The value of A was assumed to be 0.35, the value of F to be 800,000 pcf. Figure 37 shows the set of p-y curves for static loading, and Figure 38 shows curves for cyclic loading.

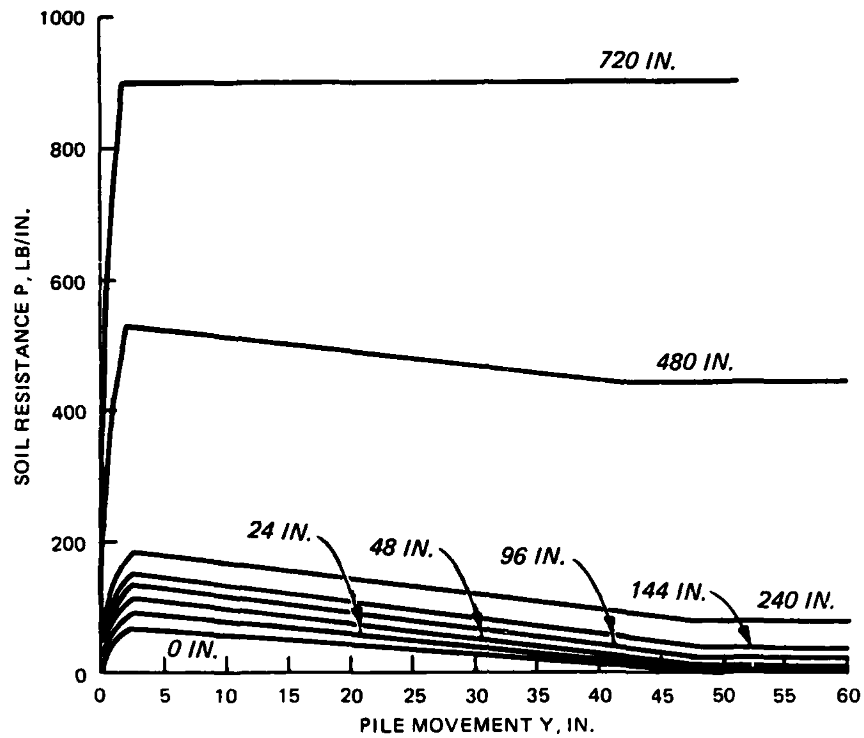


Figure 36. Example p-y curves for soft clay below the water table; unified criteria, cyclic loading

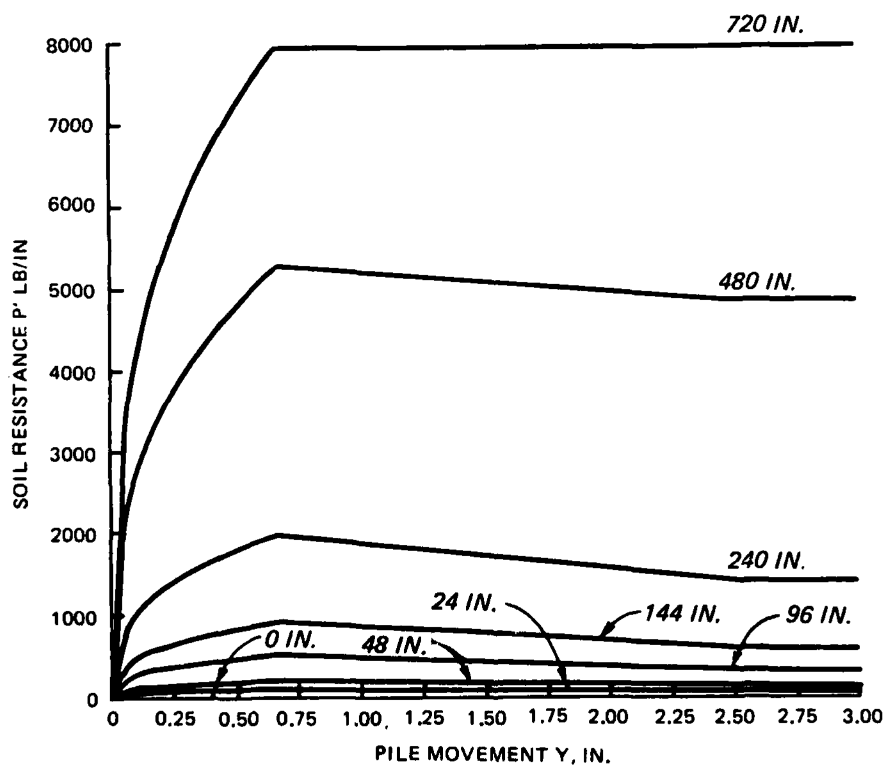


Figure 37. Example p-y curves for stiff clay below the water table; unified criteria, static loading

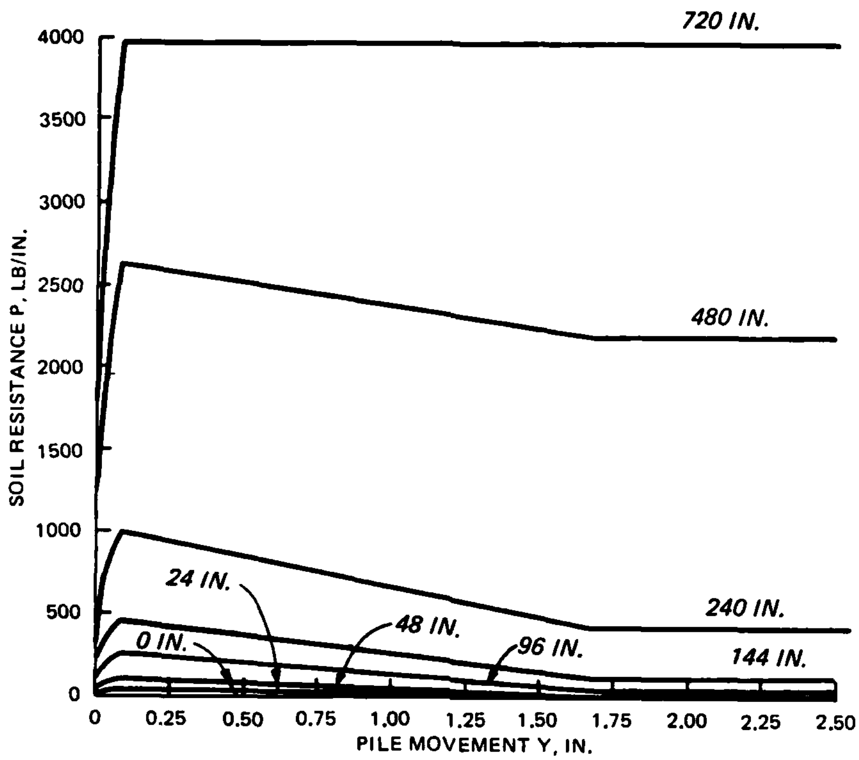


Figure 38. Example p-y curves for stiff clay below the water table; unified criteria, cyclic loading

Recommendations for p-y Curves for Sand

116. As shown below, a major experimental program was conducted on the behavior of laterally loaded piles in sand below the water table. The results can be extended to sand above the water table.

Response of sand below the water table

117. Field experiments. An extensive series of tests was performed at a site on Mustang Island, near Corpus Christi, Tex. (Cox, Reese, and Grubbs 1974). Two steel pipe piles, 24 in. in diameter, were driven into sand in a manner simulating the driving of an open-ended pipe. The piles were then subjected to lateral loading. The embedded length of the piles was 69 ft. One of the piles was subjected to short-term loading and the other to repeated loading.

118. The soil at the site was a uniformly graded fine sand with an angle of internal friction of 39 deg. The submerged unit weight was 66 pcf. The water surface was maintained a few inches above the mud line throughout the test program.

119. Recommendations for computing p-y curves. The following

procedure is for both short-term static loading and cyclic loading and is illustrated in Figure 39 (Reese, Cox, and Koop 1974).

<u>a</u>. Obtain values for the angle of internal friction ϕ , the soil unit weight γ , and pile diameter b .

<u>b</u>. Make the following preliminary computations.

$$\alpha = \frac{\phi}{2} ; \quad \beta = 45 + \frac{\phi}{2} ; \quad K_o = 0.4 ; \quad K_a = \tan^2 \left(45 - \frac{\phi}{2}\right) \tag{86}$$

<u>c</u>. Compute the ultimate soil resistance per unit length of pile using the smaller of the values given by the equations below.

$$p_{st} = \gamma x \left[\frac{K_o x \tan \phi \sin \beta}{\tan (\beta - \phi) \cos \alpha} + \frac{\tan \beta}{\tan (\beta - \phi)} \times (b + x \tan \beta \tan \alpha) + K_o x \tan \beta \times (\tan \phi \sin \beta - \tan \alpha) - K_a b \right] \tag{87}$$

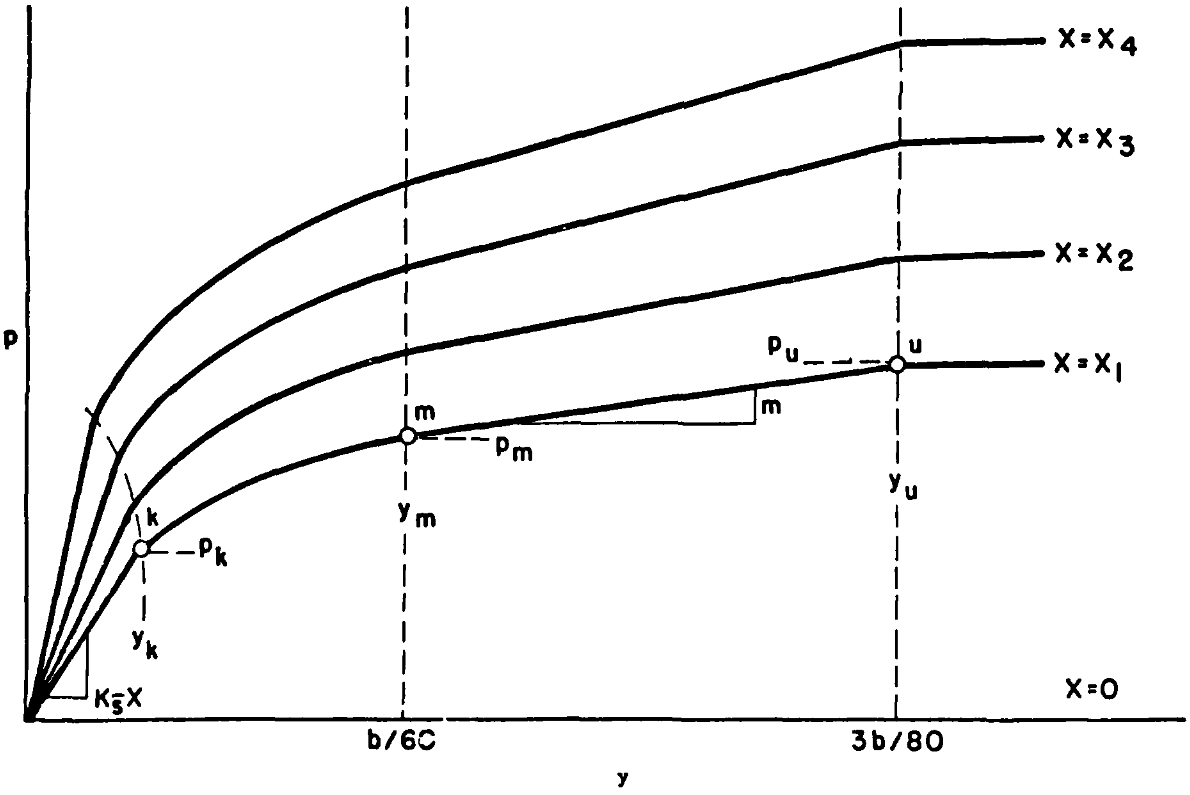


Figure 39. Characteristic shape of a family of p-y curves for static and cyclic loading in sand (Reese, Cox, and Koop 1974)

$$P_{sd} = K_a b\gamma x\ (\tan^8 \beta - 1) + K_o b\gamma x \tan \phi \tan^4 \beta \qquad (88)$$

d. In making the computations in step c, find the depth x_t at which there is an intersection between Equations 87 and 88. Above this depth, use Equation 87. Below this depth, use Equation 88.

e. Select a depth at which a p-y curve is desired.

f. Establish y_u as 3b/80 . Compute p_u from

$$p_u = \bar{A}_s p_s \quad \text{or} \quad p_u = \bar{A}_c p_s \qquad (89)$$

Use the appropriate value of $\bar{A}_s$ or $\bar{A}_c$ from Figure 40 for the particular nondimensional depth, and for either the static or cyclic case. Use the appropriate equation for p_s from Equation 87 or Equation 88 by referring to the computation in step d.

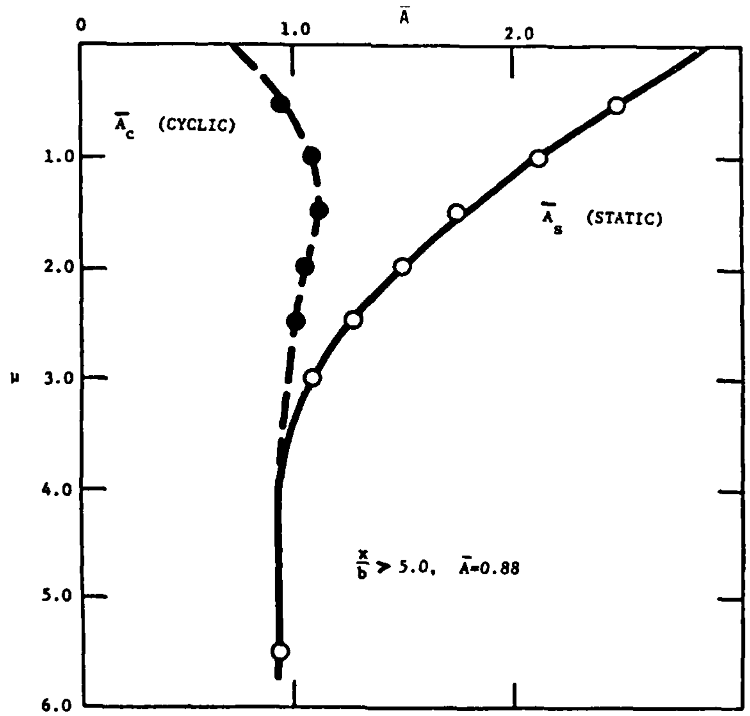


Figure 40. Values of the coefficients $\bar{A}_c$ and $\bar{A}_s$ (Reese and Sullivan 1980)

g. Establish y_m as b/60 . Compute p_m from

$$p_m = B_s p_s \quad \text{or} \quad p_m = B_c p_s \tag{90}$$

Use the appropriate value of B_s or B_c from Figure 41 for the particular nondimensional depth, and for either the static or the cyclic case. Use the appropriate equation for p_s . The two straight-line portions of the p-y curve, beyond the point where y is equal to b/60 , can now be established.

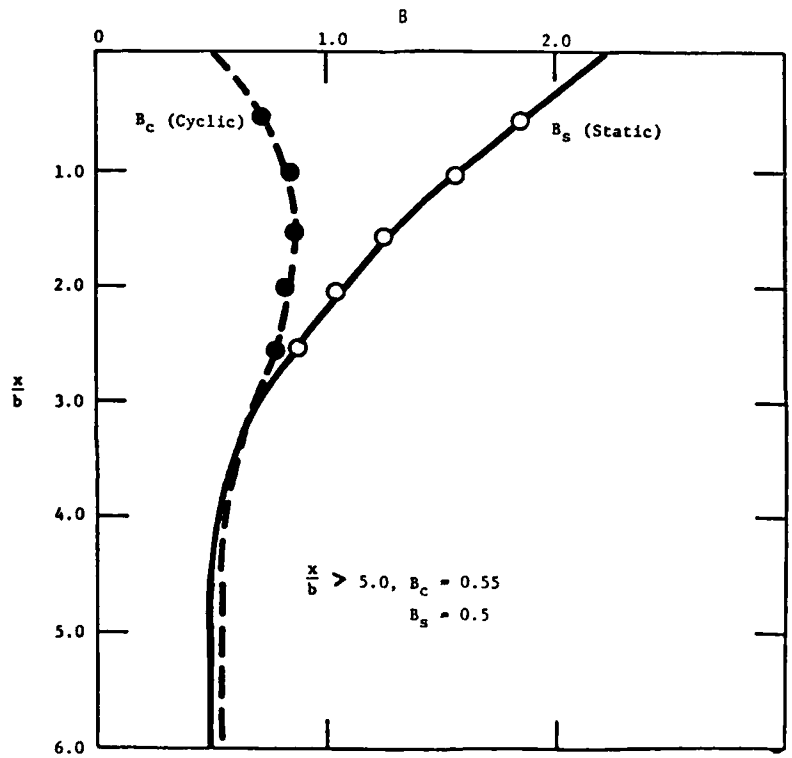


Figure 41. Nondimensional coefficient b for soil resistance versus depth (Reese and Sullivan 1980)

h. Establish the initial straight-line portion of the p-y curve,

$$p = (kx)y \tag{91}$$

Use the appropriate value of k from Table 7 or 8.

i. Establish the parabolic section of the p-y curve,

$$p = \bar{C}y^{1/n} \tag{92}$$

Table 7

Representative Values of k for Submerged Sand

	Relative Density		
	Loose	Medium	Dense
Recommended k , pci	20	60	125

Table 8

Representative Values of k for Sand Above the Water Table

	Relative Density		
	Loose	Medium	Dense
Recommended k , pci	25	90	225

Fit the parabola between points k and m as follows:

(1) Determine the slope of the line between points m and u from

$$m = \frac{p_u - p_m}{y_u - y_m} \tag{93}$$

(2) Obtain the power of the parabolic section from

$$n = \frac{p_m}{my_m} \tag{94}$$

(3) Obtain the coefficient $\bar{C}$ from

$$\bar{C} = \frac{p_m}{y_m^{1/n}} \tag{95}$$

(4) Determine point k from

$$y_k = \left(\frac{\bar{C}}{kx}\right)^{n/n-1} \tag{96}$$

(5) Compute the appropriate number of points on the parabola by using Equation 92.

Note: The step-by-step procedure is outlined, and Figure 39 is drawn, as if there is an intersection between the initial straight-line portion of the p-y curve and the parabolic portion of the curve at point k . However, in some instances, there may be no intersection with the parabola. Equation 91 defines the p-y curve until there is an intersection with another branch of the p-y curve, or, if no intersection occurs, Equation 91 defines the complete p-y curve. This completes the development of the p-y curve for the desired depth. Any number of curves can be developed by repeating the above steps for each desired depth.

120. Recommended soil tests. Triaxial compression tests are recommended for obtaining the angle of internal friction of the sand. Confining pressures should be used which are close or equal to those at the depths being considered in the analysis. If samples cannot be obtained, correlations between d and results from penetration tests can be used. Tests must be performed to determine the unit weight of the sand.

121. Example curves. An example set of p-y curves was computed for sand below the water table for a pile with a diameter of 48 in. The soil profile used is presented in Figure 42. The submerged unit weight was assumed to be 57.5 pcf, and k was taken to be 80 pci. The loading was assumed to be both static and cyclic.

122. p-y curves were computed for the following depths below the mud line: 0, 1, 2, 4, 8, 12, 20, 40, and 60 ft. The plotted curves are shown in Figure 43 for static loading and in Figure 44 for cyclic loading.

Response of sand above the water table

123. The procedure described in the previous section can be used for sand above the water table if appropriate adjustments are made to the unit weight and angle of internal friction of the sand. Some small-scale experiments were performed by Parker and Reese (1971), and recommendations for p-y curves for dry sand were developed from those experiments. The results of the Parker and Reese experiments should be useful in checking solutions which were obtained using results from the test program for full-scale piles.

Summary

124. This part of the report has described procedures which can be used in developing soil response curves for laterally loaded piles in soft clay,

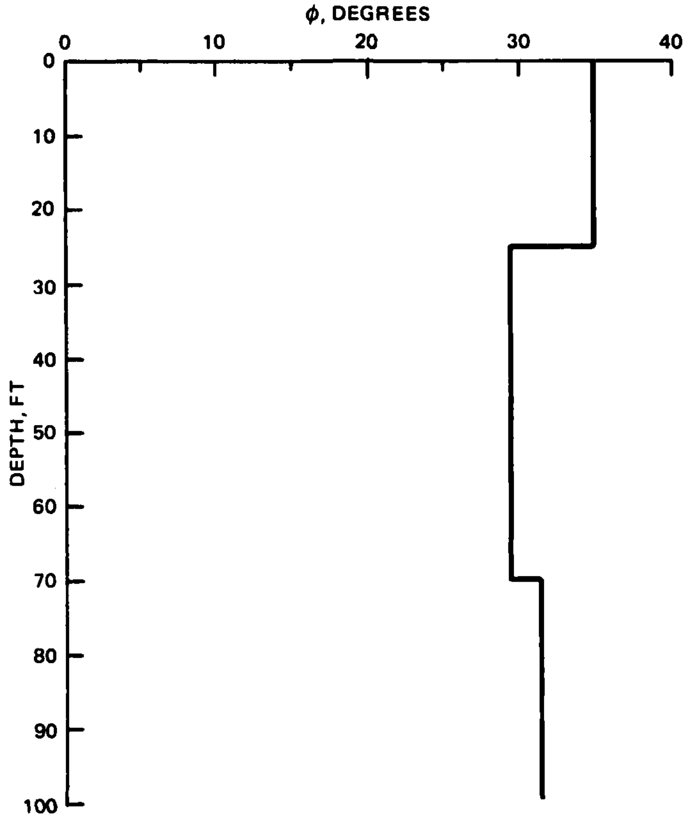


Figure 42. Soil profile used for example p-y curves for sand below the water table; Reese criteria

stiff clay, or sands. Most of the material covered in this part of the report was extracted from reports of work done and documented at UT by Prof. Reese and his associates. The examples are selected from Corps of Engineers' files.

125. It must be emphasized that development of proper soil-response curves requires experience and a feel for the problem. At best, the procedures described in this part should only be used as guidelines. In every case, a user is responsible for developing these curves, and it is assumed that he will apply judgment in using the guidance provided here.

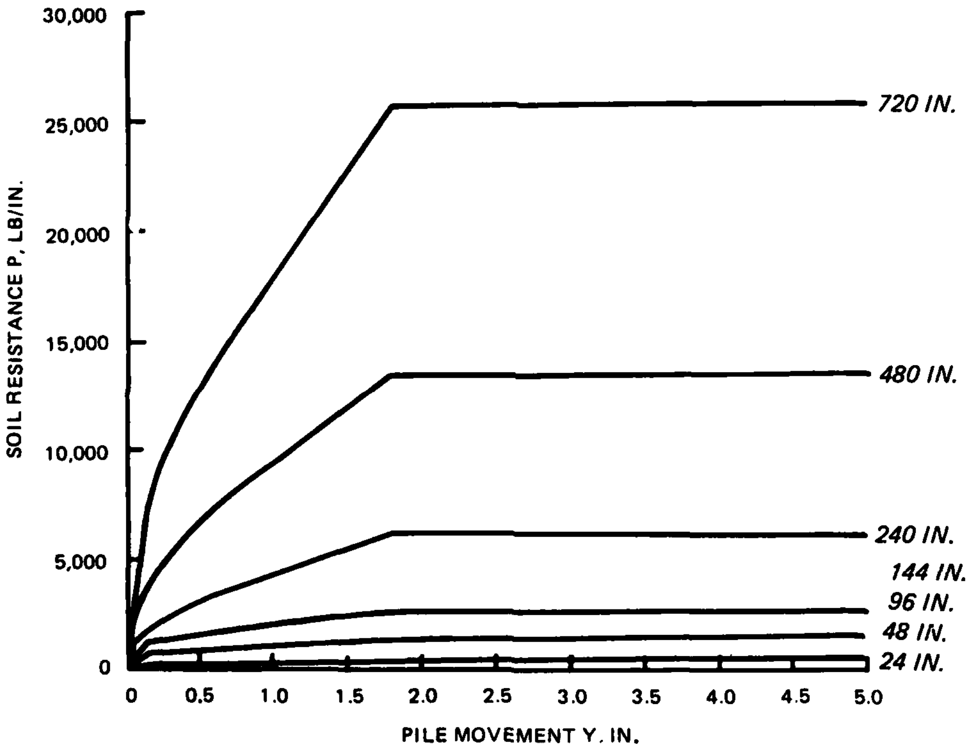


Figure 43. Example p-y curves for sand below the water table, static loading

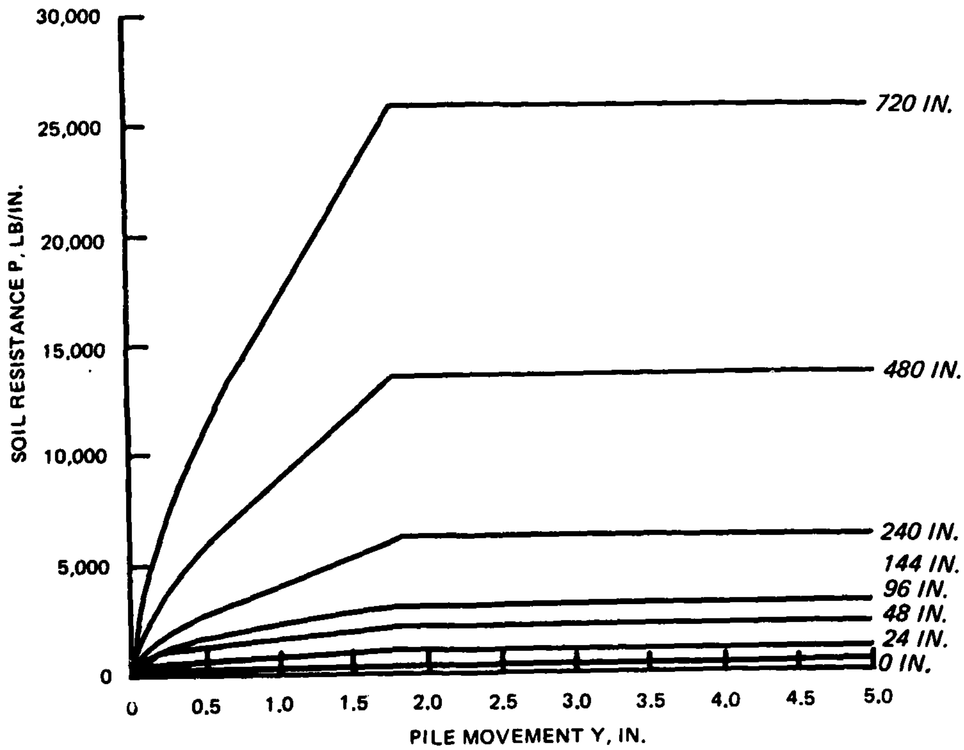


Figure 44. Example p-y curves for sand below the water table, cyclic loading

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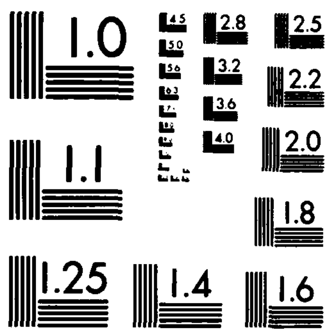

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APPENDIX A: NONDIMENSIONAL SOLUTIONS FOR ANALYSIS OF LATERALLY LOADED PILES

Introduction

1. The principle of dimensional analysis is usually applied to physical models; however, Reese and Matlock (1956)* applied the principle to mathematical models as well. They used the principle of dimensional analysis to produce a set of nondimensional coefficients which can be used to solve the governing differential equation for laterally loaded piles.

2. The development of the nondimensional solution method was a result of extensive experience gained at The University of Texas at Austin through manual use of the difference equation method. Parts of the method were done a few times for each boundary condition, using a range of values for the variables. It was found that these solutions could then be applied to many similar problems. The theoretical legitimacy of this method of approach was confirmed by applying the principles of engineering similitude to derive the method.

3. At the time of the development of nondimensional methods of analysis, computers were available to few engineers outside of research. The nondimensional methods were developed because they included many of the advantages of the finite difference solutions, yet could be performed relatively easily by using a hand calculator. Their primary advantage was that the nonlinear soil response could be taken into account through successive iterations of the solution. The main disadvantage was that a predetermined variation of soil modulus with depth must be assumed. Today, the nondimensional methods are important because they: (a) provide a hand solution method to verify computer results by the finite difference technique, (b) pr(˙de a better understanding of the mechanics of the response of a pile under lateral loading, and (c) can be used on occasion to obtain results for use in design if a computer is not available.

4. Readers are referred to Reese and Sullivan (1980), Reese and Allen (1977), Reese and Matlock (1956) and Matlock and Reese (1960) for the concept and theory of nondimensional solutions and the details of the solution procedure for analyses of laterally loaded piles. This appendix presents a

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* References cited in this appendix are included in the References at the end of the main text.

step-by-step procedure and an example solution, including the manual generation of a p-y curve using soft clay criteria.

Solution Procedure (Extracted from Reese and Sullivan 1980)

5. The solution procedure is described below for three sets of boundary conditions at the top of the pile: (a) pile head free to rotate, (b) pile head fixed against rotation, and (c) pile head restrained against rotation. These boundary conditions are shown in Figure A1 along with the sign convention used in the solutions.

6. Limitations imposed by the nondimensional solutions are as follows:

   a. The effect on bending moment of the axial load cannot be investigated.

   b. A constant value of flexural rigidity of the pile must be used.

   c. The nondimensional curves included herein are valid only for the case of a linearly varying soil modulus with zero at the groundline.

Case I: Pile head free to rotate

7. The solution procedure for Case I is as follows:

   a. Construct p-y curves at various depths by procedures recommended in the main text, with the spacing between p-y curves being closer near the ground surface than near the bottom of the pile.

   b. Assume a value of T, the relative stiffness factor, from

$$T = \sqrt[5]{\frac{EI}{k}} \tag{A1}$$

   where

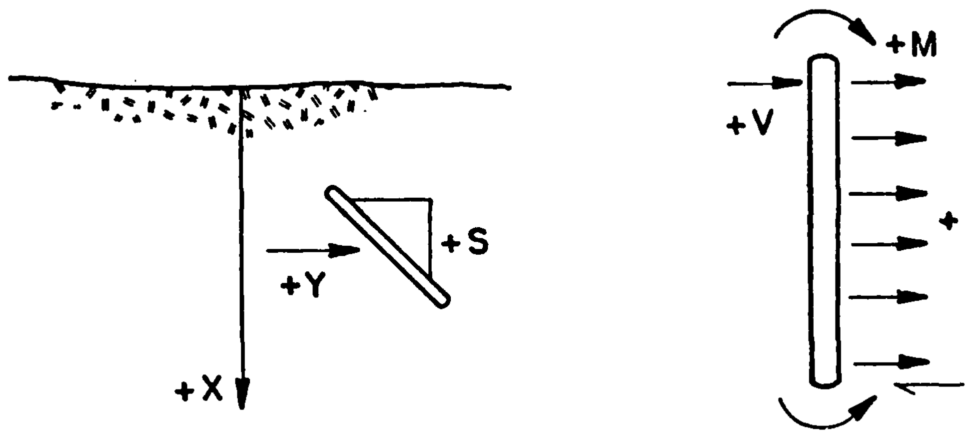
   EI = flexural rigidity of pile

   k = constant relating the secant modulus of soil reaction to depth ($E_s = kx$)

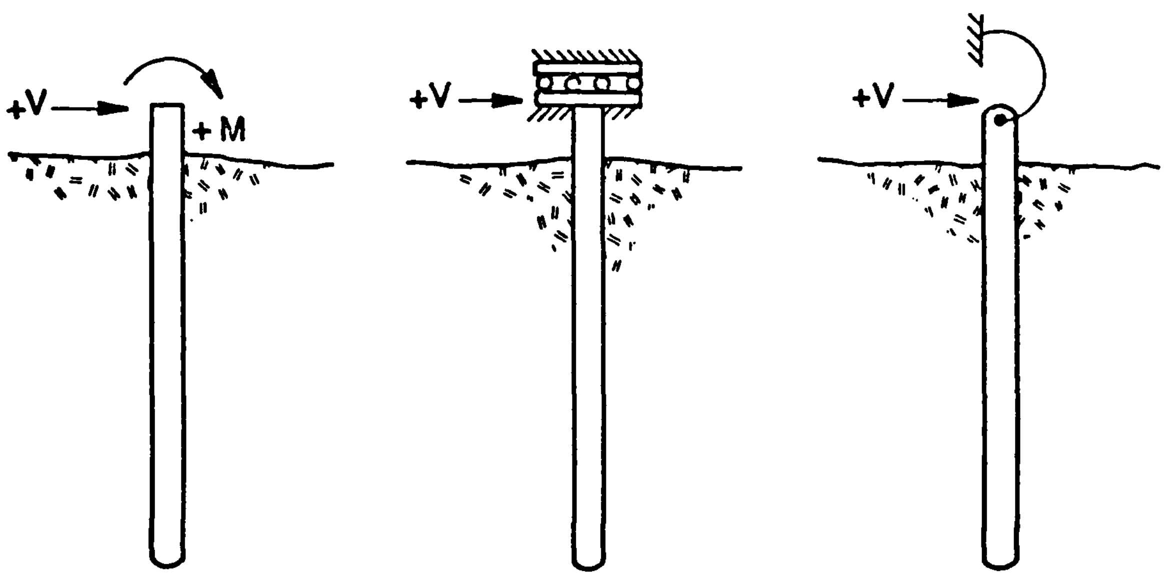
   c. Compute the depth coefficient $z_{max} = L/T$. (A2)

   d. Compute the deflection y at each depth x along the pile where a p-y curve is available from

$$y = A_y \frac{P_T T^3}{EI} + B_y \frac{M_T T^2}{EI} \tag{A3}$$



a. Sign convention



Case I. Pile Head Free to Rotate

Case II. Pile Head Fixed Against Rotation

Case III. Pile Head Restricted Against Rotation

b. Boundary conditions

Figure A1. Sign convention and boundary conditions considered in the solution procedure (Reese and Sullivan 1980) where

A_y = deflection coefficient (from Figure A2)

P_T = shear at top of pile

T = relative stiffness factor

B_y = deflection coefficient (from Figure A3)

M_T = moment at top of pile

The particular curves to be employed in determining the A_y and B_y coefficients depend on the value of z_{max} computed in step c.

e. From a p-y curve, select the value of soil resistance p that corresponds to the pile deflection value y at the depth of the p-y curve. Repeat this procedure for every p-y curve that is available.

f. Compute a secant modulus of soil reaction E_s using the equation

$$E_s = \frac{p}{y}$$

Plot the E_s values versus depth.

g. From the E_s-versus-depth plot in step f, compute the constant k which relates E_s to depth ($k = E_s/x$). Give more weight to the E_s values near the ground surface.

h. Compute a value of the relative stiffness factor T from the value of p found in step g. Repeat steps b through g using the new value of T each time, until the assumed value of T equals the calculated value of T.

i. When the iterative procedure has been completed, the values of deflection along the pile are known from step d of the final iteration. Values of soil reactions may be computed from the basic expression

$$p = E_s y$$

Values of slope, moment, and shear along the pile can be determined from

$$S = A_s \frac{P_t T^2}{EI} + B_s \frac{M_t T}{EI} \tag{A4}$$

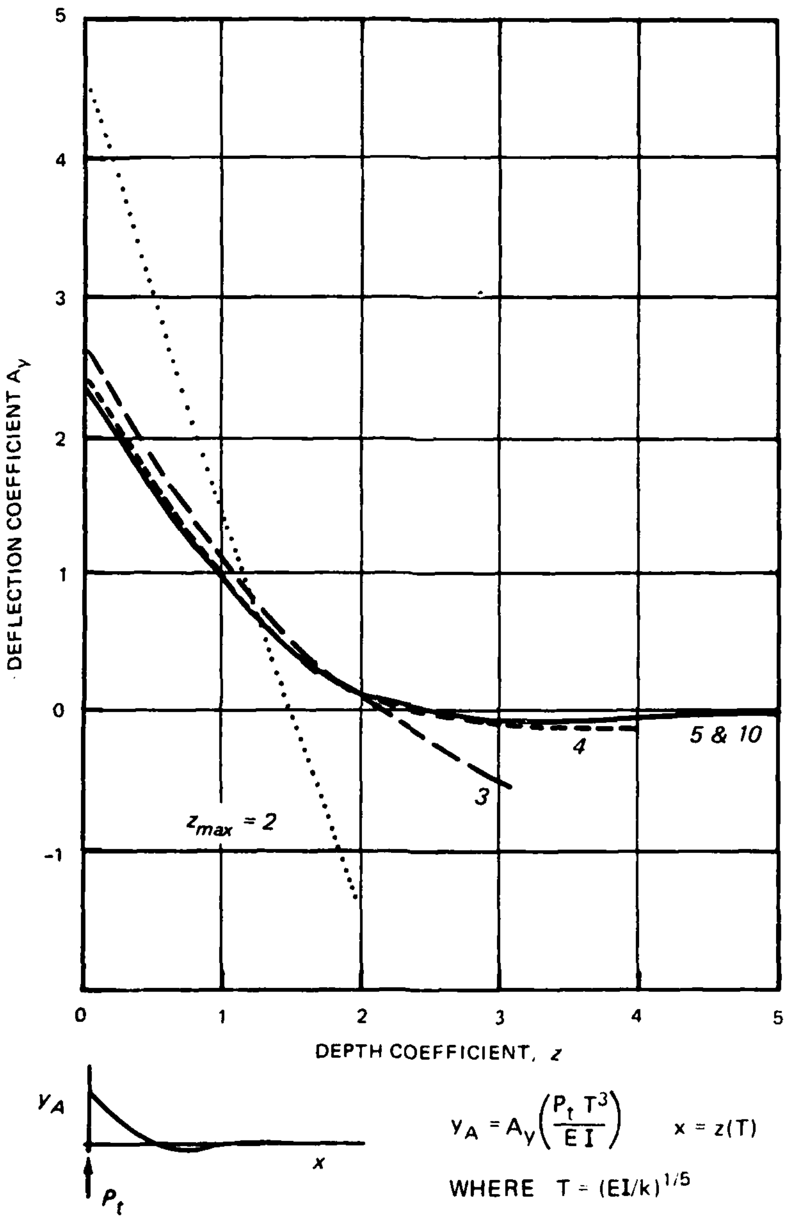
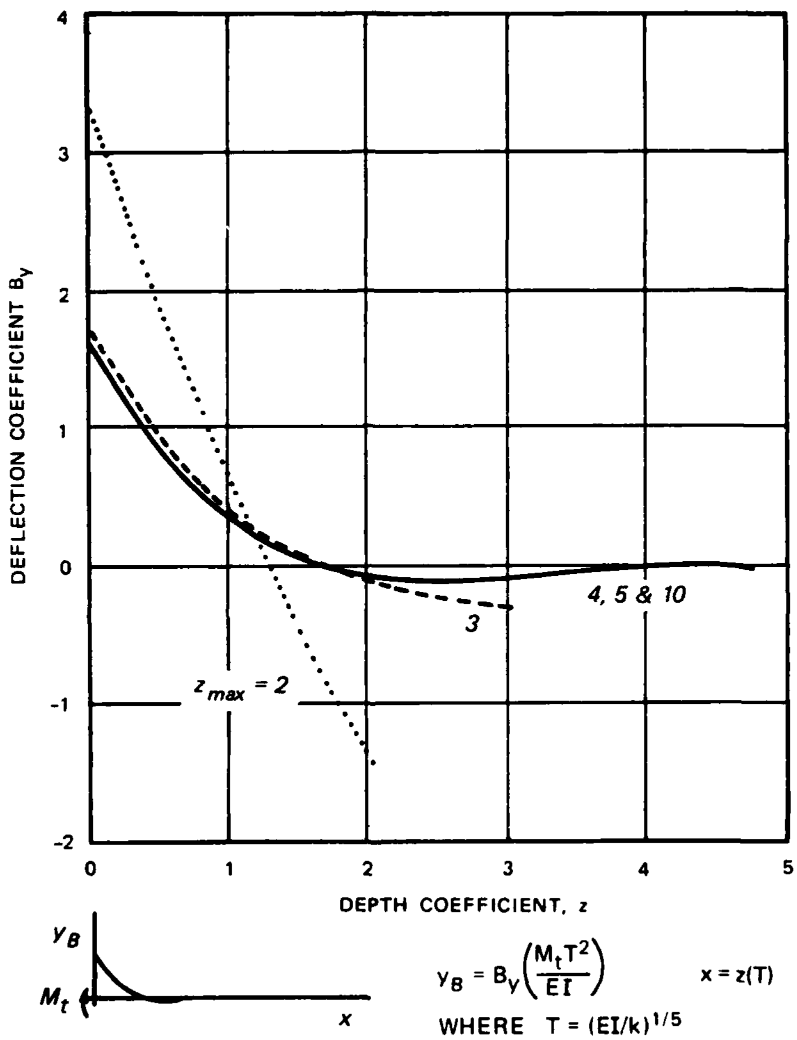


Figure A2. Pile deflection produced by lateral load at mud line (Reese and Sullivan 1980)



$$y_B = B_y\left(\frac{M_t T^2}{EI}\right) \qquad x = z(T)$$

WHERE $T = (EI/k)^{1/5}$

Figure A3. Pile deflection produced by moment applied at mud line (Reese and Sullivan 1980)

$$M = A_m P_t T + B_M M_t \qquad (A5)$$

and

$$V = A_v P_t + B_v \frac{M_t}{T} \qquad (A6)$$

The appropriate coefficients to be used in the above equations may be obtained from Figures A4 through A9.

Case II: Pile head
fixed against rotation

8. Case II may be used to obtain a solution for the case where the superstructure translates under load but does not rotate and where the superstructure is very stiff in relation to the pile.

a. Perform steps a, b, and c of the solution procedure for free-head piles (Case I).

b. Compute the deflection y at each depth along the pile where a p-y curve is available from

$$y_F = F_y \frac{P_t T^3}{EI} \qquad (A7)$$

The deflection coefficients F_y may be found by entering Figure A10 with the appropriate value of z_{max}.

c. The solution proceeds in a manner similar to steps e through h for the free-head case (Case I).

d. Compute the moment at the top of the pile M_T from

$$M_t = F_{MT} P_t T \qquad (A8)$$

The value of F_{MT} may be found by entering Table A1 with the appropriate value of z_{max}.

e. Compute values of slope, moment, shear, and soil reaction along the pile by following the procedure in step i for the free-head pile.

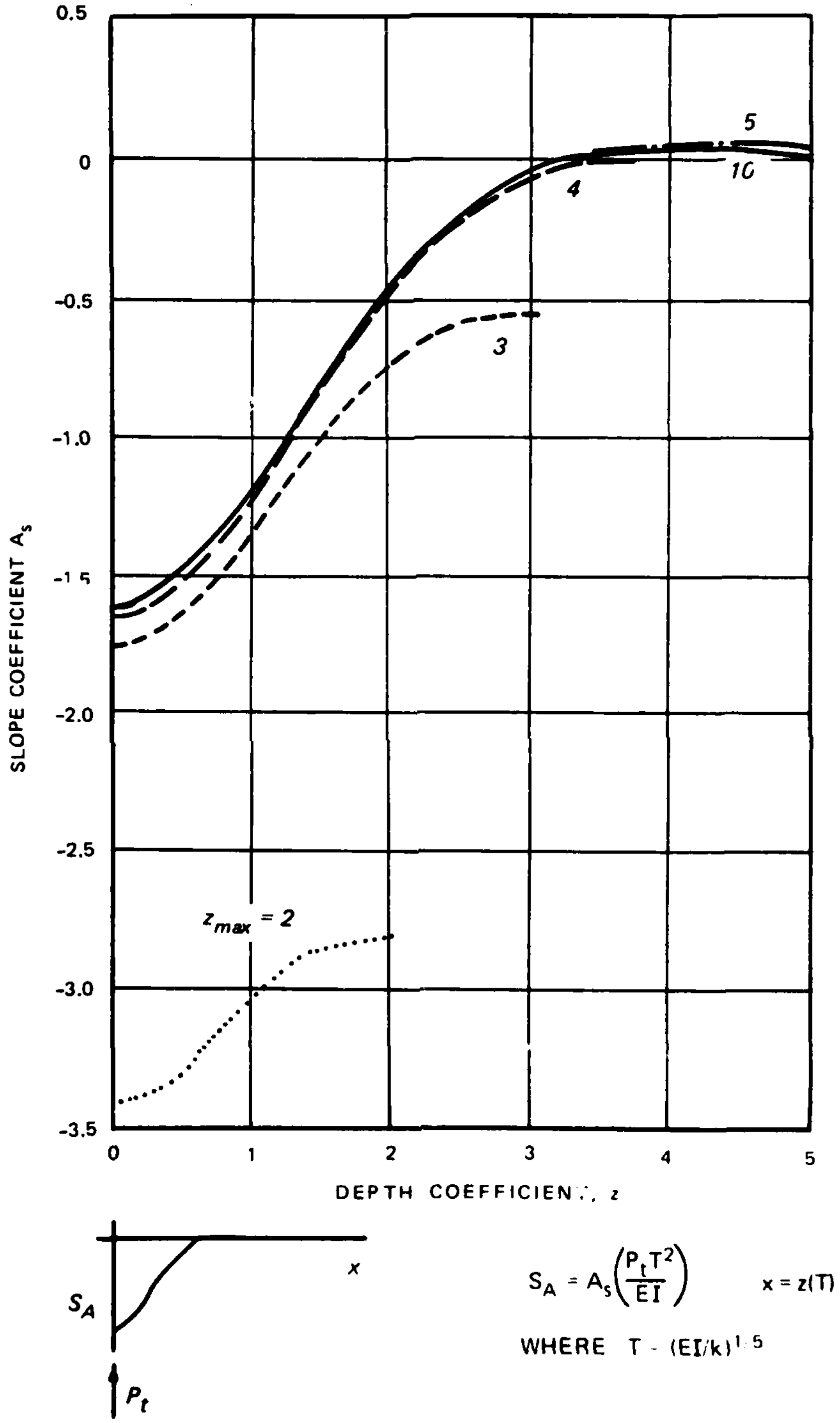
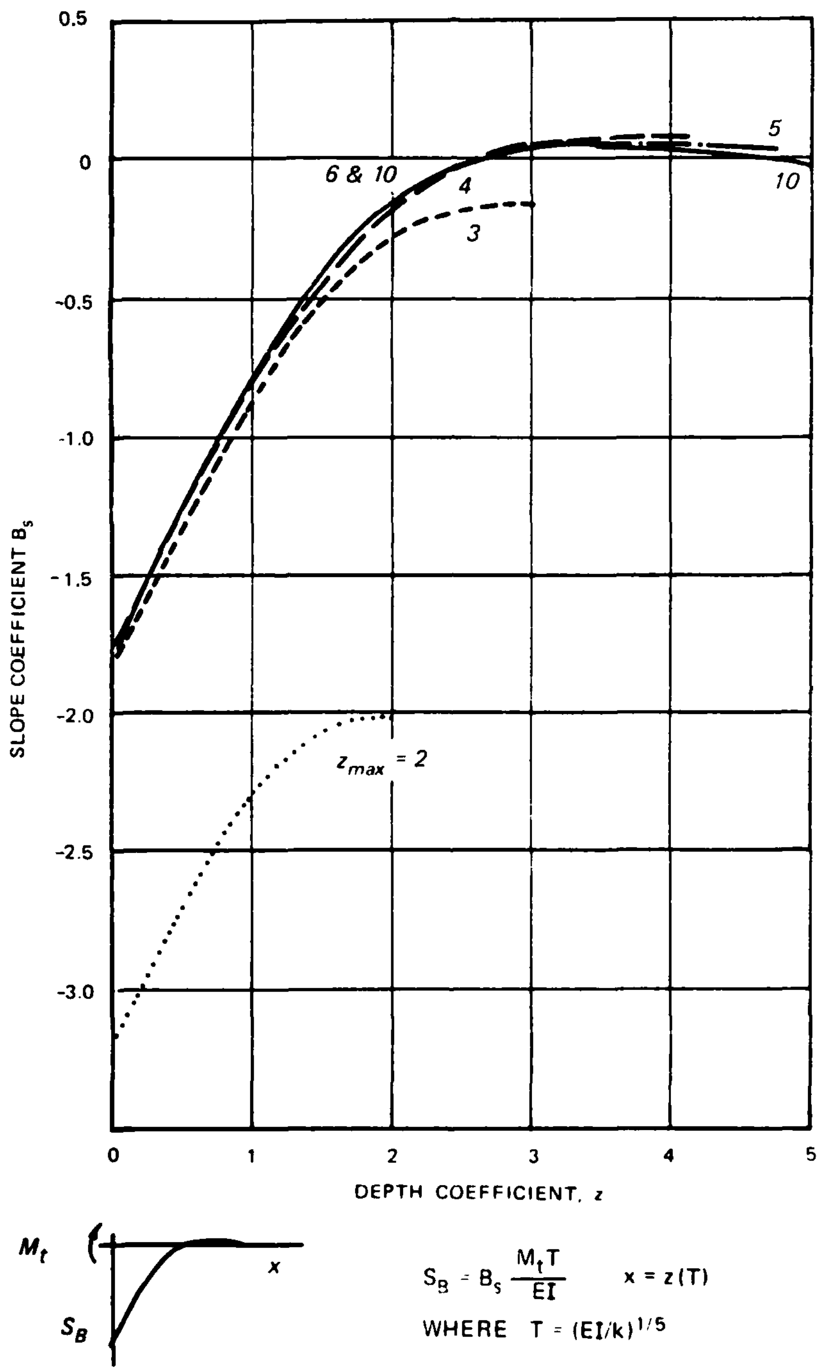


Figure A4. Slope of pile caused by lateral load at mud line (Reese and Sullivan 1980)



$$S_B = B_s \frac{M_t T}{EI} \qquad x = z(T)$$

WHERE $T = (EI/k)^{1/5}$

Figure A5. Slope of pile caused by moment applied at mud line (Reese and Sullivan 1980)

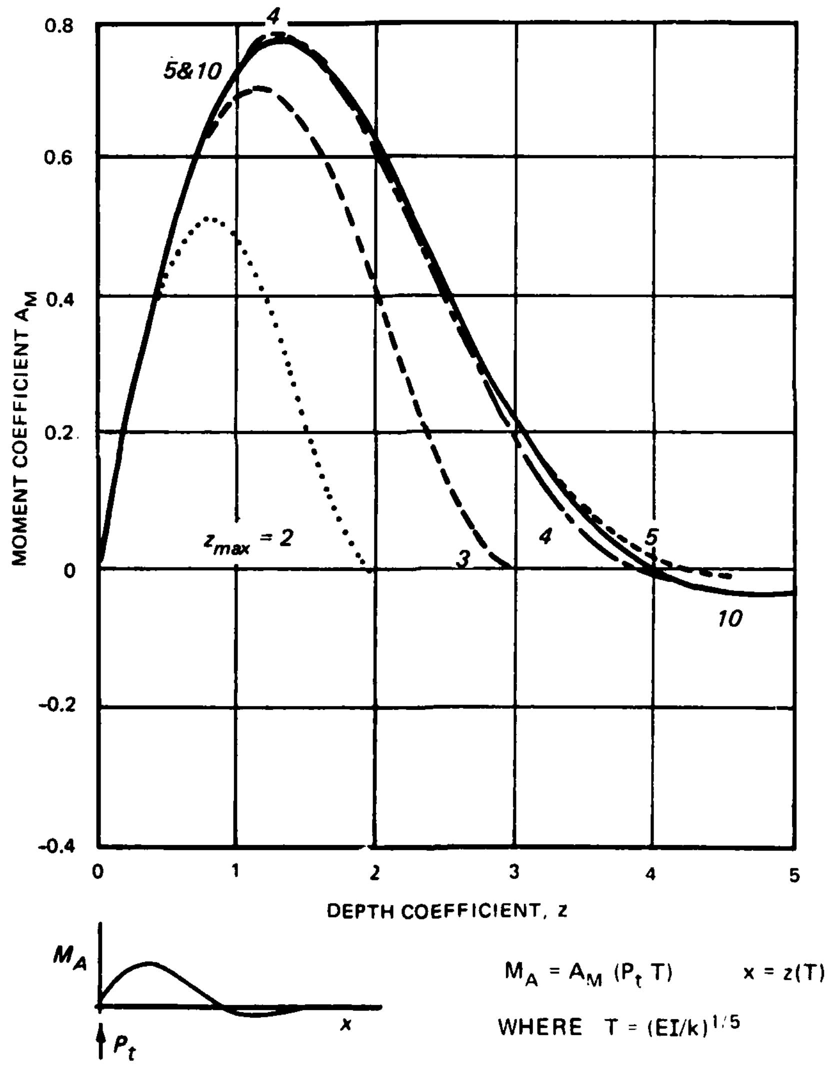


Figure A6. Bending moment produced by lateral load at mud line (Reese and Sullivan 1980)

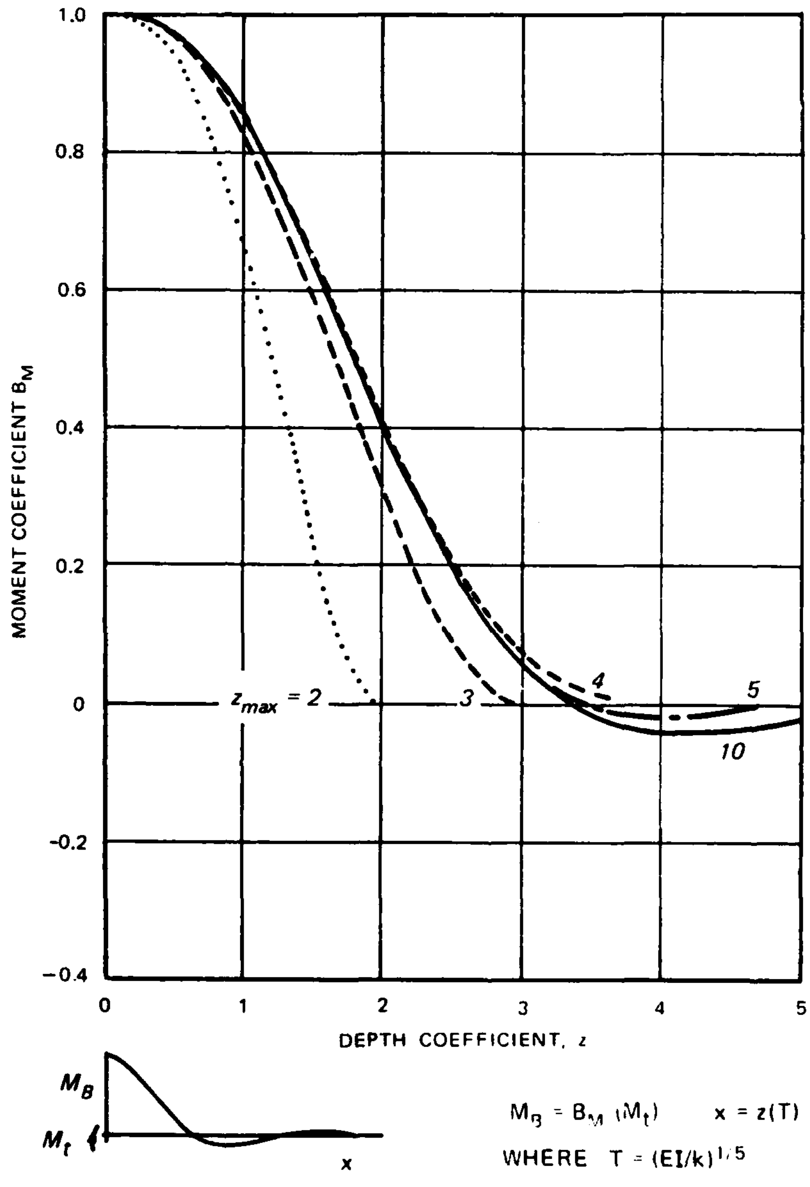


Figure A7. Bending moment produced by moment applied at mud line (Reese and Sullivan 1980)

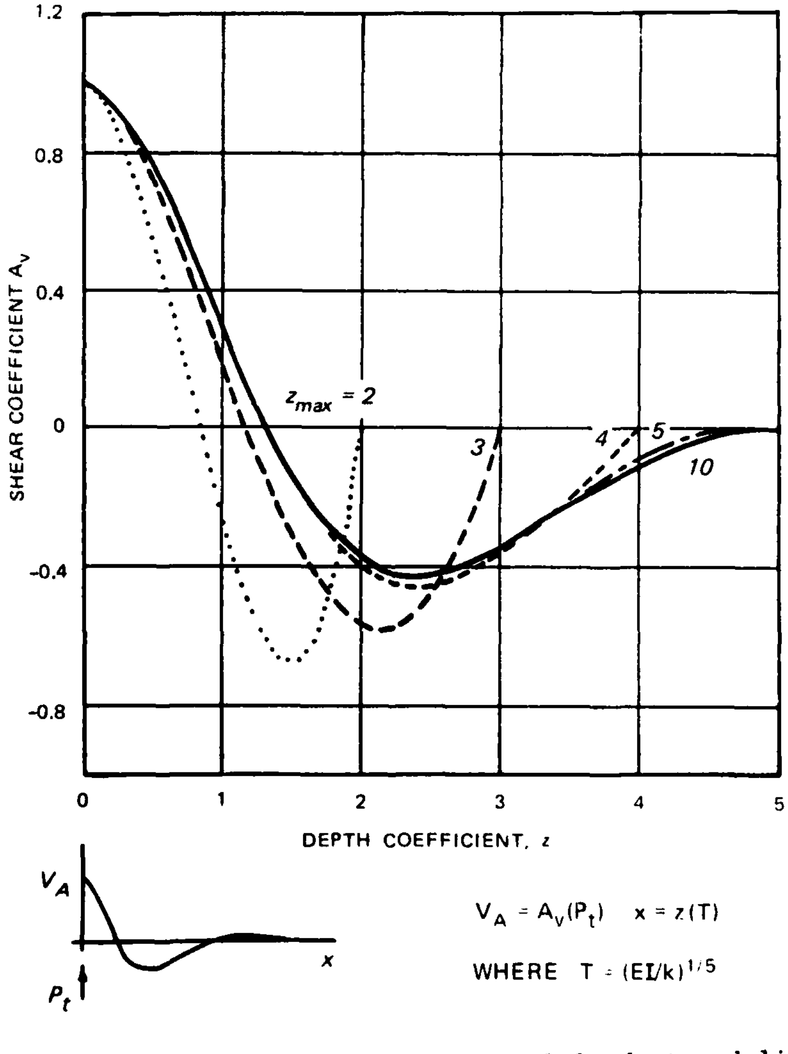


Figure A8. Shear produced by lateral load at mud line (Reese and Sullivan 1980)

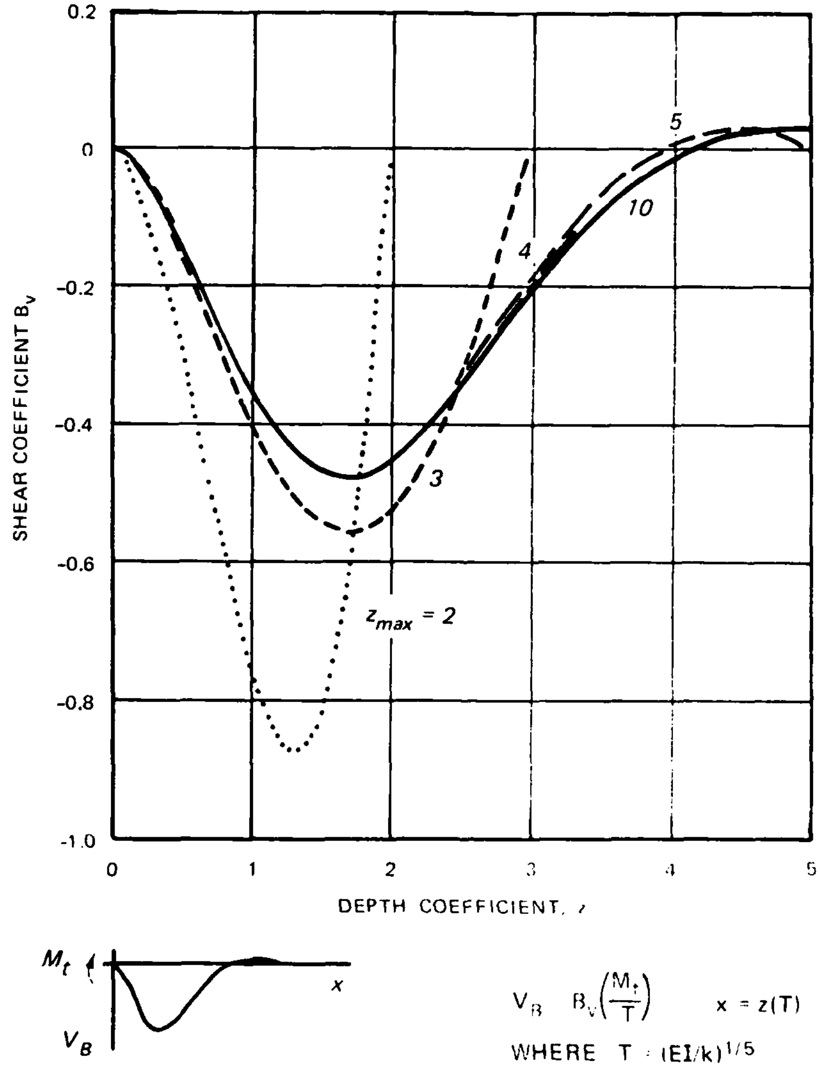


Figure A9. Shear produced by moment applied at mud line (Reese and Sullivan 1980)

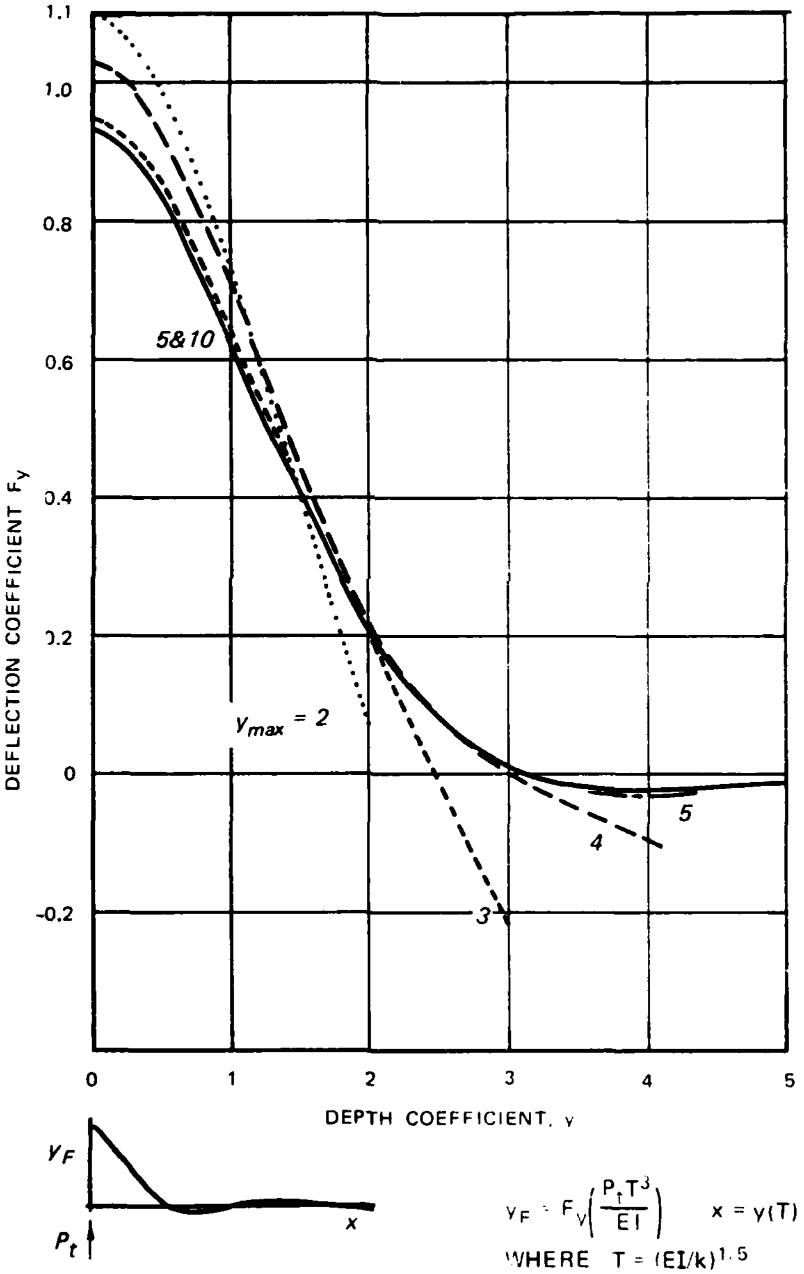


Figure A10. Deflection of pile fixed against rotation at mud line (Reese and Sullivan 1980)

Table A1

Moment Coefficients at Top of Pile for Fixed-Head Case

z_{max}	F_{Mt}
2	-1.06
3	-0.97
4	-0.93
5 and above	-0.93

Case III: Pile head restrained against rotation

9. Case III may be used to obtain a solution for the case where the superstructure translates under load but does not rotate.

a. Perform steps a, b, c of the solution procedure free-head piles (Case I).

b. Obtain the value of the spring stiffness k_θ of the pile superstructure system. The spring stiffness is defined as

$$k_\theta = \frac{M_t}{S_t} \tag{A9}$$

where

M_t = moment at top of pile

S_t = slope at top of pile

c. Compute the slope at the top of the pile S_t from

$$S_t = A_{st} \frac{P_T T^2}{EI} + B_{st} \frac{M_T T}{EI} \tag{A10}$$

where

A_{st} = slope coefficient (From Figure A4)

B_{st} = slope coefficient (from Figure A5)

d. Solve Equations A9 and A10 for the moment at the top of the pile M_t .

e. Perform steps a through i of the solution procedure for free-head piles (Case I).

10. This process completes the solution of the laterally loaded pile problem for three sets of boundary conditions. The solution gives values of deflection, slope, moment, shear, and soil reaction as a function of depth. To illustrate the nondimensional method, an example solution is presented next.

Example Solution

11. The following paragraphs present an example analysis using the nondimensional method and a comparison of the results with the computer solution of the same problem.

Problem statement

12. Figure A11 illustrates the problem to be solved by the nondimensional method as well as pertinent soils data. This same problem, as solved by COM624G, is presented in Appendix D as example problem 1. A comparison of the two solutions is presented following the nondimensional solution.

Nondimensional solution

13. The solution will proceed in the step-by-step manner described for Case I.

14. Step 1. Compute and construct p-y curves. The p-y curves for the example problem as generated by COM624G (using the soft clay criteria) are presented in Appendix D, example problem 1. These same curves are generated manually in the following steps to illustrate the hand procedure. The computations follow the step-by-step procedure given for soft clay criteria in Part III of the main report. Computations for both static and cyclic curves are presented; however, only cyclic curves are utilized in the pile analysis. The depths for which curves are to be computed are: 0, 16, 32, 48, 80, 128, 154, and 240 in. Only the static and cyclic curves for $x = 48$ in. are computed in the following example:

a. Static curves:

(1) Obtain the variation of shear strength and submerged unit weight with depth and determine ε_{50} . (See Table 3, Part III of the main text.)

The following properties are used:

$c = 500$ psf $= 3.47$ psi

$\gamma' = 30$ pcf $= 0.0168$ pci

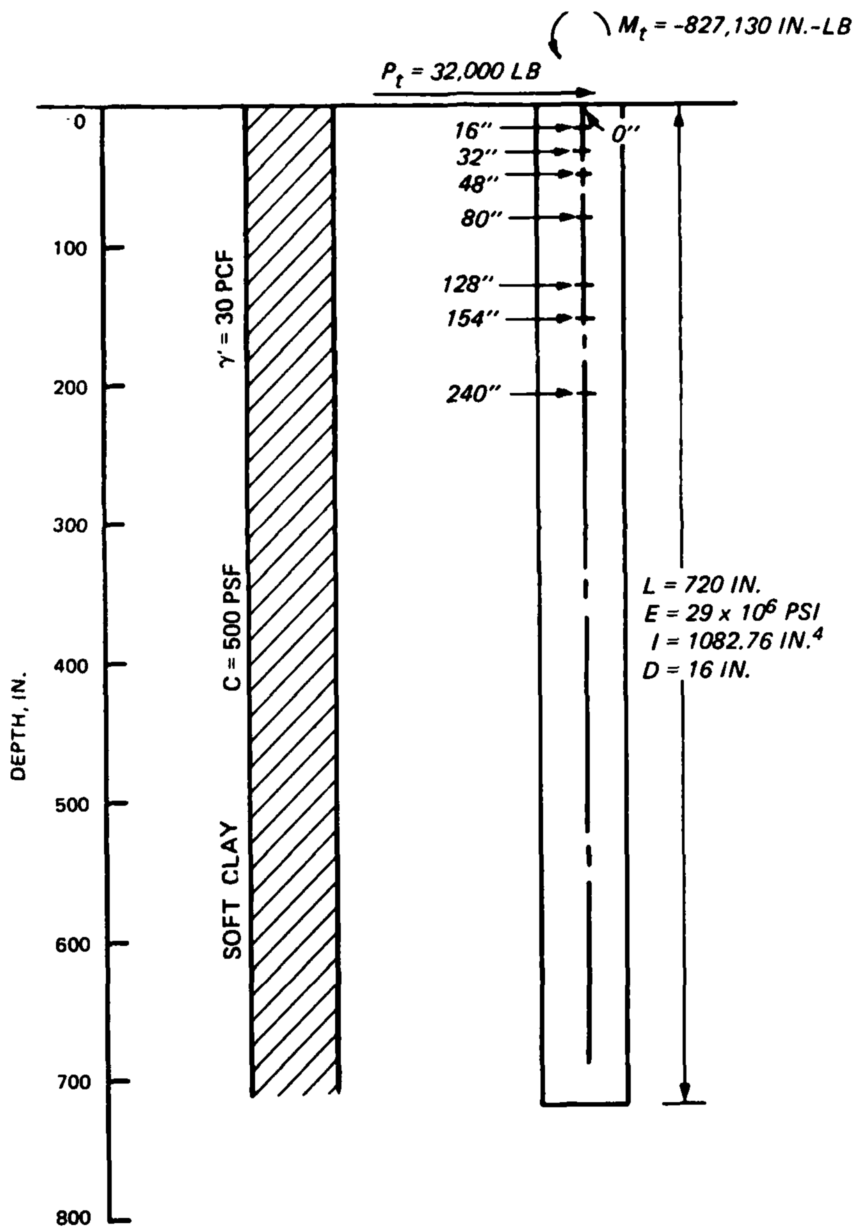


Figure A11. Example problem for solution by nondimensional methods

$$\varepsilon_{50} = 0.010$$
$$b = 16 \text{ in.}$$
$$x = 48 \text{ in.}$$

(2) Compute p_u using the smaller of the values from

$$p_u = \left(3 + \frac{\gamma'}{c} x + \frac{0.5}{b} x\right) cb$$

and

$$p_u = 9cb$$

$$p_u = \left[3 + \frac{0.0168}{3.47} (48) + \frac{0.5}{16} (48)\right] 3.47(16)$$
$$= 262.7 \text{ lb/in.}$$

$$p_u = 9(3.47)(16) = 499.7 \text{ lb/in.}$$

Therefore, use

$$p_u = 262.7 \text{ lb/in.}$$

---

(3) Compute y_{50} at half p_u :

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

$$y_{50} = 2.5(0.010)(16) = 0.40 \text{ in.}$$

---

(4) Compute points describing the p-y curve:

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

p is constant beyond $y = 8y_{50}$.

---

y , in.	p , lb/in.
0.2	104.3
0.4	131.4
0.8	165.5
1.2	189.4
2.0	224.6
3.2	262.7

$$8y_{50} = 8(0.40) = 3.2 \text{ in.}$$

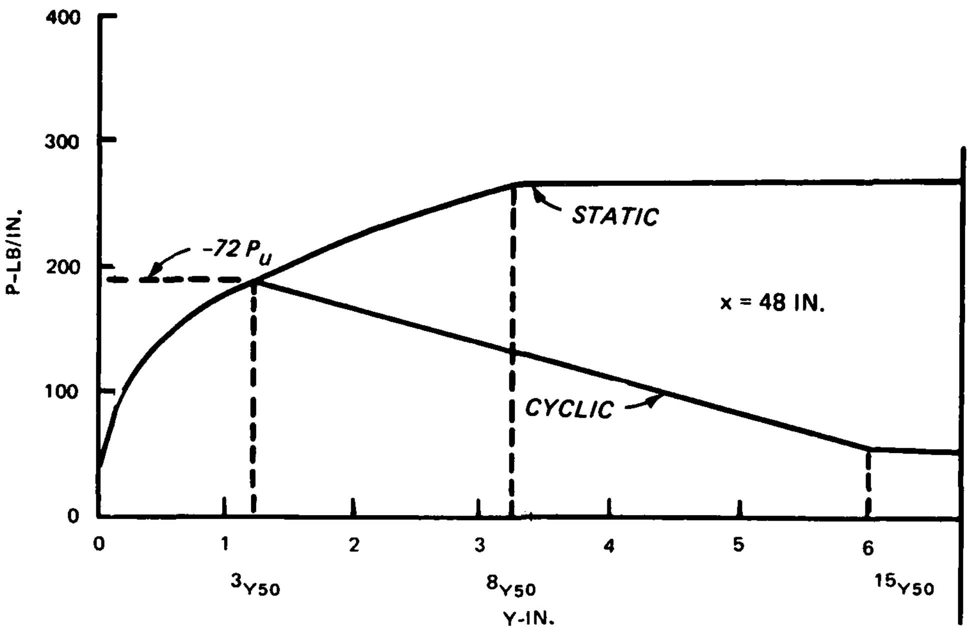


Figure A12. Computed static and cyclic p-y curves for x = 48 in.

(5) The computed static p-y curve is plotted in Figure A12.

b. Cyclic curves:

(1) The cyclic curve is the same as the static curve for p less than $0.72p_u$.

(2) Solve for x_r:

$$x_r = \frac{6cb}{\gamma' b + 0.5c}$$

$$x_r = \frac{6(3.47)(16)}{0.0168(16) + 0.5(3.47)}$$

$$x_r = 166.2 \text{ in.}$$

---

(3) If $x \geq x_r$, $p = 0.72p_u$ for $y > 3y_{50}$.

(4) If $x < x_r$, p decreases from $0.72p_u$ at $y = 3y_{50}$ to p in the following equation at $y = 15y_{50}$:

$$p = 0.72p_u \frac{x}{x_r}$$

$$p = 0.72(262.7)\ \frac{48}{166.2} = 54.6 \text{ lb/in.}$$

$$y = 15y_{50} = 15(0.40) = 6.0 \text{ in.}$$

$$p = 0.72p_u = 0.72(262.7) = 189.1 \text{ lb/in.}$$

$$y = 3y_{50} = 3(0.40) = 1.2 \text{ in.}$$

---

(5) The computed cyclic p-y curve is plotted in Figure A12.

c. The remainder of the p-y curves for the other values of x are computed using the same procedure. These computed curves are presented in Figure A13.

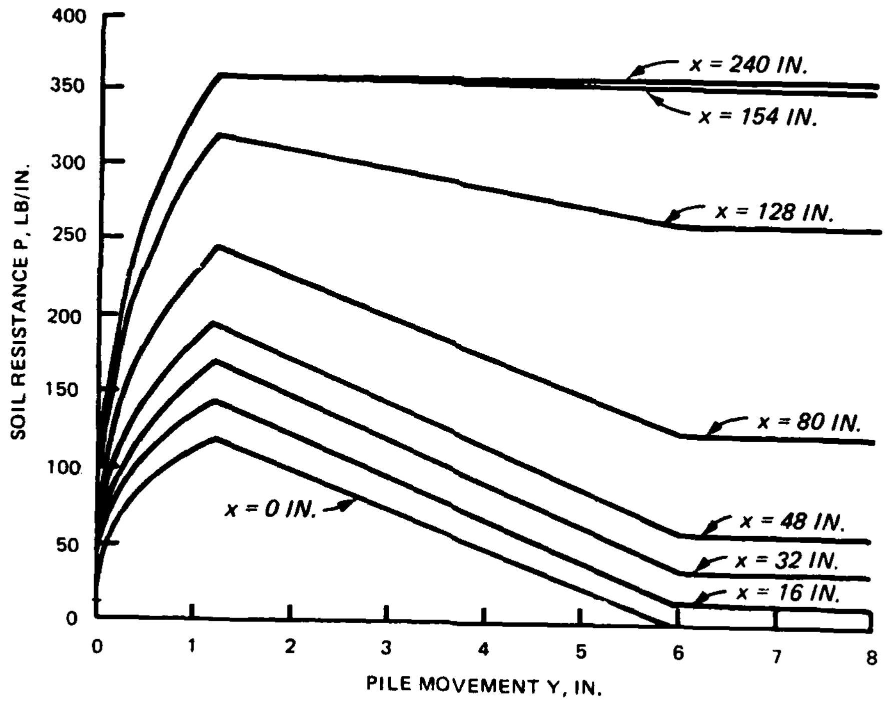


Figure A13. Plot of p-y curves for example problem solved by nondimensional method; soft clay criteria, cyclic loading

15. Step 2. Assume T : T = 95 in.

16. Step 3. Compute z_{max} :

$$z_{max} = \frac{L}{T} = \frac{720}{95} = 7.58$$

17. Step 4. Compute the deflection y at depths of 0, 16, 32, 48, 80, 128, 154, and 240 in. using Equation A3 and Figures A2 and A3. The computations are presented in tabular form in Table A2.

18. Step 5. From the set of p-y curves (Figure A13) the values of p are determined corresponding to the y values computed in step 4 (see the tabulation in Table A2).

19. Step 6. Compute the E_s value at each depth (see the tabulation in Table A2).

20. Step 7. Prepare a plot of E_s versus depth as shown in Figure A14. In fitting the straight line to the plotted points, more weight should be given to the points near the ground surface. The k value is determined as the slope of this line:

$$k = \frac{E_s}{x} = \frac{500}{142} = 3.52 \text{ lb/in.}^3$$

21. Step 8. Compute T :

$$T = 5\ \frac{EI}{k} = \sqrt[5]{\frac{(3.14)10^{10}}{3.52}} = 97.9 \text{ in.}$$

Step 8 completes the first iteration of the solution procedure. Before proceeding to the next iteration, the results thus far should be examined to provide guidance in further computations. It is evident from Figure A14 that $E_s = kx$ is not a good representation of the variation of the soil modulus with depth. A straight line through the origin does not fit the plotted points. However, the constraints of the method required that the line pass through the origin to satisfy the assumption that $E_s = kx$. Figure A14 also reveals that the solution has not been found because the k value of 4.0 pci that was assumed is not equal to the k of 3.52 pci that was obtained. Correspondingly, the assumed value of T was not equal to the T value obtained. From comparisons, it appears that the value of k will decrease and T will increase with successive iterations. The iterations are continued until the desired degree of convergence is achieved. In the example problem, the computations were continued for three additional iterations. The additional computations are shown in Tables A3-A5; the corresponding plots of E_s versus x are shown in Figures A15-A17. For this example, the computations were continued until the deflections at the groundline agreed within 5 percent for the

Table A2

Nondimensional Analysis of Laterally Loaded Piles with Pile Head Free to Rotate Computations for Iteration No. 1

P_t = 32,000 lb M_t = -827,130 in.-lb EI = 3.14×10^{10} lb-in.^{2}

Trial 1 $k_{assumed}$ = 4.0 lb-in.^{3} (or $T_{assumed}$ = 95 in.)

$T = \left(\frac{EI}{k}\right)^{1/5}$ = 95 in. $z_{max} = \frac{L}{T}$ = 7.58

Depth in.	Depth Coefficient	Deflection Coefficient	Deflection Coefficient	Deflection in.	Soil Resistance lb/in.	Soil Modulus lb/in.^{2}
x	$z = \frac{x}{T}$	A_y, from Figure A2	B_y, from Figure A3	$y = A_y \frac{P_t T^3}{EI} + B_y \frac{M_t T^2}{EI}$ $0.874A_y + 0.238B_y$	p , from p-y Curve	$E_s = -\frac{p}{y}$
0	0.0	2.40	1.60	1.72	110	64
16	0.17	2.15	1.33	1.56	138	88
32	0.34	1.85	1.10	1.35	163	121
48	0.51	1.60	0.85	1.20	195	163
80	0.84	1.15	0.50	0.89	220	247
128	1.35	0.58	0.13	0.48	233	485
154	1.62	0.32	0.02	0.27	220	815
240	2.53	-0.03	-0.10	0.00		

$k = \frac{E_s}{x}$ = 3.5 $T_{obtained} = \left(\frac{EI}{k}\right)^{1/5}$ = 97.9 in.

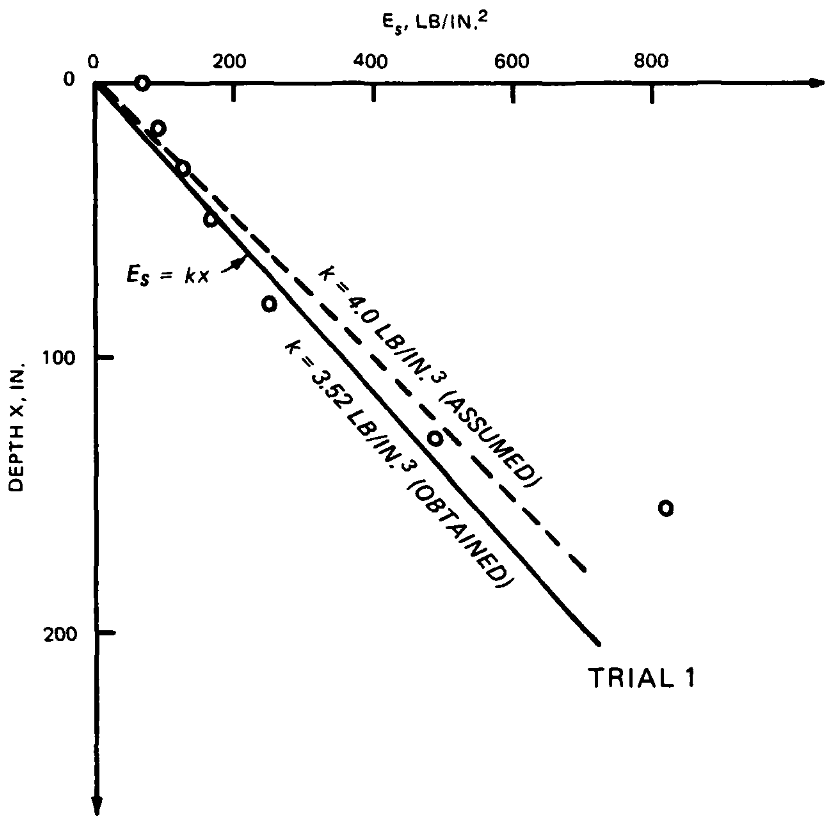


Figure A14. Plot of E_s versus x for example problem; first iteration

Table A3

Nondimensional Analysis of Laterally Loaded Piles with Pile Head Free to Rotate Computations for Iteration No. 2

P_t = 32,000 lb M_t = -827,130 in.-lb   EI = 3.14×10^{10} lb-in.^{2}

Trial 2 $k_{assumed}$ = 3.5 lb-in.^{3} (or $T_{assumed}$ = 97.9 in.)

$T = \left(\frac{EI}{k}\right)^{1/5}$ = 97.9 in. $z_{max} = \frac{L}{T}$ = 7.35

Depth in.	Depth Coefficient	Deflection Coefficient	Deflection Coefficient	Deflection in.	Soil Resistance lb/in.	Soil Modulus lb/in.^{2}
x	$z = \frac{x}{T}$	A_y, from Figure A2	B_y, from Figure A3	$y = A_y \frac{P_t T^3}{EI} + B_y \frac{M_t T^2}{EI}$	p, from p-y Curve	$E_s = -\frac{p}{y}$
0	0.00	2.40	1.60	1.89	103	54
16	0.16	2.17	1.36	1.73	132	76
32	0.33	1.86	1.07	1.51	160	106
48	0.49	1.61	0.83	1.33	190	126
80	0.82	1.17	0.52	0.99	225	227
128	1.31	0.62	0.15	0.56	250	446
154	1.58	0.35	0.03	0.33	240	727
240	2.46	-0.03	-0.10	0.00		

$k = \frac{E_s}{x}$ = 3.14 $T_{obtained} = \left(\frac{EI}{k}\right)^{1/5}$ = 100.0 in.

Table A4

Nondimensional Analysis of Laterally Loaded Piles with Pile Head Free to Rotate Computations for Iteration No. 3

P_t = 32,000 lb M_t = -827,130 in.-lb EI = 3.14×10^{10} lb-in.^{2}

Trial 3 $k_{assumed}$ = 3.14 lb-in.^{3} (or $T_{assumed}$ = 100.0 in.)

$T = \left(\frac{EI}{k}\right)^{1/5}$ = 100.0 in. $z_{max} = \frac{L}{T}$ = 7.20

Depth in.	Depth Coefficient	Deflection Coefficient	Deflection Coefficient	Deflection in.	Soil Resistance lb/in.	Soil Modulus lb/in.^{2}
x	$z = \frac{x}{T}$	A_y, from Figure A2	B_y, from Figure A3	$y = A_y \frac{P_t T^3}{EI} + B_y \frac{M_t T^2}{EI}$	p , from p-y Curve	$E_s = -\frac{p}{y}$
0	0.00	2.40	1.60	2.02	100	50
16	0.16	2.20	1.35	1.89	128	68
32	0.32	1.87	1.10	1.62	160	99
48	0.48	1.63	C.85	1.44	190	132
80	0.80	1.20	0.55	1.08	237	219
128	1.28	0.65	0.15	0.62	250	403
154	1.54	0.37	0.05	0.36	24(	667
240	2.40	0.00	0.10	0.03	75	2500

$k = \frac{E_s}{x}$ = 2.91 $T_{obtained} = \left(\frac{EI}{k}\right)^{1/5}$ = 101.5 in.

Table A5

Nondimensional Analysis of Laterally Loaded Piles with Pile Head Free to Rotate Computations for Iteration No. 4

P_t = 32,000 lb M_t = -827,130 in.-lb EI = 3.14×10^{10} lb-in.^{2}

Trial 4 $k_{assumed}$ = 2.91 lb-in.^{3} (or $T_{assumed}$ = 101.5 in.)

$T = \left(\frac{EI}{k}\right)^{1/5}$ = 101.5 in. $z_{max} = \frac{L}{T}$ = 7.09

Depth in.	Depth Coefficient	Deflection Coefficient	Deflection Coefficient	Deflection in.	Soil Resistance lb/in.	Soil Modulus lb/in.^{2}
x	$z = \frac{x}{T}$	A_y, from Figure A2	B_y, from Figure A3	$y = A_y \frac{P_t T^3}{EI} + B_y \frac{M_t T^2}{EI}$	p , from p-y Curve	$E_s = -\frac{p}{y}$
0	0.00	2.40	1.60	2.12	95	45
16	0.16	2.20	1.35	1.98	125	63
32	0.32	1.87	1.10	1.69	158	93
48	0.47	1.63	0.85	1.51	187	124
80	0.79	1.20	0.55	1.13	240	212
128	1.26	0.65	0.15	0.65	257	395
154	1.52	0.37	0.05	0.38	243	639
240	2.36	0.00	-0.10	0.03	75	2500
480	4.73	0.00	0.00	0.00		
720	7.09	0.00	0.00	0.00		

$k = \frac{E_s}{x}$ = 2.9 $T_{obtained} = \left(\frac{EI}{k}\right)^{1/5}$ = 102 in.

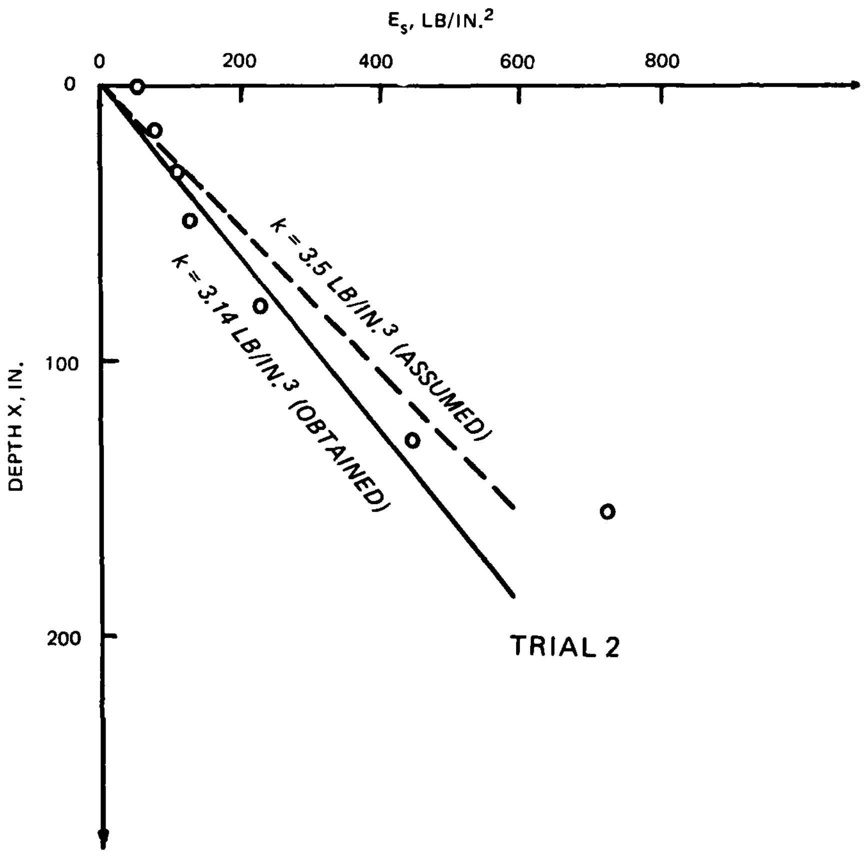


Figure A15. Plot of E_s versus x for example problem; second iteration

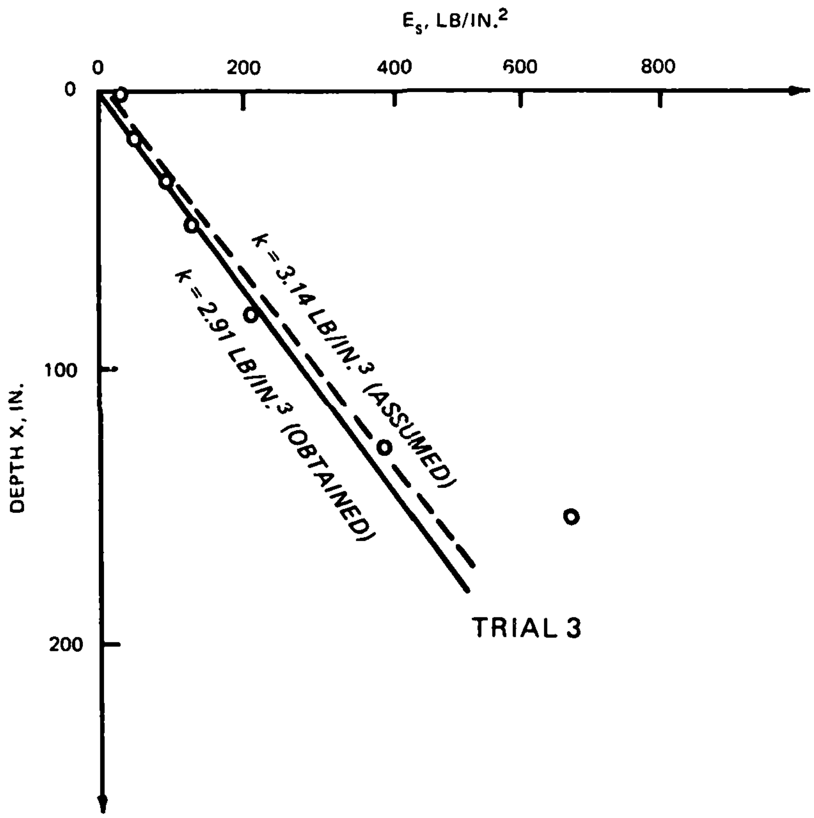


Figure A16. Plot of E_s versus x for example problem; third iteration

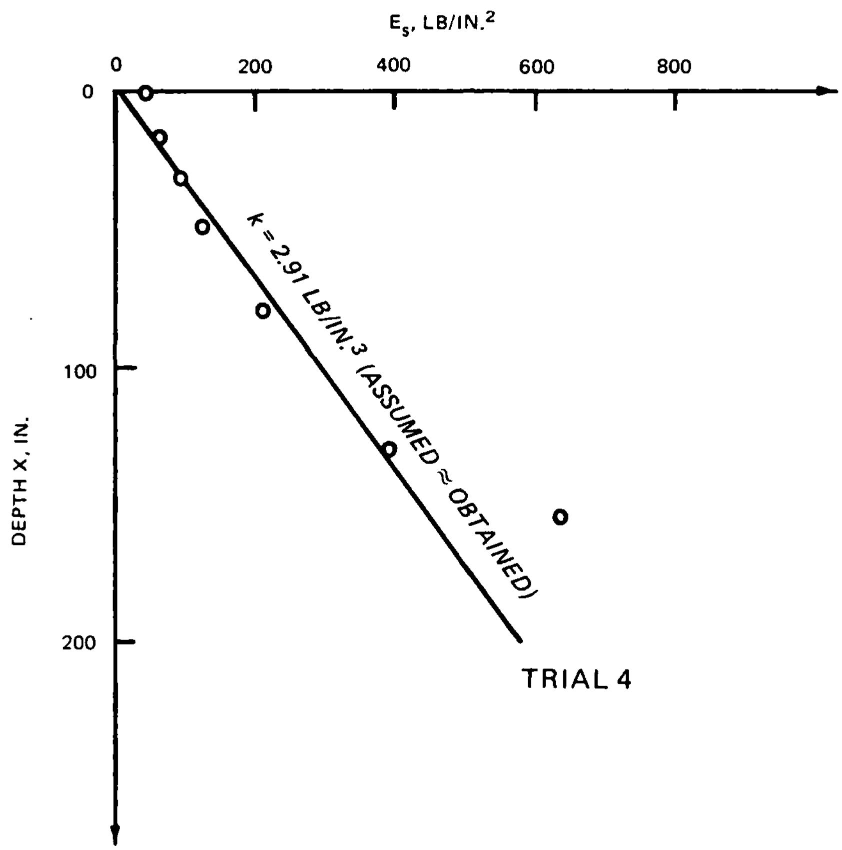


Figure A17. Plot of E_s versus x for example problem; fourth iteration last two iterations. However, the number of iterations for a particular problem should be determined by the user after giving due consideration to the degree of accuracy required and to the limitations inherent in the method. After the final iteration is complete, continue with step 9.

22. Step 9. The final step in the computation procedure is to determine the results of the analysis as follows:

a. The value of deflection y and soil reaction p along the pile are known from step 4 of the final iteration (Table A5). These results are presented in Figures A18 and A19 and are compared with the computer solution of example problem 1 from Appendix D.

b. Compute slope S versus depth from Equation A4:

$$S = A_s \frac{P_t T^z}{EI} + B_s \frac{M_t T}{EI} \qquad \text{(A4 bis)}$$

where A_s and B_s are slope coefficients taken from Figures A4 and A5, respectively. Results of the computations are presented in tabular form in Table A6 and in graphic form in Figure A20.

c. Compute moment M versus depth from Equation A5:

$$M = A_m P_t T + B_m M_t \qquad \text{(A5 bis)}$$

where A_m and B_m are moment coefficients taken from Figures A6 and A7, respectively. Results of these computations are presented in tabular form in Table A7 and in graphic form in Figure A21. Also plotted in Figure A21 are results from the computer solution.

d. Compute shear V versus depth from Equation A6:

$$V = A_v P_t + \frac{B_v M_t}{T} \qquad \text{(A6 bis)}$$

where A_v and B_v are shear coefficients taken from Figures A8 and A9, respectively. Results of these computations are presented in tabular form in Table A8 and in graphic form in Figure A22.

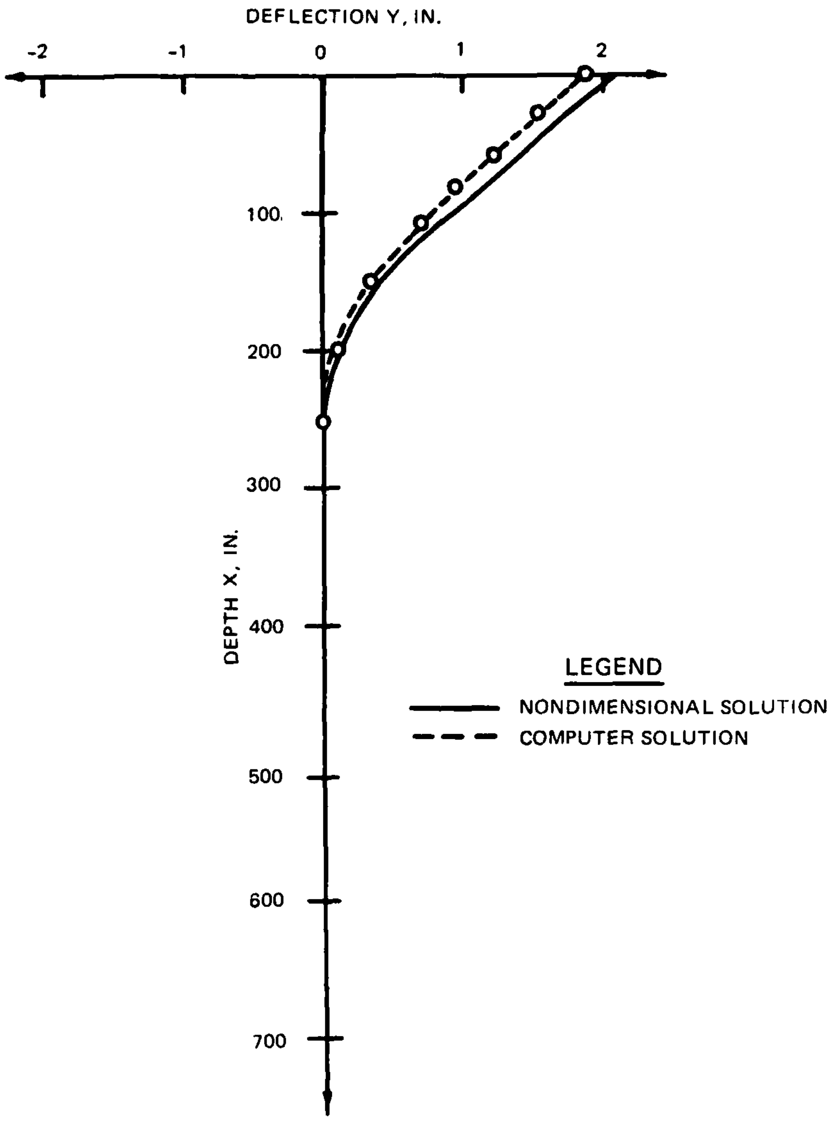


Figure A18. Plots of deflection y versus depth x for example problem

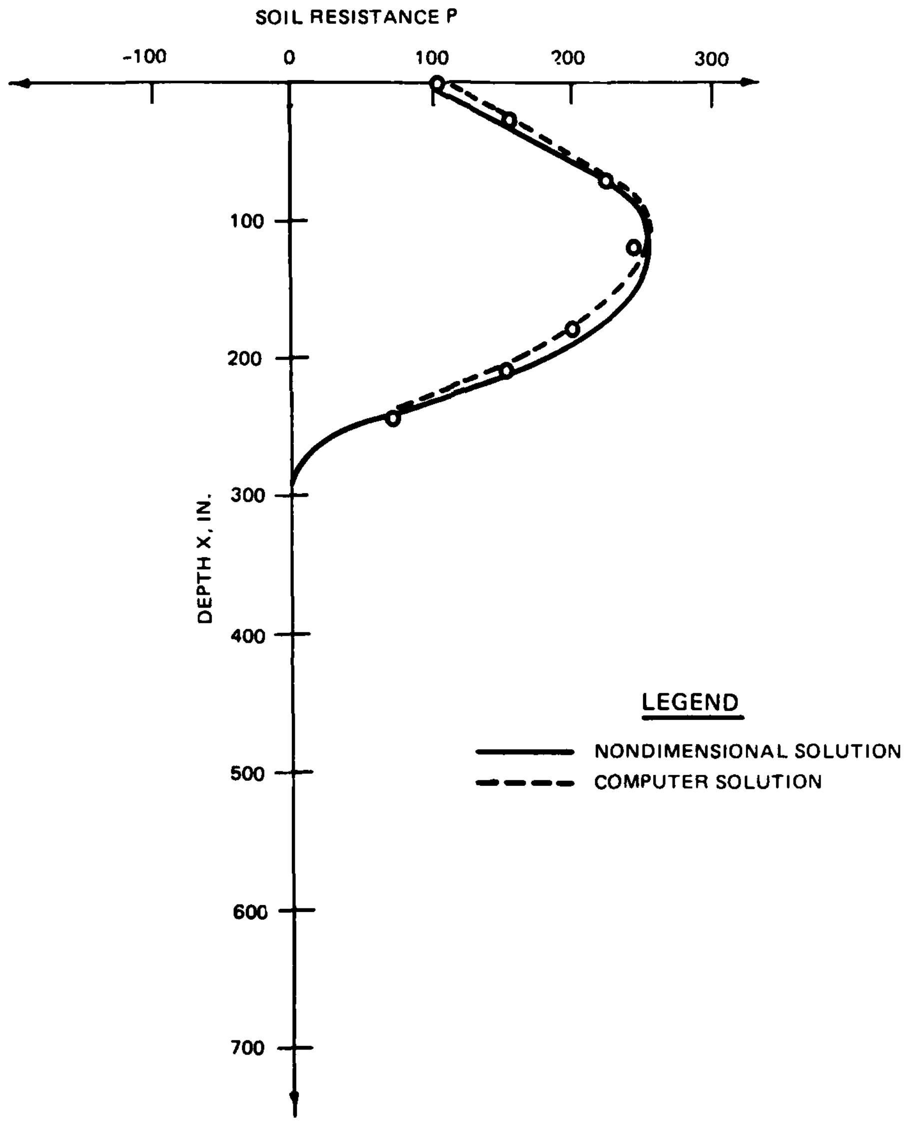


Figure A19. Plot of soil resistance p versus depth x for example problem

Table A6

Computed Slopes

Depth in.	Depth Coefficient	Slope Coefficient	Slope Coefficient	Slope
x	$z = \frac{x}{T}$	A_s , from Figure A4	B_s , from Figure A5	$S = A_s \frac{P_T T^2}{EI} + B_s \frac{M_T T}{EI}$
0	0.0	-1.625	-1.750	-0.0124
16	0.16	-1.600	-1.625	-0.0125
32	0.32	-1.560	-1.425	-0.0126
48	0.47	-1.510	-1.285	-0.0124
80	0.79	-1.350	-0.975	-0.0116
128	1.26	-1.000	-0.575	-0.0090
154	1.52	-0.800	-0.400	-0.0073
240	2.36	-0.260	-0.048	-0.0026
480	4.73	0.035	0.025	0.0003
720	7.09	0.000	0.000	0.0000

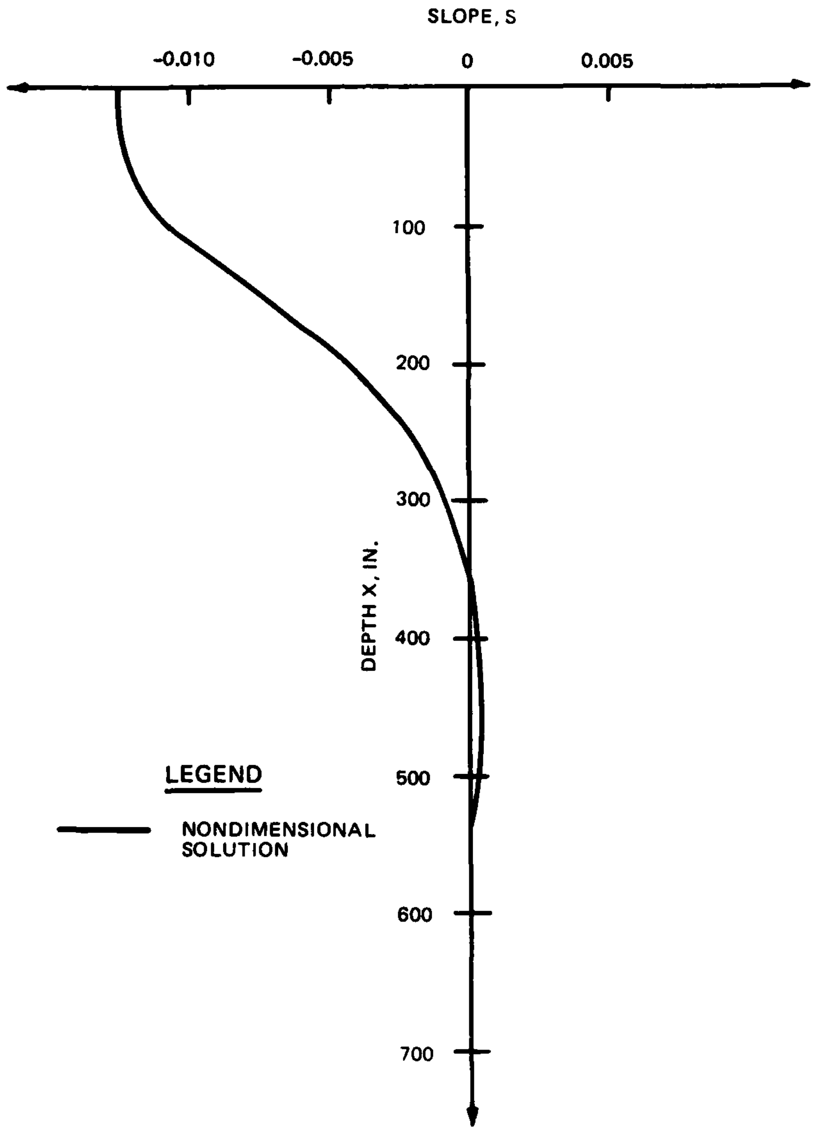


Figure A20. Plot of slope versus depth for example problem

Table A7

Computed Moments

Depth in.	Depth Coefficient	Moment Coefficient	Moment Coefficient	Moment in.-lb
x	$z = \frac{x}{T}$	A_M, from Figure A6	B_M, from Figure A7	$M = A_M P_t T + B_M M_t$
0	0.0	0.00	1.00	-8.27×10^5
16	0.16	0.16	1.00	-3.07×10^5
32	0.32	0.32	0.99	2.21×10^5
48	0.47	0.44	0.98	6.19×10^5
80	0.79	0.65	0.92	1.35×10^6
128	1.26	0.77	0.75	1.88×10^6
154	1.52	0.76	0.63	1.95×10^6
240	2.36	0.49	0.25	1.38×10^6
480	4.73	-0.01	-0.02	-1.59×10^4
720	7.09	0.00	0.00	0.0

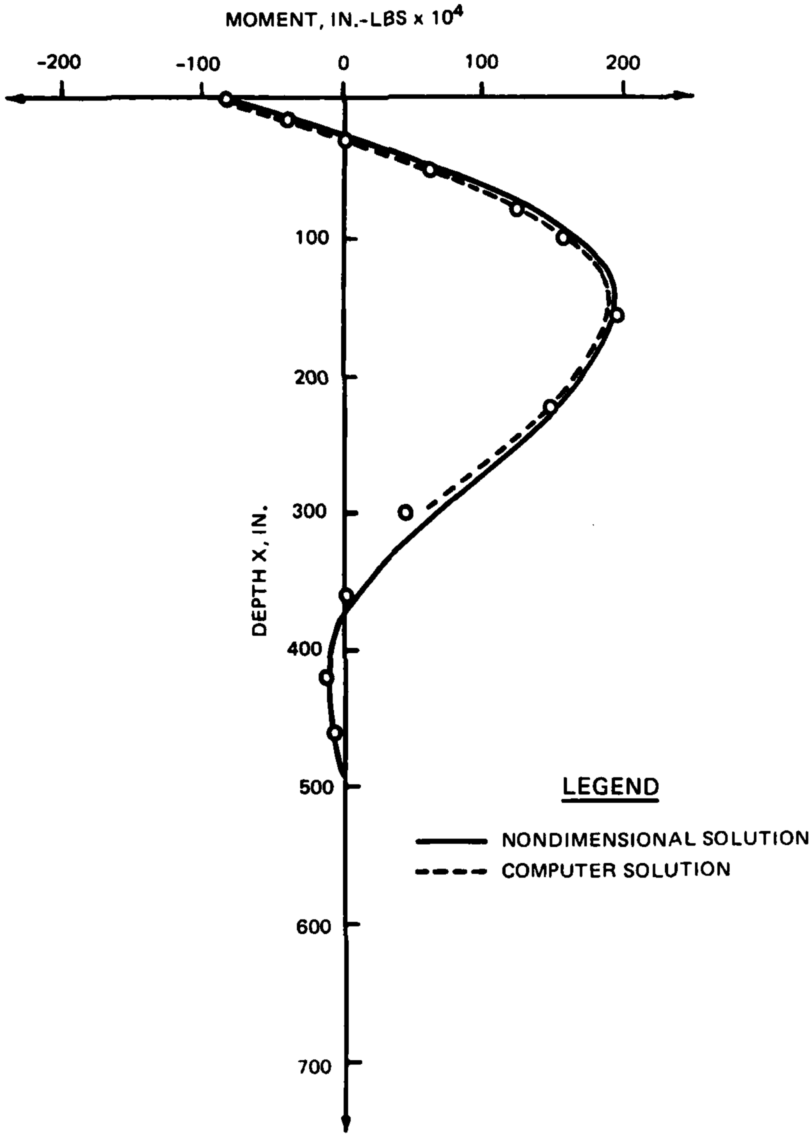


Figure A21. Plot of moment versus depth for example problem

Table A8

Computed Shears

Depth in.	Depth Coefficient	Shear Coefficient	Shear Coefficient	Shear lb
x	$z = \frac{x}{T}$	A_v, from Figure A8	B_v, from Figure A9	$V = A_v P_t + B_v \frac{M_t}{T}$
0	0.00	1.00	0.00	32,000
16	0.16	0.97	-0.02	30,400
32	0.32	0.89	-0.07	29,050
48	0.47	0.78	-0.13	26,019
80	0.79	0.50	-0.26	18,119
128	1.26	0.05	-0.43	5,104
154	1.52	-0.15	-0.47	-970
240	2.36	-0.43	-0.39	-10,582
480	4.73	0.0	0.02	-163
720	7.09	0.0	0.00	0

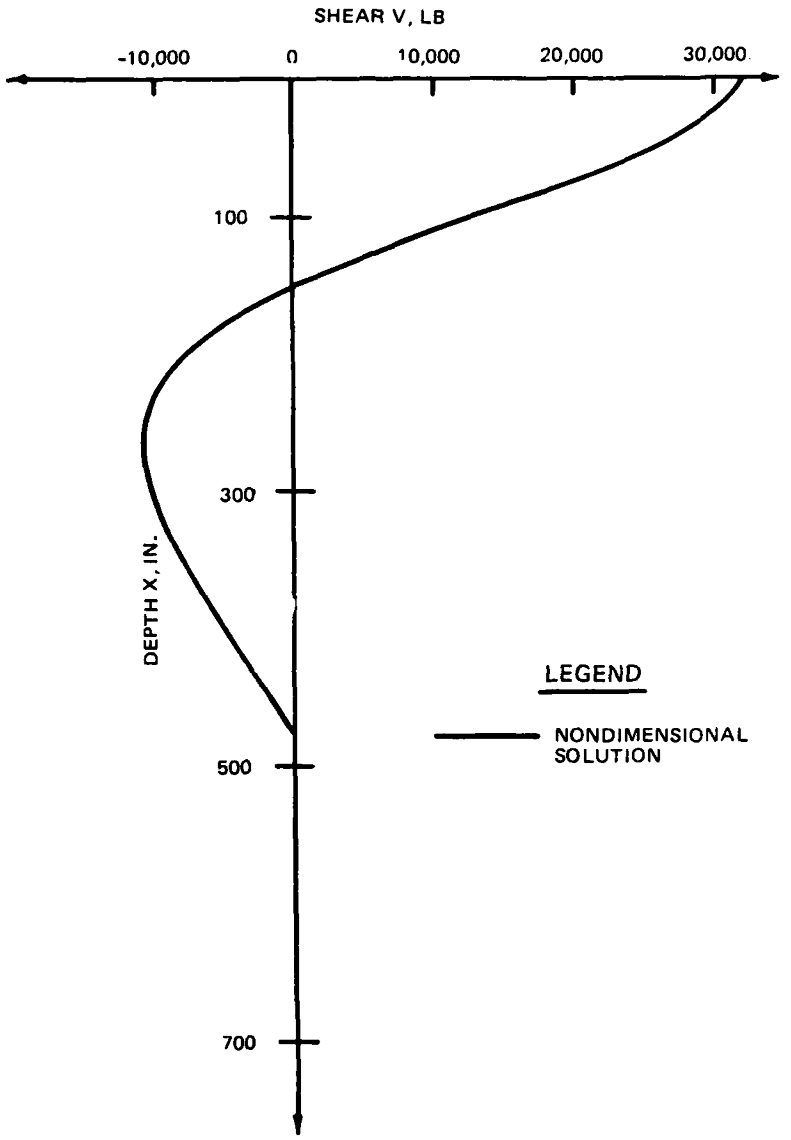


Figure A22. Plot of shear versus depth for example problem

23. Tables A9 through A11 present forms which are included for convenience of the user when making nondimensional analyses.

Comparison between nondimensional and computer solutions

24. Comparisons between the nondimensional solution and the computer solution (Appendix D, example problem 1) are presented in Figures A18, A19, and A21. Figure A18 presents a comparison of deflection versus depth. As is shown, the maximum variation occurs at the ground surface and is approximately 12 percent. Figure A19 presents a comparison of soil resistance versus depth. The maximum percentage difference occurs at the ground surface and is approximately 10 percent. The maximum numerical difference occurs at the depth of maximum soil resistance (120 in.) and is approximately 12 lb/in. Figure A21 presents a comparison of moment versus depth. The maximum variation is approximately 6 percent and occurs at a depth of approximately 100 in. The maximum moment occurs at a depth of approximately 150 in. and the two methods yield essentially equal results.

25. The comparisons presented above indicate good to excellent agreement between the nondimensional and computer solutions. However, the user should be aware that the variations presented above apply only to this particular problem and variations for other problems may be larger or smaller. When considering whether or not the nondimensional solution yields a satisfactory degree of accuracy, the user should consider the variables inherent in computing the response of a laterally loaded pile.

Table A9

Nondimensional Analysis of Laterally Loaded Piles with Pile Head Free to Rotate

P_t = ______ lb M_t = ______ in.-lb EI = ______ lb-in.^{2}

Trial ______ $k_{assumed}$ = ______ lb-in.^{3} (or $T_{assumed}$ = ______ in.)

$T = \left(\frac{EI}{k}\right)^{1/5}$ = ______ in. $z_{max} = \frac{L}{T}$ = ______

Depth in.	Depth Coefficient	Deflection Coefficient	Deflection Coefficient	Deflection in.	Soil Resistance lb/in.	Soil Modulus lb/in.^{2}
x	$z = \frac{x}{T}$	A_y, from Figure A2	B_y, from Figure A3	$y = A_y \frac{P_t T^3}{EI} + B_y \frac{M_t T^2}{EI}$	p , from p-y Curve	$E_s = -\frac{p}{y}$

$k = \frac{E_s}{x}$ = ______ $T_{obtained} = \left(\frac{EI}{k}\right)^{1/5}$ = ______ in.

Table A10

Nondimensional Analysis of Laterally Loaded Piles with Pile Head Restrained Against Rotation

P_t = ______ lb   EI = ____________ $lb\text{-}in.^2$

Trial _____ $k_{assumed}$ = _____ $lb\text{-}in.^3$ (or $T_{assumed}$ = ______ in.)

$T = \left(\frac{EI}{k}\right)^{1/5}$ = ______ in. k_θ = ____________ $\frac{in.\text{-}lb}{radian}$ A_{st} = __________ B_{st} = __________

$M_t = \frac{k_\theta A_{st} P_t T^2}{EI} \bigg/ \left(1 - \frac{B_{st} k_\theta T}{EI}\right)$ = ____________ in.-lb

Depth in.	Depth Coefficient	Deflection Coefficient	Deflection Coefficient	Deflection in.	Soil Resistance lb/in.	Soil Modulus $lb/in.^2$
x	$z = \frac{x}{T}$	A_y, from Figure A2	B_y, from Figure A3	$y = A_y \frac{P_t T^3}{EI} + B_y \frac{M_t T^2}{EI}$	p , from p-y Curve	$E_s = -\frac{p}{y}$

$k = \frac{E_s}{x}$ = _______ $T_{obtained} = \left(\frac{EI}{k}\right)^{1/5}$ = _______ in.

Table A11

Nondimensional Analysis of Laterally Loaded Piles with Pile Head Fixed Against Rotation

P_t = ______ lb M_t = ______ in.-lb EI = ______ lb-in.^{2}

Trial________ $k_{assumed}$ = ______ lb/in.^{3} (or $T_{assumed}$ = ______ in.)

$T = \left(\frac{EI}{k}\right)^{1/5}$ = ______in. $z_{max} = \frac{L}{T}$ = ______________________________

Depth in.	Depth Coefficient	Deflection Coefficient	Deflection in.	Soil Resistance lb/in.	Soil Modulus lb/in.^{2}
x	$z = \frac{x}{T}$	F_y, from Figure A10	$y = F_y \frac{P_t T^3}{EI}$	p , from p-y Curve	$E_s = \frac{P}{y}$

$k = \frac{E_s}{x}$ = ______________ lb/in.^{3} $T_{obtained} = \left(\frac{EI}{k}\right)^{1/5}$ = ____________ in.

APPENDIX B: EXAMPLE DESIGN PROBLEM

Introduction

1. The behavior of a laterally loaded pile is a complex function of soil and pile parameters and loading conditions. In many cases, complexity of behavior combined with the uncertainty of loading conditions requires the designer to investigate a range of parameters and loading conditions before arriving at a final design. This appendix presents a design problem in which soil and loading conditions are not known with certainty and illustrates some of the decisions that must be made by the designer. Meyer and Reese (1979)* present an excellent study on the effects of variations in soil parameters on computed pile behavior which should provide the user with further insight. From the example in this appendix and the study by Meyer and Reese (1979), the user should be aware of the sensitivity of the analysis to variations in parameters and loading conditions and the necessity for sound engineering judgment based on a thorough understanding of the design variables and analysis procedures.

Example Design Problem

2. The example problem, which is illustrated in Figure B1, is taken from design studies of mooring dolphin facilities for Columbia Lock and Dam on the Ouachita River in central Louisiana. The example considers one particular load case for a single-pile dolphin.

Loading case

3. The loading case presented in the example is one of several cases that might be analyzed. The specific case is for collision impact between the end of a barge and the dolphin. Other cases that might be analyzed are mooring forces from current and wind, berthing impact from the end and side of a barge, and collision impact between the end and side of a barge and the dolphin.

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* References cited in this appendix are included in the References at the end of the main text.

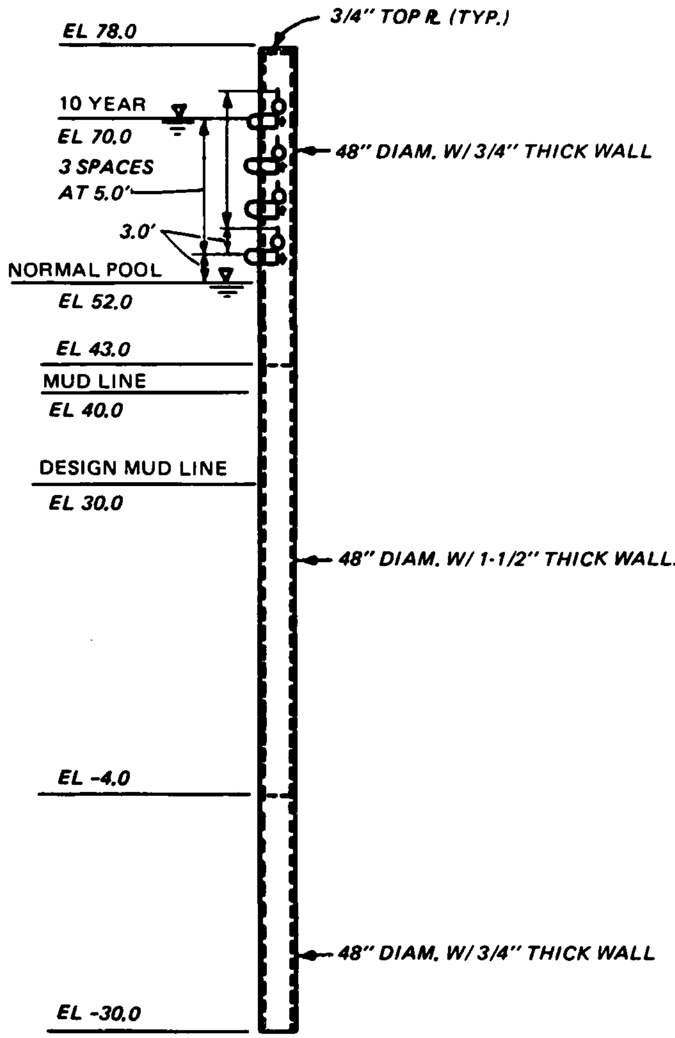

Figure B1. Example design problem, single-pile mooring dolphin

Computation of loads

4. Loads for the case presented were computed as follows:

a. Energy. Barge impact energy was computed from

$$E = f \frac{WV^2}{2g} \tag{B1}$$

where

E = impact energy, ft-lb

f = dissipation factor

W = weight of barge (tow and cargo), lb

V = velocity, normal to the dolphin, at impact, ft/sec

g = acceleration of gravity, ft/sec^2

The factor f reflects the energy dissipation created by the swing of the vessel about the dolphin after impact and is calculated from

$$f = \frac{1}{1 + 16 \frac{d^2}{L^2}} \tag{B2}$$

where

d = distance from point of contact, measured tangent to the point of contact, to the center of gravity of the barge, ft

L = length of the barge, ft

Equation B2 for the dissipation factor reveals that, for end impact, an 80 percent reduction in energy is effected.

b. Normal force. Barge impact force was computed from

$$P_{max} = \frac{2E}{\delta}$$

where

P_{max} = maximum normal force required to resist impact, lb

E = impact energy, ft-lb

δ = deflection of dolphin, ft

5. Computing the force P_{max} involves an iterative procedure in which a deflection is assumed, a trial P_{max} is computed, the analysis is performed using the trial P_{max} to obtain a new deflection, and the procedure is

continued until the trial deflection and the computed deflection agree. The forces, moments, shears, etc., are then taken from the final iteration. P_{max} can also be determined by computing a curve of P_{max} versus δ, plotting the curve, and integrating the area under the curve by trial until an energy balance is obtained.

6. Because of the dependence of P_{max} on deflection and the fact that deflection is a function of the bending moment and stiffness of the pile, a pile with a larger section modulus will not necessarily have smaller bending stresses than a pile with a smaller section modulus.

Design conditions

7. Surveys indicated the mud line to be at el 40,* as indicated in Figure B1. The top of the dolphin was set by the design criteria which required 8 ft of stickup above the 10-year frequency high-water stage (el 70). The low-water stage is el 52 which is controlled by the minimum upper pool of the lock. The design considered the force P_{max} to be applied 3 ft above the water surface. Because of the dependence of P_{max} on deflection, which in turn was dependent on bending moment and pile stiffness, it was necessary to perform analyses with P_{max} applied as a low-level force (3 ft above low water) and as a high-level force (3 ft above high water). The example presented herein considers only the high-level force. Another important variable in the design was the velocity of the barge upon impact. Based on the hydraulic analysis for the design, a velocity of 1.0 ft/sec was selected as the best estimate.

Design soil parameters

8. Borings at the site indicated the soil to be silts from the river bottom down to a depth of 15 ft. Below this, sands are indicated to extend beyond the penetration of the piling. Because p-y criteria are not available for silts, it was necessary to make a design decision as to the appropriate p-y criteria to use. The decision was to use soft clay criteria for the silts, then vary the criteria to determine the influence of the variation on the pile behavior. Sand criteria were used for the sands. The soil profile used and the design parameters are shown in Figure B2. Figure B3 presents the generated p-y curves. Cyclic p-y curves were used for both soils.

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* All elevations (el) cited herein are in feet referenced to the National Geodetic Vertical Datum (NGVD).

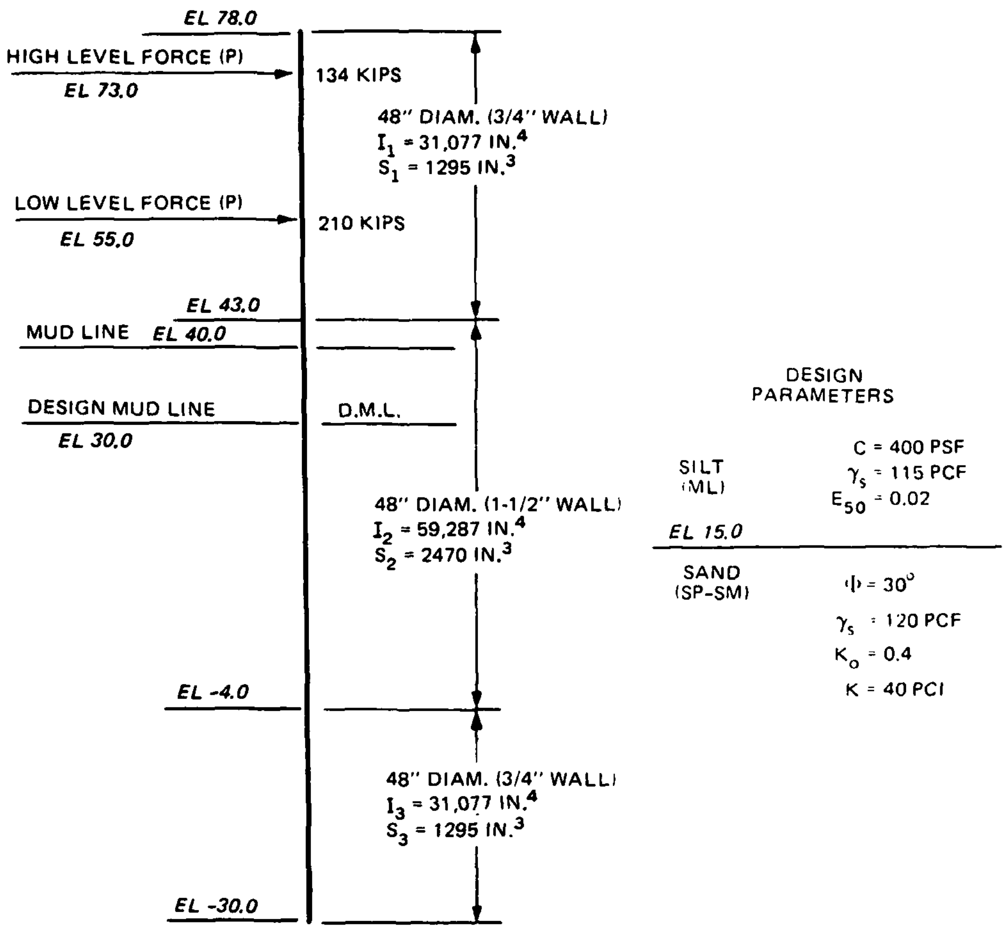


Figure B2. Pile and soil properties; single-pile mooring dolphin

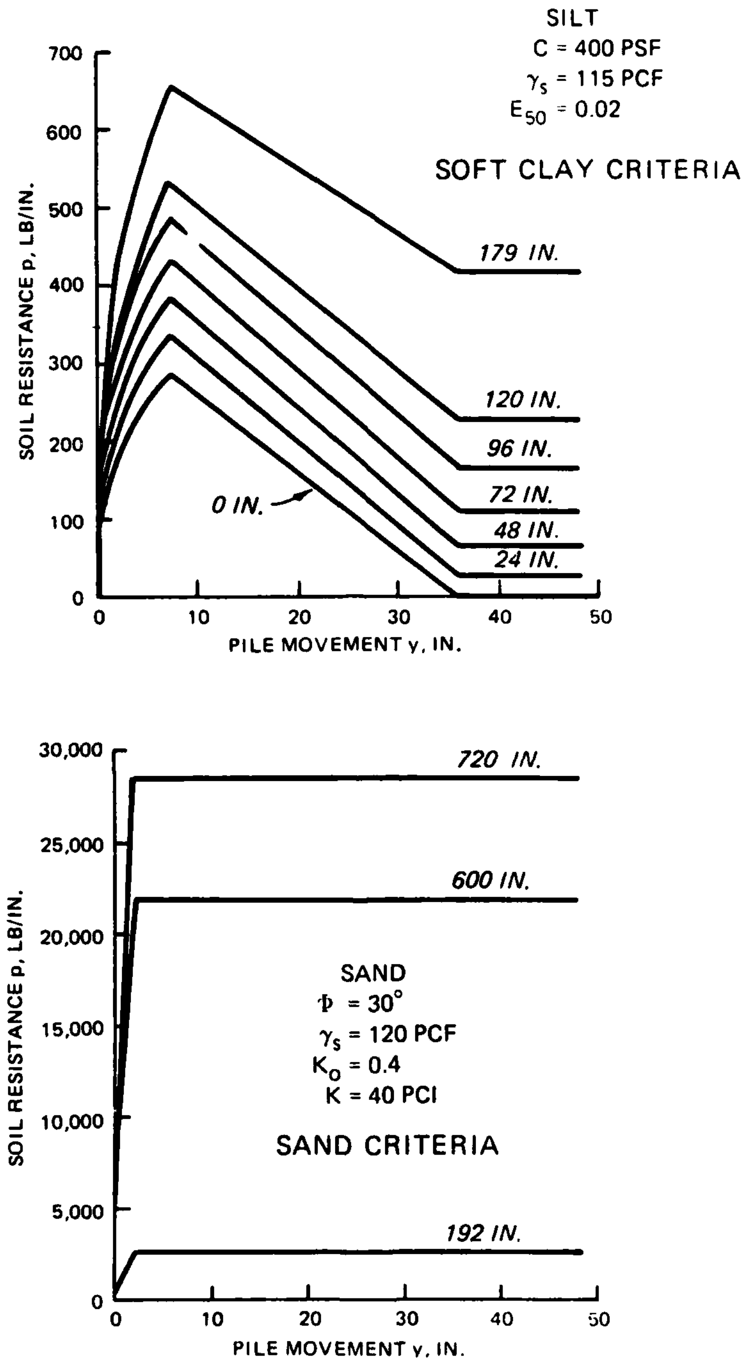


Figure B3. p-y curves; single-pile mooring dolphin

Design analyses

9. The various conditions investigated under the load case are tabulated in Table B1. Results of the analysis are presented in tabular form in Table B2 and in graphical form in Figures B4 and B5.

Conclusions

10. As can be seen in Figures B4 and B5 and Table B2, the results from an analysis can vary considerably depending on the input assumptions. For this particular example, the variation in shear strength of ±40 percent did not have a significant effect. The conditions which exhibit the most influence are the assumed 10 ft of scour and the increase in the barge velocity, with the combined effect of scour and increased barge velocity yielding the most critical condition. As shown in Table B2, the factor of safety for the combined condition drops drastically. This response is caused by the fact that the location of the maximum moment dropped into a segment of the pile which had a reduced section modulus. Obviously, this pile would not have an adequate section modulus if the conditions of scour and/or increased barge velocity were considered realistic. The final decisions in an example of this type must be made by the designer after considering the degree of certainty with which the design conditions are known.

11. A detailed input and output for computer analysis of one load case is presented in Appendix D, example 2.

Table B1

Description of Conditions Analyzed for Load Case IIIA

Condition No.	Description of Condition
1	Analyzed with a barge velocity of 1.0 ft/sec, groundline at mud line, and conventionally generated p-y curves
2	Loaded as in Condition 1 except 10 ft of scour assumed below mud line
3	Loaded as in Condition 1 except 40 percent reduction in estimated strength of the silts
4	Loaded as in Condition 1 except 40 percent increase in estimated strength of the silts
5	Velocity of barge assumed to be 1.5 ft/sec. All other factors same as in Condition 1
6	Same as Condition 5 except 10 ft of scour assumed below mud line

Table B2

Summary of Analysis

Condition No.	Pile Head Deflection in.	Deflection at Groundline in.	Maximum Bending Moment ft-kips	Factor of Safety*
1	20.3	7.5	7,442	1.62
2	28.4	12.3	4,417	0.98
3	20.9	7.9	7,642	1.62
4	19.6	7.2	7,258	1.62
5	28.1	10.7	10,083	1.21
6	41.0	18.2	11,250	0.67

* Yield strength of steel = 60 ksi.

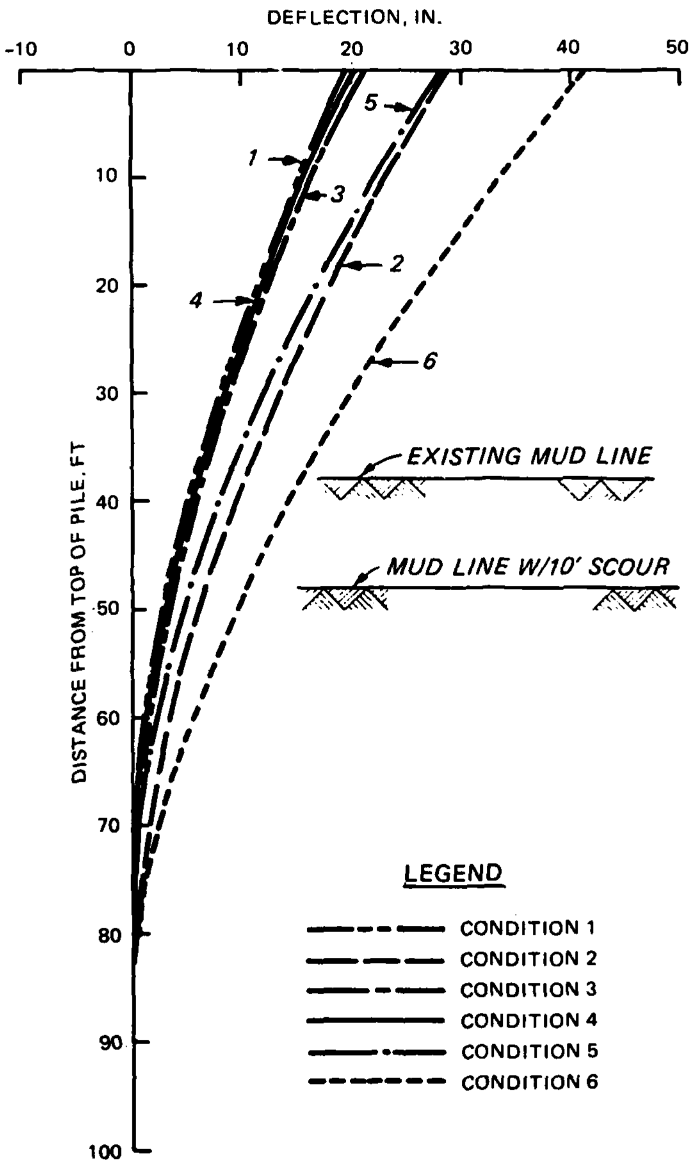


Figure B4. Plot of deflection versus depth

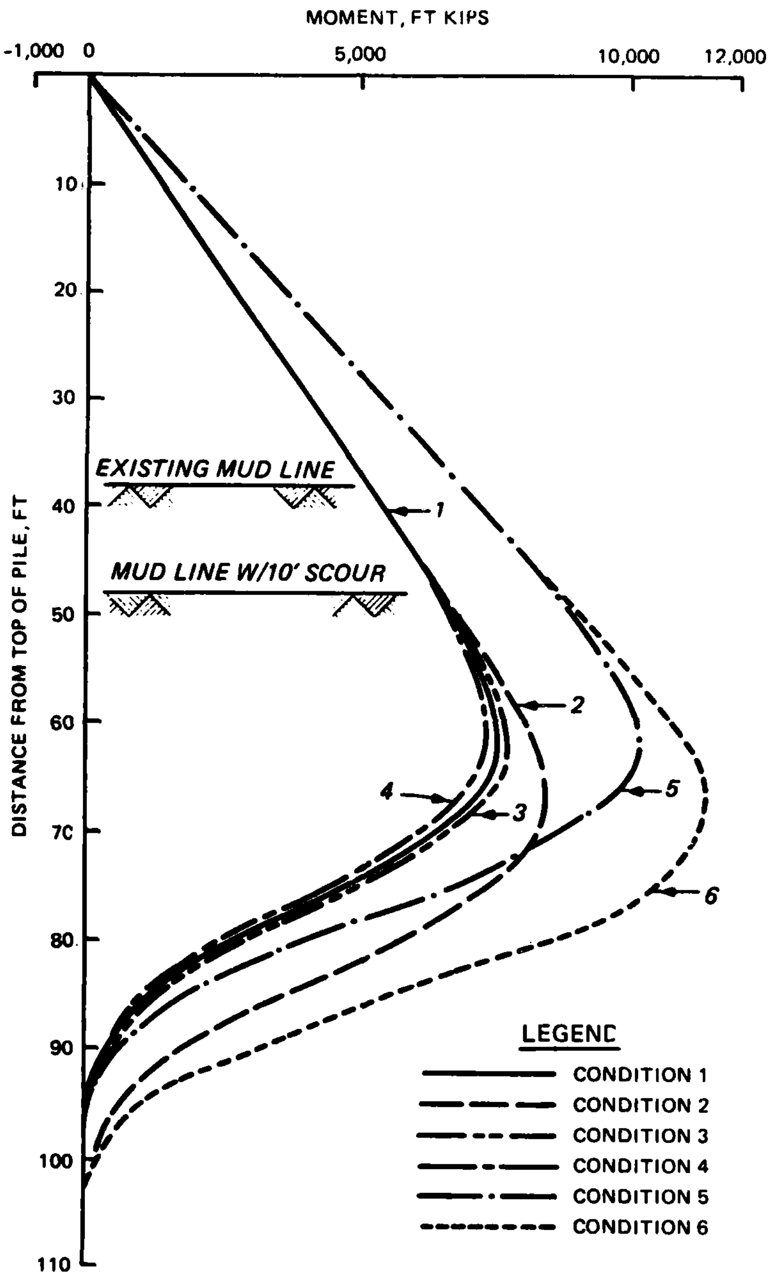


Figure B5. Plot of moment versus depth

APPENDIX C: INPUT GUIDE FOR COM624G

Introduction

1. COM624G is a computer program that facilitates analysis of laterally loaded piles for various boundary conditions. The program was originally written by Prof. L. C. Reese and W. R. Sullivan at The University of Texas at Austin and was labelled COM624 (Reese and Sullivan 1980).* In the COM624G version of the program, the input format was changed, a conversational mode for inputting data loads added, and graphical options were provided for plotting both input and output data. The program was also double-precisioned for use on the Honeywell DPS-1 computer. These modifications were programmed by Messrs. Michael Pace and Reed L. Mosher of the Automatic Data Processing Center, U. S. Army Engineer Waterways Experiment Station (WES).

2. Complete documentation of COM624 is provided in Reese and Sullivan (1980), and the reader should refer to this source for detailed information on the program. This appendix provides an input guide only to COM624G. The order of the input data by major groups (identified by a keyword) is immaterial, although input within each major group should be together in sequential order. All major groups are not required for problem solution, and within each group some data are optional. The optional data are indicated by inclosing them in parentheses.

3. Example problems are included at the end of the input guide. These problems are the same as those used in Reese and Sullivan (1980) for COM624 and are included so that verification is possible.

Accessing the Program

4. To run COM624G on the WES or Office of Personnel Management, Macon, Ga., computer systems, sign on to the particular system. Then

```
* FORT
* OLD WESLIB/CORPS/I0012,R
* GCS2D
* device - TK4 (4014)
           ALP (Alphanumeric Terminal)
```

---

* References cited in this appendix are included in the References at the end of the main text.

Cybernet System

5. /OLD,CORPS/UN = CECELB

   /CALL,CORPS,I0012

Input Guide for COM624G

Keyword [Line Number] (Optional Information)

I. Title

TITLE One line for identifying the individual problem in a computer run. It may be any alphanumeric information up to 72 characters including the line number and embedded blanks.

[LN] TITLE

[LN] Any alphanumeric information up to 72 characters.

II. System Units

UNITS One line identifying the units to be used in the program. This information is only used to insure proper unit identification on output (i.e., no conversions are made in the program).

[LN] UNITS

[LN] ISYSTM (IDUM1 IDUM2 IDUM3)

ISYSTM = ENGL - for English units (L=inches, F=lbs.)

= METR - for metric units or any other system

(IDUM1 IDUM2 IDUM3) = Alphanumeric information describing the system of units selected. (i.e., feet and kips, cm and grams, etc.)

III. Pile Descriptions

PILE Two to eleven lines that describe the pile geometry and properties.

[LN] PILE NI NDIAM LENGTH EPILE XGS

[LN] XDIAM(I) DIAM(I) MINER(I) (AREA(I))
(I = 1, NDIAM)

1st Group

NI = Number of increments into which pile is divided

NDIAM = Number of segments of pile with different diameters

LENGTH = Length of pile

EPILE = Modulus of elasticity

XGS = Depth below top of pile to ground surface

2nd Group

XDIAM = Depth below top of pile

DIAM = Diameter of pile at XDIAM

MINERT = Moment of inertia at XDIAM

(AREA) = Cross-sectional area of pile (L^2) (If left blank, computed assuming a pipe section)

IV. Soil Description

<u>SOIL</u> Two to ten lines that describe soil system and its properties.

[LN] SOIL NL

[LN] LAYER(I) KSOIL(I) XTOP(I) XBOT(I) K(I) (AE(I) FR(I)) (I = 1, NL)

1st Group

NL = Number of layers of soil.

2nd Group

LAYER(I) = Layer number

KSOIL(I) = Code to control the type of p-y curves

= 1 to have p-y curves computed internally using Matlock's (1970) criteria for soft clay

= 2 to have p-y curves computed internally using Reese's and Welch's (1975) criteria for stiff clay below the water table

= 3 to have p-y curves computed internally using Reese's and Welch's (1975) criteria for stiff clay above the water table

= 4 to have p-y curves computed internally using Reese et al. (1974) criteria for sand

= 5 to use linear interpolation between input p-y curves

= 6 to have p-y curves computed internally using Sullivan et al. (1979) unified clay criteria

XTOP(I) = X-coordinate of top of layer

XBOT(I) = X-coordinate of bottom of layer

K(I) = Constant (F/L^3) in equation $E_s = Kx$. This is used to define initial soil moduli for the first iteration and to determine initial slope of p-y curve where KSOIL = 2, 4, or 6

(AE(I)) = Factor "A" in uniform clay criteria

(FR(I)) = Factor "F" in uniform clay criteria. (Leave blank unless KSOIL(I) = 6)

V. Unit Weight Profile (Optional)

WEIGHT One to eleven lines that describe the effective unit weights of soil in the soil profile.

[LN] WEIGHT NGI

[LN] XGI(I) GAM1(I)
I = 1, NG1

1st Group

NGI = Number of points on plot of effective unit weight versus depth

2nd Group

XG1(I) = X-coordinate below top of pile to point where effective unit weight of soil is specified

GAM1(I) = Effective unit weight of soil corresponding to XG1

VI. Soil Strength Profile (Optional)

Strength Two to eleven lines that describe the variation in strength properties of soil with depth.

[LN] STRENGTH NSTR

[LN] XSTR(I) C1(I) PHI1(I) EE50(I)
(I = 1, NSTR)

1st Group

NSTR = Number of points on input curve of strength versus depth

2nd Group

XSTR(I) = X-Coordinate below top of pile for which C, 0, and e_{50} are specified

C1(I) = Undrained shear strength of soil corresponding to XSTR(I)

PHI1(I) = Angle of internal friction in degrees corresponding to XSTR(I)

EE50(I) = Strain at 50 percent stress level corresponding to XSTR(I)

VII. Input for p-y Curves (Optional)

[LN] PY Up to 930 lines that define the p-y curves for soil response to lateral load.

[LN] PY NPY NPPY

[LN] XPY(I)

[LN] YP(I,J) PP(I,J)
(I = 1, NPY; J = 1, NPPY)

1st Group

NPY = Number of p-y curves (maximum 30)

NPPY = Number of points on p-y curves (maximum 30)

2nd Group

XPY(I) = X-distance from top of pile to input p-y curve

3rd Group (Defines the p-y curve at distance = XPY(I).)

YP(I,J) = Deflection of a point on a p-y curve

PP(I,J) = Soil resistance corresponding to YP

VIII. Boundary Conditions at the Pile Head

BOUNDARY Specifies the boundary condition at the pile head

[LN] BOUNDARY KBC NRUN

[LN] KOPSUB(I) PTSUB(I) BC2SUB(I) PXSUB(I) (I = 1, NRUN)

1st Group

KBC = Code to control boundary condition at top of pile

= 1 for free head (user specified lateral load and moment)

= 2 for specified lateral load and slope at pile head. (Slope is 0 for fixed-head pile)

= 3 for a specified lateral load and rotational restraint at the pile head

NRUN = Number of sets of boundary conditions (load cases)

2nd Group

KOPSUB(I) = Pile head printout code

= 0 if only the pile head deflection and slope, maximum bending moment, and maximum combined stress are to be printed for the associated loads

= 1 if complete output is desired for the associated loads

PTSUB(I) = Lateral load at top of pile

BC2SUB(I) = Value of second boundary condition

= Moment (if KBC = 1)

= Slope (if KBC = 2)

= Rotational stiffness (if KBC = 3)

PXSUB(I) = Axial load on pile (assumed to be uniform over whole length of pile)

IX. Distributed Lateral Load on Pile (Optional)

LOAD Describes a distributed lateral load applied to the pile.

[LN] LOAD NLD NW(J)

[LN] XW(J,I) WW(J,I)
(I = 1, NW); (J = 1, NRUN)

NLD = Load case number

NW = Number of points on plot of distributed lateral load on pile versus depth for specified NLD

XW(I) = X-coordinate where distributed loads are specified

WW(I) = Distributed lateral load

X. For Cyclic Load (Optional)

CYCLIC Specifies if the loading is cyclic or static.

[LN] CYCLIC KCYCL RCYCL

KCYCL = 0 for cyclic loading

= 1 for static loading

RCYCL = Number of cycles of loading (need only for p-y curves generated criteria for stiff clay above the water table)

XI. Control of output

OUTPUT Describes the amount of output to be printed.

[LN] OUTPUT KOUTPT INC KPYOP NNSUB

[LN] XNSUB(I) ... XNSUB(NNSUB)

KOUTPT = 0 if data are to be printed only to depth where moment first changes sign

= 1 if data are to be printed for full length of pile

= 2 for extra output to help with debugging

INC = Increment used in printing output

= 1 to print values at every node

= 2 to print values at every second node

= 3 to print values at every third node, etc. (up to NI + 1)

KPYOP = 0 if no p-y curves are to be generated and printed for verification purposes

= 1 if p-y curves are to be generated and printed for verification

NNSUB = Number of depths for which internally generated p-y curves are to be printed (maximum 305)

2nd Group

XNSUB(I) = X-coordinate at which internally generated p-y curves are to be generated for printing

XII. Program Control

CONTROL Specified maximum number of interactions and tolerance of solution convergence maximum deflections.

[LN] CONTROL MAXIT YTOL EXDEFL

MAXIT = Maximum number of iterations for analysis of load case

YTOL = Tolerance on solution convergence

EXDEFL = Value of deflection of pile head that is considered grossly excessive and which stops the run. Default to pile diameter

XIII. Termination of Input Sequence

END Terminates the input sequence and initiates the analysis.

[LN] END

Example Problems

6. Pile properties and the soil profile to be used in all four problems are shown in Figure C1.

Example problem 1

7. A free-head pile will be analyzed for lateral loads of 5,000, 10,000, 15,000, and 20,000 lb. An axial load of 100,000 lb will be used, and no moment will be applied at the pile head. The p-y curves shown in Figure C1 will be used in this analysis.

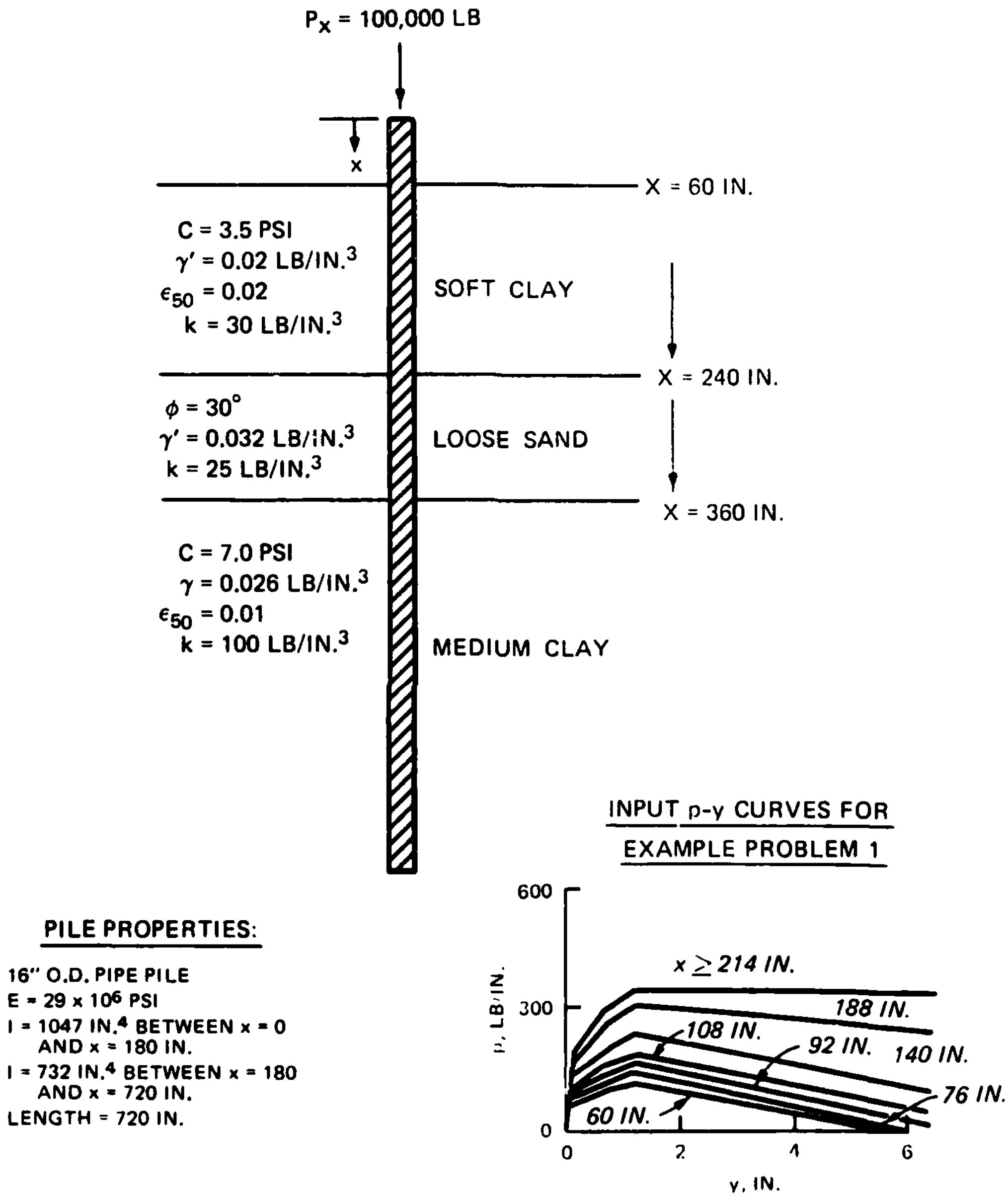


Figure C1. Pile and soil description

```
10 TITLE
20 EX. PRO. 1 FROM DOCUMENTATION OF COM. PRO. COM624 BY L.C. REESE, 1980.
30 UNITS
40 ENGL
50 PILE 120 2 720 29.E6 60  (Pile Properties - NI,NDIAM,LENGTH,EPILE,XGS)
60 0 16 1047                (XDIAM(I),DIAN(I),MINERT(I)
70 180 16 732                  where I = 1,NDIAM
80 SOIL 3                   (Soil Description - NL)
90 1 5  60 240  30           LAYER(I),KSOIL(I),XTOP(I),XBOT(I),K(I)
100 2 5 240 360  25             where I = 1,NL
110 3 5 360 800 100
120 PY 7 6                  (Input P-Y Curves - NPY,NPPY)
130 60                       XPY(I)
140 0.0   0.0                YP(I,J),   PP(I,J)
150 0.2  66.1                  .          .          where I = 1,NPY
160 0.4  83.2                  .          .                J = 1,NPPY
170 0.8 105.0                  .          .
180 1.2 120.0                  .          .
190 6.0   0.0                YP(I,NPPY),PP(I,NPPY)
200 76
210 0.0   0.0
220 0.2  79.8
230 0.4 100.0
240 0.8 127.0
250 1.2 145.0
260 6.0  15.0
270 92
280 0.0   0.0
290 0.2  93.3
300 0.4 117.0
310 0.8 148.0
320 1.2 169.0
330 6.0  34.0
340 108
350 0.0   0.0
360 0.2 107.0
370 0.4 135.0
380 0.8 170.0
390 1.2 194.0
400 6.0  61.0
410 140
420 0.0   0.0
430 0.2 134.0
440 0.4 169.0
450 0.8 213.0
460 1.2 243.0
470 6.0 123.0
480 188
490 0.0   0.0
500 0.2 175.0
510 0.4 221.0
520 0.8 278.0
530 1.2 318.0
540 6.0 264.0
550 214
560 0.0   0.0
```

```
570 0.2 198.0
580 0.4 250.0
590 0.8 315.0
600 1.2 360.0
610 6.0 360.0
620 OUTPUT 1 2 0 0              (Output Control - KOUTPT,INC,KPYOP,NNSUB)
630 BOUNDARY 1 4                (Boundary Conditions at Pile Head -KBC,NRUN)
640 1  5.E3 0.0 1.E5
650 1 10.E3 0.0 1.E5            (KOPSUB(I),PTSUB(I),BC2SUB(I),PXSUB(I)
660 1 15.E3 0.0 1.E5
670 1 20.E3 0.0 1.E5                     where I = 1,NRUN
680 CONTROL 100 .001 24         (Program Control - MAXIT,YTOL,EXDEFL)
690 END
```

*

(Input Echo)

***** UNIT DATA. *****

SYSTEM OF UNITS
(UP TO 16 CHAR.)
ENGL

***** PILE DATA. *****

NO. INCREMENTS PILE IS DIVIDED	NO. SEGMENTS WITH DIFFERENT CHARACTERISTICS	LENGTH OF PILE	MODULUS OF ELASTICITY	DEPTH
120	2	0.720E 03	0.290E 08	0.600E 02

TOP OF SEGMENT	DIAMETER OF PILE	MOMENT OF INERTIA	CROSS-SECT. AREA
0.	0.160E 02	0.105E 04	0.359E 02
0.180E 03	0.160E 02	0.732E 03	0.243E 02

***** SOIL DATA. *****

NUMBER OF LAYERS
3

LAYER NUMBER	P-Y CURVE CONTROL CODE	TOP OF LAYER	BOTTOM OF LAYER	INITIAL SOIL MODULI CONST.	FACTOR "A"	FACTOR "F"
1	5	0.600E 02	0.240E 03	0.300E 02	0.	0.
2	5	0.240E 03	0.360E 03	0.250E 02	0.	0.
3	5	0.360E 03	0.800E 03	0.100E 03	0.	0.

***** UNIT WEIGHT DATA. *****

NO. POINTS FOR PLOT
OF EFF. UNIT WEIGHT
VS. DEPTH
0

***** PROFILE DATA. *****

(p-y Data)

NO. POINTS FOR
STRENGTH PARAMETERS
VS. DEPTH
0

***** P-Y DATA. *****

NO. OF
P-Y CURVES
7

NO. POINTS ON
P-Y CURVES
6

X-COORD. TO
INPUT P-Y CURVE
0.600E 02

DEFLECTION	SOIL RESISTANCE
0.	0.
0.200E 00	0.661E 02
0.400E 00	0.832E 02
0.800E 00	0.105E 03
0.120E 01	0.120E 03
0.600E 01	0.

X-COORD. TO
INPUT P-Y CURVE
0.760E 02

DEFLECTION	SOIL RESISTANCE
0.	0.
0.200E 00	0.798E 02
0.400E 00	0.100E 03
0.800E 00	0.127E 03
0.120E 01	0.145E 03
0.600E 01	0.150E 02

X-COORD. TO
INPUT P-Y CURVE
0.920E 02

DEFLECTION	SOIL RESISTANCE
0.	0.
0.200E 00	0.933E 02
0.400E 00	0.117E 03
0.800E 00	0.148E 03
0.120E 01	0.169E 03
0.600E 01	0.340E 02

X-COORD. TO

INPUT P-Y CURVE
0.108E 03

DEFLECTION	SOIL RESISTANCE
0.	0.
0.200E 00	0.107E 03
0.400E 00	0.135E 03
0.800E 00	0.170E 03
0.120E 01	0.194E 03
0.600E 01	0.610E 02

X-COORD. TO
INPUT P-Y CURVE
0.140E 03

DEFLECTION	SOIL RESISTANCE
0.	0.
0.200E 00	0.134E 03
0.400E 00	0.169E 03
0.800E 00	0.213E 03
0.120E 01	0.243E 03
0.600E 01	0.123E 03

X-COORD. TO
INPUT P-Y CURVE
0.188E 03

DEFLECTION	SOIL RESISTANCE
0.	0.
0.200E 00	0.175E 03
0.400E 00	0.221E 03
0.800E 00	0.278E 03
0.120E 01	0.318E 03
0.600E 01	0.264E 03

X-COORD. TO
INPUT P-Y CURVE
0.214E 03

DEFLECTION	SOIL RESISTANCE
0.	0.
0.200E 00	0.198E 03
0.400E 00	0.250E 03
0.800E 00	0.315E 03
0.120E 01	0.360E 03
0.600E 01	0.360E 03

***** OUTPUT DATA. *****

DATA OUTPUT	OUTPUT INCREMENT	P-Y PRINTOUT	NO. DEPTHS TO PRINT FOR

CODE 1	CODE 2	CODE 0	P-Y CURVES 0

DEPTH FOR
PRINTING
P-Y CURVES
0.

***** PILE HEAD (BOUNDARY) DATA. *****

BOUNDARY CONDITION CODE	NO. OF SETS OF BOUNDARY CONDITIONS
1	4

PILE HEAD PRINTOUT CODE	LATERAL LOAD AT TOP OF PILE	VALUE OF SECOND BOUNDARY CONDITION	AXIAL LOAD ON PILE
1	0.500E 04	0.	0.100E 06
1	0.100E 05	0.	0.100E 06
1	0.150E 05	0.	0.100E 06
1	0.200E 05	0.	0.100E 06

***** CYCLIC DATA. *****

CYCLIC(0) OR STATIC(1) LOADING	NO. CYCLES OF LOADING
0	0.

***** PROGRAM CONTROL DATA. *****

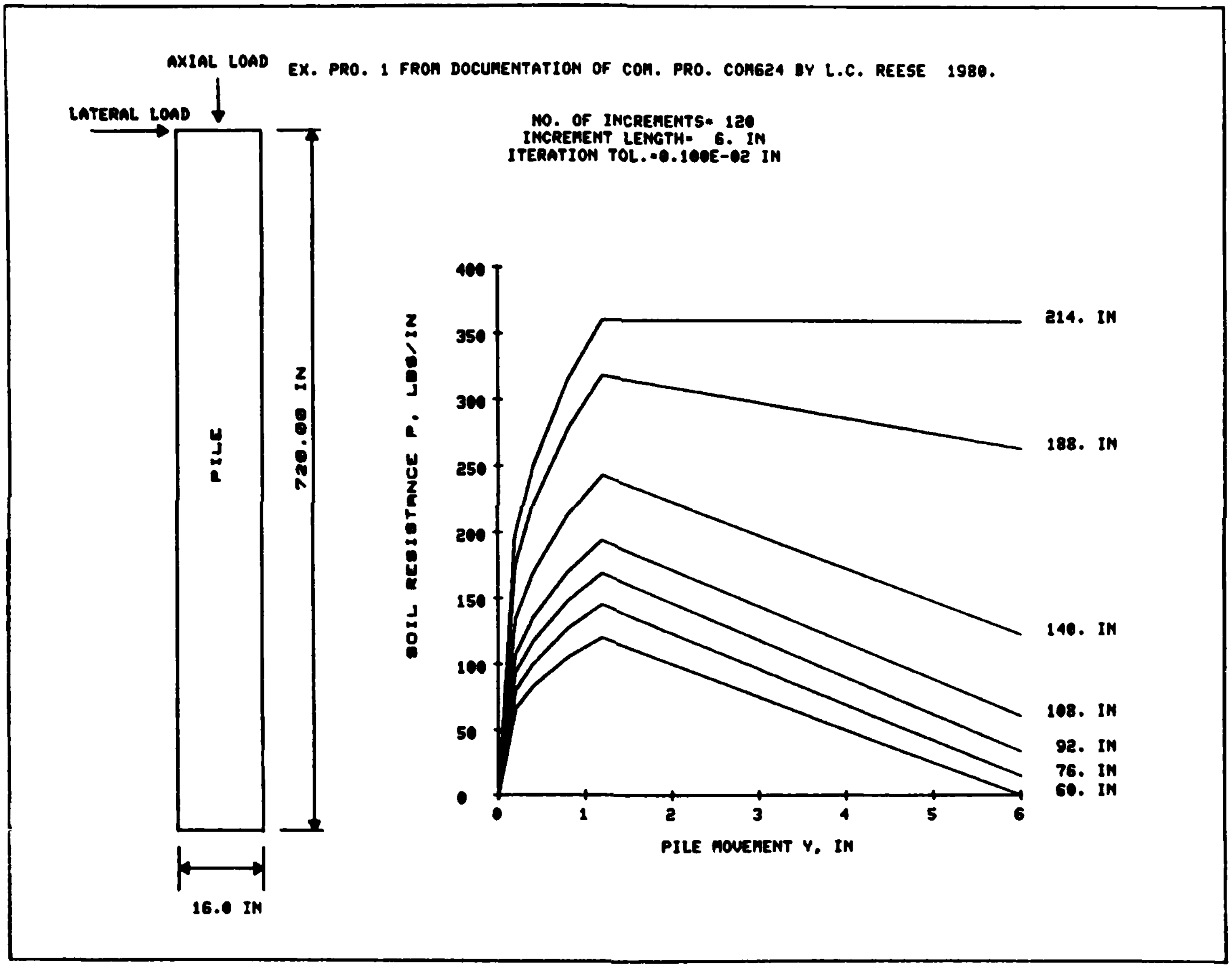
MAX. NO. OF ITERATIONS	TOLERENCE ON SOLUTION CONVERGENCE	PILE HEAD DEFLECTION FLAG(STOPS RUN)
100	0.100E-02	0.240E 02

***** LOAD DATA. *****

BOUNDARY SET NO.	NO. POINTS FOR DISTRIB. LATERAL LOAD VS. DEPTH
1	0

BOUNDARY SET NO.   NO. POINTS FOR DISTRIB. LATERAL

	LOAD VS. DEPTH
2	0
BOUNDARY SET NO.	NO. POINTS FOR DISTRIB. LATERAL LOAD VS. DEPTH
3	0
BOUNDARY SET NO.	NO. POINTS FOR DISTRIB. LATERAL LOAD VS. DEPTH
4	0


AXIAL LOAD
EX. PRO. 1 FROM DOCUMENTATION OF COM. PRO. COM624 BY L.C. REESE 1980.
LATERAL LOAD
NO. OF INCREMENTS= 120
INCREMENT LENGTH= 6. IN
ITERATION TOL.=0.100E-02 IN
PILE
720.00 IN
16.0 IN
SOIL RESISTANCE P, LBS/IN
400
350
300
250
200
150
100
50
0
1
2
3
4
5
6
PILE MOVEMENT Y, IN
214. IN
188. IN
140. IN
108. IN
92. IN
76. IN
60. IN

EX. PRO. 1 FROM DOCUMENTATION OF COM. PRO. COM624 BY L.C. REESE 1980.
LOADING CONDITIONS

LOAD CASE NO.	LATERAL LOAD AT PILE HEAD(LBS)	AXIAL LOAD AT PILE HEAD(LBS)	APPLIED MOMENT AT PILE HEAD(LBS-IN)
1	5000.	100000.	0.
2	10000.	100000.	0.
3	15000.	100000.	0.
4	20000.	100000.	0.

EX. PRO. 1 FROM DOCUMENTATION OF COM. PRO. COM624 BY L.C. REESE, 19
80.

UNITS--ENGL

O U T P U T   I N F O R M A T I O N
****

(Load Case 1)

NO. OF ITERATIONS = 5
MAXIMUM DEFLECTION ERROR = 0.409E-03 IN

PILE LOADING CONDITION
LATERAL LOAD AT PILE HEAD = 0.500E 04 LBS
APPLIED MOMENT AT PILE HEAD = 0. LBS-IN
AXIAL LOAD AT PILE HEAD = 0.100E 06 LBS

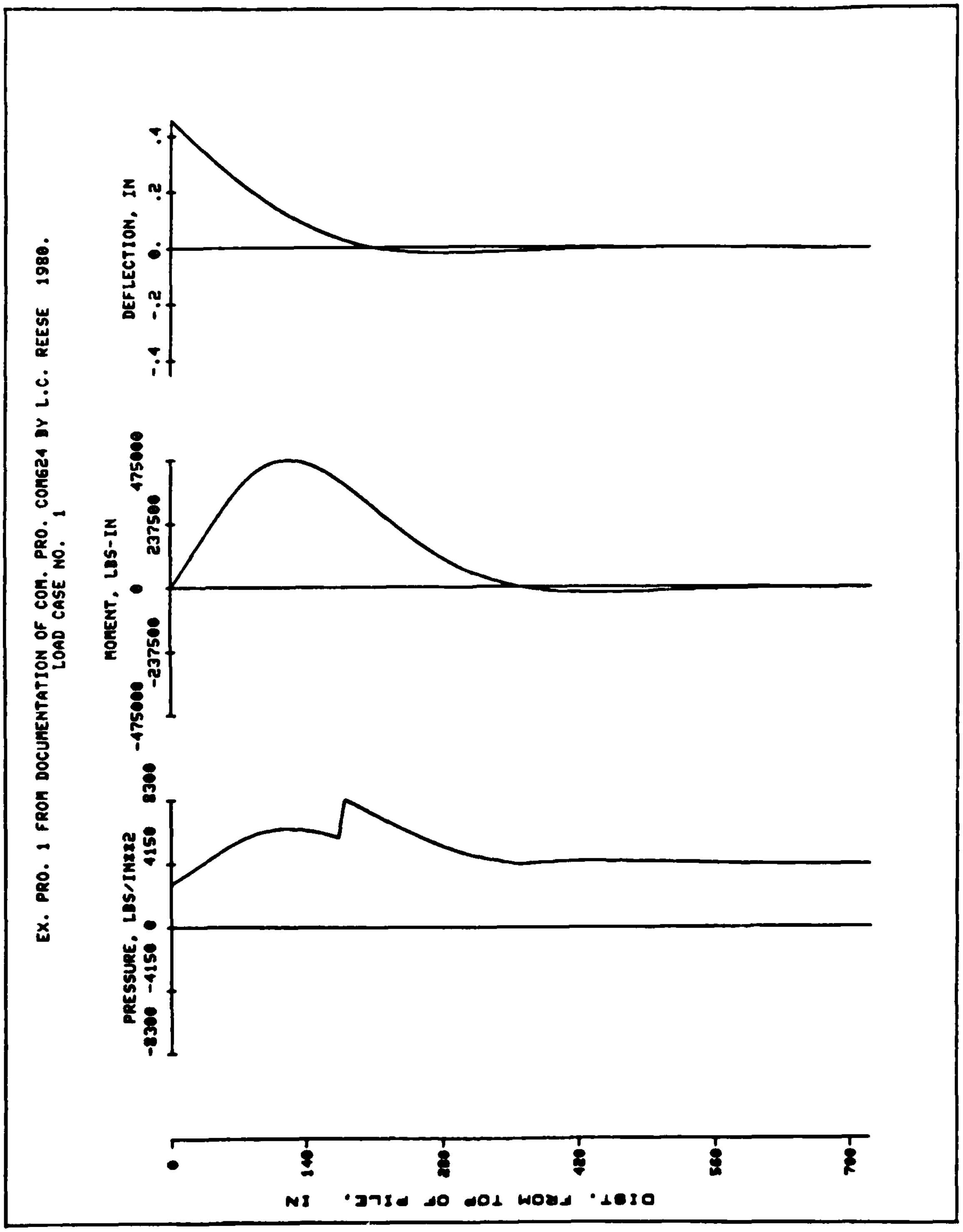
X IN	DEFLEC IN	MOMENT LBS-IN	TOTAL STRESS LBS/IN**2	DISTR. LOAD LBS/IN	SOIL MODULUS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2
*******	*********	*********	*********	*********	*********	*********
0.	0.452E 00	0.	0.278E 04	0.	0.	0.304E 11
12.00	0.414E 00	0.638E 05	0.327E 04	0.	0.	0.304E 11
24.00	0.376E 00	0.128E 06	0.376E 04	0.	0.	0.304E 11
36.00	0.339E 00	0.191E 06	0.424E 04	0.	0.	0.304E 11
48.00	0.303E 00	0.255E 06	0.473E 04	0.	0.	0.304E 11
60.00	0.268E 00	0.318E 06	0.522E 04	0.	0.269E 03	0.304E 11
72.00	0.235E 00	0.374E 06	0.564E 04	0.	0.340E 03	0.304E 11
84.00	0.203E 00	0.418E 06	0.597E 04	0.	0.429E 03	0.304E 11
636.00	0.794E-03	-0.135E 04	0.412E 04	0.	0.990E 03	0.212E 11
648.00	0.712E-03	-0.921E 03	0.412E 04	0.	0.990E 03	0.212E 11
660.00	0.623E-03	-0.591E 03	0.411E 04	0.	0.990E 03	0.212E 11
672.00	0.530E-03	-0.349E 03	0.411E 04	0.	0.990E 03	0.212E 11
684.00	0.435E-03	-0.183E 03	0.411E 04	0.	0.990E 03	0.212E 11
696.00	0.339E-03	-0.780E 02	0.411E 04	0.	0.990E 03	0.212E 11
708.00	0.242E-03	-0.218E 02	0.411E 04	0.	0.990E 03	0.212E 11
720.00	0.145E-03	0.	0.411E 04	0.	0.990E 03	0.212E 11

OUTPUT VERIFICATION

```
THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT      = -0.296E-02 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT =  0.383E-03 LBS

COMPUTED LATERAL FORCE AT PILE HEAD            =  0.50000E 04 LBS
    COMPUTED MOMENT AT PILE HEAD               =  0.           IN-LBS
    COMPUTED SLOPE AT PILE HEAD                = -0.31710E-02

THE OVERALL MOMENT IMBALANCE              =  0.193E-03 IN-LBS
THE OVERALL LATERAL FORCE IMBALANCE       = -0.388E-09 LBS
```

OUTPUT SUMMARY

```
PILE HEAD DEFLECTION    =  0.452E 00 IN
MAXIMUM BENDING MOMENT  =  0.475E 06 IN-LBS
MAXIMUM TOTAL STRESS    =  0.831E 04 LBS/IN**2
MAXIMUM SHEAR FORCE     =  0.532E 04 LBS
```


EX. PRO. 1 FROM DOCUMENTATION OF COM. PRO. COM624 BY L.C. REESE 1980.
LOAD CASE NO. 1
DEFLECTION, IN
-.4
-.2
0.
.2
.4
MOMENT, LBS-IN
-475000
-237500
0
237500
475000
PRESSURE, LBS/IN**2
-8300
-4150
0
4150
8300
DIST. FROM TOP OF PILE, IN
0
140
280
420
560
700

(Load Case 2)

NO. OF ITERATIONS = 8
MAXIMUM DEFLECTION ERROR = 0.921E-03 IN

PILE LOADING CONDITION
LATERAL LOAD AT PILE HEAD = 0.100E 05 LBS
APPLIED MOMENT AT PILE HEAD = 0. LBS-IN
AXIAL LOAD AT PILE HEAD = 0.100E 06 LBS

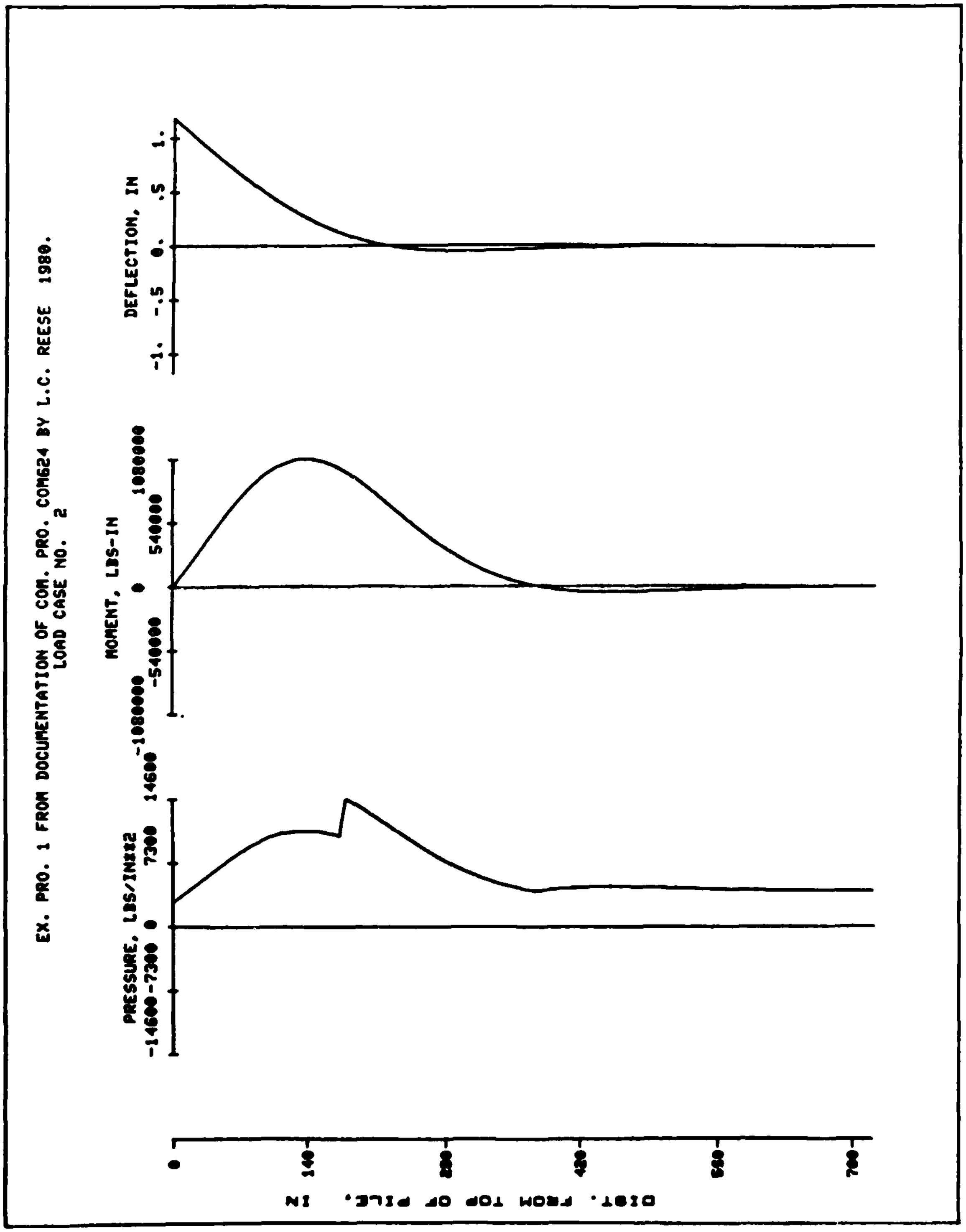
X	DEFLEC	MOMENT	TOTAL STRESS	DISTR. LOAD	SOIL MODULUS	FLEXURAL RIGIDITY
IN	IN	LBS-IN	LBS/IN**2	LBS/IN	LBS/IN**2	LBS-IN**2
*******	*********	*********	*********	*********	*********	*********
0.	0.118E 01	0.	0.278E 04	0.	0.	0.304E 11
12.00	0.109E 01	0.129E 06	0.377E 04	0.	0.	0.304E 11
24.00	0.995E 00	0.258E 06	0.476E 04	0.	0.	0.304E 11
36.00	0.904E 00	0.387E 06	0.574E 04	0.	0.	0.304E 11
48.00	0.816E 00	0.516E 06	0.673E 04	0.	0.	0.304E 11
60.00	0.730E 00	0.645E 06	0.771E 04	0.	0.139E 03	0.304E 11
72.00	0.646E 00	0.762E 06	0.861E 04	0.	0.173E 03	0.304E 11
84.00	0.567E 00	0.863E 06	0.938E 04	0.	0.213E 03	0.304E 11
636.00	0.205E-02	-0.432E 04	0.415E 04	0.	0.990E 03	0.212E 11
648.00	0.190E-02	-0.302E 04	0.414E 04	0.	0.990E 03	0.212E 11
660.00	0.172E-02	-0.200E 04	0.413E 04	0.	0.990E 03	0.212E 11
672.00	0.154E-02	-0.122E 04	0.412E 04	0.	0.990E 03	0.212E 11
684.00	0.134E-02	-0.657E 03	0.411E 04	0.	0.990E 03	0.212E 11
696.00	0.114E-02	-0.286E 03	0.411E 04	0.	0.990E 03	0.212E 11
708.00	0.936E-03	-0.762E 02	0.411E 04	0.	0.990E 03	0.212E 11
720.00	0.732E-03	0.	0.411E 04	0.	0.990E 03	0.212E 11

OUTPUT VERIFICATION

```
THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT      = -0.984E-02 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT =  0.108E-02 LBS

COMPUTED LATERAL FORCE AT PILE HEAD             =  0.10000E 05 LBS
    COMPUTED MOMENT AT PILE HEAD                =  0.          IN-LBS
    COMPUTED SLOPE AT PILE HEAD                 = -0.76937E-02

THE OVERALL MOMENT IMBALANCE                =  0.102E-02 IN-LBS
THE OVERALL LATERAL FORCE IMBALANCE         = -0.135E-08 LBS
```

OUTPUT SUMMARY

```
PILE HEAD DEFLECTION    =  0.118E 01 IN
MAXIMUM BENDING MOMENT =  0.108E 07 IN-LBS
MAXIMUM TOTAL STRESS    =  0.146E 05 LBS/IN**2
MAXIMUM SHEAR FORCE     =  0.108E 05 LBS
```


EX. PRO. 1 FROM DOCUMENTATION OF COM. PRO. COM624 BY L.C. REESE 1980.
LOAD CASE NO. 2
DEFLECTION, IN
-1.
-.5
0.
.5
1.
MOMENT, LBS-IN
-1080000
-540000
0
540000
1080000
PRESSURE, LBS/IN**2
-14600
-7300
0
7300
14600
0
140
280
420
560
700
DIST. FROM TOP OF PILE, IN

(Load Case 3)

NO. OF ITERATIONS = 11
MAXIMUM DEFLECTION ERROR = 0.968E-03 IN

PILE LOADING CONDITION
LATERAL LOAD AT PILE HEAD = 0.150E 05 LBS
APPLIED MOMENT AT PILE HEAD = 0. LBS-IN
AXIAL LOAD AT PILE HEAD = 0.100E 06 LBS

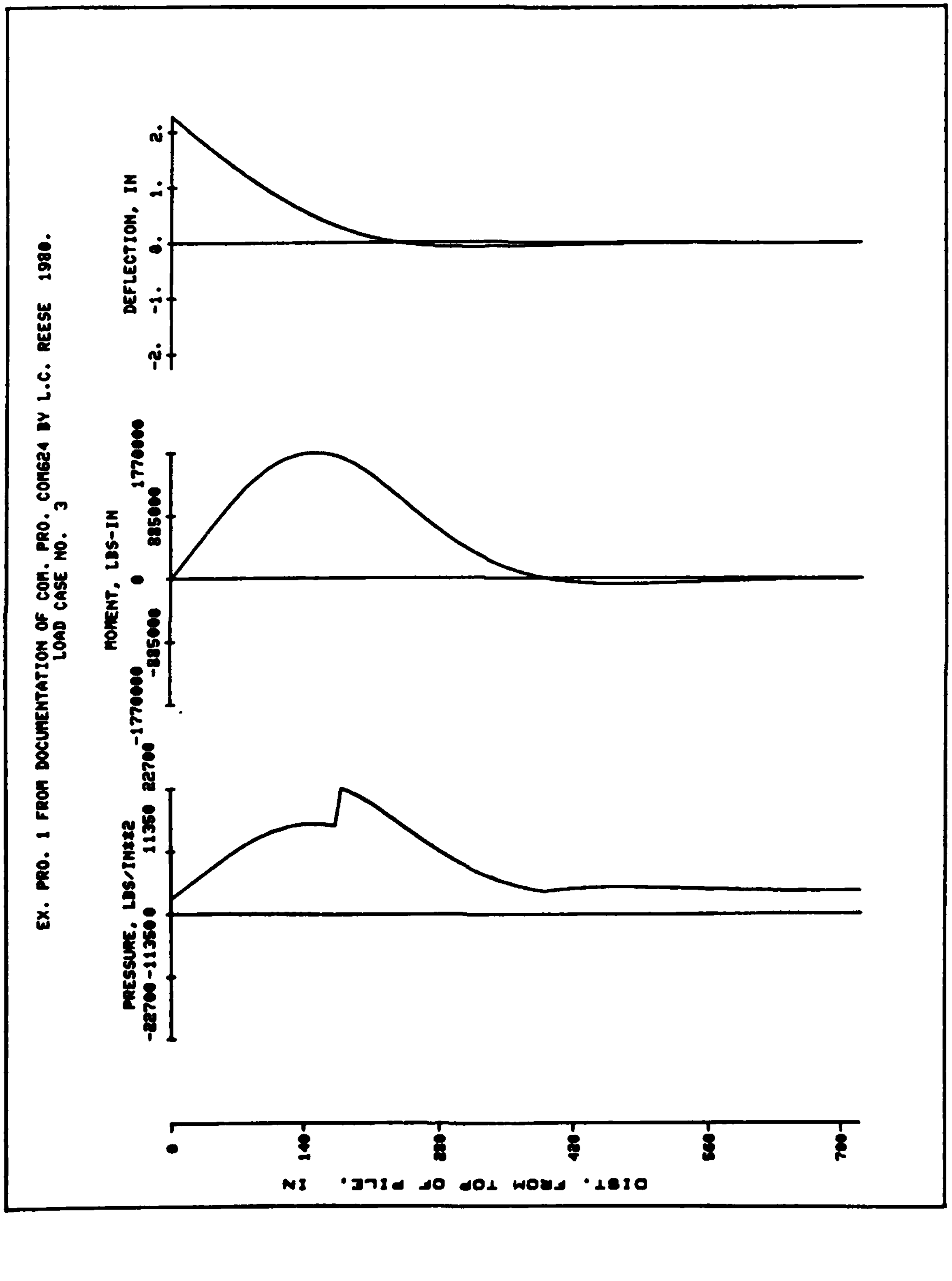
X	DEFLEC	MOMENT	TOTAL STRESS	DISTR. LOAD	SOIL MODULUS	FLEXURAL RIGIDITY
IN	IN	LBS-IN	LBS/IN**2	LBS/IN	LBS/IN**2	LBS-IN**2
*******	*********	*********	*********	*********	*********	*********
0.	0.226E 01	0.	0.278E 04	0.	0.	0.304E 11
12.00	0.210E 01	0.196E 06	0.428E 04	0.	0.	0.304E 11
24.00	0.193E 01	0.393E 06	0.578E 04	0.	0.	0.304E 11
36.00	0.177E 01	0.589E 06	0.728E 04	0.	0.	0.304E 11
48.00	0.161E 01	0.785E 06	0.878E 04	0.	0.	0.304E 11
60.00	0.146E 01	0.980E 06	0.103E 05	0.	0.781E 02	0.304E 11
72.00	0.131E 01	0.116E 07	0.117E 05	0.	0.104E 03	0.304E 11
84.00	0.116E 01	0.133E 07	0.129E 05	0.	0.134E 03	0.304E 11
600.00	0.368E-02	-0.217E 05	0.434E 04	0.	0.990E 03	0.212E 11
612.00	0.382E-02	-0.173E 05	0.430E 04	0.	0.990E 03	0.212E 11
624.00	0.384E-02	-0.134E 05	0.425E 04	0.	0.990E 03	0.212E 11
636.00	0.378E-02	-0.100E 05	0.422E 04	0.	0.990E 03	0.212E 11
648.00	0.364E-02	-0.717E 04	0.419E 04	0.	0.990E 03	0.212E 11
660.00	0.346E-02	-0.486E 04	0.416E 04	0.	0.990E 03	0.212E 11
672.00	0.324E-02	-0.304E 04	0.414E 04	0.	0.990E 03	0.212E 11
684.00	0.300E-02	-0.167E 04	0.413E 04	0.	0.990E 03	0.212E 11
696.00	0.275E-02	-0.736E 03	0.411E 04	0.	0.990E 03	0.212E 11
708.00	0.250E-02	-0.190E 03	0.411E 04	0.	0.990E 03	0.212E 11
720.00	0.224E-02	0.	0.411E 04	0.	0.990E 03	0.212E 11

OUTPUT VERIFICATION

```
THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT     =  0.120E-01 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = -0.167E-02 LBS

COMPUTED LATERAL FORCE AT PILE HEAD           =  0.15000E 05 LBS
    COMPUTED MOMENT AT PILE HEAD              =  0.           IN-LBS
    COMPUTED SLOPE AT PILE HEAD               = -0.13733E-01

THE OVERALL MOMENT IMBALANCE              = -0.443E-02 IN-LBS
THE OVERALL LATERAL FORCE IMBALANCE       = -0.223E-08 LBS
```

OUTPUT SUMMARY

```
PILE HEAD DEFLECTION    =  0.226E 01 IN
MAXIMUM BENDING MOMENT =  0.177E 07 IN-LBS
MAXIMUM TOTAL STRESS    =  0.227E 05 LBS/IN**2
MAXIMUM SHEAR FORCE     =  0.164E 05 LBS
```



EX. PRO. 1 FROM DOCUMENTATION OF COM. PRO. COM624 BY L.C. REESE 1980.
LOAD CASE NO. 3
DEFLECTION, IN
-2. -1. 0. 1. 2.
MOMENT, LBS-IN
-1770000 -885000 0 885000 1770000
PRESSURE, LBS/IN**2
-22700 -11350 0 11350 22700
0 140 280 420 560 700
DIST. FROM TOP OF PILE, IN

(Load Case 4)

NO. OF ITERATIONS = 25
MAXIMUM DEFLECTION ERROR = 0.818E-03 IN

PILE LOADING CONDITION
LATERAL LOAD AT PILE HEAD = 0.200E 05 LBS
APPLIED MOMENT AT PILE HEAD = 0. LBS-IN
AXIAL LOAD AT PILE HEAD = 0.100E 06 LBS

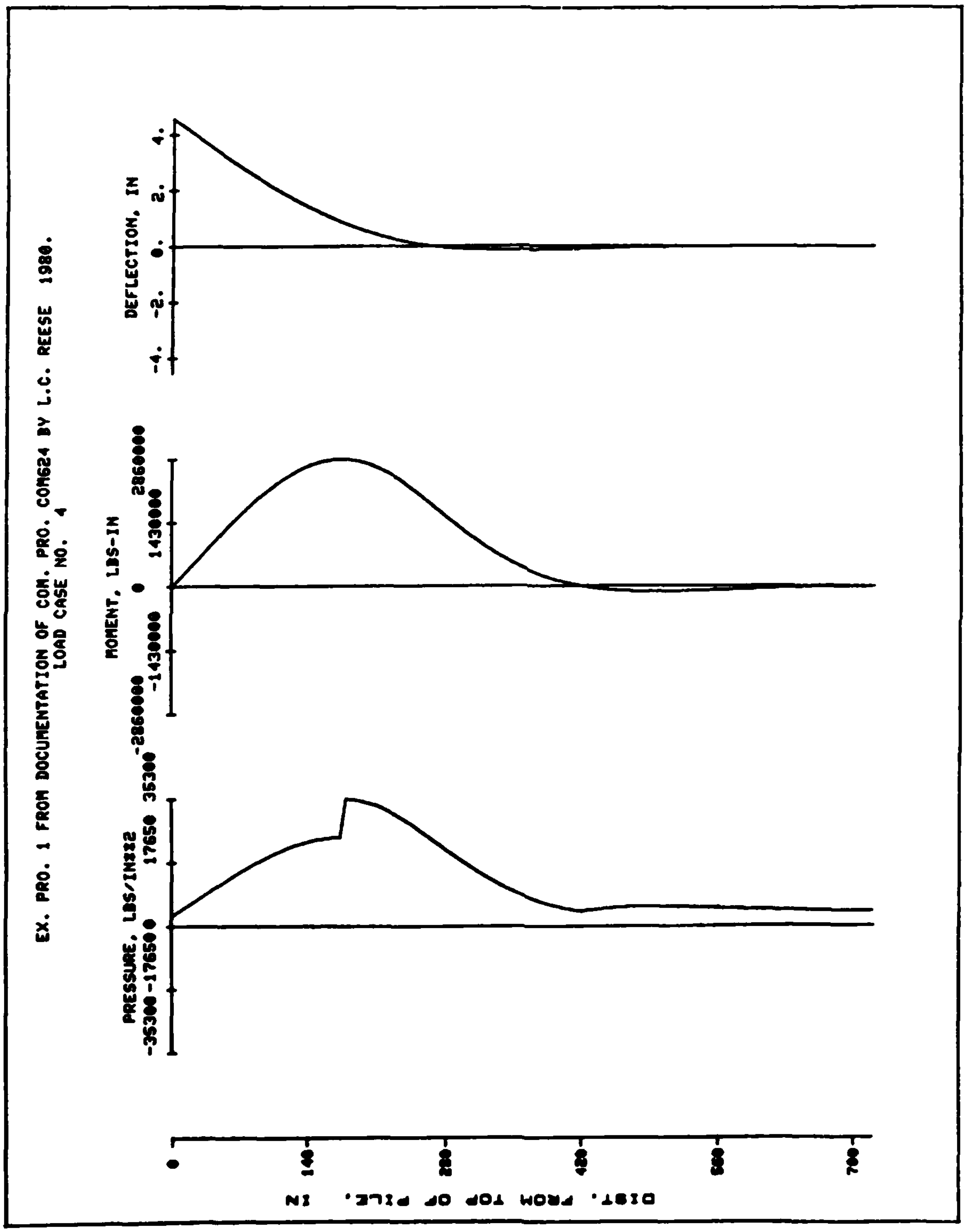
X IN	DEFLEC IN	MOMENT LBS-IN	TOTAL STRESS LBS/IN**2	DISTR. LOAD LBS/IN	SOIL MODULUS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2
*******	*********	*********	*********	*********	*********	*********
0.	0.456E 01	0.	0.278E 04	0.	0.	0.304E 11
12.00	0.427E 01	0.270E 06	0.484E 04	0.	0.	0.304E 11
24.00	0.397E 01	0.539E 06	0.690E 04	0.	0.	0.304E 11
36.00	0.368E 01	0.809E 06	0.896E 04	0.	0.	0.304E 11
48.00	0.339E 01	0.108E 07	0.110E 05	0.	0.	0.304E 11
60.00	0.310E 01	0.135E 07	0.131E 05	0.	0.234E 02	0.304E 11
72.00	0.282E 01	0.161E 07	0.151E 05	0.	0.339E 02	0.304E 11
84.00	0.255E 01	0.185E 07	0.169E 05	0.	0.469E 02	0.304E 11
636.00	0.662E-02	-0.254E 05	0.438E 04	0.	0.990E 03	0.212E 11
648.00	0.695E-02	-0.187E 05	0.431E 04	0.	0.990E 03	0.212E 11
660.00	0.714E-02	-0.130E 05	0.425E 04	0.	0.990E 03	0.212E 11
672.00	0.725E-02	-0.834E 04	0.420E 04	0.	0.990E 03	0.212E 11
684.00	0.730E-02	-0.470E 04	0.416E 04	0.	0.990E 03	0.212E 11
696.00	0.732E-02	-0.209E 04	0.413E 04	0.	0.990E 03	0.212E 11
708.00	0.733E-02	-0.522E 03	0.411E 04	0.	0.990E 03	0.212E 11
720.00	0.733E-02	0.	0.411E 04	0.	0.990E 03	0.212E 11

OUTPUT VERIFICATION

```
THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT      = -0.233E-01 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT =  0.266E-02 LBS

COMPUTED LATERAL FORCE AT PILE HEAD              =  0.20000E 05 LBS
    COMPUTED MOMENT AT PILE HEAD                 =  0.          IN-LBS
    COMPUTED SLOPE AT PILE HEAD                  = -0.24829E-01

THE OVERALL MOMENT IMBALANCE                =  0.546E-02 IN-LBS
THE OVERALL LATERAL FORCE IMBALANCE         = -0.480E-08 LBS
```

OUTPUT SUMMARY

```
PILE HEAD DEFLECTION    =  0.456E 01 IN
MAXIMUM BENDING MOMENT =  0.286E 07 IN-LBS
MAXIMUM TOTAL STRESS    =  0.353E 05 LBS/IN**2
MAXIMUM SHEAR FORCE     =  0.225E 05 LBS
```



EX. PRO. 1 FROM DOCUMENTATION OF COM. PRO. COM624 BY L.C. REESE 1980.
LOAD CASE NO. 4
DEFLECTION, IN
-4. -2. 0. 2. 4.
MOMENT, LBS-IN
-2860000 -1430000 0 1430000 2860000
PRESSURE, LBS/IN**2
-35300 -17650 0 17650 35300
0 140 280 420 560 700
DIST. FROM TOP OF PILE, IN

EX. PRO. 1 FROM DOCUMENTATION OF COM. PRO. COM624 BY L.C. REESE, 19
80.

SUMMARY TABLE
****

LATERAL LOAD (LBS)	BOUNDARY CONDITION BC2	AXIAL LOAD (LBS)	YT (IN)	ST (IN/IN)	MAX. MOMENT (IN-LBS)	MAX. STRESS (LBS/IN**2)
0.500E 04	0.	0.100E 06	0.452E 00	-0.317E-02	0.475E 06	0.831E 04
0.100E 05	0.	0.100E 06	0.118E 01	-0.769E-02	0.108E 07	0.146E 05
0.150E 05	0.	0.100E 06	0.226E 01	-0.137E-01	0.177E 07	0.227E 05
0.200E 05	0.	0.100E 06	0.456E 01	-0.248E-01	0.286E 07	0.353E 05

Example problem 2

8. A free-head pile with no applied moment and a lateral load of 10,000 lb will be analyzed. An axial load of 100,000 lb will be applied at the pile head. p-y curves will be generated internally using the soft clay criteria for the soft clay, sand criteria for the sand, and unified clay criteria for the medium clay (A = 1.0 and F = 0.7 for the unified criteria). Loading will be assumed to be cyclic. Output will include points on the p-y curves at x coordinates of 60, 80, 100, 150, 200, 250, 300, and 500 in.

```
10 TITLE
20 EX. PRO. 2 FROM DOCUMENTATION OF COM. PRO. COM624 BY L.C. REESE, 1980.
30 UNITS
40 ENGL
50 PILE 120 2 720 29.E6 60          (Pile Properties - NI,NDIAM,LENGTH,EPILE,XGS)
60 0 16 1047                        (XDIAM(I),DIAM(I),MINERT(I)
70 180 16 732                           Where I = 1,NDIAM
80 SOIL 3                           (Soil Description - NL)
90 1 1  60 240  30                   LAYER(I),KSOIL)I),XTOP)I),XBOT(I),K(I),(AE(I),FR(I))
100 2 4 240 360  25                     Where I = 1,NL
110 3 6 360 800 100 1.0 0.7
120 STRENGTH 6                      (Soil Strength Profile - NSTR)
130 60 3.5 0 .02                     XSTR(I),C1(I),PHI1(I),EE50(I)
140 240 3.5 0. .02
150 240 0 30 .02                         Where I = 1,NSTR
160 360 0 30 .02
170 360 7 0 .01
180 800 7 0 .01
190 WEIGHT 6                        (Unit Weight Profile - NGI)
200 60 .02
210 240 .02                            XG1(I),GAM1(I)
220 240 .032
230 360 .032                             Where I=1,NGI
240 360 .026
250 800 .026
260 OUTPUT 1 2 1 8                       (Output Control - KOUTPT,INC,KPYOP,NNSUB)
270 60 80 100 150 200 250 300 500        (XNSUB(I) .... XNSUB(NNSUB)
280 BOUNDARY 1 1               (Boundary Condition at Pile Head - KBC,NRUN)
290 1 10000 0 1.E5             (KOPSUB(I),PTSUB(I),BC2SUB(I),PXSUB(I), Where I = 1,NRUN)
300 CYCLIC .0 0                (Cyclic Load Indicator - KCYCL,RCYCL)
310 CONTROL 100 .001 24        (Program Control - MAXIT,YTOL,EXDEFL)
320 END
```

*

(Input Echo)

***** UNIT DATA. *****

SYSTEM OF UNITS
(UP TO 16 CHAR.)
ENGL

***** PILE DATA. *****

NO. INCREMENTS PILE IS DIVIDED	NO. SEGMENTS WITH DIFFERENT CHARACTERISTICS	LENGTH OF PILE	MODULUS OF ELASTICITY	DEPTH
120	2	0.720E 03	0.290E 08	0.600E 02

TOP OF SEGMENT	DIAMETER OF PILE	MOMENT OF INERTIA	CROSS-SECT. AREA
0.	0.160E 02	0.105E 04	0.359E 02
0.180E 03	0.160E 02	0.732E 03	0.243E 02

***** SOIL DATA. *****

NUMBER OF LAYERS
3

LAYER NUMBER	P-Y CURVE CONTROL CODE	TOP OF LAYER	BOTTOM OF LAYER	INITIAL SOIL MODULI CONST.	FACTOR "A"	FACTOR "F"
1	1	0.600E 02	0.240E 03	0.300E 02	0.	0.
2	4	0.240E 03	0.360E 03	0.250E 02	0.	0.
3	6	0.360E 03	0.800E 03	0.100E 03	0.100E 01	0.700E 00

***** UNIT WEIGHT DATA. *****

NO. POINTS FOR PLOT
OF EFF. UNIT WEIGHT
VS. DEPTH
6

DEPTH BELOW TOP TO POINT	EFFECTIVE UNIT WEIGHT
0.600E 02	0.200E-01
0.240E 03	0.200E-01
0.240E 03	0.320E-01

0.360E 03	0.320E-01
0.360E 03	0.260E-01
0.800E 03	0.260E-01

***** PROFILE DATA. *****

NO. POINTS FOR
STRENGTH PARAMETERS
VS. DEPTH
6

DEPTH BELOW TOP OF PILE	UNDRAINED SHEAR STRENGTH OF SOIL	ANGLE OF INTERNAL FRICTION IN RADIANS	STRAIN AT 50% STRESS LEVEL
0.600E 02	0.350E 01	0.	0.200E-01
0.240E 03	0.350E 01	0.	0.200E-01
0.240E 03	0.	0.524E 00	0.200E-01
0.360E 03	0.	0.524E 00	0.200E-01
0.360E 03	0.700E 01	0.	0.100E-01
0.800E 03	0.700E 01	0.	0.100E-01

***** P-Y DATA. *****

NO. OF
P-Y CURVES
0

***** OUTPUT DATA. *****

DATA OUTPUT CODE	OUTPUT INCREMENT CODE	P-Y PRINTOUT CODE	NO. DEPTHS TO PRINT FOR P-Y CURVES
1	20	1	8

DEPTH FOR PRINTING P-Y CURVES
0.600E 02
0.800E 02
0.100E 03
0.150E 03
0.200E 03
0.250E 03
0.300E 03
0.500E 03

***** PILE HEAD (BOUNDARY) DATA. *****

BOUNDARY CONDITION CODE	NO. OF SETS OF BOUNDARY CONDITIONS
1	1

PILE HEAD PRINTOUT CODE	LATERAL LOAD AT TOP OF PILE	VALUE OF SECOND BOUNDARY CONDITION	AXIAL LOAD ON PILE
1	0.100E 05	0.	0.100E 06

***** CYCLIC DATA. *****

CYCLIC(0) OR STATIC(1) LOADING	NO. CYCLES OF LOADING
0	0.100E 03

***** PROGRAM CONTROL DATA. *****

MAX. NO. OF ITERATIONS	TOLERENCE ON SOLUTION CONVERGENCE	PILE HEAD DEFLECTION FLAG(STOPS RUN)
100	0.100E-02	0.240E 02

***** LOAD DATA. *****

BOUNDARY SET NO.	NO. POINTS FOR DISTRIB. LATERAL LOAD VS. DEPTH
1	0

GENERATED P-Y CURVES

THE NUMBER OF CURVES = 8
THE NUMBER OF POINTS ON EACH CURVE = 17

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
0.	16.000	0.4E 01	0.2E-01	0.200E-01

Y,IN	P,LBS/IN
0.	0.
0.006	16.800
0.200	52.917
0.400	66.671
0.600	76.319
0.800	84.000
1.000	90.486
1.200	96.156
1.400	101.226
1.600	105.833
1.800	110.071
2.000	114.006
2.200	117.686
2.400	121.149
6.400	70.560
12.000	0.000
16.000	0.

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
20.00	16.000	0.4E 01	0.2E-01	0.200E-01

Y,IN	P,LBS/IN
0.	0.
0.006	20.940
0.200	65.957
0.400	83.100
0.600	95.126
0.800	104.700
1.000	112.785
1.200	119.852
1.400	126.171
1.600	131.914
1.800	137.196
2.000	142.100
2.200	146.687
2.400	151.004
6.400	95.688
12.000	18.577
16.000	18.577

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
40.00	16.000	0.4E 01	0.2E-01	0.200E-01

Y,IN	P,LBS/IN
0.	0.
0.006	25.080
0.200	78.997
0.400	99.530
0.600	113.933
0.800	125.400
1.000	135.083
1.200	143.547
1.400	151.116
1.600	157.994
1.800	164.320
2.000	170.194
2.200	175.688
2.400	180.858
6.400	123.877
12.000	44.499
16.000	44.499

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
90.00	16.000	0.4E 01	0.2E-01	0.200E-01

Y,IN	P,LBS/IN
0.	0.
0.006	35.430
0.200	111.598
0.400	140.604
0.600	160.951
0.800	177.150
1.000	190.829
1.200	202.786
1.400	213.478
1.600	223.195
1.800	232.132
2.000	240.430
2.200	248.191
2.400	255.495
6.400	207.740
12.000	141.442
16.000	141.442

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
140.00	16.000	0.4E 01	0.2E-01	0.200E-01

Y,IN	P,LBS/IN
0.	0.
0.006	45.780
0.200	144.198

0.400	181.678
0.600	207.969
0.800	228.900
1.000	246.575
1.200	262.025
1.400	275.841
1.600	288.396
1.800	299.944
2.000	310.665
2.200	320.693
2.400	330.131
6.400	310.732
12.000	284.294
16.000	284.294

DEPTH IN	DIAM IN	PHI DEG	GAMMA LBS/IN**3	A	B	PCT	PCD
190.00	16.00	30.0	0.2E-01	0.88	0.55	0.16E 04	0.18E 04

Y IN	P LBS/IN
0.	0.
0.022	105.556
0.044	211.111
0.067	316.667
0.089	422.222
0.111	527.778
0.133	627.427
0.156	675.613
0.178	720.334
0.200	762.232
0.222	801.772
0.244	839.304
0.267	875.100
0.600	1400.160
5.733	1400.160
10.867	1400.160
16.000	1400.160

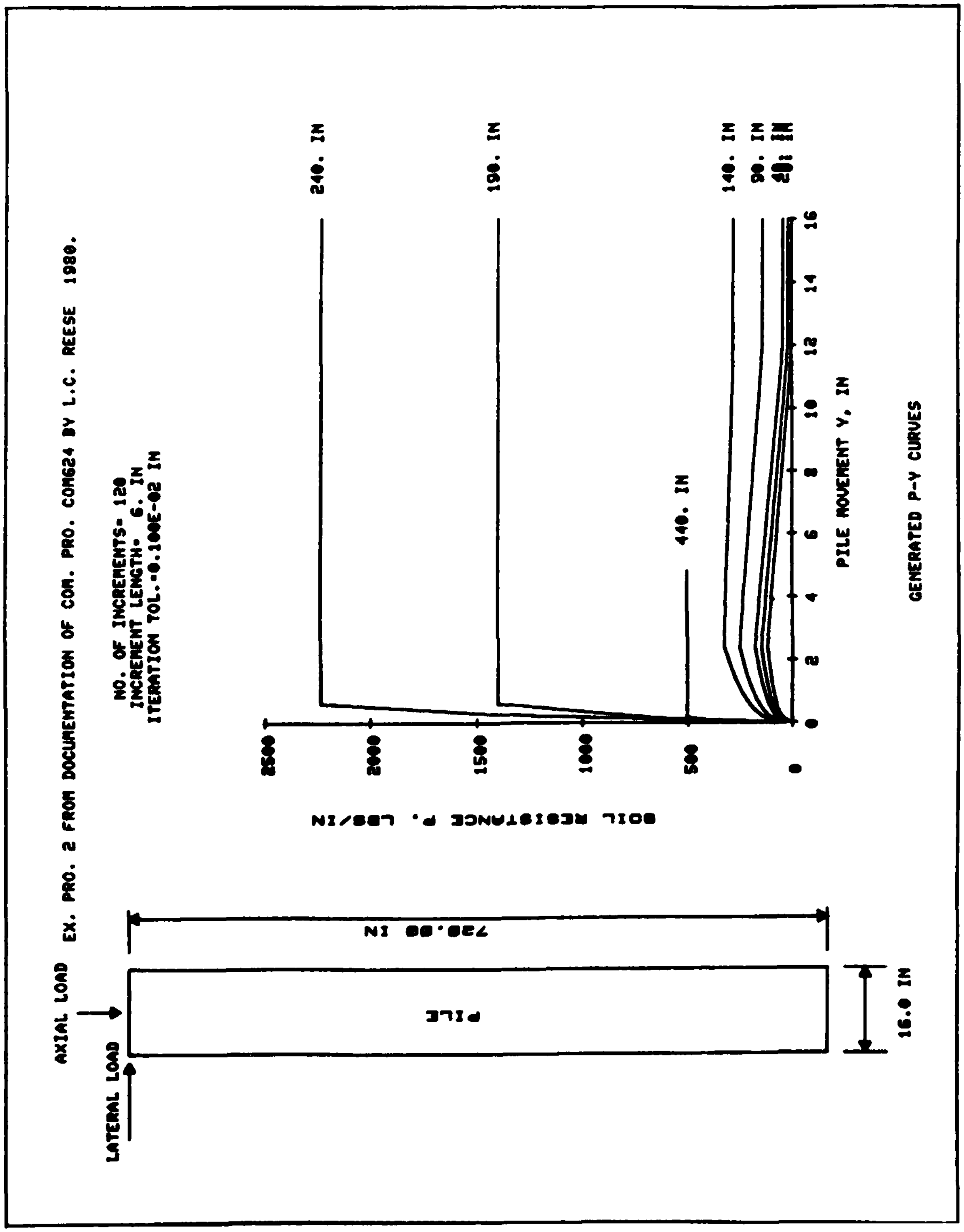
DEPTH IN	DIAM IN	PHI DEG	GAMMA LBS/IN**3	A	B	PCT	PCD
240.00	16.00	30.0	0.2E-01	0.88	0.55	0.28E 04	0.25E 04

Y IN	P LBS/IN
0.	0.
0.022	133.333
0.044	266.667
0.067	400.000
0.089	533.333
0.111	666.667
0.133	800.000
0.156	933.333
0.178	1066.667
0.200	1200.000
0.222	1279.319

0.244	1339.206
0.267	1396.323
0.600	2234.117
5.733	2234.117
10.867	2234.117
16.000	2234.117

DEPTH	DIAM	C	CAVG	GAMMA	E50
IN	IN	LBS/IN**2	LBS/IN**3	LBS/IN**3	
440.00	16.000	0.7E 01	0.4E 01	0.3E-01	0.100E-01

Y IN	P LBS/IN
0.	0.
0.013	220.142
0.027	277.362
0.040	317.500
0.053	349.454
0.067	376.438
0.080	400.025
0.093	421.117
0.107	440.285
0.120	457.914
0.133	474.282
0.147	489.592
0.160	504.000
1.173	504.000
2.187	504.000
3.200	504.000
4.800	504.000


EX. PRO. 2 FROM DOCUMENTATION OF COM. PRO. COM624 BY L.C. REESE 1980.
NO. OF INCREMENTS= 120
INCREMENT LENGTH= 6. IN
ITERATION TOL.=0.100E-02 IN
AXIAL LOAD
LATERAL LOAD
PILE
720.00 IN
16.0 IN
SOIL RESISTANCE P, LBS/IN
0
500
1000
1500
2000
2500
PILE MOVEMENT Y, IN
0
2
4
6
8
10
12
14
16
240. IN
190. IN
440. IN
140. IN
90. IN
GENERATED P-Y CURVES

EX. PRO. 2 FROM DOCUMENTATION OF COM. PRO. COM624 BY L.C. REESE 1980.
LOADING CONDITIONS

LOAD CASE NO.	LATERAL LOAD AT PILE HEAD(LBS)	AXIAL LOAD AT PILE HEAD(LBS)	APPLIED MOMENT AT PILE HEAD(LBS-IN)
1	10000.	100000.	0.

EX. PRO. 2 FROM DOCUMENTATION OF COM. PRO. COM624 BY L.C. REESE, 19
80.

UNITS--ENGL

O U T P U T   I N F O R M A T I O N
****

NO. OF ITERATIONS = 14
MAXIMUM DEFLECTION ERROR = 0.562E-03 IN

PILE LOADING CONDITION
- LATERAL LOAD AT PILE HEAD = 0.100E 05 LBS
- APPLIED MOMENT AT PILE HEAD = 0. LBS-IN
- AXIAL LOAD AT PILE HEAD = 0.100E 06 LBS

X	DEFLEC	MOMENT	TOTAL STRESS	DISTR. LOAD	SOIL MODULUS	FLEXURAL RIGIDITY
IN	IN	LBS-IN	LBS/IN**2	LBS/IN	LBS/IN**2	LBS-IN**2
*******	*********	*********	*********	*********	*********	*********
0.	0.135E 01	0.	0.115E 04	0.	0.	0.304E 11
12.00	0.125E 01	0.130E 06	0.214E 04	0.	0.	0.304E 11
24.00	0.115E 01	0.260E 06	0.313E 04	0.	0.	0.304E 11
36.00	0.105E 01	0.390E 06	0.413E 04	0.	0.	0.304E 11
48.00	0.954E 00	0.520E 06	0.512E 04	0.	0.	0.304E 11
60.00	0.859E 00	0.649E 06	0.611F 04	0.	0.100E 03	0.304E 11
72.00	0.767E 00	0.769E 06	0.702E 04	0.	0.124E 03	0.304E 11
84.00	0.679E 00	0.875E 06	0.783E 04	0.	0.152E 03	0.304E 11
636.00	-0.203E-06	-0.944E 02	0.199E 04	0.	0.576E 05	0.212E 11
648.00	0.511E-06	-0.649E 02	0.199E 04	0.	0.588E 05	0.212E 11
660.00	0.783E-06	-0.395E 02	0.199E 04	0.	0.600E 05	0.212E 11
672.00	0.785E-06	-0.207E 02	0.198E 04	0.	0.612E 05	0.212E 11
684.00	0.642E-06	-0.874E 01	0.198E 04	0.	0.624E 05	0.212E 11
696.00	0.438E-06	-0.249E 01	0.198E 04	0.	0.636E 05	0.212E 11
708.00	0.216E-06	-0.243E 00	0.198E 04	0.	0.648E 05	0.212E 11
720.00	-0.931E-08	0.	0.198E 04	0.	0.660E 05	0.212E 11

OUTPUT VERIFICATION

```
THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT      =  0.106E-01 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT =  0.143E-02 LBS

COMPUTED LATERAL FORCE AT PILE HEAD            =  0.10000E 05 LBS
    COMPUTED MOMENT AT PILE HEAD               =  0.          IN-LBS
    COMPUTED SLOPE AT PILE HEAD                = -0.84314E-02

THE OVERALL MOMENT IMBALANCE               =  0.285E-02 IN-LBS
THE OVERALL LATERAL FORCE IMBALANCE        = -0.131E-08 LBS
```

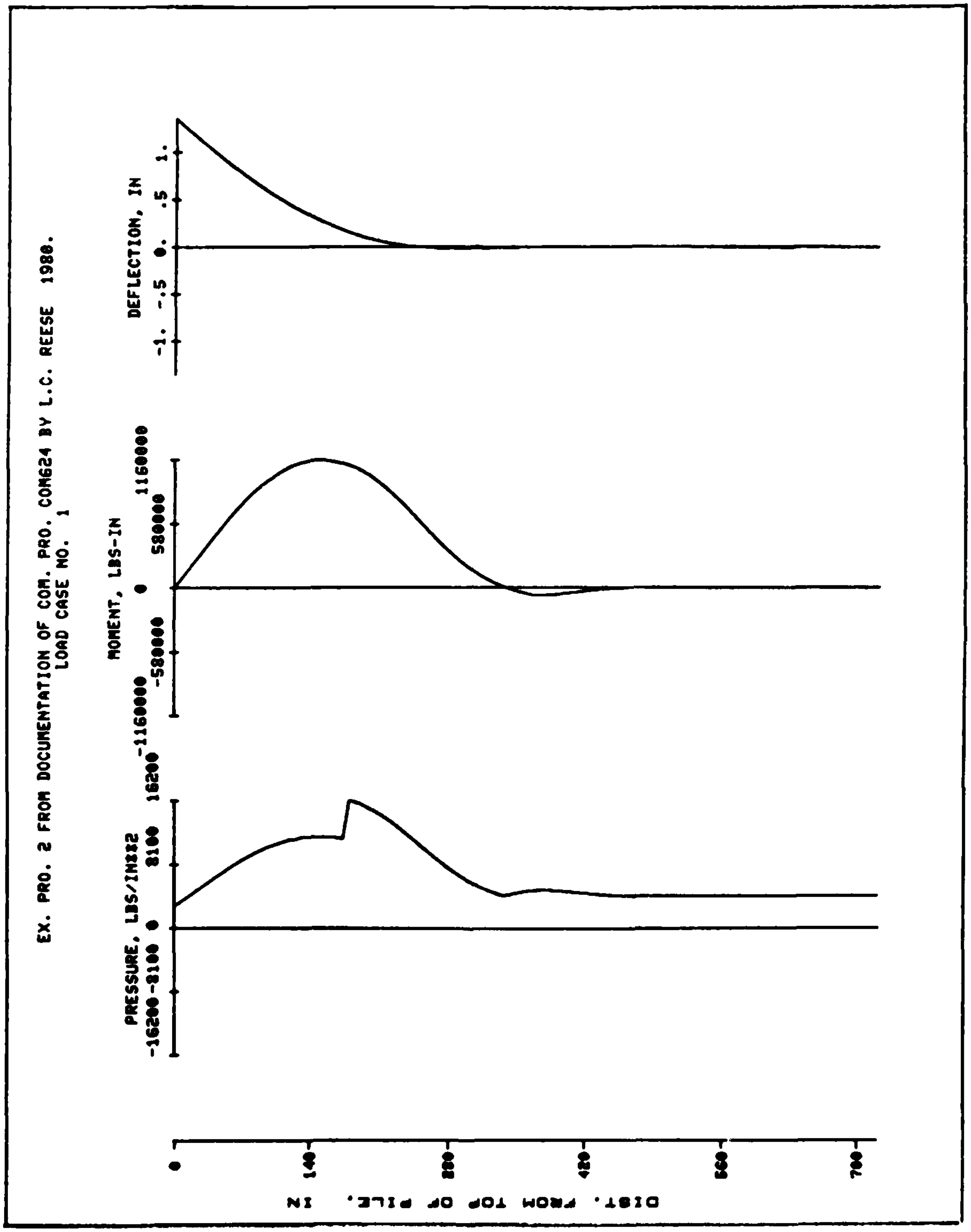
OUTPUT SUMMARY

```
PILE HEAD DEFLECTION    =  0.135E 01 IN
MAXIMUM BENDING MOMENT  =  0.116E 07 IN-LBS
MAXIMUM TOTAL STRESS    =  0.141E 05 LBS/IN**2
MAXIMUM SHEAR FORCE     =  0.108E 05 LBS
```

EX. PRO. 2 FROM DOCUMENTATION OF COM. PRO. COM624 BY L.C. REESE, 19
80.

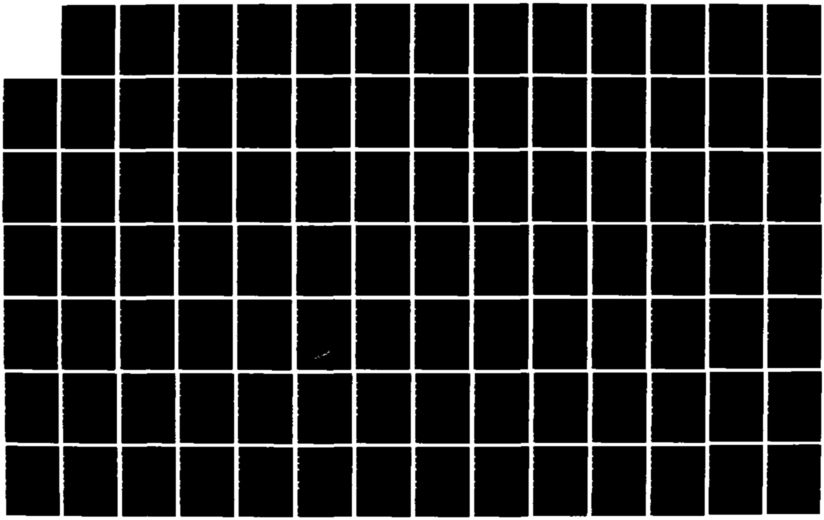
S U M M A R Y   T A B L E
****

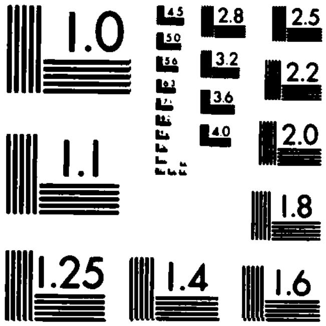
```
 LATERAL    BOUNDARY   AXIAL                                  MAX.        MAX.
  LOAD     CONDITION   LOAD        YT          ST            MOMENT      STRESS
  (LBS)       BC2       (LBS)      (IN)       (IN/IN)      (IN-LBS)  (LBS/IN**2)
 0.100E 05 0.         0.100E 06 0.135E 01-0.843E-02 0.116E 07 0.141E 05
```


EX. PRO. 2 FROM DOCUMENTATION OF COM. PRO. CON624 BY L.C. REESE 1980.
LOAD CASE NO. 1
DEFLECTION, IN
-1. -.5 0. .5 1.
MOMENT, LBS-IN
-1160000 -580000 0 580000 1160000
PRESSURE, LBS/IN**2
-16200 -8100 0 8100 16200
DIST. FROM TOP OF PILE, IN
0 140 280 420 560 700

AD-A144 641 LATERALLY LOADED PILES AND COMPUTER PROGRAM COM624G(U) 3/4
TEXAS UNIV AT AUSTIN L C REESE ET AL. APR 84
WES-TR-K-84-2

UNCLASSIFIED F/G 13/13 NL





MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

Example problem 3

9. A fixed-head pile will be analyzed under a lateral load of 10,000 lb and an axial load of 100,000 lb. p-y curves will be generated internally using the soft clay criteria for both clay layers and sand criteria for the sand layer. A p-y curve will be output at $x = 500$ in.

```
10 TITLE
20 EX. PRO. 3 FROM DOCUMENTATION OF COM. PRO. COM624 BY L.C. REESE, 1980.
30 UNITS
40 ENGL
50 PILE 120 2 720 29.E6 60  (Pile Properties - NI,NDIAM,LENGTH,EPILE,XGS)
60 0 16 1047                (XDIAM(I),DIAM(I),MINERT(I)
70 180 16 732                  where I = 1,NDIAM
80 STRENGTH 6                (Soil Strength Profile - NSTR)
90  60 3.5 0.0 .02
100 240 3.5 0.0 .02           XSTR(I),C1(I),PHI1(I),EE50(I)
110 240 0.0 30. .02
120 360 0.0 30. .02               where I = 1,NSTR
130 360 7.0 0.0 .01
140 800 7.0 0.0 .01
150 WEIGHT 6                 (Unit Weight Profile - NGI)
160  60 .02
170 240 .02                     XG1(I),GAM1(I)
180 240 .032
190 360 .032                    where I = 1,NGI
200 360 .026
210 800 .026
220 SOIL 3                   (Soil Description - NL)
230 1 1  60 240  30           LAYER(I),KSOIL(I),XTOP(I),XBOT(I),K(I)
240 2 4 240 360  25                where I = 1,NL
250 3 1 360 800 100
260 BOUNDARY 2 1             (Boundary Conditions at Pile Head - KBC,NRUN)
270 1 10000 0.0 1.E5         (KOPSUB(I),PTSUB(I),BC2SUB(I),PXSUB(I) Where I=1,NRUN)
280 OUTPUT 1 2 1 1           (Output Control - KOUTPT,INC,KPYOP,NNSUB)
290 500                      (XNSUB(I) ... XNSUB(NNSUB)
300 CYCLIC .0 0              (Cyclic Load Indicator - KCYCL,RCYCL)
310 END
```

*

(Input Echo)

***** UNIT DATA. *****

SYSTEM OF UNITS
(UP TO 16 CHAR.)
ENGL

***** PILE DATA. *****

NO. INCREMENTS PILE IS DIVIDED	NO. SEGMENTS WITH DIFFERENT CHARACTERISTICS	LENGTH OF PILE	MODULUS OF ELASTICITY	DEPTH
120	2	0.720E 03	0.290E 08	0.600E 02

TOP OF SEGMENT	DIAMETER OF PILE	MOMENT OF INERTIA	CROSS-SECT. AREA
0.	0.160E 02	0.105E 04	0.359E 02
0.180E 03	0.160E 02	0.732E 03	0.243E 02

***** SOIL DATA. *****

NUMBER OF LAYERS
3

LAYER NUMBER	P-Y CURVE CONTROL CODE	TOP OF LAYER	BOTTOM OF LAYER	INITIAL SOIL MODULI CONST.	FACTOR "A"	FACTOR "F"
1	1	0.600E 02	0.240E 03	0.300E 02	0.	0.
2	4	0.240E 03	0.360E 03	0.250E 02	0.	0.
3	2	0.360E 03	0.800E 03	0.100E 03	0.100E 01	0.700E 00

***** UNIT WEIGHT DATA. *****

NO. POINTS FOR PLOT
OF EFF. UNIT WEIGHT
VS. DEPTH
6

DEPTH BELOW TOP TO POINT	EFFECTIVE UNIT WEIGHT
0.600E 02	0.200E-01
0.240E 03	0.200E-01
0.240E 03	0.320E-01

0.360E 03	0.320E-01
0.360E 03	0.260E-01
0.800E 03	0.260E-01

***** PROFILE DATA. *****

NO. POINTS FOR
STRENGTH PARAMETERS
VS. DEPTH
6

DEPTH BELOW TOP OF PILE	UNDRAINED SHEAR STRENGTH OF SOIL	ANGEL OF INTERNAL FRICTION IN RADIANS	STRAIN AT 50% STRESS LEVEL
0.600E 02	0.350E 01	0.	0.200E-01
0.240E 03	0.350E 01	0.	0.200E-01
0.240E 03	0.	0.524E 00	0.200E-01
0.360E 03	0.	0.524E 00	0.200E-01
0.360E 03	0.700E 01	0.	0.100E-01
0.800E 03	0.700E 01	0.	0.100E-01

***** P-Y DATA. *****

NO. OF
P-Y CURVES
0

***** OUTPUT DATA. *****

DATA OUTPUT CODE	OUTPUT INCREMENT CODE	P-Y PRINTOUT CODE	NO. DEPTHS TO PRINT FOR P-Y CURVES
1	2	1	1

DEPTH FOR
PRINTING
P-Y CURVES
0.500E 03

***** PILE HEAD (BOUNDARY) DATA. *****

BOUNDARY CONDITION CODE	NO. OF SETS OF BOUNDARY CONDITIONS
2	1

PILE HEAD PRINTOUT CODE	LATERAL LOAD AT TOP OF PILE	VALUE OF SECOND BOUNDARY CONDITION	AXIAL LOAD ON PILE
1	0.100E 05	0.	0.100E 06

***** CYCLIC DATA. *****

CYCLIC(0) OR STATIC(1) LOADING	NO. CYCLES OF LOADING
0	0.100E 03

***** PROGRAM CONTROL DATA. *****

MAX. NO. OF ITERATIONS	TOLERENCE ON SOLUTION CONVERGENCE	PILE HEAD DEFLECTION FLAG(STOPS RUN)
100	0.100E-02	0.240E 02

***** LOAD DATA. *****

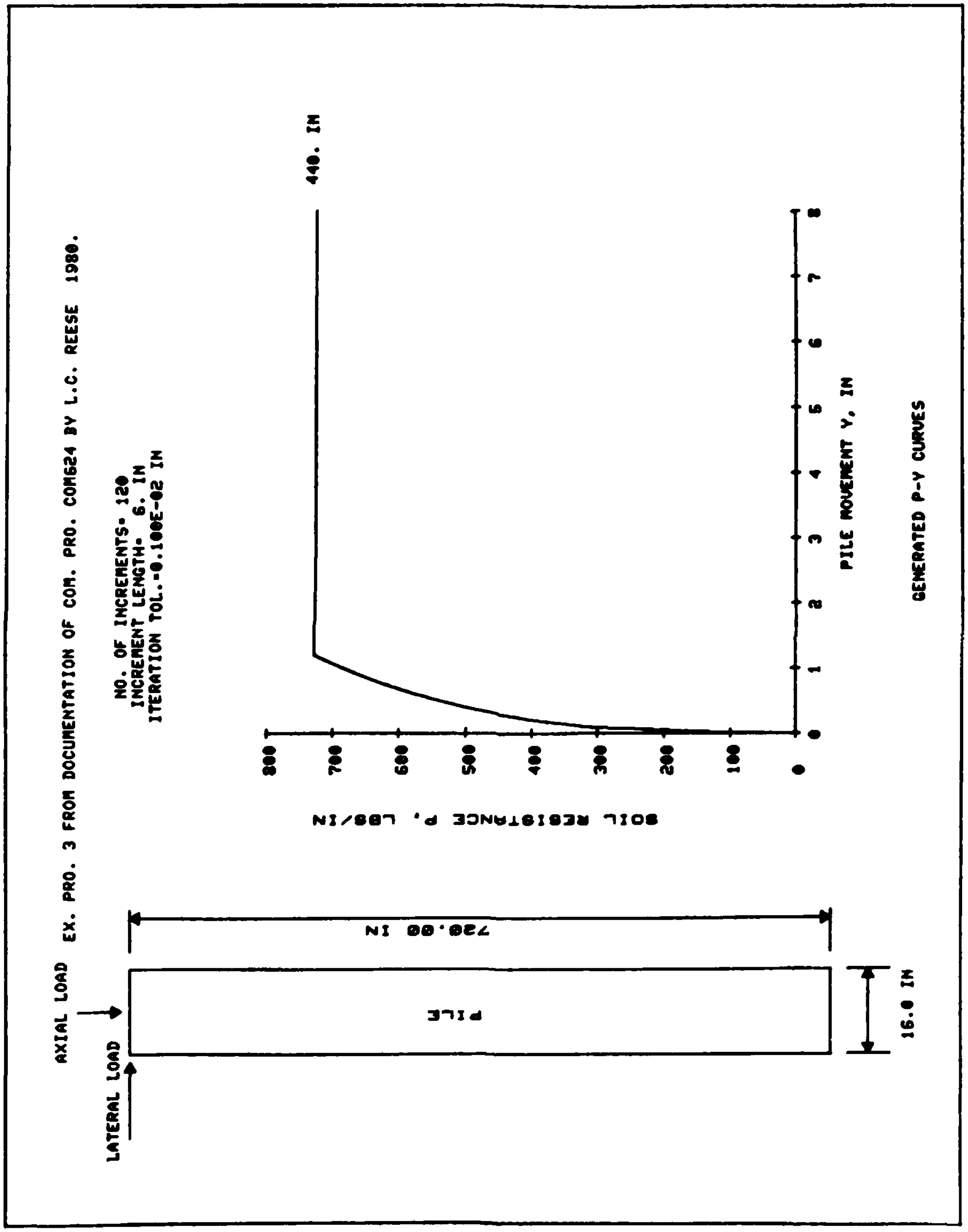
BOUNDARY SET NO.	NO. POINTS FOR DISTRIB. LATERAL LOAD VS. DEPTH
1	0

GENERATED P-Y CURVES

THE NUMBER OF CURVES = 1
THE NUMBER OF POINTS ON EACH CURVE = 17

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
440.00	16.000	0.7E 01	0.3E-01	0.100E-01

Y,IN	P,LBS/IN
0.	0.
0.003	100.800
0.100	317.500
0.200	400.025
0.300	457.914
0.400	504.000
0.500	542.918
0.600	576.936
0.700	607.356
0.800	635.000
0.900	660.427
1.000	684.033
1.100	706.114
1.200	726.894
3.200	725.760
6.000	725.760
8.000	725.760



EX. PRO. 3 FROM DOCUMENTATION OF COM. PRO. COM624 BY L.C. REESE 1980.
NO. OF INCREMENTS= 120
INCREMENT LENGTH= 6. IN
ITERATION TOL.=0.100E-02 IN
440. IN
SOIL RESISTANCE P, LBS/IN
PILE MOVEMENT Y, IN
GENERATED P-Y CURVES
AXIAL LOAD
LATERAL LOAD
PILE
720.00 IN
16.0 IN

EX. PRO. 3 FROM DOCUMENTATION OF COM. PRO. COM624 BY L.C. REESE 1980.
LOADING CONDITIONS

LOAD CASE NO.	LATERAL LOAD AT PILE HEAD(LBS)	AXIAL LOAD AT PILE HEAD(LBS)	SLOPE AT PILE HEAD
1	10000.	100000.	0.

EX. PRO. 3 FROM DOCUMENTATION OF COM. PRO. COM624 BY L.C. REESE, 19
80.

UNITS--ENGL

O U T P U T   I N F O R M A T I O N
****

NO. OF ITERATIONS = 9
MAXIMUM DEFLECTION ERROR = 0.796E-03 IN

PILE LOADING CONDITION

- LATERAL LOAD AT PILE HEAD = 0.100E 05 LBS
- SLOPE AT PILE HEAD = 0. IN/IN
- AXIAL LOAD AT PILE HEAD = 0.100E 06 LBS

X IN	DEFLEC IN	MOMENT LBS-IN	TOTAL STRESS LBS/IN**2	DISTR. LOAD LBS/IN	SOIL MODULUS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2
*******	*********	*********	*********	*********	*********	*********
0.	0.269E 00	-0.986E 06	0.103E 05	0.	0.	0.304E 11
12.00	0.267E 00	-0.866E 06	0.940E 04	0.	0.	0.304E 11
24.00	0.261E 00	-0.745E 06	0.848E 04	0.	0.	0.304E 11
36.00	0.251E 00	-0.624E 06	0.755E 04	0.	0.	0.304E 11
48.00	0.238E 00	-0.503E 06	0.663E 04	0.	0.	0.304E 11
60.00	0.223E 00	-0.381E 06	0.570E 04	0.	0.247E 03	0.304E 11
72.00	0.206E 00	-0.266E 06	0.481E 04	0.	0.299E 03	0.304E 11
84.00	0.187E 00	-0.159E 06	0.400E 04	0.	0.359E 03	0.304E 11
636.00	0.100E-36	0.	0.411E 04	0.	0.196E 12	0.212E 11
648.00	0.100E-36	0.	0.411E 04	0.	0.196E 12	0.212E 11
660.00	0.100E-36	0.	0.411E 04	0.	0.196E 12	0.212E 11
672.00	0.100E-36	0.	0.411E 04	0.	0.196E 12	0.212E 11
684.00	0.100E-36	0.	0.411E 04	0.	0.196E 12	0.212E 11
696.00	0.100E-36	0.	0.411E 04	0.	0.196E 12	0.212E 11
708.00	0.100E-36	0.	0.411E 04	0.	0.196E 12	0.212E 11
720.00	0.100E-36	0.	0.411E 04	0.	0.196E 12	0.212E 11

OUTPUT VERIFICATION

```
THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT      =  0.481E-02 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = -0.743E-03 LBS

COMPUTED LATERAL FORCE AT PILE HEAD            =  0.10000E 05 LBS
    COMPUTED SLOPE AT PILE HEAD                =  0.            IN/IN

THE OVERALL MOMENT IMBALANCE                 = -0.179E-02 IN-LBS
THE OVERALL LATERAL FORCE IMBALANCE          = -0.406E-09 LBS
```

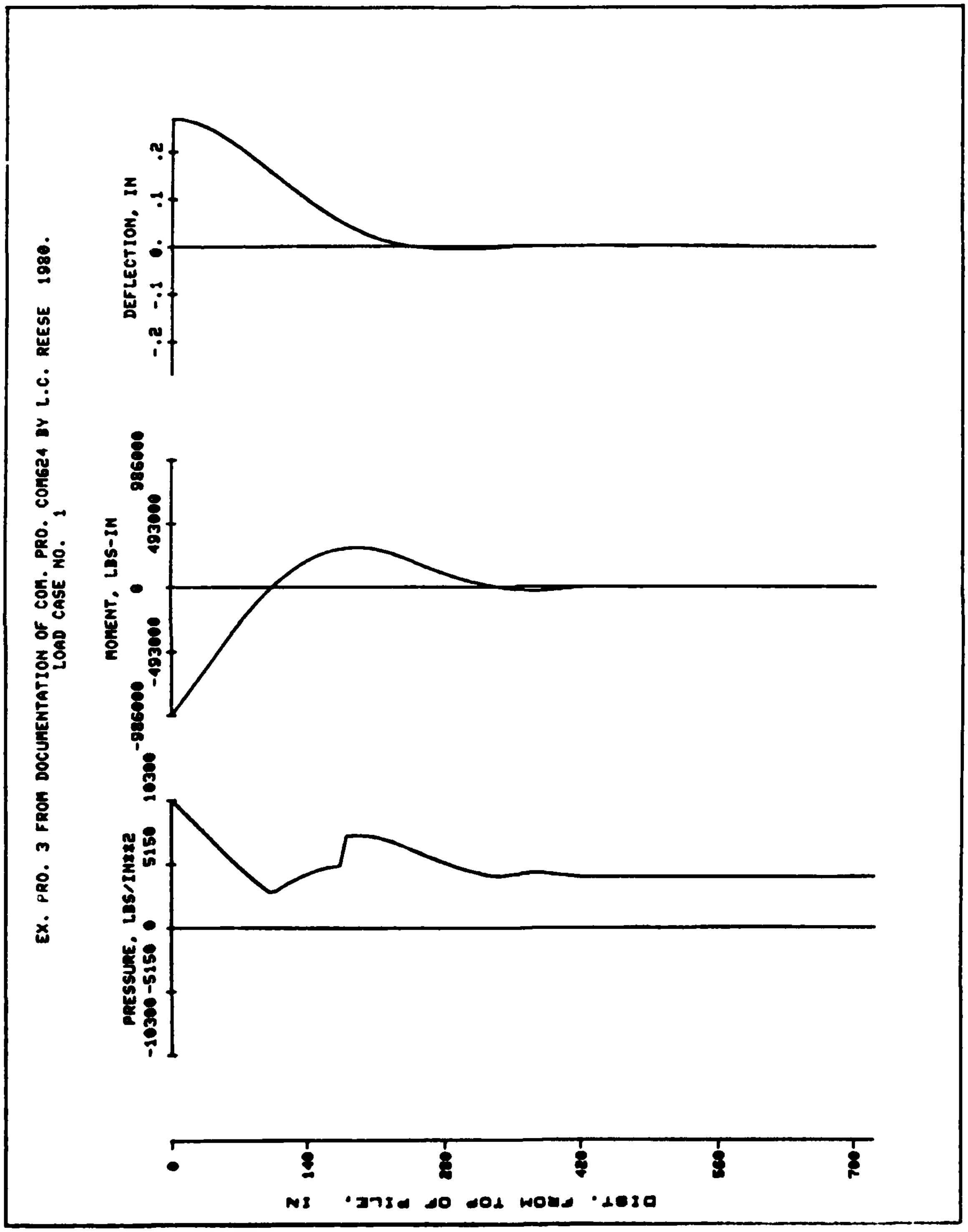
OUTPUT SUMMARY

```
PILE HEAD DEFLECTION    =  0.269E 00 IN
MAXIMUM BENDING MOMENT = -0.986E 06 IN-LBS
MAXIMUM TOTAL STRESS    =  0.103E 05 LBS/IN**2
MAXIMUM SHEAR FORCE     =  0.101E 05 LBS
```

EX. PRO. 3 FROM DOCUMENTATION OF COM. PRO. COM624 BY L.C. REESE, 19
80.

S U M M A R Y   T A B L E
****

LATERAL LOAD (LBS)	BOUNDARY CONDITION BC2	AXIAL LOAD (LBS)	YT (IN)	ST (IN/IN)	MAX. MOMENT (IN-LBS)	MAX. STRESS (LBS/IN**2)
0.100E 05	0.	0.100E 06	0.269E 00	0.	-0.986E 06	0.103E 05


EX. PRO. 3 FROM DOCUMENTATION OF COM. PRO. COM624 BY L.C. REESE 1980.
LOAD CASE NO. 1
PRESSURE, LBS/IN**2
-10300 -5150 0 5150 10300
MOMENT, LBS-IN
-986000 -493000 0 493000 986000
DEFLECTION, IN
-.2 -.1 0. .1 .2
DIST. FROM TOP OF PILE, IN
0 140 280 420 560 700

Example problem 4

10. A pile with a rotational restraint of $M_s/S_t = 1 \times 10^6$ in.-lb will be analyzed under a lateral load of 10,000 lb and an axial load of 100,000 lb. p-y curves will be generated internally using soft clay criteria for the soft clay, sand criteria for sand, and the criteria for stiff clay below the water table for the medium clay. Coordinates of a p-y curve at x = 500 in. will be output.

```
10 TITLE
20 EX. PRO. 4 FROM DOCUMENTATION OF COM. PRO. COM624 BY L.C. REESE, 1980.
30 UNITS
40 ENGL
50 PILE 120 2 720 29.E6 60 (Pile Properties - NI,NDIAM,LENGTH,EPILE,XGS)
60 0 16 1047               (XDIAM(I),DIAM(I),MINERT(I)
70 180 16 732                   where I = 1,NDIAM
80 SOIL 3                  (Soil Description - NL)
90 1 1 60 240 30           (LAYER(I),KSOIL(I),XTOP(I)XBOT(I),K(I)
100 2 4 240 360 25              where I = 1,NL
110 3 2 360 800 100
120 OUTPUT 1 2 1 1         (Output Control - KOUTPT,INC,KPYOP,NNSUB)
130 500                    (XNSUB(I) ... XNSUB(NNSUB))
140 BOUN 3 1               (Boundary Condition at Pile Head - KBC,NRUN)
150 1 10000 1.E6 1.E5      (KOPSUB(I),PTSUB(I),BC2SUB(I),PXSUB(I), Where I = 1,NRUN)
160 CONTROL 100 .001 24    (Program Control - MAXIT,YTOL,EXDEFL)
170 STRENGTH 6             (Soil Strength Profile - NSTR)
180 60 3.5 0 .02
190 240 3.5 0 .02          (XSTR(I),C1(I),PHI1(I),EE50(I)
200 240 0 30 .02
210 360 0 30 .02                where I = 1,NSTR
220 360 7 0 .01
230 800 7 0 .01
240 WEIGHT 6               (Unit Weight Profile - NGI)
250 60 .02
260 240 .02                 XG1(I),GAM1(I)
270 240 .032
280 360 .032                    where I = 1,NGI
290 360 .026
300 800 .026
310 CYCLIC 0 0             (Cyclic Load Indicator - KCYCL,RCYCL)
320 END
```

*

(Input Echo)

***** UNIT DATA. *****

SYSTEM OF UNITS
(UP TO 16 CHAR.)
ENGL

***** PILE DATA. *****

NO. INCREMENTS PILE IS DIVIDED	NO. SEGMENTS WITH DIFFERENT CHARACTERISTICS	LENGTH OF PILE	MODULUS OF ELASTICITY	DEPTH
120	2	0.720E 03	0.290E 08	0.600E 02

TOP OF SEGMENT	DIAMETER OF PILE	MOMENT OF INERTIA	CROSS-SECT. AREA
0.	0.160E 02	0.105E 04	0.359E 02
0.180E 03	0.160E 02	0.732E 03	0.243E 02

***** SOIL DATA. *****

NUMBER OF LAYERS
3

LAYER NUMBER	P-Y CURVE CONTROL CODE	TOP OF LAYER	BOTTOM OF LAYER	INITIAL SOIL MODULI CONST.	FACTOR "A"	FACTOR "F"
1	1	0.600E 02	0.240E 03	0.300E 02	0.	0.
2	4	0.240E 03	0.360E 03	0.250E 02	0.	0.
3	1	0.360E 03	0.800E 03	0.100E 03	0.100E 01	0.700E 00

***** UNIT WEIGHT DATA. *****

NO. POINTS FOR PLOT
OF EFF. UNIT WEIGHT
VS. DEPTH
6

DEPTH BELOW TOP TO POINT	EFFECTIVE UNIT WEIGHT
0.600E 02	0.200E-01
0.240E 03	0.200E-01
0.240E 03	0.320E-01

0.360E 03	0.320E-01
0.360E 03	0.260E-01
0.800E 03	0.260E-01

***** PROFILE DATA. *****

NO. POINTS FOR
STRENGTH PARAMETERS
VS. DEPTH
6

DEPTH BELOW TOP OF PILE	UNDRAINED SHEAR STRENGTH OF SOIL	ANGLE OF INTERNAL FRICTION IN RADIANS	STRAIN AT 50% STRESS LEVEL
0.600E 02	0.350E 01	0.	0.200E-01
0.240E 03	0.350E 01	0.	0.200E-01
0.240E 03	0.	0.524E 00	0.200E-01
0.360E 03	0.	0.524E 00	0.200E-01
0.360E 03	0.700E 01	0.	0.100E-01
0.800E 03	0.700E 01	0.	0.100E-01

***** P-Y DATA. *****

NO. OF
P-Y CURVES
0

***** OUTPUT DATA. *****

DATA OUTPUT CODE	OUTPUT INCREMENT CODE	P-Y PRINTOUT CODE	NO. DEPTHS TO PRINT FOR P-Y CURVES
1	20	1	1

DEPTH FOR
PRINTING
P-Y CURVES
0.500E 03

***** PILE HEAD (BOUNDARY) DATA. *****

BOUNDARY CONDITION CODE	NO. OF SETS OF BOUNDARY CONDITIONS
3	1

PILE HEAD PRINTOUT CODE	LATERAL LOAD AT TOP OF PILE	VALUE OF SECOND BOUNDARY CONDITION	AXIAL LOAD ON PILE
1	0.100E 05	0.100E 07	0.100E 06

***** CYCLIC DATA. *****

CYCLIC(0) OR STATIC(1) LOADING	NO. CYCLES OF LOADING
0	0.100E 03

***** PROGRAM CONTROL DATA. *****

MAX. NO. OF ITERATIONS	TOLERENCE ON SOLUTION CONVERGENCE	PILE HEAD DEFLECTION FLAG(STOPS RUN)
100	0.100E-02	0.240E 02

***** LOAD DATA. *****

BOUNDARY SET NO.	NO. POINTS FOR DISTRIB. LATERAL LOAD VS. DEPTH
1	0

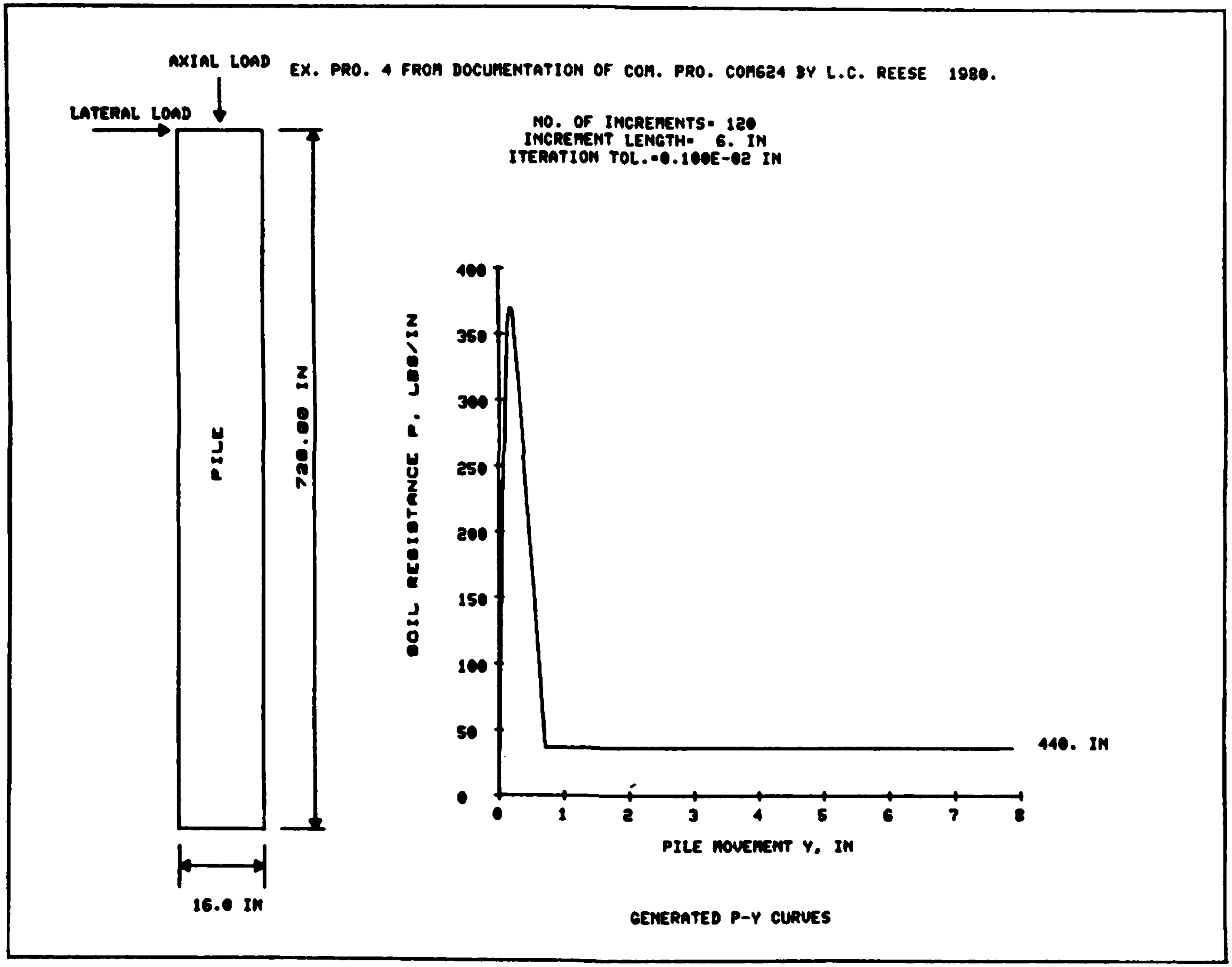
GENERATED P-Y CURVES

THE NUMBER OF CURVES = 1
THE NUMBER OF POINTS ON EACH CURVE = 17

DEPTH	DIAM	C	CAVG	GAMMA	E50
IN	IN	LBS/IN**2	LBS/IN**2	LBS/IN**3	
440.00	16.000	0.7E 01	0.4E 01	0.3E-01	0.100E-01

AS =0.60 AC =0.30

Y,IN	P,LBS/IN
0.	0.
0.020	94.272
0.039	172.416
0.059	235.477
0.079	284.574
0.098	320.928
0.118	345.890
0.138	360.996
0.157	368.079
0.177	369.600
0.197	368.079
0.216	360.996
0.236	345.890
0.394	242.901
0.551	139.857
0.708	36.812
7.872	36.812


EX. PRO. 4 FROM DOCUMENTATION OF COM. PRO. COM624 BY L.C. REESE 1980.
AXIAL LOAD
LATERAL LOAD
PILE
720.00 IN
16.0 IN
NO. OF INCREMENTS= 120
INCREMENT LENGTH= 6. IN
ITERATION TOL.=0.100E-02 IN
SOIL RESISTANCE P, LBS/IN
0
50
100
150
200
250
300
350
400
0 1 2 3 4 5 6 7 8
PILE MOVEMENT Y, IN
440. IN
GENERATED P-Y CURVES

EX. PRO. 4 FROM DOCUMENTATION OF COM. PRO. COM624 BY L.C. REESE, 1980.
LOADING CONDITIONS

LOAD CASE NO.	LATERAL LOAD AT PILE HEAD(LBS)	AXIAL LOAD AT PILE HEAD(LBS)	ROTATIONAL STIFFNESS AT PILE HEAD(LBS)
1	10000.	100000.	1000000.

EX. PRO. 4 FROM DOCUMENTATION OF COM. PRO. COM624 BY L.C. REESE, 19
80.

UNITS--ENGL

OUTPUT INFORMATION
****

NO. OF ITERATIONS = 14
MAXIMUM DEFLECTION ERROR = 0.568E-03 IN

PILE LOADING CONDITION
LATERAL LOAD AT PILE HEAD = 0.100E 05 LBS
ROTATIONAL RESTRAINT = 0.100E 07 LBS-IN
AXIAL LOAD AT PILE HEAD = 0.100E 06 LBS

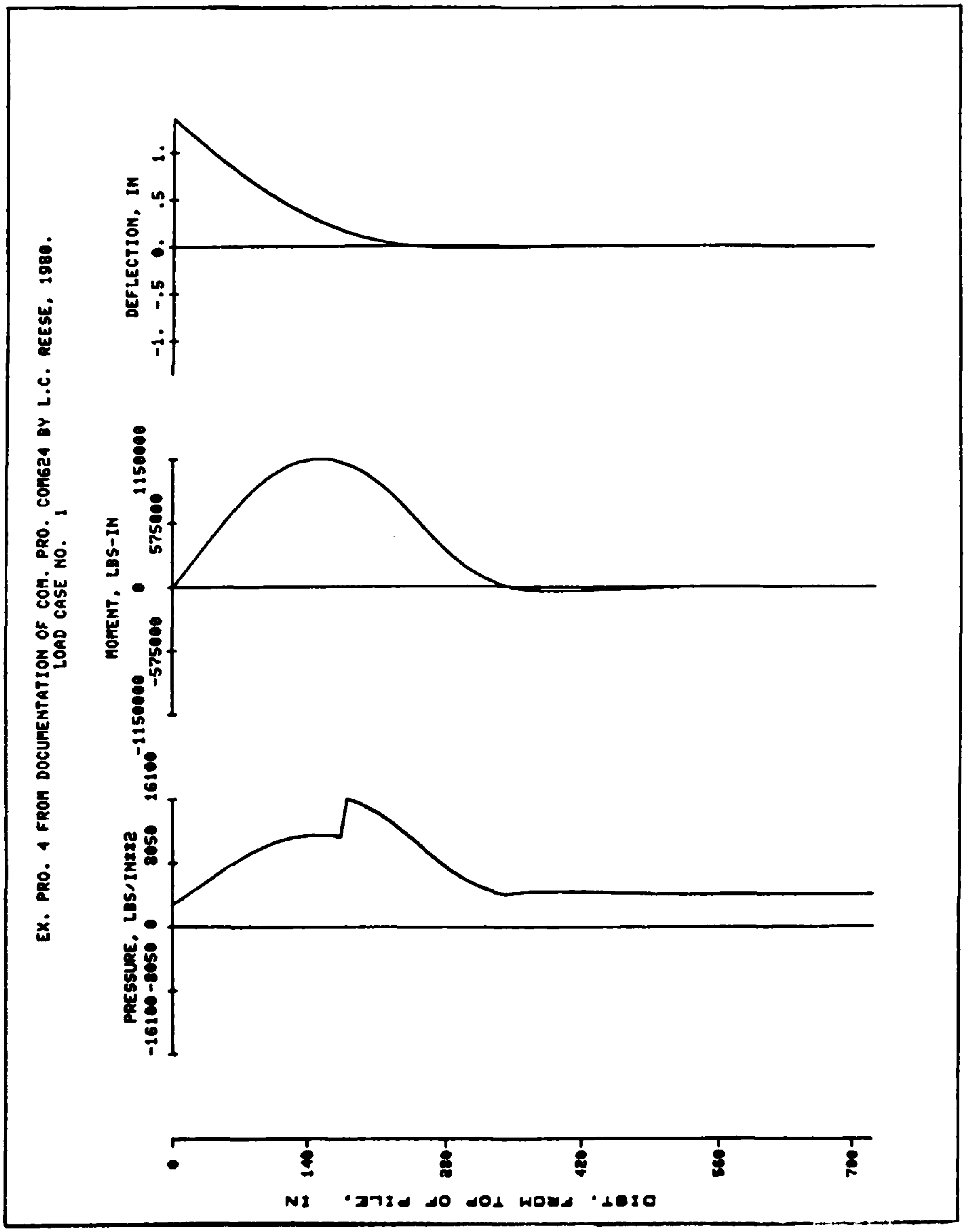
X	DEFLEC	MOMENT	TOTAL STRESS	DISTR. LOAD	SOIL MODULUS	FLEXURAL RIGIDITY
IN	IN	LBS-IN	LBS/IN**2	LBS/IN	LBS/IN**2	LBS-IN**2
*******	*********	*********	*********	*********	*********	*********
0.	0.135E 01	-0.837E 04	0.121E 04	0.	0.	0.304E 11
12.00	0.125E 01	0.122E 06	0.208E 04	0.	0.	0.304E 11
24.00	0.115E 01	0.252E 06	0.307E 04	0.	0.	0.304E 11
36.00	0.105E 01	0.382E 06	0.406E 04	0.	0.	0.304E 11
48.00	0.950E 00	0.511E 06	0.505E 04	0.	0.	0.304E 11
60.00	0.856E 00	0.641E 06	0.604E 04	0.	0.100E 03	0.304E 11
72.00	0.765E 00	0.760E 06	0.696E 04	0.	0.124E 03	0.304E 11
84.00	0.677E 00	0.866E 06	0.776E 04	0.	0.152E 03	0.304E 11
636.00	0.744E-05	0.105E 04	0.200E 04	0.	0.522E 04	0.212E 11
648.00	-0.147E-04	0.817E 03	0.199E 04	0.	0.522E 04	0.212E 11
660.00	-0.312E-04	0.596E 03	0.199E 04	0.	0.522E 04	0.212E 11
672.00	-0.438E-04	0.398E 03	0.199E 04	0.	0.522E 04	0.212E 11
684.00	-0.536E-04	0.232E 03	0.199E 04	0.	0.522E 04	0.212E 11
696.00	-0.618E-04	0.107E 03	0.199E 04	0.	0.522E 04	0.212E 11
708.00	-0.693E-04	0.273E 02	0.198E 04	0.	0.522E 04	0.212E 11
720.00	-0.765E-04	0.	0.198E 04	0.	0.522E 04	0.212E 11

```
    OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT      =  0.104E-01 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = -0.145E-02 LBS

COMPUTED LATERAL FORCE AT PILE HEAD             =  0.10000E 05 LBS
        COMPUTED ROTATIONAL STIFFNESS AT PILE HEAD =  0.10000E 07 IN-LB
S
    COMPUTED SLOPE AT PILE HEAD                   = -0.83710E-02

THE OVERALL MOMENT IMBALANCE                  = -0.324E-02 IN-LBS
THE OVERALL LATERAL FORCE IMBALANCE           = -0.132E-08 LBS


    OUTPUT SUMMARY

PILE HEAD DEFLECTION    =  0.135E 01 IN
MAXIMUM BENDING MOMENT =  0.115E 07 IN-LBS
MAXIMUM TOTAL STRESS    =  0.140E 05 LBS/IN**2
MAXIMUM SHEAR FORCE     =  0.108E 05 LBS
```

EX. PRO. 4 FROM DOCUMENTATION OF COM. PRO. COM624 BY L.C. REESE, 1980.

S U M M A R Y   T A B L E
****

LATERAL LOAD (LBS)	BOUNDARY CONDITION BC2	AXIAL LOAD (LBS)	YT (IN)	ST (IN/IN)	MAX. MOMENT (IN-LBS)	MAX. STRESS (LBS/IN**2)
0.100E 05	0.100E 07	0.100E 06	0.135E 01	-0.837E-02	0.115E 07	0.140E 05


EX. PRO. 4 FROM DOCUMENTATION OF COM. PRO. COM624 BY L.C. REESE, 1980.
LOAD CASE NO. 1
DEFLECTION, IN
-1. -.5 0. .5 1.
MOMENT, LBS-IN
-1150000 -575000 0 575000 1150000
PRESSURE, LBS/IN**2
-16100 -8050 0 8050 16100
0 140 280 420 560 700
DIST. FROM TOP OF PILE, IN

APPENDIX D: ADDITIONAL EXAMPLE PROBLEMS

Example 1

1. This example is provided to illustrate program sequence and also for comparison to the problem analyzed earlier by nondimensional methods in Appendix A. Pile properties and soil description are shown in Figure D1. Prompts, data and output echoes, and graphics are presented as they would appear at the user's terminal. Input is from a data file, and p-y curves will be generated for verification at x coordinates of 0, 16, 32, 48, 80, 128, 154, 240, 480, and 720 in.

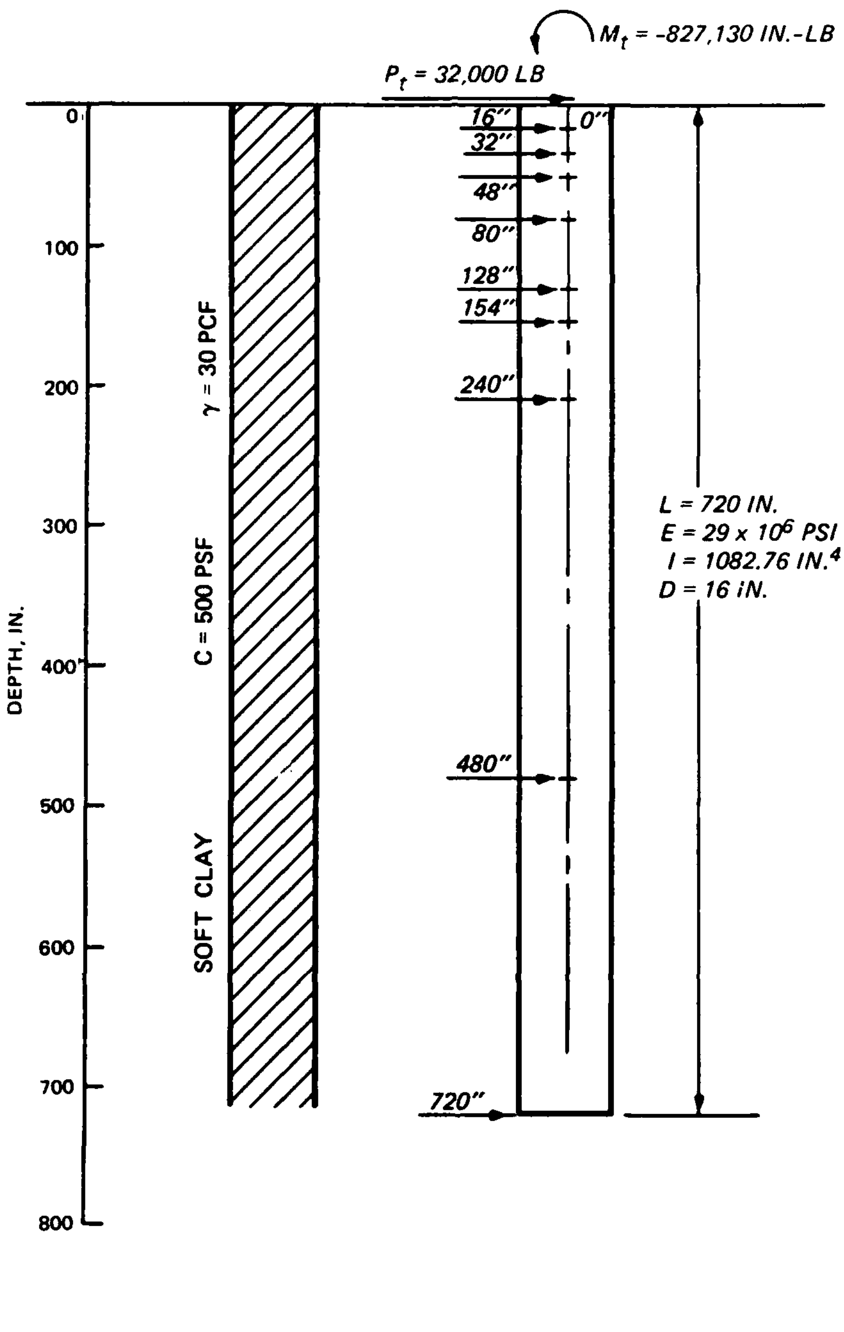


Figure D1. Pile and soil properties

```
10 TITLE
20 COMPARISON SOLUTION FOR EXAMPLE SOLVED BY NON-DIMENSIONAL METHOD
30 UNITS
40 ENGL
50 PILE 72 1 720 29.E6 0 (Pile Properties - NI,NDIAM,LENGTH,EPILE,XGS)
60 0 16 1082.79          (XDIAM(I),DIAM(I),MINERT(I), Where I=1,NDIAM)
70 SOIL 1                (Soil Description - NL)
80 1 1 0 720 25          (LAYER(I),KSOIL(I),XTOP(I),XBOT(I),K(I) Where I = 1,NL)
90 WEIGHT 2              (Unit Weight Profile - NGI)
100 0 .0174              (XG1(I),GAM1(I)
110 720 .0174              Where I = 1,NGI
120 STRENGTH 2           (Soil Strength Profile - NSTR)
130 0 3.472 0 .01         XSTR(I),C1(I),PHI1(I),EE50(I)
140 720 3.472 0 .01        Where I = 1,NSTR
150 OUTPUT 1 2 1 10      (Output Control - KOUTPT,INC,KPYOP,NNSUB)
160 0 16 32 48 80 128 154 240 480 720 (XNSUB(I) ... XNSUB(NNSUB)
170 BOUN 1 1    (Boundary Conditions at Pile Head - KBC,NRUN)
180 1 32000 -827130 0 (KOPSUB(I),PTSUB(I),BC2SUB(I),PXSUB(I), Where I = 1,NRUN)
190 CYCLIC 0 0          (Cyclic Load Indicator - KCYCL,RCYCL)
200 CONTROL 100 .001 40  (Program Control - MAXIT,YTOL,EXDEFL)
210 END

*
```

```
        02/09/82    08.700

IS INPUT FROM TERMINAL OR A FILE
ENTER T OR F
=F

ENTER DATA FILE NAME
=EDCOMND


COMPARISON SOLUTION FOR EXAMPLE SOLVED BY NON-DIMENSIONAL METHOD
INPUT COMPLETE. DO YOU WANT INPUT DATA
ECHOPRINTED TO YOUR TERMINAL, A FILE,
BOTH, OR NEITHER? (ENTER T, F, B, OR N)
=B
ENTER NAME FOR INPUT ECHOPRINT FILE
=INPUT

THIS FILE ALREADY EXISTS: INPUT
ENTER ANOTHER NAME-
=INEX


         ***** UNIT DATA. *****


   SYSTEM OF UNITS
   (UP TO 16 CHAR.)
ENGL


         ***** PILE DATA. *****


  NO. INCREMENTS     NO. SEGMENTS      LENGTH  MODULUS OF     DEPTH
 PILE IS DIVIDED    WITH DIFFERENT       OF    ELASTICITY
                    CHARACTERISTICS     PILE
        72                  1        0.720E 03 0.290E 08    0.


  TOP OF     DIAMETER     MOMENT OF     CROSS-SECT.
 SEGMENT     OF PILE       INERTIA         AREA
0.          0.160E 02    0.108E 04      0.373E 02


         ***** SOIL DATA. *****


  NUMBER OF LAYERS
         1
```

LAYER NUMBER	P-Y CURVE CONTROL CODE	TOP OF LAYER	BOTTOM OF LAYER	INITIAL SOIL MODULI CONST.	FACTOR "A"	FACTOR "F"
1	1	0.	0.720E 03	0.250E 02	0.	0.

***** UNIT WEIGHT DATA. *****

NO. POINTS FOR PLOT
OF EFF. UNIT WEIGHT
VS. DEPTH
2

DEPTH BELOW TOP TO POINT	EFFECTIVE UNIT WEIGHT
0.	0.174E-01
0.720E 03	0.174E-01

***** PROFILE DATA. *****

NO. POINTS FOR
STRENGTH PARAMETERS
VS. DEPTH
2

DEPTH BELOW TOP OF PILE	UNDRAINED SHEAR STRENGTH OF SOIL	ANGLE OF INTERNAL FRICTION IN RADIANS	STRAIN AT 50% STRESS LEVEL
0.	0.347E 01	0.	0.100E-01
0.720E 03	0.347E 01	0.	0.100E-01

***** P-Y DATA. *****

NO. OF
P-Y CURVES
0

***** OUTPUT DATA. *****

DATA OUTPUT CODE	OUTPUT INCREMENT CODE	P-Y PRINTOUT CODE	NO. DEPTHS TO PRINT FOR P-Y CURVES
1	2	1	10

DEPTH FOR
PRINTING
P-Y CURVES
0.

```
0.160E 02
0.320E 02
0.480E 02
0.800E 02
0.128E 03
0.154E 03
0.240E 03
0.480E 03
0.720E 03


    ***** PILE HEAD (BOUNDARY) DATA. *****


BOUNDARY      NO. OF SETS
CONDITION     OF BOUNDARY
  CODE        CONDITIONS
   1              1


 PILE HEAD      LATERAL LOAD AT    VALUE OF SECOND      AXIAL LOAD
PRINTOUT CODE     TOP OF PILE     BOUNDARY CONDITION     ON PILE
   1              0.320E 05          -.827E 06          0.


    ***** CYCLIC DATA. *****


  CYCLIC(0)       NO. CYCLES
 OR STATIC(1)     OF LOADING
  LOADING
   0              0.100E 03


    ***** PROGRAM CONTROL DATA. *****


MAX. NO. OF    TOLERENCE ON     PILE HEAD DEFLECTION
 ITERATIONS      SOLUTION       FLAG(STOPS  RUN)
               CONVERGENCE
   100          0.100E-02           0.400E 02


    ***** LOAD DATA. *****

  BOUNDARY      NO. POINTS FOR
  SET NO.      DISTRIB. LATERAL
                LOAD VS. DEPTH
     1               0

DO YOU WANT TO EDIT INPUT DATA? (YES OR NO)
=N
```

```
WILL OUTPUT GO TO THE TERMINAL, FILE OR BOTH?
ENTER T, F, OR B
=B

ENTER NAME FOR OUTPUT FILE
=OUTEX
```

(P-Y curves generated for verification)

GENERATED P-Y CURVES

THE NUMBER OF CURVES = 10
THE NUMBER OF POINTS ON EACH CURVE = 17

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
0.	16.000	0.3E 01	0.2E-01	0.100E-01

Y, IN	P, LBS/IN
0.	0.
0.003	16.666
0.100	52.493
0.200	66.137
0.300	75.709
0.400	83.328
0.500	89.762
0.600	95.387
0.700	100.416
0.800	104.987
0.900	109.191
1.000	113.093
1.100	116.744
1.200	120.180
3.200	69.996
6.000	0.000
8.000	0.

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
16.00	16.000	0.3E 01	0.2E-01	0.100E-01

Y, IN	P, LBS/IN
0.	0.
0.003	19.889
0.100	62.645
0.200	78.928
0.300	90.350
0.400	99.443
0.500	107.122
0.600	113.834
0.700	119.836
0.800	125.291
0.900	130.307
1.000	134.965

Y, IN	P, LBS/IN
1.100	139.322
1.200	143.422
3.200	89.302
6.000	13.847
8.000	13.847

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
32.00	16.000	0.3E 01	0.2E-01	0.100E-01

Y, IN	P, LBS/IN
0.	0.
0.003	23.112
0.100	72.797
0.200	91.719
0.300	104.992
0.400	115.558
0.500	124.482
0.600	132.281
0.700	139.256
0.800	145.594
0.900	151.424
1.000	156.837
1.100	161.900
1.200	166.664
3.200	110.478
6.000	32.182
8.000	32.182

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
48.00	16.000	0.3E 01	0.2E-01	0.100E-01

Y, IN	P, LBS/IN
0.	0.
0.003	26.335
0.100	82.949
0.200	104.509
0.300	119.633
0.400	131.674
0.500	141.841
0.600	150.729
0.700	158.676
0.800	165.898
0.900	172.541
1.000	178.709
1.100	184.477
1.200	189.906
3.200	133.524
6.000	55.004
8.000	55.004

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
80.00	16.000	0.3E 01	0.2E-01	0.100E-01

Y,IN	P,LBS/IN
0.	0.
0.003	32.781
0.100	103.253
0.200	130.091
0.300	148.917
0.400	163.904
0.500	176.560
0.600	187.623
0.700	197.516
0.800	206.506
0.900	214.775
1.000	222.452
1.100	229.633
1.200	236.390
3.200	185.227
6.000	114.113
8.000	114.113

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
128.00	16.000	0.3E 01	0.2E-01	0.100E-01

Y,IN	P,LBS/IN
0.	0.
0.003	42.450
0.100	133.709
0.200	168.463
0.300	192.842
0.400	212.250
0.500	228.639
0.600	242.965
0.700	255.776
0.800	267.418
0.900	278.126
1.000	288.067
1.100	297.366
1.200	306.117
3.200	276.805
6.000	236.436
8.000	236.436

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
154.00	16.000	0.3E 01	0.2E-01	0.100E-01

Y,IN	P,LBS/IN
0.	0.
0.003	47.687
0.100	150.206
0.200	189.247
0.300	216.634
0.400	238.437
0.500	256.848
0.600	272.942

0.700	287.333
0.800	300.412
0.900	312.441
1.000	323.609
1.100	334.055
1.200	343.885
3.200	333.437
6.000	319.559
8.000	319.559

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
240.00	16.000	0.3E 01	0.2E-01	0.100E-01

Y,IN	P,LBS/IN
0.	0.
0.003	49.997
0.100	157.480
0.200	198.412
0.300	227.126
0.400	249.984
0.500	269.287
0.600	286.160
0.700	301.249
0.800	314.960
0.900	327.572
1.000	339.280
1.100	350.232
1.200	360.539
3.200	359.977
6.000	359.977
8.000	359.977

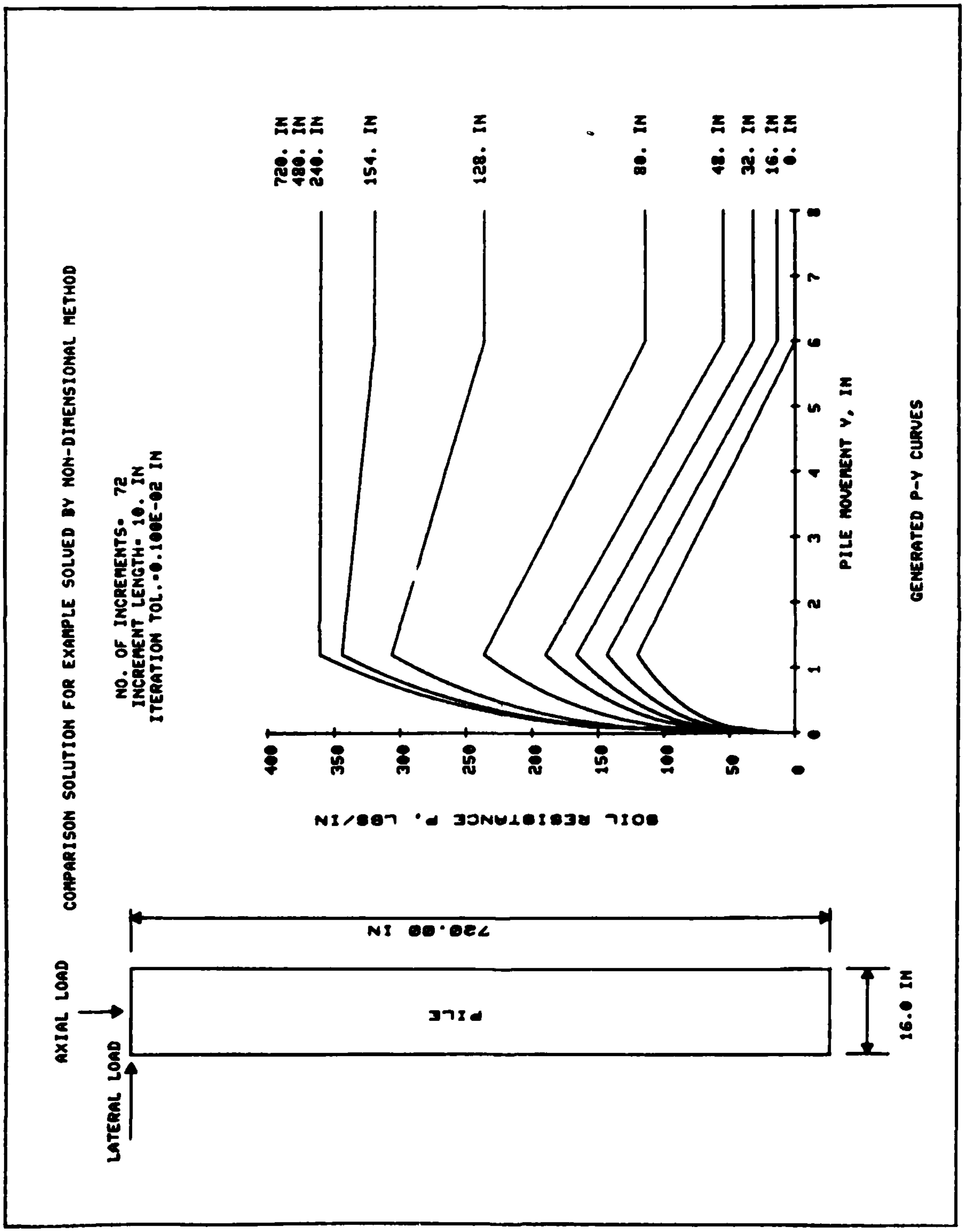
DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
480.00	16.000	0.3E 01	0.2E-01	0.100E-01

Y,IN	P,LBS/IN
0.	0.
0.003	49.997
0.100	157.480
0.200	198.412
0.300	227.126
0.400	249.984
0.500	269.287
0.600	286.160
0.700	301.249
0.800	314.960
0.900	327.572
1.000	339.280
1.100	350.232
1.200	360.539
3.200	359.977
6.000	359.977
8.000	359.977

DEPTH	DIAM	C	GAMMA	E50
IN	IN	LBS/IN**2	LBS/IN**3	
720.00	16.000	0.3E 01	0.2E-01	0.100E-01

Y,IN	P,LBS/IN
0.	0.
0.003	49.997
0.100	157.480
0.200	198.412
0.300	227.126
0.400	249.984
0.500	269.287
0.600	286.160
0.700	301.249
0.800	314.960
0.900	327.572
1.000	339.280
1.100	350.232
1.200	360.539
3.200	359.977
6.000	359.977
8.000	359.977

COMPARISON SOLUTION FOR EXAMPLE SOLVED BY NON-DIMENSIONAL METHOD
DO YOU WANT TO PLOT INPUT DATA? (Y OR N)
=Y


COMPARISON SOLUTION FOR EXAMPLE SOLVED BY NON-DIMENSIONAL METHOD
NO. OF INCREMENTS= 72
INCREMENT LENGTH= 10. IN
ITERATION TOL.=0.100E-02 IN
AXIAL LOAD
LATERAL LOAD
PILE
720.00 IN
16.0 IN
SOIL RESISTANCE P, LBS/IN
PILE MOVEMENT Y, IN
GENERATED P-Y CURVES
720. IN
480. IN
240. IN
154. IN
128. IN
80. IN
48. IN
32. IN
16. IN
0. IN
0
50
100
150
200
250
300
350
400
1
2
3
4
5
6
7
8

COMPARISON SOLUTION FOR EXAMPLE SOLVED BY NON-DIMENSIONAL METHOD
LOADING CONDITIONS

LOAD CASE NO.	LATERAL LOAD AT PILE HEAD(LBS)	AXIAL LOAD AT PILE HEAD(LBS)	APPLIED MOMENT AT PILE HEAD(LBS-IN)
1	32000.	0.	-827130.

COMPARISON SOLUTION FOR EXAMPLE SOLVED BY NON-DIMENSIONAL METHOD

UNITS--ENGL

OUTPUT INFORMATION
****

PILE LOADING CONDITION

LATERAL LOAD AT PILE HEAD = 0.320E 05 LBS
APPLIED MOMENT AT PILE HEAD = -0.827E 06 LBS-IN
AXIAL LOAD AT PILE HEAD = 0. LBS

X IN	DEFLEC IN	MOMENT LBS-IN	TOTAL STRESS LBS/IN**2	DISTR. LOAD LBS/IN	SOIL MODULUS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2
*******	*********	*********	*********	*********	*********	*********
0.	0.198E 01	-0.827E 06	0.611E 04	0.	0.507E 02	0.314E 11
20.00	0.175E 01	-0.209E 06	0.154E 04	0.	0.769E 02	0.314E 11
40.00	0.151E 01	0.356E 06	0.263E 04	0.	0.113E 03	0.314E 11
60.00	0.127E 01	0.852E 06	0.630E 04	0.	0.162E 03	0.314E 11
80.00	0.105E 01	0.127E 07	0.936E 04	0.	0.216E 03	0.314E 11
100.00	0.836E 00	0.159E 07	0.118E 05	0.	0.281E 03	0.314E 11
120.00	0.648E 00	0.182E 07	0.135E 05	0.	0.370E 03	0.314E 11
140.00	0.483E 00	0.196E 07	0.145E 05	0.	0.495E 03	0.314E 11
160.00	0.342E 00	0.200E 07	0.148E 05	0.	0.678E 03	0.314E 11
180.00	0.227E 00	0.194E 07	0.144E 05	0.	0.912E 03	0.314E 11
200.00	0.137E 00	0.181E 07	0.134E 05	0.	0.128E 04	0.314E 11
220.00	0.696E-01	0.160E 07	0.118E 05	0.	0.201E 04	0.314E 11
240.00	0.226E-01	0.134E 07	0.991E 04	0.	0.426E 04	0.314E 11
260.00	-0.734E-02	0.104E 07	0.771E 04	0.	0.893E 04	0.314E 11
280.00	-0.240E-01	0.760E 06	0.562E 04	0.	0.408E 04	0.314E 11
300.00	-0.309E-01	0.516E 06	0.381E 04	0.	0.345E 04	0.314E 11
320.00	-0.312E-01	0.314E 06	0.232E 04	0.	0.342E 04	0.314E 11
340.00	-0.274E-01	0.154E 06	0.114E 04	0.	0.373E 04	0.314E 11
360.00	-0.217E-01	0.354E 05	0.261E 03	0.	0.436E 04	0.314E 11
380.00	-0.155E-01	-0.455E 05	0.336E 03	0.	0.545E 04	0.314E 11
400.00	-0.987E-02	-0.925E 05	0.683E 03	0.	0.737E 04	0.314E 11
420.00	-0.538E-02	-0.110E 06	0.816E 03	0.	0.111E 05	0.314E 11
440.00	-0.228E-02	-0.105E 06	0.773E 03	0.	0.196E 05	0.314E 11
460.00	-0.491E-03	-0.810E 05	0.598E 03	0.	0.546E 05	0.314E 11
480.00	0.272E-03	-0.476E 05	0.352E 03	0.	0.807E 05	0.314E 11
500.00	0.423E-03	-0.204E 05	0.150E 03	0.	0.603E 05	0.314E 11
520.00	0.306E-03	-0.318E 04	0.235E 02	0.	0.749E 05	0.314E 11
540.00	0.141E-03	0.490E 04	0.362E 02	0.	0.126E 06	0.314E 11
560.00	0.329E-04	0.594E 04	0.439E 02	0.	0.333E 06	0.314E 11
580.00	-0.292E-05	0.264E 04	0.195E 02	0.	0.160E 07	0.314E 11
600.00	-0.356E-05	0.178E 03	0.132E 01	0.	0.145E 07	0.314E 11

620.00	-0.288E-06	-0.290E 03	0.214E 01	0.	0.793E 07	0.314E 11
640.00	0.153E-07	-0.208E 01	0.153E-01	0.	0.599E 08	0.314E 11
660.00	-0.632E-12	0.343E-02	0.254E-04	0.	0.759E 11	0.314E 11
680.00	0.203E-16	-0.136E-06	0.101E-08	0.	0.975E 11	0.314E 11
700.00	-0.652E-21	0.521E-11	0.385E-13	0.	0.975E 11	0.314E 11
720.00	0.419E-25	0.	0.	0.	0.975E 11	0.314E 11

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = 0.832E-02 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = -0.652E-03 LBS

COMPUTED LATERAL FORCE AT PILE HEAD = 0.32000E 05 LBS
COMPUTED MOMENT AT PILE HEAD = -0.82713E 06 IN-LBS
COMPUTED SLOPE AT PILE HEAD = -0.11650E-01

THE OVERALL MOMENT IMBALANCE = 0.933E-02 IN-LBS
THE OVERALL LATERAL FORCE IMBALANCE = -0.296E-09 LBS

OUTPUT SUMMARY

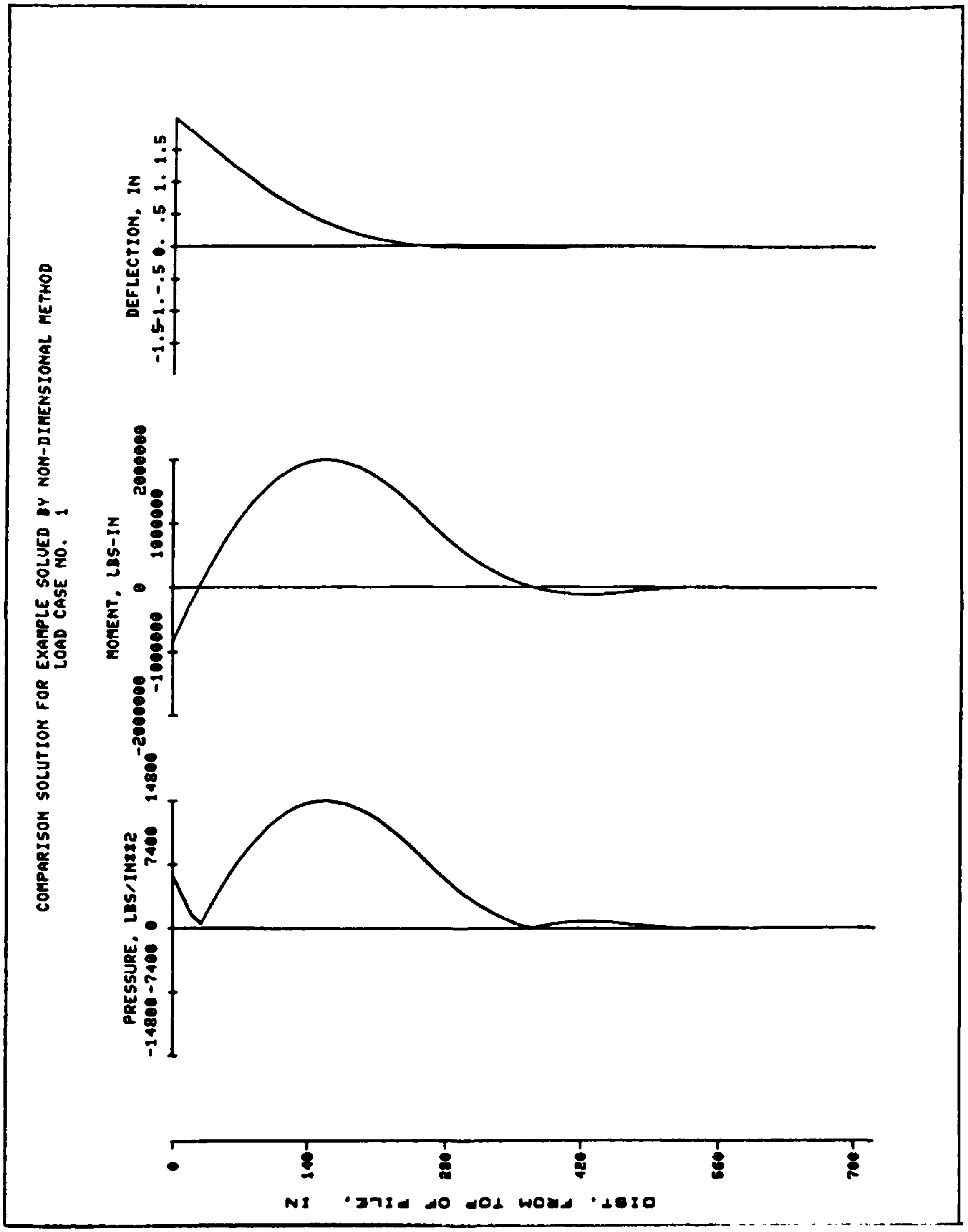
PILE HEAD DEFLECTION = 0.198E 01 IN
MAXIMUM BENDING MOMENT = 0.200E 07 IN-LBS
MAXIMUM TOTAL STRESS = 0.148E 05 LBS/IN**2
MAXIMUM SHEAR FORCE = 0.320E 05 LBS

COMPARISON SOLUTION FOR EXAMPLE SOLVED BY NON-DIMENSIONAL METHOD

S U M M A R Y   T A B L E
****

LATERAL LOAD (LBS)	BOUNDARY CONDITION BC2	AXIAL LOAD (LBS)	YT (IN)	ST (IN/IN)	MAX. MOMENT (IN-LBS)	MAX. STRESS (LBS/IN**2)
0.320E 05	-0.827E 06	0.	0.198E 01	-0.117E-01	0.200E 07	0.148E 05

COMPARISON SOLUTION FOR EXAMPLE SOLVED BY NON-DIMENSIONAL METHOD
DO YOU WANT TO PLOT OUTPUT? (Y OR N)
=Y


COMPARISON SOLUTION FOR EXAMPLE SOLVED BY NON-DIMENSIONAL METHOD
LOAD CASE NO. 1
DEFLECTION, IN
-1.5 -1. -.5 0. .5 1. 1.5
MOMENT, LBS-IN
-2000000 -1000000 0 1000000 2000000
PRESSURE, LBS/IN**2
-14800 -7400 0 7400 14800
0 140 280 420 560 700
DIST. FROM TOP OF PILE, IN

Example 2

2. This example is taken from the example design of a single-pile dolphin at Columbia Lock and Dam on the Ouachita River presented earlier in Appendix B. The analysis presented here is for one particular load case for a single-pile dolphin as shown in Figure D2. Pile properties and soil stratification are shown in Figure D3.

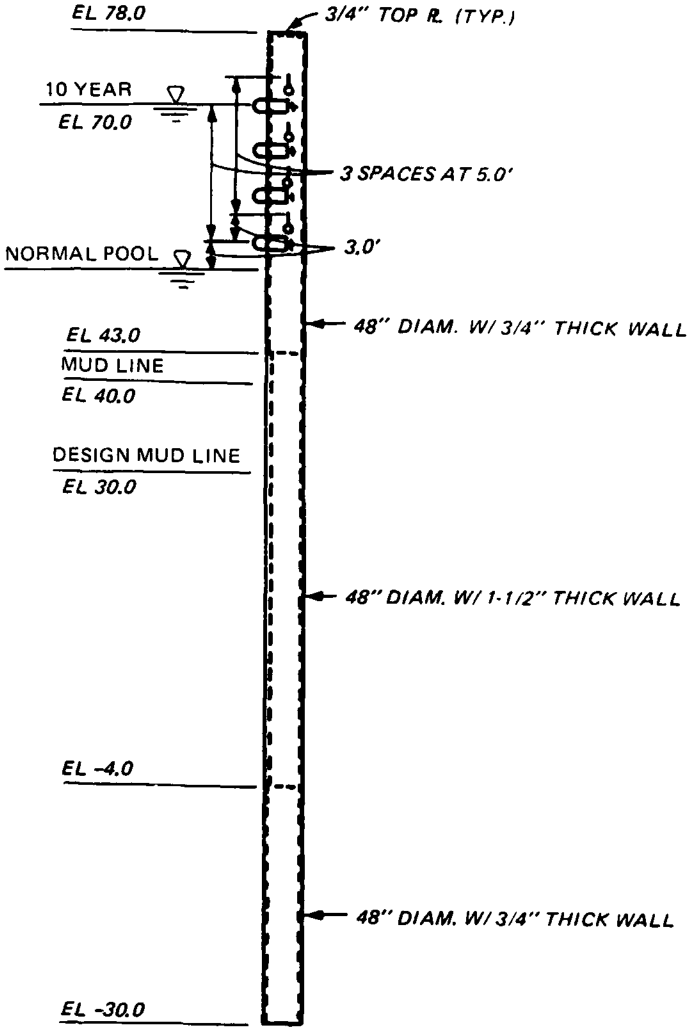


Figure D2. Example design problem; single-pile mooring dolphin

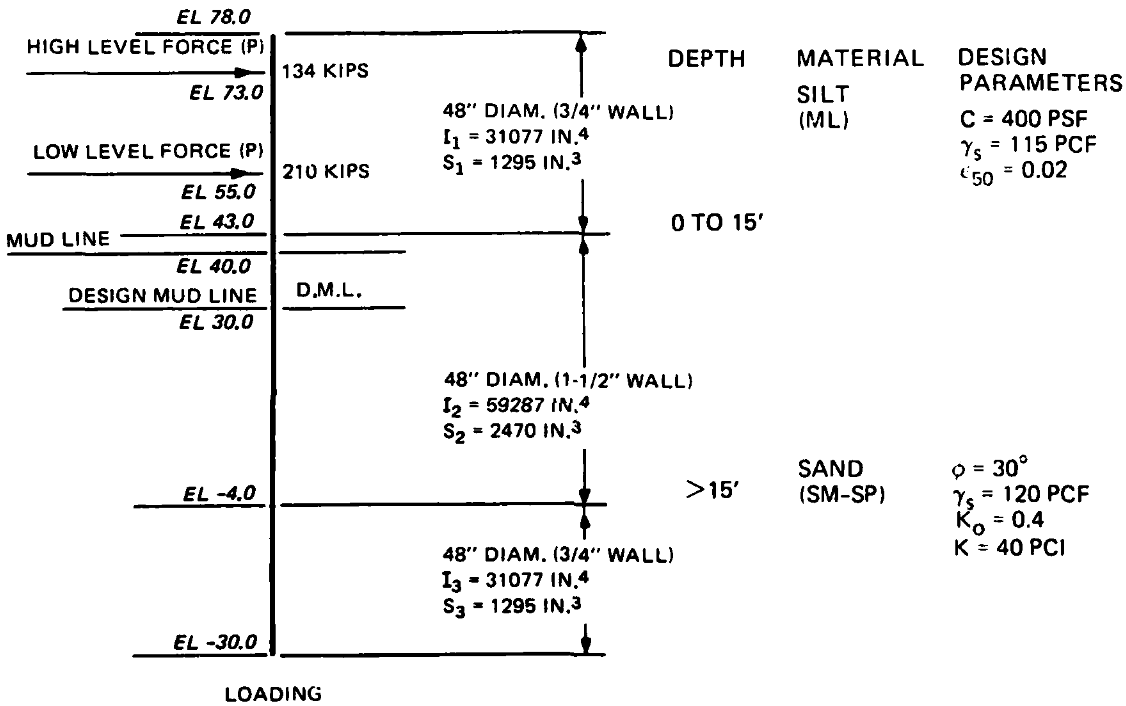


Figure D3. Pile and soil properties; single-pile mooring dolphin

```
010 TITLE
020 COLUMBIA LOCK & DAM - SINGLE PILE DOLPHIN
030 UNITS
040 ENGL
050 PILE 100 3 1236 29.E6 516 (PILE PROPERTIES-NI,NDIAM,LENGTH,EPILE,XGS)
070 0 48 31077   )
080 360 48 59287 } <-------- XDIAN(I),DIAM(I),MINERT(I)
090 924 48 31077 )              where I=1,NDIAM
100 SOIL 2                   (SOIL DESCRIPTION-NL)
120 1 1 516 696 25   } <---- (LAYER(I),KSOIL(I),XTOP(I),XBOT(I),K(I)
130 2 4 696 1240 40  }           where I=1,NL)
140 WEIGHT 4                 (UNIT WEIGHT PROFILE-NGI)
160 516 .0304   )
170 696 .0304   }
180 696 .0333   } <--------- (XG1(I),GAM1(I) where I=1,NGI)
190 1240 .0333  )
200 STRENGTH 4               (SOIL STRENGTH PROFILE-NSTR
220 516 2.778 0 .02  )
230 696 2.778 0 .02  } <----(XSTR(I),C1(I), PHI1(I),EE50(I) where I=1,NSTR)
240 696 0 30 .01     }
250 1240 0 30 .01    )
260 OUTPUT 1 2 1 10          (OUTPUT CONTROL-KOUTPT,INC,KPYOP,NNSUB)
280 516 540 564 588 612 636 695 708 1116 1236 (XNSUB(I)....XNSUB(NNSUB))
290 BOUN 1 1                 (BOUNDARY CONDITION AT PILEHEAD-KBC,NRUN)
310 1 134000 0 0             (KOPSUB(I),PTSUB(I),BC2SUB(I),PXSUB(I),
                                 where I=1,NRUN)
330 CYCLIC 0 0               (CYCLIC LOAD INDICATOR-KCYCL,RCYCL)
350 CONTROL 100 .001 100     (PROGRAM CONTROL-MAXIT,YTOL,EXDEFL)
370 END
```

(Input Echo for Mooring Dolphin Analysis)

***** UNIT DATA. *****

SYSTEM OF UNITS
(UP TO 16 CHAR.)
ENGL

***** PILE DATA. *****

NO. INCREMENTS PILE IS DIVIDED	NO. SEGMENTS WITH DIFFERENT CHARACTERISTICS	LENGTH OF PILE	MODULUS OF ELASTICITY	DEPTH
100	3	0.124E 04	0.290E 08	0.516E 03

TOP OF SEGMENT	DIAMETER OF PILE	MOMENT OF INERTIA	CROSS-SECT. AREA
0.	0.480E 02	0.311E 05	0.111E 03
0.360E 03	0.480E 02	0.593E 05	0.219E 03
0.924E 03	0.480E 02	0.311E 05	0.111E 03

***** SOIL DATA. *****

NUMBER OF LAYERS
2

LAYER NUMBER	P-Y CURVE CONTROL CODE	TOP OF LAYER	BOTTOM OF LAYER	INITIAL SOIL MODULI CONST.	FACTOR "A"	FACTOR "F"
1	1	0.516E 03	0.696E 03	0.250E 02	0.	0.
2	4	0.696E 03	0.124E 04	0.400E 02	0.	0.

***** UNIT WEIGHT DATA. *****

NO. POINTS FOR PLOT
OF EFF. UNIT WEIGHT
VS. DEPTH
4

DEPTH BELOW TOP TO POINT	EFFECTIVE UNIT WEIGHT

0.516E 03	0.304E-01
0.696E 03	0.304E-01
0.696E 03	0.333E-01
0.124E 04	0.333E-01

***** PROFILE DATA. *****

NO. POINTS FOR
STRENGTH PARAMETERS
VS. DEPTH
4

DEPTH BELOW TOP OF PILE	UNDRAINED SHEAR STRENGTH OF SOIL	ANGLE OF INTERNAL FRICTION IN RADIANS	STRAIN AT 50% STRESS LEVEL
0.516E 03	0.278E 01	0.	0.200E-01
0.696E 03	0.278E 01	0.	0.200E-01
0.696E 03	0.	0.524E 00	0.100E-01
0.124E 04	0.	0.524E 00	0.100E-01

***** P-Y DATA. *****

NO. OF
P-Y CURVES
0

***** OUTPUT DATA. *****

DATA OUTPUT CODE	OUTPUT INCREMENT CODE	P-Y PRINTOUT CODE	NO. DEPTHS TO PRINT FOR P-Y CURVES
1	2	1	10

DEPTH FOR PRINTING P-Y CURVES
0.516E 03
0.540E 03
0.564E 03
0.588E 03
0.612E 03
0.636E 03
0.695E 03
0.708E 03
0.112E 04
0.124E 04

***** PILE HEAD (BOUNDARY) DATA. *****

BOUNDARY CONDITION CODE	NO. OF SETS OF BOUNDARY CONDITIONS
1	1

PILE HEAD PRINTOUT CODE	LATERAL LOAD AT TOP OF PILE	VALUE OF SECOND BOUNDARY CONDITION	AXIAL LOAD ON PILE
1	0.134E 06	0.	0.

***** CYCLIC DATA. *****

CYCLIC(0) OR STATIC(1) LOADING	NO. CYCLES OF LOADING
0	0.100E 03

***** PROGRAM CONTROL DATA. *****

MAX. NO. OF ITERATIONS	TOLERENCE ON SOLUTION CONVERGENCE	PILE HEAD DEFLECTION FLAG(STOPS RUN)
100	0.100E-02	0.100E 03

***** LOAD DATA. *****

BOUNDARY SET NO.	NO. POINTS FOR DISTRIB. LATERAL LOAD VS. DEPTH
1	0

(P-Y curves for Mooring Dolphin Analysis)

GENERATED P-Y CURVES

THE NUMBER OF CURVES = 10
THE NUMBER OF POINTS ON EACH CURVE = 17

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
0.	48.000	0.3E 01	0.3E-01	0.200E-01

Y, IN	P, LBS/IN
0.	0.
0.019	40.003
0.600	126.002
1.200	158.753
1.800	181.727
2.400	200.016
3.000	215.461
3.600	228.961
4.200	241.034
4.800	252.004
5.400	262.095
6.000	271.463
6.600	280.226
7.200	288.473
19.200	168.013
36.000	0.000
48.000	0.

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
24.00	48.000	0.3E 01	0.3E-01	0.200E-01

Y, IN	P, LBS/IN
0.	0.
0.019	46.839
0.600	147.533
1.200	185.880
1.800	212.780
2.400	234.194
3.000	252.278
3.600	268.086
4.200	282.221
4.800	295.066
5.400	306.881
6.000	317.851
6.600	328.111
7.200	337.767

Y, IN	P, LBS/IN
19.200	208.729
36.000	28.813
48.000	28.813

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
48.00	48.000	0.3E 01	0.3E-01	0.200E-01

Y, IN	P, LBS/IN
0.	0.
0.019	53.675
0.600	169.064
1.200	213.008
1.800	243.833
2.400	268.373
3.000	289.096
3.600	307.210
4.200	323.408
4.800	338.129
5.400	351.668
6.000	364.238
6.600	375.996
7.200	387.061
19.200	252.949
36.000	66.037
48.000	66.037

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
72.00	48.000	0.3E 01	0.3E-01	0.200E-01

Y, IN	P, LBS/IN
0.	0.
0.019	60.510
0.600	190.595
1.200	240.135
1.800	274.886
2.400	302.551
3.000	325.913
3.600	346.335
4.200	364.596
4.800	381.191
5.400	396.454
6.000	410.625
6.600	423.880
7.200	436.354
19.200	300.673
36.000	111.671
48.000	111.671

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50

96.00	48.000	0.3E 01	0.3E-01	0.200E-01

Y,IN	P,LBS/IN
0.	0.
0.019	67.346
0.600	212.126
1.200	267.262
1.800	305.939
2.400	336.730
3.000	362.731
3.600	385.459
4.200	405.783
4.800	424.253
5.400	441.241
6.000	457.012
6.600	471.765
7.200	485.648
19.200	351.901
36.000	165.715
48.000	165.715

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
120.00	48.000	0.3E 01	0.3E-01	0.200E-01

Y,IN	P,LBS/IN
0.	0.
0.019	74.182
0.600	233.657
1.200	294.390
1.800	336.992
2.400	370.908
3.000	399.549
3.600	424.584
4.200	446.971
4.800	467.315
5.400	486.027
6.000	503.400
6.600	519.649
7.200	534.942
19.200	406.633
36.000	228.169
48.000	228.169

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
179.00	48.000	0.3E 01	0.3E-01	0.200E-01

Y,IN	P,LBS/IN
0.	0.
0.019	90.986
0.600	286.588
1.200	361.078

1.800	413.331
2.400	454.930
3.000	490.058
3.600	520.765
4.200	548.223
4.800	573.176
5.400	596.127
6.000	617.435
6.600	637.366
7.200	656.122
19.200	556.079
36.000	417.451
48.000	417.451

DEPTH IN	DIAM IN	PHI DEG	GAMMA LBS/IN**3	A	B	PCT	PCD
192.00	48.00	30.0	0.3E-01	0.90	0.55	0.29E 04	0.81E 04

Y IN	P LBS/IN
0.	0.
0.067	451.195
0.133	642.120
0.200	789.337
0.267	913.835
0.333	1023.773
0.400	1123.348
0.467	1215.056
0.533	1300.527
0.600	1380.895
0.667	1456.986
0.733	1529.424
0.800	1598.696
1.800	2616.048
17.200	2616.048
32.600	2616.048
48.000	2616.048

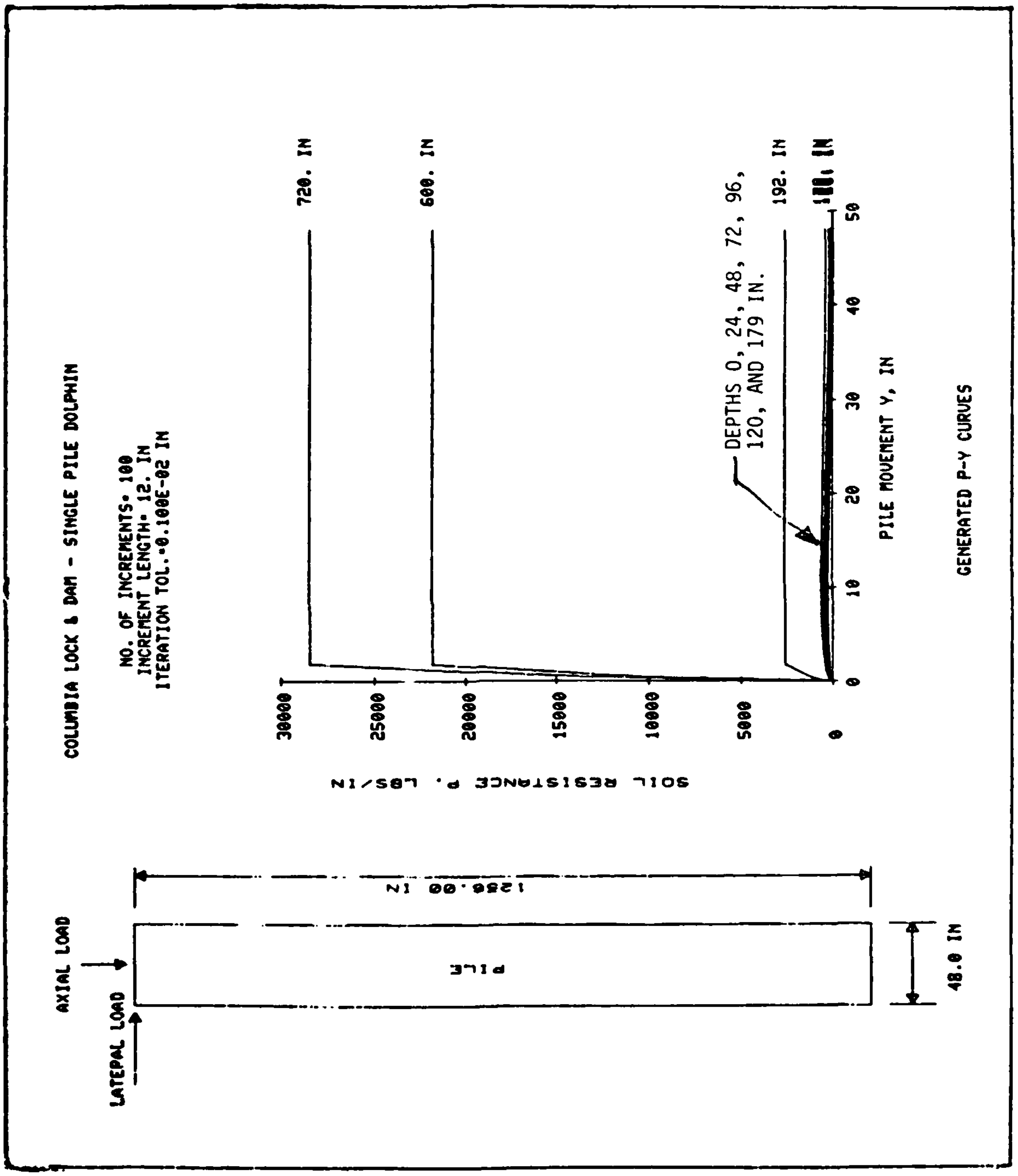
DEPTH IN	DIAM IN	PHI DEG	GAMMA LBS/IN**3	A	B	PCT	PCD
600.00	48.00	30.0	0.3E-01	0.88	0.55	0.25E 05	0.27E 05

Y IN	P LBS/IN
0.	0.
0.067	1600.000
0.133	3200.000
0.200	4800.000
0.267	6400.000
0.333	8000.000
0.400	9600.000
0.467	10534.620
0.533	11231.946

0.600	11885.247
0.667	12501.779
0.733	13087.006
0.800	13645.166
1.800	21832.265
17.200	21832.265
32.600	21832.265
48.000	21832.265

DEPTH	DIAM	PHI	GAMMA	A	B	PCT	PCD
IN	IN	DEG	LBS/IN**3				
720.00	48.00	30.0	0.3E-01	0.88	0.55	0.35E 05	0.32E 05

Y	P
IN	LBS/IN
0.	0.
0.067	1920.000
0.133	3840.000
0.200	5760.000
0.267	7680.000
0.333	9600.000
0.400	11520.000
0.467	13440.000
0.533	14650.798
0.600	15502.955
0.667	16307.151
0.733	17070.513
0.800	17798.569
1.800	28477.711
17.200	28477.711
32.600	28477.711
48.000	28477.711


COLUMBIA LOCK & DAM - SINGLE PILE DOLPHIN
NO. OF INCREMENTS= 100
INCREMENT LENGTH= 12. IN
ITERATION TOL.=0.100E-02 IN
720. IN
600. IN
192. IN
DEPTHS 0, 24, 48, 72, 96, 120, AND 179 IN.
SOIL RESISTANCE P, LBS/IN
PILE MOVEMENT Y, IN
GENERATED P-Y CURVES
AXIAL LOAD
LATERAL LOAD
PILE
1256.00 IN
48.0 IN

COLUMBIA LOCK & DAM - SINGLE PILE DOLPHIN
LOADING CONDITIONS

LOAD CASE.NO.	LATERAL LOAD AT PILE HEAD(LBS)	AXIAL LOAD AT PILE HEAD(LBS)	APPLIED MOMENT AT PILE HEAD(LBS-IN)
1	134000.	0.	0.

COLUMBIA LOCK & DAM - SINGLE PILE DOLPHIN

UNITS--ENGL

O U T P U T   I N F O R M A T I O N
****

NO. OF ITERATIONS = 11
MAXIMUM DEFLECTION ERROR = 0.410E-03 IN

PILE LOADING CONDITION
LATERAL LOAD AT PILE HEAD = 0.134E 06 LBS
APPLIED MOMENT AT PILE HEAD = 0. LBS-IN
AXIAL LOAD AT PILE HEAD = 0. LBS

X IN	DEFLEC IN	MOMENT LBS-IN	TOTAL STRESS LBS/IN**2	DISTR. LOAD LBS/IN	SOIL MODULUS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2
*******	*********	*********	*********	*********	*********	*********
0.	0.199E 02	0.	0.	0.	0.	0.901E 12
24.72	0.190E 02	0.331E 07	0.256E 04	0.	0.	0.901E 12
49.44	0.182E 02	0.662E 07	0.512E 04	0.	0.	0.901E 12
74.16	0.173E 02	0.994E 07	0.767E 04	0.	0.	0.901E 12
98.88	0.164E 02	0.132E 08	0.102E 05	0.	0.	0.901E 12
123.60	0.156E 02	0.166E 08	0.128E 05	0.	0.	0.901E 12
148.32	0.147E 02	0.199E 08	0.153E 05	0.	0.	0.901E 12
173.04	0.139E 02	0.232E 08	0.179E 05	0.	0.	0.901E 12
197.76	0.131E 02	0.265E 08	0.205E 05	0.	0.	0.901E 12
222.48	0.123E 02	0.298E 08	0.230E 05	0.	0.	0.901E 12
247.20	0.115E 02	0.331E 08	0.256E 05	0.	0.	0.901E 12
271.92	0.107E 02	0.364E 08	0.281E 05	0.	0.	0.901E 12
296.64	0.100E 02	0.397E 08	0.307E 05	0.	0.	0.901E 12
321.36	0.929E 01	0.431E 08	0.333E 05	0.	0.	0.901E 12
346.08	0.862E 01	0.464E 08	0.358E 05	0.	0.	0.901E 12
370.80	0.797E 01	0.497E 08	0.201E 05	0.	0.	0.172E 13
395.52	0.734E 01	0.530E 08	0.215E 05	0.	0.	0.172E 13
420.24	0.673E 01	0.563E 08	0.228E 05	0.	0.	0.172E 13
444.96	0.614E 01	0.596E 08	0.241E 05	0.	0.	0.172E 13
469.68	0.557E 01	0.629E 08	0.255E 05	0.	0.	0.172E 13
494.40	0.503E 01	0.662E 08	0.268E 05	0.	0.	0.172E 13
519.12	0.450E 01	0.696E 08	0.282E 05	0.	0.560E 02	0.172E 13
543.84	0.400E 01	0.728E 08	0.295E 05	0.	0.710E 02	0.172E 13
568.56	0.353E 01	0.758E 08	0.307E 05	0.	0.885E 02	0.172E 13

593.28	0.309E 01	0.786E 08	0.318E 05	0.	0.109E 03	0.172E 13
618.00	0.267E 01	0.812E 08	0.329E 05	0.	0.134E 03	0.172E 13
642.72	0.228E 01	0.836E 08	0.339E 05	0.	0.164E 03	0.172E 13
667.44	0.192E 01	0.858E 08	0.347E 05	0.	0.201E 03	0.172E 13
692.16	0.159E 01	0.878E 08	0.355E 05	0.	0.247E 03	0.172E 13
716.88	0.130E 01	0.892E 08	0.361E 05	0.	0.176E 04	0.172E 13
741.60	0.103E 01	0.892E 08	0.361E 05	0.	0.238E 04	0.172E 13
766.32	0.798E 00	0.877E 08	0.355E 05	0.	0.326E 04	0.172E 13
791.04	0.595E 00	0.847E 08	0.343E 05	0.	0.453E 04	0.172E 13
815.76	0.422E 00	0.800E 08	0.324E 05	0.	0.636E 04	0.172E 13
840.48	0.278E 00	0.737E 08	0.298E 05	0.	0.916E 04	0.172E 13
865.20	0.160E 00	0.658E 08	0.266E 05	0.	0.140E 05	0.172E 13
889.92	0.657E-01	0.566E 08	0.229E 05	0.	0.150E 05	0.172E 13
914.64	-0.886E-02	0.468E 08	0.189E 05	0.	0.159E 05	0.172E 13
939.36	-0.656E-01	0.371E 08	0.286E 05	0.	0.169E 05	0.901E 12
964.08	-0.994E-01	0.281E 08	0.217E 05	0.	0.179E 05	0.901E 12
988.80	-0.114E 00	0.201E 08	0.155E 05	0.	0.189E 05	0.901E 12
1013.52	-0.115E 00	0.134E 08	0.104E 05	0.	0.199E 05	0.901E 12
1038.24	-0.107E 00	0.818E 07	0.631E 04	0.	0.209E 05	0.901E 12
1062.96	-0.933E-01	0.427E 07	0.330E 04	0.	0.219E 05	0.901E 12
1087.68	-0.766E-01	0.162E 07	0.125E 04	0.	0.229E 05	0.901E 12
1112.40	-0.588E-01	0.296E 05	0.229E 02	0.	0.239E 05	0.901E 12
1137.12	-0.409E-01	-0.703E 06	0.543E 03	0.	0.248E 05	0.901E 12
1161.84	-0.235E-01	-0.816E 06	0.630E 03	0.	0.258E 05	0.901E 12
1186.56	-0.658E-02	-0.558E 06	0.431E 03	0.	0.268E 05	0.901E 12
1211.28	0.994E-02	-0.194E 06	0.150E 03	0.	0.278E 05	0.901E 12
1236.00	0.263E-01	0.	0.	0.	0.288E 05	0.901E 12

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = 0.750E 00 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = -0.567E-01 LBS

COMPUTED LATERAL FORCE AT PILE HEAD = 0.13400E 06 LBS
COMPUTED MOMENT AT PILE HEAD = 0. IN-LBS
COMPUTED SLOPE AT PILE HEAD = -0.35652E-01

THE OVERALL MOMENT IMBALANCE = -0.922E 00 IN-LBS
THE OVERALL LATERAL FORCE IMBALANCE = -0.111E-06 LBS

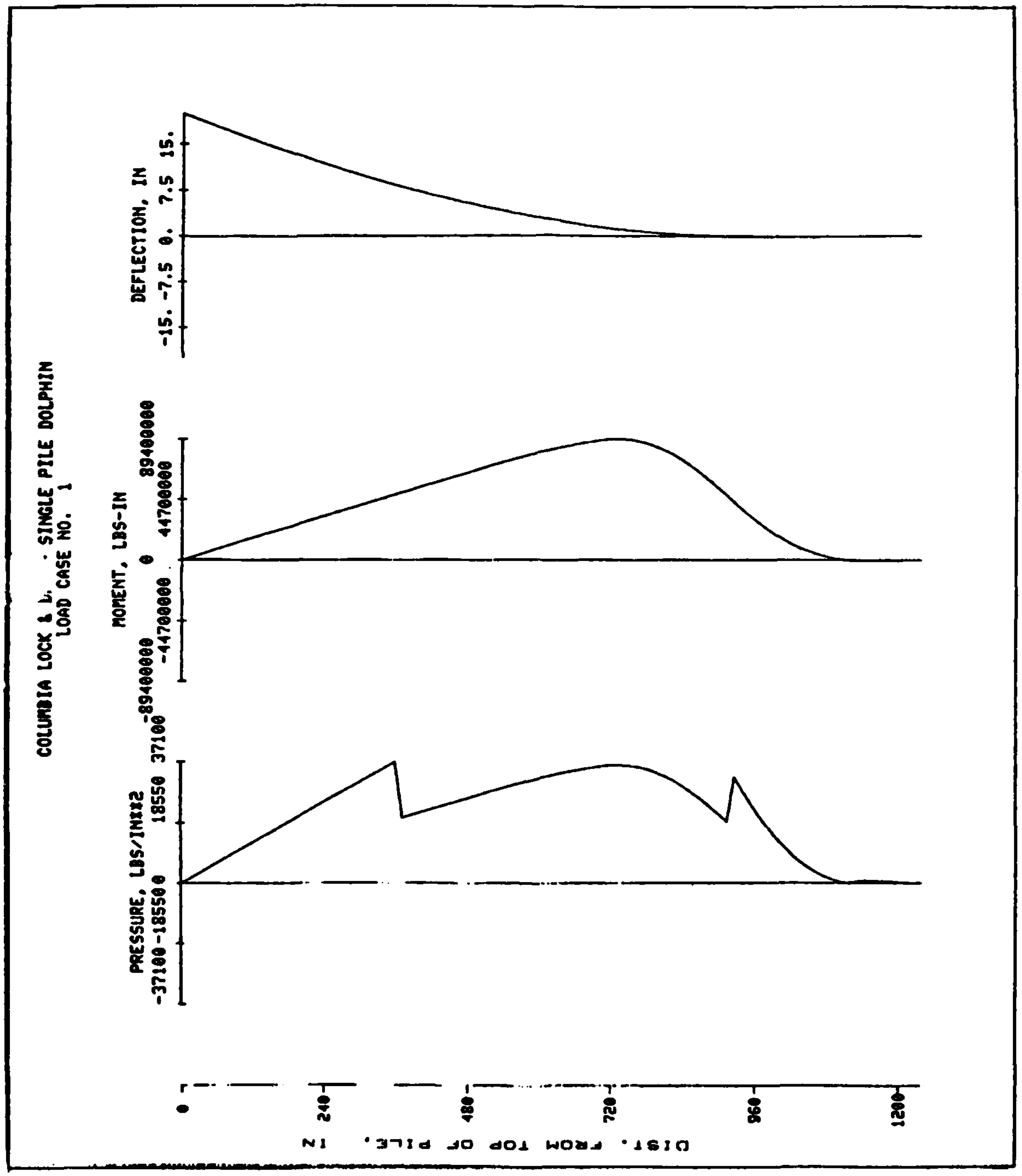
OUTPUT SUMMARY

PILE HEAD DEFLECTION = 0.199E 02 IN
MAXIMUM BENDING MOMENT = 0.894E 08 IN-LBS
MAXIMUM TOTAL STRESS = 0.371E 05 LBS/IN**2
MAXIMUM SHEAR FORCE = 0.134E 06 LBS

COLUMBIA LOCK & DAM - SINGLE PILE DOLPHIN

S U M M A R Y   T A B L E
****

LATERAL LOAD (LBS)	BOUNDARY CONDITION BC2	AXIAL LOAD (LBS)	YT (IN)	ST (IN/IN)	MAX. MOMENT (IN-LBS)	MAX. STRESS (LBS/IN**2)
0.134E 06	0.	0.	0.199E 02	-0.357E-01	0.894E 08	0.371E 05


COLUMBIA LOCK & D. · SINGLE PILE DOLPHIN
LOAD CASE NO. 1
DEFLECTION, IN
-15. -7.5 0. 7.5 15.
MOMENT, LBS-IN
-89400000 -44700000 0 44700000 89400000
PRESSURE, LBS/IN**2
-37100 -18550 0 18550 37100
DIST. FROM TOP OF PILE, IN
0 240 480 720 960 1200

Example 3

3. The pile shown in Figure D4 will be analyzed under various loads and pile head boundary conditions. The soil profile used is shown in Figure D5. Four variations will be analyzed in a single run.

Free-head pile: p-y curves by soft clay criteria, Example 3a

4. The pile is treated as a free-head pile with an applied moment of 300,000 in.-lb. Lateral loads of 25,000, 30,000, and 35,000 lb, along with an axial load of 15,000 lb, will be analyzed. p-y curves will be generated internally using the soft clay criteria and cyclic loading. The strain at 50 percent of the maximum deviator stress is assumed to be a constant 0.02 to a depth of 336 in. and to decrease linearly to 0.01 at a depth of 1176 in.

Free-head pile: p-y curves by unified criteria, Example 3b

5. This problem is identical with Example 3a except that the p-y curves will be generated by the unified criteria with cyclic loading, and a lateral load of 25,000 lb will be analyzed. Values of $A = 2.5$, $F = 1.0$, and $k = 116$ pci are assumed. Output will include points on the p-y curves at x coordinates of 96, 120, 144, 192, 240, 336, 576, and 960 in.

Fixed-head pile: p-y curves by unified criteria, Example 3c

6. This problem is identical with Example 3b for unified criteria except that the pile head is fixed against rotation. A p-y curve will be output at a depth of $x = 576$ in. for verification.

Rotational restraint at pile head of 1.5×10^6 in.-lb, Example 3d

7. This problem is identical with Example 3b for unified criteria except that the boundary condition at the pile head will be one of rotational restraint with $M_t/S_t = 1.5 \times 10^6$ in.-lb. A p-y curve will be output at a depth of $x = 576$ in. for verification.

Comparison of Examples 3a, 3b, 3c, and 3d

8. Comparisons between soil resistance, moment, and deflection for examples 3a, 3b, 3c, and 3d for a lateral load of 25,000 lb are shown in Figure D6.

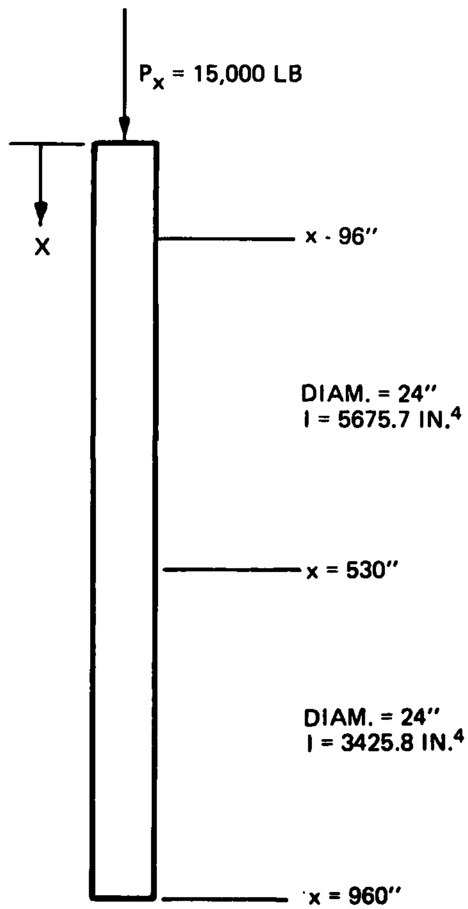


Figure D4. Pile properties for example problems

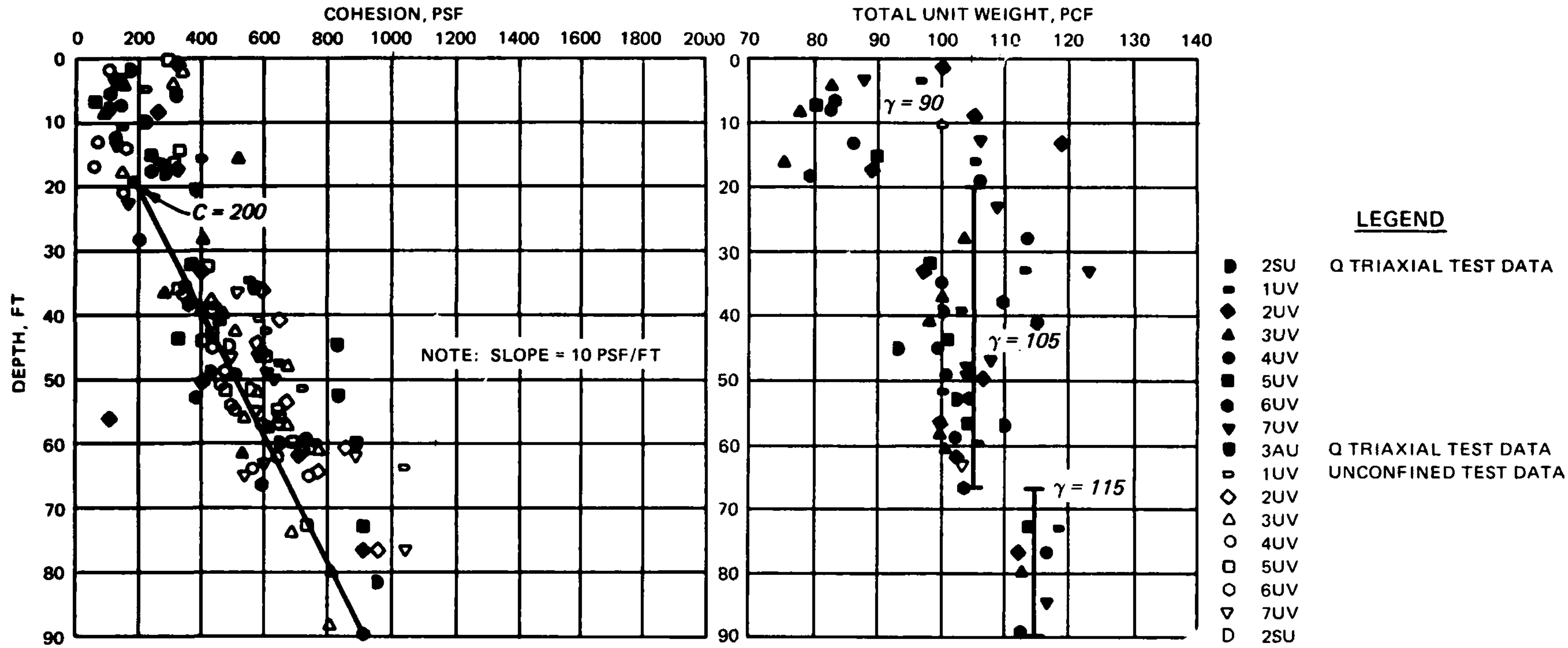


Figure D5. Soil profile used in example problem

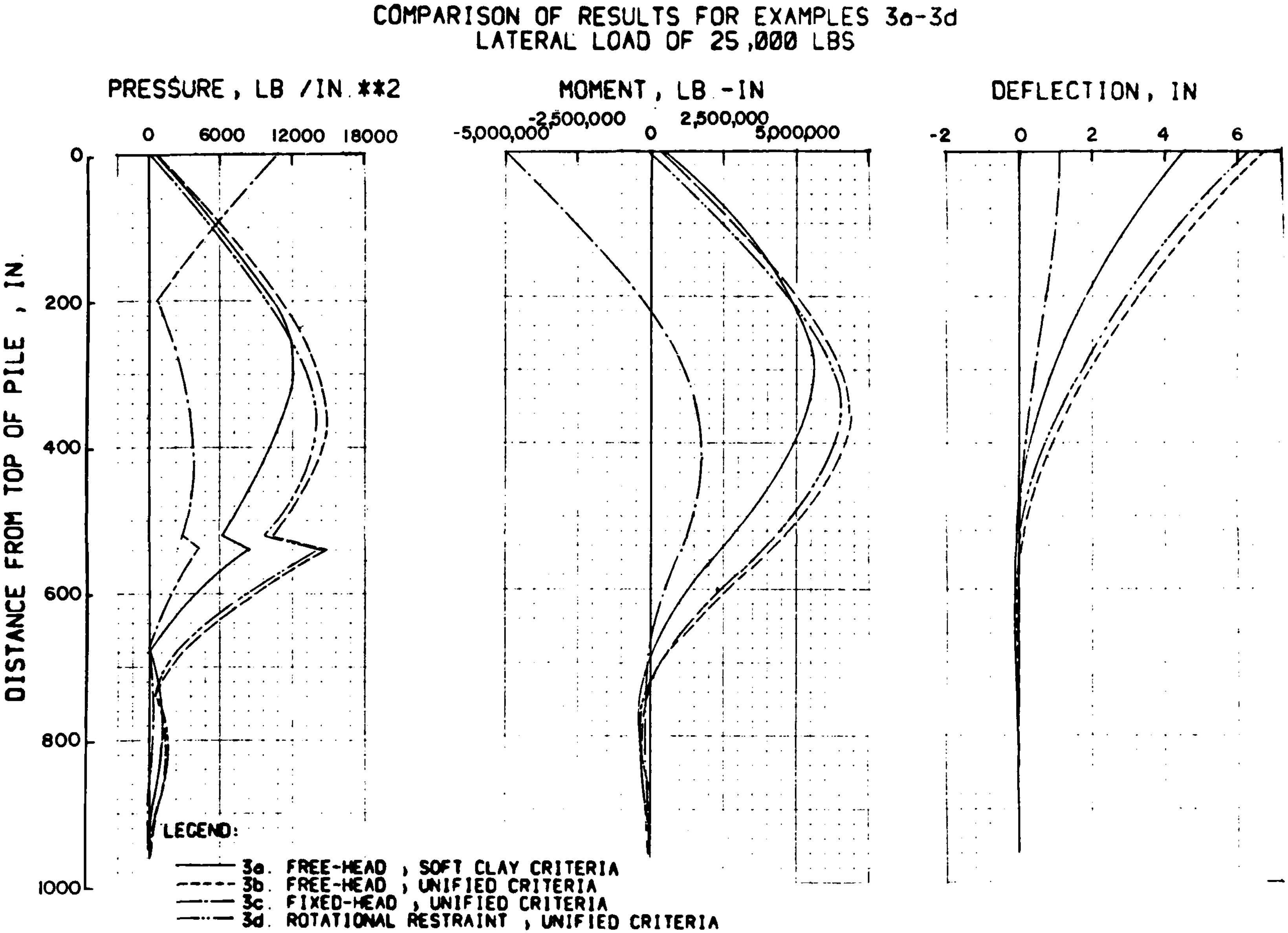


Figure D6. Comparison of results for Examples 3a-3d, lateral load of 25,000 lb

```
10 TITLE
20 FREE HEAD PILE - P-Y CURVES BY SOFT CLAY CRITERIA
30 UNITS
40 ENGL
50 PILE 96 2 960 29.E6 96 (Pile properties - NI,NDIAM,LENGTH,EPILE,XGS)
60 0 24 5675.7  (XDIAM(I),DIAM(I),MINERT(I)
70 530 24 3425.8  Where I = 1,NDIAM
80 SOIL 1         (Soil Description - NL)
90 1 1 96 1176 116  (LAYER(I),KSOIL(I),XTOP(I),XBOT(I),K(I) where I = 1,NL)
100 WEIGHT 6         (Unit Weight Profile - NGI)
110 96   .0159
120 336 .0159
130 336 .0246        XG1(I), GAM1(I)
140 900 .0246        Where I = 1,NGI
150 900 .0304
160 1176 .0304
170 STRENGTH 3  (Soil Strength Profile - NSTR)
180   96 1.389 0.0 .02   XSTR(I),C1(I),PHI1(I),EE50(I)
190  336 1.389 0.0 .02      Where I = 1,NSTR
200 1176 6.250 0.0 .01
210 BOUNDARY 1 3        (Boundary Condition at Pile Head - KBC,NRUN)
220 1 25.E3 3.E5 1.5E4   KOPSUB(I),PTSUB(I),BC2SUB(I),PXSUB(I)
230 1 30.E3 3.E5 1.5E4       Where I = 1,NRUN
240 1 35.E3 3.E5 1.5E4
250 CYCLIC 0 0          (Cyclic Load Indicator - KCYCL,RCYCL)
260 OUTPUT 1 2 1 8      (Output Control - KOUTPT,INC,KPYOP,NNSUB)
270 96 120 144 192 240 336 576 960   (XNSUB(I) .... XNSUB(NNSUB))
280 CONTROL 100 .001 40 (Program Control - MAXIT,YTOL,EXDEFL)
290 END
300 TITLE
310 FREE HEAD PILE - P-Y CURVES BY UNIFIED CRITERIA
320 SOIL 1                     (Soil Description - NL)
330 1 6 96 1176 116 2.5 1.0 (LAYER(I),KSOIL(I),XTOP(I),XBOT(I),K(I) Where I=1,NL)
340 BOUNDARY 1 1            (Boundary Condition at Pile Head - KBC,NRUN)
350 1 25.E3 3.E5 1.5E4      (KOPSUB(I),PTSUB(I),BC2SUB(I),PXSUB(I) where I=1,NRUN)
360 OUTPUT 1 2 1 8          (Output Control - KOUTPT,INC,KPYOP,NNSUB)
370 96 120 144 192 240 336 576 960  (XNSUB(I), ... XNSUB(NNSUB))
380 END
390 TITLE
400 FIXED HEAD PILE -  P-Y CURVES BY UNIFIED CRITERIA
410 BOUNDARY 2 1          (Boundary Condition at Pile Head - KBC,NRUN)
420 1 25.E3 0.0 1.5E4     (KOPSUB(I),PTSUB(I),BC2SUB(I),PXSUB(I) Where I=1,NRUN)
430 OUTPUT 1 2 1 1        (output Control - KOUTPT,INC,KPYOP,NNSUB)
440 576                   (XNSUB(I) ... XNSUB(NNSUB))
450 END
460 TITLE
470 ROTATIONAL RESTRAINT AT PILE HEAD OF 1.5 E6 IN-LBS
480 BOUNDARY 3 1         (Boundary Condition at Pile Head - KBC,NRUN)
490 1 25.E3 1.5E6 1.5E4(KOPSUB(I),PTSUB(I),BC2SUB(I),PXSUB(I) Where I-1,NRUN)
500 END
```

*

(Input Echo for Problem 1 - Free head pile - P-Y curves by Soft Clay Criteria)

***** UNIT DATA. *****

SYSTEM OF UNITS
(UP TO 16 CHAR.)
ENGL

***** PILE DATA. *****

NO. INCREMENTS PILE IS DIVIDED	NO. SEGMENTS WITH DIFFERENT CHARACTERISTICS	LENGTH OF PILE	MODULUS OF ELASTICITY	DEPTH
96	2	0.960E 03	0.290E 08	0.960E 02

TOP OF SEGMENT	DIAMETER OF PILE	MOMENT OF INERTIA	CROSS-SECT. AREA
0.	0.240E 02	0.568E 04	0.872E 02
0.530E 03	0.240E 02	0.343E 04	0.504E 02

***** SOIL DATA. *****

NUMBER OF LAYERS
1

LAYER NUMBER	P-Y CURVE CONTROL CODE	TOP OF LAYER	BOTTOM OF LAYER	INITIAL SOIL MODULI CONST.	FACTOR "A"	FACTOR "F"
1	1	0.960E 02	0.118E 04	0.116E 03	0.	0.

***** UNIT WEIGHT DATA. *****

NO. POINTS FOR PLOT
OF EFF. UNIT WEIGHT
VS. DEPTH
6

DEPTH BELOW TOP TO POINT	EFFECTIVE UNIT WEIGHT
0.960E 02	0.159E-01
0.336E 03	0.159E-01
0.336E 03	0.246E-01
0.900E 03	0.246E-01
0.900E 03	0.304E-01

0.118E 04   0.304E-01

***** PROFILE DATA. *****

NO. POINTS FOR
STRENGTH PARAMETERS
VS. DEPTH
3

DEPTH BELOW TOP OF PILE	UNDRAINED SHEAR STRENGTH OF SOIL	ANGLE OF INTERNAL FRICTION IN RADIANS	STRAIN AT 50% STRESS LEVEL
0.960E 02	0.139E 01	0.	0.200E-01
0.336E 03	0.139E 01	0.	0.200E-01
0.118E 04	0.625E 01	0.	0.100E-01

***** P-Y DATA. *****

NO. OF
P-Y CURVES
0

***** OUTPUT DATA. *****

DATA OUTPUT CODE	OUTPUT INCREMENT CODE	P-Y PRINTOUT CODE	NO. DEPTHS TO PRINT FOR P-Y CURVES
1	2	1	8

DEPTH FOR
PRINTING
P-Y CURVES
0.960E 02
0.120E 03
0.144E 03
0.192E 03
0.240E 03
0.336E 03
0.576E 03
0.960E 03

***** PILE HEAD (BOUNDARY) DATA. *****

BOUNDARY CONDITION CODE	NO. OF SETS OF BOUNDARY CONDITIONS

1	3

PILE HEAD PRINTOUT CODE	LATERAL LOAD AT TOP OF PILE	VALUE OF SECOND BOUNDARY CONDITION	AXIAL LOAD ON PILE
1	0.250E 05	0.300E 06	0.150E 05
1	0.300E 05	0.300E 06	0.150E 05
1	0.350E 05	0.300E 06	0.150E 05

***** CYCLIC DATA. *****

CYCLIC(0) OR STATIC(1) LOADING	NO. CYCLES OF LOADING
0	0.100E 03

***** PROGRAM CONTROL DATA. *****

MAX. NO. OF ITERATIONS	TOLERENCE ON SOLUTION CONVERGENCE	PILE HEAD DEFLECTION FLAG(STOPS RUN)
100	0.100E-02	0.400E 02

***** LOAD DATA. *****

BOUNDARY SET NO.	NO. POINTS FOR DISTRIB. LATERAL LOAD VS. DEPTH
1	0

BOUNDARY SET NO.	NO. POINTS FOR DISTRIB. LATERAL LOAD VS. DEPTH
2	0

BOUNDARY SET NO.	NO. POINTS FOR DISTRIB. LATERAL LOAD VS. DEPTH
3	0

(P-Y Curves generated for verification - Problem 1)

GENERATED P-Y CURVES

THE NUMBER OF CURVES = 8
THE NUMBER OF POINTS ON EACH CURVE = 17

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
0.	24.000	0.1E 01	0.2E-01	0.200E-01

Y,IN	P,LBS/IN
0.	0.
0.010	10.001
0.300	31.501
0.600	39.688
0.900	45.432
1.200	50.004
1.500	53.865
1.800	57.240
2.100	60.258
2.400	63.001
2.700	65.524
3.000	67.866
3.300	70.057
3.600	72.118
9.600	42.003
18.000	0.000
24.000	0.

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
24.00	24.000	0.1E 01	0.2E-01	0.200E-01

Y,IN	P,LBS/IN
0.	0.
0.010	12.583
0.300	39.635
0.600	49.937
0.900	57.164
1.200	62.917
1.500	67.775
1.800	72.022
2.100	75.820
2.400	79.271
2.700	82.445
3.000	85.392
3.300	88.148
3.600	90.742
9.600	57.725
18.000	11.699
24.000	11.699

DEPTH	DIAM	C	GAMMA	E50
IN	IN	LBS/IN**2	LBS/IN**3	
48.00	24.000	0.1E 01	0.2E-01	0.200E-01

Y,IN	P,LBS/IN
0.	0.
0.010	15.166
0.300	47.770
0.600	60.187
0.900	68.896
1.200	75.830
1.500	81.686
1.800	86.804
2.100	91.381
2.400	95.540
2.700	99.366
3.000	102.918
3.300	106.240
3.600	109.366
9.600	75.447
18.000	28.199
24.000	28.199

DEPTH	DIAM	C	GAMMA	E50
IN	IN	LBS/IN**2	LBS/IN**3	
96.00	24.000	0.1E 01	0.2E-01	0.200E-01

Y,IN	P,LBS/IN
0.	0.
0.010	20.331
0.300	64.040
0.600	80.685
0.900	92.361
1.200	101.657
1.500	109.506
1.800	116.368
2.100	122.504
2.400	128.080
2.700	133.208
3.000	137.970
3.300	142.423
3.600	146.614
9.600	116.894
18.000	75.606
24.000	75.606

DEPTH	DIAM	C	GAMMA	E50
IN	IN	LBS/IN**2	LBS/IN**3	
144.00	24.000	0.1E 01	0.2E-01	0.200E-01

Y,IN	P,LBS/IN
0.	0.
0.010	25.497
0.300	80.309

0.600	101.183
0.900	115.826
1.200	127.483
1.500	137.327
1.800	145.932
2.100	153.626
2.400	160.619
2.700	167.050
3.000	173.021
3.300	178.606
3.600	183.863
9.600	166.345
18.000	142.222
24.000	142.222

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
240.00	24.000	0.1E 01	0.2E-01	0.200E-01

Y,IN	P,LBS/IN
0.	0.
0.010	30.002
0.300	94.502
0.600	119.065
0.900	136.295
1.200	150.012
1.500	161.596
1.800	171.721
2.100	180.775
2.400	189.003
2.700	196.571
3.000	203.598
3.300	210.170
3.600	216.355
9.600	216.017
18.000	216.017
24.000	216.017

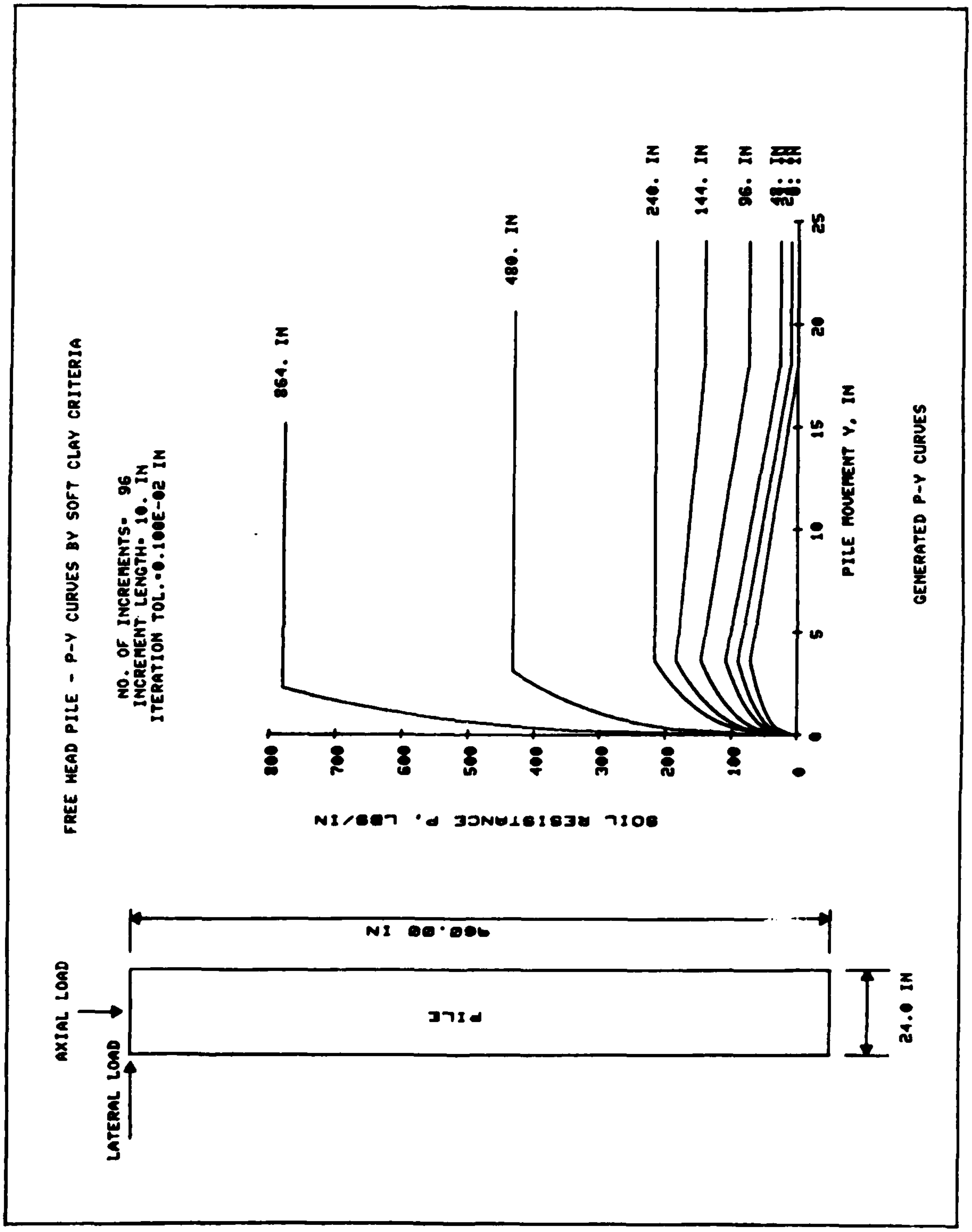
DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
480.00	24.000	0.3E 01	0.2E-01	0.171E-01

Y,IN	P,LBS/IN
0.	0.
0.008	60.002
0.257	188.994
0.514	238.117
0.771	272.576
1.029	300.009
1.286	323.174
1.543	343.424
1.800	361.532
2.057	377.987
2.314	393.122
2.571	407.174
2.829	420.318

3.086	432.687
8.229	432.012
15.429	432.012
20.571	432.012

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
864.00	24.000	0.5E 01	0.2E-01	0.126E-01

Y,IN	P,LBS/IN
0.	0.
0.006	108.001
0.189	340.181
0.377	428.601
0.566	490.625
0.754	540.003
0.943	581.701
1.131	618.149
1.320	650.742
1.509	680.361
1.697	707.604
1.886	732.897
2.074	756.555
2.263	778.819
6.034	777.604
11.314	777.604
15.086	777.604


FREE HEAD PILE - P-Y CURVES BY SOFT CLAY CRITERIA
NO. OF INCREMENTS= 96
INCREMENT LENGTH= 10. IN
ITERATION TOL.=0.100E-02 IN
864. IN
480. IN
240. IN
144. IN
96. IN
SOIL RESISTANCE P, LBS/IN
0
100
200
300
400
500
600
700
800
0
5
10
15
20
25
PILE MOVEMENT Y, IN
GENERATED P-Y CURVES
AXIAL LOAD
LATERAL LOAD
PILE
960.00 IN
24.0 IN

FREE HEAD PILE - P-Y CURVES BY SOFT CLAY CRITERIA
LOADING CONDITIONS

LOAD CASE NO.	LATERAL LOAD AT PILE HEAD(LBS)	AXIAL LOAD AT PILE HEAD(LBS)	APPLIED MOMENT AT PILE HEAD(LBS-IN)
1	25000.	15000.	300000.
2	30000.	15000.	300000.
3	35000.	15000.	300000.

FREE HEAD PILE - P-Y CURVES BY SOFT CLAY CRITERIA

UNITS--ENGL

O U T P U T   I N F O R M A T I O N

****

(Load Case 1 - Problem 1)

NO. OF ITERATIONS = 19
MAXIMUM DEFLECTION ERROR = 0.647E-03 IN

PILE LOADING CONDITION

- LATERAL LOAD AT PILE HEAD = 0.250E 05 LBS
- APPLIED MOMENT AT PILE HEAD = 0.300E 06 LBS-IN
- AXIAL LOAD AT PILE HEAD = 0.150E 05 LBS

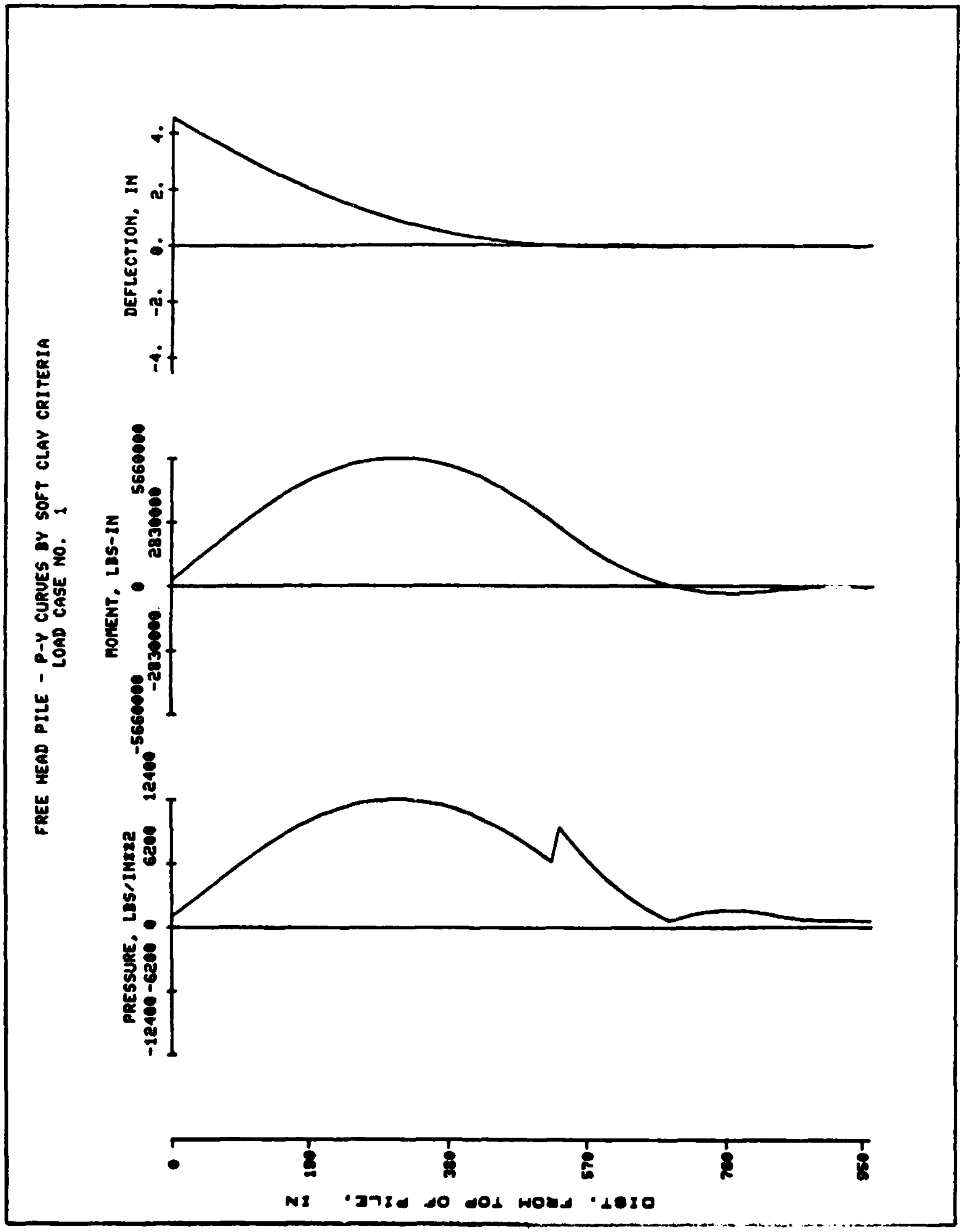
X	DEFLEC	MOMENT	TOTAL STRESS	DISTR. LOAD	SOIL MODULUS	FLEXURAL RIGIDITY
IN	IN	LBS-IN	LBS/IN**2	LBS/IN	LBS/IN**2	LBS-IN**2
*******	*********	*********	*********	*********	*********	*********
0.	0.454E 01	0.300E 06	0.806E 03	0.	0.	0.165E 12
20.00	0.425E 01	0.804E 06	0.187E 04	0.	0.	0.165E 12
40.00	0.397E 01	0.131E 07	0.294E 04	0.	0.	0.165E 12
60.00	0.369E 01	0.181E 07	0.400E 04	0.	0.	0.165E 12
80.00	0.341E 01	0.232E 07	0.507E 04	0.	0.	0.165E 12
100.00	0.314E 01	0.282E 07	0.614E 04	0.	0.229E 02	0.165E 12
120.00	0.287E 01	0.330E 07	0.716E 04	0.	0.293E 02	0.165E 12
140.00	0.261E 01	0.375E 07	0.810E 04	0.	0.365E 02	0.165E 12
↓						↓
820.00	-0.315E-03	-0.181E 06	0.932E 03	0.	0.994E 05	0.993E 11
840.00	0.266E-03	-0.111E 06	0.687E 03	0.	0.129E 06	0.993E 11
860.00	0.396E-03	-0.546E 05	0.489E 03	0.	0.102E 06	0.993E 11
880.00	0.302E-03	-0.141E 05	0.347E 03	0.	0.131E 06	0.993E 11
900.00	0.147E-03	0.107E 05	0.335E 03	0.	0.241E 06	0.993E 11
920.00	0.317E-04	0.213E 05	0.372E 03	0.	0.103E 07	0.993E 11
940.00	-0.123E-05	0.672E 04	0.321E 03	0.	0.576E 08	0.993E 11
960.00	0.633E-09	0.	0.298E 03	0.	0.379E 11	0.993E 11

OUTPUT VERIFICATION

```
THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT      =  0.365E-01 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = -0.378E-02 LBS

COMPUTED LATERAL FORCE AT PILE HEAD              =  0.25000E 05 LBS
    COMPUTED MOMENT AT PILE HEAD                 =  0.30000E 06 IN-LBS
    COMPUTED SLOPE AT PILE HEAD                  = -0.14385E-01

THE OVERALL MOMENT IMBALANCE               = -0.134E-01  IN-LBS
THE OVERALL LATERAL FORCE IMBALANCE        = -0.750E-08 LBS
```

OUTPUT SUMMARY

```
PILE HEAD DEFLECTION    =  0.454E 01 IN
MAXIMUM BENDING MOMENT =  0.566E 07 IN-LBS
MAXIMUM TOTAL STRESS    =  0.121E 05 LBS/IN**2
MAXIMUM SHEAR FORCE     =  0.252E 05 LBS
```


FREE HEAD PILE - P-Y CURVES BY SOFT CLAY CRITERIA
LOAD CASE NO. 1
DEFLECTION, IN
-4.
-2.
0.
2.
4.
MOMENT, LBS-IN
-5660000
-2830000
0
2830000
5660000
PRESSURE, LBS/IN**2
-12400
-6200
0
6200
12400
DIST. FROM TOP OF PILE, IN
0
190
380
570
760
950

(Load Case 2 - Problem 1)

NO. OF ITERATIONS = 14
MAXIMUM DEFLECTION ERROR = 0.855E-03 IN

PILE LOADING CONDITION
LATERAL LOAD AT PILE HEAD = 0.300E 05 LBS
APPLIED MOMENT AT PILE HEAD = 0.300E 06 LBS-IN
AXIAL LOAD AT PILE HEAD = 0.150E 05 LBS

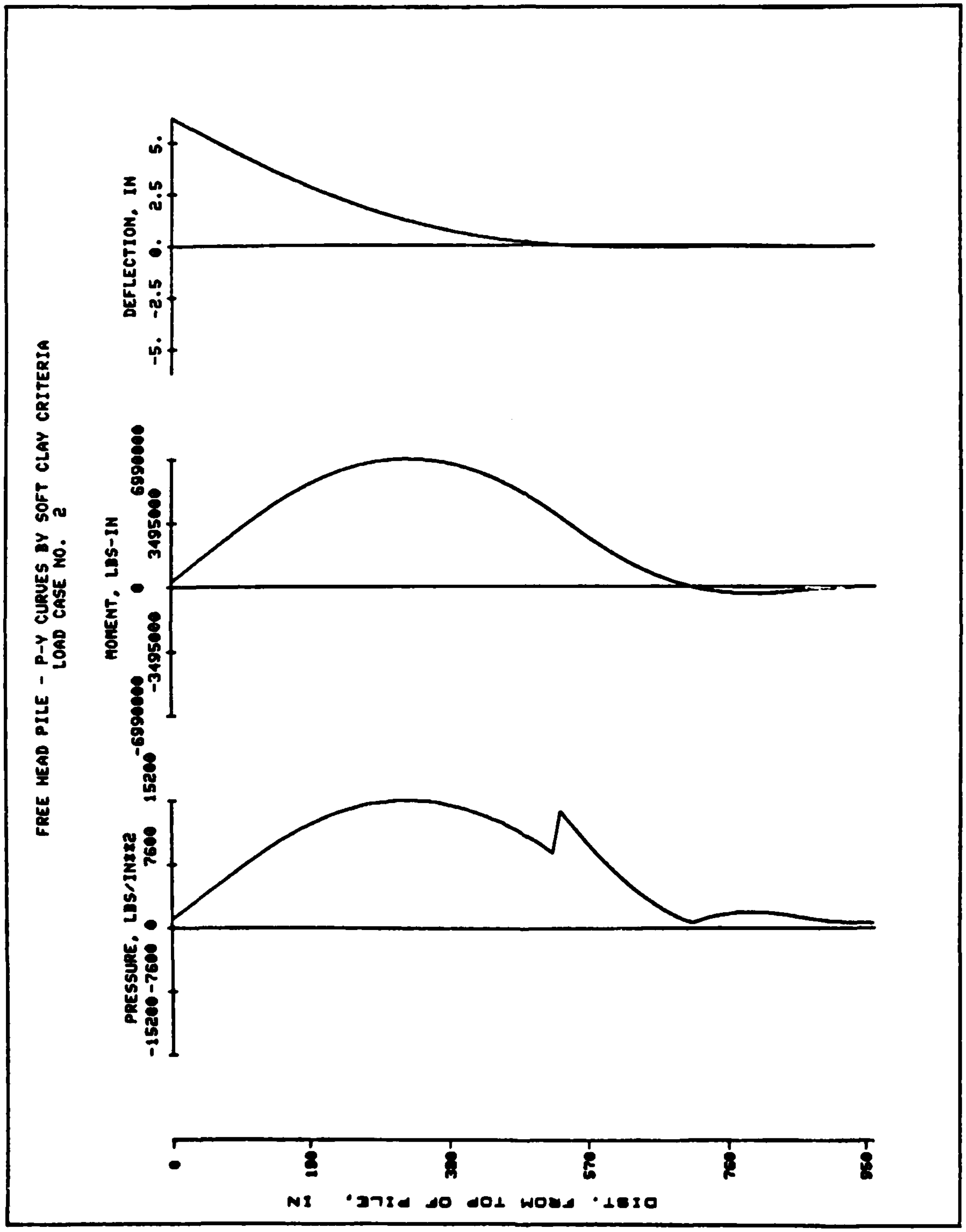
X	DEFLEC	MOMENT	TOTAL STRESS	DISTR. LOAD	SOIL MODULUS	FLEXURAL RIGIDITY
IN	IN	LBS-IN	LBS/IN**2	LBS/IN	LBS/IN**2	LBS-IN**2
*******	*********	*********	*********	*********	*********	*********
0.	0.616E 01	0.300E 06	0.806E 03	0.	0.	0.165E 12
20.00	0.579E 01	0.906E 06	0.209E 04	0.	0.	0.165E 12
40.00	0.542E 01	0.151E 07	0.337E 04	0.	0.	0.165E 12
60.00	0.505E 01	0.212E 07	0.465E 04	0.	0.	0.165E 12
80.00	0.469E 01	0.272E 07	0.593E 04	0.	0.	0.165E 12
100.00	0.434E 01	0.333E 07	0.721E 04	0.	0.165E 02	0.165E 12
120.00	0.399E 01	0.391E 07	0.844E 04	0.	0.222E 02	0.165E 12
140.00	0.365E 01	0.446E 07	0.960E 04	0.	0.290E 02	0.165E 12
↓						↓
820.00	-0.316E-02	-0.329E 06	0.145E 04	0.	0.222E 05	0.993E 11
840.00	-0.893E-03	-0.266E 06	0.123E 04	0.	0.535E 05	0.993E 11
860.00	0.309E-03	-0.183E 06	0.940E 03	0.	0.109E 06	0.993E 11
880.00	0.769E-03	-0.109E 06	0.681E 03	0.	0.622E 05	0.993E 11
900.00	0.785E-03	-0.542E 05	0.488E 03	0.	0.635E 05	0.993E 11
920.00	0.577E-03	-0.188E 05	0.364E 03	0.	0.805E 05	0.993E 11
940.00	0.288E-03	-0.192E 04	0.304E 03	0.	0.132E 06	0.993E 11
960.00	-0.119E-04	0.	0.298E 03	0.	0.959E 06	0.993E 11

OUTPUT VERIFICATION

```
THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT     =  0.358E-01 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = -0.371E-02 LBS

COMPUTED LATERAL FORCE AT PILE HEAD              =  0.30000E 05 LBS
    COMPUTED MOMENT AT PILE HEAD                 =  0.30000E 06 IN-LBS
    COMPUTED SLOPE AT PILE HEAD                  = -0.18615E-01

THE OVERALL MOMENT IMBALANCE               =  0.124E-02 IN-LBS
THE OVERALL LATERAL FORCE IMBALANCE        = -0.995E-08 LBS
```

OUTPUT SUMMARY

```
PILE HEAD DEFLECTION    =  0.616E 01 IN
MAXIMUM BENDING MOMENT  =  0.699E 07 IN-LBS
MAXIMUM TOTAL STRESS    =  0.149E 05 LBS/IN**2
MAXIMUM SHEAR FORCE     =  0.303E 05 LBS
```


FREE HEAD PILE - P-Y CURVES BY SOFT CLAY CRITERIA
LOAD CASE NO. 2
DEFLECTION, IN
-5. -2.5 0. 2.5 5.
MOMENT, LBS-IN
-6990000 -3495000 0 3495000 6990000
PRESSURE, LBS/IN**2
-15200 -7600 0 7600 15200
DIST. FROM TOP OF PILE, IN
0 190 380 570 760 950

(Load Case 3 - Problem 1)

NO. OF ITERATIONS = 18
MAXIMUM DEFLECTION ERROR = 0.754E-03 IN

PILE LOADING CONDITION

LATERAL LOAD AT PILE HEAD = 0.350E 05 LBS
APPLIED MOMENT AT PILE HEAD = 0.300E 06 LBS-IN
AXIAL LOAD AT PILE HEAD = 0.150E 05 LBS

X IN	DEFLEC IN	MOMENT LBS-IN	TOTAL STRESS LBS/IN**2	DISTR. LOAD LBS/IN	SOIL MODULUS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2
*******	*********	*********	*********	*********	*********	*********
0.	0.836E 01	0.300E 06	0.806E 03	0.	0.	0.165E 12
20.00	0.788E 01	0.101E 07	0.230E 04	0.	0.	0.165E 12
40.00	0.740E 01	0.171E 07	0.380E 04	0.	0.	0.165E 12
60.00	0.693E 01	0.242E 07	0.529E 04	0.	0.	0.165E 12
80.00	0.646E 01	0.313E 07	0.679E 04	0.	0.	0.165E 12
100.00	0.601E 01	0.384E 07	0.828E 04	0.	0.105E 02	0.165E 12
120.00	0.556E 01	0.452E 07	0.973E 04	0.	0.144E 02	0.165E 12
140.00	0.512E 01	0.518E 07	0.111E 05	0.	0.191E 02	0.165E 12
880.00	-0.291E-03	-0.252E 06	0.118E 04	0.	0.121E 06	0.993E 11
900.00	0.129E-02	-0.149E 06	0.819E 03	0.	0.456E 05	0.993E 11
920.00	0.227E-02	-0.686E 05	0.538E 03	0.	0.323E 05	0.993E 11
940.00	0.297E-02	-0.178E 05	0.360E 03	0.	0.279E 05	0.993E 11
960.00	0.359E-02	0.	0.298E 03	0.	0.253E 05	0.993E 11

OUTPUT VERIFICATION

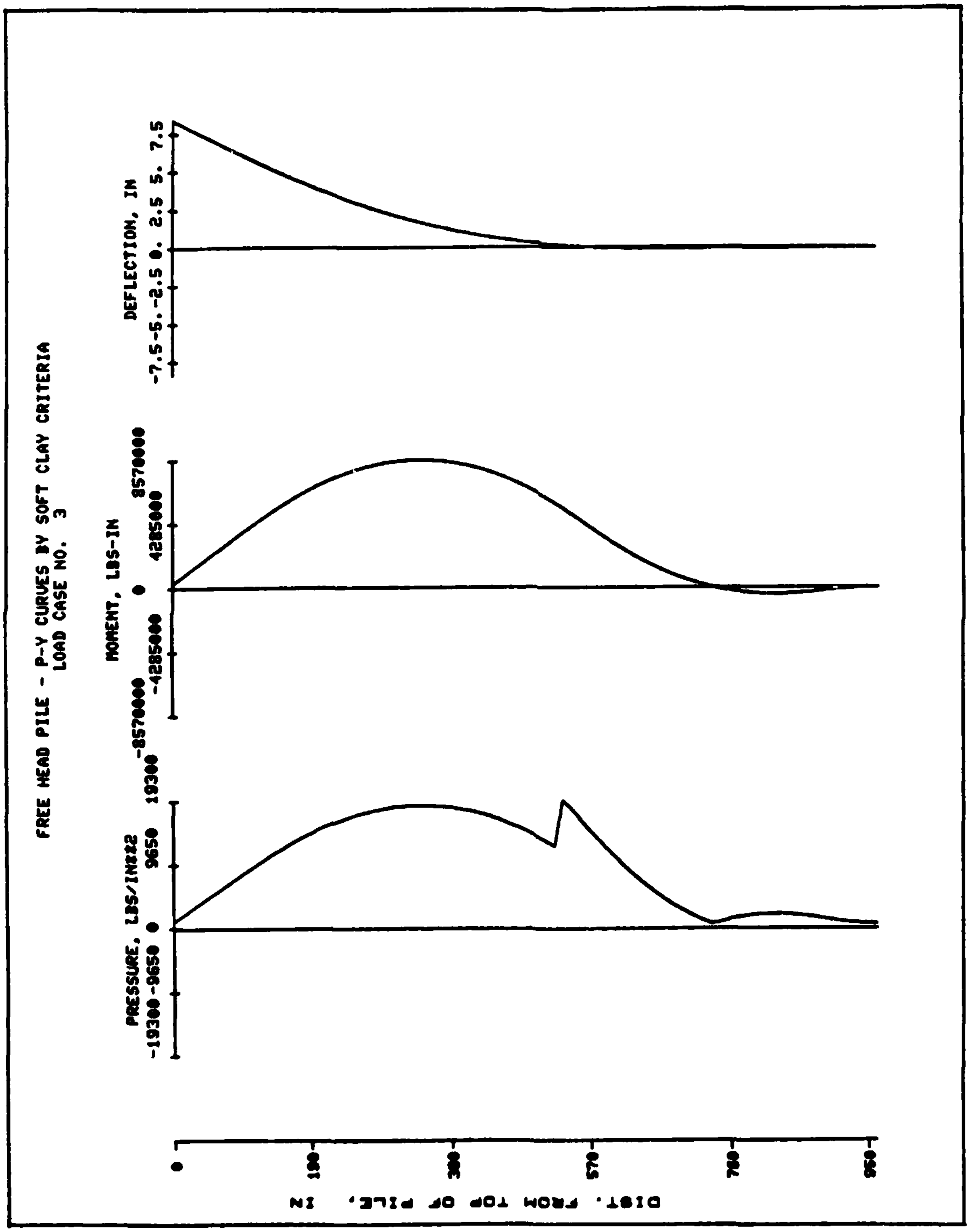
THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = -0.551E-01 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = 0.692E-02 LBS

COMPUTED LATERAL FORCE AT PILE HEAD = 0.35000E 05 LBS
COMPUTED MOMENT AT PILE HEAD = 0.30000E 06 IN-LBS
COMPUTED SLOPE AT PILE HEAD = -0.23999E-01

THE OVERALL MOMENT IMBALANCE = 0.426E-01 IN-LBS
THE OVERALL LATERAL FORCE IMBALANCE = -0.187E-07 LBS

OUTPUT SUMMARY

PILE HEAD DEFLECTION = 0.836E 01 IN
MAXIMUM BENDING MOMENT = 0.857E 07 IN-LBS
MAXIMUM TOTAL STRESS = 0.190E 05 LBS/IN**2
MAXIMUM SHEAR FORCE = 0.354E 05 LBS



FREE HEAD PILE - P-Y CURVES BY SOFT CLAY CRITERIA
LOAD CASE NO. 3
DEFLECTION, IN
-7.5 -5. -2.5 0. 2.5 5. 7.5
MOMENT, LBS-IN
-8570000 -4285000 0 4285000 8570000
PRESSURE, LBS/IN**2
-19300 -9650 0 9650 19300
0 190 380 570 760 950
DIST. FROM TOP OF PILE, IN

FREE HEAD PILE - P-Y CURVES BY SOFT CLAY CRITERIA

S U M M A R Y   T A B L E

****

LATERAL LOAD (LBS)	BOUNDARY CONDITION BC2	AXIAL LOAD (LBS)	YT (IN)	ST (IN/IN)	MAX. MOMENT (IN-LBS)	MAX. STRESS (LBS/IN**2
0.250E 05	0.300E 06	0.150E 05	0.454E 01	-0.144E-01	0.566E 07	0.121E 05
0.300E 05	0.300E 06	0.150E 05	0.616E 01	-0.186E-01	0.699E 07	0.149E 05
0.350E 05	0.300E 06	0.150E 05	0.836E 01	-0.240E-01	0.857E 07	0.190E 05

(Input Echo for Problem 2 - Free head pile - P-Y curves by Unified Criteria)

***** UNIT DATA. *****

SYSTEM OF UNITS
(UP TO 16 CHAR.)
ENGL

***** PILE DATA. *****

NO. INCREMENTS PILE IS DIVIDED	NO. SEGMENTS WITH DIFFERENT CHARACTERISTICS	LENGTH OF PILE	MODULUS OF ELASTICITY	DEPTH
96	2	0.960E 03	0.290E 08	0.960E 02

TOP OF SEGMENT	DIAMETER OF PILE	MOMENT OF INERTIA	CROSS-SECT. AREA
0.	0.240E 02	0.568E 04	0.872E 02
0.530E 03	0.240E 02	0.343E 04	0.504E 02

***** SOIL DATA. *****

NUMBER OF LAYERS
1

LAYER NUMBER	P-Y CURVE CONTROL CODE	TOP OF LAYER	BOTTOM OF LAYER	INITIAL SOIL MODULI CONST.	FACTOR "A"	FACTOR "F"
1	6	0.960E 02	0.118E 04	0.116E 03	0.250E 01	0.100E 01

***** UNIT WEIGHT DATA. *****

NO. POINTS FOR PLOT
OF EFF. UNIT WEIGHT
VS. DEPTH
6

DEPTH BELOW TOP TO POINT	EFFECTIVE UNIT WEIGHT
0.960E 02	0.159E-01
0.336E 03	0.159E-01
0.336E 03	0.246E-01
0.900E 03	0.246E-01
0.900E 03	0.304E-01

0.118E 04 0.304E-01

***** PROFILE DATA. *****

NO. POINTS FOR
STRENGTH PARAMETERS
VS. DEPTH
3

DEPTH BELOW TOP OF PILE	UNDRAINED SHEAR STRENGTH OF SOIL	ANGLE OF INTERNAL FRICTION IN RADIANS	STRAIN AT 50% STRESS LEVEL
0.960E 02	0.139E 01	0.	0.200E-01
0.336E 03	0.139E 01	0.	0.200E-01
0.118E 04	0.625E 01	0.	0.100E-01

***** P-Y DATA. *****

NO. OF
P-Y CURVES
0

***** OUTPUT DATA. *****

DATA OUTPUT CODE	OUTPUT INCREMENT CODE	P-Y PRINTOUT CODE	NO. DEPTHS TO PRINT FOR P-Y CURVES
1	2	1	8

DEPTH FOR
PRINTING
P-Y CURVES
0.960E 02
0.120E 03
0.144E 03
0.192E 03
0.240E 03
0.336E 03
0.576E 03
0.960E 03

***** PILE HEAD (BOUNDARY) DATA. *****

BOUNDARY CONDITION CODE	NO. OF SETS OF BOUNDARY CONDITIONS

1	1

PILE HEAD PRINTOUT CODE	LATERAL LOAD AT TOP OF PILE	VALUE OF SECOND BOUNDARY CONDITION	AXIAL LOAD ON PILE
1	0.250E 05	0.300E 06	0.150E 05

***** CYCLIC DATA. *****

CYCLIC(0) OR STATIC(1) LOADING	NO. CYCLES OF LOADING
0	0.100E 03

***** PROGRAM CONTROL DATA. *****

MAX. NO. OF ITERATIONS	TOLERENCE ON SOLUTION CONVERGENCE	PILE HEAD DEFLECTION FLAG(STOPS RUN)
100	0.100E-02	0.400E 02

***** LOAD DATA. *****

BOUNDARY SET NO.	NO. POINTS FOR DISTRIB. LATERAL LOAD VS. DEPTH
1	0

(P-Y curves generated by verification - Problem 2)

GENERATED P-Y CURVES

THE NUMBER OF CURVES = 8
THE NUMBER OF POINTS ON EACH CURVE = 17

DEPTH IN	DIAM IN	C LBS/IN**2	CAVG LBS/IN**3	GAMMA LBS/IN**3	E50
0.	24.000	0.1E 01	0.1E 01	0.2E-01	0.200E-01

Y IN	P LBS/IN
0.	0.
0.100	11.600
0.200	18.346
0.300	21.000
0.400	23.114
0.500	24.899
0.600	26.459
0.700	27.854
0.800	29.122
0.900	30.288
1.000	31.370
1.100	32.383
1.200	33.336
8.800	22.224
16.400	11.112
24.000	0.000
36.000	0.

DEPTH IN	DIAM IN	C LBS/IN**2	CAVG LBS/IN**3	GAMMA LBS/IN**3	E50
24.00	24.000	0.1E 01	0.1E 01	0.2E-01	0.200E-01

Y IN	P LBS/IN
0.	0.
0.100	22.626
0.200	28.506
0.300	32.632
0.400	35.916
0.500	38.689
0.600	41.113
0.700	43.281
0.800	45.251
0.900	47.063
1.000	48.745

1.100	50.319
1.200	51.800
8.800	35.972
16.400	20.144
24.000	4.317
36.000	4.317

DEPTH IN	DIAM IN	C LBS/IN**2	CAVG LBS/IN**3	GAMMA LBS/IN**3	E50
48.00	24.000	0.1E 01	0.1E 01	0.2E-01	0.200E-01

Y IN	P LBS/IN
0.	0.
0.100	29.122
0.200	36.691
0.300	42.001
0.400	46.228
0.500	49.797
0.600	52.918
0.700	55.708
0.800	58.243
0.900	60.576
1.000	62.741
1.100	64.766
1.200	66.672
8.800	48.152
16.400	29.632
24.000	11.112
36.000	11.112

DEPTH IN	DIAM IN	C LBS/IN**2	CAVG LBS/IN**3	GAMMA LBS/IN**3	E50
96.00	24.000	0.1E 01	0.1E 01	0.2E-01	0.200E-01

Y IN	P LBS/IN
0.	0.
0.100	36.402
0.200	45.864
0.300	52.501
0.400	57.785
0.500	62.247
0.600	66.147
0.700	69.635
0.800	72.804
0.900	75.719
1.000	78.426
1.100	80.958
1.200	83.340
8.800	64.820
16.400	46.300
24.000	27.780
36.000	27.780

DEPTH IN	DIAM IN	C LBS/IN**2	CAVG LBS/IN**3	GAMMA LBS/IN**3	E50
144.00	24.000	0.1E 01	0.1E 01	0.2E-01	0.200E-01

Y IN	P LBS/IN
0.	0.
0.100	43.683
0.200	55.037
0.300	63.001
0.400	69.342
0.500	74.696
0.600	79.376
0.700	83.562
0.800	87.365
0.900	90.863
1.000	94.111
1.100	97.149
1.200	100.008
8.800	83.340
16.400	66.672
24.000	50.004
36.000	50.004

DEPTH IN	DIAM IN	C LBS/IN**2	CAVG LBS/IN**3	GAMMA LBS/IN**3	E50
240.00	24.000	0.1E 01	0.1E 01	0.2E-01	0.200E-01

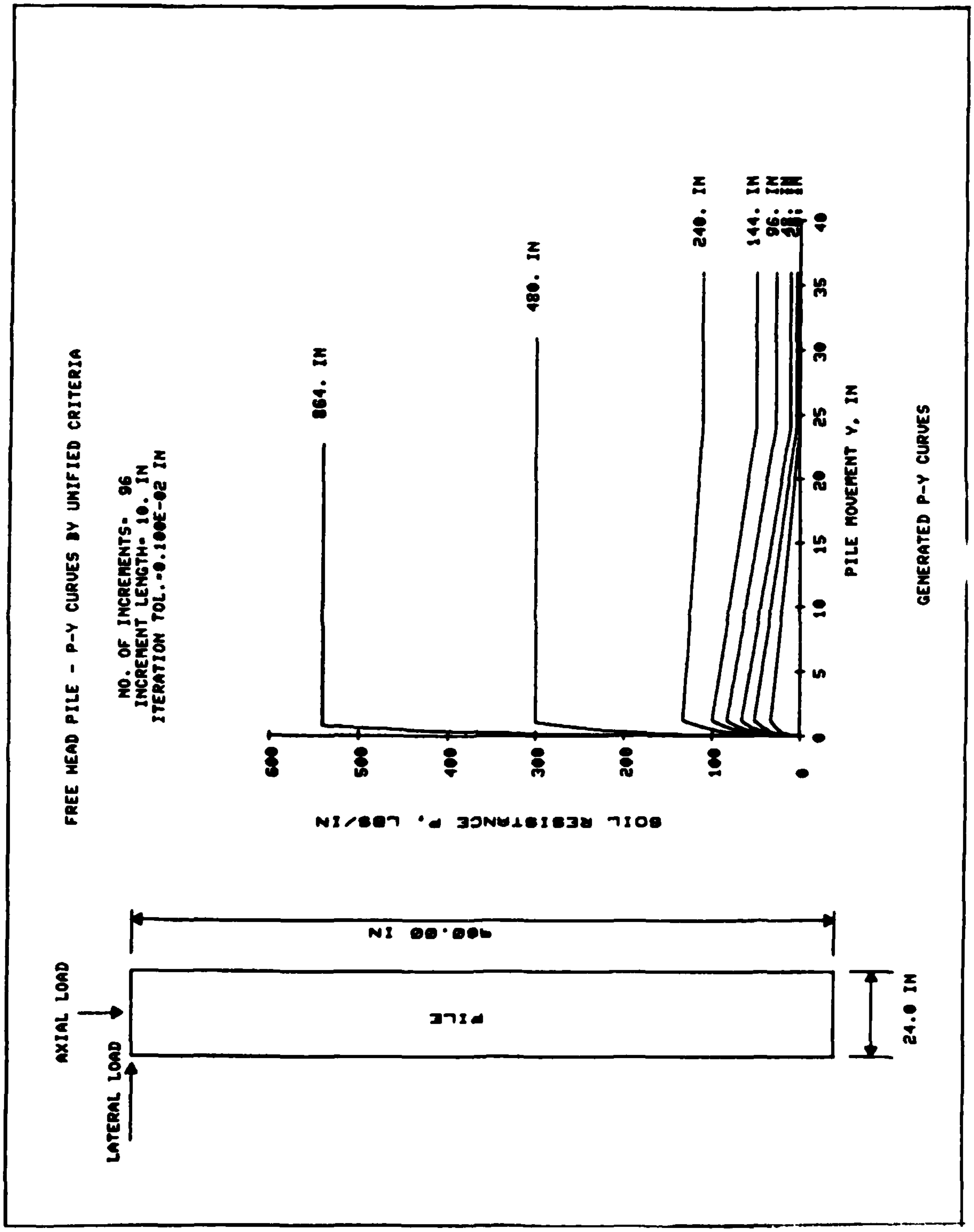
Y IN	P LBS/IN
0.	0.
0.100	58.243
0.200	73.382
0.300	84.001
0.400	92.456
0.500	99.595
0.600	105.835
0.700	111.416
0.800	116.487
0.900	121.151
1.000	125.482
1.100	129.532
1.200	133.344
8.800	125.936
16.400	118.528
24.000	111.120
36.000	111.120

DEPTH	DIAM	C	CAVG	GAMMA	E50
IN	IN	LBS/IN**2	LBS/IN**3	LBS/IN**3	
480.00	24.000	0.3E 01	0.2E 01	0.2E-01	0.171E-01

Y	P
IN	LBS/IN
0.	0.
0.086	131.041
0.171	165.101
0.257	188.994
0.343	208.014
0.429	224.077
0.514	238.117
0.600	250.672
0.686	262.082
0.771	272.576
0.857	282.319
0.943	291.432
1.029	300.009
7.543	300.009
14.057	300.009
20.571	300.009
30.857	300.009

DEPTH	DIAM	C	CAVG	GAMMA	E50
IN	IN	LBS/IN**2	LBS/IN**3	LBS/IN**3	
864.00	24.000	0.5E 01	0.3E 01	0.2E-01	0.126E-01

Y	P
IN	LBS/IN
0.	0.
0.063	235.868
0.126	297.175
0.189	340.181
0.251	374.417
0.314	403.329
0.377	428.601
0.440	451.199
0.503	471.736
0.566	490.625
0.629	508.162
0.691	524.566
0.754	540.003
5.531	540.003
10.309	540.003
15.086	540.003
22.629	540.003


FREE HEAD PILE - P-Y CURVES BY UNIFIED CRITERIA
NO. OF INCREMENTS= 96
INCREMENT LENGTH= 10. IN
ITERATION TOL.=0.100E-02 IN
864. IN
480. IN
240. IN
144. IN
96. IN
48. IN
600
500
400
300
200
100
0
0
5
10
15
20
25
30
35
40
SOIL RESISTANCE P, LBS/IN
PILE MOVEMENT Y, IN
GENERATED P-Y CURVES
AXIAL LOAD
LATERAL LOAD
PILE
480.00 IN
24.0 IN

FREE HEAD PILE - P-Y CURVES BY UNIFIED CRITERIA
LOADING CONDITIONS

LOAD CASE NO.	LATERAL LOAD AT PILE HEAD(LBS)	AXIAL LOAD AT PILE HEAD(LBS)	APPLIED MOMENT AT PILE HEAD(LBS-IN)
1	25000.	15000.	300000.

FREE HEAD PILE - P-Y CURVES BY UNIFIED CRITERIA

UNITS--ENGL

OUTPUT INFORMATION
****

NO. OF ITERATIONS = 27
MAXIMUM DEFLECTION ERROR = 0.765E-03 IN

PILE LOADING CONDITION
LATERAL LOAD AT PILE HEAD = 0.250E 05 LBS
APPLIED MOMENT AT PILE HEAD = 0.300E 06 LBS-IN
AXIAL LOAD AT PILE HEAD = 0.150E 05 LBS

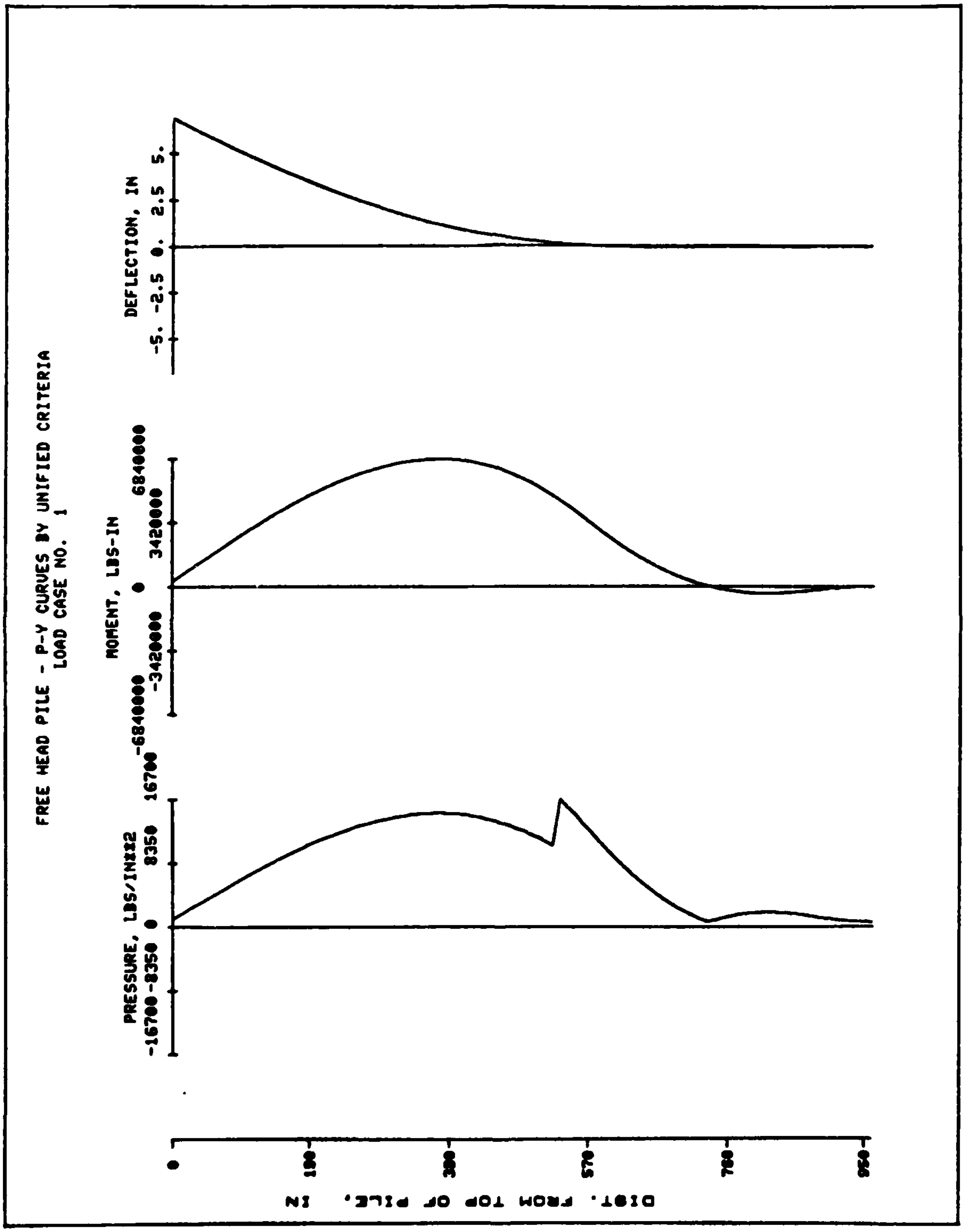
X IN	DEFLEC IN	MOMENT LBS-IN	TOTAL STRESS LBS/IN**2	DISTR. LOAD LBS/IN	SOIL MODULUS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2
*******	*********	*********	*********	*********	*********	*********
0.	0.688E 01	0.300E 06	0.806E 03	0.	0.	0.165E 12
20.00	0.650E 01	0.806E 06	0.188E 04	0.	0.	0.165E 12
40.00	0.611E 01	0.131E 07	0.294E 04	0.	0.	0.165E 12
60.00	0.574E 01	0.182E 07	0.401E 04	0.	0.	0.165E 12
80.00	0.536E 01	0.232E 07	0.508E 04	0.	0.	0.165E 12
100.00	0.499E 01	0.283E 07	0.615E 04	0.	0.610E 01	0.165E 12
120.00	0.463E 01	0.332E 07	0.720E 04	0.	0.964E 01	0.165E 12
140.00	0.428E 01	0.380E 07	0.821E 04	0.	0.135E 02	0.165E 12
820.00	-0.753E-02	-0.363E 06	0.157E 04	0.	0.124E 05	0.993E 11
840.00	-0.371E-02	-0.331E 06	0.146E 04	0.	0.206E 05	0.993E 11
860.00	-0.123E-02	-0.269E 06	0.124E 04	0.	0.445E 05	0.993E 11
880.00	0.185E-03	-0.186E 06	0.949E 03	0.	0.909E 05	0.993E 11
900.00	0.846E-03	-0.107E 06	0.672E 03	0.	0.606E 05	0.993E 11
920.00	0.107E-02	-0.481E 05	0.466E 03	0.	0.535E 05	0.993E 11
940.00	0.110E-02	-0.121E 05	0.340E 03	0.	0.543E 05	0.993E 11
960.00	0.107E-02	0.	0.298E 03	0.	0.570E 05	0.993E 11

OUTPUT VERIFICATION

```
THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT      =  0.525E-01 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = -0.425E-02 LBS

COMPUTED LATERAL FORCE AT PILE HEAD             =  0.25000E 05 LBS
    COMPUTED MOMENT AT PILE HEAD                =  0.30000E 06 IN-LBS
    COMPUTED SLOPE AT PILE HEAD                 = -0.19210E-01

THE OVERALL MOMENT IMBALANCE                 =  0.146E-01 IN-LBS
THE OVERALL LATERAL FORCE IMBALANCE          = -0.113E-07 LBS
```

OUTPUT SUMMARY

```
PILE HEAD DEFLECTION    =  0.688E 01 IN
MAXIMUM BENDING MOMENT =  0.684E 07 IN-LBS
MAXIMUM TOTAL STRESS    =  0.164E 05 LBS/IN**2
MAXIMUM SHEAR FORCE     =  0.253E 05 LBS
```


FREE HEAD PILE - P-Y CURVES BY UNIFIED CRITERIA
LOAD CASE NO. 1
DEFLECTION, IN
-5. -2.5 0. 2.5 5.
MOMENT, LBS-IN
-6840000 -3420000 0 3420000 6840000
PRESSURE, LBS/IN**2
-16700 -8350 0 8350 16700
DIST. FROM TOP OF PILE, IN
0 190 380 570 760 950

FREE HEAD PILE - P-Y CURVES BY UNIFIED CRITERIA

S U M M A R Y   T A B L E

****

LATERAL LOAD (LBS)	BOUNDARY CONDITION BC2	AXIAL LOAD (LBS)	YT (IN)	ST (IN/IN)	MAX. MOMENT (IN-LBS)	MAX. STRESS (LBS/IN**2)
0.250E 05	0.300E 06	0.150E 05	0.688E 01	-0.192E-01	0.684E 07	0.164E 05

(Input Echo for Problem 3 - Fixed head pile - P-Y curves by Unified Criteria)

***** UNIT DATA. *****

SYSTEM OF UNITS
(UP TO 16 CHAR.)
ENGL

***** PILE DATA. *****

NO. INCREMENTS PILE IS DIVIDED	NO. SEGMENTS WITH DIFFERENT CHARACTERISTICS	LENGTH OF PILE	MODULUS OF ELASTICITY	DEPTH
96	2	0.960E 03	0.290E 08	0.960E 02

TOP OF SEGMENT	DIAMETER OF PILE	MOMENT OF INERTIA	CROSS-SECT. AREA
0.	0.240E 02	0.563E 04	0.872E 02
0.530E 03	0.240E 02	0.343E 04	0.504E 02

***** SOIL DATA. *****

NUMBER OF LAYERS
1

LAYER NUMBER	P-Y CURVE CONTROL CODE	TOP OF LAYER	BOTTOM OF LAYER	INITIAL SOIL MODULI CONST.	FACTOR "A"	FACTOR "F"
1	6	0.960E 02	0.118E 04	0.116E 03	0.250E 01	0.100E 01

***** UNIT WEIGHT DATA. *****

NO. POINTS FOR PLOT
OF EFF. UNIT WEIGHT
VS. DEPTH
6

DEPTH BELOW TOP TO POINT	EFFECTIVE UNIT WEIGHT
0.960E 02	0.159E-01
0.336E 03	0.159E-01
0.336E 03	0.246E-01
0.900E 03	0.246E-01
0.900E 03	0.304E-01

0.118E 04	0.304E-01

***** PROFILE DATA. *****

NO. POINTS FOR
STRENGTH PARAMETERS
VS. DEPTH
3

DEPTH BELOW TOP OF PILE	UNDRAINED SHEAR STRENGTH OF SOIL	ANGLE OF INTERNAL FRICTION IN RADIANS	STRAIN AT 50% STRESS LEVEL
0.960E 02	0.139E 01	0.	0.200E-01
0.336E 03	0.139E 01	0.	0.200E-01
0.118E 04	0.625E 01	0.	0.100E-01

***** P-Y DATA. *****

NO. OF
P-Y CURVES
0

***** OUTPUT DATA. *****

DATA OUTPUT CODE	OUTPUT INCREMENT CODE	P-Y PRINTOUT CODE	NO. DEPTHS TO PRINT FOR P-Y CURVES
1	2	1	1

DEPTH FOR
PRINTING
P-Y CURVES
0.576E 03

***** PILE HEAD (BOUNDARY) DATA. *****

BOUNDARY CONDITION CODE	NO. OF SETS OF BOUNDARY CONDITIONS
2	1

PILE HEAD PRINTOUT CODE	LATERAL LOAD AT TOP OF PILE	VALUE OF SECOND BOUNDARY CONDITION	AXIAL LOAD ON PILE
1	0.250E 05	0.	0.150E 05

***** CYCLIC DATA. *****

CYCLIC(0) OR STATIC(1) LOADING	NO. CYCLES OF LOADING
0	0.100E 03

***** PROGRAM CONTROL DATA. *****

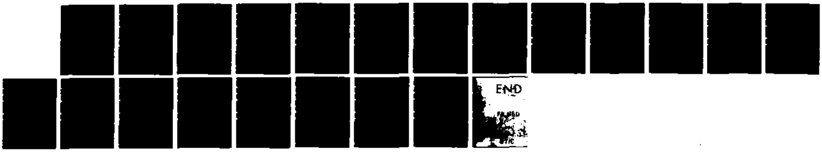
MAX. NO. OF ITERATIONS	TOLERENCE ON SOLUTION CONVERGENCE	PILE HEAD DEFLECTION FLAG(STOPS RUN)
100	0.100E-02	0.400E 02

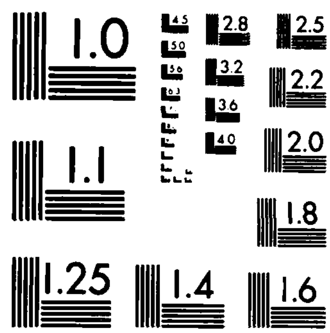
***** LOAD DATA. *****

BOUNDARY SET NO.	NO. POINTS FOR DISTRIB. LATERAL LOAD VS. DEPTH
1	0

AD-A144 641 LATERALLY LOADED PILES AND COMPUTER PROGRAM COM624G(U) 4/4
TEXAS UNIV AT AUSTIN L C REESE ET AL. APR 84
WES-TR-K-84-2

UNCLASSIFIED F/G 13/13 NL





MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

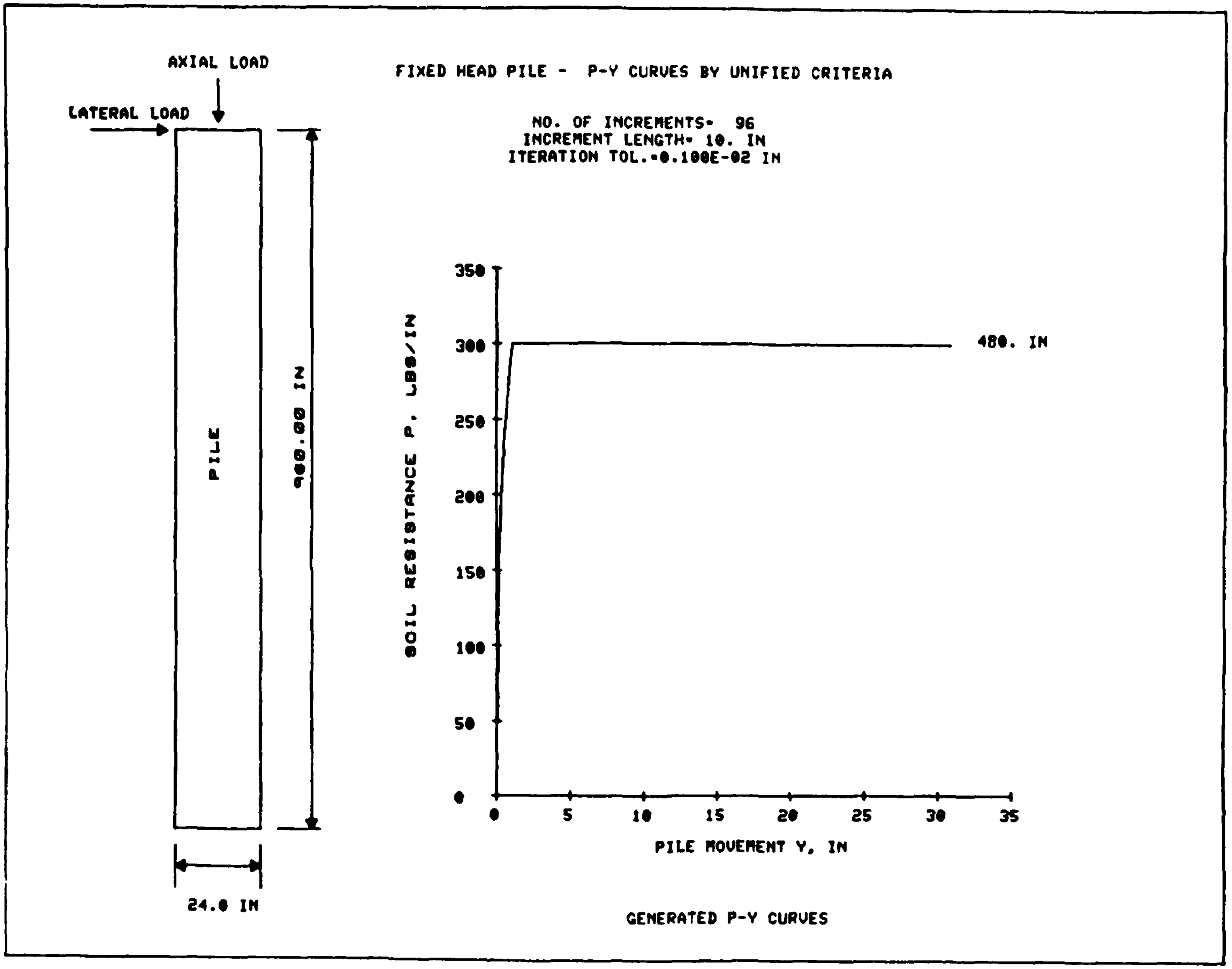
(P-Y curves generated for verification - Problem 3)

GENERATED P-Y CURVES

THE NUMBER OF CURVES = 1
THE NUMBER OF POINTS ON EACH CURVE = 17

DEPTH	DIAM	C	CAVG	GAMMA	E50
IN	IN	LBS/IN**2	LBS/IN**3	LBS/IN**3	
480.00	24.000	0.3E 01	0.2E 01	0.2E-01	0.171E-01

Y	P
IN	LBS/IN
0.	0.
0.086	131.041
0.171	165.101
0.257	188.994
0.343	208.014
0.429	224.077
0.514	238.117
0.600	250.672
0.686	262.082
0.771	272.576
0.857	282.319
0.943	291.432
1.029	300.009
7.543	300.009
14.057	300.009
20.571	300.009
30.857	300.009


FIXED HEAD PILE - P-Y CURVES BY UNIFIED CRITERIA
NO. OF INCREMENTS= 96
INCREMENT LENGTH= 10. IN
ITERATION TOL.=0.100E-02 IN
AXIAL LOAD
LATERAL LOAD
PILE
960.00 IN
24.0 IN
SOIL RESISTANCE P, LBS/IN
350
300
250
200
150
100
50
0
0
5
10
15
20
25
30
35
PILE MOVEMENT Y, IN
480. IN
GENERATED P-Y CURVES

FIXED HEAD PILE - P-Y CURVES BY UNIFIED CRITERIA
LOADING CONDITIONS

LOAD CASE NO.	LATERAL LOAD AT PILE HEAD(LBS)	AXIAL LOAD AT PILE HEAD(LBS)	SLOPE AT PILE HEAD
1	25000.	15000.	0.

FIXED HEAD PILE - P-Y CURVES BY UNIFIED CRITERIA

UNITS--ENGL

O U T P U T   I N F O R M A T I O N
****

NO. OF ITERATIONS = 16
MAXIMUM DEFLECTION ERROR = 0.680E-03 IN

PILE LOADING CONDITION
- LATERAL LOAD AT PILE HEAD = 0.250E 05 LBS
- SLOPE AT PILE HEAD = 0. IN/IN
- AXIAL LOAD AT PILE HEAD = 0.150E 05 LBS

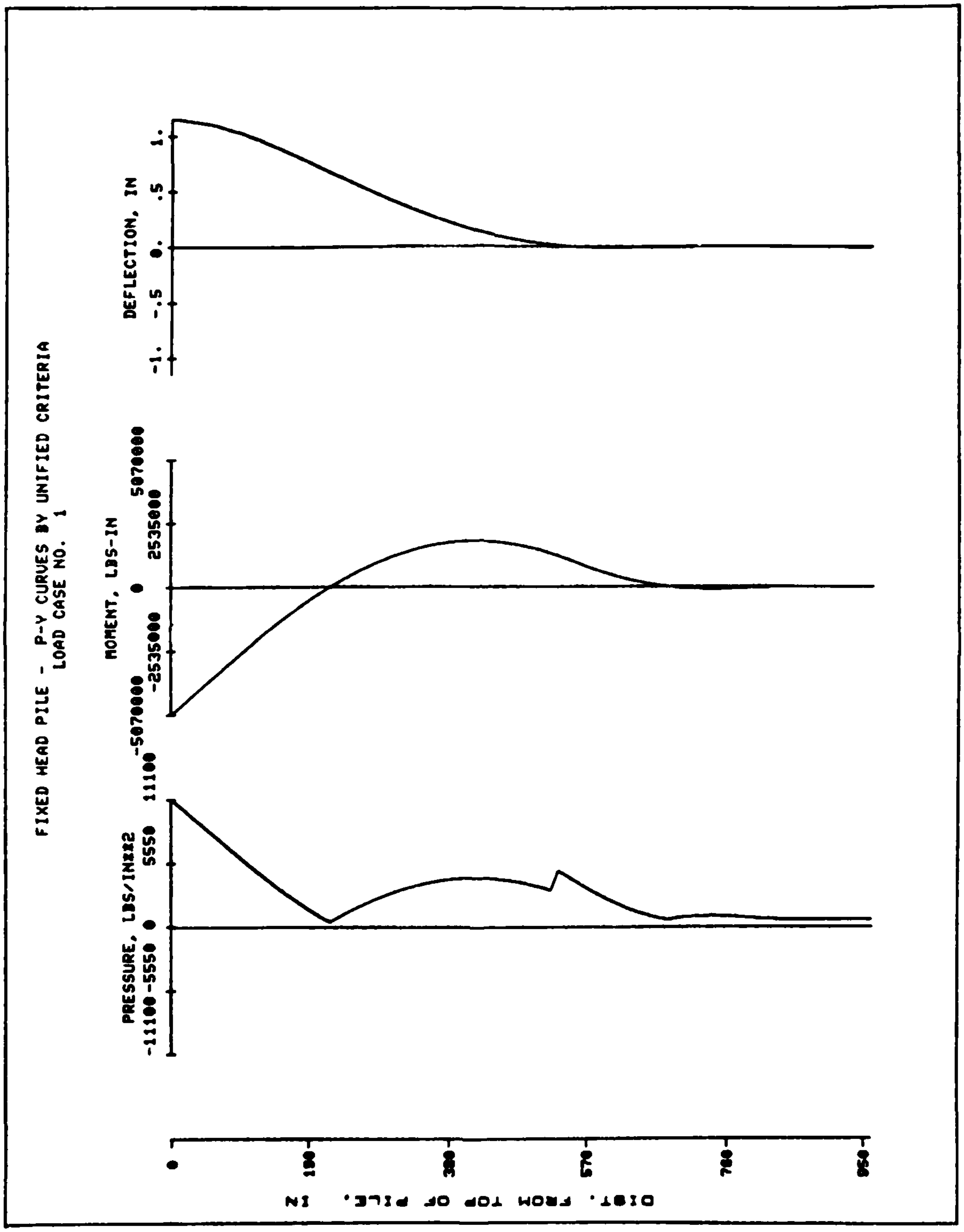
X	DEFLEC	MOMENT	TOTAL STRESS	DISTR. LOAD	SOIL MODULUS	FLEXURAL RIGIDITY
IN	IN	LBS-IN	LBS/IN**2	LBS/IN	LBS/IN**2	LBS-IN**2
*******	*********	*********	*********	*********	*********	*********
0.	0.115E 01	-0.507E 07	0.109E 05	0.	0.	0.165E 12
20.00	0.114E 01	-0.457E 07	0.983E 04	0.	0.	0.165E 12
40.00	0.112E 01	-0.407E 07	0.878E 04	0.	0.	0.165E 12
60.00	0.110E 01	-0.357E 07	0.772E 04	0.	0.	0.165E 12
80.00	0.106E 01	-0.307E 07	0.666E 04	0.	0.	0.165E 12
100.00	0.102E 01	-0.257E 07	0.560E 04	0.	0.339E 02	0.165E 12
120.00	0.969E 00	-0.208E 07	0.457E 04	0.	0.498E 02	0.165E 12
140.00	0.914E 00	-0.161E 07	0.357E 04	0.	0.652E 02	0.165E 12
820.00	0.212E-03	-0.187E 05	0.363E 03	0.	0.840E 05	0.993E 11
840.00	0.180E-03	-0.579E 04	0.318E 03	0.	0.863E 05	0.993E 11
860.00	0.124E-03	0.100E 04	0.301E 03	0.	0.886E 05	0.993E 11
880.00	0.698E-04	0.341E 04	0.310E 03	0.	0.909E 05	0.993E 11
900.00	0.292E-04	0.325E 04	0.309E 03	0.	0.933E 05	0.993E 11
920.00	0.140E-05	0.197E 04	0.305E 03	0.	0.956E 05	0.993E 11
940.00	-0.185E-04	0.626E 03	0.300E 03	0.	0.979E 05	0.993E 11
960.00	-0.357E-04	0.	0.298E 03	0.	0.100E 06	0.993E 11

OUTPUT VERIFICATION

```
THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT     =  0.403E-01 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = -0.248E-02 LBS

COMPUTED LATERAL FORCE AT PILE HEAD             =  0.25000E 05 LBS
    COMPUTED SLOPE AT PILE HEAD                 =  0.21684E-19 IN/IN

THE OVERALL MOMENT IMBALANCE                =  0.147E-01 IN-LBS
THE OVERALL LATERAL FORCE IMBALANCE         = -0.252E-08 LBS
```

OUTPUT SUMMARY

```
PILE HEAD DEFLECTION    =  0.115E 01 IN
MAXIMUM BENDING MOMENT = -0.507E 07 IN-LBS
MAXIMUM TOTAL STRESS    =  0.109E 05 LBS/IN**2
MAXIMUM SHEAR FORCE     =  0.250E 05 LBS
```


FIXED HEAD PILE - P-Y CURVES BY UNIFIED CRITERIA
LOAD CASE NO. 1
DEFLECTION, IN
-1. -.5 0. .5 1.
MOMENT, LBS-IN
-5070000 -2535000 0 2535000 5070000
PRESSURE, LBS/IN**2
-11100 -5550 0 5550 11100
DIST. FROM TOP OF PILE, IN
0 190 380 570 760 950

FIXED HEAD PILE - P-Y CURVES BY UNIFIED CRITERIA

S U M M A R Y   T A B L E

****

LATERAL LOAD (LBS)	BOUNDARY CONDITION BC2	AXIAL LOAD (LBS)	YT (IN)	ST (IN/IN)	MAX. MOMENT (IN-LBS)	MAX. STRESS (LBS/IN**2)
0.250E 05	0.	0.150E 05	0.115E 01	0.217E-19	-0.507E 07	0.109E 05

(Input Echo for Problem 4 - Rotational Restraint at Pile Head)

***** UNIT DATA. *****

SYSTEM OF UNITS
(UP TO 16 CHAR.)
ENGL

***** PILE DATA. *****

NO. INCREMENTS PILE IS DIVIDED	NO. SEGMENTS WITH DIFFERENT CHARACTERISTICS	LENGTH OF PILE	MODULUS OF ELASTICITY	DEPTH
96	2	0.960E 03	0.290E 08	0.960E 02

TOP OF SEGMENT	DIAMETER OF PILE	MOMENT OF INERTIA	CROSS-SECT. AREA
0.	0.240E 02	0.568E 04	0.872E 02
0.530E 03	0.240E 02	0.343E 04	0.504E 02

***** SOIL DATA. *****

NUMBER OF LAYERS
1

LAYER NUMBER	P-Y CURVE CONTROL CODE	TOP OF LAYER	BOTTOM OF LAYER	INITIAL SOIL MODULI CONST.	FACTOR "A"	FACTOR "F"
1	6	0.960E 02	0.118E 04	0.116E 03	0.250E 01	0.100E 01

***** UNIT WEIGHT DATA. *****

NO. POINTS FOR PLOT
OF EFF. UNIT WEIGHT
VS. DEPTH
6

DEPTH BELOW TOP TO POINT	EFFECTIVE UNIT WEIGHT
0.960E 02	0.159E-01
0.336E 03	0.159E-01
0.336E 03	0.246E-01
0.900E 03	0.246E-01
0.900E 03	0.304E-01

0.118E 04	0.304E-01

***** PROFILE DATA. *****

NO. POINTS FOR
STRENGTH PARAMETERS
VS. DEPTH
3

DEPTH BELOW TOP OF PILE	UNDRAINED SHEAR STRENGTH OF SOIL	ANGLE OF INTERNAL FRICTION IN RADIANS	STRAIN AT 50% STRESS LEVEL
0.960E 02	0.139E 01	0.	0.200E-01
0.336E 03	0.139E 01	0.	0.200E-01
0.118E 04	0.625E 01	0.	0.100E-01

***** P-Y DATA. *****

NO. OF
P-Y CURVES
0

***** OUTPUT DATA. *****

DATA OUTPUT CODE	OUTPUT INCREMENT CODE	P-Y PRINTOUT CODE	NO. DEPTHS TO PRINT FOR P-Y CURVES
1	2	1	1

DEPTH FOR
PRINTING
P-Y CURVES
0.576E 03

***** PILE HEAD (BOUNDARY) DATA. *****

BOUNDARY CONDITION CODE	NO. OF SETS OF BOUNDARY CONDITIONS
3	1

PILE HEAD PRINTOUT CODE	LATERAL LOAD AT TOP OF PILE	VALUE OF SECOND BOUNDARY CONDITION	AXIAL LOAD ON PILE
1	0.250E 05	0.150E 07	0.150E 05

***** CYCLIC DATA. *****

CYCLIC(0) OR STATIC(1) LOADING	NO. CYCLES OF LOADING
0	0.100E 03

***** PROGRAM CONTROL DATA. *****

MAX. NO. OF ITERATIONS	TOLERENCE ON SOLUTION CONVERGENCE	PILE HEAD DEFLECTION FLAG(STOPS RUN)
100	0.100E-02	0.400E 02

***** LOAD DATA. *****

BOUNDARY SET NO.	NO. POINTS FOR DISTRIB. LATERAL LOAD VS. DEPTH
1	0

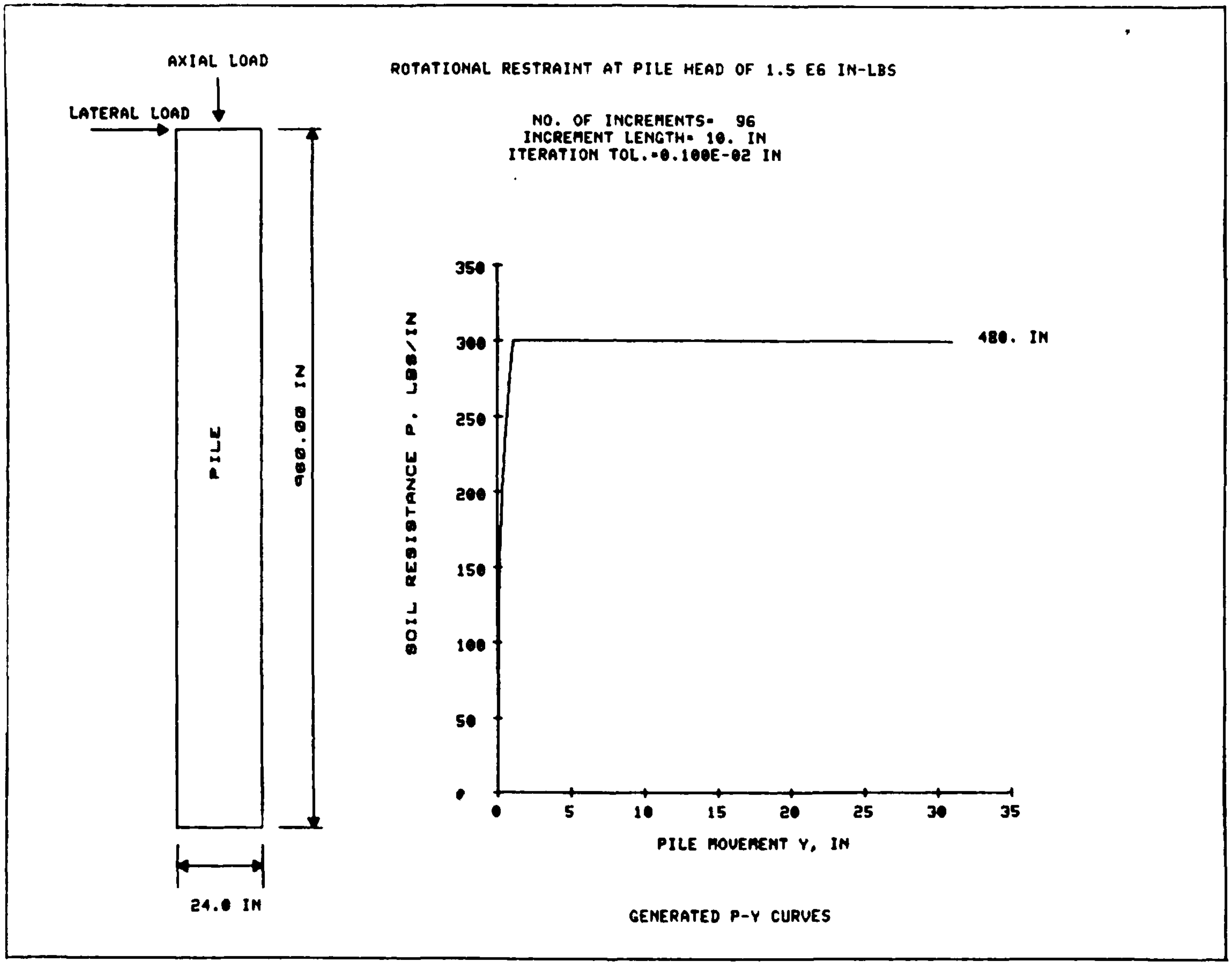
(P-Y curve generated for verification - Problem 4)

GENERATED P-Y CURVES

THE NUMBER OF CURVES = 1
THE NUMBER OF POINTS ON EACH CURVE = 17

DEPTH IN	DIAM IN	C LBS/IN**2	CAVG LBS/IN**3	GAMMA LBS/IN**3	E50
480.00	24.000	0.3E 01	0.2E 01	0.2E-01	0.171E-01

Y IN	P LBS/IN
0.	0.
0.086	131.041
0.171	165.101
0.257	188.994
0.343	208.014
0.429	224.077
0.514	238.117
0.600	250.672
0.686	262.082
0.771	272.576
0.857	282.319
0.943	291.432
1.029	300.009
7.543	300.009
14.057	300.009
20.571	300.009
30.857	300.009


AXIAL LOAD
LATERAL LOAD
PILE
960.00 IN
24.0 IN
ROTATIONAL RESTRAINT AT PILE HEAD OF 1.5 E6 IN-LBS
NO. OF INCREMENTS= 96
INCREMENT LENGTH= 10. IN
ITERATION TOL.=0.100E-02 IN
SOIL RESISTANCE P, LBS/IN
350
300
250
200
150
100
50
0
480. IN
0
5
10
15
20
25
30
35
PILE MOVEMENT Y, IN
GENERATED P-Y CURVES

ROTATIONAL RESTRAINT AT PILE HEAD OF 1.5 E6 IN-LBS
LOADING CONDITIONS

LOAD CASE NO.	LATERAL LOAD AT PILE HEAD(LBS)	AXIAL LOAD AT PILE HEAD(LBS)	ROTATIONAL STIFFNESS AT PILE HEAD(LBS)
1	25000.	15000.	1500000.

ROTATIONAL RESTRAINT AT PILE HEAD OF 1.5 E6 IN-LBS

UNITS--ENGL

O U T P U T   I N F O R M A T I O N
****

NO. OF ITERATIONS = 28
MAXIMUM DEFLECTION ERROR = 0.794E-03 IN

PILE LOADING CONDITION
- LATERAL LOAD AT PILE HEAD = 0.250E 05 LBS
- ROTATIONAL RESTRAINT = 0.150E 07 LBS-IN
- AXIAL LOAD AT PILE HEAD = 0.150E 05 LBS

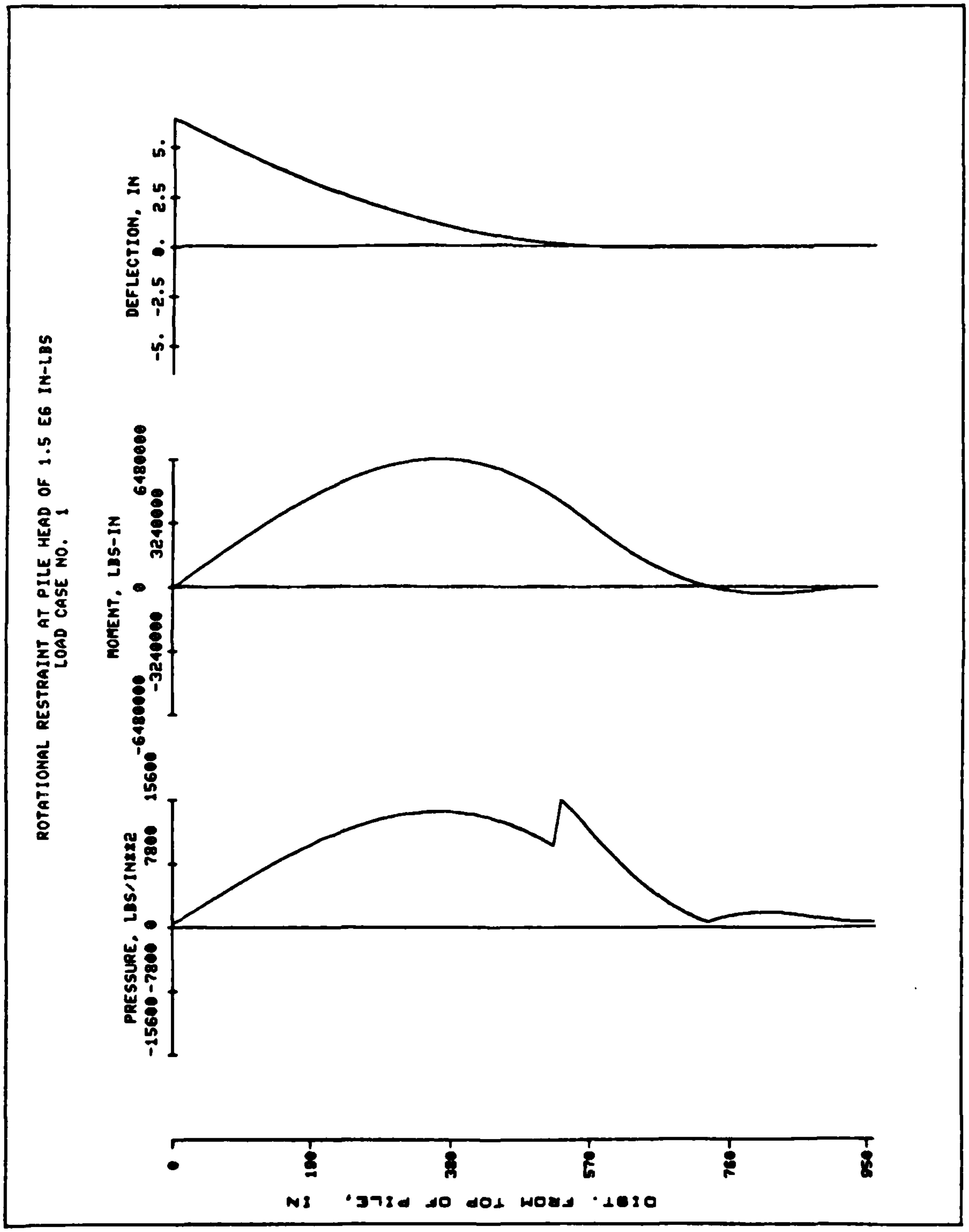
X IN	DEFLEC IN	MOMENT LBS-IN	TOTAL STRESS LBS/IN**2	DISTR. LOAD LBS/IN	SOIL MODULUS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2
*******	*********	*********	*********	*********	*********	*********
0.	0.641E 01	-0.267E 05	0.228E 03	0.	0.	0.165E 12
20.00	0.606E 01	0.479E 06	0.118E 04	0.	0.	0.165E 12
40.00	0.570E 01	0.984E 06	0.225E 04	0.	0.	0.165E 12
60.00	0.535E 01	0.149E 07	0.332E 04	0.	0.	0.165E 12
80.00	0.500E 01	0.199E 07	0.439E 04	0.	0.	0.165E 12
100.00	0.466E 01	0.250E 07	0.546E 04	0.	0.665E 01	0.165E 12
120.00	0.432E 01	0.299E 07	0.650E 04	0.	0.105E 02	0.165E 12
140.00	0.399E 01	0.347E 07	0.751E 04	0.	0.147E 02	0.165E 12
↓						↓
820.00	-0.579E-02	-0.338E 06	0.148E 04	0.	0.148E 05	0.993E 11
840.00	-0.262E-02	-0.301E 06	0.135E 04	0.	0.259E 05	0.993E 11
860.00	-0.656E-03	-0.236E 06	0.113E 04	0.	0.675E 05	0.993E 11
880.00	0.361E-03	-0.157E 06	0.847E 03	0.	0.909E 05	0.993E 11
900.00	0.744E-03	-0.878E 05	0.605E 03	0.	0.660E 05	0.993E 11
920.00	0.768E-03	-0.384E 05	0.432E 03	0.	0.666E 05	0.993E 11
940.00	0.633E-03	-0.934E 04	0.330E 03	0.	0.782E 05	0.993E 11
960.00	0.455E-03	0.	0.298E 03	0.	0.100E 06	0.993E 11

OUTPUT VERIFICATION

```
THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT     = -0.452E-01 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = -0.422E-02 LBS

COMPUTED LATERAL FORCE AT PILE HEAD            =  0.25000E 05 LBS
       COMPUTED ROTATIONAL STIFFNESS AT PILE HEAD =  0.15000E 07 IN-LB
S
    COMPUTED SLOPE AT PILE HEAD                   = -0.17819E-01

THE OVERALL MOMENT IMBALANCE                  =  0.152E-01 IN-LBS
THE OVERALL LATERAL FORCE IMBALANCE           = -0.966E-08 LBS
```

OUTPUT SUMMARY

```
PILE HEAD DEFLECTION    =  0.641E 01 IN
MAXIMUM BENDING MOMENT =  0.648E 07 IN-LBS
MAXIMUM TOTAL STRESS    =  0.153E 05 LBS/IN**2
MAXIMUM SHEAR FORCE     =  0.253E 05 LBS
```


ROTATIONAL RESTRAINT AT PILE HEAD OF 1.5 E6 IN-LBS
LOAD CASE NO. 1
DEFLECTION, IN
-5. -2.5 0. 2.5 5.
MOMENT, LBS-IN
-6480000 -3240000 0 3240000 6480000
PRESSURE, LBS/IN**2
-15600 -7800 0 7800 15600
0 190 380 570 760 950
DIST. FROM TOP OF PILE, IN

ROTATIONAL RESTRAINT AT PILE HEAD OF 1.5 E6 IN-LBS

SUMMARY TABLE
****

LATERAL LOAD (LBS)	BOUNDARY CONDITION BC2	AXIAL LOAD (LBS)	YT (IN)	ST (IN/IN)	MAX. MOMENT (IN-LBS)	MAX. STRESS (LBS/IN**2)
0.250E 05	0.150E 07	0.150E 05	0.641E 01	-0.178E-01	0.648E 07	0.153E 05

APPENDIX E: NOTATION

Symbol	Definition	Definition on Page
A	Factor	35
b	Width of the pile	32
	Footing width	34
	Pile diameter	35
c	Cohesion	36
C	Parameter describing the effect of repeated loading on deformation	68
c_a	Average undrained shear strength	39
EI	Flexural rigidity	13
E_s	Soil modulus	18
H	Depth to the point under consideration	39
k	Constant giving variation of soil modulus with depth	33
K_a	Rankine active earth pressure coefficient (minimum coefficient of active earth pressure)	41
k_h	Coefficient of horizontal subgrade reaction	32
K_o	At-rest earth pressure coefficient	41
$\bar{k}_{s1}$	Coefficient of vertical subgrade reaction for a 1-ft-wide beam	32
LI	Liquidity index	73
m	Reduction factor to be multiplied by c_a to yield the average sliding stress between the pile and the stiff clay	39
M	Moment	13
M_i	Moment at joint i	22
M_t	Moment at the top of the pile	25
M_t/S_t	Rotational-restraint constant at the top of the pile	25
N	Number of cycles of load application	69

Symbol	Definition	Definition on Page
O_R	Overconsolidation ratio	73
p	Soil resisting pressure applied to beam (soil resistance)	14
PI	Plasticity index	73
p_t	Lateral load at the top of the pile	25
p_u	Ultimate soil resistance	35
p_x	Axial load	12
q	Uniformly distributed vertical load on beam	13
R	Variation in pile bending stiffness	21
S	Slope	13
St	Slope of the elastic curve at the top of the pile	25
S_t	Sensitivity	73
V	Shear	13
W_L	Liquid limit	73
x	Depth from the ground surface	33
y	Deflection at point x along the length of the pile (pile deflection)	13
y_c	Deflection under N cycles of load	69
y_s	Deflection under a short-term static load	69
y_{50}	Deflection under a short-term static load at half the ultimate resistance	69
δ	Deflection of dolphin, ft	B3
ε	Strain	34
ε_{50}	Strain at half the maximum principal stress difference	35
ρ_1	Mean settlement of the foundation	34
σ	Stress	36

Symbol	Definition	Definition on Page
$\bar{\sigma}_v$	Average effective stress	71
σ_Δ	Deviator stress	35
γ	Average unit weight of the soil (submerged unit weight if the soil is below the water table)	39
γ'	Average effective unit weight from the ground surface to the p-y curve	52
ϕ	Angle of internal friction	36

END

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