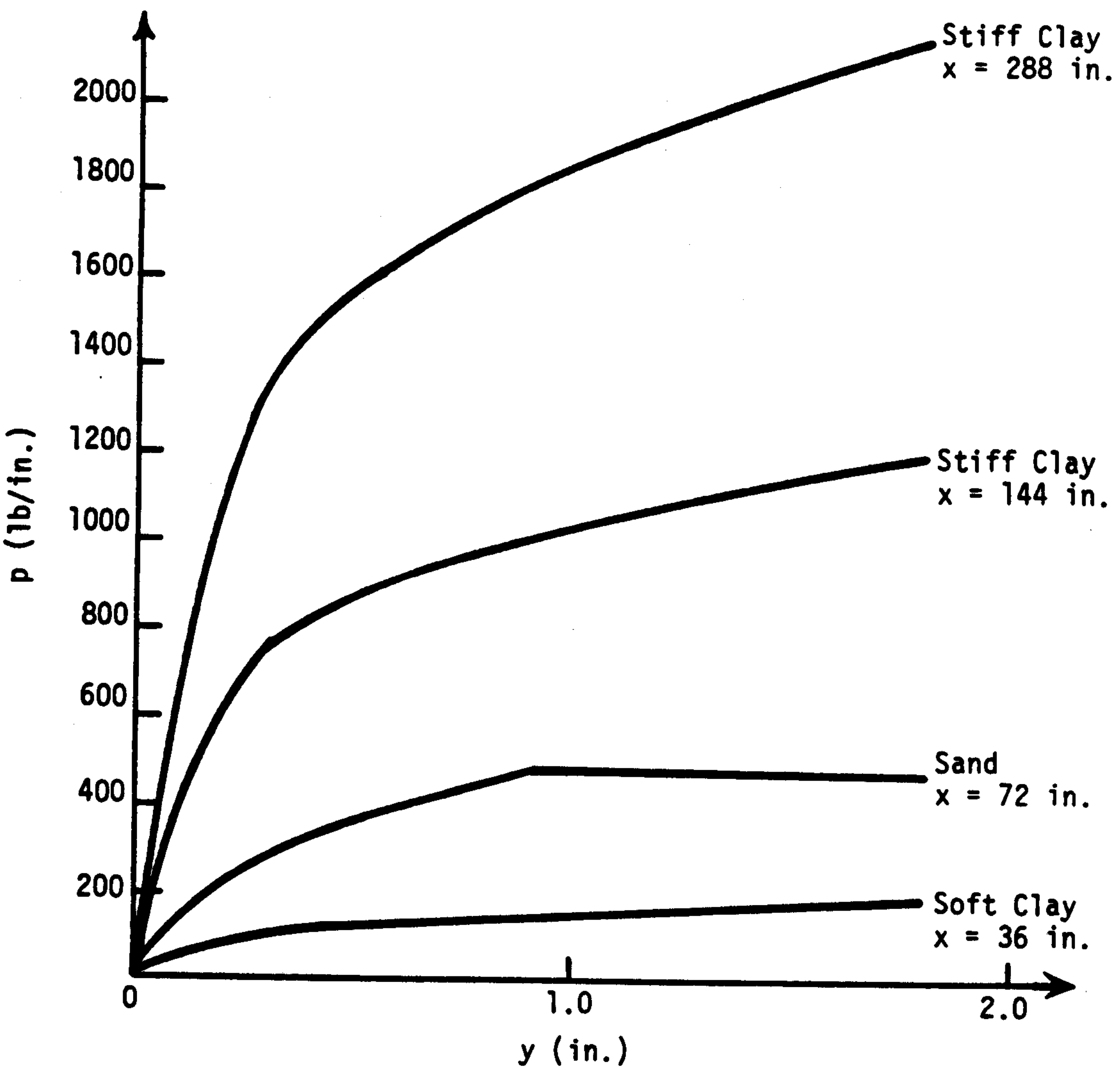


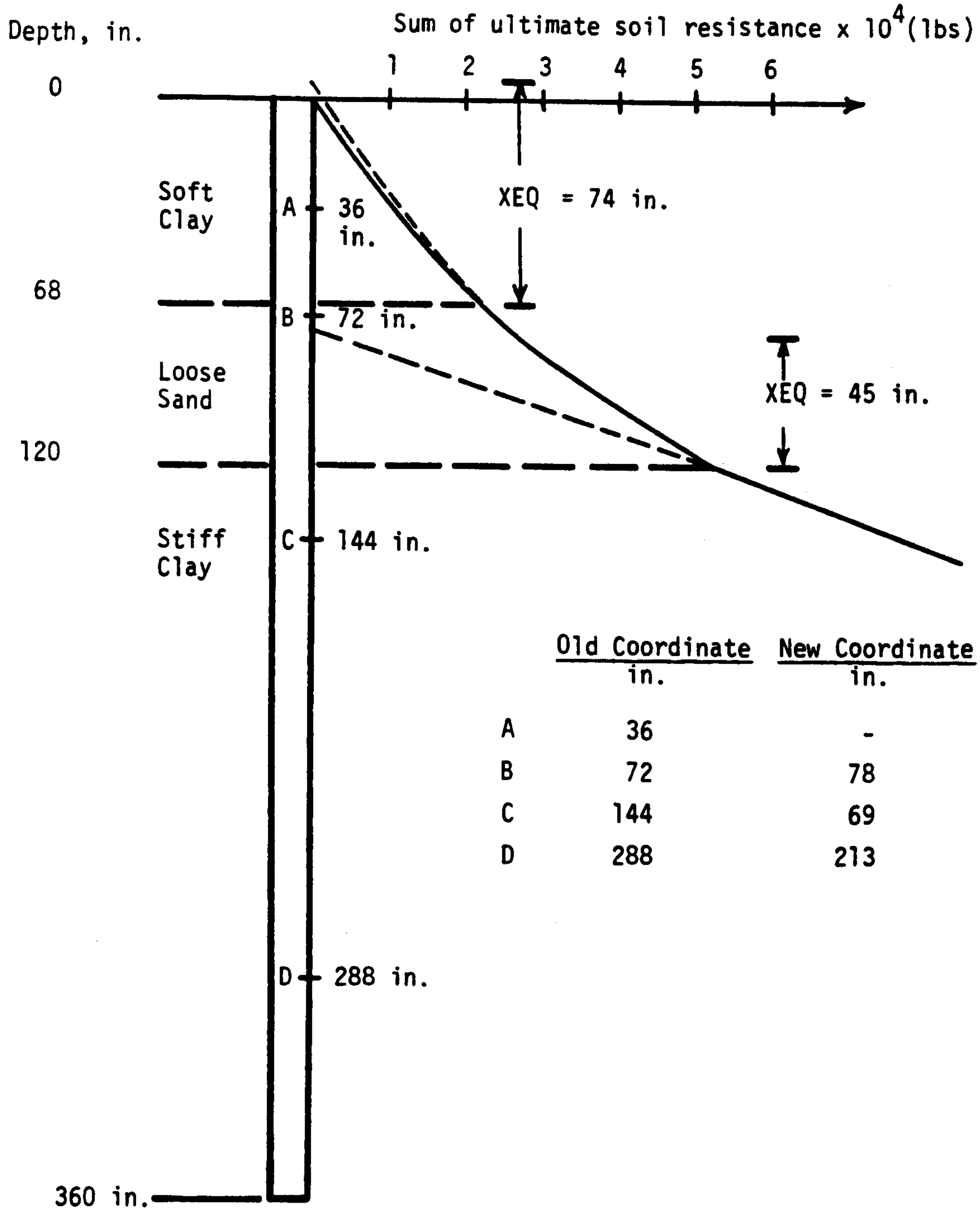
Figure 3.16. Example p-y curves for layered soils.

just below the soft clay; and the curve for depths of 144 ins and 228 ins in the lower zone of stiff clay.

Following the procedure suggested by Georgiadis (1983), the p-y curve for soft clay can be computed as if the profile consists altogether of that soil. When dealing with the sand, an equivalent depth of sand is found such that the value of the sum of the ultimate soil resistance for the equivalent sand and the soft clay are equal at the interface. The equivalent depth of loose sand to substitute for the effect of the clay was computed to be 74 inches. Thus, 68 ins of soft clay is replaced by 74 ins of loose sand, and point B that defines the position of the p-y curve in the sand is 78 ins below the assumed ground surface that the actual depth of 72 inches. Figure 3.17 shows a plot of the sum of the ultimate soil resistances with the equivalent thickness of the soft clay layer (computed as H_2 by use of Eqs. 3.41 and 3.42) as shown as XEQ.

An equivalent depth of stiff clay was found such that the sum of the ultimate soil resistance for the stiff clay is equal to the sum of the ultimate soil resistance of the loose sand and soft clay. That equivalent depth was found to be 45 ins and is indicated in Fig. 3.17. Thus, the depths to the two p-y curves in the stiff clay are assumed to be 69 ins and 213 ins rather than the actual depths of 144 ins and 288 ins (the actual thickness of 120 ins of the two upper-layers was reduced to 45 ins, a reduction of 75 inches.)

Another point of considerable interest is that the presence of no free water was used for the stiff clay in the recommendations for p-y curves for stiff clay. This decision is based on the assumption that the sand above the stiff clay can move downward and fill any gap that develops between the clay and



	Old Coordinate in.	New Coordinate in.
A	36	-
B	72	78
C	144	69
D	288	213

Figure 3.17. Equivalent depths of overlying soil for use in computing p-y curves for a layered system.

the pile. Furthermore, in the stiff-clay experiment where free water was present, the free water moved upward along the face of the pile with each cycle of loading. The presence of soft clay and sand to a depth of 10 ft above the stiff clay is believed to suppress the hydraulic action of free water even though the sand did not serve to close the potential gaps in the stiff clay.

MODIFICATIONS TO p-y CURVES FOR SLOPING GROUND

Introduction

The recommendations for p-y curves presented to this point are developed for a horizontal ground surface. In order to allow designs to be made if a pile is installed on a slope, modifications must be made in the p-y curves. The modifications involve revisions in the manner in which the ultimate soil resistance is computed. In this regard, the assumption is made that the flow-around failure will not be influenced by sloping ground; therefore, only the equations for the wedge-type failure need modification.

The solutions presented herein are entirely analytical and must be considered as preliminary. Additional modifications may be indicated if it is possible to implement an extensive laboratory and field study.

Equations for Ultimate Resistance in Clay

The ultimate soil resistance near the ground surface for saturated clay where the pile was installed in ground with a horizontal slope was derived by Reese (1958) and was shown in Eq. 3.2.

$$(p_u)_{ca} = 2c_a b + \gamma b H + 2.83\ c_a H \tag{3.2}$$

If the ground surface has a slope angle θ as shown in Fig. 3.18, the soil resistance in the front of the pile, following the Reese approach is:

$$(p_u)_{ca} = (2c_a b + \gamma b H + 2.83\ c_a H)\ \frac{1}{1 + \tan\theta}. \tag{3.43}$$

The soil resistance in the back of the pile is:

$$(p_u)_{ca} = (2c_a b + \gamma b H + 2.83\ c_a H)\ \frac{\cos\theta}{\sqrt{2}\ \cos\ (45^\circ + \theta)}. \tag{3.44}$$

where

$(p_u)_{ca}$ = ultimate soil resistance near ground surface,

c_a = average undrained shear strength,

b = pile diameter,

γ = average unit weight of soil,

H = depth from ground surface to point along pile where soil resistance is computed, and

θ = angle of slope as measured from the horizontal.

A comparison of Eqs. 3.43 and 3.44 shows that the equations are identical except for the terms at the right side of the

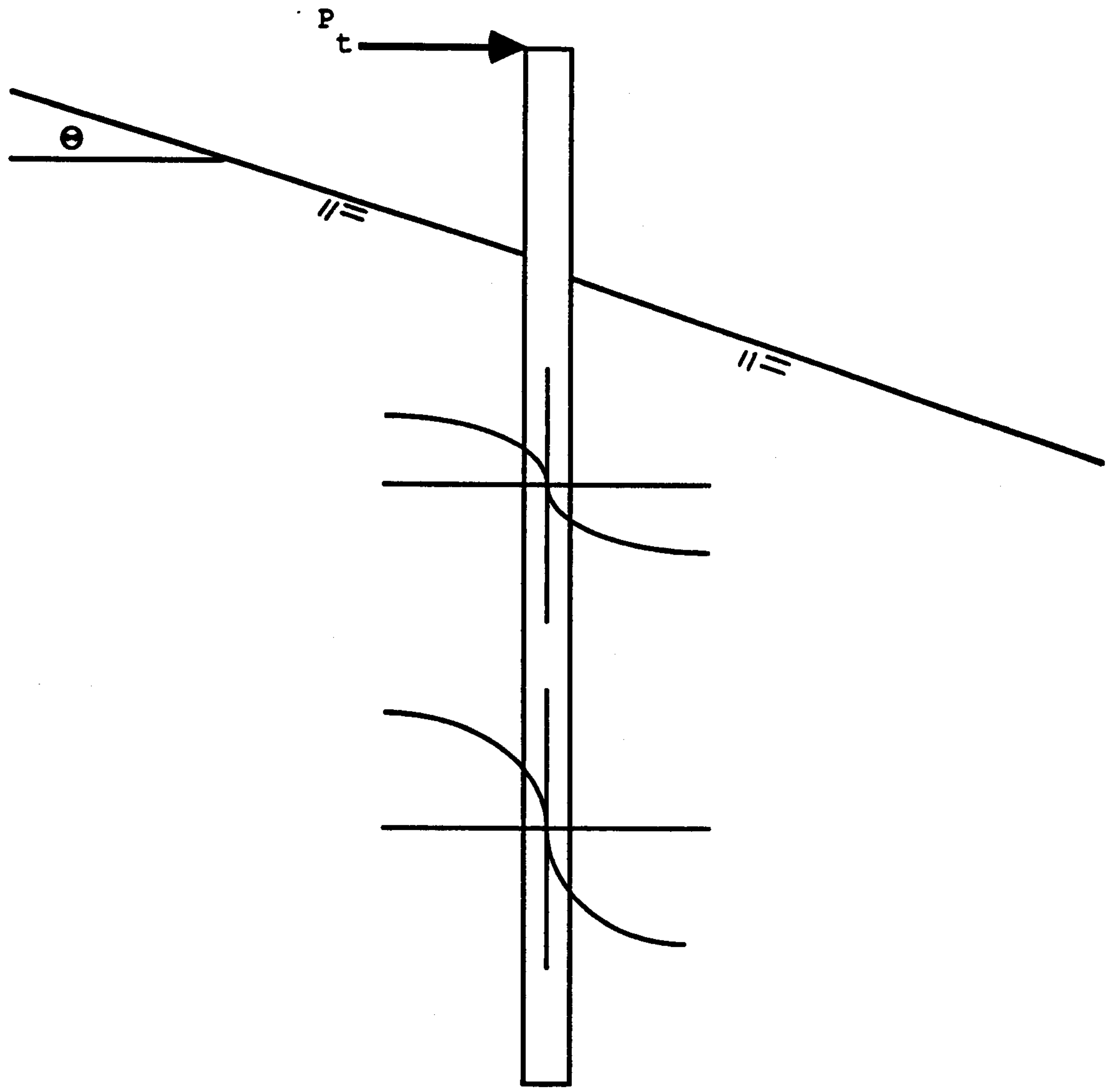


Figure 3.18. Pile installed in sloping ground.

parenthesis. If θ is equal to zero, the equations become equal to the original equation.

Equations for Ultimate Resistance in Sand

The ultimate soil resistance near the ground surface for sand where the pile was installed in ground with a horizontal slope is:

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

$$\left. + K_o H \tan\beta (\tan\phi \sin\beta - \tan\alpha) - K_a b \right]. \qquad (3.5)$$

If the ground surface has a slope angle θ, the ultimate soil resistance in the front of the pile is:

$$(p_u)_{sa} = \gamma H \left[\frac{K_o H \tan\phi \sin\beta}{\tan(\beta-\phi)\cos\alpha} \left(4D_1^3 - 3D_1^2 + 1 \right) + \right.$$

$$\frac{\tan\beta}{\tan(\beta-\phi)} \left(bD_2 + H \tan\beta \tan\alpha D_2^2 \right) +$$

$$\left. K_o H \tan\beta(\tan\phi \sin\beta - \tan\alpha) \left(4D_1^3 - 3D_1^2 + 1 \right) - K_a b \right]. \qquad (3.45)$$

where

$$D_1 = \frac{\tan\beta \tan\theta}{\tan\beta \tan\theta + 1}, \qquad (3.46)$$

$$D_2 = 1 - D_1, \text{ and} \tag{3.47}$$

$$K_a = \cos\beta \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}. \tag{3.48}$$

(β is defined in Fig. 3.3)

The ultimate soil resistance in the back of the pile is:

$$(p_u)_{sa} = \gamma H \left[\frac{K_o H \tan\phi \sin\beta}{\tan(\beta-\phi)\cos\alpha} \left(4D_3^3 - 3D_3^2 + 1 \right) + \frac{\tan\beta}{\tan(\beta-\phi)} \left(bD_4 + H \tan\beta \tan\alpha D_4^2 \right) + K_o H \tan\beta(\tan\phi\sin\beta - \tan\alpha)\left(4D_3^3 - 3D_3^2 + 1\right) - K_a b \right] \tag{3.49}$$

where

$$D_3 = \frac{\tan\beta\tan\theta}{1 - \tan\beta\tan\theta}, \text{ and} \tag{3.50}$$

$$D_4 = 1 + D_3. \tag{3.51}$$

CHAPTER 4. COMPUTATION OF ULTIMATE MOMENT AND FLEXURAL RIGIDITY OF PILE

INTRODUCTION

Application

The designer of piles under lateral loading must usually make computations to ascertain that three factors are within tolerable limits: combined stress (including bending stress), shear stress, and pile-head deflection. The flexural rigidity of the pile (bending stiffness) is an important parameter that influences the computations (Reese and Wang, 1988).

In general, the flexural rigidity of reinforced concrete with a specified cross section varies nonlinearly with the applied bending moment, and a constant EI employed in the p-y analysis for a concrete pile will result in some degree of inaccuracy in the computation.

Because the response of a pile is nonlinear with respect to load (the soil has a nonlinear response), the load-factor approach is recommended. Therefore, the ultimate bending moment of a reinforced-concrete member, and of any other type of section being analyzed, is needed.

The code PMEIX has been developed to yield the ultimate-moment capacity of a reinforced-concrete or steel-pipe pile and to give the bending stiffness of such piles as a function of applied moment. With this information the designer can make a correct judgement in the selection of a representative EI value in accordance with the loading range and can compute the ultimate lateral load for a given cross-section.

Significant Assumptions

The program solves for the behavior of a slice from a pile or from a beam-column. It is of interest to note that the EI of the concrete member will experience a significant change when cracking occurs. In the coding used herein, the assumption is made that the tensile strength of concrete is minimal and that cracking will be closely spaced when it appears. Actually, such cracks will initially be spaced at some distance apart and the change in the EI will not be so drastic. In respect to the cracking of concrete, therefore, the EI for a beam will change more gradually than is given by the coding.

The ultimate bending moment of a reinforced-concrete section is taken at a maximum strain of concrete of 0.003 and is not affected by the crack spacing. The ultimate bending moment for steel, because of the large amount of deformation of steel when stressed to beyond the proportional limit, is taken at a maximum strain of 0.015 which is five times that of concrete.

STRESS-STRAIN CURVES FOR CONCRETE AND STEEL

Any number of models can be used for the stress-strain curves for concrete and steel. For the purposes of the computations presented herein, some relatively simple curves are used.

Figure 4.1 shows the stress-strain curve for concrete. The following equations apply to the branches of the curve. The value of f'_c is specified by the engineer; the other symbols are defined below or in the figure.

$$f''_c = 0.85\, f'_c \qquad (4.1)$$

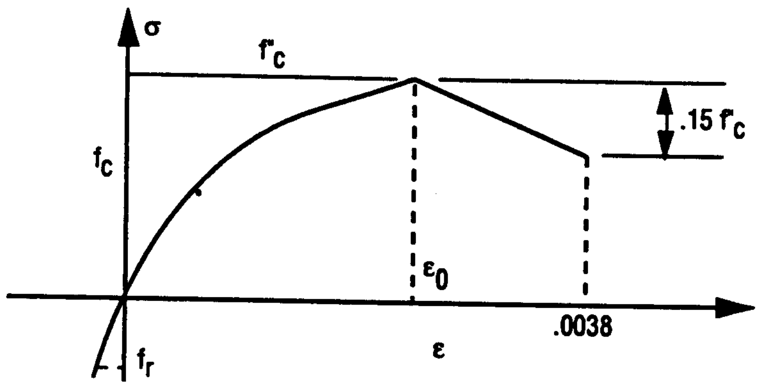


Figure 4.1. Stress-strain curve for concrete.

$$f_c = f''_c \left[2\frac{\varepsilon}{\varepsilon_0} - \left(\frac{\varepsilon}{\varepsilon_0}\right)^2\right] \tag{4.2}$$

$$f_r = 7.5\ (f'_c)^{0.5} \tag{4.3}$$

$$E_c = 57{,}000\ (f'_c)^{0.5} \tag{4.4}$$

$$\varepsilon_0 = 1.7\ \frac{f'_c}{E_c} \tag{4.5}$$

where

E_c = initial modulus of the concrete and the units of E_c, f_r and f'_c are psi.

Figure 4.2 shows the stress-strain (σ–ε) curve for steel and, as may be seen, there is no limit to the amount of plastic deformation. The curves for tension and compression are identical. The yield strength of the steel f_y is selected according to the material being used, and the following equations apply.

$$\varepsilon_y = \frac{f_y}{E_s} \tag{4.6}$$

$$E_s = 29{,}000{,}000 \text{ psi.} \tag{4.7}$$

The models and the equations shown here are employed in the derivations that are shown subsequently.

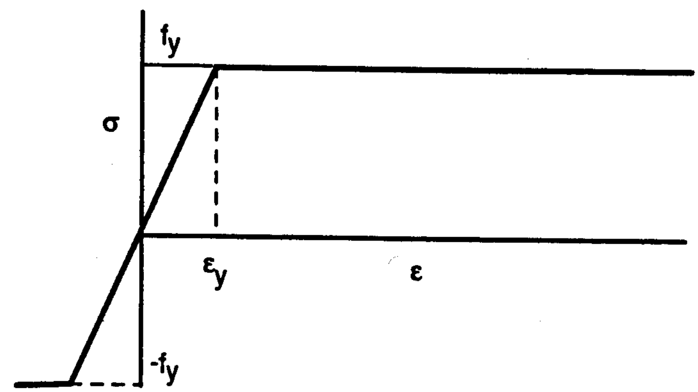


Figure 4.2. Stress-strain (α-ε) curve for steel.

CROSS SECTIONS THAT CAN BE ANALYZED

The following types of cross sections can be analyzed:

1. square or rectangular, reinforced concrete,
2. circular, reinforced concrete,
3. circular, reinforced concrete, with steel tubular shell around concrete,
4. circular, reinforced concrete, with steel tubular shell and tubular core, and
5. circular, steel tubular shell

The output consists of a set of values for M versus EI for different axial loads ranging from zero to the axial-load capacity for the column. The number of load cases in one run is limited to 10.

COMPUTATION PROCEDURE

The flexural behavior of a structural element such as a beam, column, or a pile subjected to bending is dependent upon its flexural rigidity, EI, where E is the modulus of elasticity of the material of which it is made and I is the moment of inertia of the cross section about the axis of bending. In some instances the values of E and I remain constant for all ranges of stresses to which the member is subjected, but there are situations where both E and I vary with changes in stress conditions.

This variation is most pronounced in reinforced concrete members. Because of nonlinearity in stress-strain relationships, the value of E varies; and because the concrete in the tensile zone below the neutral axis becomes ineffective due to cracking, the value of I is reduced. When a member is made up of a composite cross section there is no way to calculate directly the value of E for the member as a whole. Reinforced concrete is a composite material; other examples are concrete encased in a steel tube or a steel section encased in concrete.

An element from a beam with an unloaded shape of abcd is shown by the dashed lines in Fig. 4.3. The beam is subjected to pure bending and the element changes in shape as shown by the solid lines. The relative rotation of the sides of the element is given by the small angle $d\theta$ and the radius of curvature of the elastic element is signified by the length ρ. The unit strain ε_x along the length of the beam is given by Eq. 4.8.

$$\varepsilon_x = \frac{\Delta}{dx} \tag{4.8}$$

where

Δ = deformation at any distance from the neutral axis, and

dx = length of the element.

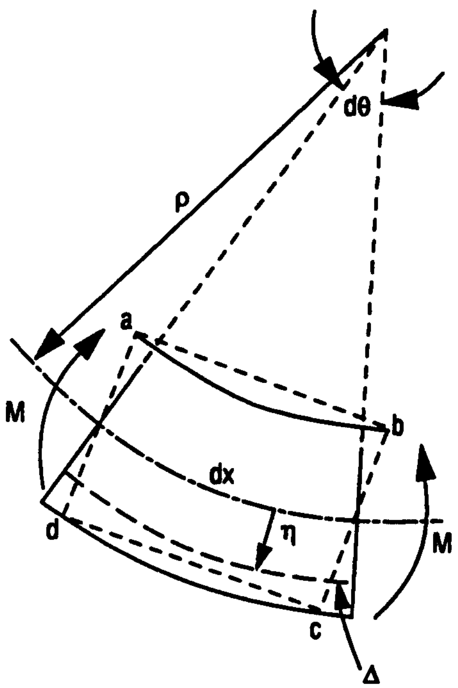


Figure 4.3. Element from a beam with an unloaded shape of abcd.

From similar triangles

$$\frac{\rho.}{dx} = \frac{\eta}{\Delta} \tag{4.9}$$

where

η = distance from neutral axis.

Equation 4.10 is obtained from Eqs. 4.8 and 4.9, as follows:

$$\varepsilon_x = \frac{\eta}{\rho} \tag{4.10}$$

From Hooke's law

$$\varepsilon_x = \frac{\sigma_x}{E} \tag{4.11}$$

where

σ_x = unit stress along the length of the beam, and

E = Young's modulus.

Therefore

$$\sigma_x = \frac{E\eta}{\rho} \tag{4.12}$$

From beam theory

$$\sigma_x = \frac{M\eta}{I} \tag{4.13}$$

where

M = applied moment, and

I = moment of inertia of the section.

From Eqs. 4.12 and 4.13

$$\frac{M\eta}{I} = \frac{E\eta}{\rho} \tag{4.14}$$

Rewriting Eq. 4.14

$$\frac{M}{EI} = \frac{1}{\rho} \tag{4.15}$$

Continuing with the derivation, it can be seen that $dx = \rho d\theta$ and

$$\frac{1}{\rho} = \frac{d\theta}{dx} \tag{4.16}$$

For convenience, the symbol ϕ is substituted for $\frac{d\theta}{dx}$; therefore, from this substitution and Eqs. 4.15 and 4.16, the following equation is found.

$$EI = \frac{M}{\phi} \tag{4.17}$$

Also, because $\Delta = \eta\, d\theta$ and $\varepsilon_x = \frac{\Delta}{dx}$ then,

$$\varepsilon_x = \phi\, \eta \tag{4.18}$$

The computation for a reinforced-concrete section, or a section consisting partly or entirely of a pile, proceeds by selecting a value of ϕ and estimating the position of the neutral axis. The strain at points along the depth of the beam can be computed by use of Eq. 4.18, which in turn will lead to the forces in the concrete and steel. In this step, assumptions are made that the stress-strain curves for concrete and steel are as shown in Figs. 4.1 and 4.2.

With the magnitude of the forces, both tension and compression, the equilibrium of the section can be checked, taking into account the external compressive loading. If the section is not in equilibrium, a revised position of the neutral axis is selected and iterations proceed until the neutral axis is found.

The bending moment is found from the forces in the concrete and steel by taking moments about the centroidal axis of the section. Thus, the externally-applied, axial load does not enter the equations. Then, the value of EI is found from Eq. 4.10. The maximum strain is tabulated and the solution proceeds by incrementing the value of ϕ. The computations continue until the maximum strain selected for failure, in the concrete or in a steel pipe, is reached or exceeded. Thus, the ultimate moment that can be sustained by the section can be found.

EXAMPLE CALCULATION BY HAND

Figure 4.4 shows the cross section of a beam subjected to bending moment. The axial load is 200 kips, ϕ = .0001 in^{-1}, E_c = 4,000 kip/sq in, and E_s = 30,000 kips/sq inches. The value of M and EI are to be found.

Step 1

As the first step, the position of the neutral axis should be determined by trial, such that the net force on the cross section equals the applied load of 200 kips. Concrete below the neutral axis will be neglected if the tensile stress in the concrete is high enough to cause the concrete to crack. A linear stress-strain relationship will be assumed here for simplicity.

Strains:

At top fiber of concrete: (.0001)(9.2) = .00092

1st row of bars:	(.0001)(6.2) = .00062
2nd row of bars:	(.0001)(1.8) = .00018
3rd row of bars:	(.0001)(9.8) = .00098
4th row of bars:	(.0001)(17.8) = 00178

Forces:

Concrete: [(.00092)(4000/2)][(20)(9.2)] = 338 kips comp

1st row of bars:	44 kips comp
2nd row of bars:	8 kips tension
3rd row of bars:	46 kips tension
4th row of bars:	127 kips tension

Net forces = 201 kips - OK

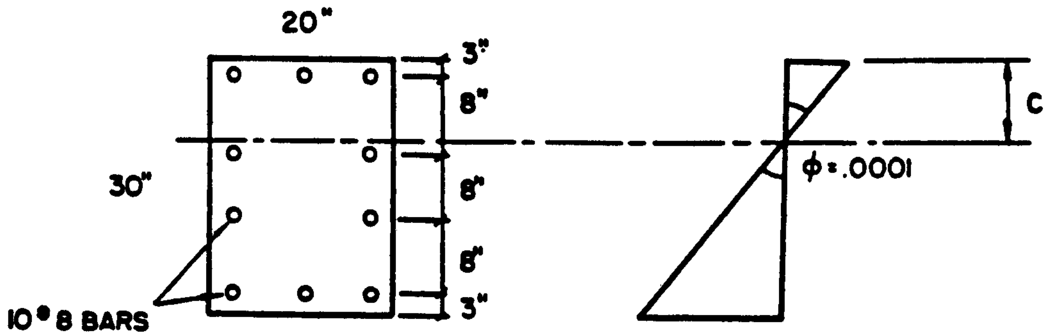


Figure 4.4. Cross section of a beam for example problem.

Step 2

The bending moment due to all these forces about the centroidal axis of the cross section now is to be found. Clockwise moments are taken as positive.

Moment due to compression in concrete

$$= 338\ (15 - \frac{9.2}{3})$$

$$= +4{,}033 \text{ in-kips.}$$

Trial 1

c = 9 inches.

Strains:

At top fiber of concrete	(.0001)(9) = .0009
1st row of bars:	(.0001)(6) = .0006
2nd row of bars:	(.0001)(2) = .0002
3rd row of bars:	(.0001)(10) = .001
4th row of bars:	(.0001)(18) = .00018

Forces (stress x area):

Concrete: [(.0009)(4000/2)] [(20)(9.0)] = 324 kips comp

1st row of bars:	(.0006)(30,000)(3)(.79) = 43 kips comp
2nd row of bars:	(.0002)(30,000)(2)(.79) = 9 kips tension
3rd row of bars:	(.001)(30,000)(2)(.79) = 47 kips tension
4th row of bars:	(.0018)(30,000)(2)(.79) = 128 kips tension

Net forces = 183 kips comp - no good

Trial 2

c = 9.2 inches.

Moment due to compression in Row 1 bars	= (44)(12) = +528 in-kips
Moment due to tension in Row 2 bars	= (8)(4) = -32 in-kips
Moment due to tension in Row 3 bars	= (46)(4) = +184 in-kips
Moment due to tension in Row 4 bars	= (127)(12) = +1524 in-kips
Net moment M	= +6237 in-kips

The net moment from the computer solution is 6169 in-kips. The discrepancy between hard calculation and computer solution can be further reduced if more trials by hand calculations can be done.

$$EI = M/\phi = 6237/.0001 = 62{,}370{,}000 \text{ kips-sq inches.}$$

The above method, though simple in cases like rectangular cross sections, becomes tedious when cross sections with varying widths are considered. Further, because the actual stress-strain relationship of concrete is a nonlinear function, for a circular cross section, the computation of forces will involve double integration, one for the area and one for the stress. This is not feasible by hand calculations. However, with the aid of the high-speed digital computer, the solution has been made possible for these complicated cases.

CHAPTER 5. VERIFICATION OF ACCURACY OF SOLUTION

The accuracy of the output of any computer run must be verified because of a compelling and urgent reason. An error in the output could lead to an incorrect design with unforeseen and undesirable results. Incidents can be cited with regard to placing too much reliance on the accuracy of any output. In another context, the late Dr. Karl Terzaghi wrote about his early experiences as an engineer in Russia. He looked at the plans for a major building and by his experience he knew that some reinforced-concrete beams were too small. As noted earlier, the analysis of a pile under lateral loading, or lateral and axial loading, requires the full attention of an experienced engineer.

Verification is necessary for several reasons: the input boundary conditions and soil properties could be in error; the particular computer could be operating with an inadequate word length; some problem could exist with the operating system of the computer; the number of increments into which the pile is divided could be improper; and, finally, there could be a "bug" in the computer program itself. Some teachers of methods of coding the solution to engineering problems have stated the following truism, "It is impossible to write a computer code of any length without an error."

With regard to the accuracy of the coding of COM624P, several comments can be made: the code was written by a programmer with extensive experience in writing codes and with an excellent educational background in mechanics, many parts of the code have been tested against existing codes, the program was checked thoroughly before any release, and the program will have been used by a number of beta sites before any general release. Furthermore, by agreement with FHWA, ENSOFT will answer questions with the view of maintaining the code for a considerable period of time.

Nevertheless, the verification of the output for any problem should be viewed as an integral part of using the program.

The verification may be accomplished by one of the methods given herein or, preferably, by means devised specifically by the engineer for the particular problem that is being addressed. The following sections of this chapter present specific suggestions for verification.

SIGNIFICANT FIGURES

The solution of the differential equation is done numerically by use of difference equations as presented in Chapter 2. The differences between deflections at adjacent points will disappear unless a sufficient number of significant figures are carried in the computations. COM624P is written in double-precision arithmetic and, using IBM PC's XT and AT, the word length for computations is 64 bits, resulting in 15 significant figures.

The first step to be taken by the user of COM624P is to investigate the operating system of the particular computer being used to make sure that a sufficient number of significant figures is being used in the computations. Also, the identical problems can be run that are solved in Part I, Chapter 5, and the output can be compared in detail.

SELECTION OF NUMBER OF INCREMENTS

A fundamental aspect of the solution of the differential equation by difference techniques, as shown in Chapter 2, is the selection of the number of increments into which the pile is divided, or, in effect, the selection of the length of an increment. The length of the pile that is to be sub-divided is the embedded length plus the portion above the groundline.

The first step in the selection of the increment is to eliminate the lower portion of a pile where there may be several points of zero deflection. As discussed earlier, the groundline deflection and the maximum bending moment are unaffected if the length of a pile extends so that there are at least two points of zero deflection at the bottom of the pile.

With the length of the pile adjusted so that there is not a large number of points of zero deflection, the engineer may wish to make a few runs with a relatively large lateral loading with the pile subdivided into different numbers of increments. The results for an example of such a study are shown in Fig. 5.1 where computed values of groundline deflection and maximum bending moment are plotted. Serious errors were introduced when the number of increments was less than 50, and virtually the same results were obtained if the number of increments was 100 or more. Because of the nonlinearity of the problem and because of the number of parameters that are involved, the selection of an appropriate number of increments cannot easily be made automatic. The engineer-user should study a number of the kinds of problems that are usually encountered in the local practice and make enough studies of the sort shown in Fig. 5.1 to be assured of introducing no errors due to too few increments.

CHECKING AGAINST EXAMPLE PROBLEMS

Six examples of computer output are presented in Part I, Chapter 5. The engineer may wish to code one or more of those problems for the particular computer being used. If agreement is not obtained between the outputs, the operating system of the local computer needs to be evaluated; it may be necessary to make use of another computer.

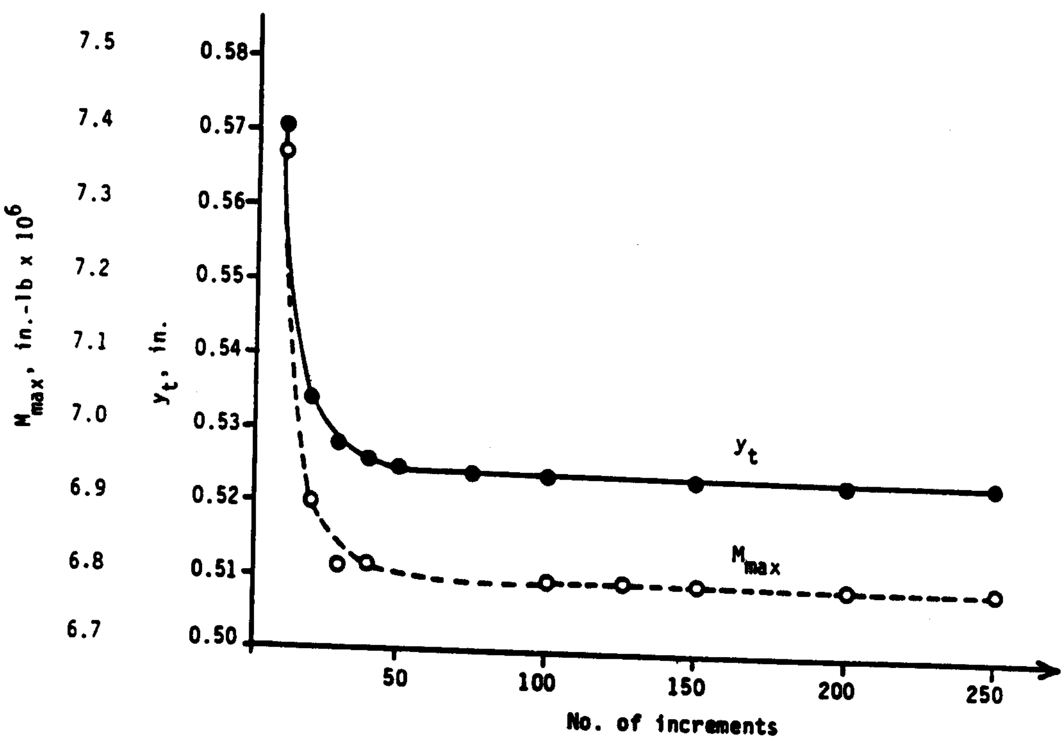


Figure 5.1. Influence of length of increment on pile-head deflection and maximum bending moment.

CHECK OF ECHO-PRINT

The code for COM624P is written so that there is an echo-print of the input data. A careful check of the echo-print to see that the coding was correct is necessary for each computer run.

CHECK OF SOIL RESISTANCE

The engineer-user has the option of asking that p-y curves be printed for various positions along the length of the pile. As COM624P is being implemented and on occasion thereafter, the user may have some p-y curves printed for the purpose of checking. The first step is to check one or more of the curves against the recommendations shown in Chapter 3. The computations may be tedious but the assurance of accurate computations is necessary.

The next step in the check is to read off the tabulated value of deflection from a table of output for one of the p-y curves. With that value of deflection as an argument, the p-y curve is consulted and the soil resistance corresponding to the deflection is interpolated. The soil resistance should agree closely with the value tabulated in the output. In this connection, the point should be made that the equations for the p-y curves are employed for every point that a soil resistance is needed as the computer is doing the internal computations; however, the soil resistances that are output for a p-y curve are for discrete deflections. Therefore, the interpolation mentioned above could be very slightly in error.

The procedures of verification with respect to a specific computer run will be implemented with respect to Example 1 in Part I, Chapter 5. With regard to the p-y curves, the values of ultimate resistance will be computed for the four curves that are tabulated; the values were computed by calculator to be 778, 1098, 1419, and 1820 lb/in. These values agree with the values that are tabulated. The deflection was checked at which the soil resistance became constant, or 16 y_{50}. The value of y_{50} was computed to be 0.257 in and 16 y_{50} was computed to be 4.12 inches. This value agrees with the tabulated values.

Next, a check will be made to determine whether or not an appropriate value of soil resistance was computed. The p-y curve at a depth of 20 ins and the run for a lateral load of 20,000 lbs were selected. The deflection at a value of x of 20 ins was 0.101 inch. Employing the equation for the early part of the p-y curve: $p = (0.5)\ (778)\ (0.101/0.257)^{0.25} = 308$ lbs/inches. This value agrees with the value shown in the output for a depth of 20 inches.

CHECK OF MECHANICS

Check of Results of Analysis of Buckling Load

As noted earlier, the computation of the buckling load does not involve the solution of an eigenvalue problem, but is accomplished by incrementing the axial load until there is excessive deflection. The computer output can be examined and a point below the groundline can be found where the moment is zero. This point can be selected as a hinge and the Euler equation can be used to check the buckling load for the column that consists of that portion of the pile from the assumed hinge and above.

Check of Values of Shear and Bending Moment

A plot can be made by hand or with the computer of the values of soil resistance that are computed for a particular run. The boundary conditions can be used and the shear can be computed point-by-point along the pile. The values of shear computed in this way should agree closely with the values that are tabulated. Also, the area under the p-x curve can be integrated approximately to obtain a concentrated load that is equivalent to the distributed load. The equilibrium of the pile in shear can then be checked. If the pile is subjected to an axial load, the computation will have some error, but the engineer can reach a reasonable conclusion about the accuracy of the computer results.

A check of the results for Example 1 with a lateral load of 20,000 lbs is continued by making a plot of the soil resistance, as shown in Fig. 5.2. The concentrated loads with their estimated line of action are shown in the figure. The concentrated loads were found by counting squares. The lines of action were "eyeballed." The out-of-balance of the shear is 1,800 lbs; a satisfactory solution in consideration of the method and that the effect of axial loading is ignored. The out-of-balance of the moment, if moments are taken about the top of the pile, is only 9,200 ins-lbs; again, a satisfactory solution.

Check of Deflection

An examination of the tabulation of the values of deflection for the output being studied shows that a zero slope occurred at a value of x of about 130 inches. A plot of the moment diagram to a depth of 130 ins was made, not shown here, and the second area-moment proposition was employed to compute the deflection at the top of the pile. A value of 0.15 in was computed, which agrees well with the value from COM624P.

Relationships Between Computed Values

The tabulated results from the computer of the graphical results can be checked to see that the equations of mechanics are satisfied at significant points. The following checks can be made: the shear must have maximum values where the values of soil resistance are zero, the moment must have maximum values where the

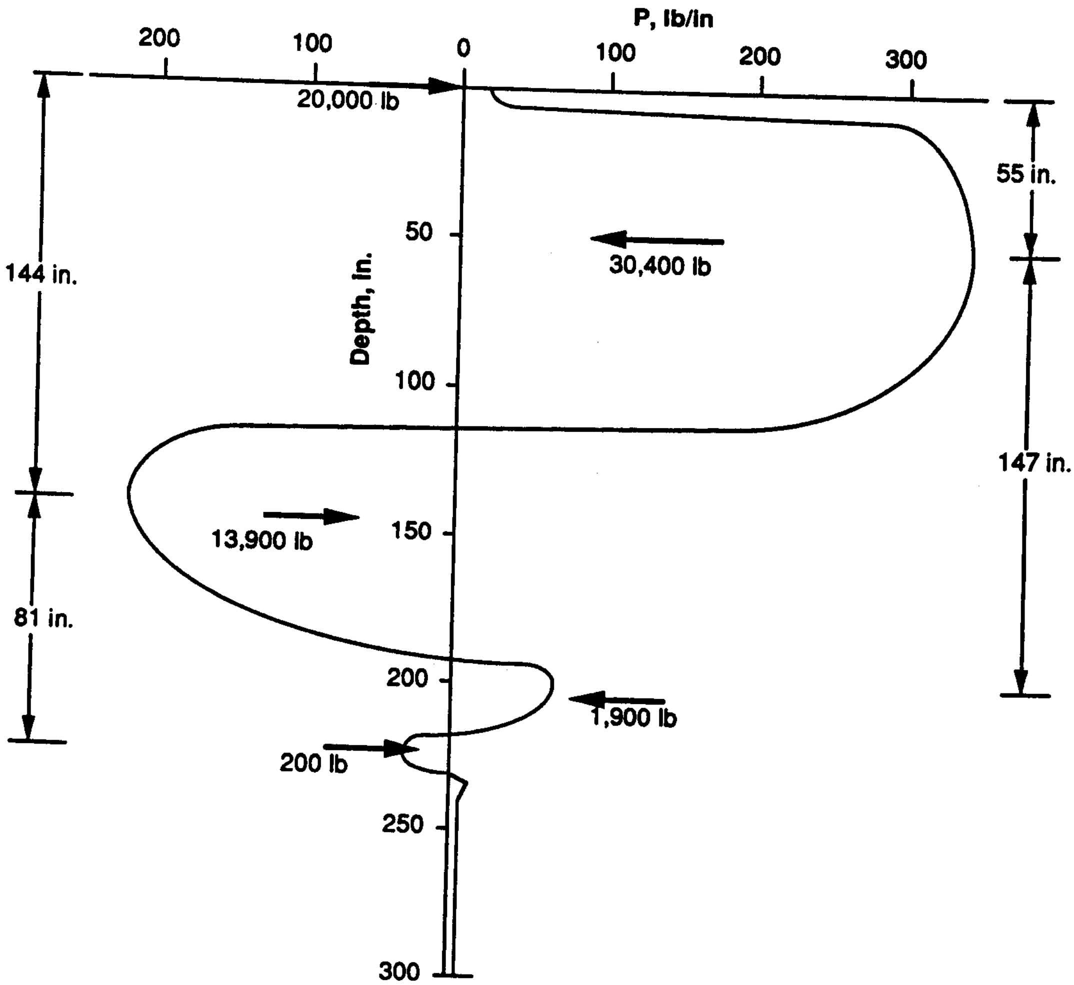


Figure 5.2. Plot of soil resistance for Example 1, lateral load of 20,000 pounds.

values of shear are zero, and the soil resistance must be zero where the deflection is zero.

An examination of the output for Example 1 in Part I, Chapter 5 shows that the checks that are indicated are satisfied.

CHECK BY IMPLEMENTING APPROXIMATE METHODS OF ANALYSIS

The two publications on laterally loaded piles sponsored by FHWA (FHWA, 1984; FHWA, 1986) contain sections on two methods of analysis that yield approximate results. The methods proposed by Broms, (1964a, 1964b, 1965), allow for the computation of the load at which the pile will develop a plastic hinge; then the load can be compared to the comparable load obtained by the computer. The method is based on the equations of static equilibrium and it is approximate; nevertheless, the engineer can obtain a crude evaluation of the accuracy of the computer solution.

The second method of analysis can provide a much closer check of the computer results. With the p-y curves that are either tabulated or presented in graphical form by COM624P, the engineer can use nondimensional curves and check the results of the computer for any particular run. The bending stiffness of the pile should be taken as that for the upper section, the boundary conditions should be used as the lateral loading on the pile at the groundline, and axial loading must be ignored. Even with the approximations that are made in the nondimensional method, the agreement with the computer solutions should be fair to good.

CHECK BY AN ASSOCIATE OR A COLLEAGUE

One of the emerging methods of verification of engineering studies is peer review. Such a technique should be advantageous with respect to the verification of the results of computations with COM624P. If such a procedure were to be implemented, the review should probably be delayed until the computations had been completed for a particular design.

CONCLUDING COMMENT

The use of a calculator to make checks of the output of the computer program is a time-consuming and tedious process. However, such a procedure pays dividends in preventing errors and will give the engineer an excellent understanding of the computational process that is employed in the program. However, after the engineer gains some experience in analyzing the types of piles in the kinds of soils that are usually encountered in the local practice, the correctness of a computer run can readily be judged on the basis of past experience. Some serious checking is advised, however, when a new situation is encountered.

CHAPTER 6. FULL-SCALE TESTING OF PILES

INTRODUCTION

The testing of piles in the field under axial loading is a well-established practice and has been common since piles were first used. The testing of piles under lateral loading is less frequent, perhaps because the means of establishing failure of a pile under lateral load has not become common knowledge. However, with the availability of the technology presented herein, there are benefits to be gained from the performance of full-scale tests of piles under lateral loading. The photograph in Fig. 6.1 is of a test of a drilled shaft at Los Angeles. The testing arrangement is as described later in this chapter.

Figure 6.1. Testing of a drilled shaft under lateral loading, Los Angeles.

There are two general reasons for performing tests of piles in the field under axial loading: to prove a particular design, and to gain information to allow for a redesign (Reese, 1978). These reasons are valid for the test of a pile under lateral loading. Thirdly, in some cases a valuable contribution to the technical literature can be made.

With regard to a proof test of a pile under lateral load, the procedure is not straightforward, because the response of the pile under lateral load is affected strongly by the way the pile is connected to the superstructure. A single-pile foundation for an overhead sign, for example, will be subjected to a shear and a moment. The exact simulation of the pile-head conditions for the sign structure and for a particular kind of loading is impractical if not impossible; therefore, analytical procedures must be employed to interpret the results of virtually any lateral-load test. Such analytical procedures are presented later in this chapter. A standard test is described where no internal instrumentation is used in the pile and where only a relatively small amount of instrumentation is used above the groundline. The standard test can be employed to prove any given design and, in some instances, the standard test can provide information for redesign.

Some information is given on a comprehensive testing program where a pile is instrumented internally for the measurement of bending moment along its length. Precise information on soil response at a particular site can be gained from such a testing program, design information will be specific and valuable, and a contribution to the technical literature can be made. The comprehensive program is expensive and advisable when the benefits are worth the cost.

SELECTION OF TEST SITE

Site selection is simplified if a test is to be performed in connection with the design of a particular structure. However, even in such a case, care should be taken in selecting the precise location of the test pile. In general, the test location should be where the soil profile reveals the weakest condition. In evaluating a soil profile, the soils from the ground surface to a depth equal to five to ten pile diameters are of principal importance. If designed on the basis of the results from the weakest soil, the piles at other places on the construction site should behave more favorably than the test pile.

The selection of a site where a fully instrumented pile is to be tested is usually difficult. The principal aim of such a test is to obtain experimental p-y curves that can be employed in developing predictions of soil response. Thus, the soil at the site must be relatively homogeneous and representative of a type of soil for which predictive equations are needed. For many of the past experiments, the finding of a suitable site was a major problem.

After a site has been selected, attention must be given to the moisture content of the near-surface soils. If cohesive soils exist at the site and are partially saturated, steps may be taken to saturate the soils. If the cohesive soils will be submerged during the life of the structure, the site should be flooded during the testing period.

The position of the water table and the moisture content are also important if the soil at the test site is granular. Partial saturation of sand will result in an apparent cohesion that will not be present if the sand dries or if it becomes submerged.

INVESTIGATION OF SOIL PROPERTIES

The recommendations for obtaining soil properties should be consulted when obtaining data on soils for use in analyzing the results of the lateral-load experiments. Those recommendations make use of the soil parameters of significance.

For cohesive soils, high-quality tube samples should be obtained and laboratory tests should be performed. The Standard Penetration Test is the principal investigative technique for cohesionless soils, but the static-cone-penetrometer test is also recommended.

In performing the soil investigation, careful attention should be given to the near-surface soils, a zone that is frequently given little attention for design of piles under axial loading. As noted previously, the soil strata within a few diameters of the ground surface provide the principal support for a laterally-loaded pile.

SELECTION OF TEST PILE

If a lateral load test is being performed to confirm the design at a particular site, the diameter, stiffness, and length of the test pile should be as close as possible to similar properties of the piles proposed for production. Because the purpose of the test is to obtain information on soil response, consideration should be given to increasing the stiffness and bending-moment capacity of the test pile in order to allow the test pile to be deflected as far as possible. The increased load that will be necessary will usually cause no significant problem.

The length of the test pile must be considered with care. As shown in Fig. 5.1, the deflection of a pile will be significantly greater if it is in the "short" pile range. Tests of these short piles could be hard to interpret because a small change in the pile penetration could cause a large change in the ground-line deflection.

The selection of the test pile for the case of complete instrumentation involves a considerable amount of preliminary analysis. Factors to be considered are: the pile diameter for which the soil response is desired, the soil conditions, the kind of instrumentation to be employed for determining bending moment along the length of the pile, the method of installing instrumentation in the pile, the magnitude of the desired ground-line deflection, and the nature of the loading.

INSTALLATION OF TEST PILE

For cases where information is desired on pile response at a particular site, the installation of the test pile should agree as closely as possible to the procedure proposed for the production piles. The response of a pile to load is affected considerably by the installation procedure; thus, the detailed procedure used for pile placement is important.

For the case of a test pile in cohesive soil, the placing of the pile can cause excess porewater pressures to occur. As a rule, these porewater pressures should have dissipated before testing begins; therefore, the use of piezometers at the test site may be important.

The installation of a pile that has been instrumented for the measurement of bending moment along the length of the pile must consider the possible damage of the instrumentation due to pile driving or other installation effects. The instrumentation must be especially rugged where the pile is to be installed by an impact hammer and where hard driving is expected. However, the installation must be such that it is consistent with methods used in practice. In no case would jetting be allowed with wash water flowing up and along the outside of the test pile.

The influence of the installation procedures on the soil properties should be investigated if possible. However, almost any testing technique prior to the loading would cause soil disturbance and would be undesirable. Some non-intrusive methods are available, based on the use of dynamic methods, that can be considered.

Testing of the near-surface soils close to the pile wall at the completion of the load tests is useful and can be done without any undesirable effects. The kinds of tests that are desirable are indicated where criteria for p-y curves are discussed. In general, laboratory tests of undisturbed samples are recommended.

TESTING PROCEDURES

Excellent guidance for the procedures for testing a pile under lateral loading is given by the ASTM Standard D 3966-81, "Standard Method of Testing Piles Under Lateral Loads." Some general comments on the ASTM standard are given in this section, and detailed recommendations are given in the following sections.

For the standard test as well as for the instrumented test, two principles should guide the testing procedure: (1) the loading (static, repeated, sustained, or dynamic) should be consistent with that expected for the production piles and (2) the testing arrangement should be such that deflection, rotation, bending moment, and shear at the groundline (or at the point of load application) are measured or can be computed.

With regard to loading, even though static (short-term) loading is seldom encountered in practice, the soil response from that loading is usually desirable so that correlations can be made with soil properties. The combination of static and repeated loading may be desirable. A load can be applied, readings taken, and the same load can be reapplied a number of times with readings taken after specific numbers of cycles. Then, a larger load is applied and the procedure repeated. The assumption is made that the readings for the first application at a larger load are unaffected by the repetitions of a smaller load. While that important assumption may not be strictly true, errors are on the conservative side.

Sustained loads will probably have little influence on the behavior of granular materials or on overconsolidated clays if the computed values of soil stresses are well below ultimate. If a pile is installed in soft, inorganic clay or other compressible soil, sustained loading would obviously influence the soil response. In general, loads would have to be maintained for a long period of time and a special testing program would have to be designed. However, data can be obtained in a period of several days or a few weeks that can allow extrapolation to results for a long period.

The application of a dynamic load to a single pile is feasible and desirable if the production piles sustain such loads. The loading equipment and instrumentation for such a testing program would have to be designed to yield results that would be relevant to a particular application, and a special study would be required. The design of piles to withstand the effects of an earthquake involves several levels of computation. Soil-response curves must include an inertia effect and the free-field motion of the earth must be estimated. Therefore, p-y curves that are determined from the tests described herein have only a limited application to the earthquake problem. No method is currently available for performing field tests of piles to gain information on soil response that can be used directly in design of piles to sustain seismic loadings.

The testing of battered piles is mentioned in ASTM D 3966-81 (also see FHWA-IP-84-11, Appendix 8). The analysis of a pile group, some of which are batter piles, is discussed in the technical literature. In such analyses, information is required on the behavior of battered piles under a load that is normal to the axis of the pile. Unless the batter is large, the behavior of battered and vertical piles under this normal load (lateral load) is similar. For large batter, an approximate solution is given in FHWA-IP-84-11 (page 300).

The testing of pile groups, also mentioned in D 3966-81, is desirable but is expensive in time, material, and instrumentation. If a large-scale test of a group of piles is proposed, detailed analyses should precede the design of the test in order that measurements can be made that will provide critical information. Such analyses may reveal the desirability of internal instrumentation to measure bending moment in each of the piles.

The analysis of test results is not covered in D 3966-81. The argument can be made, as presented earlier in this chapter, that test results can fail to reveal critical information unless combined with analytical methods. The next section of this chapter suggests procedures that demonstrate the close connection between testing and analysis. A testing program should not be initiated unless preceded and followed by analytical studies.

The ASTM standard mentions methods of dealing with the lateral soil resistance against a pile cap. A conservative procedure, and one that is consistent with reality in many instances, is to assume that there is no soil resistance either against the sides or the bottom of the cap. A small amount of settlement would eliminate the bottom resistance, and shrinkage would eliminate the side resistance. Therefore, it is recommended that a pile cap not be used in the testing program or, if used, that the cap not be placed against the soil.

ASTM D 3966-81 gives a number of procedures for applying load and for measuring movements. Some details, generally consistent with the ASTM standard, about methods that have been found to be satisfactory are given in the next section. With regard to a loading schedule, ASTM indicates that loading should be applied in increments to a maximum of 200 to 250 percent of the design load. However, it is rarely possible to perform a test with the rotational restraint at the pile-head exactly the same as for production piles; thus, an alternate suggestion is made that the loading be continued in increments until the pile actually fails due to the development of a plastic hinge. Or, the loading can be continued until the bending stress becomes equal to a certain percentage of the ultimate, as indicated by computations.

The sections in D 3966-81 on safety requirements and report presentation are worthy of careful consideration. Safety is an important concern in load testing and safety meetings prior to any load test should be held. The detailed list in the section on reporting is useful and indicates most of the items that should be addressed in preparing a report.

TESTING PILE WITH NO INTERNAL INSTRUMENTATION

A step-by-step procedure is given in the following paragraphs for the testing of a pile or piles with no internal instrumentation, termed the standard test because of its simplicity and ease of performance. The test program is initiated with a study to indicate the economic advantages of the experiment. It is presumed that a careful subsurface investigation with laboratory testing has been carried out and that soil properties are well known. The soil properties near the ground surface are especially important.

Preliminary Computations

After the type and size of pile has been selected for testing, preliminary computations should be made using the computer code described herein. The computations should anticipate that the pile head should be free to rotate and that the shear should be applied near the ground surface. Analyses should be done using p-y curves for both static and repeated loading. Curves showing pile-head deflection and pile-head rotation should be developed for a range of loading up to the point where the ultimate moment is developed.

Computations should be done with parameters varied, and the length of the test pile and its bending stiffness should be selected on the basis of the computations.

Obtaining Stiffness of Test Pile

The bending stiffness of the test pile or piles can be found by computation, but it is preferable to obtain the stiffness experimentally. If the pile consists of a pipe or some other prefabricated section, rather than a cast-in-place pile, it is possible to support the pile near its ends in the laboratory and load it as a beam. The stiffness of the pile can be computed from the deflection.

For a cast-in-place section, or for a prefabricated section as well, several feet of soil around the pile can be excavated after the primary testing program is completed. The pile can be reloaded and deflections can be measured at several points along the exposed portion of the pile. If this latter procedure is to be employed, the lateral loading should have been stopped before the pile was damaged.

The stiffness of drilled shafts and other reinforced-concrete sections will vary with bending moment. Some information on this variation can be obtained from the field measurements described above. That information, along with the use of the code for PMEIX, should provide engineers with adequate data on stiffness of reinforced-concrete sections.

Pile Installation

As noted earlier, the installation of a test pile should be done in the same manner as for the production piles. Small amounts of accidental batter will have little influence on the performance of a pile under lateral load. Care should be exercised in installation that the near-surface soils have the same properties as for the production piles.

Loading Arrangement

A wide variety of arrangements for the test pile and the reaction system are possible. The arrangement to be selected is the one that has the greatest advantage for the particular design. There are some advantages, however, in testing two piles simultaneously as shown in Fig. 2 of D 3966-81. A reaction system must be supplied; thus, a second pile can supply that need. Furthermore, and more importantly, a comparison of the results of two tests performed simultaneously will give the designer some idea of the natural variations that can be expected in pile performance. It is important to note, however, that spacing between the two piles should be such that the pile-soil-pile interaction is negligible.

Drawings of two two-pile arrangements are shown in Figs. 6.2 and 6.3. In both instances the pile head is free to rotate and the loads are applied as near the ground surface as convenient. In both instances free water should be maintained above the ground surface, if that situation can exist during the life of the structure.

The details of a system where the piles can be shoved apart or pulled together are shown in Fig. 6.2. This two-way loading is important if the production piles can be loaded in that manner. The lateral loading on a pile will be predominantly in one direction, termed the forward direction here. If the loading is repeated or cyclic, a smaller load in the reverse direction could conceivably cause the soil response to be different than if the load is applied only in the forward direction. As noted earlier, it is important that the shear and moment be known at the ground line; therefore, the loading arrangement should be designed as shown so that shear only is applied at the point of application of load.

Figure 6.3 shows the details of a second arrangement for testing two piles simultaneously. In this case, however, the load can be applied in only one direction. A single bar of high-strength steel that passes along the diameter of each of the piles is employed in the arrangement shown in Fig. 6.3. Two high-strength bars are utilized in the arrangement shown in Fig. 6.3. Not shown in the sketches are the means to support the ram and load cell that extend horizontally from the pile. Care must be taken in employing the arrangement shown in either Figs. 6.3a or 6.3b to ensure that the loading and measuring systems will be stable under the applied loads.

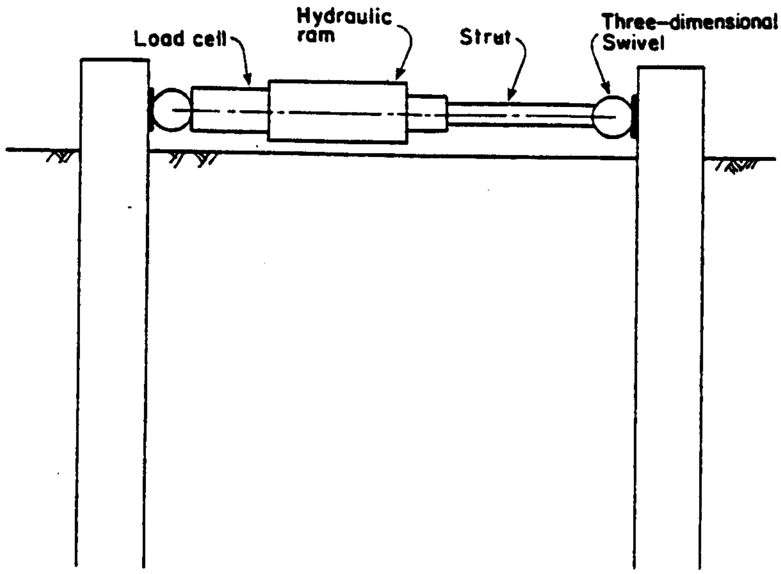


Figure 6.2. Two-pile test arrangement for two-way loading.

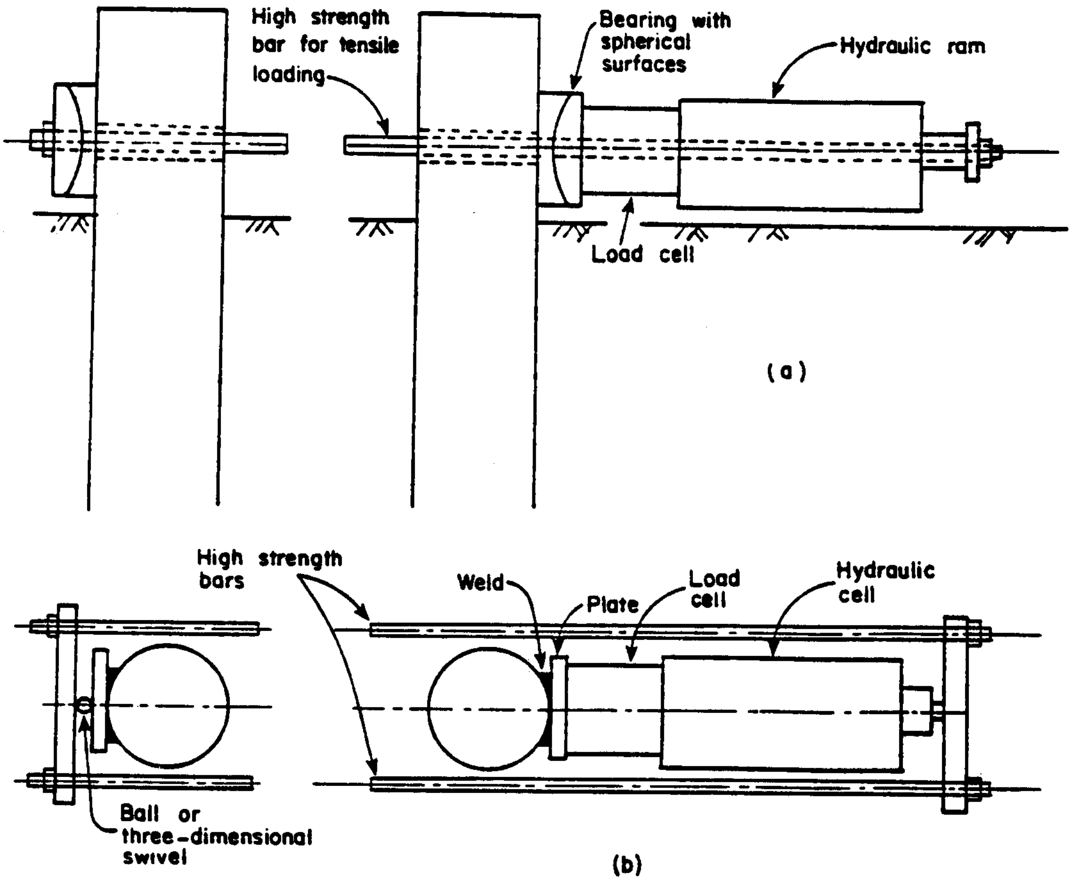


Figure 6.3. Two-pile test arrangement with one-way loading.

The most convenient way to apply the lateral load is to employ a hydraulic ram with hydraulic pressure developed by an air-operated or electricity-operated pump. The capacity of a ram is computed by multiplying the piston area by the maximum pressure. Some rams, of course, are double acting and can apply a forward or reverse load on the test pile or piles. The preliminary computations should ensure that the capacity and the travel of the piston are ample.

If the rate of loading is important (and it may be if the test is in clay soils beneath water, and erosion at the pile face is important), the maximum rate of flow of the pump is important along with the volume required per inch of stroke of the ram. The seals on the pump and on the ram, along with hydraulic lines and connections, must be checked ahead of time and spare parts should be available.

High pressures in the operating system constitute a safety problem and can cause operating difficulties. On some projects, the use of an automatic controller for the hydraulic system is justified. A backup control must be available to allow the override of the automatic system in case of malfunction. On at least one important project the malfunction of the hydraulic system caused a large monetary loss.

The loading system shown in Fig. 6.3 will ensure that no eccentricity will be applied to the load cell and the hydraulic ram. If the two-bar system shown in Fig. 6.3(b) is employed, it should be even simpler to achieve concentric loading. However, the system shown in Fig. 6.2 will require that the load cell and the ram be connected rigidly and that bearings be placed at the face of each of the piles so that no eccentric loading is applied to the ram or to the load cell. The arrangement shown in Fig. 6.2

may require that the points of load application be adjustable in order to prevent torsional loading of the piles.

Instrumentation

A simple system for obtaining the deflection and rotation of the pile head is shown in Fig. 6.4. The slope or rotation of the portion of the pile above the point of load application can be found by knowing the gauge readings and the distance between them. The same data will yield the deflection at the point of load application. In the test shown in Fig. 6.1, a casing was attached to the rebar cage prior to concreting and a slope indicator was used to measure the slope (or rotation) of the drilled shaft over its full length.

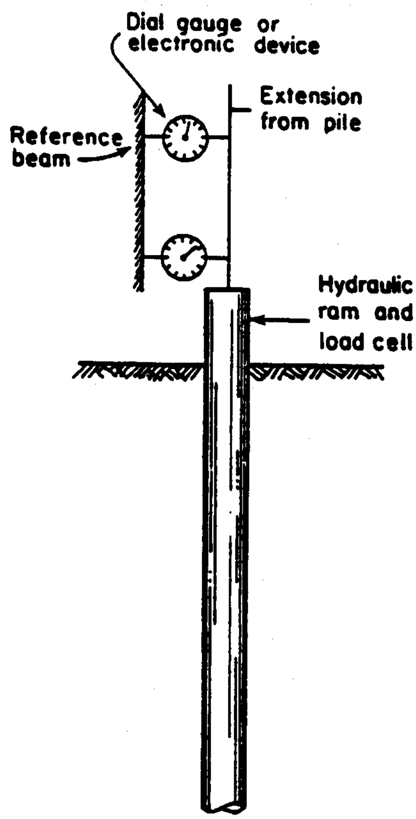


Figure 6.4. Schematic drawing of deflection-measuring system.

An alternate or redundant method of obtaining the pile-head rotation is shown in Fig. 6.5. A sensitive bubble for leveling a bar is attached as shown. A micrometer is fixed to one end of the bar and a hardened point to the other. A sturdy bracket is attached to the pile, or to an extension of the pile, at a convenient distance above the point of load application. Readings of the micrometer when the instrument is carefully leveled for each load will allow pile-head rotation to be computed.

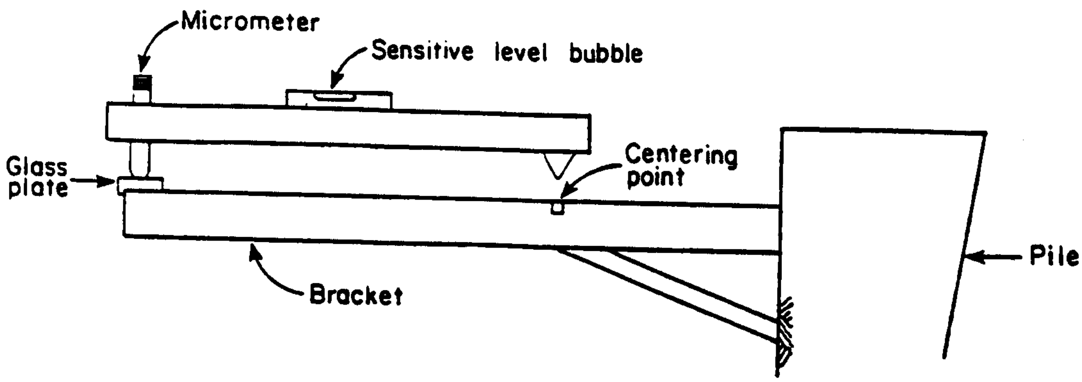


Figure 6.5. Device for measuring pile-head rotation.

Electronic load cells are available for routine purchase. These cells can be used with a minimum of difficulty and can be read with a high-speed data-acquisition system, if desired.

The motion of the pile head can be measured with dial gauges, but a more convenient way is to employ electronic gauges. In either case, gauges with sufficient travel should be obtained or difficulty will be encountered during the test program. Two types of electronic motion transducers are in common use: linear potentiometers or LVDT's (linear variable differential transformers). The LVDT may have a longer life than the differential potentiometers; in either case the motion transducer should be attached so that there is no binding as the motion rod moves in and out.

Two other comments about instrumentation are important. The verification of the output of each instrument should be an important step in the testing program. Also, the instruments should be checked for temperature sensitivity. In some cases it may be necessary to perform tests at night or to protect the various instruments from all but minor changes in temperature.

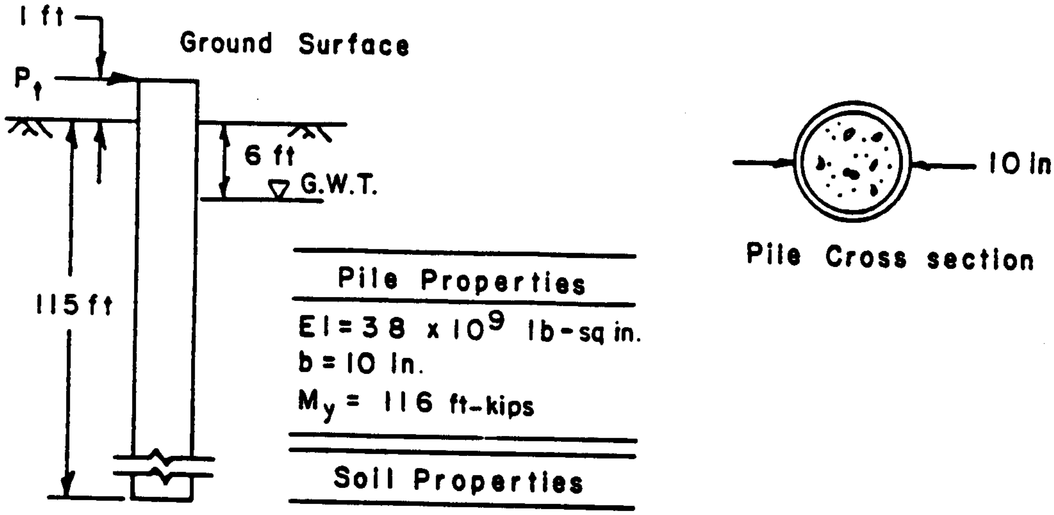
Interpretation of Data

The interpretation of data from a test of an uninstrumented piles is a straightforward process. Plots are made of deflection versus applied load and rotation versus applied load (for the ground line or for the point of load application). Computer Program COM624P is then used, and computations of pile-head deflection and rotation are made for the same loads that were used for the field test. The results are plotted against the field results. If the results do not agree, the soil parameters (probably the shear strength of clay and angle of internal friction of sand) are changed by trial to bring the computed and experimental results into agreement. (Most of the interpretation will be done in the office; however, it is desirable to do some plotting in the field as a means of checking the validity of the data that are being taken).

The soil parameters as modified are then used in making a design for the site. An appropriate factor of safety, normally introduced as a load factor to increase the working load, is employed, taking into account all of the relevant considerations.

Example Computations

The test selected for study was performed by Capozzoli (1968) near St. Gabriel, Louisiana. The pile and soil properties are shown in Fig. 6.6. The loading was short term. The soil at the site was a soft-to-medium, intact, silty clay. The natural moisture content of the clay varied from 35 to 46 percent in the upper 10 ft of soil. The undrained shear strength, shown in Fig. 6.6, was obtained from triaxial tests. The unit weight of the soil was 110 lb/ft^3 above the water table and 48 lb/ft^3 below the water table.



Pile Properties
EI = 38 x 10^9 lb-sq in.
b = 10 in.
M_y = 116 ft-kips

Soil Properties
Depth = 0–115 ft
c = 600 lb/sq ft
ϵ_{50} = 1%
γ = 110 lb/cu ft

Figure 6.6. Information for analysis of test at St. Gabriel.

The results from the field experiment and computed results are shown in Fig. 6.7. The experimental results are shown by the open circles; the results from Computer Program COM624P with the reported shear strength of 600 lb/ft^2 and with an ε_{50} of 1.0 percent are shown by the solid line. The soil properties were varied by trial and the best fit to the experimental results was found for an undrained strength of 887 lb/ft^2 and an ε_{50} of 0.9 percent. These values of the modified soil properties should be used in design computations for the production piles if the production piles are to be identical with the one employed in the load test.

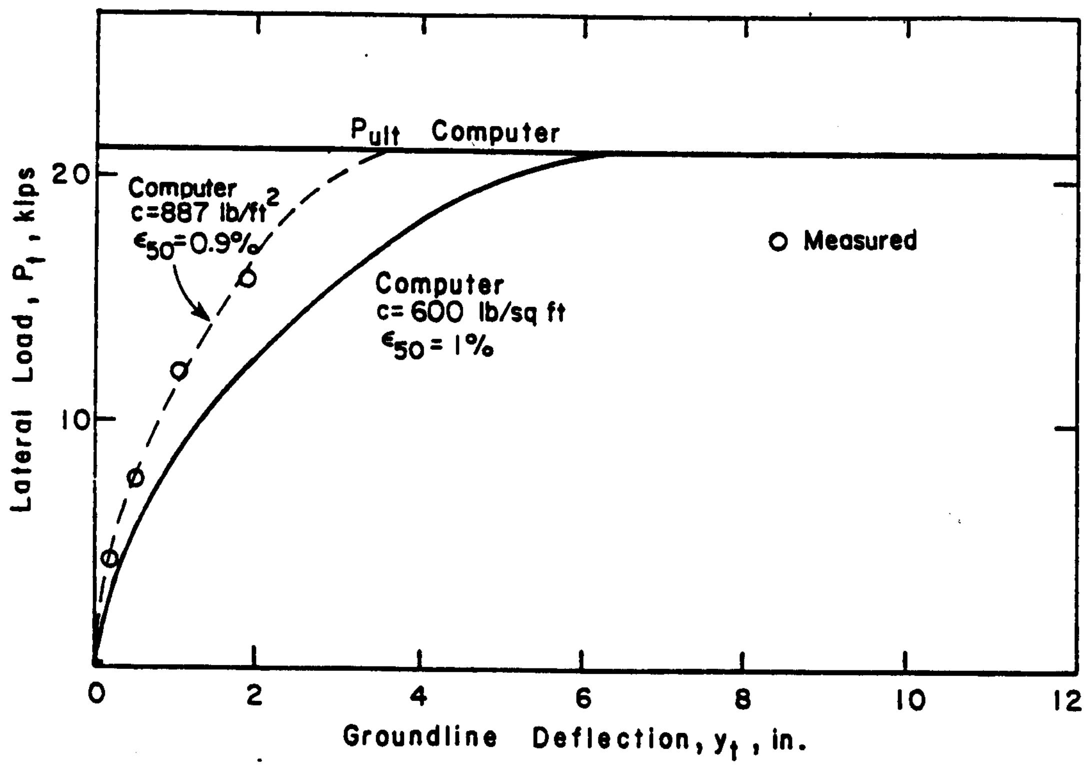


Figure 6.7. Comparison of measured and computed results for St. Gabriel Test.

Computer Program PMEIX was employed and an ultimate bending moment for the section in Fig. 6.6 was computed to be 1392 in-kips. In making the design computations with the modified soil properties, the computed maximum bending moment should be no greater than the ultimate moment (1392 in-kips) divided by an appropriate factor of safety. In computing the maximum bending moment, the rotational restraint at the pile head must be estimated as accurately as possible. If it is assumed that the pile will be unrestrained against rotation and that the load is applied one ft above the ground line, a load of 21 kips will cause the ultimate bending moment to develop. The deflection of the pile must be considered because deflection can control some designs rather than the design being controlled by the bending resistance of the section.

Two other factors must also be considered in design. These are: the nature of the loading and the spacing of the piles. The experiment employed short-term loading; if the loading on the production piles is to be different, an appropriate adjustment must be made in the p-y curves. Also, if the production piles are to be in a closely-spaced group, consideration must be given to pile-soil-pile interaction.

TESTING PILE WITH INTERNAL INSTRUMENTATION

The performance of experiments with piles that are instrumented internally for the measurement of bending moment along the length of the pile is highly desirable. The results of experiments that are carefully performed will allow experimental p-y curves to be developed; thus, significant information can be added to the technical literature. In addition, of course, excellent data will be available to guide the design of piles at the test site. However, the performance of experiments with piles that have internal instrumentation is expensive, both in labor and materials. In addition, instrumentation specialists with excellent skills are required. Therefore, a detailed cost-benefit study should be undertaken before such a test program is begun.

Preliminary Computations

If a major experiment with a pile with internal instrumentation is to be undertaken, the preliminary computations should be exhaustive. Assuming that the test site and the pile geometry have been selected and that soil properties are known, computations must be performed to get the best possible estimate of the response of the pile. On the basis of these computations, the nature of the loading system is decided upon and a detailed design of the system is made.

The preliminary computations also allow the selection of the kind of internal instrumentation that is to be employed and a detailed design of the instrumentation is then made. Electrical-resistance strain gauges are frequently employed to read strains in the pile material. The test pile can be calibrated by supporting the pile as a beam, applying known measurements at positions of strain gauges, and reading the output of each gauging point. If a drilled shaft is employed in the test, an instrumented pipe can be cast along the axis of the shaft and calibration can be done after the test is over by removing soil around the pile to as great a depth as possible and reloading the pile (Welch and Reese, 1972).

Further computations must be carried out to ensure that the pile is not damaged if it is to be installed by driving. Diligence in planning and in performing preliminary computations can do much to ensure the success of the expensive instrument.

Instrumentation

The instrumentation that is placed above the ground is similar, if not identical, to that described for the pile with no instrumentation. While the principal item of internal instrumentation pertains to a direct determination of bending moment from point to point along the pile, the use of a slope indicator from which deflections can be obtained is sometimes desirable. If space allows and if the loading schedule that is proposed will allow a slope indicator to be used, the installation of slope-indicator casing may be warranted.

As noted above, the use of strain gauges to enable bending moments to be obtained is a common practice. However, innovative techniques are being developed regularly, and the selection and installation of the internal instrumentation should follow a careful study of available methods.

Some investigators have made measurements of ground-surface movements during the lateral loading of a pile. The placing of markers on a grid pattern around the test pile and the measurement of the movement of those markers are time-consuming and cumbersome. The use of photographic techniques to obtain ground-surface movements has much to recommend it.

Analysis of Data and Correlations with Theory

The principal analytical technique is to perform two integrations of the bending moment curves and two differentiations. The boundary conditions at the head of the pile must be employed in the analysis. The integrations yield the pile deflections; with reasonably good moment curves and with good measurements of the boundary conditions at the pile head, an accurate family of curves giving deflection of the pile as a function of depth can be obtained.

The two differentiations are another matter. Errors in the moment values are accentuated. Therefore, it is usually necessary to employ curve-fitting techniques and obtain analytical expressions for selected portions of the moment curves. If the differentiations can be carried out successfully, the result will be a family of curves showing soil resistance as a function of depth. Specific depths can be selected and cross-plotting will yield a family of p-y curves.

An additional step in the analytical process is to employ the principles of soil mechanics and of elasticity to develop predictive equations for pile response. Ideally, the predictive equations should agree with the experimental results at the test site and should further serve to predict the behavior of piles of different geometry at the test site and at other sites where the soils are similar. The predictive equations will be valid, of course, only for the kind of loading employed at the test site.

Review of Three Experiments Using Piles with Internal Instrumentation

Matlock (1970) performed experiments near Austin, Texas, and near Sabine, Texas, in soft to medium clay. The pile was a steel pipe, 12.75 ins in diameter. Thirty-five pairs of electrical-resistance strain gauges were installed in the interior of the pipe. The gauges were spaced 6 ins apart near the top of the embedded portion with wider spacings being used below. The embedded portion of the pile was 45 ft long. The pipe was split along a diameter, the gauges were installed, and the two halves were welded together.

The pile was calibrated prior to driving so that extremely accurate determinations of bending moment could be made. The experimental p-y curves that were obtained from the testing program form the basis of recommendations that are widely used for design of piles in soft clay under lateral loading.

Cox, Reese, and Grubbs (1974) performed experiments in sand near Corpus Christi, Texas. The piles were steel pipes, 24 ins in diameter. Forty electrical-resistance strain gauges were installed in each of the two piles by placing the piles horizontally and by working from a trolley. Two piles were driven at the same site; one pile was tested under static loading and the other under cyclic loading. The embedded length of each pile was 69 feet.

The piles were calibrated in the laboratory prior to installation. The experimental p-y curves that were obtained from the testing program form the basis of recommendations that are widely used for the design of piles in sand under lateral loading.

Welch and Reese (1972) conducted a test of an instrumented drilled shaft with a nominal diameter of 30 in, a total length of 44 ft, and a penetration of 42 feet. The soil at the site was an overconsolidated clay with an average undrained shear strength in the upper 20 ft of approximately 2,200 lb/ft^2. Average values of liquid limit and plastic limit were 70 and 20, respectively. The water table was at a depth of 18 feet.

A steel pipe, with an outside diameter of 10.75 in and with a wall thickness of 0.25 in, was split longitudinally and strain-gauge rosettes were installed on each half to form a full bridge at 28 points along the drilled shaft. Twenty-three gauge points were at spacings of 15 in from the top and there were 5 spaces at 30 in near the bottom.

The lateral loads were applied at the groundline and in one direction only. Readings were taken after 1, 2, 5, 10, 15, and 20 applications of load. Readings were taken with a data logger of the gauges at the top of the drilled shaft and of all of the gauges for the measurement of bending moment. After the loadings were completed, an excavation to a depth of 20 ft was made around the drilled shaft and a loading was made to obtain data for determination of the as-built values of bending stiffness (EI), as well as calibrations for each of the gauge points so that strain-gauge readings could be converted directly into bending moment.

Curves for each of the applied loads were plotted to show bending moment as a function of depth. A study of curve-fitting techniques was done and the result indicated that the best results could be obtained with least-squares using a 7-degree polynominal. The values of deflection (y) and soil resistance (p) were obtained at various points along the drilled shaft by using the following equations.

$$y = \iint (M/EI)\ dx \tag{6.1}$$

$$p = \frac{d^2M}{dx^2} \tag{6.2}$$

The resulting p-y curves for the depth of 37.5 in below the groundline are shown in Fig. 6.8. The curves illustrate the effect of cyclic loading. The soil resistance is decreased or the deflections are increased with cyclic loading. The authors decided to take cyclic loading into account by increasing the computed deflection for static loading by an increment that takes into account the stress level and the number of cycles (Welch and Reese, 1972). The resulting equations are implemented in the recommendations for p-y curves that are shown earlier in this document.

CONCLUDING COMMENTS

Only a brief presentation is possible concerning the details of a program of testing of piles under lateral load. The brevity of the presentation is consistent with the purposes of this document and is not meant to detract from the importance of the topic.

Simple, inexpensive experiments can be performed with piles with no internal instrumentation and data of great value can be obtained concerning the response of a pile at a particular site.

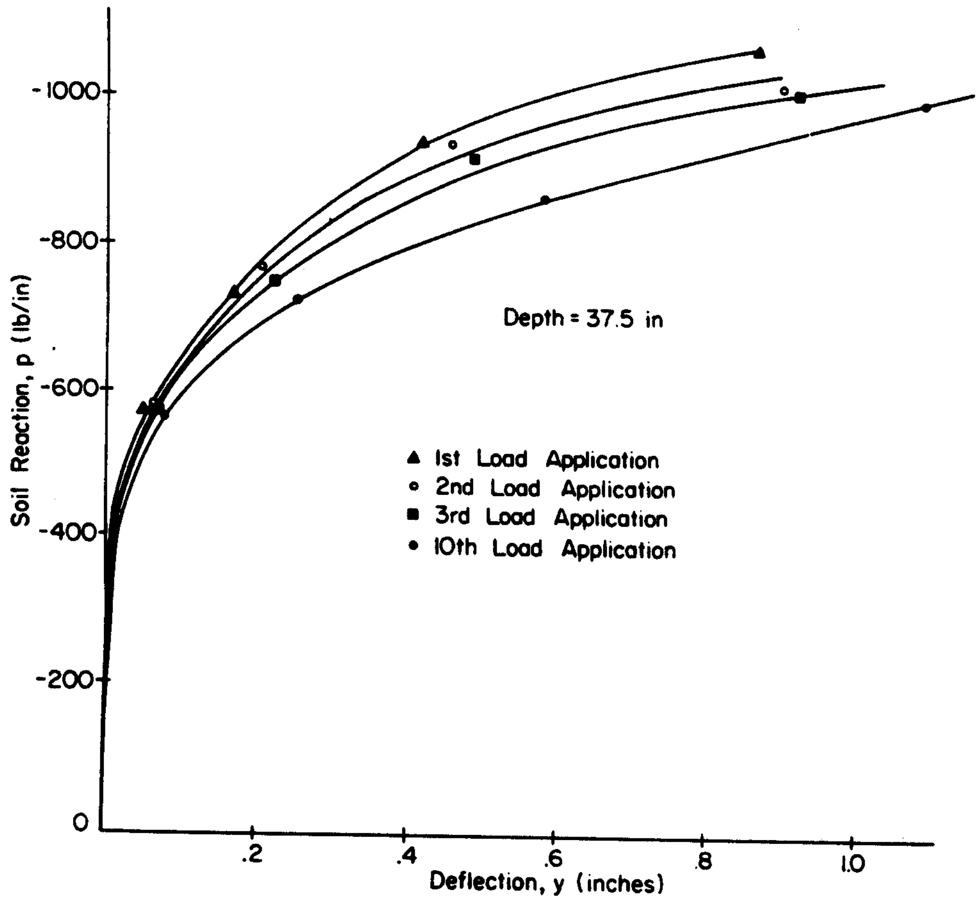


Figure 6.8. Effect of repeated loading on p-y curves.

The performance of tests of piles with internal instrumentation can well be justified at the site of a major project, especially if the current methods of predicting p-y curves are not exactly applicable to the soil, pile, and loading to be employed at the site. In addition to getting data for the design of a particular project, data will be obtained for use at similar sites. Also, a contribution can be made to the engineering profession.

Redundancy in load-measuring and deflection-measuring systems is good practice. Rams can be calibrated as a means of checking readings from load cells. Stretched wires or surveying instruments can be employed to check deflection. Such redundancy can be extremely useful in case of the failure of a primary system of measurement.

The available data are insufficient to allow a comment to be made that all field tests of piles under lateral loading are cost effective. However, the tests that have been performed appear to have saved money on specific projects. The tests of instrumented piles have paid for themselves many times over. The investigation of the benefits from performing field tests of piles under lateral loading for a specific project is strongly advised.

CHAPTER 7. SUGGESTIONS FOR DESIGN

A conference on deep foundations several years ago was opened by an address by Dr. Ralph B. Peck (1967). He gave factors that related to a successful design: loading, soil conditions, theory, tolerance to differential movement, and quality of construction. Dr. Peck concluded his address by saying that very few problems with foundations could be attributed to inadequacy of theory but that most of them were due to misjudgment of loading or of soil conditions or to construction defects. The computer program COM624P might fit into the category of theory in Dr. Peck's list; however, it will be exceedingly difficult to design a pile properly to sustain lateral loading without a suitable analytical tool. The point that there are many important aspects of design in addition to analysis is well taken.

There is a movement in several countries to design on the basis of "limit states" and "partial safety factors." The concept of limit states is to identify all of the reasons that a particular design could fail to perform its assigned function during the life of the structure; in other words, reasons that would limit the usefulness of the construction. Partial safety factors are employed to find a global safety factor for a component of a structure, say a pile. A partial factor would be applied to loads, soil properties, theory, construction, and so on, in consideration of the engineer's evaluation of how well each of the items could be evaluated. The two concepts are used rarely in the United States but the engineer brings experience, training, and judgment to bear rather then using a formalized approach to the selection of the factor of safety. Such attributes of the engineer are essential with respect to the design of piles under lateral loading.

SELECTING AN APPROPRIATE MODEL FOR A PILE

As noted earlier, two aspects of the response of a pile must be considered in design: the development of a plastic hinge, or hinges, along the length of the pile; and excessive deflection. In both instances, the pile must be modeled properly. As an example, a pile is considered that is attached to the superstructure at some distance above the groundline. In many instances, the pile head is neither free to rotate nor fully fixed against rotation but is somewhere in between. In some instances, the determination of the degree of rotational restraint is difficult. Then, some parametric studies can be done with the rotational restraint varied. The computer output usually will allow the engineer to decide whether or not to proceed with an uncertain pile-head condition or to design one with a predictable amount of rotational restraint.

With regard to the axial load and bending moment that will result in the formation of a plastic hinge, formulas are available for steel piles. However, a computer program is desirable for the determination of the ultimate moment for a reinforced-concrete pile. The bending stiffness, EI, of the reinforced-concrete pile will vary with the loading. Thus, the output from the computer will enable the engineer to select the EI to agree with the bending moment when modeling the pile.

The axial load that acts on a pile subjected to lateral load usually has only a small effect on the bending moment; therefore, the assumption is made that the axial load is constant over the length of the pile being analyzed by COM624P. Any error resulting from that assumption is thought to be extremely small. In almost all instances the reduction in axial load from the groundline to the point of maximum bending moment will be negligible.

If the above assumption is valid, the behavior of a pile under axial load and under lateral load can be solved independently. However, in computing axial load of a pile in clay by use of soil properties, the engineer could decide to eliminate the clay from the first point of zero lateral deflection to the groundline in computing the axial load. The clay can be molded away by lateral deflection and lose at least some of it ability to transfer axial load.

STUDY OF INFLUENCE OF VARIOUS PARAMETERS

A number of suggestions have been made in previous sections about the desirability of varying certain parameters in the input and observing their influence. The engineer-user is urged to extend the procedure to most of the factors that must be input. The results of some such studies are given elsewhere (FHWA/RD-85/106, pp. 197-210) but the studies are more meaningful if done for local subsurface conditions and for the piles that are more commonly used in practice.

The parametric studies are especially useful with respect to the properties of the soil. An informative set of computations is to put the maximum and minimum values of soil strength that could be reasonably expected at a given site, and obtain upper-bound and lower-bound solutions. Such a study would give excellent guidance on the benefits of a comprehensive study of the subsurface conditions. In connection with the study of the influence of soil properties, the shear strength and other relevant properties should be varied with depth. Contrary to the need for piles under axial loading, the properties of the near-surface soils are very significant. Studies of this sort could lead to a change in the way that soils are sampled and tested when a pile under lateral loading is to be designed.

CASE STUDIES

An exercise that is of considerable use is to compare results from analysis with those for experiments. Some offices have a number of such cases in the engineering files and the technical literature contains a number of reports on lateral-load tests. One of the difficulties in making such comparisons is that the results of the experiments are frequently incomplete. Typical data that are missing are the point of application of the lateral load and the bending stiffness of the test pile.

Comparisons of the results from analysis and from experiments have been reported (FHWA/RD-85/106, pp. 211-244). Reference to these studies and to those made by the user-engineer will provide valuable information of the accuracy that can be expected from COM624P.

FIELD TESTS

Upper-bound and lower-bound studies and case studies will provide an excellent background for making a decision about the desirability of running a field test at the construction site. A full-scale test of the proposed production piles might be necessary at locations where a number of piles are to be installed and where the lateral loading is significant. Procedures for such tests are given in some standards; however, one feature in some such recommendations is inappropriate. That is, it is virtually impossible in most cases to provide the rotational restraint at the pile head that will exist in the structure to be constructed. Therefore, the testing program should be aimed at determining the response of the soil (FHWA, 1984, pp. 176-178) The response so obtained can be used to design piles of various diameters and with various bending stiffnesses.

A desirable testing technique is to test two identical piles simultaneously by jacking them apart or pulling them together (FHWA, 1984, pp. 169-175). The difference in the response of the two piles will give the engineer some idea of the amount of variability that could be expected with the production piles.

TECHNICAL ADVANCES

The engineer-user of COM624P may wish to search the new technical literature for articles on piles under lateral loading. Of particular interest will be articles that deal with the testing of fully instrumented piles to obtain p-y curves and articles on the response of pile groups to lateral loading. Research is underway in the United States and in many foreign countries on lateral loading of piles, and there undoubtedly will be a number of advances in the state-of-the-art from time to time.

COM624P
LATERALLY LOADED PILE ANALYSIS PROGRAM FOR THE MICROCOMPUTER
Version 2.0

Part III: System Maintenance Manual

CHAPTER 1. GENERAL

The documentation for Computer Program COM624P consists of three documents: Part I, Users Guide; Part II Engineering Background; and Part III, Systems Maintenance.

The necessity of maintaining a computer program, regardless of its length and simplicity, is essential and procedures for the maintenance must be clear. Maintenance assures the user that the program is operating satisfactorily and that information can be obtained about the program when necessary. As a part of the maintenance, the user should be able to obtain a rapid response to questions as they arise.

The expected use of COM624P in the solution of problems in foundation engineering imposes some special requirements regarding the maintenance of the program. It is expected that copies of the program will be distributed to one or more offices of the departments of highways of the States, and possibly to other transportation agencies in the States. The components of the program to be distributed are the user guide, and floppy disks with an executable object code, example problems, and graphics capabilities. The object codes are to be permanently identified as being distributed to a particular engineering office.

The source code will remain at an appropriate office in the Federal Highway Administration where the maintenance of COM624P will reside. After the initial period when ENSOFT will maintain the program according to the terms of the contract, it is expected that FHWA will assign maintenance responsibilities to two persons: a geotechnical engineer, and a computer programmer. However, only minimal questions are expected to arise that must be answered.

The terms of the contract for Version I require that ENSOFT provide technical assistance for 18 months from the date the program was submitted to June of 1990. The principal activities related to the maintenance of COM624P were handled by ENSOFT during that period. It is expected that many of the users will receive Version 2.0 of the program and that FHWA staff, along with the assistance of Ensoft, will help to address most of the questions that are expected to arise from the users in 1993.

The USERS GUIDE was prepared in a manner to allow almost all of the questions that arise on the part of the user to be answered by referring to the document. In this connection, some of the material that might be normally placed in the SYSTEM MAINTENANCE MANUAL is included in the USERS GUIDE. The experience of ENSOFT with similar programs has led to the careful selection of material so that the user can use COM624P with assurance.

CHAPTER 2. SYSTEM DESCRIPTION

SYSTEM APPLICATION

The program computes deflection, shear, bending moment, and soil response with respect to depth for a laterally-loaded pile in nonlinear soils. There are three major components of this software for the microcomputer for the analysis of laterally-loaded piles. The system consists of a menu-input preprocessor (C624EDIT), a main program for the analyses (COM624P), and a postprocessor for display of graphics (C624VIEW). A flowchart showing the interrelationships of the major components of the system is provided in Fig. 2.1.

SYSTEM ENVIRONMENT

Hardware Requirements

The system was developed for the IBM-XT and the IBM-AT machines or for any other microcomputer that is compatible with those IBM machines. The microcomputer should have at least 256k of RAM memory. A high-speed, floating-point math coprocessor is highly recommended but it is not required.

Two peripheral devices are required for the display of graphics: a graphics adapter and a graphics printer. The specific graphics adapters and printers supported by the system are described in Chapter 3 of the USERS GUIDE.

Software Requirements

The operating system that is required to run the program is MS-DOS 2.1 or a later version. The main program is written in FORTRAN language with double-precision arithmetic. A FORTRAN

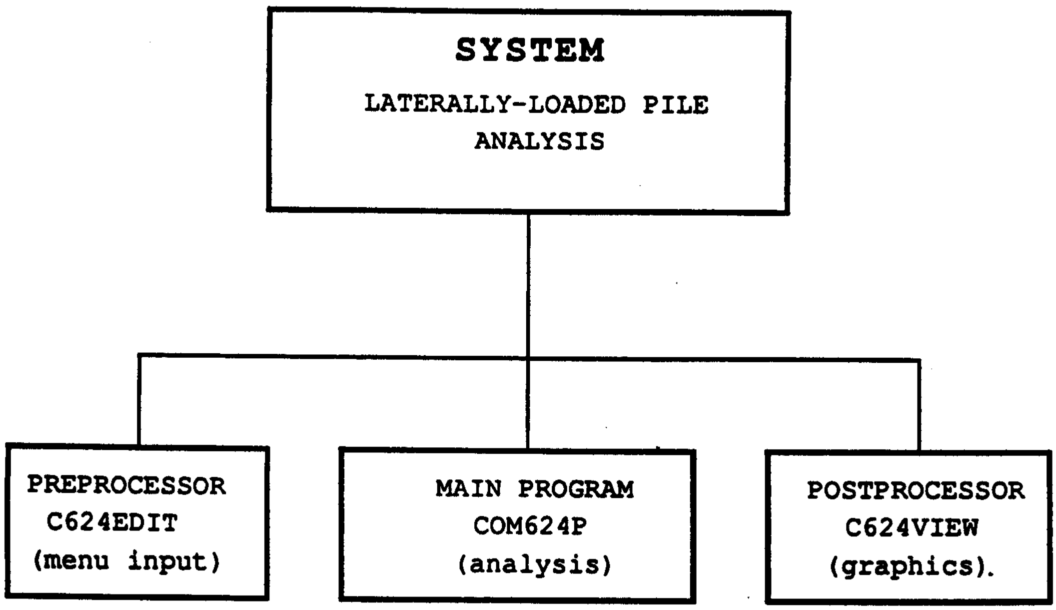


Figure 2.1. The interrelationships of the major components of the system.

compiler in compliance with ANSI FORTRAN 77 is required to compile the program if any modifications are made in the future.

The preprocessor and the postprocessor are written in PASCAL language to utilize fully the screen functions. A TURBO PASCAL compiler from Borland International should be used to recompile these two programs if any upgrade is required in the future.

COMMUNICATIONS

No requirements are necessary.

INTERFACES

No requirements are necessary.

SECURITY

No data base in included and no security is required. Security of the source code that is supplied should be maintained by FHWA.

COM624P
LATERALLY LOADED PILE ANALYSIS PROGRAM FOR THE MICROCOMPUTER
Version 2.0

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LINE-BY-LINE INPUT GUIDE FOR BATCH FILE

The free-field format is used for all the data entry. Each line can have a maximum of 80 characters. The variables can be separated by either a space or a comma. It is recommended that the user follow the line arrangement described later in entering the data.

1. Title Line

Number of lines: 1

Explanation:

Any characters, including blanks, are allowed in this descriptive title. However, do not type the word END in columns 1 through 3 as end is used to indicate the end of the data input.

2. Units and Computation Selection Line

Variables: ISYSTM, NCOM, MEI

Number of lines: 1

Explanation:

```
   ISYSTM = 1 if English units of pounds and inches are to
              be used,
          = 2 if SI units of kilonewtons and meters are
              to be used, and
          = 3 if some other consistent set of units for force
              and length are to be used (the program will not
              try to determine which set of units is used but
              will indicate units on output by F for force and L
              for length, e.g., stress would be F/L**2).
   NCOM   = 1 for analysis of piles under lateral loading,
          = 2 for computation of ultimate bending moment,
          = 3 for both analyses,
 If NCOM  = 1, omit input lines 21 to 26,
 If NCOM  = 2, omit input lines 3 through 20,
   MEI    = 0 for no internal variation of EI employed in
                   computation,
          = 1 for using the internal variation of EI generated
              by the program during computation, and
 If NCOM  = 1 or 2, set MEI = 0.
```

3. **Input Control Line**

Variables: NI, NL, NDIAM, NW

Number of lines: 1

Explanation:

NI = number of increments into which the pile is divided (maximum is 3ØØ),
NL = number of layers of soil (maximum is 9),
NDIAM = number of segments of pile with different diameter, area, or moment of inertia (maximum is 1Ø), and
NW = number of points on plot of distributed lateral load on the pile versus depth (minimum is Ø, maximum is 1Ø).

Set NW = Ø if there are no distributed loads on the pile.

4. **Input Control Line**

Variables: NG1, NSTR, NPY

Number of lines: 1

Explanation:

NG1 = number of points on plot of effective unit weight versus depth (minimum = 2, maximum = 1Ø),
NSTR = number of points on input curves of strength parameters (c, ϕ, ε_{50}) versus depth (minimum = 2, maximum = 1Ø), and
NPY = number of input p-y curves (minimum = Ø, maximum = 3Ø).

Set NGI = Ø and NSTR = Ø if all p-y curves are to be input by the user (if no p-y curves are to be generated internally).

5. **Geometry Line**

Variables: LENGTH, EPILE, XGS, SLOPE

Number of lines: 1

Explanation:

LENGTH = length of pile (L),
EPILE = modulus of elasticity of pile (F/L^2),

XGS = depth below top of pile to ground surface (L), and
SLOPE = slope angle of ground surface to horizontal plane (degrees).

6. **Output Control Line**

Variables: KPYOP, INC

Number of lines: 1

Explanation:

KPYOP = Ø 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 (see 17 and 18 for input of depths at which p-y curves will be generated and printed),
INC = increment used in printing output,
= 1 to print values at every node,
= 2 to print values at every second node, and
= 3 to print values at every third node, etc. (up to NI + 1).

7. **Analysis Control Line**

Variables: KBC, KOUTPT, KCYCL, RCYCL

Number of lines: 1

Explanation:

KBC = code to control boundary condition at top of pile,
= 1 for a free head (user specifies shear P_t and moment M_t at the pile head),
= 2 for specified shear P_t and slope S_t at the pile head (S_t = Ø for a fixed-head pile),
= 3 for a specified shear P_t and rotational restraint M_t/S_t at the pile head,
= 4 for a specified deflection y_t and moment at the pile head,
KOUTPT = Ø 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,
KCYCL = Ø for cyclic loading,
= 1 for static loading, and
RCYCL = number of cycles of loading (need only for p-y curves generated with criteria for stiff clay above the water table).

8. Run Control Line

Variables: MAXIT, YTOL, EXDEFL

Number of lines: 1

Explanation:

MAXIT = maximum number of iterations allowed for analysis of single set of loads,
YTOL = tolerance (L) on solution convergence. When the maximum change in deflection at any node for successive iterations is less than YTOL, iteration stops, and
EXDEFL = value of deflection of pile head (L) that is considered grossly excessive and which stops the run.

9. Distributed Loads

Omit if NW = Ø

Variables: XW(I), WW(I)

Number of lines: NW

Explanation:

XW = depth (L) below top of pile to a point where distributed load is specified, and
WW = distributed lateral load (F/L) on pile.

The program uses linear interpolation between points on the WW-XW curve to determine the distributed load at every node. For best results, points on the WW-XW curve should fall on the pile-node points. Wherever no distributed load is specified, it is assumed to be zero. Data must be arranged with ascending values of XW.

10. Pile Properties Line

Variables: XDIAM(I), DIAM(I), MINERT(I), AREA(I)

Number of lines: NDIAM

Explanation:

XDIAM = x-coordinate (depth below top of pile) of the top of a segment of pile with uniform cross-section (L). The first depth (XDIAM (1)) must equal Ø.Ø,

DIAM = diameter of pile corresponding to XDAIM (L). For non-circular cross sections, use of minimum width will produce conservative results,

MINERT = moment of inertia of pile cross sections (L^4), and

AREA = cross-sectional area of pile (L^2). If left blank, program will compute area assuming a pipe section.

Data must be arranged with ascending values of XDAIM. Note that at a depth between XDIAM(I) and XDIAM(I + 1), the pile properties associated with XDIAM(I) will be used. For a pile with uniform cross section, just one pile property value is needed. The last value of XDIAM need not be greater than or equal to the length of pile.

11. Soil Profile Line

Variables: LAYER, KSOIL, XTOP, XBOT, K

Number of Lines: NL

Explanation:

LAYER(I) = Layer identification number (use 1 for the top layer, 2 for the second layer, etc.),

KSOIL = code to control the type of p-y curves that will be used for L-th layer,
= 1 to have p-y curves computed internally using Matlock's (197Ø) criteria for soft clay,
= 2 to have p-y curves computed internally using Reese et al.'s (1975) criteria for stiff clay below the water table,
= 3 to have p-y curves computed internally using Welch and Reese's (1972) criteria for stiff clay above the water table,
= 4 to have p-y curves computed internally using Reese et al.'s (1974) criteria for sand,
= 5 to use linear interpolation between input p-y curves,
= 6 to have p-y curves computed internally using Reese and Nyman's (1978) limestone criteria,

XTOP(I) = x-coordinate of top of layer (L),

XBOT(I) = x-coordinate of bottom of layer (L), and

K(I) = constant (F/L^3) in equation $E_s = kx$. This is used (1) to define initial soil moduli for the first iteration and (2) to determine initial slope of p-y curve where

KSOIL = 2 and 4.

Arrange data in ascending order of LAYER(I).

12. Unit Weight Line

Variables: XG1(I), GAM1(I)

Number of lines: NG1

Explanation:

XG1 = depth below top of pile to point where effective unit weight of soil is specified (L), and

GAM1 = effective unit weight of soil(F/L^3) corresponding to XG1.

The first depth (XG1(1)) must not be greater that the x-coordinate of the ground surface and the last depth (XG1(NG1)) must not be less than the length of the pile. The program interpolates linearly between points on the XG1 - GAM1 curve to determine effective unit weight of soil at a particular depth. The data must be arranged with ascending values of XG1.

13. Strength Parameter Line

Omit this line if NSTR = Ø.

Variables: XSTR(I), C1(I), PHI1(I), EE5Ø(I)

Number of lines: NSTR

Explanation:

XSTR = x-coordinate (depth below top of pile) for which c, ϕ, and ε_{50} are specified (L),

C1 = undrained shear strength of soil (F/L^2) corresponding to XSTR,

PHI1 = angle of internal friction (ϕ, in degrees) corresponding to XSTR, and

EE5Ø = strain at 5Ø percent stress level (ε_{50}, dimensionless) corresponding to XSTR.

The program uses linear interpolation to find c, ϕ, and ε_{50} at points between input XSTR's. XSTR(1) should not be greater than the x-coordinate of the ground surface and XSTR(NSTR) should not be less than the length of the pile. Arrange data with ascending values of XSTR. For clay layers (KSOIL = 1, 2, 3, or 6), PHI1 will not be used and may be left blank. For sand layers (KSOIL = 4), C1 and EE5Ø are not used and may be left blank.

14. Control Line for Input of p-y Curves

Omit this line if NPY = Ø

Number of lines: 1

Explanation:

NPPY = number of points on input p-y curves (minimum = 2, maximum = 3Ø).

15. Line for Depth of p-y Curve

Omit this line if NPY = Ø

Variables: XPY(I)

Number of lines: 1

Explanation:

XPY = x-coordinate (depth below top of pile) to an input p-y curve (L).

Data must be arranged in ascending order of XPY. Input XPY, then data to define the associated p-y curve (see next line), then the next XPY, etc.

16. Data Line for p-y Curve

Omit if NPY = Ø

Number of lines: NPY * NPPY

Explanation:

YP = deflection (L) of a point on a p-y curve, and
PP = soil resistance (F/L corresponding to YP).

Data must be arranged in ascending order of YP. Sequence of input is as follows:

```
      DO 3Ø I=1, NPY
      READ (5,1Ø), XPY(I)
1Ø    FORMAT (D1Ø.3)
      READ (5,2Ø), (YP(I,J), PP(I,J),J=1,NPPY)
2Ø    FORMAT (2D1Ø.3)
3Ø    CONTINUE
```

The program interpolates linearly between points on a p-y curve and between p-y curves. The program uses the deepest p-y curve available for any nodes that extend below the depth of the deepest p-y curve.

17. Control Line for Output of Internally-Generated p-y Curves

Omit this line if KPYOP = Ø

Variable: NN

Number of lines: 1

Explanation:

NN = number of depths for which internally-generated p-y curves are to be printed (maximum = 3Ø5).

Internally-generated p-y curves may be computed for selected depths and printed for verification purposes. In the analysis of pile response, a separate p-y curve is calculated at every node. Therefore, the number of p-y curves printed will have no effect on the solution.

18. Control Line for Depths at Which Internally-Generated p-y Curves are to be Printed

Omit this line if KPYOP = Ø.

Number of lines: NN

Explanation:

XN = x - coordinate (L) at which internally-generated p-y curves are to be generated and printed.

19. Input of Number of Loading Conditions

Variables: LOAD

Number of lines: 1

Explanation:

Number of different loading conditions involved in one computer run (maximum = 2Ø)

20. <u>Line to Establish Loads on Pile Head</u>

Variables: KOP, PT, BC2, PX

Number of lines: between 1 and 2Ø

Explanation:

KOP = Ø if only the pile head deflection, pile-head 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,

PT = lateral load (F) at top of pile if KBC = 1, 2, or 3, lateral deflection (L) at top of pile if KBC = 4,

BC2 = value of second boundary condition,
= moment (F-L) at top of pile if KBC = 1 or 4,
= slope (dimensionless) at top of pile if KBC = 2,
= rotational stiffness (F-L), or moment divided by slope, if KBC = 3, and

PX = axial load (F) on pile (assumed to be uniform over whole length of pile).

21. <u>Cross-Sectional Shape and Number of Axial Loads</u>

Variables: ID, NP

Number of lines: 1

Explanation:

ID = identification number of the shape of cross section of column/pile:

1. rectangular or square,
2. circular (without shell or core),
3. circular (with shell but without core),
4. circular (with shell and core or without shell and core, and
5. circular steel pipe.

NP = number of axial loads.

22. <u>Axial Loads</u>

Variables: P(I)

Number of lines: NP

Explanation:

P = axial load (kN). The total number of axial loads per run is limited to 1Ø

23. Material Strength Parameters

Variables: FC, BARFY, TUBEFY, ET

Number of lines: 1

Explanation:

FC = cylinder strength of concrete (kPa) (Ø if ID is 5),
BARFY = yield strength of reinforcement (kPa) (Ø if ID is 5),
TUBEFY = yield strength of shell or core (kPa) (Ø if ID is 1 or 2), and
ET = modulus of elasticity of steel (kPa).

24. Cross-Sectional Dimension

Variables: WIDTH, OD, DT, T, TT

Number of lines: 1

Explanation:

WIDTH = width of section if rectangular (m) (Ø if circular),
OD = outer diameter if circular, or depth of section if rectangular (m),
DT = outer diameter of core (m) (Ø if ID is not 4),
T = thickness of shell (m). (Ø if ID is 1 or 2), and
TT = thickness of core (m) (Ø if ID is not 4).

25. Rebar Arrangement
(Skip this line if ID is 5)

Variables: NSIZE, NBARS, NROWS, COVER

Number of lines: 1

Explanation:

NSIZE = size number of rebars proposed to use. Enter number as 3, 4, 5, 6, 7,......14,

NBARS = number of reinforcing bars,
NROWS = number of rows of reinforcing bars, (a number not exceeding 5Ø), and,
COVER = cover of rebar, from center of rebar to outer edge of concrete (m).

26. <u>Area of Reinforcing in a Row</u>
(Skip this line if ID is >1)

Variables: XS(I), AS(I)

Number of lines: NP

Explanation:

XS(I) = distance of row from centroidal axis, starting from top row (m),
AS(I) = area of reinforcing in a row (m^2),
AS(1) = if for the top row (m2), and
AS(2) = if for the second row from the top (m2), etc. The total number of values should not exceed 5Ø.

Generally, rebars will be equally distributed in a circular cross section. If this is the case, the program will compute the required information internally, based on the data provided in the previous line. The user needs to input data only if the shape of the section is square or rectangular.

APPENDIX B

DESIGN PROCEDURE FOR LATERALLY LOADED DEEP FOUNDATIONS USING COM624P VERSION 2

by
Christopher Dumas, P. E.
Hydraulics & Geotech Branch
FHWA Headquarters Bridge Division

DESIGN PROCEDURE FOR LATERALLY LOADED DEEP FOUNDATIONS USING COM624P VERSION 2

The computer program COM624P is an excellent tool for designing laterally loaded deep foundations. However, currently, there is little guidance available in either the Users manual, or other publications, on its proper use as an analysis tool. A general procedure on the program's proper use was developed during the design review and analysis of the case study abutment foundation (Appendix C). This procedure is a synthesis of the FHWA manual "Handbook on Design of Piles and Drilled Shafts Under Lateral Loads," the COM624P Users Manual, and phone conversations with Dr. Shin Tower Wang of Ensoft. The method is flexible and may be modified to fit specific situations. The general procedure is broken into six major tasks:

TASK I. Identify the loading combinations to be analyzed and project serviceability criteria.

TASK II. Determine a preliminary shaft/pile size and foundation configuration.

TASK III. Based on ultimate moment capacity criteria, determine if the shaft/pile is structurally acceptable.

TASK IV. Determine if the shaft/pile is acceptable based on allowable service load deflection criteria.

TASK V. Determine minimum shaft/pile depth required for axial capacity.

TASK VI. Based on the analysis results from TASKS I through V, choose the final foundation configuration, shaft/pile diameter, and shaft/pile length.

TASK I: Identify the loading combinations to be analyzed.

Identify design loading requirements (axial, lateral and bending moments) and performance criteria for routine AASHTO loading combinations (Table 3.22.1A), and special design events such as ship impact and seismic loading. It is essential the foundation designer be positive as to whether the supplied combination loads are load factor design (LFD) values or service load values.

Each set of load combinations should be evaluated separately since combinations, load type (static, cyclic, or dynamic) and performance criteria (safety factor or deflection limit) will modify the loads used in the analysis and interpretations of the results. This is a critical step in overall design process. Inappropriate designs have resulted from designers applying incorrect load magnitudes, "piggybacking load combinations" (applying the largest axial, lateral and bending moments from different combinations simultaneously), and applying inappropriate performance criteria. This information should be supplied by the structural engineer.

i. For each critical loading combination, determine the bending moments, lateral loads, and axial loads for analysis. Be careful to determine if the applied loads are Load Factor Design (LFD) values or are Service Load values.

TASK II: Determine a preliminary shaft/pile size and foundation configuration.

i. Determine a preliminary foundation configuration using AASHTO[1] guidance on spacing and group reduction factors. The AASHTO guidelines are believed to be conservative, but currently there is insufficient evidence or implementation guidance for adopting other methods. Therefore, engineering judgment should be applied cautiously when considering any modification of the AASHTO reduction factors and spacing requirements.

TASK III: Based on ultimate moment capacity criteria, determine if the shaft/pile is structurally acceptable.

i. Determine the ultimate shaft/pile loads:

Factor of Safety x Service Loads (bending moments, axial loads, and lateral loads) **or** the Load Factor Design values.

ii. Determine a preliminary shaft/pile reinforcement configuration (Drilled Shafts Only).

---

[1]AASHTO "Standard Specifications for Highway Bridges," 1992.

iii. Determine the maximum applied bending moments (computed by COM624P subroutine of COM624P version 2 program) and maximum ultimate moment capacity (computed by the PMEIX subroutine of COM624P version 2 program) for both a free head condition (KBC=1) and a fixed head condition (KBC=2). The designer may analyze only one pile head condition if they are confident the pile head is 100% free or 100% fixed. In most cases, it is recommended the designer start with a free head condition and then perform a fixed head analysis. This will provide the designer with a maximum and minimum range of deflections and moments.

- For both the free head and fixed head conditions, execute COM624P version 2 using the ultimate axial loads from step i in both the "Data for Loading" submenu of the "Analysis" menu, and "Axial loads" submenu of the "Mult" menu. Do not input design loads in the "Data for Loading" submenu. The axial loads must match. The shaft/pile should be as long as possible for the given soils information.

iv. Determine if the shaft/pile is acceptable based on ultimate moment capacity criteria. For both the free head and fixed head conditions, perform the following:

- Review the text and graphical output. For the ultimate axial load input in step iii, compare the computed maximum applied bending moment (COM624P subroutine) in the shaft/pile and the maximum ultimate moment capacity (PMEIX ultimate capacity subroutine) of the shaft/pile.

☼. If the computed maximum bending moment in the shaft/pile is larger than the maximum ultimate moment capacity of the shaft/pile, then either: a) modify the reinforcement (shafts only); or b) increase the shaft/pile diameter and return to Task II.

☼ If no run time error messages flash on the screen and the program terminates without generating complete graphical or text output, the computed deflections are excessive and the program was terminated. Increase the shaft/pile diameter and return to TASK II.

TASK IV: Determine if the shaft/pile is acceptable based on allowable service load deflection criteria.

i. Determine the maximum shaft/pile deflection for a free head condition (KBC=1). The designer may skip to step iv for a 100% fixed head condition.

☼ Execute COM624P version 2 using <u>the internal-generated cracked/uncracked EI option</u> in the "Computational control" submenu of the "Analysis" menu. Use the service load axial values in both the "Data for Loading" submenu of the "Analysis" menu, and the "Axial loads" submenu of the "Mult" menu. The axial loads must match. Again, make the shaft/pile as long as possible for the given soils information.

ii. Determine if shaft/pile is acceptable based on free head deflection criteria.

☼ Review the graphics results (Graphics Menu) and text output to determine the maximum deflection. Based on the designer estimate of shaft/pile head fixity, determine if the deflections are acceptable. For example, if a shaft/pile has a computed maximum free head deflection of 1½ inches and a maximum allowable deflection is 1 inch, the following decisions could be made based on the estimated shaft/pile head fixity:

- *Designer estimate of 0% shaft/pile head fixity.*

  1½ inches of deflection. No good. Increase the shaft/pile diameter and return to TASK II.

- *Design assumption of a 50% shaft/pile head fixity.*

  Continue analysis.

iii. Determine preliminary maximum shaft/pile depth.

☼ Review graphical plot of shaft/pile moment versus depth. Locate the depth where the moment plot crosses the zero moment line for the second time (second point of contraflexure). This depth will negate the influence of depth on the shaft/pile deflection and is therefore a good first estimate of minimum depth. The depth can be refined later after the minimum shaft/pile size, foundation configuration, and minimum depth for axial capacity have been determined.

iv. Repeat steps i through iii for a fixed shaft/pile head condition (KBC=2). The designer may skip this step for a 100% free head condition.

v. Determine if shaft/pile is acceptable based on the range of deflections (free head and fixed head).

- Based on the designer estimate of shaft/pile head fixity, determine if the deflections are acceptable. For example, a shaft/pile has a computed free head deflection of 2 inches, a computed fixed deflection of ½ inch, and maximum allowable deflection of 1 inch. The following decisions could be made based on the estimated shaft/pile head fixity:

  - *Designer estimate of 25% shaft/pile head fixity*

    No good. Increase the shaft/pile diameter and return to TASK II.

  - *Designer estimate of 50% shaft/pile head fixity*

    Marginal. Depending on the designers confidence in the soils data, Continue analyses or increase the shaft/pile diameter and return to TASK II.

  - *Designer estimate of 75% (or higher) shaft/pile head fixity.*

    Continue analysis.

TASK V: Determine minimum shaft/pile depth required for axial capacity.

- If large lateral deflections are expected, use the COM624P deflection plot to determine the portion of the top of the shaft/pile where skin resistance should be ignored.

TASK VI: Based on the analysis results from TASK I through V above, choose the final foundation configuration and shaft/pile diameter.

i. Determine final shaft/pile depth.

- If the minimum depth required for axial capacity is less than the depth determined in TASK IV, the final shaft/pile depth could be refined.

  Repeat TASK IV with shorter shaft/pile lengths. The COM624P manual recommends the shaft/pile extend at a minimum to the depth where the deflection plot crosses the zero line a second time. If this results in unacceptable deflection, try again with the shaft/pile five feet deeper. Repeat until an optimum depth is found.

ii. Determine the final foundation configuration.

APPENDIX C

DESIGN OF DRILLED SHAFTS FOR LATERAL LOADS USING COM624P VERSION 2

ABUTMENT FOUNDATION DESIGN A CASE STUDY EXAMPLE

Presented to the Eighteenth Northwest Geotechnical Workshop
Rapid City South Dakota on September 18, 1992
by
Christopher Dumas, P. E.
Hydraulics & Geotech Branch
FHWA Headquarters Bridge Division

DESIGN OF DRILLED SHAFTS FOR LATERAL LOADS USING COM624P VERSION 2

ABUTMENT FOUNDATION DESIGN

A CASE STUDY EXAMPLE

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USING COM624P VERSION 2 AS A DESIGN TOOL
ABUTMENT FOUNDATION DESIGN
A CASE STUDY EXAMPLE

The computer program COM624P is an excellent tool for designing laterally loaded deep foundations. However, currently, there is little guidance available in either the Users manual, or other publications, on its proper use as an analysis tool. A general procedure on the program's proper use was developed during the design review and analysis of the case study abutment foundation. This procedure is a synthesis of the FHWA manual "Handbook on Design of Piles and Drilled Shafts Under Lateral Loads," the COM624P Users Manual, and phone conversations with Dr. Shin Tower Wang of Ensoft. The method is flexible and may be modified to fit specific situations.
The general procedure is broken into six major tasks:

TASK I. Identify the loading combinations to be analyzed and project serviceability criteria.

TASK II. Determine a preliminary shaft/pile size and foundation configuration.

TASK III. Based on ultimate moment capacity criteria, determine if the shaft/pile is structurally acceptable.

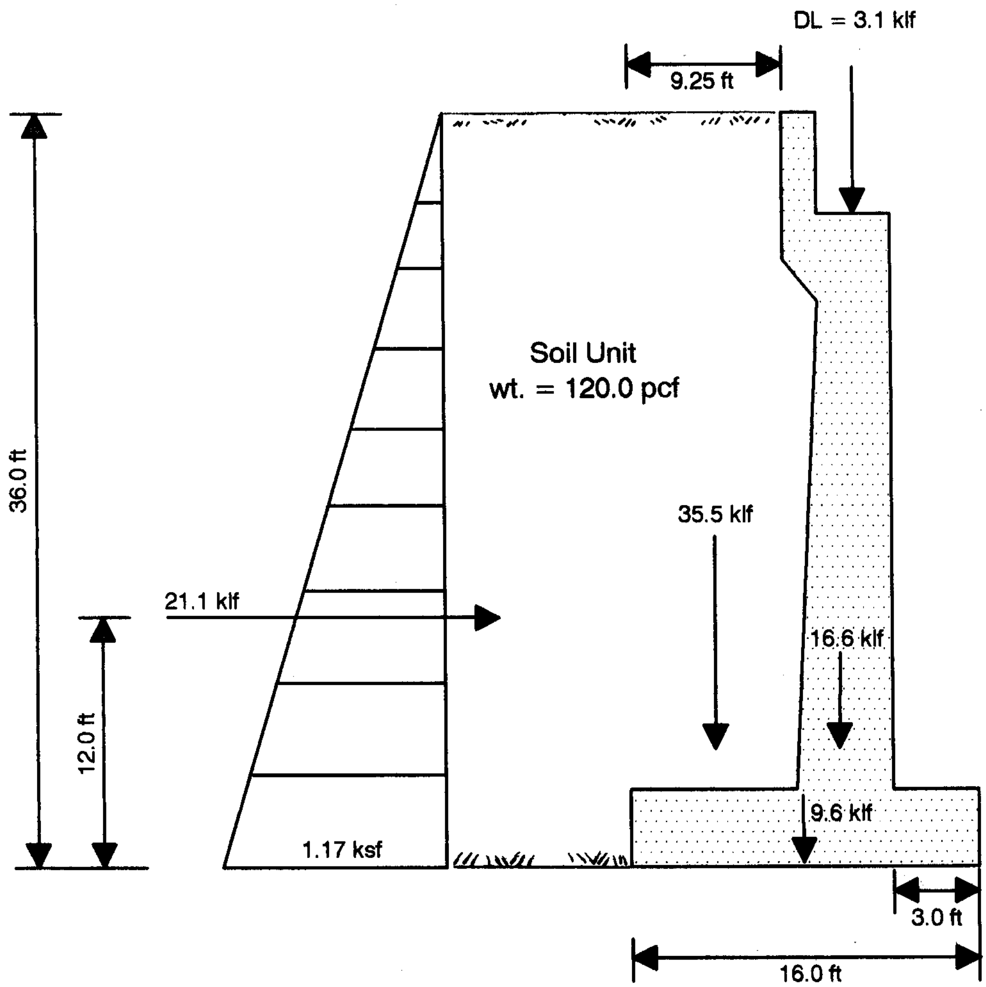
TASK IV. Determine if the shaft/pile is acceptable based on allowable service load deflection criteria.

TASK V. Determine minimum shaft/pile depth required for axial capacity.

TASK VI. Based on the analysis results from TASK I through V, choose the final foundation configuration, shaft/pile diameter, and shaft/pile length.

TASK I: Identify the loading combinations to be analyzed.

The submitted abutment design section and loads are shown below in Figure 1.



$$\Sigma P_{D.L.} = 64.8 \text{ klf} \qquad \Sigma L_{D.L.} = 21.1 \text{ klf}$$

Figure 1 - Submitted Design Section and Applied Loads.

TASK II: Determine a preliminary shaft/pile size and foundation configuration.

The submitted foundation configuration is shown below in Figure 2.

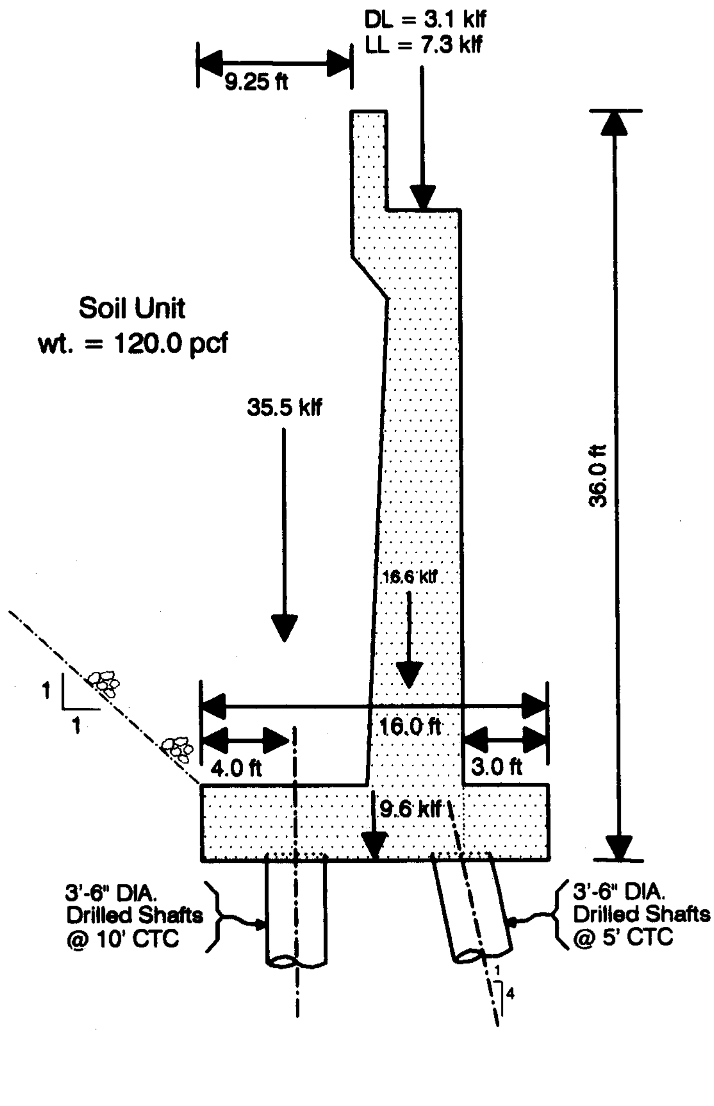


Figure 2 - Submitted Foundation Configuration.

i. Determine a preliminary foundation configuration using AASHTO guidance on shaft spacing and group reduction factors. The AASHTO guidelines are generally considered conservative, but currently there is insufficient verification of how to apply other methods. Therefore, engineering judgment should be applied cautiously when modifying the AASHTO pile reduction factors and spacing requirements.

A.A.S.H.T.O "STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES," 1992

<u>4.6.1.6 BATTER SHAFTS</u>

"The use of battered shafts to increase the lateral capacity of foundations is not recommended due to their difficulty of construction and high cost. Instead, consideration should first be given to increasing the shaft diameter to obtain the required lateral capacity."

<u>4.6.5.6.1.4 GROUP ACTION</u>

Minimum center-to-center (CTC) shaft spacing of 2.5 diameters in the direction normal to the lateral load.

<u>AASHTO GENERAL GUIDE FOR THE EFFECTS OF GROUP ACTION FOR IN-LINE LOADING CTC < 8B (NON-DISPLACEMENT PILING)</u>

CTC Shaft Spacing for In-line Loading	Ratio of Lateral Resistance of Shaft in Group to Single Shaft
8B 8 x 3.5' = 28'	1.00
6B 6 x 3.5' = 21'	0.70
4B 4 x 3.5' = 14'	0.40
3B 3 x 3.5' = 10.5'	0.25

Based on this AASHTO criteria, the submitted in-line of loading shaft spacing (Figure 2) of 2.0B (7'CTC, 3.5' clear) reduces the lateral resistance of the back row to nearly zero. In addition, the front row spacing of 1.4B (5' CTC) does not comply with the AASHTO minimum guide of 2.5B. Therefore, the submitted spacing configuration is not acceptable.

TRY A SINGLE ROW OF 3.5'Ø SHAFTS WITH A CTC SPACING OF 2.5B (9') WITH THE SHAFT ROW ℄ AT THE VERTICAL FORCE RESULTANT (NO MOMENT AT THE PILE HEAD).

TASK III: Based on ultimate moment capacity criteria, determine if the shaft/pile is structurally acceptable.

i. Determine the ultimate shaft/pile loads:

Factor of Safety x Service Loads (bending moments, axial loads, and lateral loads) <u>or</u> the Load Factor Design Values.

Single row of 3.5'Ø shafts with a CTC spacing of 2.5B (9') and F.S.= 2.0

P = (64.8 klf * 9') x (2.0) = 1,160 kips

L = (21.1 klf * 9') x (2.0) = 380 kips

---

ii. Determine a preliminary shaft/pile reinforcement configuration (Drilled Shafts Only).

f'c = 4,000 psi 14-#9 bars

---

iii. Determine the maximum applied bending moments (computed by the main subroutine of COM624P version 2 program) and maximum ultimate moment <u>capacity</u> (computed by the PMEIX subroutine of COM624P version 2 program) for both a <u>free head condition (KBC=1) and a fixed head condition (KBC=2)</u>. The designer may analyze only one pile head condition if they are confident the pile head is 100% free or 100% fixed. In most cases, it is recommended the designer start with a free head condition and then perform a fixed head analysis. This will provide the designer with a maximum and minimum range of deflections and moments.

- For both the free head and fixed head conditions, execute COM624P version 2 using the ultimate loads from step 3 in both the "Data for Loading" submenu of the "Analysis" menu, and "Axial loads" submenu of the "Mult" menu. Do not input design loads (service loads) in the "Data for Loading" submenu. The axial loads must be the same. To negate the effects of shaft length on deflection and moment magnitudes, the shaft/pile should be as long as possible for the given soils information.

The case study soil stratigraphy is shown in Figure 3.

---

iv. Determine if the shaft/pile is acceptable based on ultimate moment capacity criteria. For both the free head and fixed head conditions, perform the following:

- Review the text and graphical output. For the ultimate axial load input in step iii, compare the computed maximum applied bending moment (COM624P subroutine) in the shaft/pile and the maximum ultimate moment capacity (PMEIX ultimate capacity subroutine) of the shaft/pile.

- If the computed maximum bending moment in the shaft/pile is larger than the maximum ultimate moment capacity of the shaft/pile, then either: a) modify the reinforcement (shafts only); or b) increase the shaft/pile diameter and return to Task II.

- If no run time error messages flash on the screen and the program terminates without generating complete graphical or text output, the computed deflections are excessive and the program was terminated. Increase the shaft/pile diameter and return to TASK II.

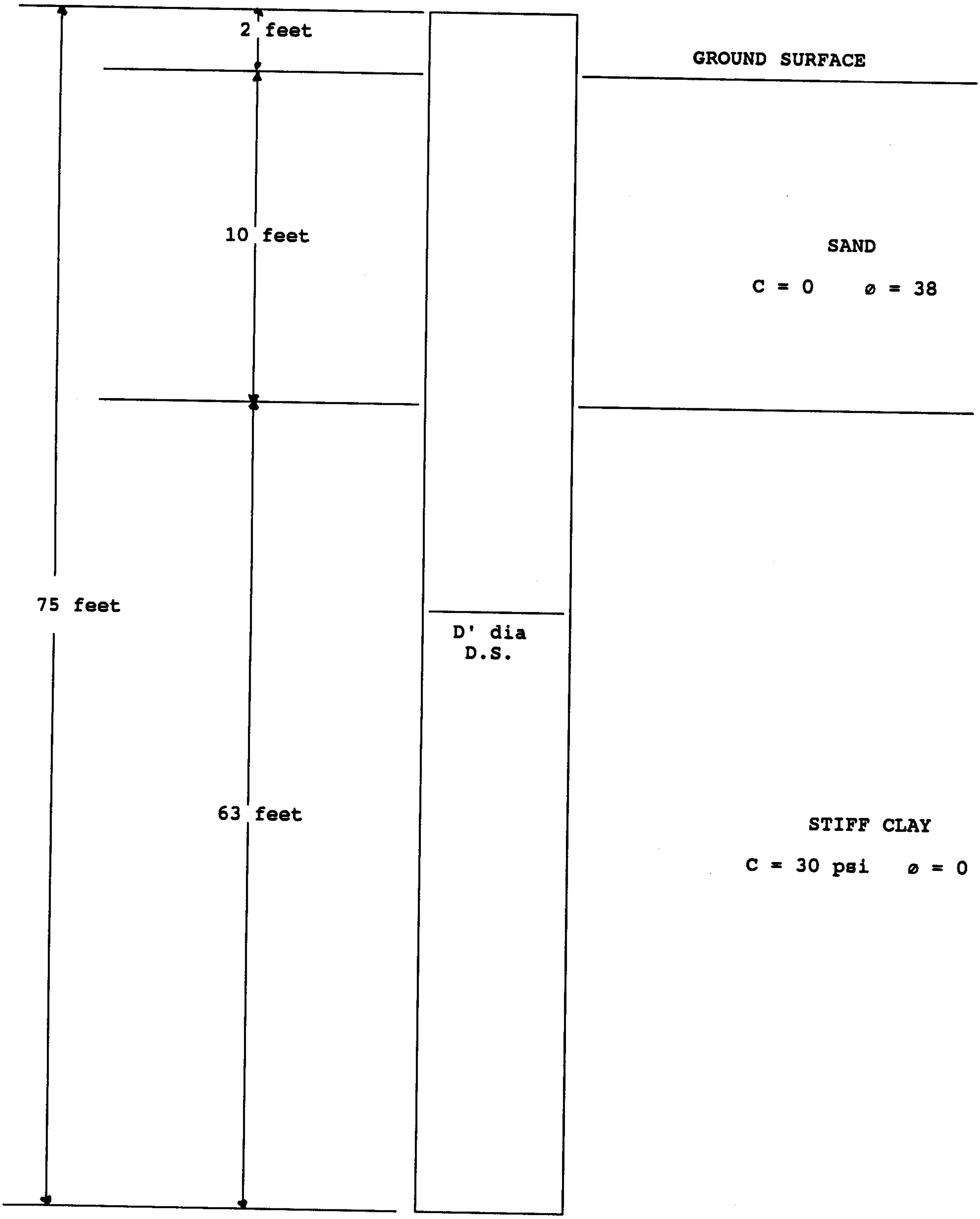


Figure 3 - Case Study Soil Stratigraphy

CHECK FREE HEAD CONDITION

COM624P VERSION 2 - RUN #1

BRIDGE ABUTMENT - 42 INCH DIA SHAFTS @ 9' CTC
FREE HEAD CONDITION AND FULL GROSS SECTION
ULTIMATE LOADING (F.S.= 2)

Maximum Bending Moment computed by the main program of COM624P version 2 program = 44,700 k-in.

Plots of computed deflections and moments versus depth are shown in Figures 4 & 5 respectively.

Maximum Ultimate Bending Moment Capacity computed by the PMEIX subroutine of COM624P version 2 = 25,100 k-in

PMEIX computed Interaction Diagram is shown in Figure 6.

44,700 k-in ≫ 25,100 k-in   NO GOOD, RETURN TO TASK II

NOTE: "Full Gross Section" refers to the Shaft EI value used in this COM624P run - E.I. for a full uncracked section. Since this will not allow the shaft to crack, the results will be conservative (larger moments).

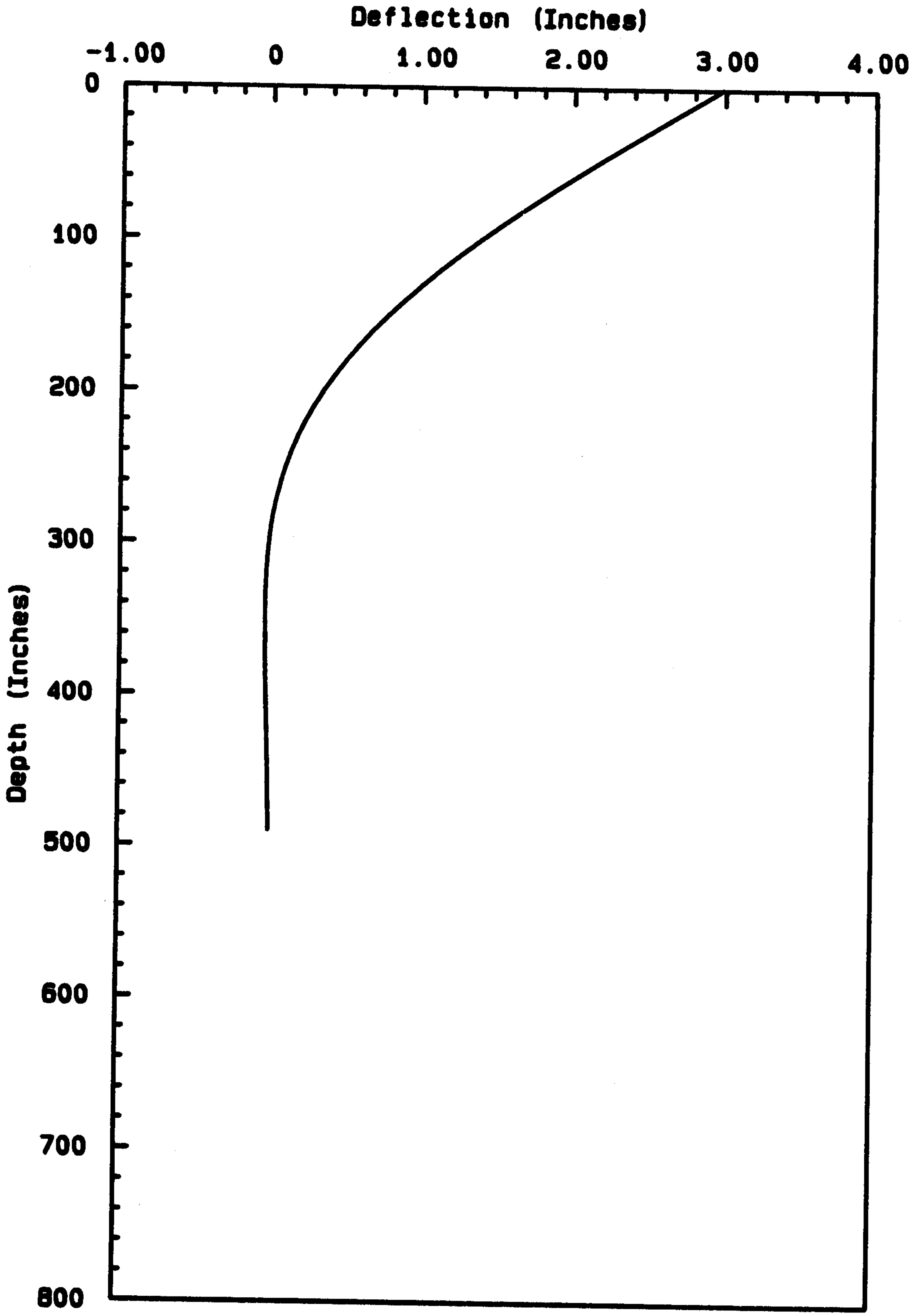


Figure 4 - COM624P computed deflection versus depth for a 42" diameter shaft with ultimate loads and free head condition.

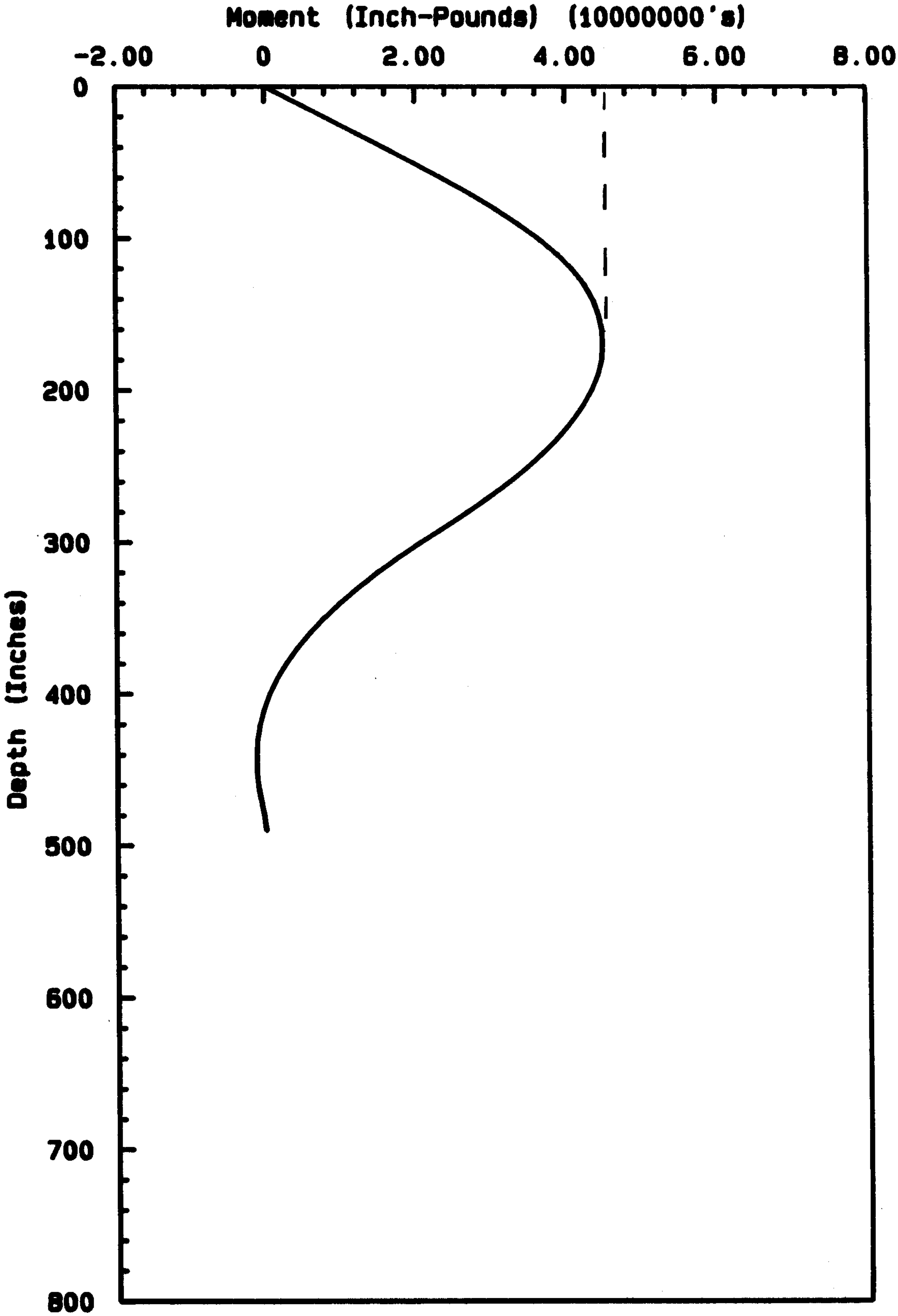


Figure 5 - COM624P computed moment versus depth for a 42" diameter shaft with ultimate loads and free head condition.

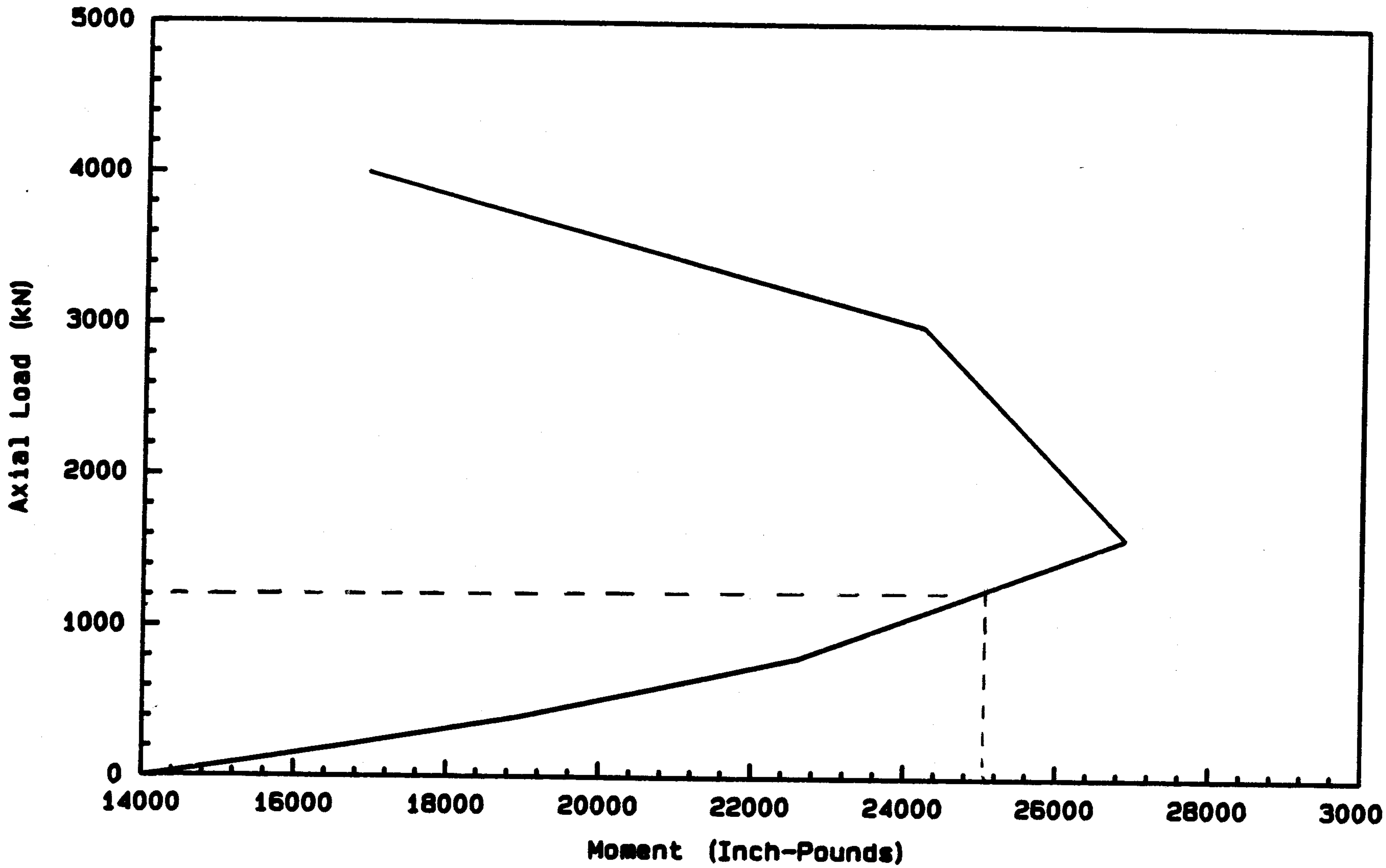


Figure 6 - PMEIX computed Interaction Diagram for a 42" diameter shaft with an ultimate axial load of 1,160 kips

TRY SINGLE ROW OF 5' Ø SHAFTS
CENTERLINE ON THE VERTICAL FORCE RESULTANT (NO MOMENT AT THE PILE HEAD)
AND SPACED AT 12' CTC (2.5B).

Check the ultimate capacity (F.S.= 2)

P = (64.8 klf * 12') x (2.0) = 1,560 kips

L = (21.1 klf * 12') x (2.0) = 506 kips

f'c = 4,000 psi   28-#10 bars

CHECK FREE HEAD CONDITION

COM624P VERSION 2 - RUN #2

BRIDGE ABUTMENT - 60" DIA SHAFTS @ 12' CTC
FREE HEAD CONDITION AND FULL GROSS SECTION
ULTIMATE LOADING (F.S.= 2)

Maximum Bending Moment computed by the main program of COM624P version 2 program = 65,300 k-in.

Plots of computed deflections and moments versus depth are shown in Figures 7 & 8 respectively.

Maximum Ultimate Bending Moment Capacity computed by the PMEIX subroutine of COM624P version 2 = 74,000 k-in

PMEIX computed Interaction Diagram is shown in Figure 9.

65,300 k-in < 74,000 k-in   OK✓

NOTE: "Full Gross Section" refers to the Shaft EI value used in this COM624P run - E.I. for a full uncracked section.

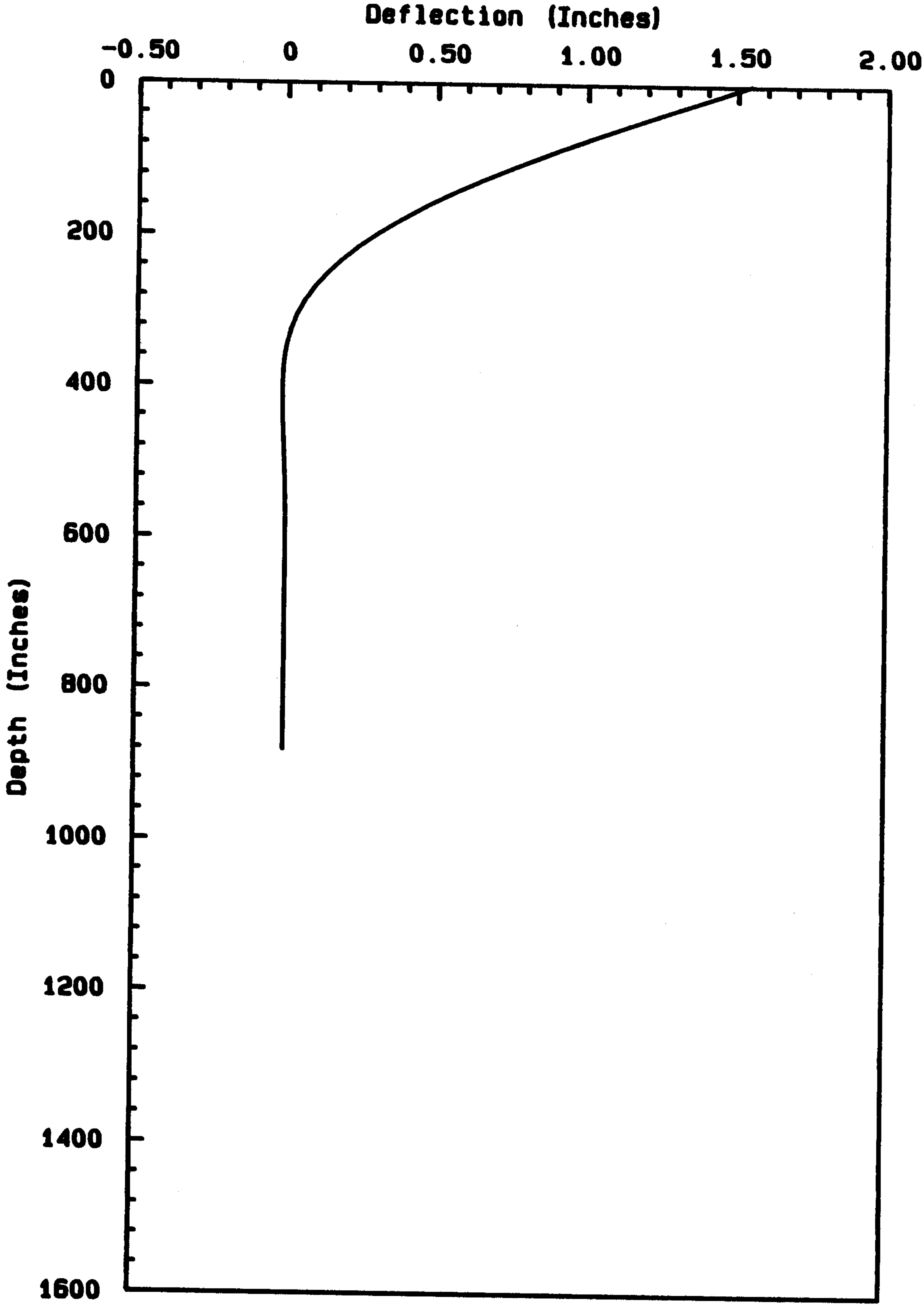


Figure 7 - COM624P computed deflection versus depth for a 60" diameter shaft with ultimate loads and free head condition.

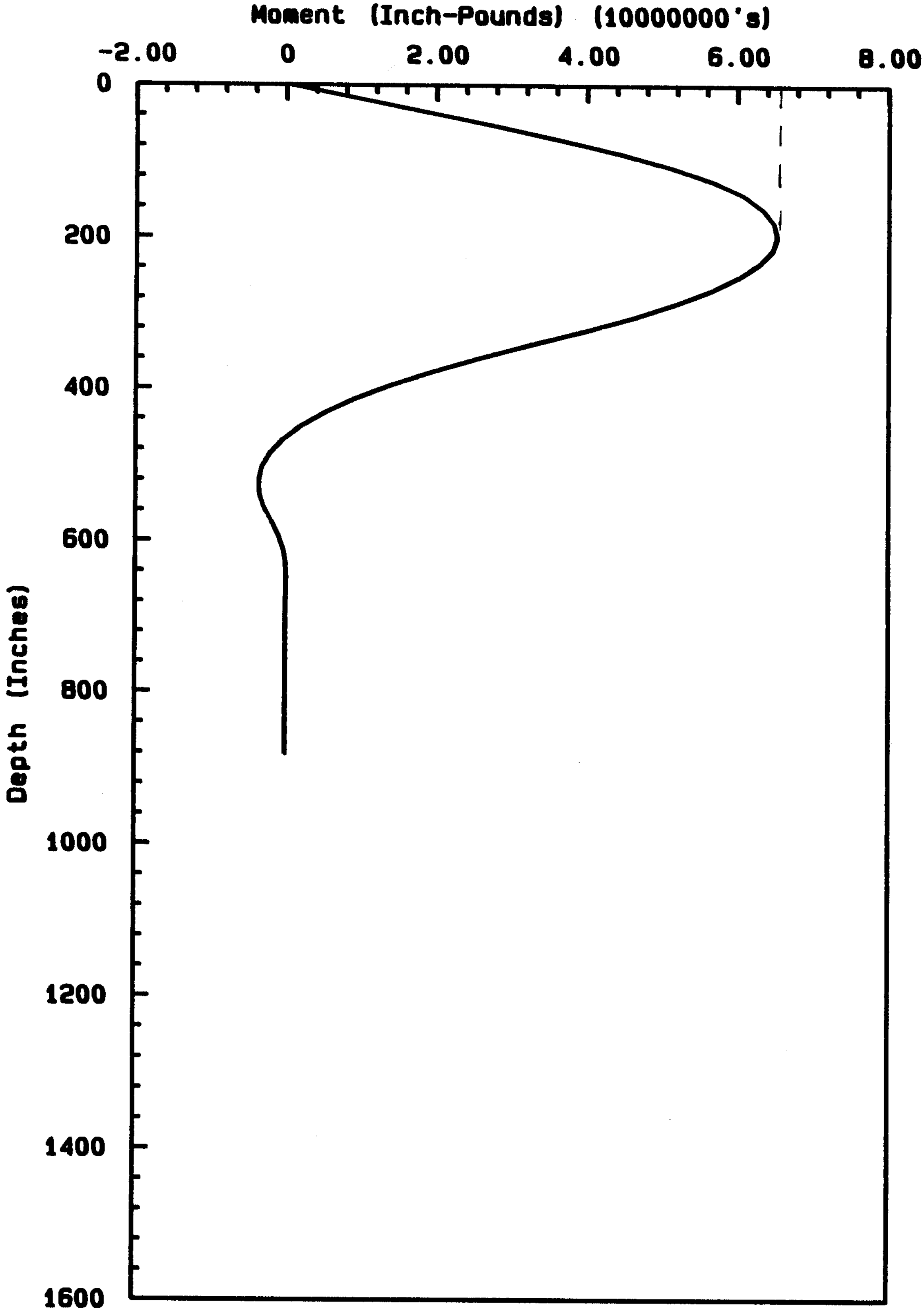


Figure 8 - COM624P computed moment versus depth for a 60" diameter shaft with ultimate loads and free head condition.

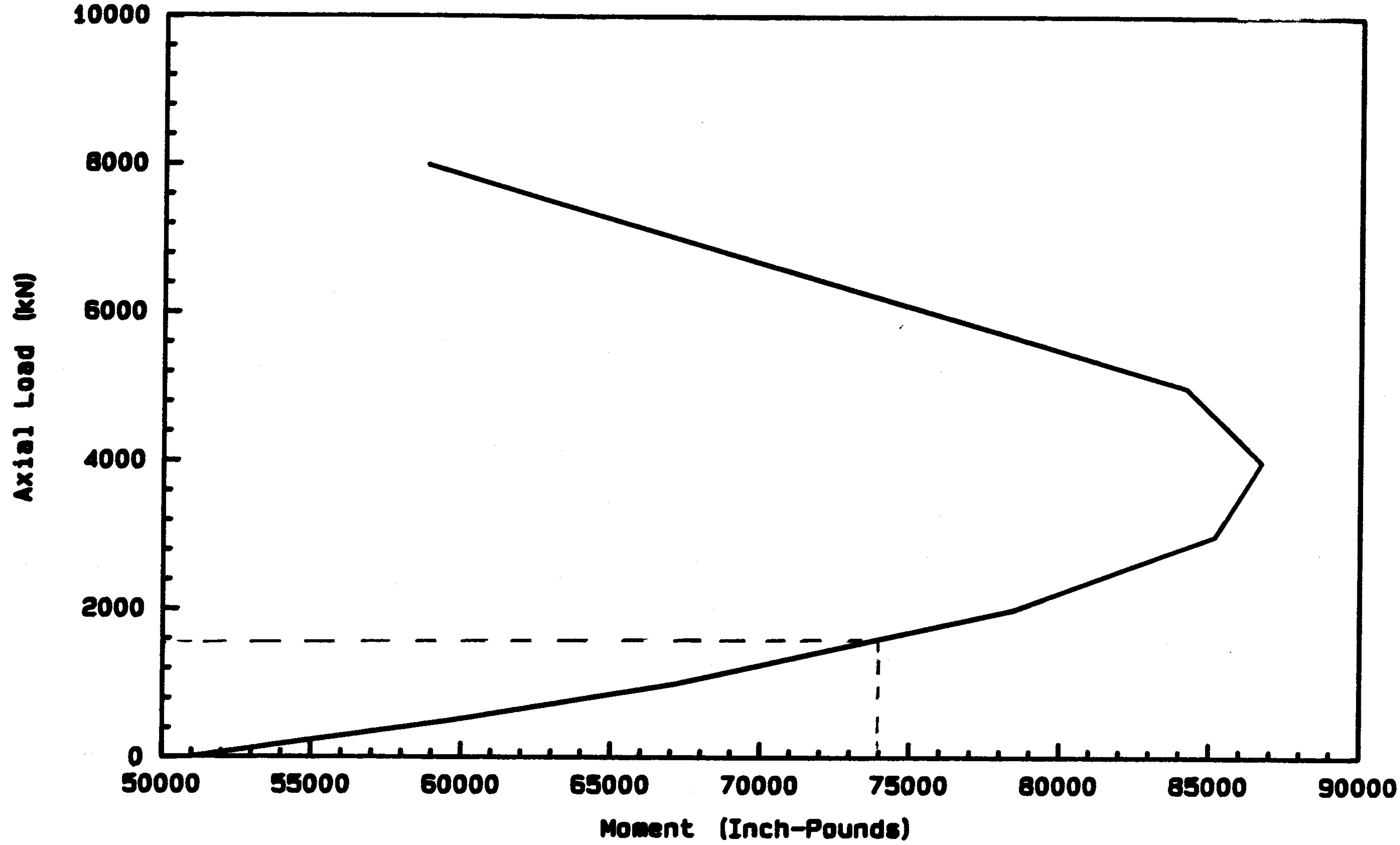


Figure 9 - PMEIX computed Interaction Diagram for a 60" diameter shaft with an ultimate axial load of 1,560 kips

CHECK FIXED HEAD CONDITION

COM624P VERSION 2 - RUN #3

BRIDGE ABUTMENT - 60" DIA SHAFTS @ 12' CTC

FIXED HEAD CONDITION AND FULL GROSS SECTION

ULTIMATE LOADING (F.S.= 2)

Maximum Bending Moment computed by the main program of COM624P version 2 program = 59,000 k-in.

59,000 k-in < 74,000 k-in OK ✓

TASK IV: Determine if the shaft/pile is acceptable based on allowable service load deflection criteria.

i. Determine the maximum shaft/pile deflection for a free head condition (KBC=1). The designer may skip to step iv for a 100% fixed head condition.

- Execute COM624P version 2 using the internally generated cracked/uncracked EI option in the "Computational control" submenu of the "Analysis" menu. Use the service load values in both the "Data for Loading" submenu of the "Analysis" menu, and the "Axial loads" submenu of the "Mult" menu. The axial loads must be the same. Again, make the shaft/pile as long as possible for the given soils information.

NOTE: internally generated cracked/uncracked EI option refers to the automatic variation of EI with stress along the shaft length. When stresses are high, the shaft cracks, deflections increase, and stresses migrate downward. Therefore, this option provides superior pile-soil interaction modeling and computed deflection magnitudes.

Service Loads

P = (64.8 klf * 12') = 778 kips

L = (21.13 klf * 12') = 253 kips

f'c = 4,000 psi   28-#10 bars

CHECK FREE HEAD CONDITION

COM624P VERSION 2 - RUN #4

BRIDGE ABUTMENT - 60 INCH DIA SHAFTS @ 12' CTC

FREE HEAD CONDITION AND CRACKED SECTION

SERVICE LOADS

Maximum pile head deflection calculated by the main program of COM624P version 2 = 0.667 inch

Plots of computed deflections and moments versus depth are shown in Figures 10 & 11 respectively.

---

ii. Determine if shaft/pile is acceptable based on free head deflection criteria.

- Review the graphics results (Graphics Menu) and text output to determine the maximum deflection. Based on the designer estimate of shaft/pile head fixity, determine if the deflections are acceptable.

MAXIMUM PILE HEAD DEFLECTION W/FREE HEAD = 0.667 INCH

0.677 INCH < 1.0 INCH MAXIMUM ALLOWABLE - OK ✓

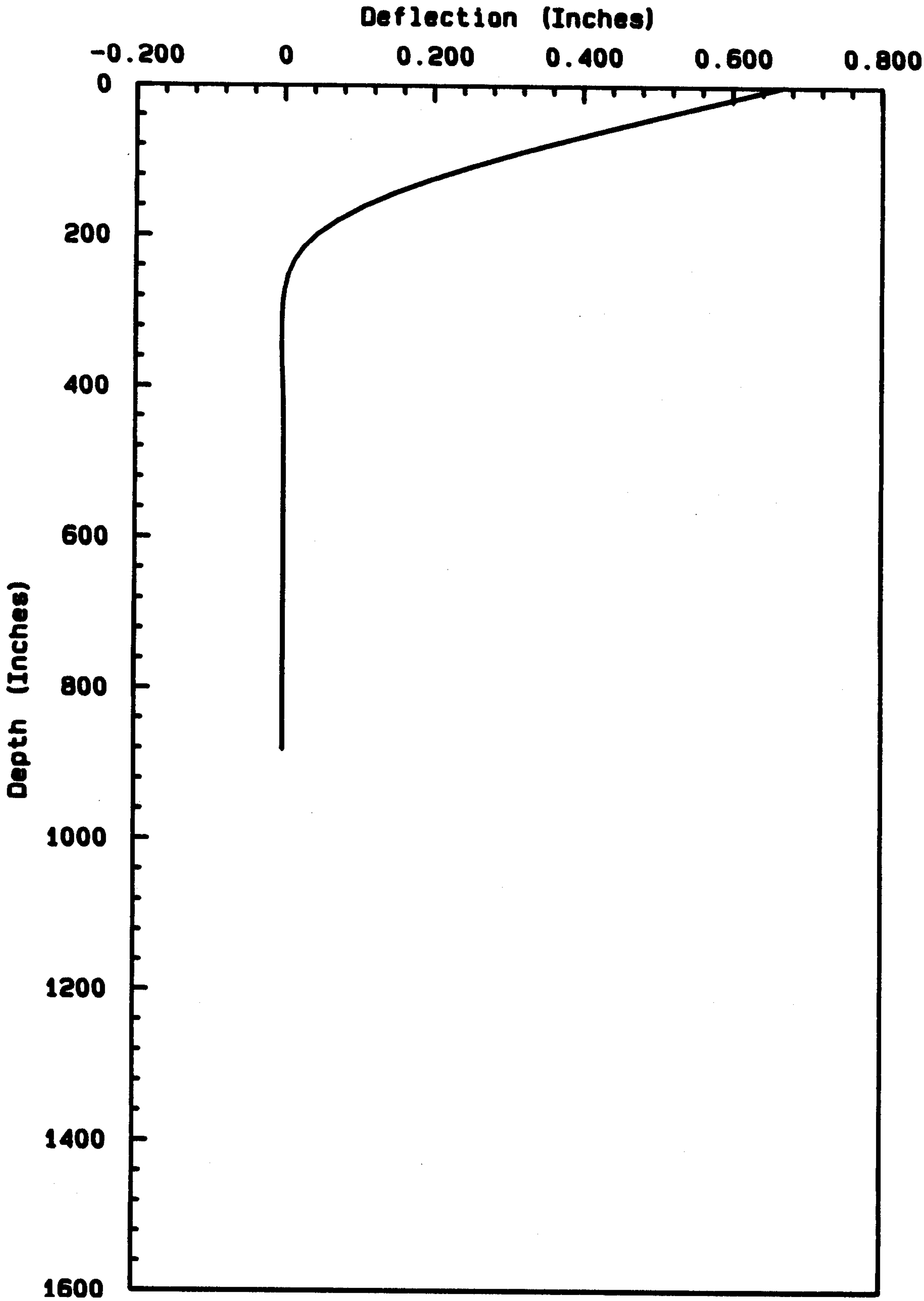


Figure 10 - COM624P computed deflection versus depth for a 60" diameter shaft with service loads and free head condition.

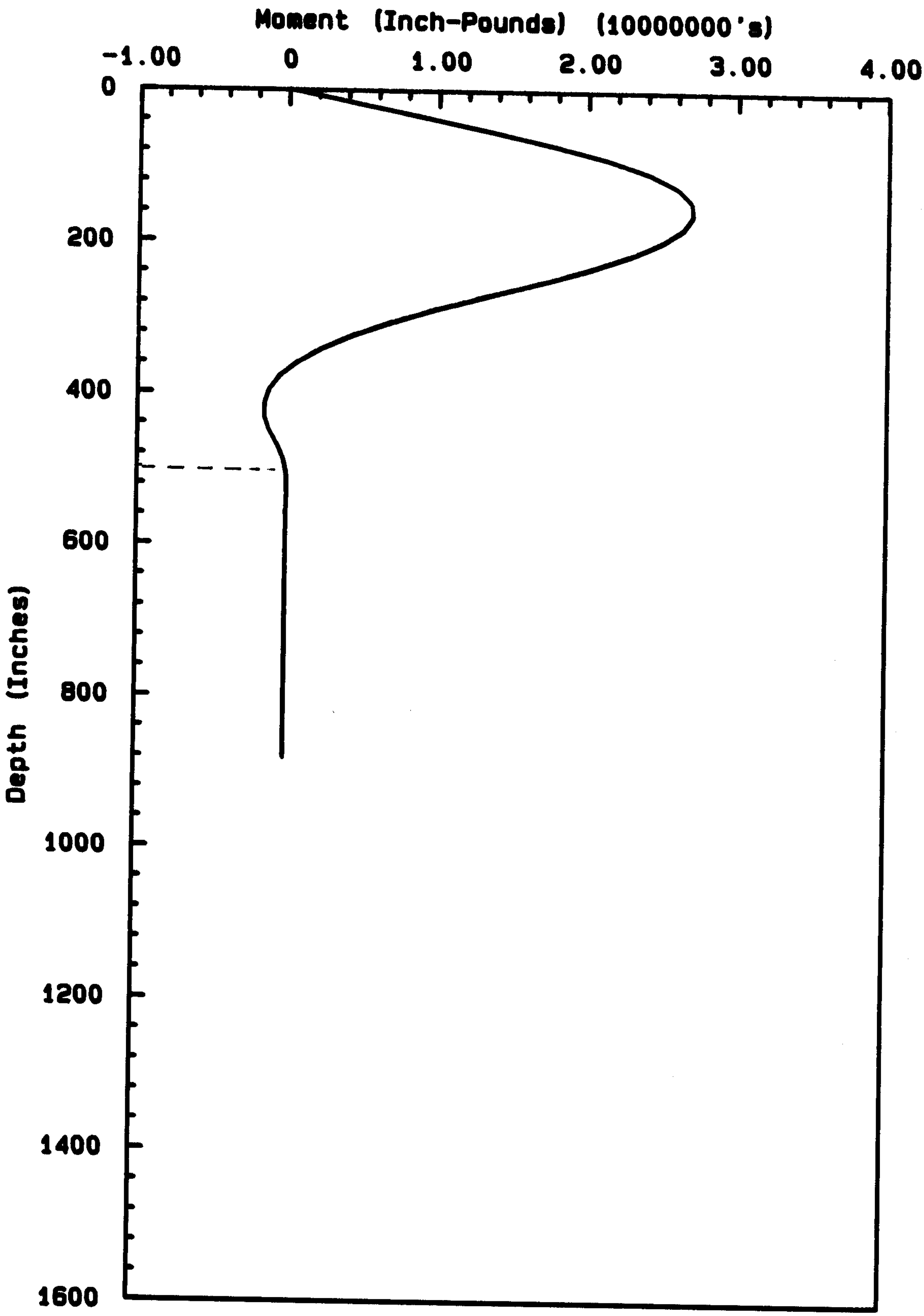


Figure 11 - COM624P computed moment versus depth for a 60" diameter shaft with service loads and free head condition.

iii. Determine preliminary maximum shaft/pile depth.

- Review graphical plot of shaft/pile moment versus depth. Locate the depth where the moment plot crosses the zero moment line for the second time (second point of contraflexure). This depth will negate the influence of depth on the shaft/pile deflection and is therefore a good first estimate of minimum depth. The depth can be refined later after the minimum shaft/pile size, foundation configuration, and minimum depth for axial capacity have been determined.

Review of Figure 11 moment versus depth plot shows the moment crossing the zero line a second time at 500 inches.

PRELIMINARY SHAFT DEPTH = 42′

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iv. Repeat steps i through iii for a fixed shaft/pile head condition (KBC=2). The designer may skip this step for a 100% free head condition.

CHECK FIXED HEAD CONDITION

COM624P VERSION 2 - RUN #5

BRIDGE ABUTMENT - 60" DIA SHAFTS @ 12' CTC

FIXED HEAD CONDITION AND CRACKED SECTION

SERVICE LOADS

Maximum pile head deflection calculated by the main program of COM624P version 2 = 0.172"

A plot of computed deflections versus depth is shown in Figure 12.

MAXIMUM PILE HEAD DEFLECTION = 0.172"

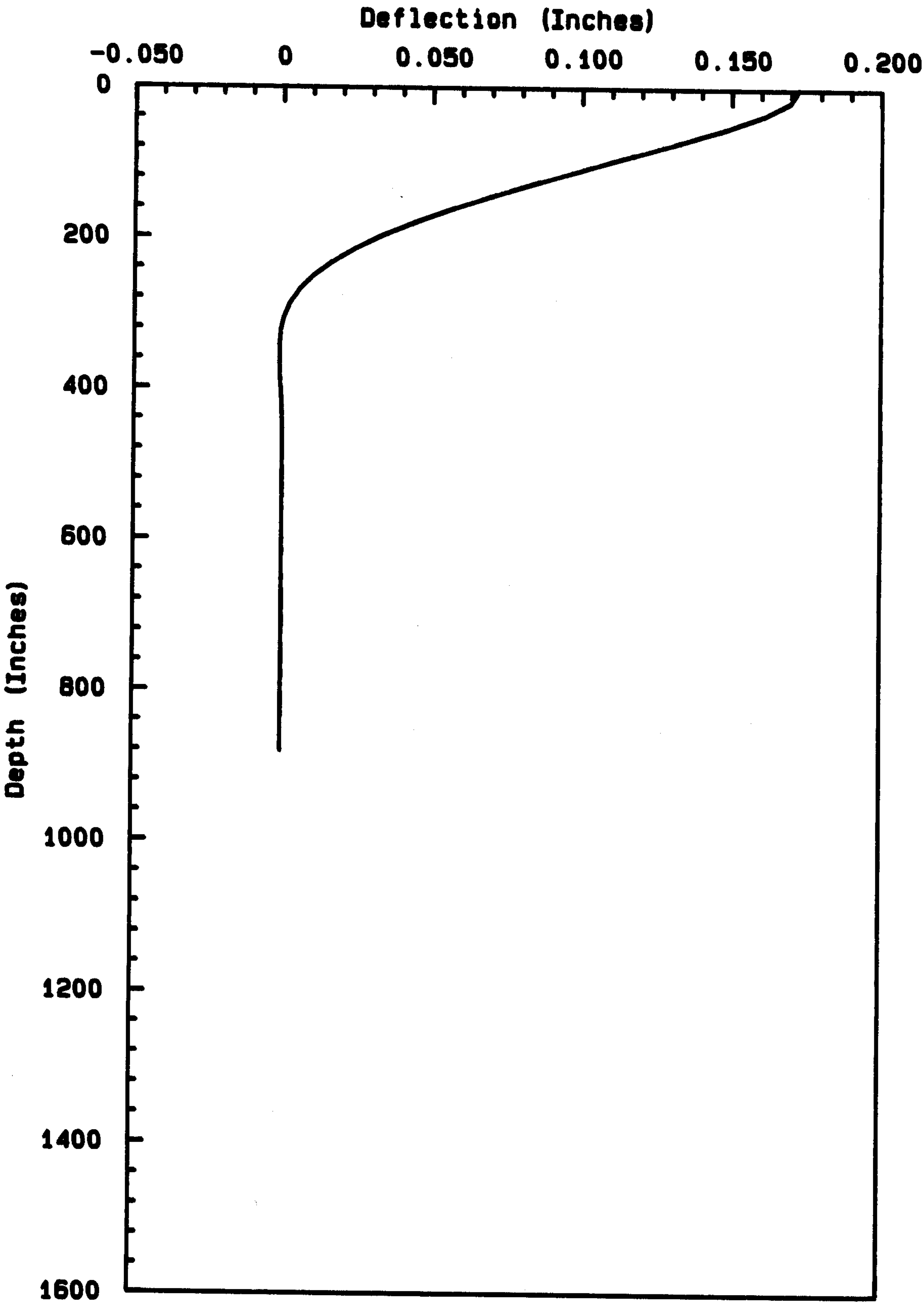


Figure 12 - COM624P computed deflection versus depth for a 60" diameter shaft with service loads and fixed head condition.

v. Determine if shaft/pile is acceptable based on the range of deflections (free head and fixed head).

SUMMARY OF COM624P LATERAL LOAD ANALYSIS RESULTS
BRIDGE ABUTMENT - 60" DIAMETER SHAFT @ 12' CTC:

Ultimate Load (F.S.= 2):	P = 1,560 kips	L = 506 kips
Service Load:	P = 778 kips	L = 253 kips
Shaft Materials:	f'c = 4,000 psi	28-#10 bars

Maximum Service Load Deflection - 0.667" (Free Head)

Minimum Service Load Deflection - 0.172" (Fixed Head)

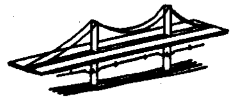
Minimum Shaft Length - 42.0 feet

Estimated % shaft head fixity = 0-10%

0.172" to 0.667" < 1.0" MAXIMUM ALLOWABLE - <u>OK ✓</u>

TASK V: Determine minimum shaft/pile depth required for axial capacity.

* If large lateral deflections are expected, use the COM624P deflection plot to determine the portion of the top of the shaft/pile where skin resistance should be ignored.



<u>CHECK AXIAL LOAD CAPACITY FOR A 5'Ø SHAFT:</u>

$P_{D.L.}$ = (64.8 klf * 12') = 778 kips

$P_{L.L}$ = (7.1 KLF * 12') = 85 kips

TOTAL = 433 TONS

ANALYSIS PERFORMED WITH ENSOFT COMPUTER PROGRAM SHAFT1

AXIAL CAPACITY OF DRILLED SHAFTS

<u>SUMMARY OF RESULTS FOR THE DESIGN LOAD OF 433 TONS (F.S.= 2)</u>

SHAFT DIAMETER	MINIMUM EMBEDMENT FOR 433 TONS
3.5'	75'
4.0'	63'
4.5'	54'
5.0'	*46'*
5.5'	39'
6.0'	33'
6.5'	34'
7.0'	30'

MINIMUM EMBEDMENT FOR A 5'Ø TO MEET LATERAL LOAD REQUIREMENTS IS 42'.

MINIMUM EMBEDMENT FOR A 5'Ø TO MEET AXIAL LOAD REQUIREMENTS IS 46'.

5' Ø SHAFT IS AN EFFICIENT SIZE✓

TASK VI: Based on the analysis results from TASKS I through V, choose the final foundation configuration and shaft/pile diameter.

i. Determine final shaft/pile depth.

* If the minimum depth required for axial capacity is less than the depth determined in TASK IV, the final shaft/pile depth could be refined.

MINIMUM REQUIRED AXIAL DEPTH OF 46' > MAXIMUM REQUIRED LATERAL DEPTH OF 42'

OK ✓ No refinement necessary

USE
A SINGLE ROW OF 5′ DIAMETER SHAFTS @ 12′ CTC CENTERLINE OF SHAFT ROW AT THE VERTICAL FORCE RESULTANT (NO MOMENT) MINIMUM OF 28 - #10 BARS FOR SHAFT REINFORCEMENT MINIMUM OF DEPTH OF EMBEDMENT - 46′

*U.S. G.P.O.:1993-343-273:80184