To the Graduate Council:

I am submitting herewith a thesis written by Christopher Allen Myers entitled "Behavior of Laterally Loaded Piles Supporting Integral Abutments." I have examined the final copy of this thesis for form and content and recommend that it be accepted in partial fulfillment of the requirements for the degree of Master of Science, with a major in Civil Engineering.

Edwin G. Burdette

Dr. Edwin G. Burdette, Major Professor

We have read this thesis and
recommend its acceptance:

Accepted for the Council:

Associate Vice Chancellor and
Dean of the Graduate School

BEHAVIOR OF LATERALLY LOADED PILES SUPPORTING INTEGRAL ABUTMENTS

A Thesis

Presented for the

Master of Science

Degree

The University of Tennessee, Knoxville

Christopher Allen Myers

December 1998

ACKNOWLEDGMENTS

I would like to thank a number of people who have assisted and supported me during the course of my graduate studies. I am extremely grateful to the members of my thesis committee: Dr. Edwin G. Burdette, Dr. J. Harold Deatherage, and Dr. David W. Goodpasture. The support of this group of individuals made my graduate studies possible. I would particularly like to thank my major professor Dr. Burdette for his assistance in completing this thesis, as well as his guidance and encouragement throughout my graduate and undergraduate studies.

There are a number of graduate and undergraduate students who have contributed to this research project and made this thesis possible. I would like to thank Samer Aljamal, Laura Arico, Eric Bonanno, Oliver Chiang, Ken Harvey, Raymond Kutai, Jay Lewis, and Tony Zeind.

I would also like to express my appreciation to the Tennessee Department of Transportation. This research was made possible with funding provided by TDOT.

ABSTRACT

The Tennessee Department of Transportation (TDOT) routinely designs and constructs jointless bridges with integral abutments. Of particular concern with jointless bridges is the effect of thermally induced deflections on piles supporting integral abutments. TDOT is currently sponsoring a research project with The University of Tennessee, Knoxville to investigate the behavior of laterally loaded piles supporting integral abutments.

This research involves full scale lateral load tests of four single piles supporting integral abutments. The abutment is cast on top of a single steel H-pile, which is instrumented along its length with strain gages and pressure sensors. The abutment is slowly subjected to a lateral load to simulate thermal loading. During testing, deflections and loads are monitored such that the load, moment, and deflection at the pile-head are known. The testing program consists of field tests to achieve ground level deflections of both one-half inch and one inch, tests involving cyclic loading, and tests where the load is increased to the point where possible failure of the abutment warrants the termination of testing. Bending moment diagrams along the length of the pile are available from field tests on the first two piles.

LPILE is a special purpose computer program which uses a finite difference approach to analyze laterally loaded piles. This program uses the same approach and methodology as COM624P, a program which is used by TDOT in the design of integral abutments. LPILE uses various soil properties to generate a soil response for a given type of soil. This soil response was found to be stiffer than the actual soil response at the test site; however, the actual soil response was found to vary widely between tests. The designer typically does not know the shear or moment at the pile-head and, therefore, cannot accurately solve for the soil response. LPILE output using the internally generated soil response, the design pile-head deflection, and the plastic moment at the pile-head was found to be conservative with respect to bending moment and unconservative with respect to buckling, which is generally a problem only in the softest soils.

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CHAPTER 1

INTRODUCTION

The evolution of the jointless bridge was significantly enhanced with the introduction in 1930 of the Hardy Cross Method for moment distribution (1). This method, combined with the increased use of computers in more recent years, allowed for the design of continuous bridges as opposed to bridges consisting of a series of simple spans. With this development came interest in integral abutments. Jointless bridges with integral abutments were constructed as early as the 1930's. One of the major advantages of jointless bridges is the exclusion of expansion joints, which are costly to install, maintain, and repair.

Design approaches for integral abutments have been based on engineering judgment, observations, and experiments. The absence of a precise design approach is partially due to the complex analysis of laterally loaded piles, a complexity caused by the nonlinear response of soil (2). The solution cannot be found using the equations of static equilibrium but must be found by solving a differential equation. The nonlinear response of the soil does not allow a closed-form solution to this differential equation. The problem is further complicated by the fact that it deals with the interaction between structure and soil; that is, the pile deflection is dependent upon the soil pressure, and, inversely, the soil pressure is dependent upon the pile deflection.

The Tennessee Department of Transportation (TDOT) routinely designs and constructs jointless bridges for short to medium bridge lengths (1). The TDOT design

approach is aided by a special purpose computer program named COM624P. This program uses a finite difference approach to analyze laterally loaded piles.

Due to the desire to design jointless bridges over longer spans than warranted by the current design approach, TDOT sponsored a current research contract with the University of Tennessee, Knoxville to investigate the behavior of laterally loaded piles supporting integral abutments. Two main areas of interest of this research are the effects of thermally induced stresses both at the interface between the pile and abutment and between the pile and soil (3).

The field research involves casting an abutment on top of a driven steel H-pile, which is instrumented with strain and pressure gages, and slowly applying a lateral load to simulate thermal loading. Testing is to be conducted on a total of four piles. Each pile is tested six times in each direction; these six tests consist of three tests to achieve a ground level pile deflection of one-half inch and three tests to achieve a ground level pile deflection of one inch. After these tests are completed on each pile, other tests are run, including cyclic loading and loading to a point where potential failure of the abutment necessitates termination of testing. Testing of the first two piles, which were driven in virgin soil, is nearing completion. The final two piles will be driven in engineered fill material. The resulting data from the first two piles are available.

This thesis focuses on the pile supporting the integral abutment and the correlation between field test data and the analysis results of a special purpose computer program named LPILE, which uses the same approach and methodology as COM624P.

Purpose. The primary purposes of this thesis are

(a) to compare field test data with LPILE analysis results using the internally generated soil response curves (p-y curves) based on input soil parameters,

(b) to modify LPILE generated p-y curves to fit field test data better, and

(c) to provide p-y curves for comparison with experimental p-y curves which will be developed in the future.

Scope. The field test data were taken from selected tests; fourteen sets of data were chosen. A one-half inch test and a one inch test were selected from each pile from tests pulling in each direction. Data sets were chosen for a ground level pile deflection of approximately one-half inch for the one-half inch tests and for deflections of one-half inch and one inch for the one inch tests. In addition, one increment from a cyclic loading test and one increment from the "loading to failure" test on the first pile were selected.

CHAPTER 2

REVIEW OF LITERATURE ON LATERALLY LOADED PILES SUPPORTING INTEGRAL ABUTMENTS

Much of the recent research pertaining to laterally loaded piles supporting integral abutments has involved laboratory, computer, and mathematical modeling. There has also been some research involving full scale field testing of laterally loaded piles which is reviewed in the following sections.

2.1 NORTHERN CALIFORNIA

Several full scale tests were run on vertical piles at three different bridge construction sites in northern California (4). Three main objectives of this research were determining the lateral load-deflection behavior of the piles, correlating the test results with theoretical results, and determining experimental values of the subgrade modulus.

The piles on two of the sites were steel pipe piles which had outside diameters of 10.75 inches, while the piles on the other site were steel H-piles. The piles were backfilled with 12 to 20 feet of compacted annular pea gravel, which was surrounded by compacted fill material. The lower portion of each pile was embedded in residual soil ranging from sandy silt to clay.

All of the test piles were used in the construction of bridge abutments. The piles were tested before construction of the abutment and bridge superstructure by jacking horizontally between two piles. Piles were also tested after construction of the abutment and bridge superstructure. The loads for these tests were taken as the post-tensioning

effects and dead load of the bridge superstructure. The tests before construction were assumed to have a free pile-head condition, while the tests after construction were assumed to have a fixed pile-head condition.

The theoretical approach used to compare with actual test results was that of a beam on an elastic foundation. This approach involves a flexural member, of stiffness EI, supported along its length by independent springs spaced infinitely close to each other (5). The stiffness of these springs represented the subgrade modulus.

From comparing the tests before and after construction, it was determined that at a pile-head deflection of 0.25 inch, a pile with a fixed-head was twice as stiff as the same pile with a free-head condition (4). It was also found that the effective length for lateral loads of 9 to 16 kips varied from 8 to 12 feet. The effective length was defined as a measure of the depth at which there is no significant lateral deflection, and in turn, no significant soil pressure on the pile. The theoretical solutions found from the beam on elastic foundation analysis were found to be a reasonable approximation of the test results. The subgrade modulus was found to vary from 10 to 65 lb/in^3. No mention was made of experimental results pertaining to the interface between the pile and abutment.

2.2 IOWA STATE

Researchers at Iowa State University performed tests in 1987 to determine if lateral loads on H-piles reduced the vertical load capacity of the piles (6). These tests were also run to verify finite element modeling. The tests were performed on two instrumented steel HP10x42 piles, which were 50 feet long and driven to a depth of 40 feet.

There were three types of tests run consisting of a vertical load test, a lateral load test, and a combined vertical and lateral load test. The vertical load test was used to determine soil characteristics pertaining to skin friction, vertical displacement, and tip displacement. A load of 280 kips was determined to be the ultimate load before a bearing capacity failure occurred. The lateral load test involved slowly pulling the pile, with bending about its strong axis, to a deflection of 2.06 inches at the ground surface. The results of the lateral load test were used to develop a relationship between lateral resistance and lateral displacement called p-y curves. These curves were developed and were within reasonable agreement with theoretical p-y curves. The combined load test showed no decrease in vertical load capacity due to the lateral load applied. The pile was initially displaced 1.93 inches laterally at the ground surface, and the ultimate vertical load, as in the vertical load test, was found to be 280 kips. This vertical load of 280 kips increased the lateral deflection to 2.40 inches.

The researchers used these test results to verify finite element modeling with a computer program named IAB2D. The pile was modeled using two-dimensional nonlinear beam elements, while the soil was modeled as nonlinear springs. The modeling was considered to be generally successful. The load-deflection results from the computer program were very close to those from the field testing, while the moment diagrams showed slightly higher moment values than those obtained from field testing.

CHAPTER 3

TESTING PROGRAM

3.1 INTRODUCTION

Testing on the first two piles is nearing completion. These two piles were driven into undisturbed, overconsolidated soil. Preparations have begun for testing the next two piles, which will be driven into engineered fill material. The test site is located on the bridge construction site at the interchange between Pellissippi Parkway and Dutchtown Road, approximately fifteen miles west of the University of Tennessee, Knoxville. The test setup, instrumentation, and test regimen described in the following sections pertain to the first two piles; however, few deviations, if any, are expected for the final two piles.

3.2 TEST SETUP

The first two piles were HP10x42 steel piles and were each forty feet long. Each pile was driven 38 feet deep, leaving one foot for embedment into the abutment and one foot exposed for instrumentation. The abutment was reinforced in accordance with a typical integral abutment designed by the Tennessee Department of Transportation (TDOT). Concrete ballast was cast on top of the abutment to provide an axial load on the pile of approximately 65 kips, typical of the load on an integral abutment pile. The rotational stiffness of the abutment was made similar to that of a typical integral abutment by allowing the abutment slab to bear on a roller support which in turn was set on a reaction beam of known stiffness. The loading was applied through the centroid of the abutment slab causing bending about the strong axis of the pile. The loading was done

through a Dywidag post-tensioning threaded bar. This bar ran through a three inch pipe which was cast into the abutment slab along its centroidal axis. The Dywidag bar had a plate and nut on one end which allowed the abutment to be pulled, and the other end went to the load frame. The load was applied at the load frame using a hydraulic jack. The test setup is shown in Figure A-1.

3.3 INSTRUMENTATION

The lateral load and the reaction between the reaction beam and the abutment beam were measured with load cells. One load cell was located at the load frame, while the other was placed between the roller support and the reaction beam. Two Linear Variable Differential Transformers (LVDTs) were used to measure horizontal deflection. One LVDT measured the pile deflection at the ground surface, while the other measured the abutment deflection just above the pile-abutment interface. The piles were instrumented with weldable strain gages attached to the inside of the flanges. Each strain gage station along the pile consisted of four gages; there was one gage on the inside of each flange, each side of the web, to eliminate torsional effects. The piles were also instrumented with pressure sensors. The pressure sensors were mounted in a hole in the flange such that the face of the sensor was flush with the face of the flange.

On the first pile, there were strain gage stations spaced at eighteen inches on center starting at the ground surface and continuing to a depth of 19.5 feet. The pressure sensors on the first pile were also spaced every eighteen inches; the sensors started at a depth of 5'-3" and ended at 12'-9" on one side and went from a depth of 6'-9" to 14'-3" on the other side. There were a few changes in the instrumentation of the second pile.

Two extra strain gage stations were added, one at a depth of nine inches and the other at a depth of 27 inches. The pressure sensors were spaced the same but started at a depth of 5'-3" and continued to a depth of 12'-9" on both sides. Also, late in the testing of the second pile, a strain gage was added to the outside of each flange above the web at the pile-abutment interface.

The strain gages and pressure sensors were protected with a strip of sheet metal; one strip was welded to each flange and then to the web. Eighteen gage galvanized sheet metal was used on the first pile, while sixteen gage ungalvanized sheet metal was used on the second. A triangular steel wedge was welded to the bottom of each strip to ensure that the sheet metal stayed attached to the pile during driving. Insulating foam was injected into the space between the pile and sheet metal to protect the gages and wires from vibrations during driving. All instrumentation was monitored with an Optim MEGADAC 3108 data acquisition system.

3.4 TEST REGIMEN

Each pile was tested six times in each direction. The first three tests in each direction achieved a ground level pile deflection of one-half inch, while the second three tests achieved a pile deflection of one inch. After these six tests, other tests were run on each pile including cyclic loading and loading to a point where potential failure of the abutment necessitated termination of testing.

The loading regimen is an important aspect of the experiment; the loading was designed to be slow and gradual to simulate thermal expansion. The loading is deflection controlled; that is, the abutment is loaded until the pile reaches a certain deflection at the

ground surface. The jacking time for each deflection step is approximately ten minutes; the loading is then stopped and remains essentially constant through the remainder of the time period. Once the maximum deflection of the test is reached, the loading is stopped for approximately ten minutes, and the load is slowly released. The loading regimens for a one-half inch and a one inch test are shown in Figures A-2 & A-3, respectively.

3.5 SELECTED RESULTS

Testing on the second pile is nearing completion. For all tests conducted on the first two piles, there are currently available plots of pile bending moment vs. depth for each deflection increment. This thesis focuses on the moment vs. depth curves described in Table 1.

Table 1. Selected Field Test Results

Test Date	**Test Description**	**Pile Deflection at Abutment**
6/24/97	Pile 1, Second ½" Test Pulling East	0.645 inches
7/24/97	Pile 1, First 1" Test Pulling East	0.631 inches
7/24/97	Pile 1, First 1" Test Pulling East	1.223 inches
8/15/97	Pile 1, First ½" Test Pulling West	0.631 inches
8/25/97	Pile 1, First 1" Test Pulling West	0.643 inches
8/25/97	Pile 1, First 1" Test Pulling West	1.274 inches
9/8/97	Pile 1, Cyclic Loading Pulling West	1.825 inches
9/12/97	Pile 1, Load to Failure Pulling West	3.127 inches
1/30/98	Pile 2, Second ½" Test Pulling East	0.609 inches
2/13/98	Pile 2, First 1" Test Pulling East	0.622 inches
2/13/98	Pile 2, First 1" Test Pulling East	1.239 inches
6/4/98	Pile 2, Third ½" Test Pulling West	0.649 inches
6/26/98	Pile 2, Second 1" Test Pulling West	0.670 inches
6/26/98	Pile 2, Second 1" Test Pulling West	1.520 inches

CHAPTER 4

LPILE

4.1 INTRODUCTION

LPILE is a special purpose computer program for analyzing laterally loaded piles and drilled shafts. LPILE uses a finite difference approach to solve the differential equation for a beam-column while taking into account the nonlinear soil response.

4.2 THEORY OF LPILE

4.2.1 DIFFERENTIAL EQUATION OF BEAM-COLUMN

LPILE solves the problem of the laterally-loaded pile by finding a solution for the differential equation for a beam-column, thus including the effects of axial and lateral loads. The differential equation is solved using a finite difference numerical method. The general form of this equation is as follows (2):

$$\frac{d^2M}{dx^2} + Q\frac{d^2y}{dx^2} - \frac{dV_V}{dx} = 0 \qquad (4-1)$$

Where:

- x = length of pile
- Q = axial load on pile
- M = bending moment on pile at any point x along length of pile
- y = lateral deflection of pile at any point x along length of pile
- V_v = shear force in pile

After several substitutions are made, the final form of this equation is as follows (2):

$$EI\frac{d^4y}{dx^4} + Q\frac{d^2y}{dx^2} - p + w = 0 \qquad (4-2)$$

Where: EI = flexural rigidity of pile

p = soil reaction per unit length

w = distributed load along length of pile

The following beam formulas are also used to analyze laterally-loaded piles (2):

$$EI\frac{d^3y}{dx^3} + Q\frac{dy}{dx} = V_v \qquad (4-3)$$

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

$$\frac{dy}{dx} = S \qquad (4-5)$$

Where: S = slope of the elastic curve defined by the axis of the pile.

The following assumptions are made in the derivation of the differential equation (2):

1. The pile is straight and has a uniform cross-section.
2. The pile has a longitudinal plane of symmetry; loads and reactions lie in that plane.
3. The pile material is homogeneous.
4. The proportional limit of the pile material is not exceeded.

5. The modulus of elasticity of the pile material is the same in tension and compression.
6. Transverse deflections of the pile are small.
7. The pile is not subjected to dynamic loading.
8. Deflections due to shearing stresses are small.

4.2.2 NONLINEAR SOIL BEHAVIOR

Before solving the differential equation for the beam-column, a plot of soil resistance (p) vs. pile deflection (y) must be determined for each point along the pile. These plots are commonly known as p-y curves. A given pile has a family of p-y curves where each curve is a function of depth. LPILE Plus 3.0 contains nine models of p-y curves to choose from:

1. Soft clay with free water
2. Stiff clay with free water
3. Stiff clay without free water
4. Sand
5. User defined p-y curves
6. Vuggy limestone (strong rock)
7. Silt (with cohesion and internal friction angle)
8. API Sand Model
9. Weak Rock

These models were determined from experimental bending moments from various field tests. The soil resistance was determined by differentiating the moment curve twice; the pile deflection was determined by integrating the moment curve twice.

$$\text{Deflection} = y(x) = \iint \frac{M(x)}{EI} dx \qquad (4-6)$$

$$\text{Soil Resistance} = p(x) = \frac{d^2M(x)}{dx^2} \qquad (4-7)$$

It is worth noting that since the moment curve is a function of depth, the deflection, soil resistance, and thus the p-y curves are functions of depth.

Since the experimental data presented are from a site consisting of stiff clay above the water table, the following is a discussion of the "stiff clay with no free water" model. This model is based on field tests performed in Houston, TX. These tests were performed using drilled shafts which were 36 inches in diameter and 42 feet long. The ultimate soil resistance is based on the most critical of two modes of failure. The first mode of failure is a wedge failure of the soil near the surface; this value of ultimate soil resistance is referred to as p_{u1}. The second mode of failure occurs when the soil flows around the pile at greater depths; this value of ultimate soil resistance is referred to as p_{u2}. Therefore, the ultimate soil resistance, p_u, is equal to the smaller of p_{u1} and p_{u2}.

$$p_{u1} \text{ is taken as } \left[3 + \frac{\gamma'}{c}x + \frac{J}{b}x\right]cb \qquad (4-8)$$

$$p_{u2} \text{ is taken as } 9cb \qquad (4-9)$$

Where: c = average undrained shear strength (stress units) from surface to point at which p-y curve (pressure vs. lateral deformation) is computed

γ' = average unit weight above the point at depth x

x = depth

b = width or diameter of pile

J = 0.5

The equation of the p-y curve is as follows:

$$p = 0.5p_u(y/y_{50})^{1/4} \qquad \text{for } y \leq 16y_{50} \qquad (4\text{-}10)$$

$$p = p_u \qquad \text{for } y > 16y_{50} \qquad (4\text{-}11)$$

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

ε_{50} = strain corresponding to one-half the average undrained shear strength

This p-y curve is shown in Figure A-4. It is worth noting that including an internal friction angle (ϕ) or soil modulus (k) with this model has no effect on the analysis. This is not the case with some of the other LPILE p-y curve models (2).

4.3 REVIEW OF LITERATURE ON LPILE

4.3.1 HISTORY OF DEVELOPMENT

After the introduction of the IBM XT Personal Computer in 1984, Ensoft, Inc. began the development of LPILE 1.0 in 1985 (7). This development was completed under the direction of Dr. Lymon C. Reese, and LPILE 1.0 was commercially distributed the

following year. LPILE 1.0 operated under the same theory and processes as COM624 which was a mainframe program at the time; however, the programming had been completely rewritten and contained a more user-friendly input method. Some of the features of LPILE 1.0 included

- the ability to internally generate p-y curves for four different soil types
- the option for user-defined p-y curves to be input
- the introduction of p-y curves for layered soil
- the availability of four pile-head boundary conditions and loading types
- the introduction of an interactive input method
- the option of considering tip-resistance curves

With a few improvements to LPILE 1.0, most notably a separate graphics package, LPILE 2.0 was introduced in 1987. Two years later LPILE 3.0 was introduced including the following features:

- more user-friendly than previous versions
- a new p-y curve criterion
- the ability to modify internally generated p-y curves
- the option to set the pile-head below the ground surface

In 1993, four years after the release of the previous version, LPILE 4.0 (DOS Version) was introduced. This updated version contained the following features:

- two new p-y curve criteria
- the ability to consider sloping ground
- new graphics plots

In the same year, LPILE Plus 1.0 (DOS Version) was introduced. This package combined LPILE 4.0 with a program called STIFF allowing the user to consider the flexural rigidity of cracked and uncracked pile sections individually.

In 1995, with the success of Microsoft Windows 3.1, LPILE 4.0 (Windows Version) was released. Also in 1995, LPILE Plus 2.0 (Windows Version) was released and contained a graphics package which ran in windows. Two years later LPILE Plus 3.0 (Windows Version) was introduced with the following features:

- a new p-y curve criterion
- the option to view a plot of pile length vs. pile-head deflection
- the output option to include a foundation stiffness matrix to be used in structural analysis programs
- more user-friendly than previous versions
- English/SI unit conversions done internally

4.3.2 REVIEW OF SUCCESSFUL USE OF LPILE

Hoyt, et. al., in their investigation of the buckling of helical anchors used for underpinning of distressed foundations, were able to confirm LPILE results with field tests. They concluded that, as in conventional pile foundations, buckling is only a concern when these anchors are long and placed in very soft soil (8).

4.4 APPLICATION IN THIS THESIS

LPILE Plus 3.0 for Windows is used in this thesis. Since the experimental data presented pertain to a stiff clay above the water table, the only p-y curve criterion used is

the stiff clay with no free water criterion. Initially the program was run using the stiff clay with no free water criterion to provide a comparison between LPILE output using this curve and experimental data. In an effort to match the experimental data more closely, the internally generated p-y curve was modified.

4.5 DETAILS OF USING LPILE

The details of using LPILE are described in Appendix B.

4.6 LIMITED SENSITIVITY STUDY

In an effort to better understand how the soil properties which are used in the p-y curves affect the final results, a limited sensitivity study is provided in Appendix C.

CHAPTER 5

COMPARISON OF LPILE RESULTS AND FIELD TEST RESULTS

5.1 INITIAL COMPARISONS

Comparisons are based on the shear and deflection at the pile-head and the moment diagram along the length of the pile. For each set of selected field data, LPILE was run using two different loading conditions. For the first condition, the moment and shear at the pile-head were input, and the output of interest was the calculated pile-head deflection and resulting moment diagram. For the second condition, the moment and deflection at the pile-head were input, and the output of interest was the calculated shear at the pile-head and resulting moment diagram.

The pile-head was taken as the interface between the pile and abutment, which left a pile length of 39 feet with a length of one foot above the ground surface. The moment at the pile-head was found by adding to the moment at the ground surface, which was calculated from strain gages, a value equal to the lateral load multiplied by one foot. This calculation is justified since the lateral load is equal to the shear at the pile-head, which is constant over the length of the pile between the ground surface and the abutment. The axial load used by LPILE was taken as the remainder of the weight of the abutment and ballast after subtracting from it the vertical reaction between the abutment beam and reaction beam. It is worth mentioning that the moment of inertia of the pile used in the LPILE analysis, as well as in the field data analysis, was slightly higher than that of an HP10x42 shape; the value used was calculated taking into account the protective sheet metal attached along the length of the pile. The soil properties input

were determined by laboratory tests; they consisted of a unit weight of 0.069 lb/in^3, an average undrained shear strength of 35.24 psi, and a strain of 0.003 corresponding to one-half the average undrained shear strength (3).

The initial comparisons were made between field test data and LPILE analysis results, using the internally generated p-y curves for a stiff clay above the water table. The LPILE analyses were not in agreement with the experimental data. The pile-head deflections calculated by LPILE were consistently smaller than actual for the first loading condition, and the shears at the pile-head calculated by LPILE were consistently larger than actual for the second loading condition. These results showed that the soil model used was too stiff. Also, the moment diagrams did not fit the test data and were not in agreement between loading conditions; however, the moment curves from the two loading conditions appeared to bound experimental data in many cases.

5.2 COMPARISONS AFTER SOIL RESPONSE CURVE MODIFICATION

Using p-y curve modification factors, the p-y curves were made less stiff by multiplying all of the y values by a factor greater than one. For each set of data, trial values of y multipliers were used until the calculated pile-head deflection from the first loading case and the shear at the pile-head from the second case were in agreement with field test data. At this point, the moment diagrams from each load case converged and were identical. The calculated moment diagrams were then compared to field data, and were found to be in general agreement. These comparisons can be seen in Figures A-5 through A-18. The LPILE analyses were generally conservative with respect to

maximum moment and predicted the second point of zero moment to be within two feet above that measured in field tests.

Unfortunately the modification factors used to unstiffen the p-y curves were by no means consistent, ranging from 1.15 to 3.89. Even without consistent p-y curves, a couple of patterns were expected. It was expected that for each pile in each direction, the one-half inch test would show a stiffer soil than a similar deflection increment of a following one inch test. The chosen data followed this pattern with one exception. It was also expected that for the larger deflection increment of a one inch test, the soil would appear to be stiffer than for the smaller deflection increment, due to the fact that the smaller deflection had been achieved three more times than the larger deflection during the one-half inch tests. The chosen data consistently followed this pattern. Although the chosen data appears to generally follow these patterns, a typical p-y curve modification factor could not be chosen due to their variability.

5.3 COMPARISONS USING PLASTIC MOMENT AT PILE-HEAD

The fact that a single p-y curve modification factor could not be determined is unfortunate since the designer typically knows only the axial load and deflection at the pile-head and cannot go through the steps above to determine the soil response. The designer can determine the deflection at which a plastic hinge forms in the pile by running LPILE analyses using a fixed pile-head and variable deflections until the pile reaches its plastic moment. At higher deflections, the moment at the pile-head will remain equal to the plastic moment. Another comparison was made by running LPILE with the plastic moment at the pile-head. LPILE was run using only one loading

condition; the loading input included the actual pile-head deflection, the plastic moment at the pile-head, and an axial load equal to the weight of the abutment and ballast. The internally generated p-y curves were used. As before, the resulting moment diagrams were compared to field data; these comparisons can be seen in Figures A-19 through A-32. These LPILE analyses were conservative with respect to both positive and negative bending moments and were not unreasonable for use in design. However, LPILE predicted the second point of zero moment generally to be two feet above that measured in field data, which is unconservative with respect to buckling when using the TDOT design approach. The unbraced length of the pile is taken as the distance between the first two points of zero moment (3).

5.4 EXPERIMENTAL SOIL RESPONSE CURVES

Experimental p-y curves can be developed from field test data. Diagrams of pressure vs. depth and deflection vs. depth are needed. The pressure diagram is obtained by differentiating the moment diagram twice, and the deflection diagram is obtained by integrating the moment diagram twice. At a given depth, the pressure and deflection at a loading increment represent a point on the p-y curve. At another loading increment, the pressure and deflection at the same depth represent another point on the same p-y curve. As mentioned in an earlier chapter, p-y curves are a function of depth. Development of these curves from experimentally determined pressure vs. depth and deflection vs. depth diagrams is beyond the scope of this thesis.

CHAPTER 6

CONCLUSIONS

LPILE output can accurately match field conditions if no more than one boundary condition is unknown. The comparisons in this thesis were made knowing the moment, shear, and deflection at the pile-head as well as the axial load on the pile; the unknown boundary condition was the soil response (p-y curves), which was solved for. On the other hand, the typical designer of an integral abutment knows only the deflection at the pile-head, the axial load on the pile, and a few soil properties; this information is inadequate to determine soil response.

The soil response is crucial for accurate LPILE analyses, and the internally generated soil response was shown to be stiffer than the actual soil at the field test site. The soil response was also shown to vary from one test to another and seemed to be sensitive to the level of lateral load applied to the abutment, the amount of soil reconstitution since the previous test, and the water content of the soil. The field tests were conducted with various periods of time between tests; there were also tests run during both very rainy months and very dry months.

As mentioned before, in the typical design case, all that is known is the pile-head deflection, axial load on the pile, and a few soil properties. The analysis based on this known deflection, plastic moment at the pile-head, and internally generated p-y curves was shown to be conservative with respect to pile bending moment. However, because the lower inflection point predicted by LPILE is higher than actual, the analysis is unconservative with respect to buckling of the pile, which is generally a problem only in

the softest soils. Thus, these comparisons indicate that LPILE does provide a reasonable, albeit approximate, method to design piles supporting integral abutments.

Further analysis of field test data in the current research will result in experimental p-y curves, which will be useful in comparing to the unstiffened curves used in this thesis. These experimentally determined curves should also provide further insight into the soil stiffness with respect to both depth and time.

LIST OF REFERENCES

LIST OF REFERENCES

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2. Reese, Lymon C., and Wang, S. T., "Technical Manual of Documentation of Computer Program LPILE Plus 3.0 for Windows." Ensoft Inc., Austin, Texas. 1997.

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APPENDICES

APPENDIX A

FIGURES

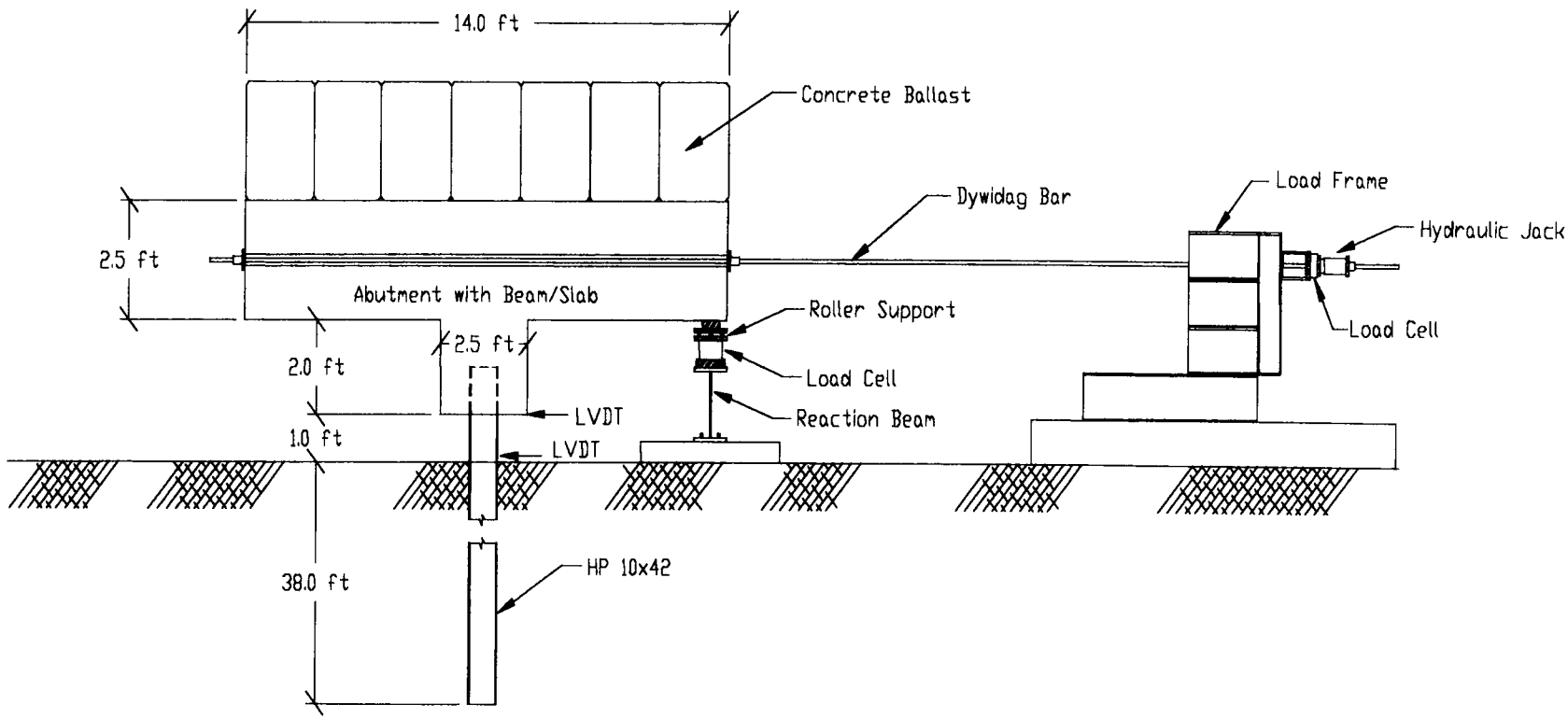


Figure A-1. Test Setup

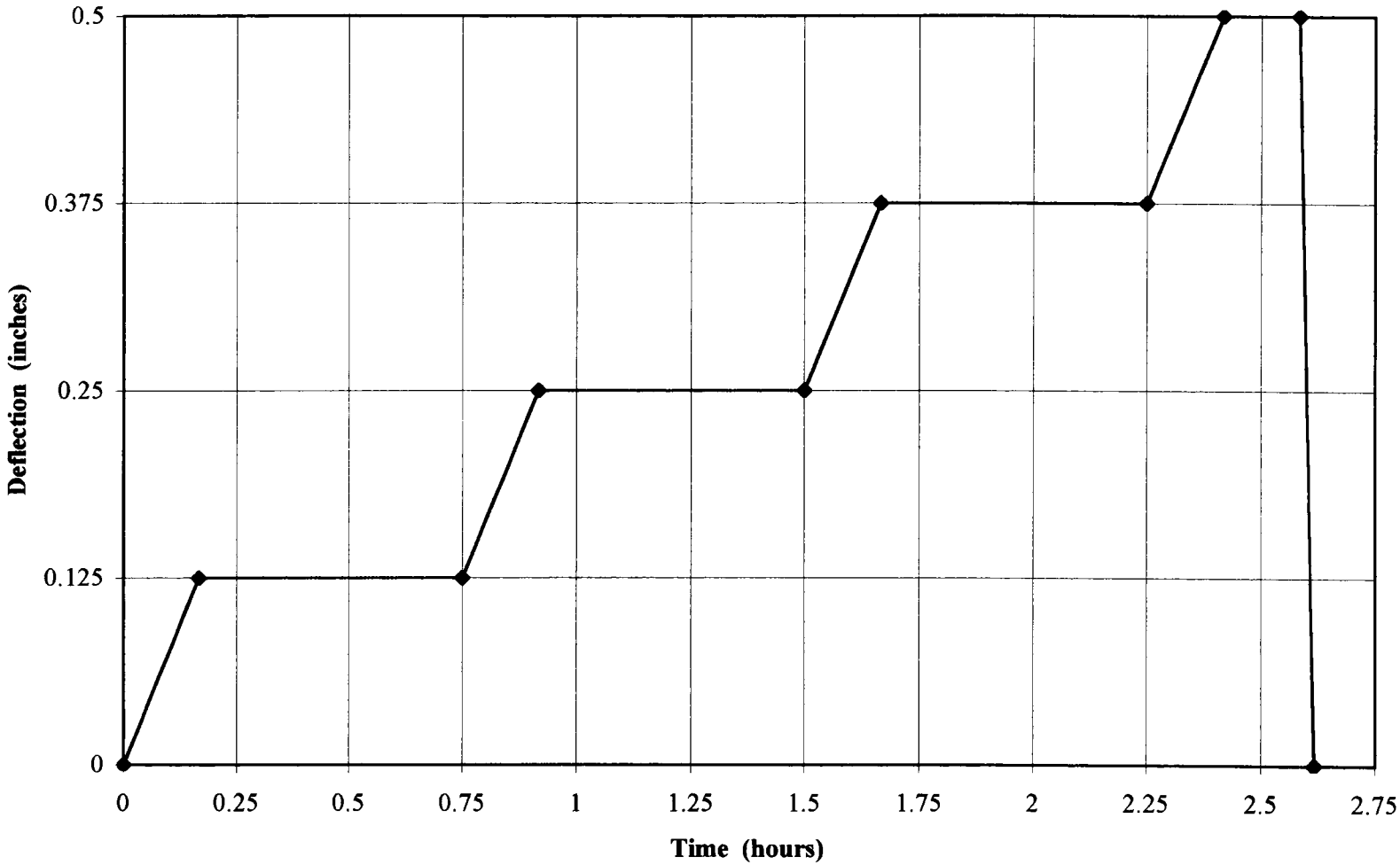


Figure A-2. Loading Regimen for a One-Half Inch Test

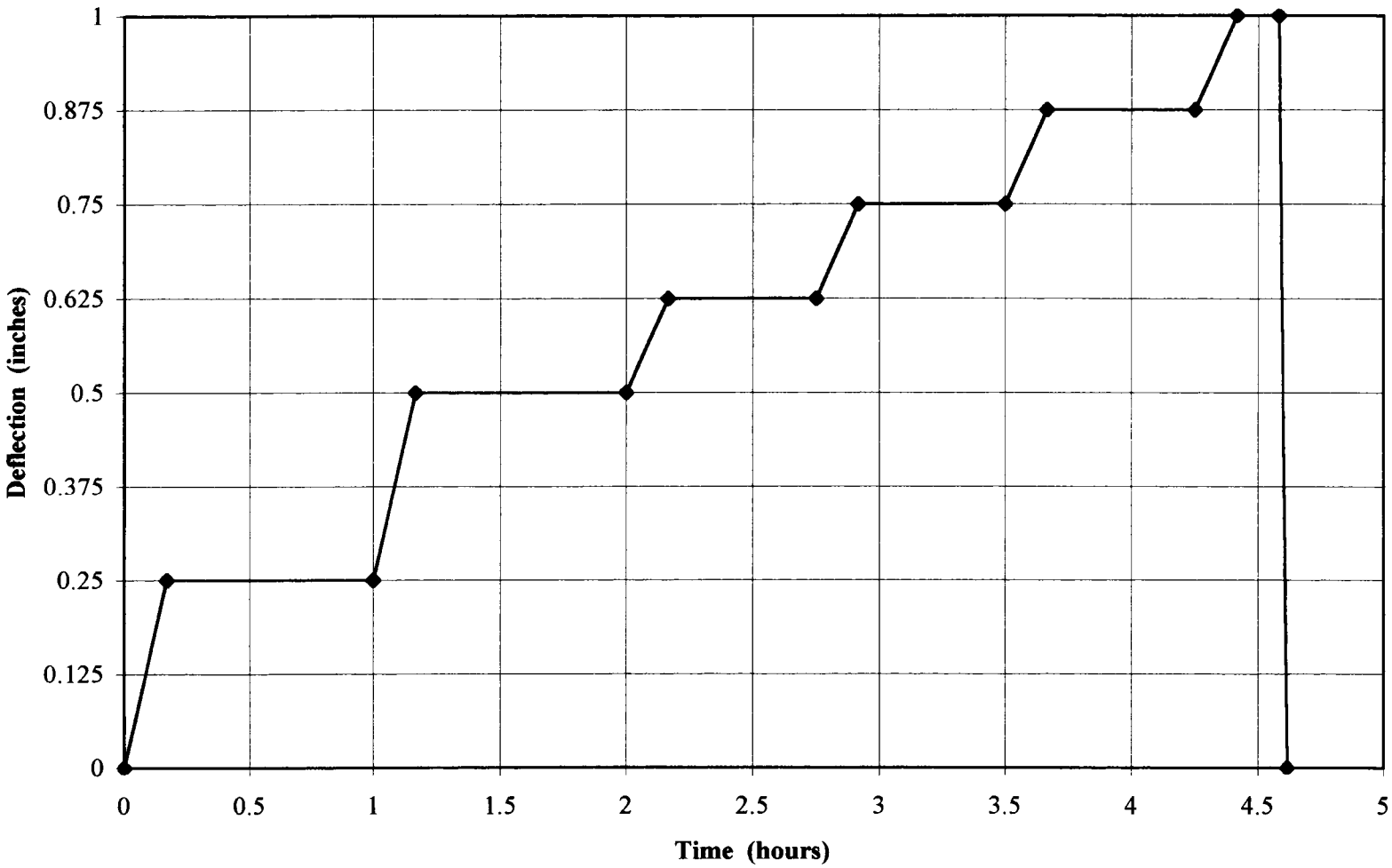


Figure A-3. Loading Regimen for a One Inch Test

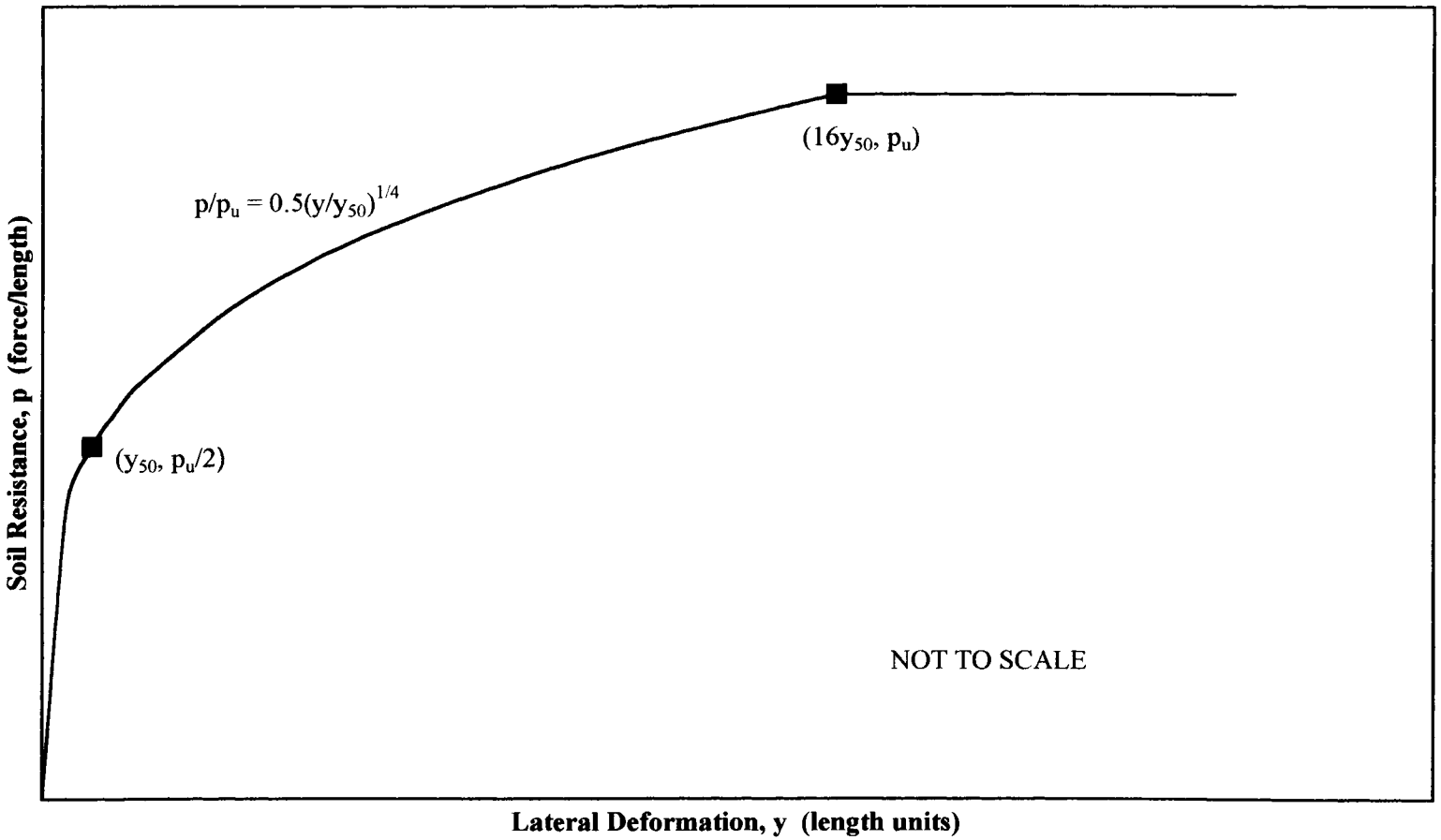


Figure A-4. General p-y curve for stiff clay with no free water

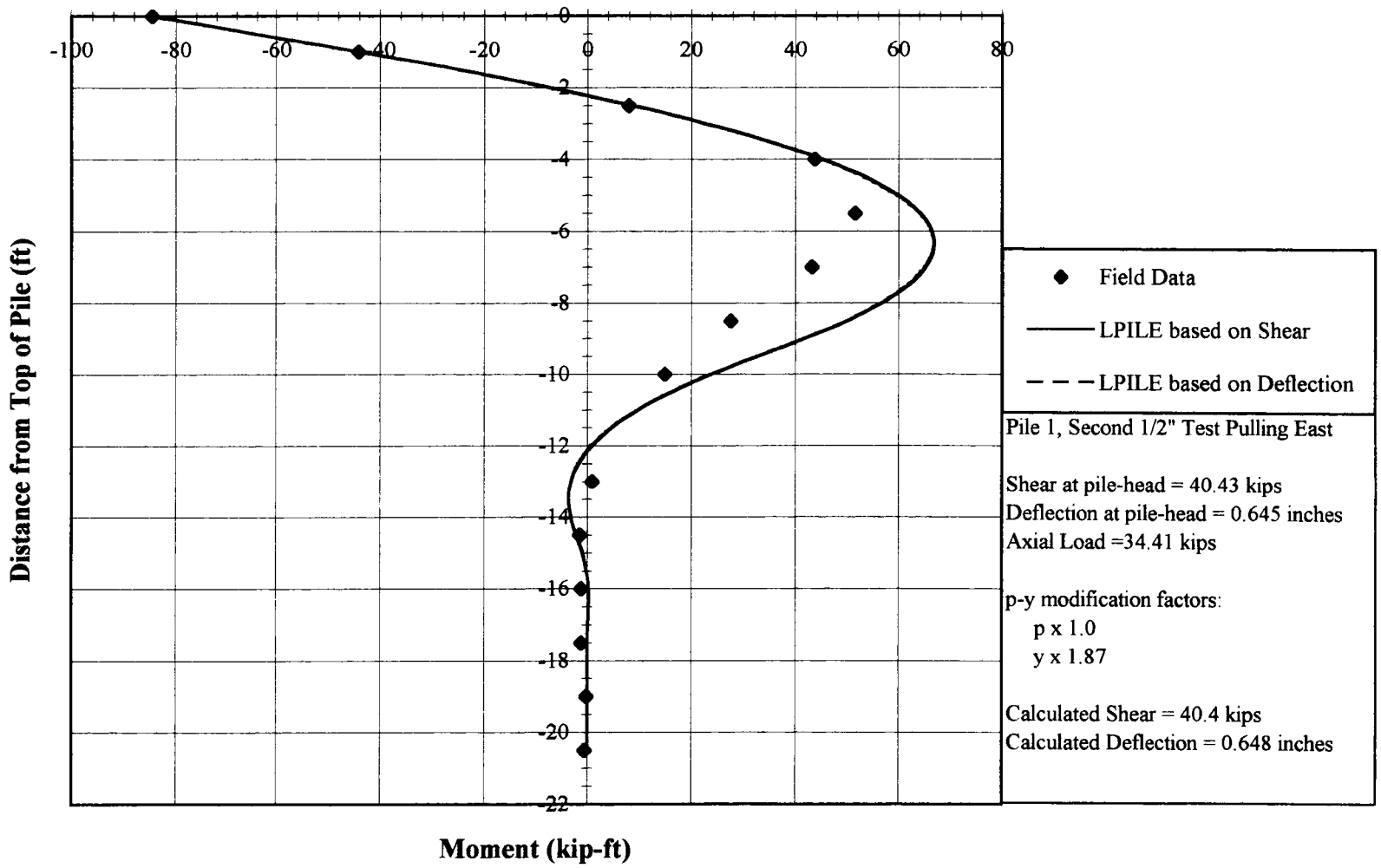


Figure A-5. Moment vs. Distance from Top of Pile Comparison Between LPILE Output and June 24, 1997 Field Test.

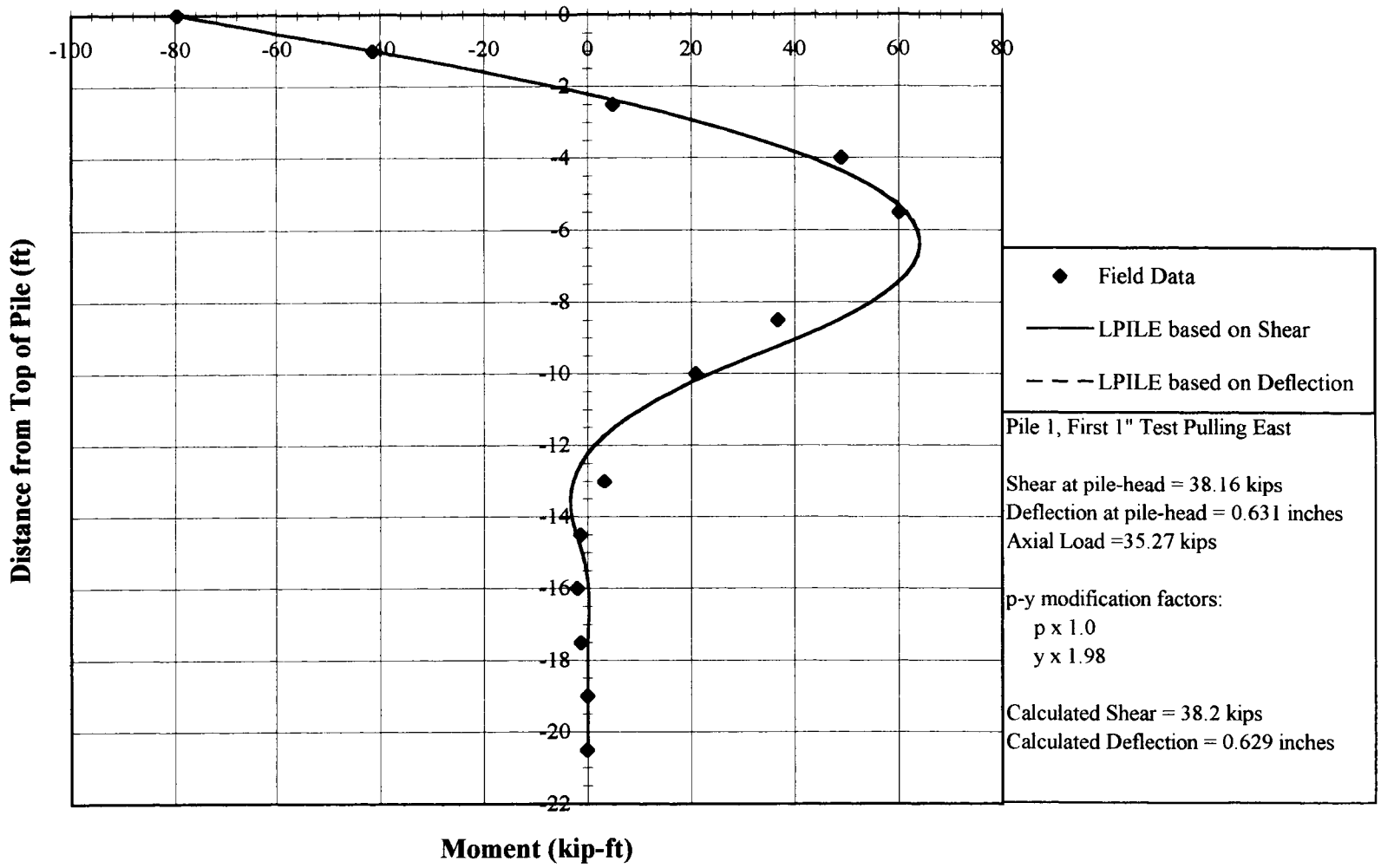


Figure A-6. First Moment vs. Distance from Top of Pile Comparison Between LPILE Output and July 24, 1997 Field Test.

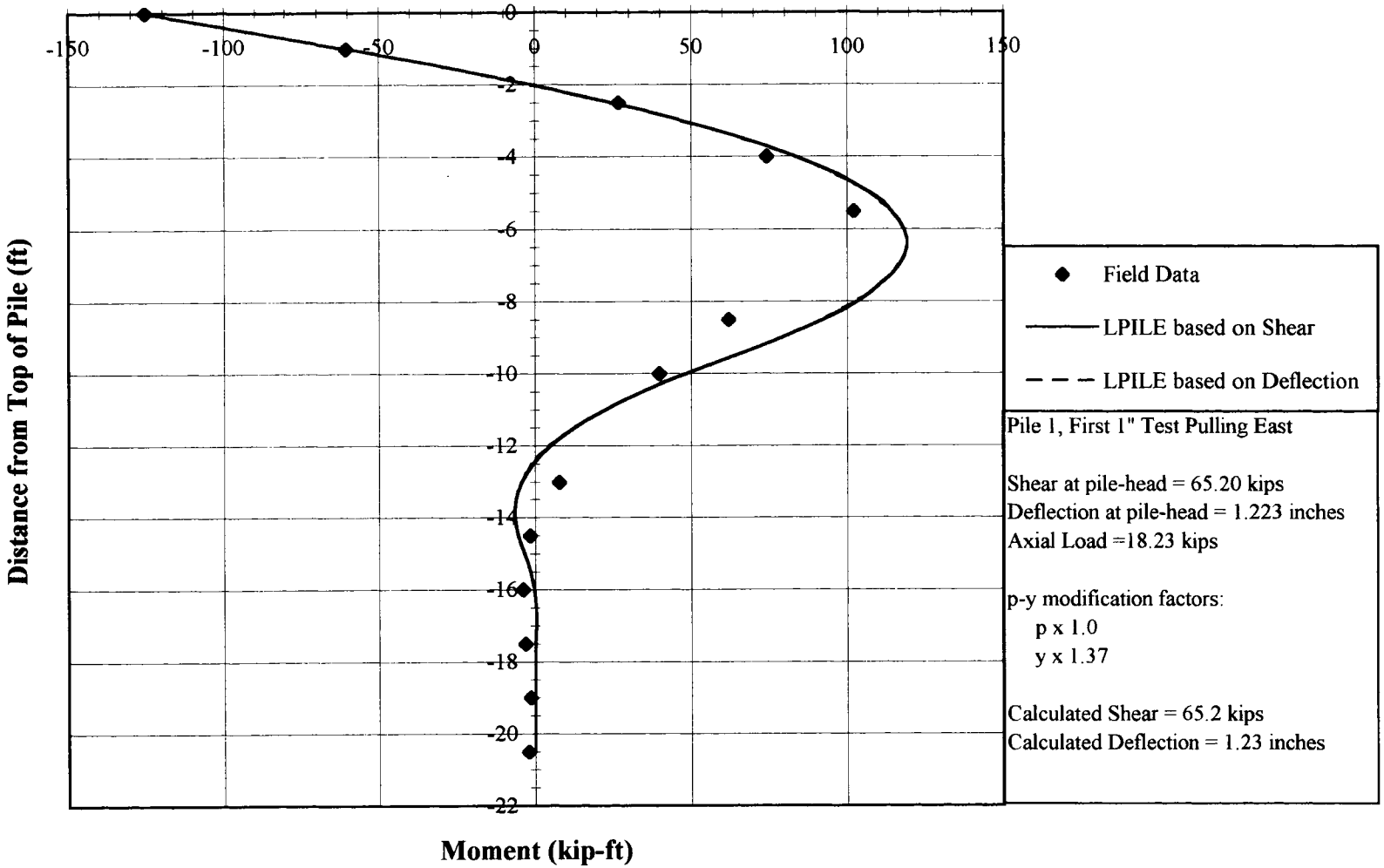


Figure A-7. Second Moment vs. Distance from Top of Pile Comparison Between LPILE Output and July 24, 1997 Field Test.

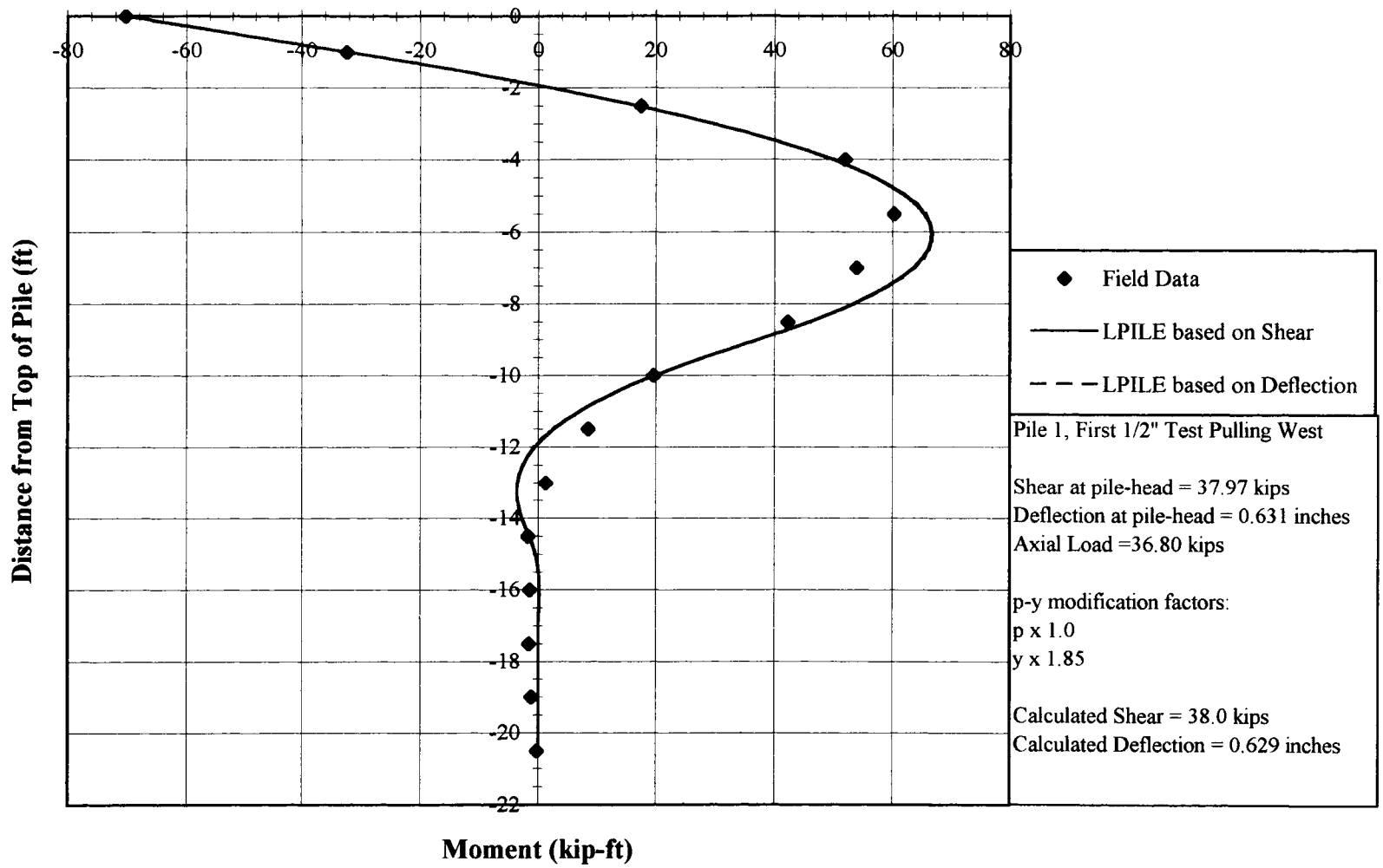


Figure A-8. Moment vs. Distance from Top of Pile Comparison Between LPILE Output and August 15, 1997 Field Test.

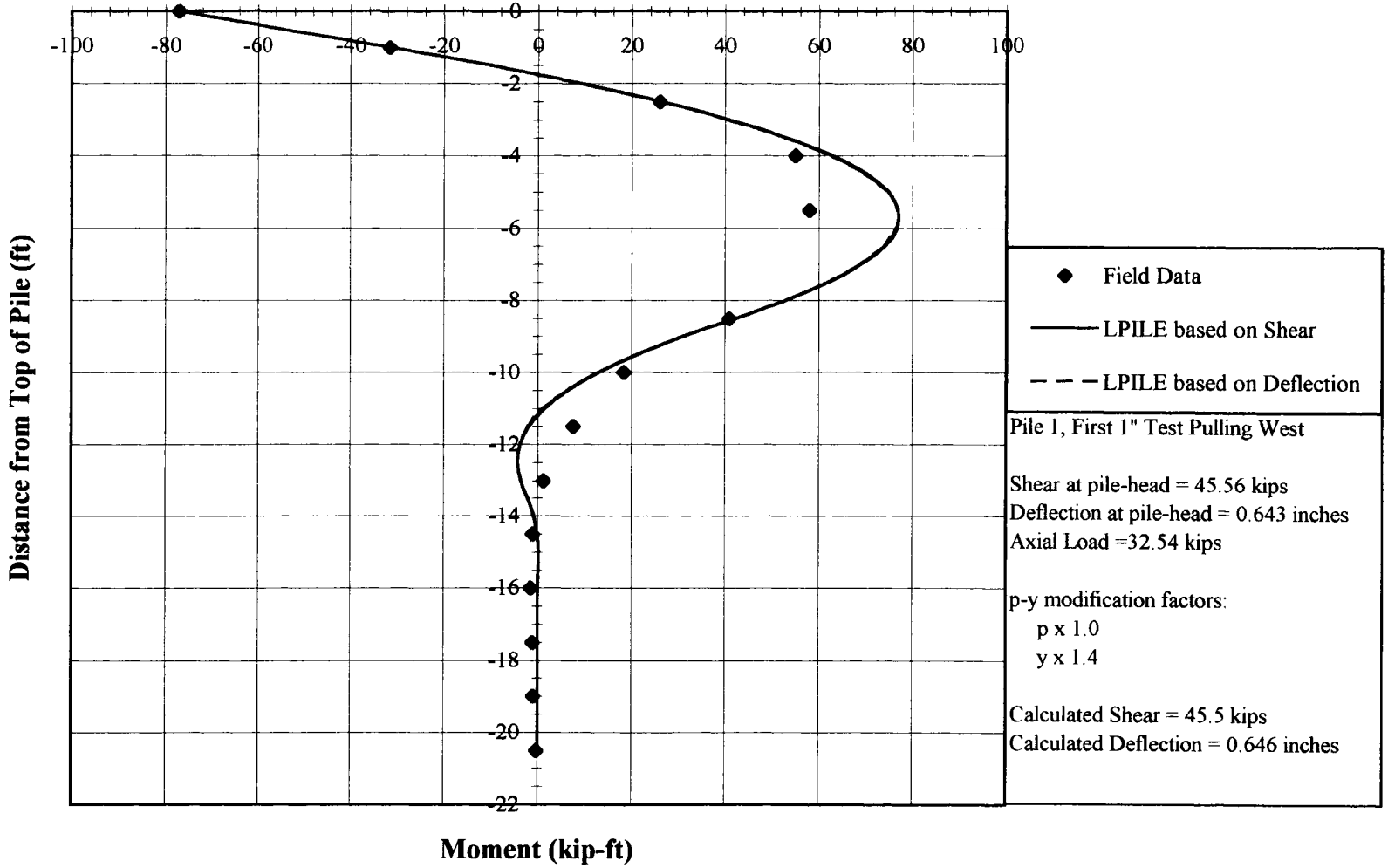


Figure A-9. First Moment vs. Distance from Top of Pile Comparison Between LPILE Output and August 25, 1997 Field Test.

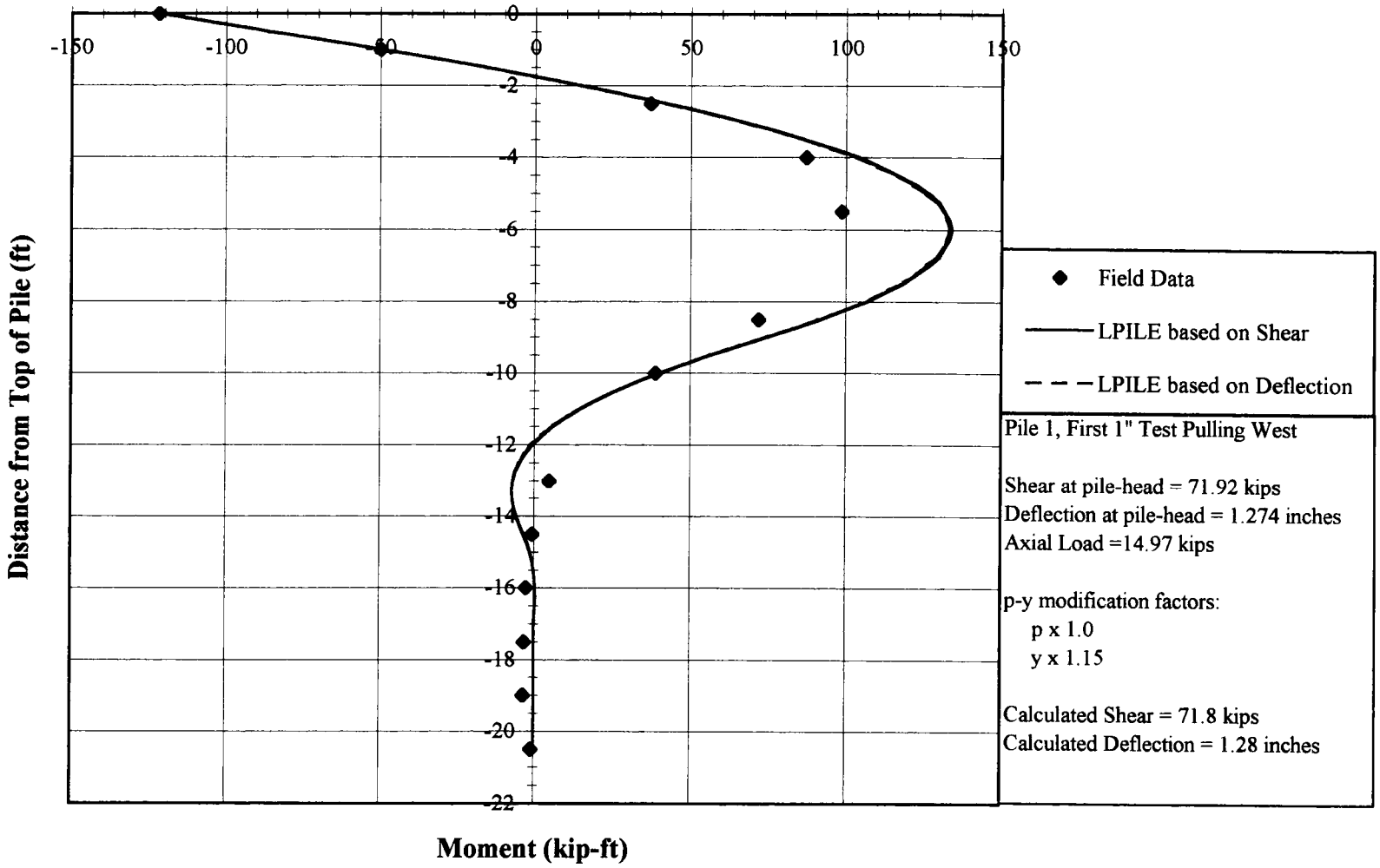


Figure A-10. Second Moment vs. Distance from Top of Pile Comparison Between LPILE Output and August 25, 1997 Field Test.

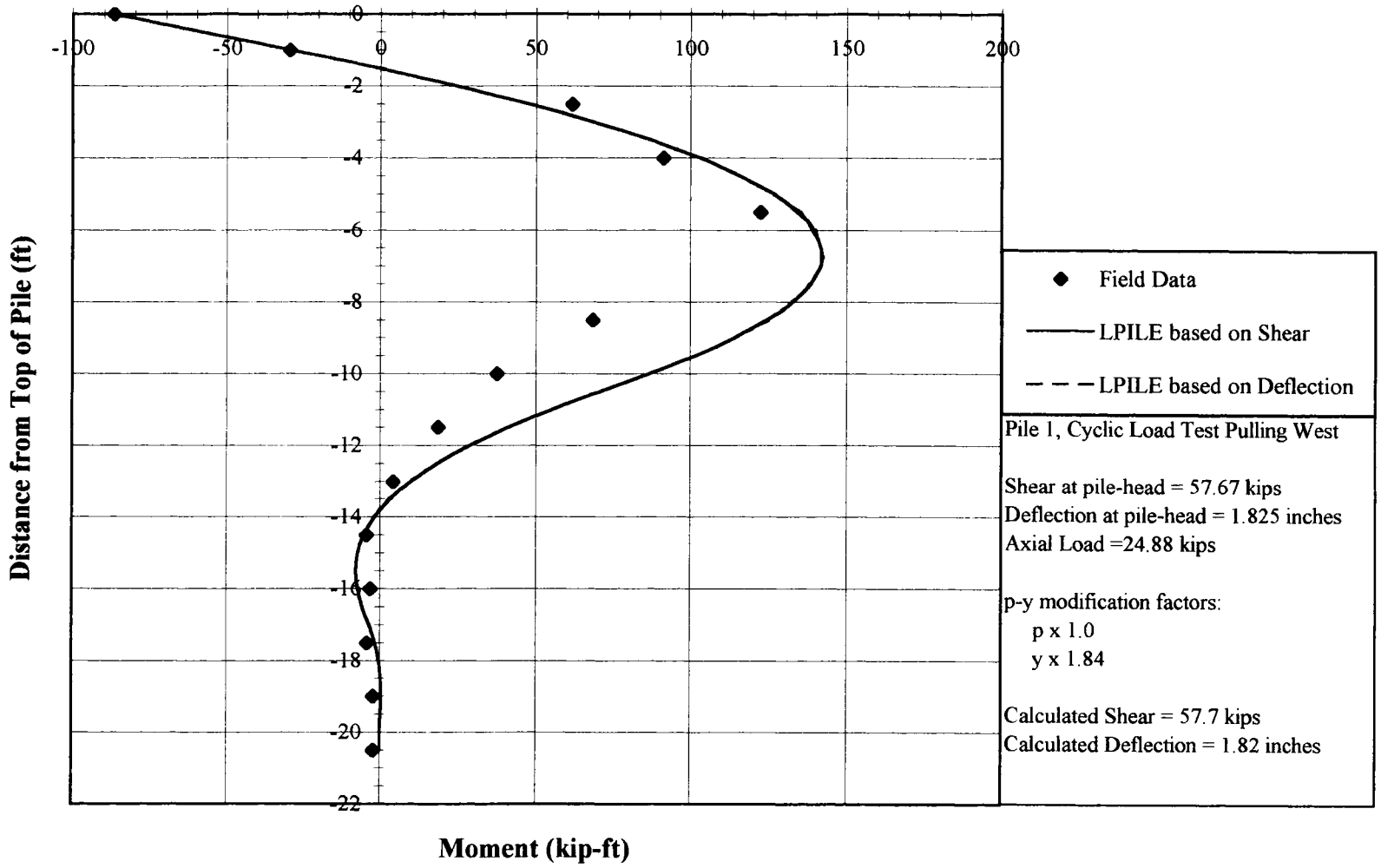


Figure A-11. Moment vs. Distance from Top of Pile Comparison Between LPILE Output and September 8, 1997 Field Test.

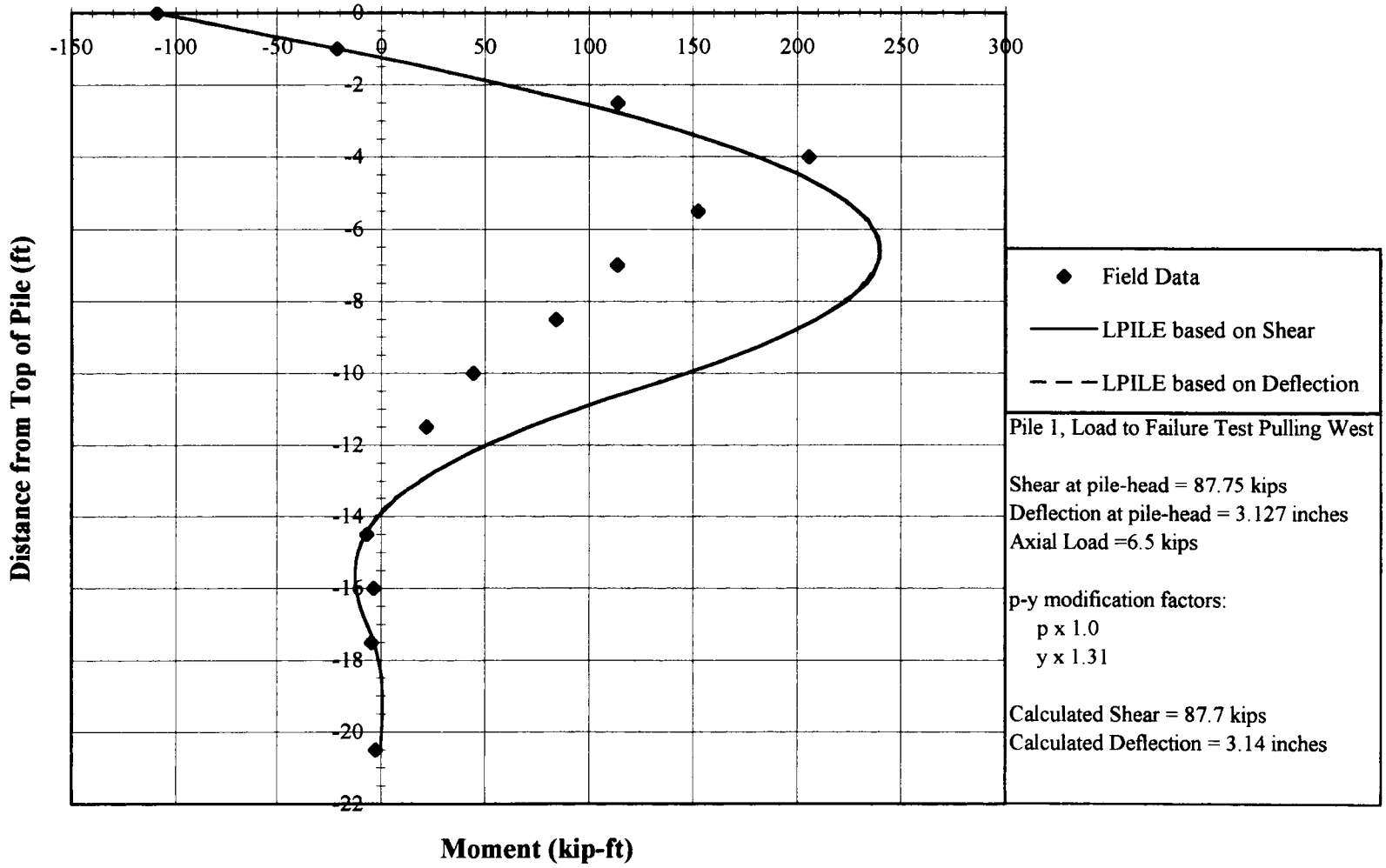


Figure A-12. Moment vs. Distance from Top of Pile Comparison Between LPILE Output and September 12, 1997 Field Test.

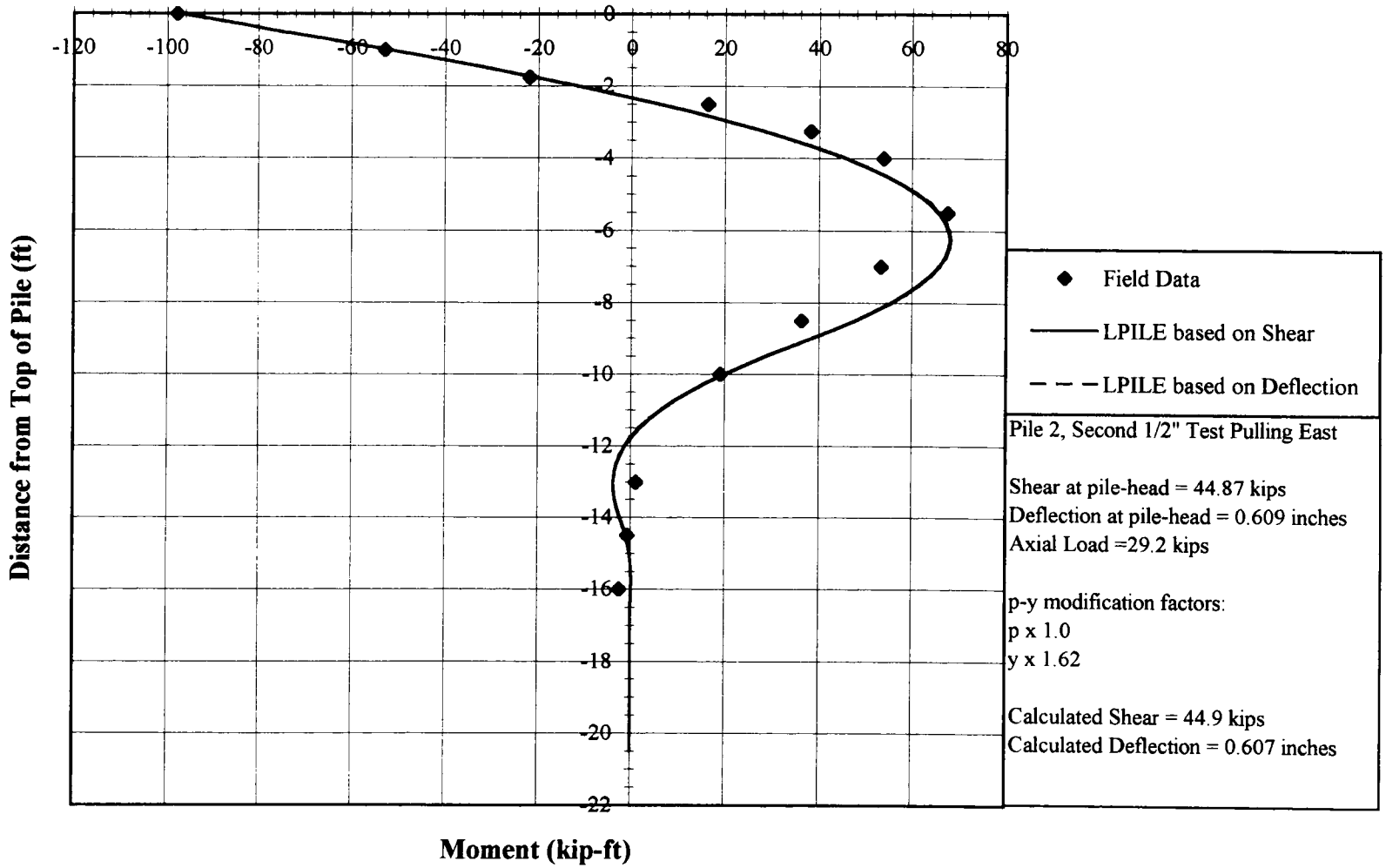


Figure A-13. Moment vs. Distance from Top of Pile Comparison Between LPILE Output and January 30, 1998 Field Test.

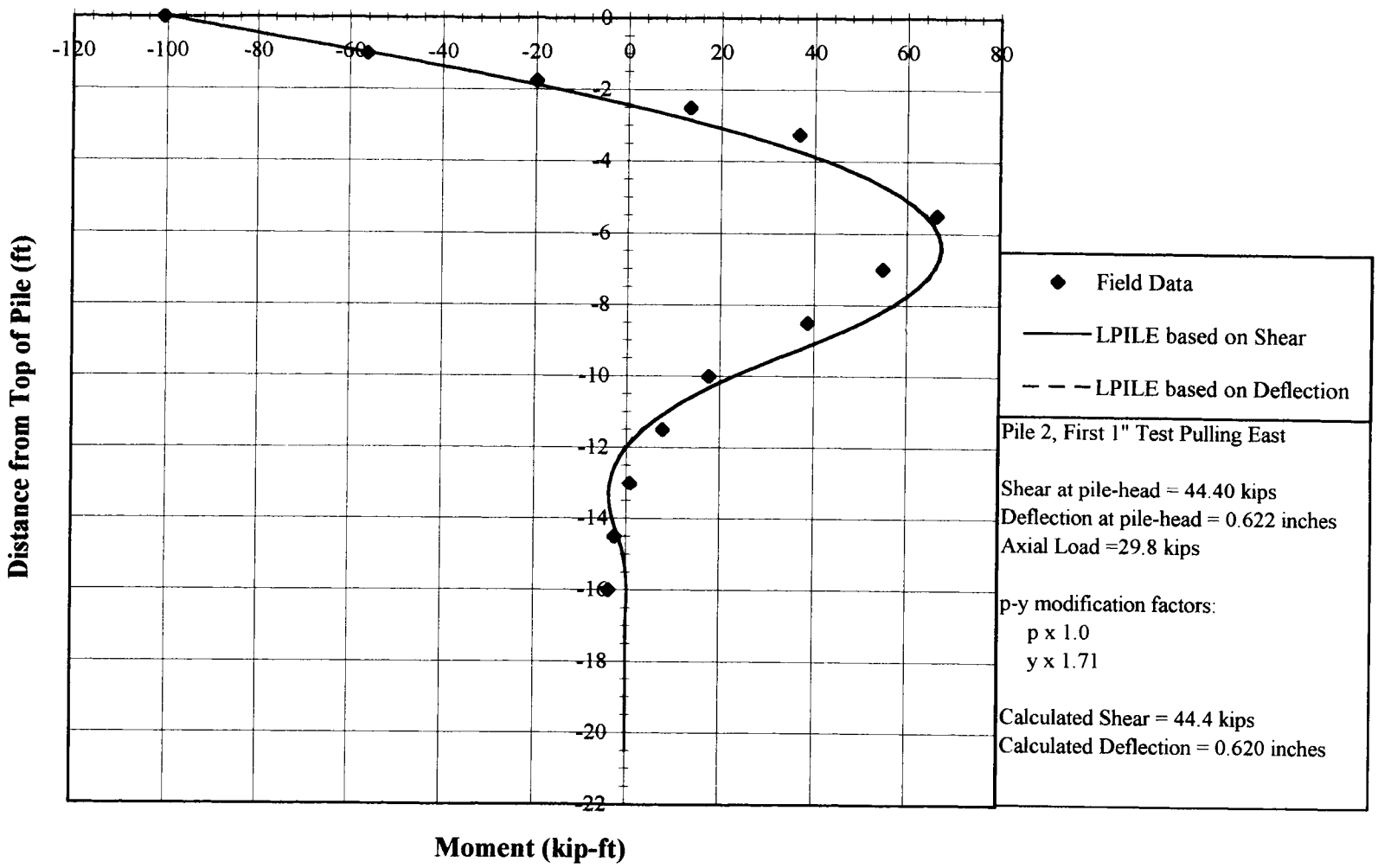


Figure A-14. First Moment vs. Distance from Top of Pile Comparison Between LPILE Output and February 13, 1998 Field Test.

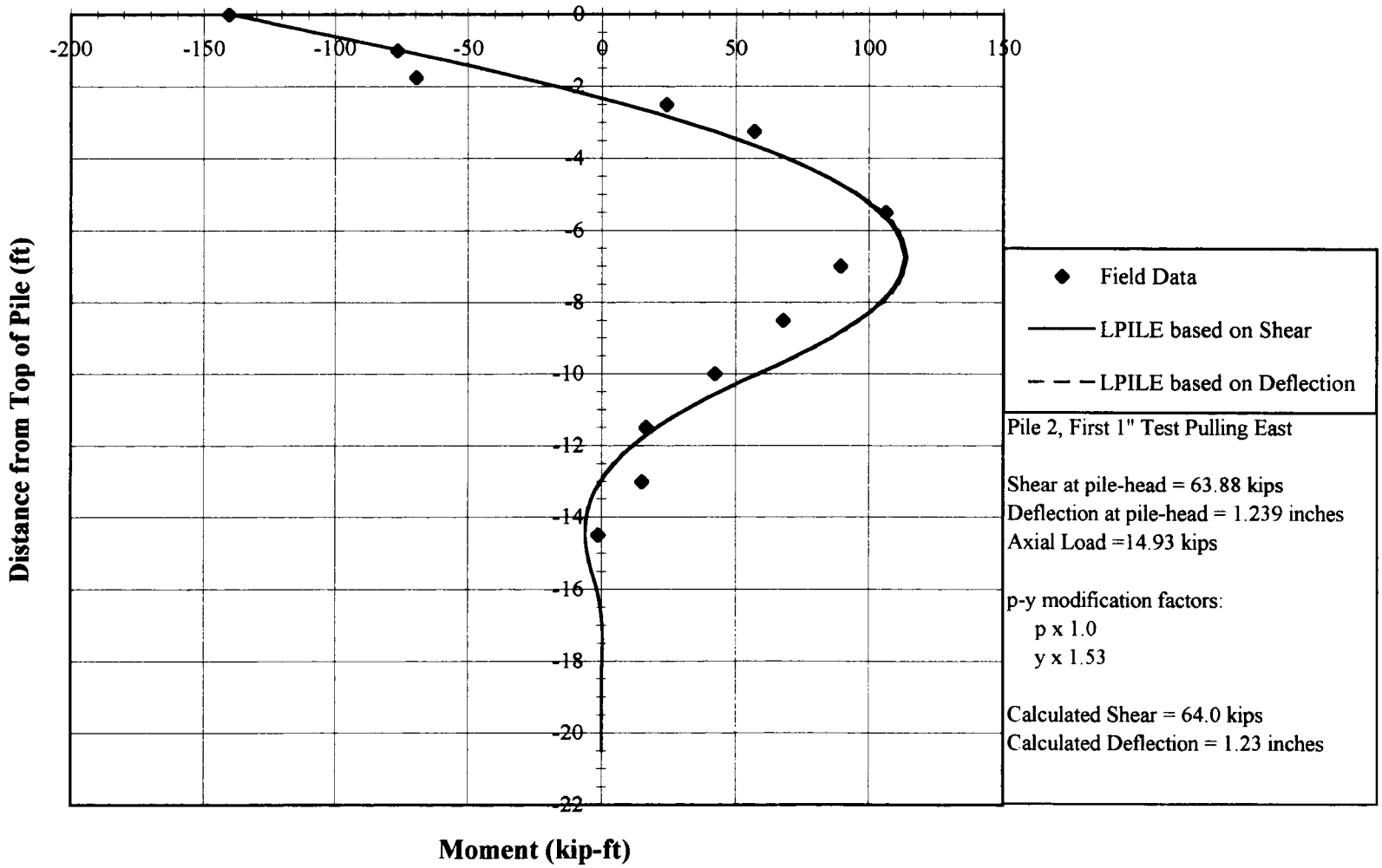


Figure A-15. Second Moment vs. Distance from Top of Pile Comparison Between LPILE Output and February 13, 1998 Field Test.

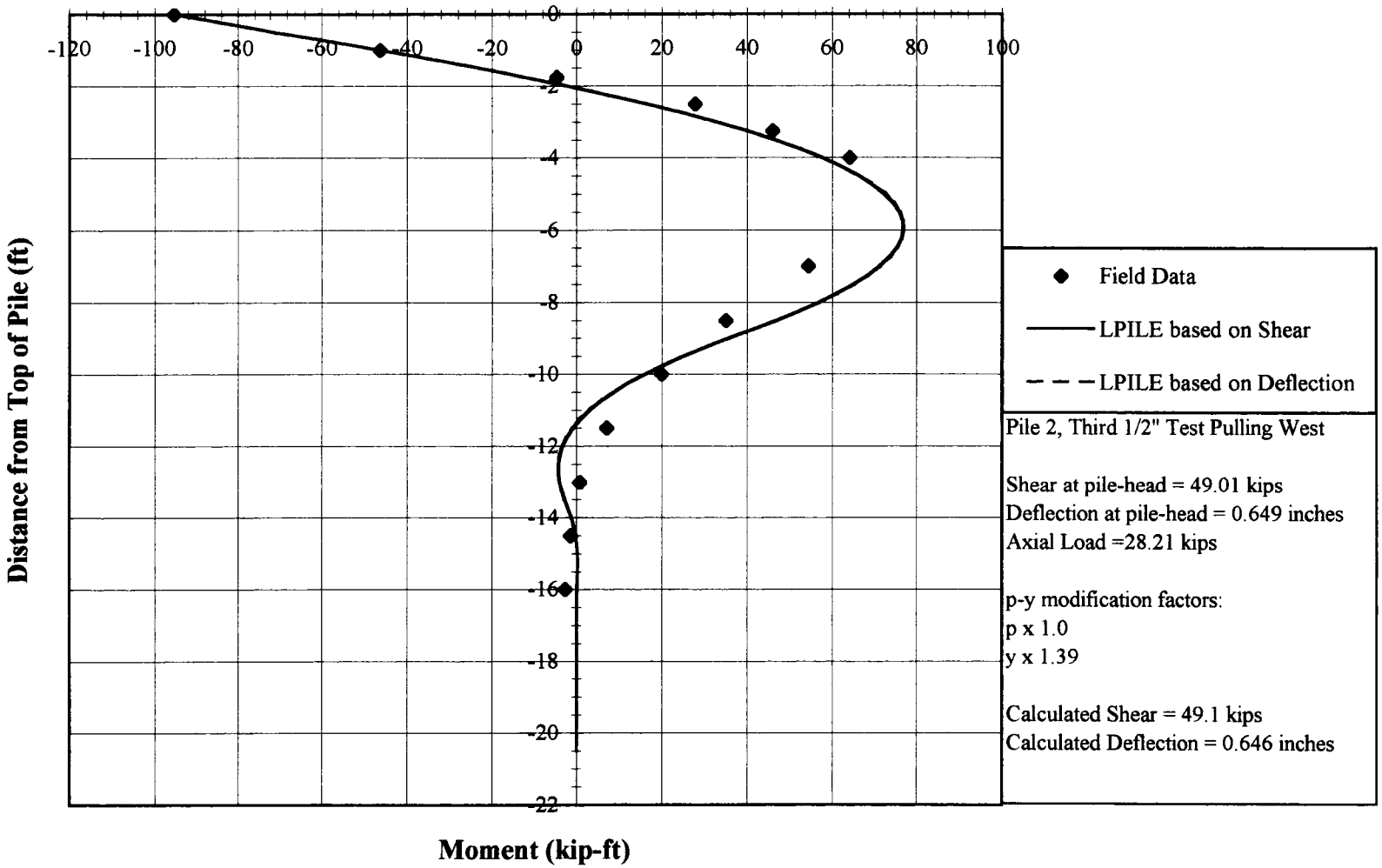


Figure A-16. Moment vs. Distance from Top of Pile Comparison Between LPILE Output and June 4, 1998 Field Test.

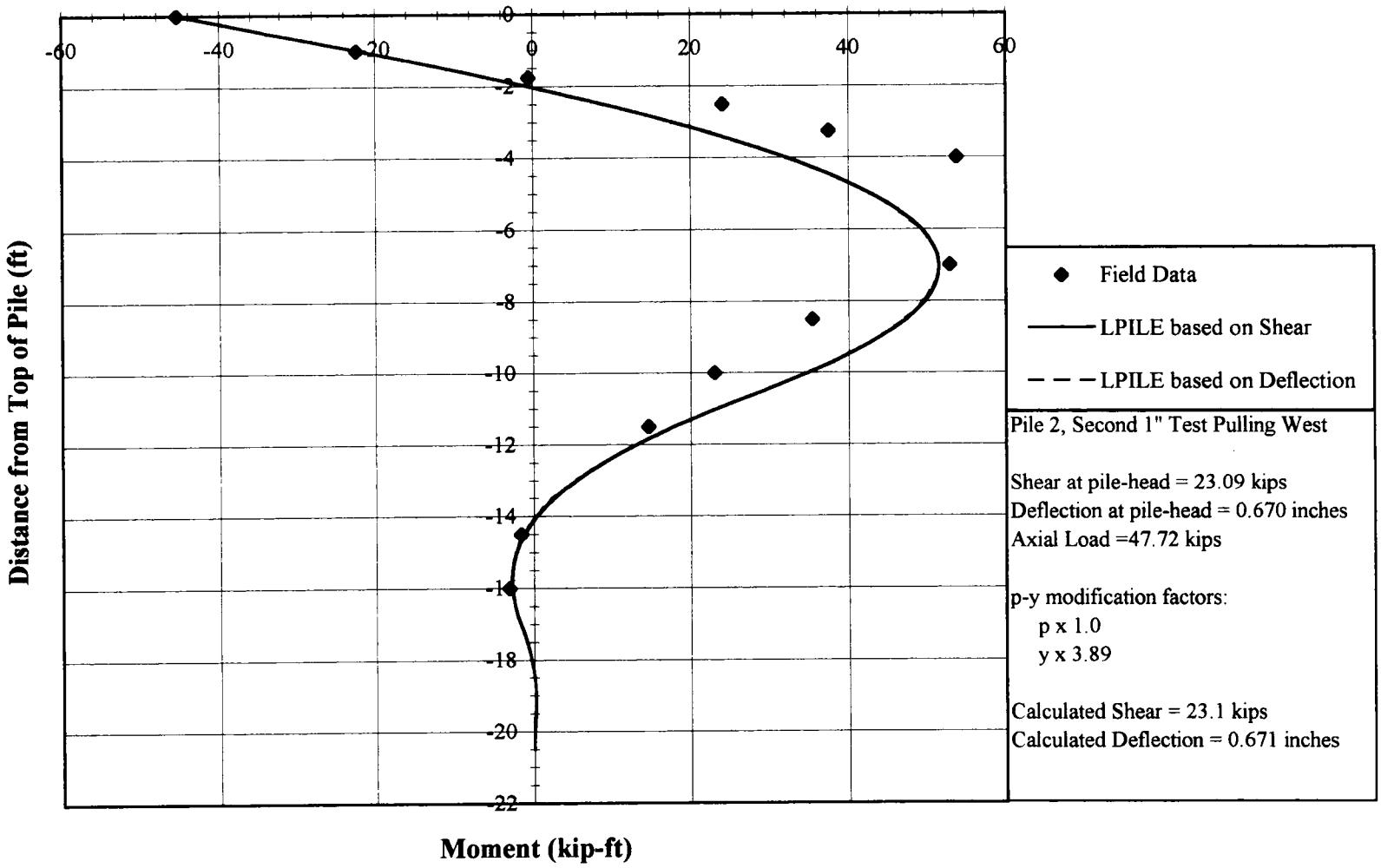


Figure A-17. First Moment vs. Distance from Top of Pile Comparison Between LPILE Output and June 26, 1998 Field Test.

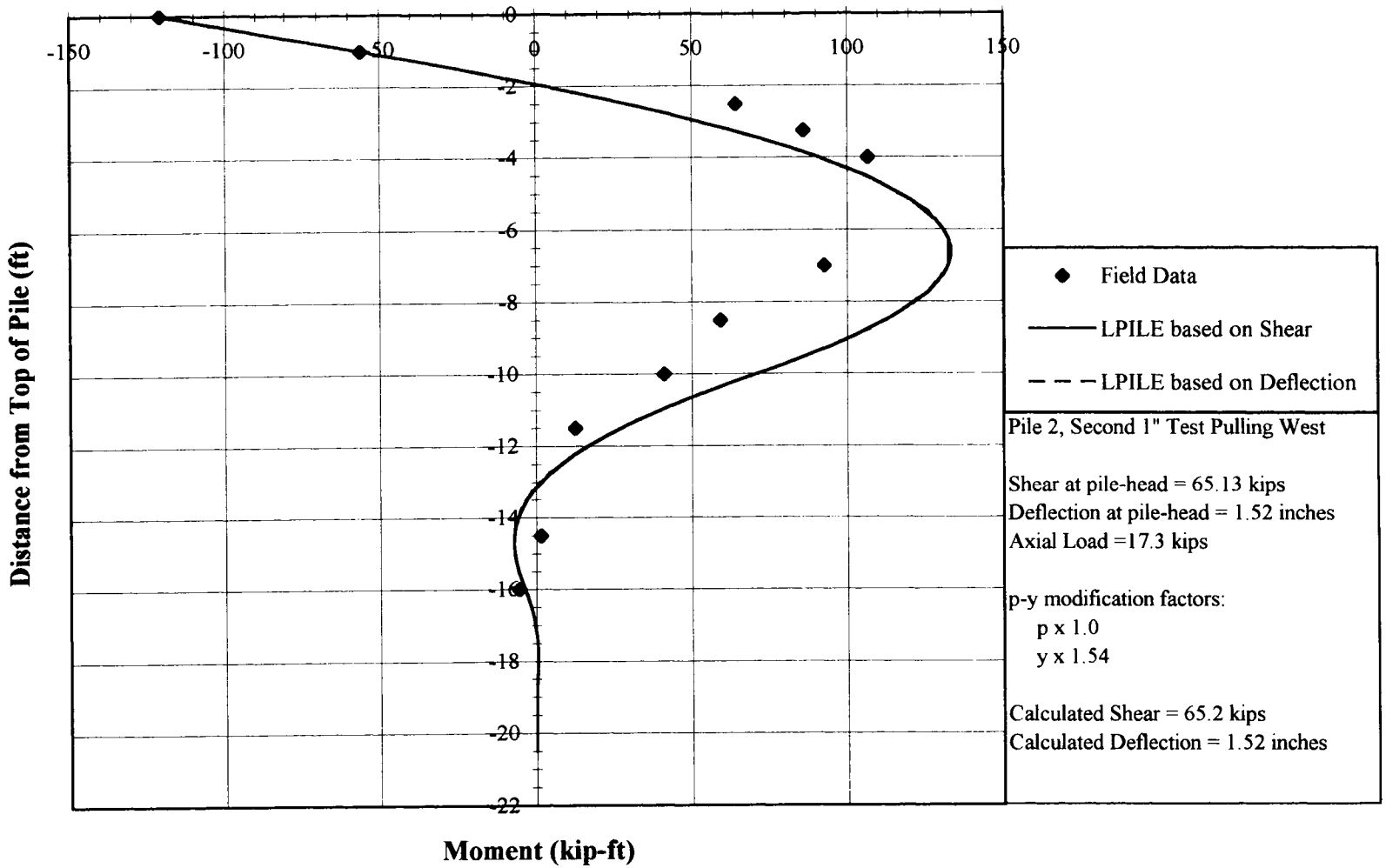


Figure A-18. Second Moment vs. Distance from Top of Pile Comparison Between LPILE Output and June 26, 1998 Field Test.

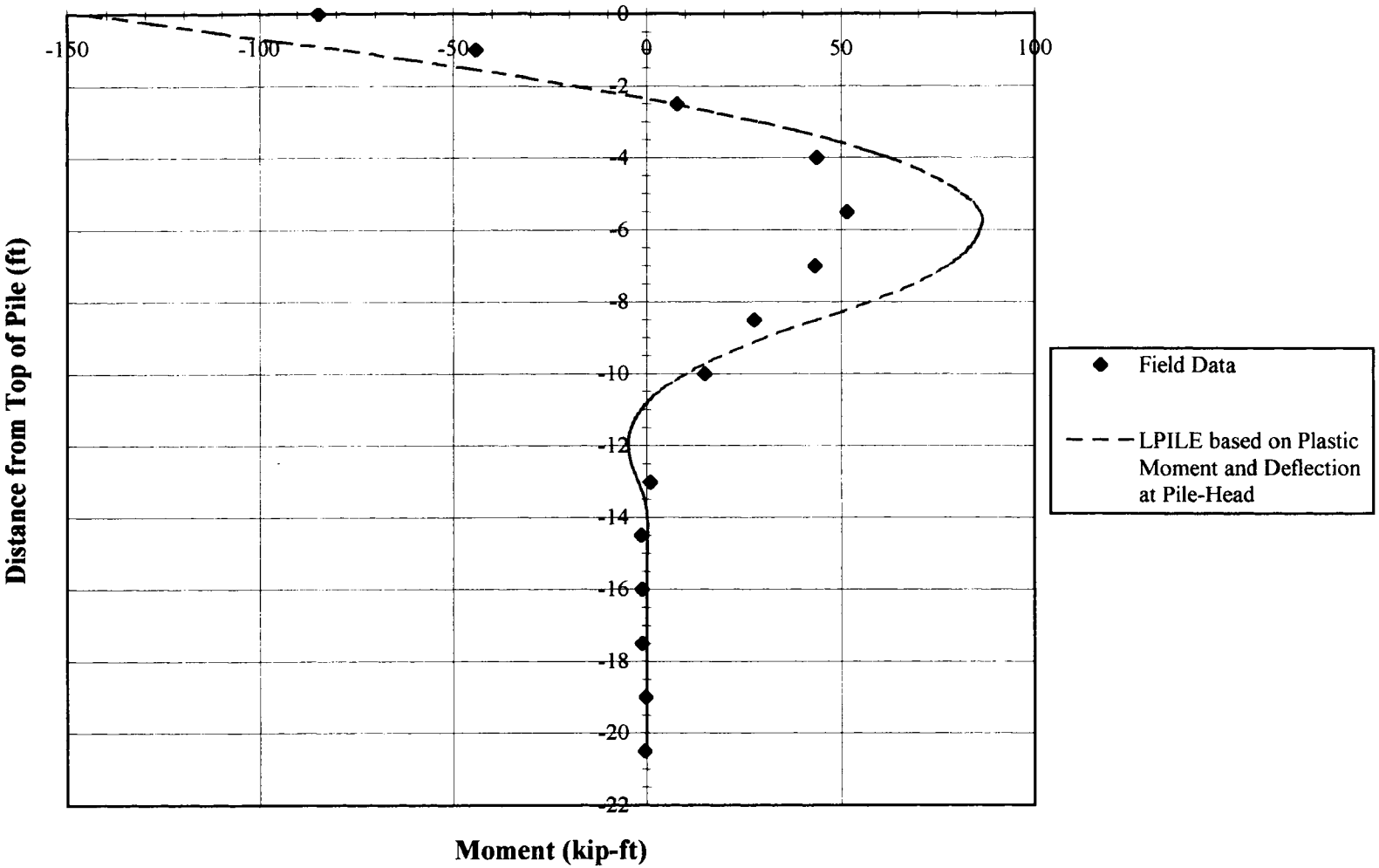


Figure A-19. Moment vs. Distance from Top of Pile Comparison between June 24, 1997 Field Data and LPILE using Plastic Moment and Pile-Head Deflection (0.645").

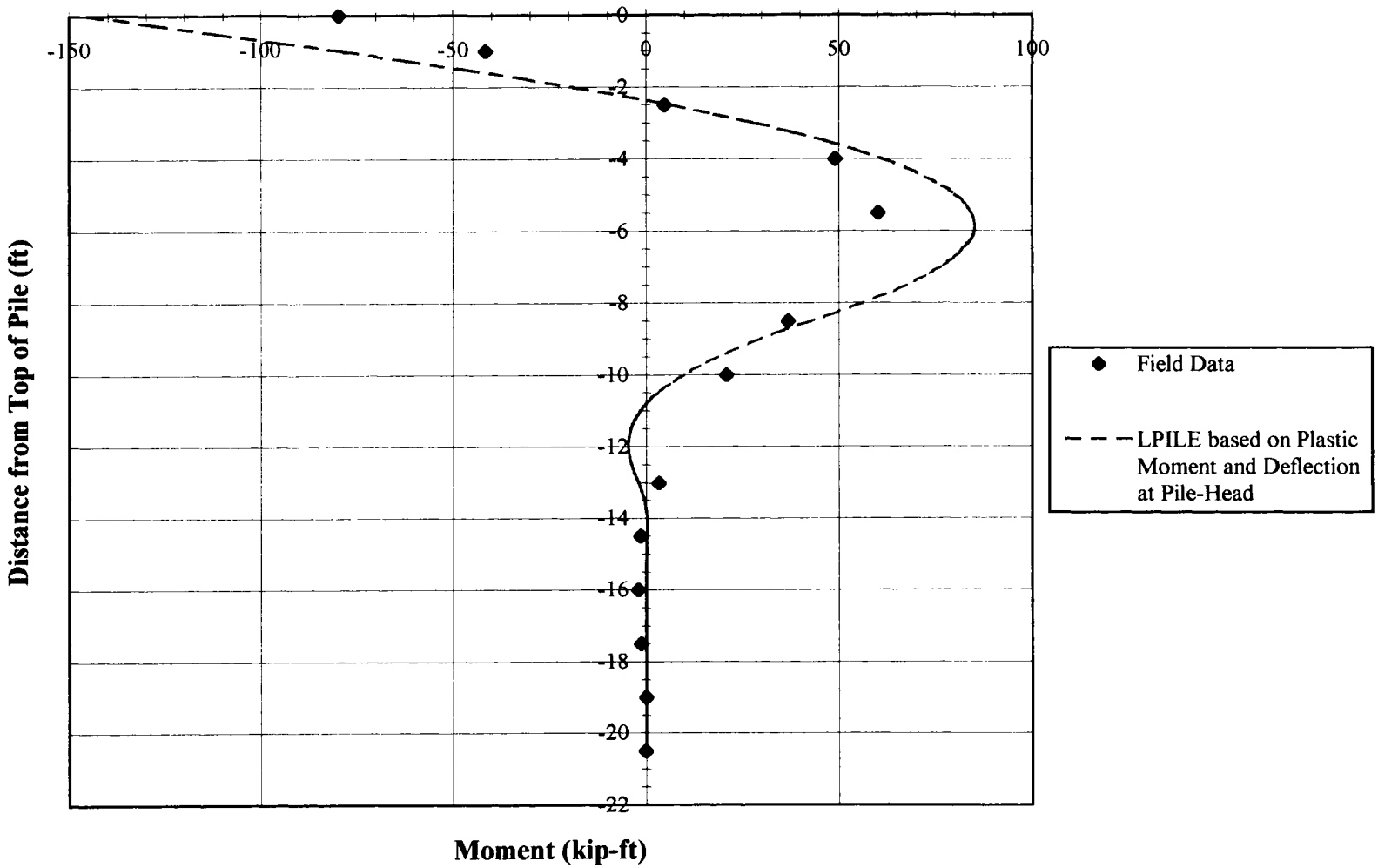


Figure A-20. Moment vs. Distance from Top of Pile Comparison between July 24, 1997 Field Data and LPILE using Plastic Moment and Pile-Head Deflection (0.631").

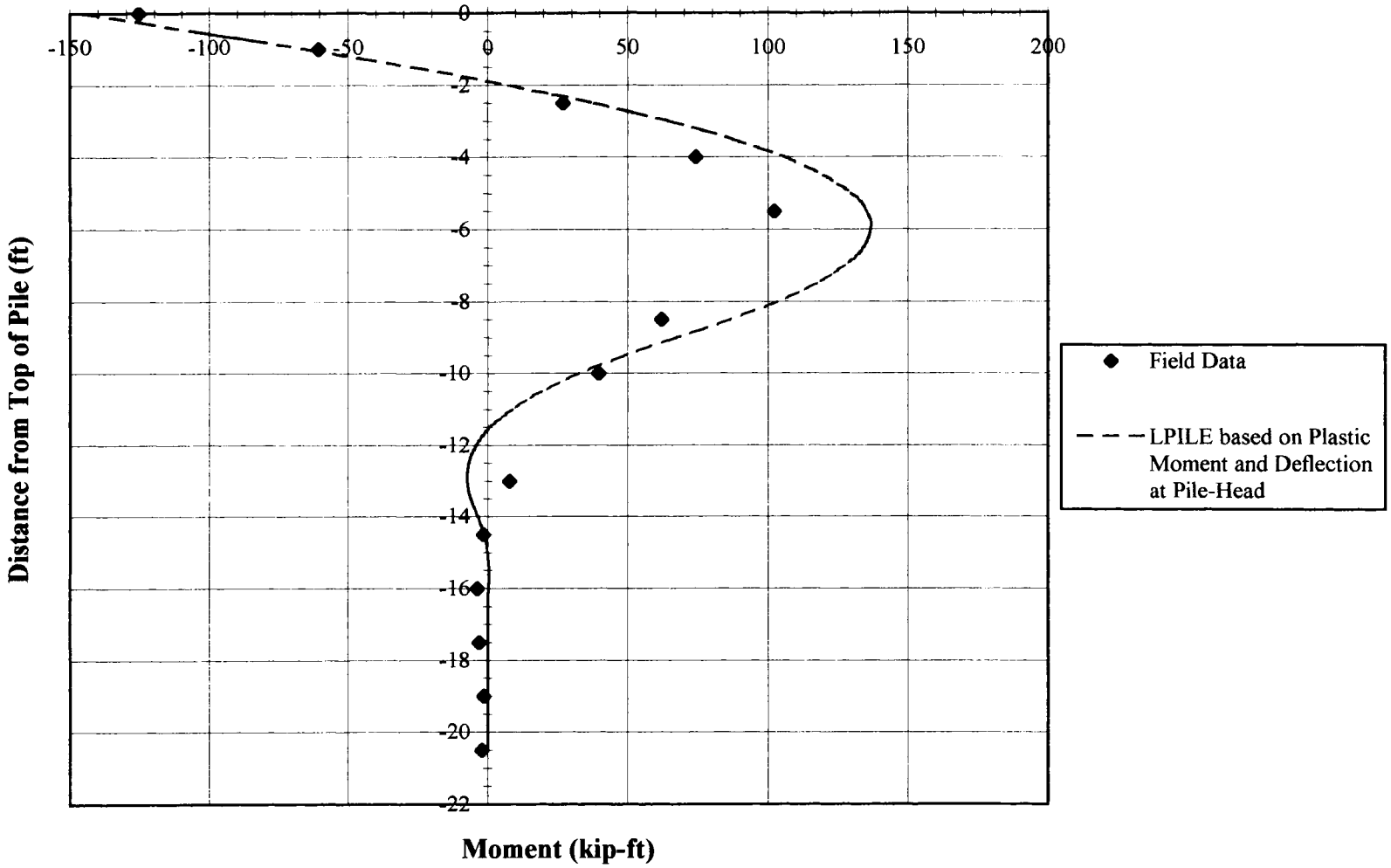


Figure A-21. Moment vs. Distance from Top of Pile Comparison between July 24, 1997 Field Data and LPILE using Plastic Moment and Pile-Head Deflection (1.223").

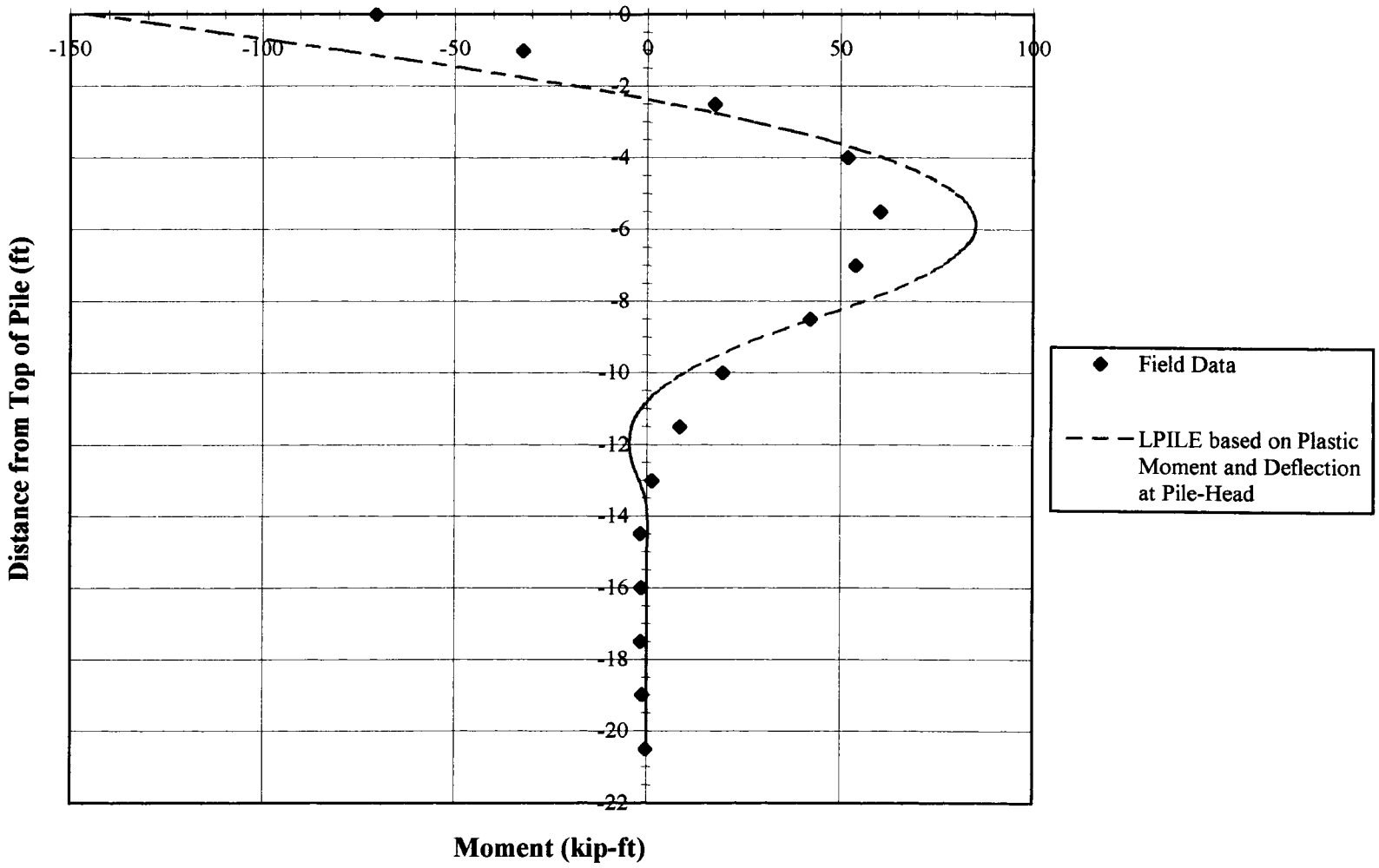


Figure A-22. Moment vs. Distance from Top of Pile Comparison between August 15, 1997 Field Data and LPILE using Plastic Moment and Pile-Head Deflection (0.631").

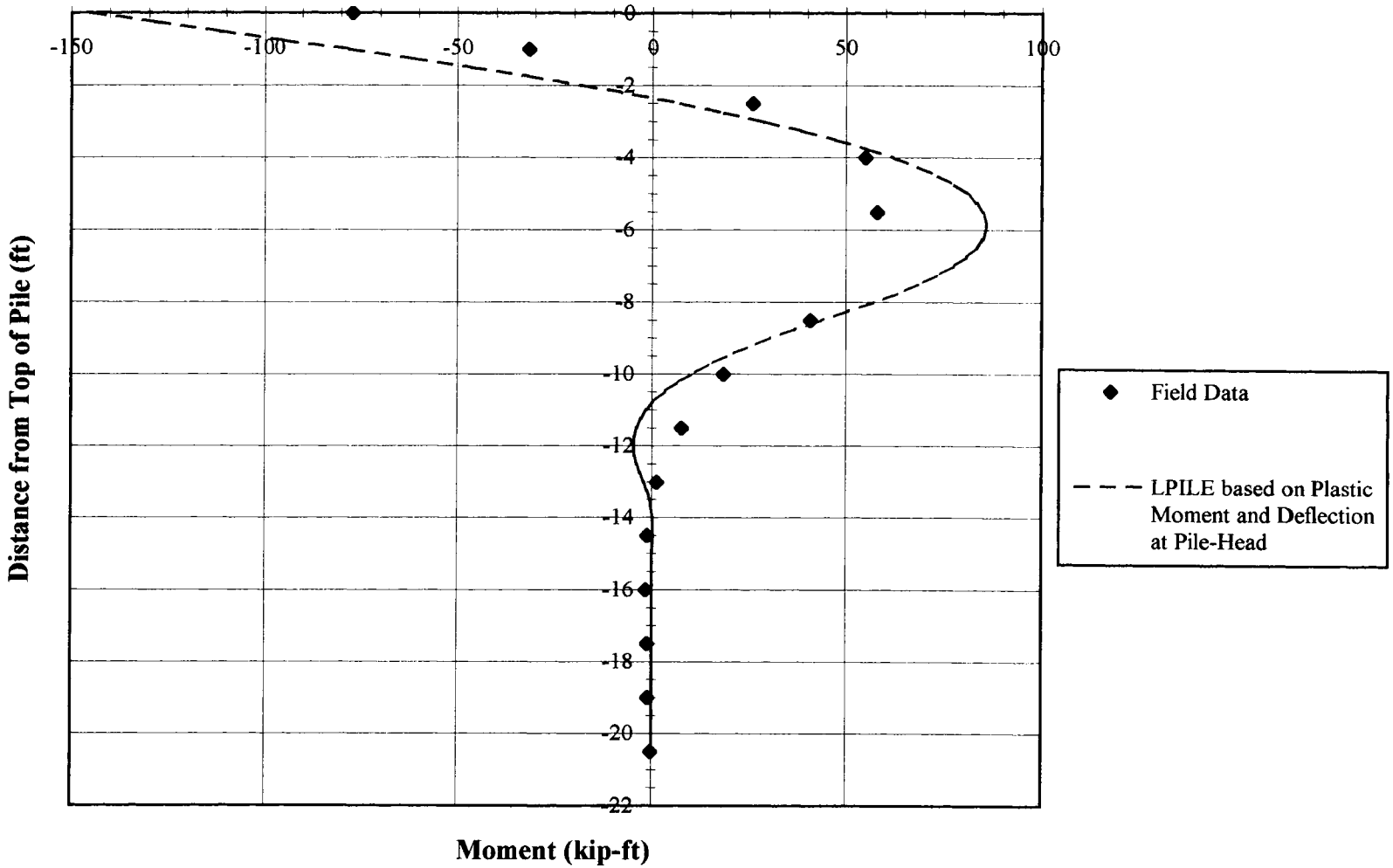


Figure A-23. Moment vs. Distance from Top of Pile Comparison between August 25, 1997 Field Data and LPILE using Plastic Moment and Pile-Head Deflection (0.643").

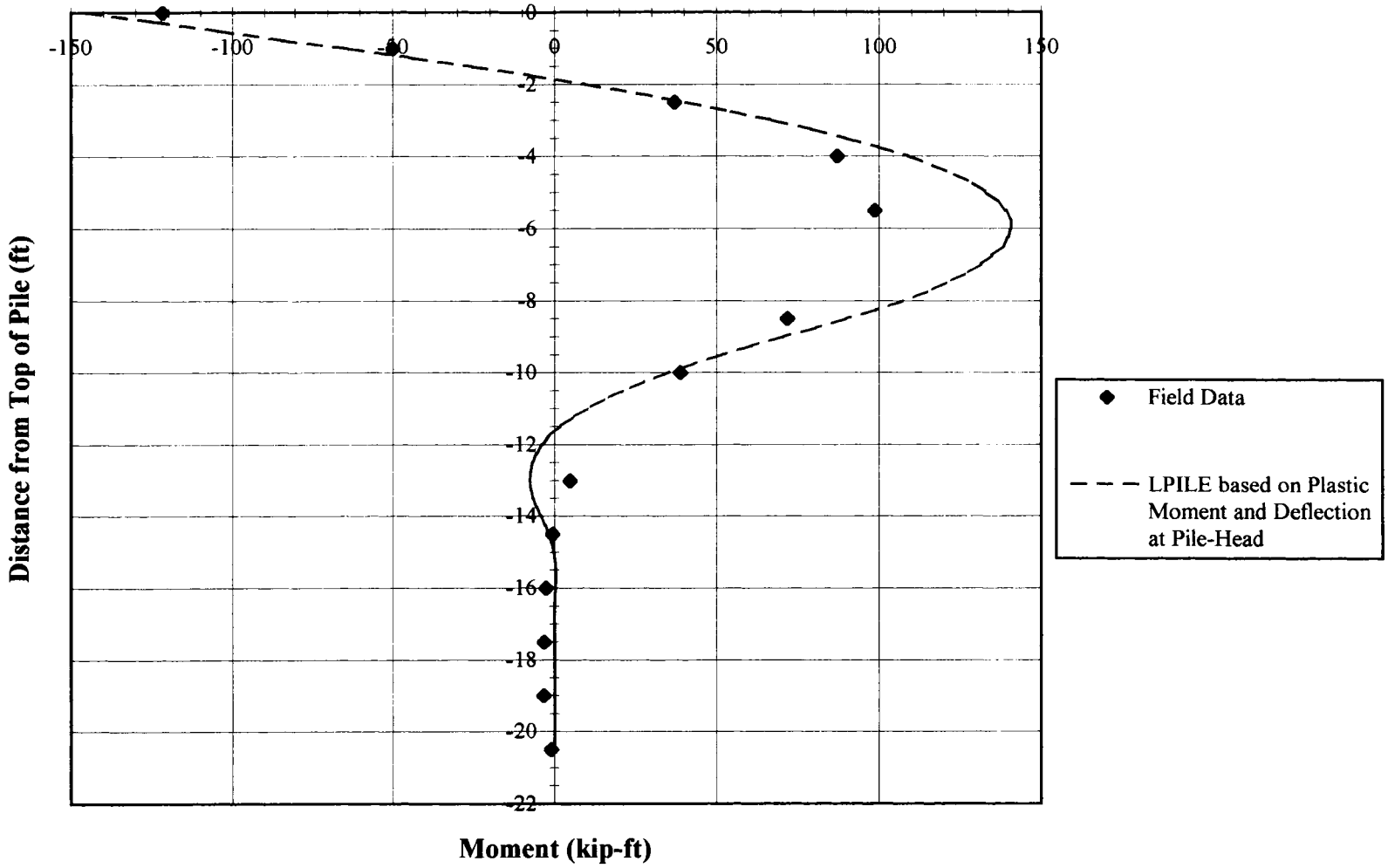


Figure A-24. Moment vs. Distance from Top of Pile Comparison between August 25, 1997 Field Data and LPILE using Plastic Moment and Pile-Head Deflection (1.274").

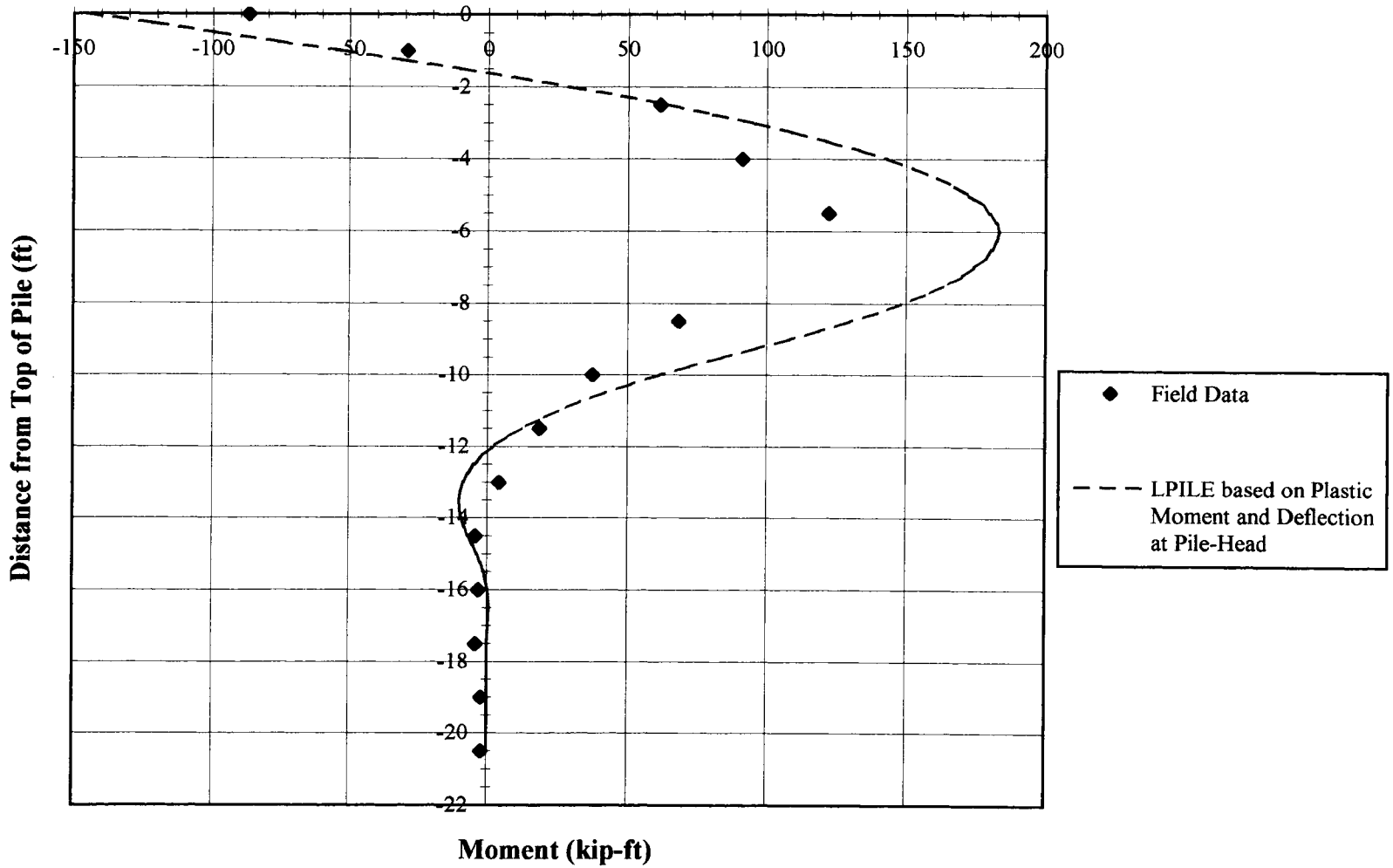


Figure A-25. Moment vs. Distance from Top of Pile Comparison between September 8, 1997 Field Data and LPILE using Plastic Moment and Pile-Head Deflection (1.825").

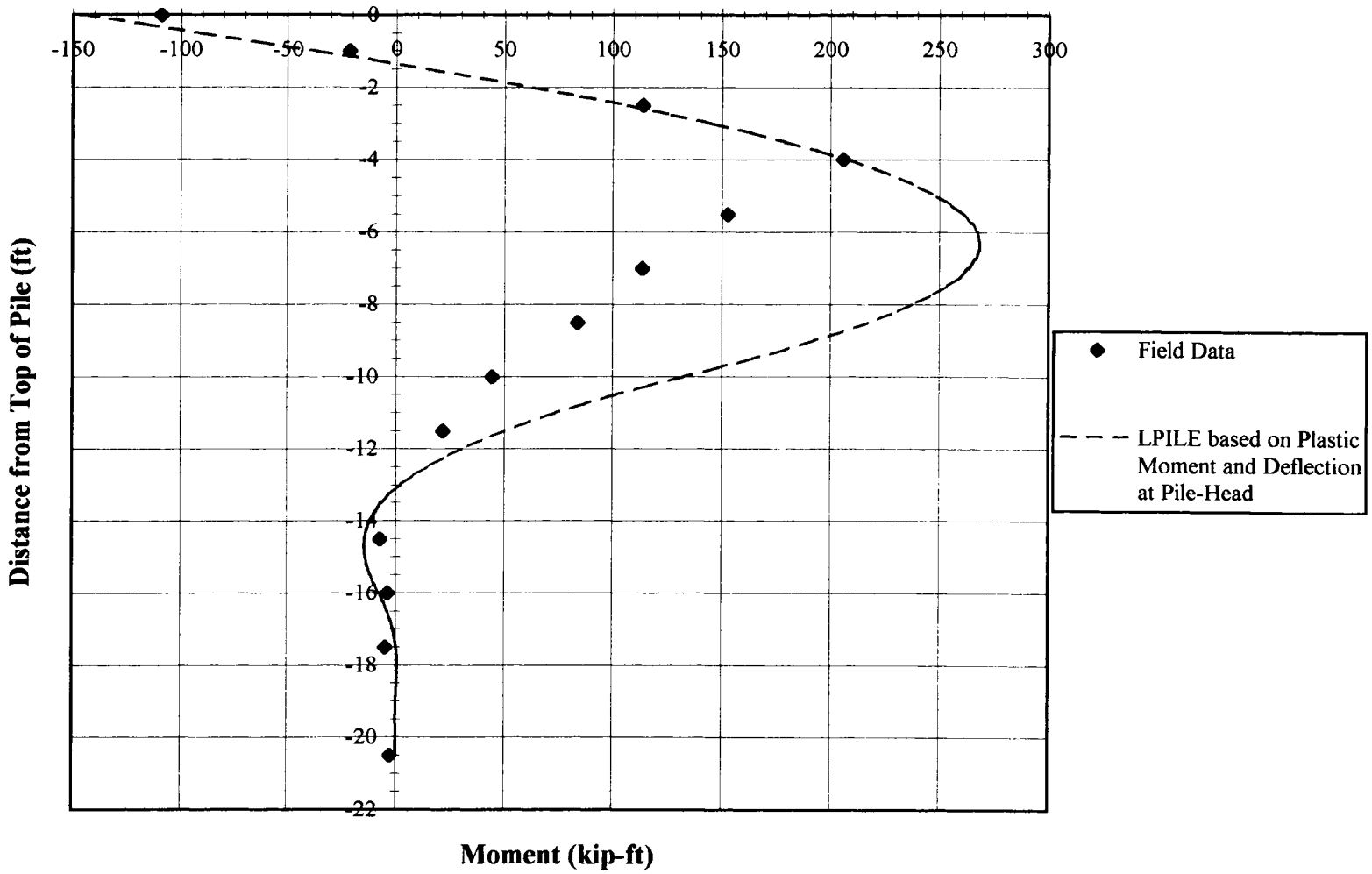


Figure A-26. Moment vs. Distance from Top of Pile Comparison between September 12, 1997 Field Data and LPILE using Plastic Moment and Pile-Head Deflection (3.127").

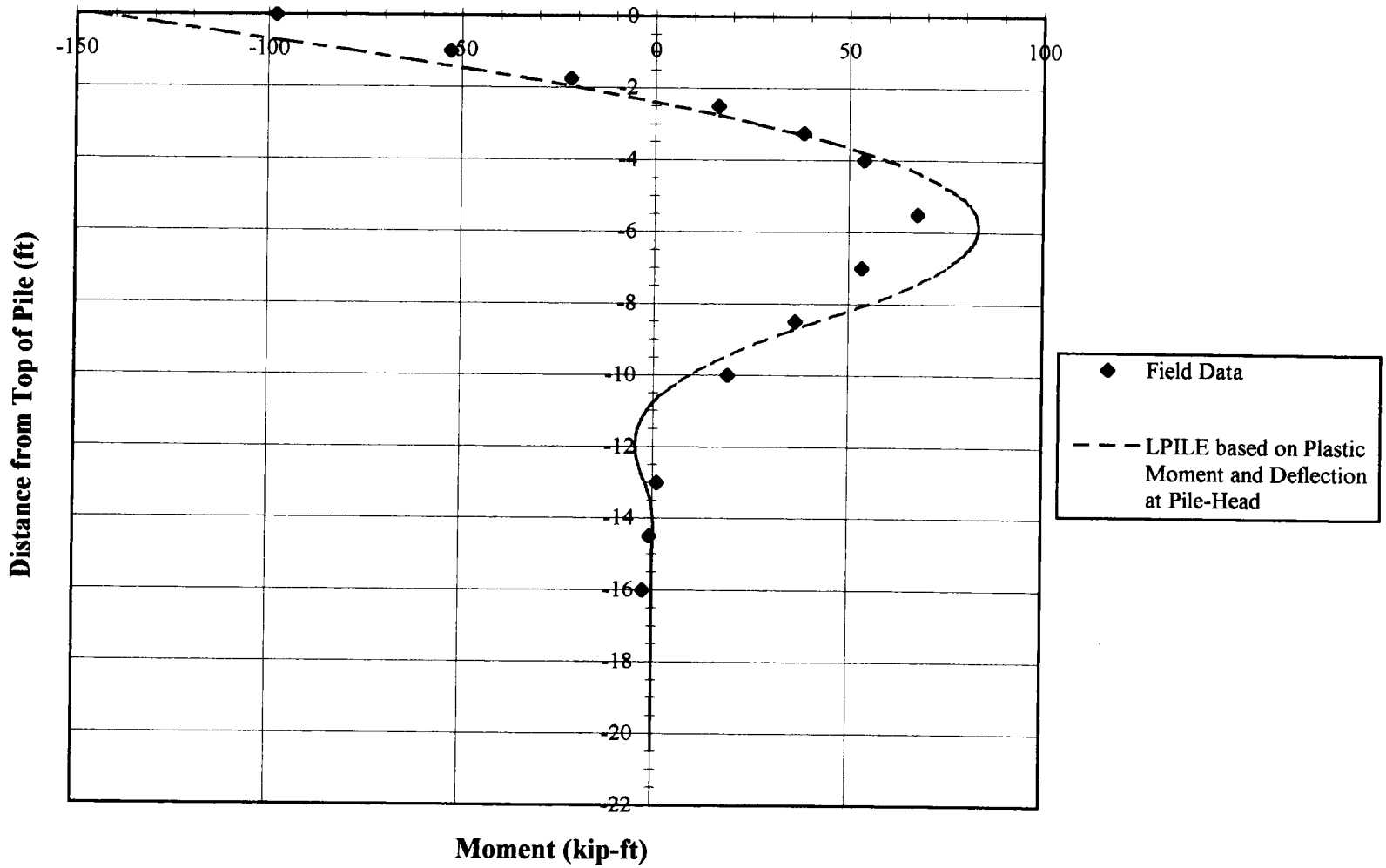


Figure A-27. Moment vs. Distance from Top of Pile Comparison between January 30, 1998 Field Data and LPILE using Plastic Moment and Pile-Head Deflection (0.609").

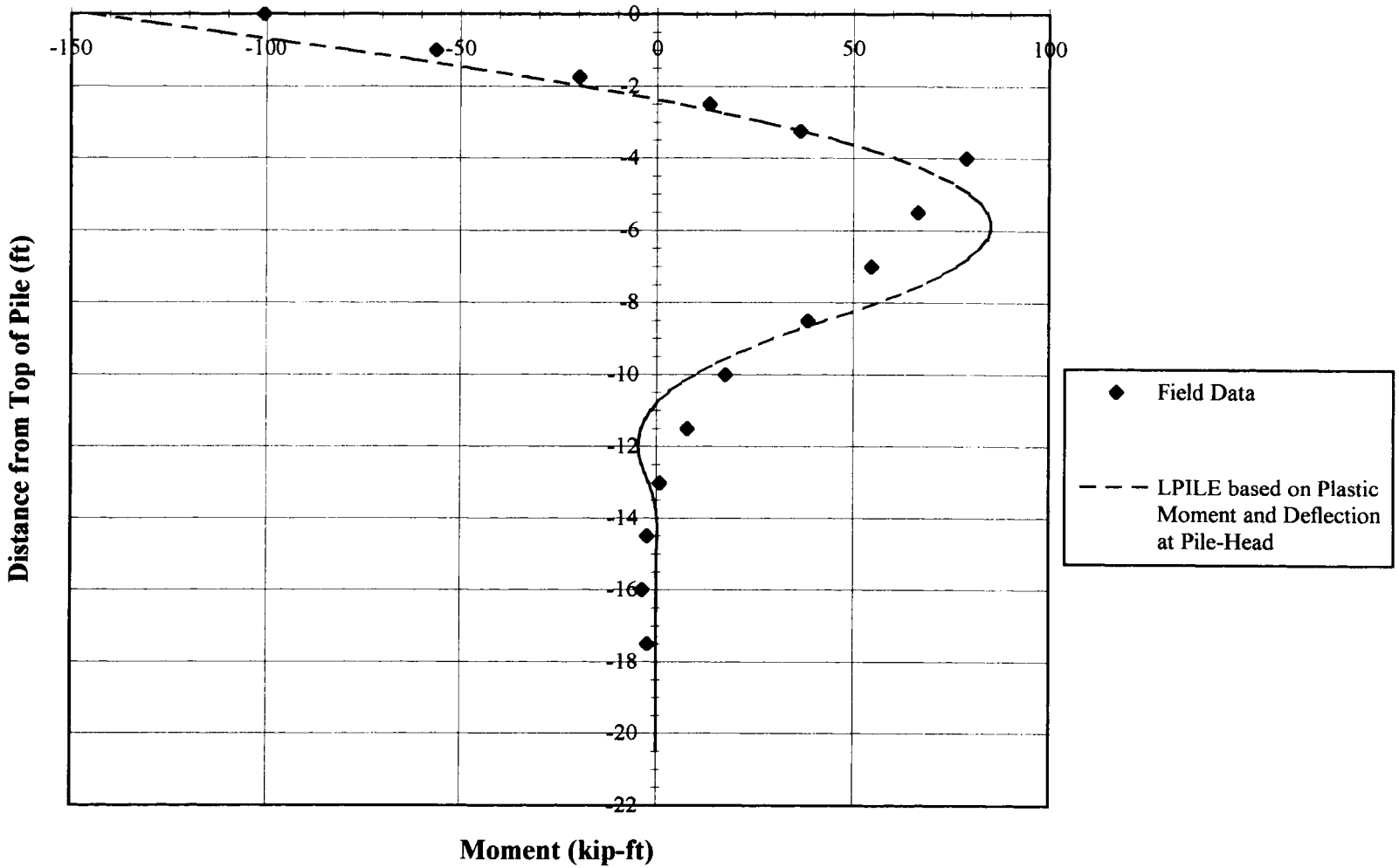


Figure A-28. Moment vs. Distance from Top of Pile Comparison between February 13, 1998 Field Data and LPILE using Plastic Moment and Pile-Head Deflection (0.622").

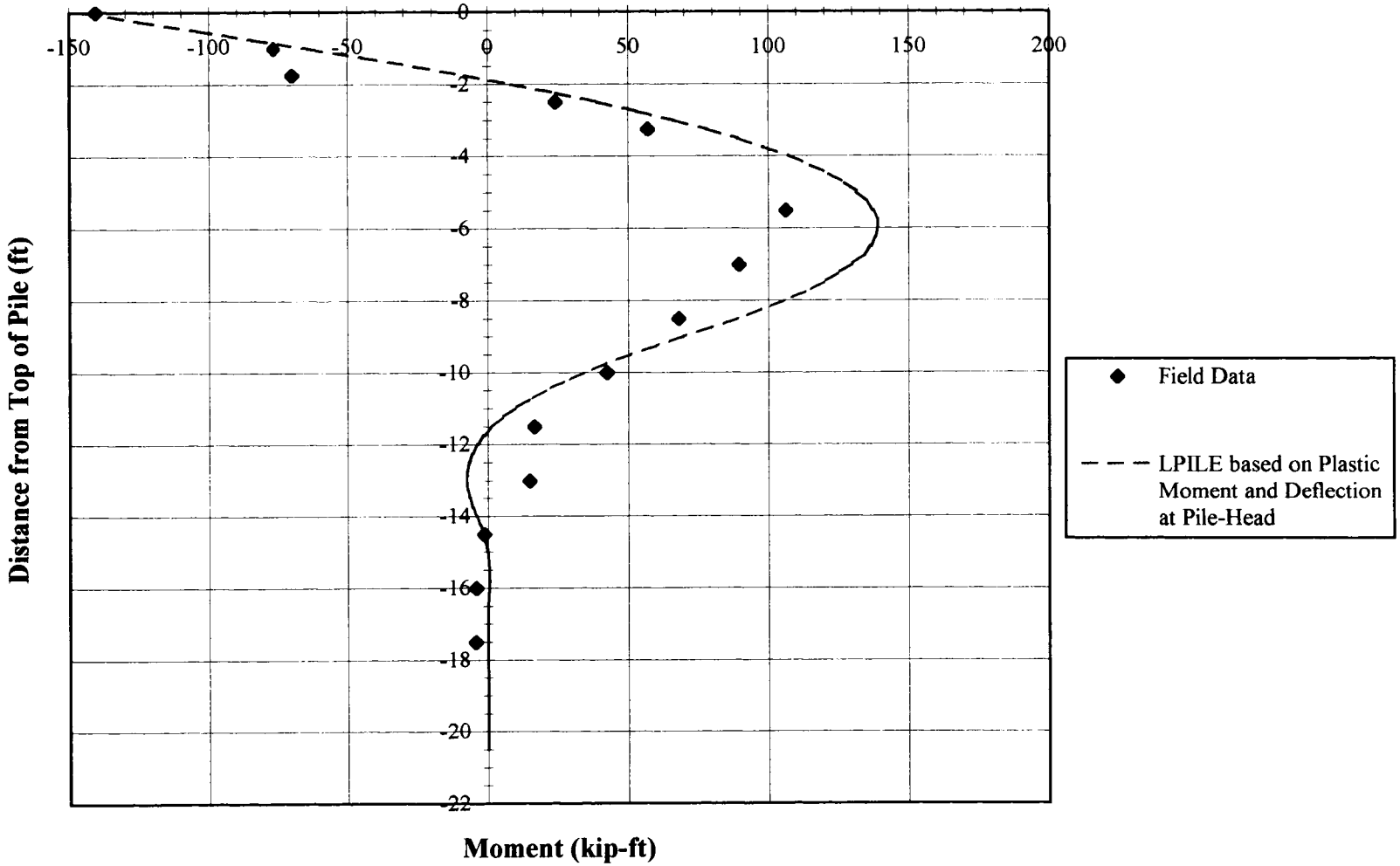


Figure A-29. Moment vs. Distance from Top of Pile Comparison between February 13, 1998 Field Data and LPILE using Plastic Moment and Pile-Head Deflection (1.239").

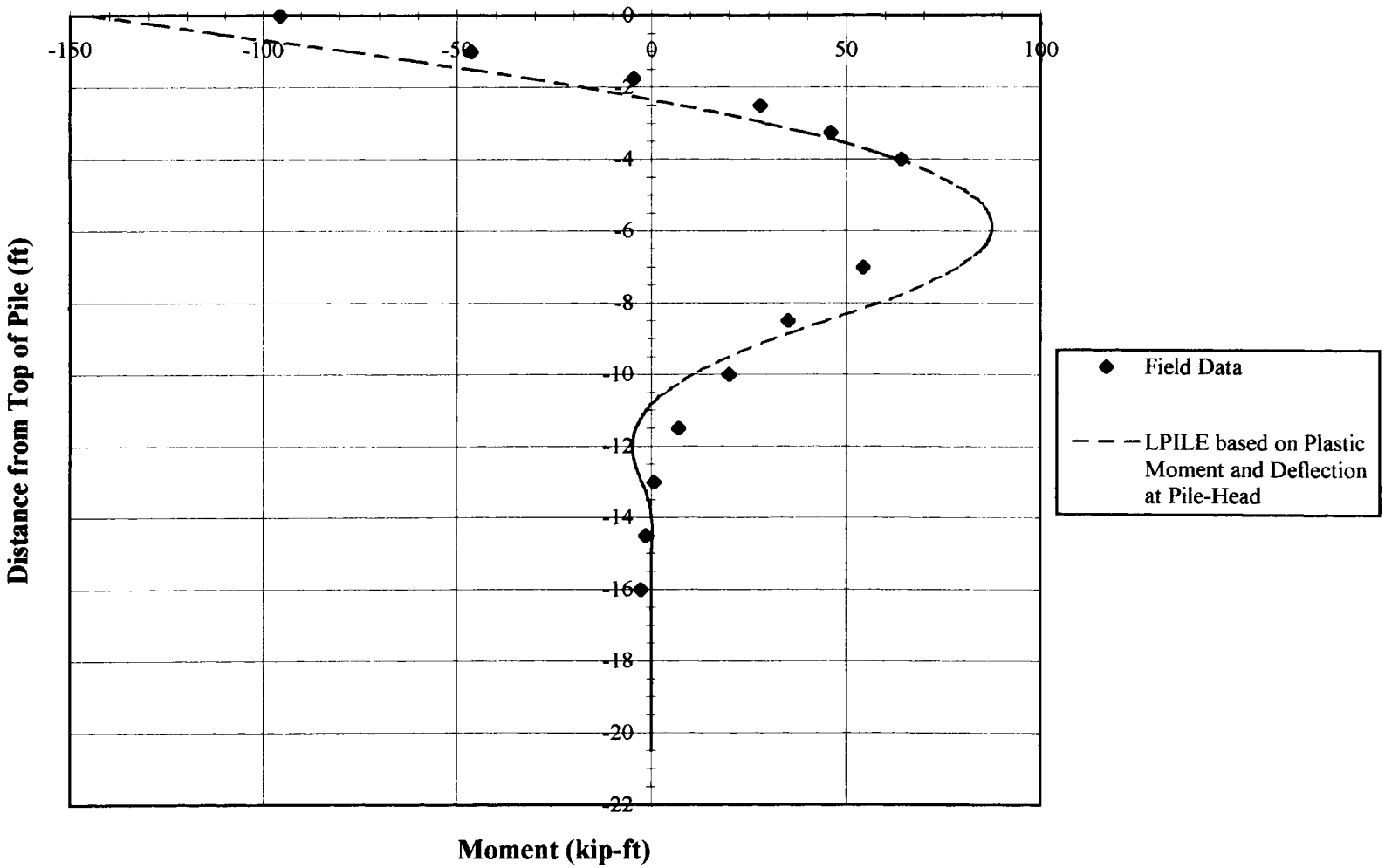


Figure A-30. Moment vs. Distance from Top of Pile Comparison between June 4, 1998 Field Data and LPILE using Plastic Moment and Pile-Head Deflection (0.649").

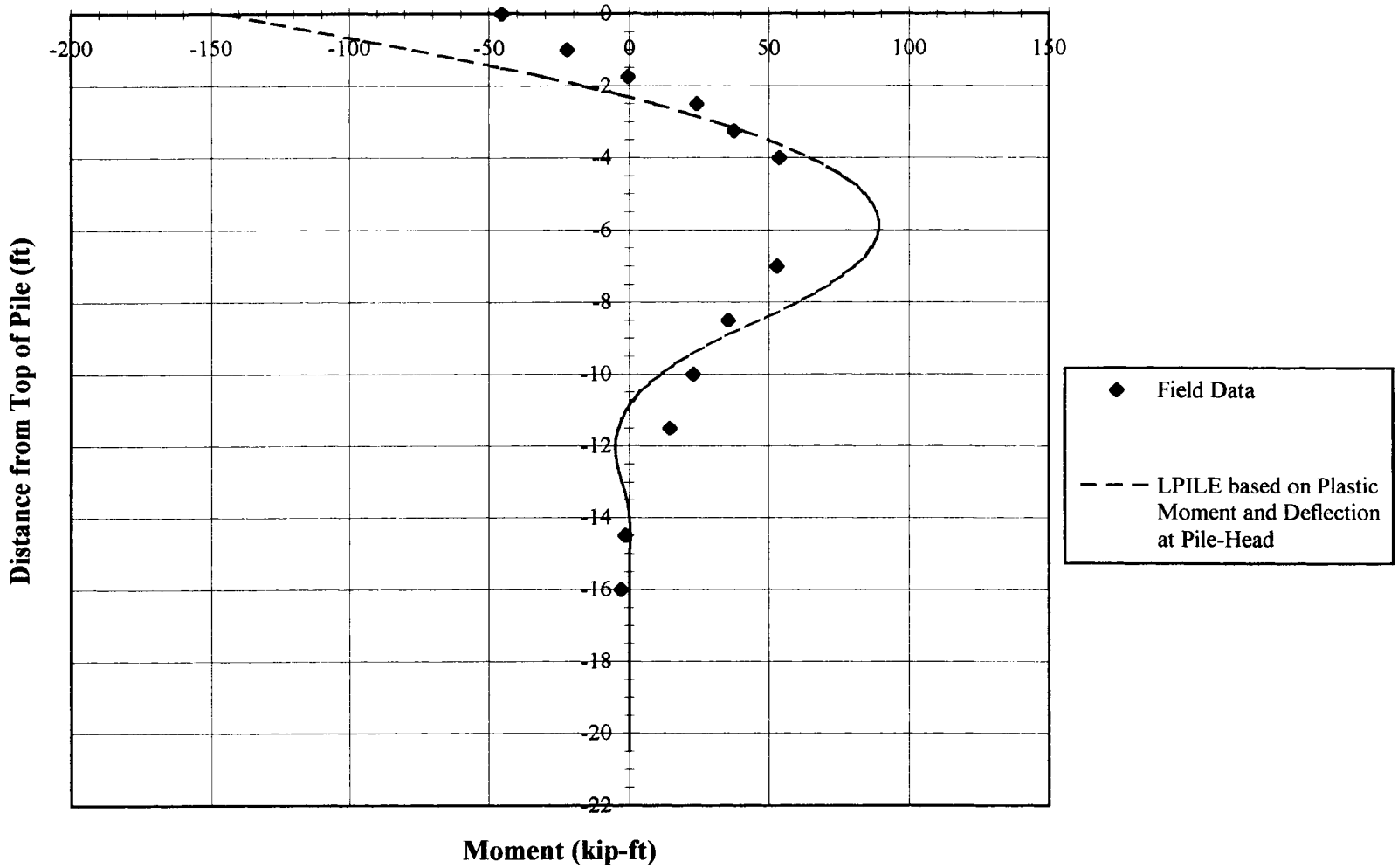


Figure A-31. Moment vs. Distance from Top of Pile Comparison between June 26, 1998 Field Data and LPILE using Plastic Moment and Pile-Head Deflection (0.670").

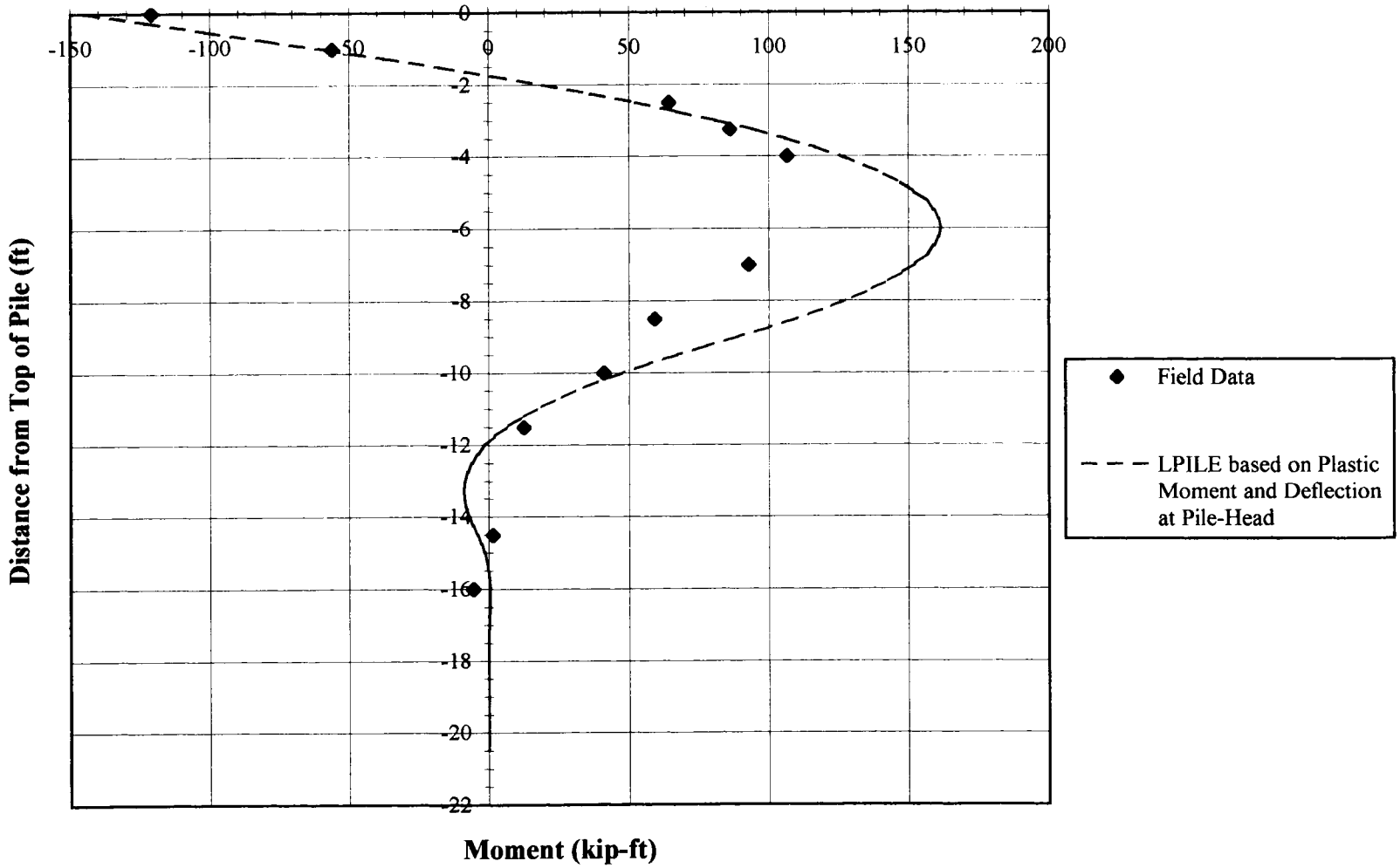


Figure A-32. Moment vs. Distance from Top of Pile Comparison between June 26, 1998 Field Data and LPILE using Plastic Moment and Pile-Head Deflection (1.52").

APPENDIX B

DETAILS OF USING LPILE

The following input screens, output screens, and brief discussion are given for the example problem below to illustrate the details of using LPILE.

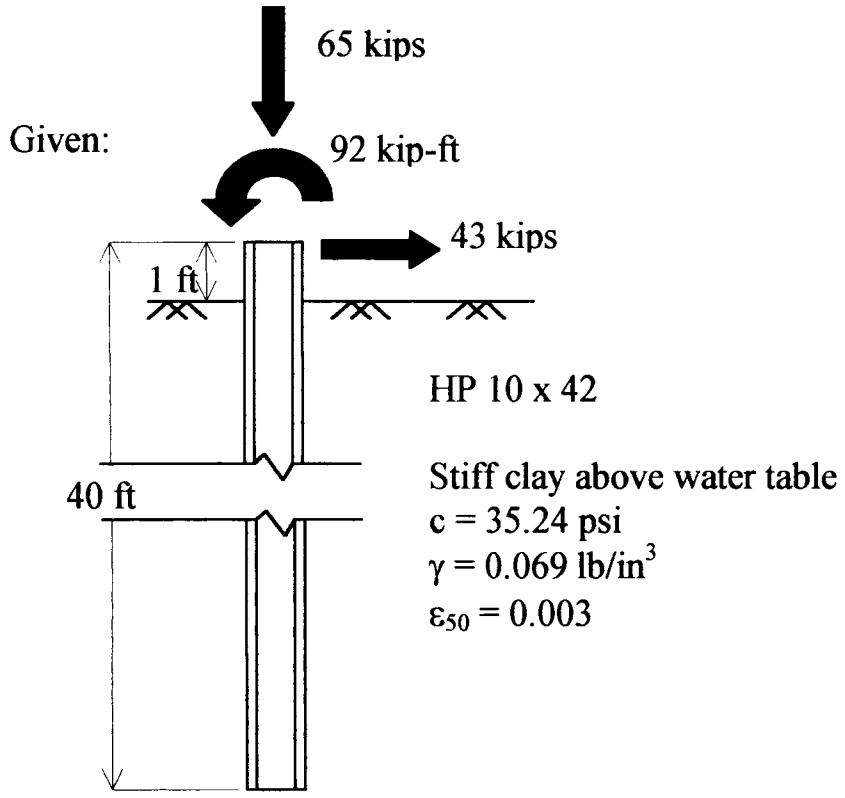


Figure B-1. Example Problem

Using LPILE is much like using any other program in Windows. When LPILE is started, the following pull-down menus are available: File, Data, Options, Computation, Graphics, Window, and Help. The File, Window, and Help menus are essentially the same as for any other program and are not discussed here. The following pages show the options contained in the Data menu as well as the input screens for the example problem.

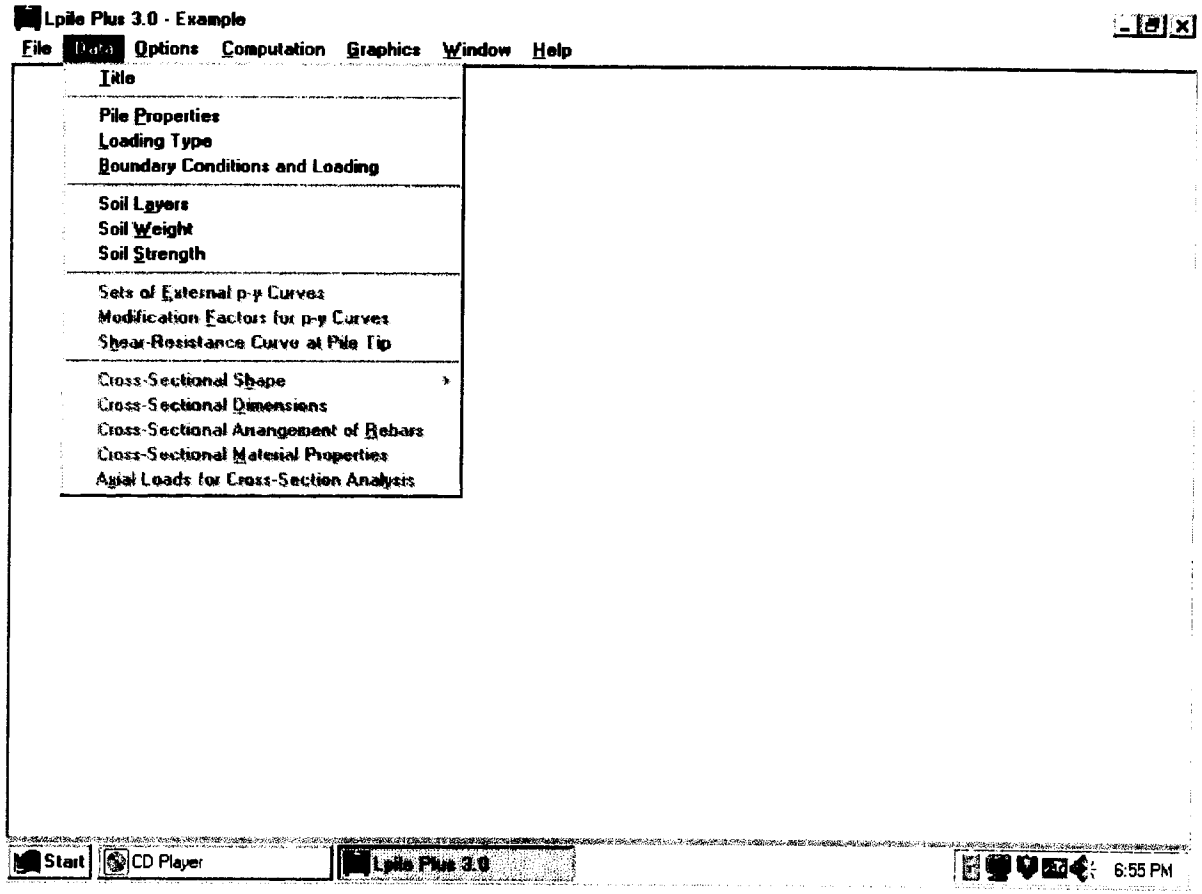


Figure B-2. LPILE Data Menu

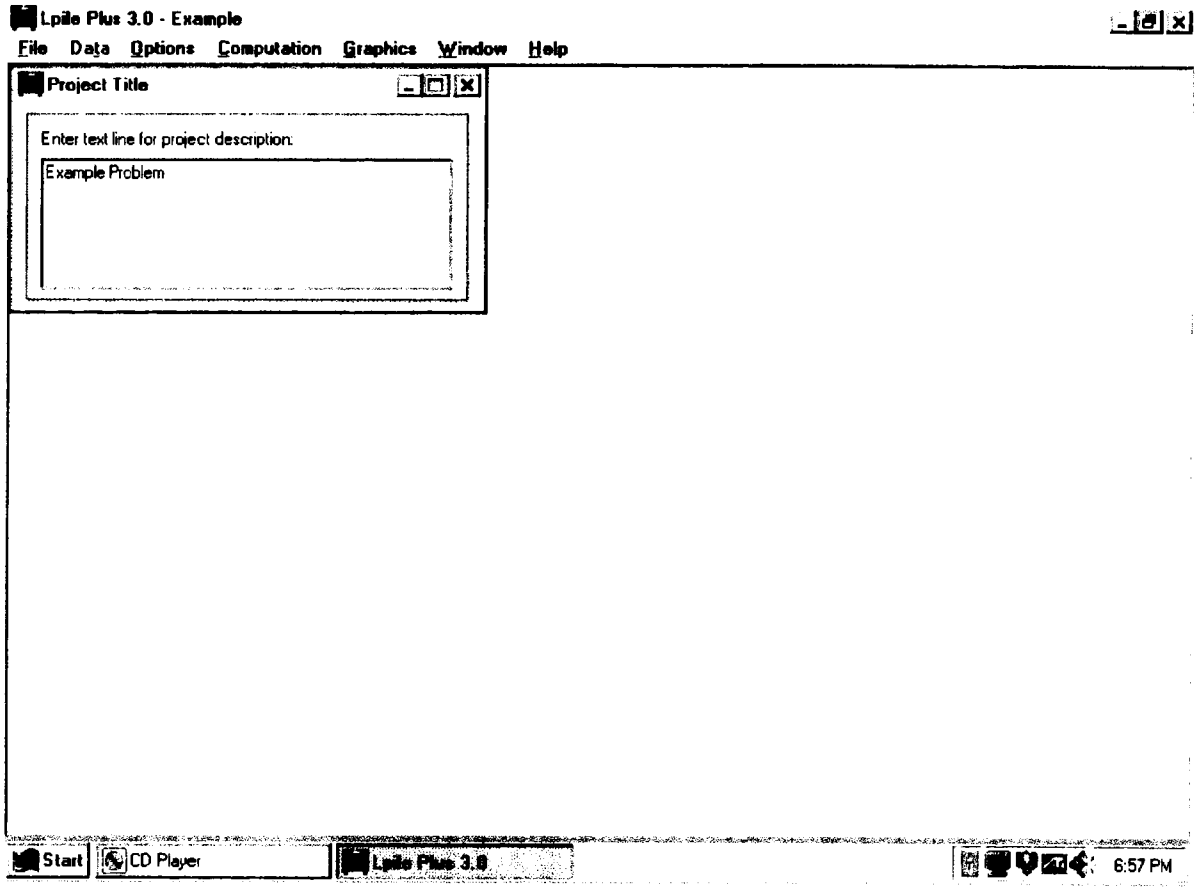


Figure B-3. LPILE Input Box for Project Title

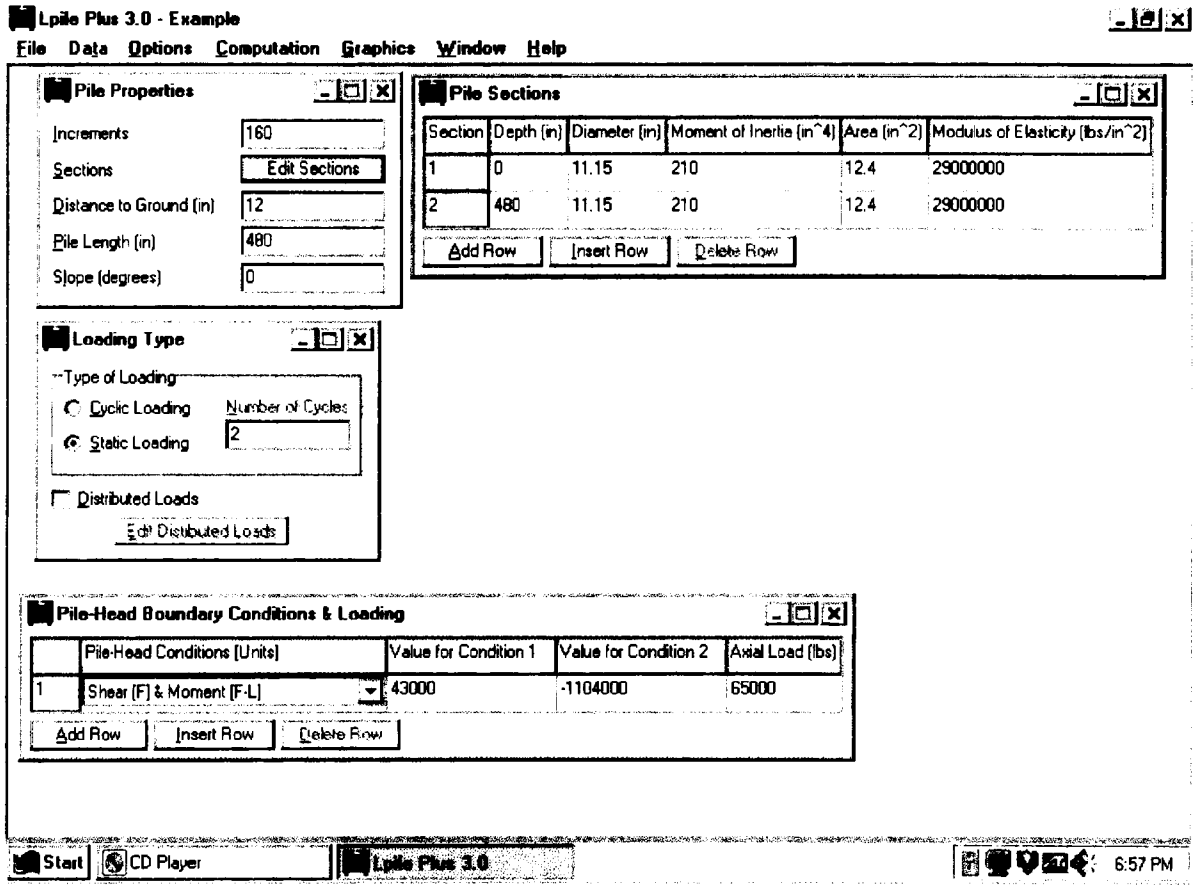


Figure B-4. LPILE Input Boxes for Pile Properties and Loading Conditions

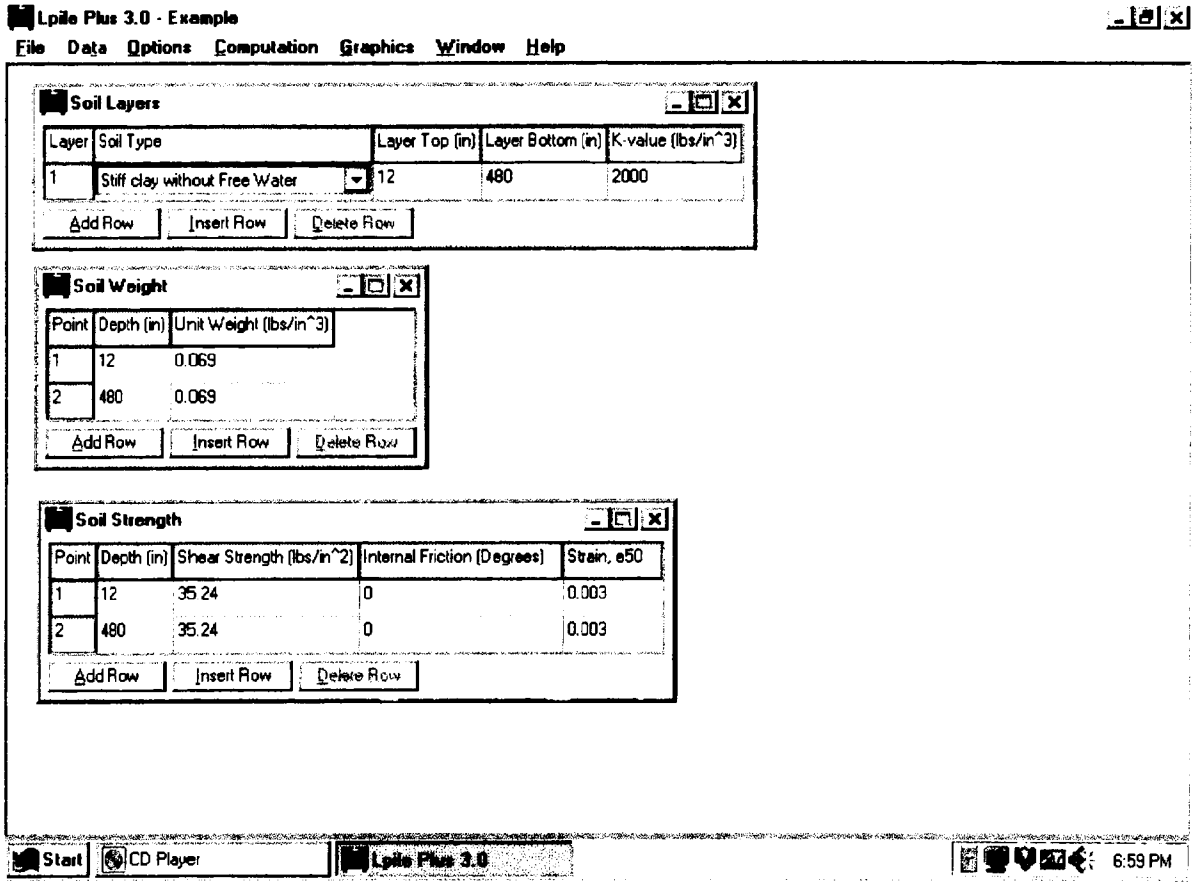


Figure B-5. LPILE Input Boxes for Soil Properties

- The number of increments in the Pile Properties box refers to the number of points along the length of the pile for which output is desired. It is recommended to use three to four increments per linear foot.
- The depth in the Pile Sections box and in all input boxes is measured from the pile head which is not necessarily from the ground level.
- The pile diameter was calculated from an equivalent circular cross-sectional area equal to the depth times the width of the HP section.
- The units of all input values shown consist of inches for length and pounds for force.
- There are four options for the pile-head conditions:

1. Shear & Moment
    - Lateral force is positive when applied from left to right.
    - Moment is positive when acting clockwise.
2. Shear & Slope
    - The user can analyze fixed-head condition if slope at pile-head is set equal to zero.
3. Shear & Rotational Stiffness
    - The user can also analyze fixed-head condition if rotational stiffness is very large.
    - Rotational stiffness is equal to the moment divided by the slope.
4. Displacement & Moment
    - Displacement is positive when applied from left to right.
    - Moment is positive when acting clockwise.

- Axial load is used to calculate second order moment effects.
- The Soil Layers box is where the p-y curve to be used is chosen.
- The soil modulus, K, was taken as recommended in LPILE documentation.

There are several options on the Data menu which are generally inactive. The first set of these options allows the user to include user-defined p-y curves, modify internally-generated p-y curves, and/or include shear resistance at the pile tip. These options can be activated from the Options menu. The next set of inactive options on the Data menu allow the user to define a general cross-section and can also be activated from the Options menu.

The following pages show the Options menu, the Control Options input box, and the Computation menu.

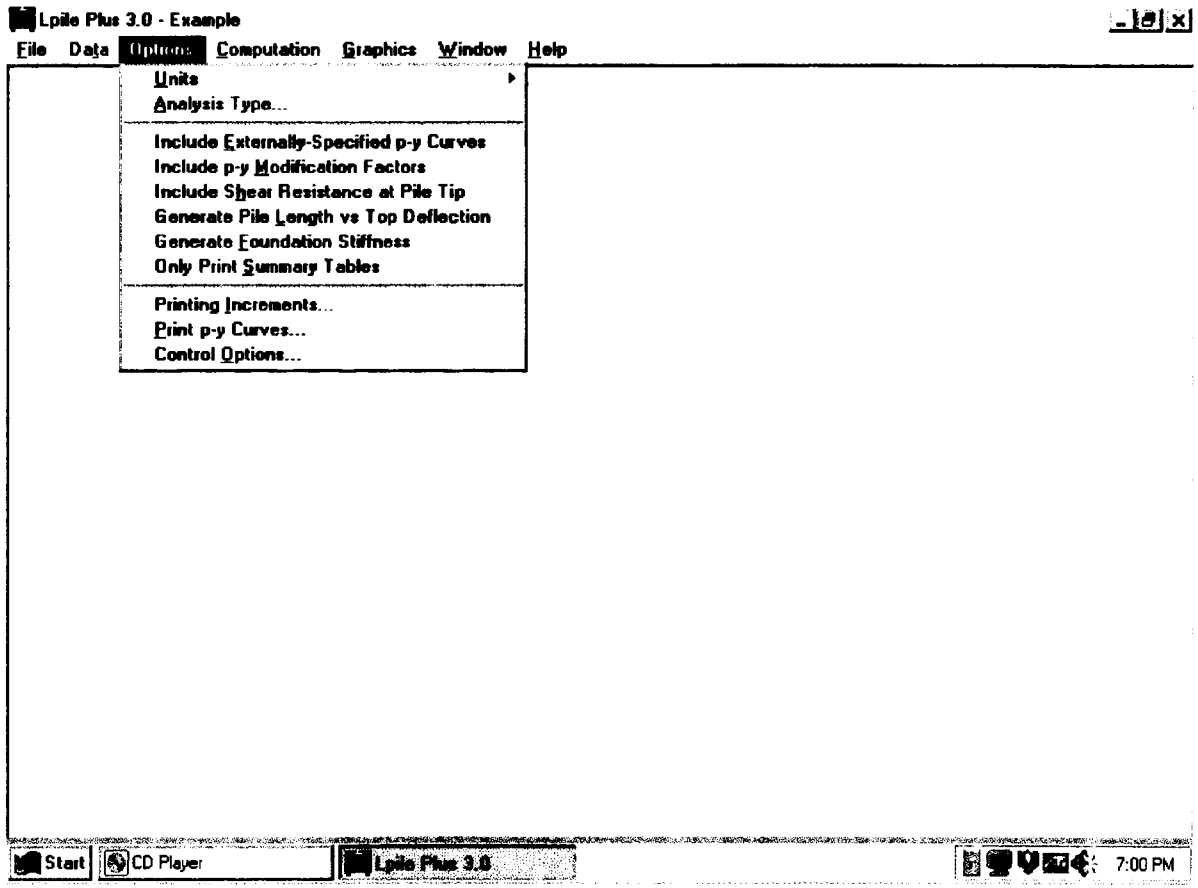


Figure B-6. LPILE Options Menu

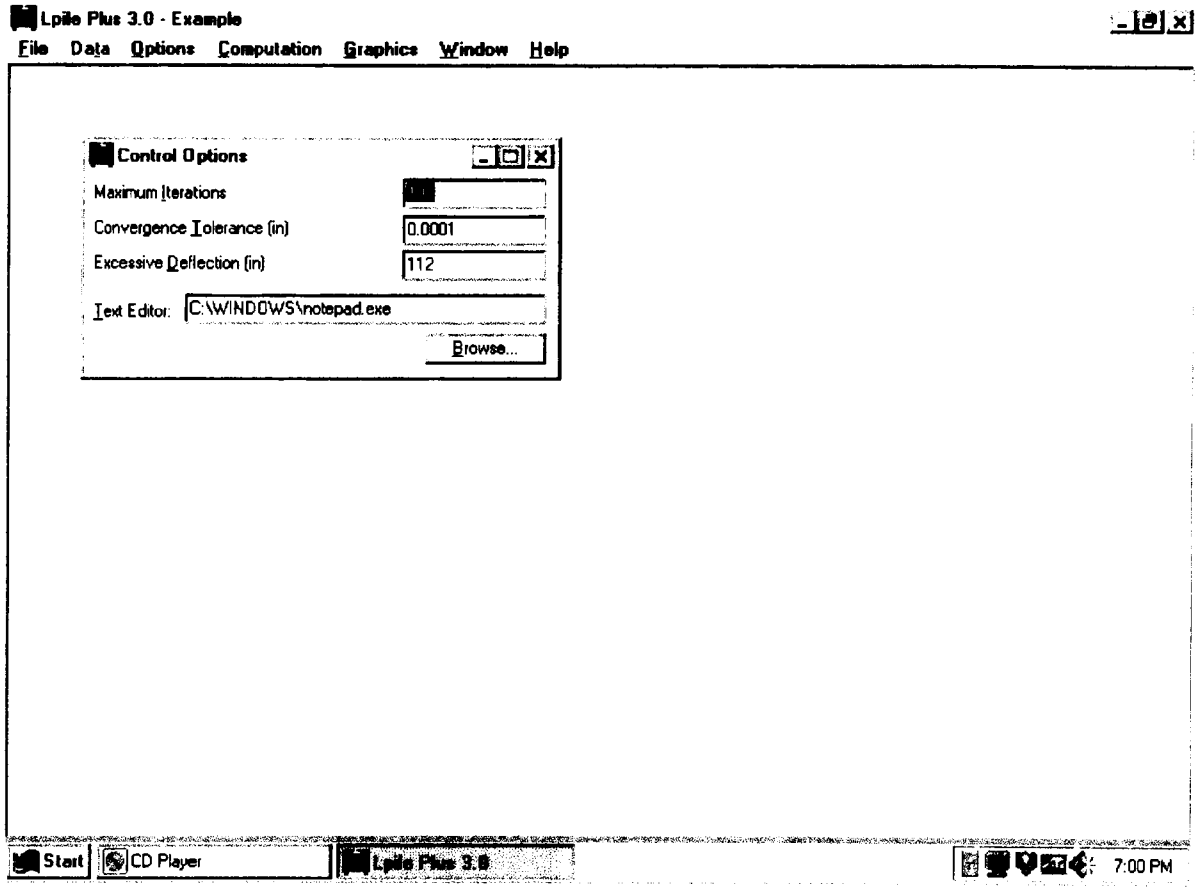


Figure B-7. LPILE Control Options

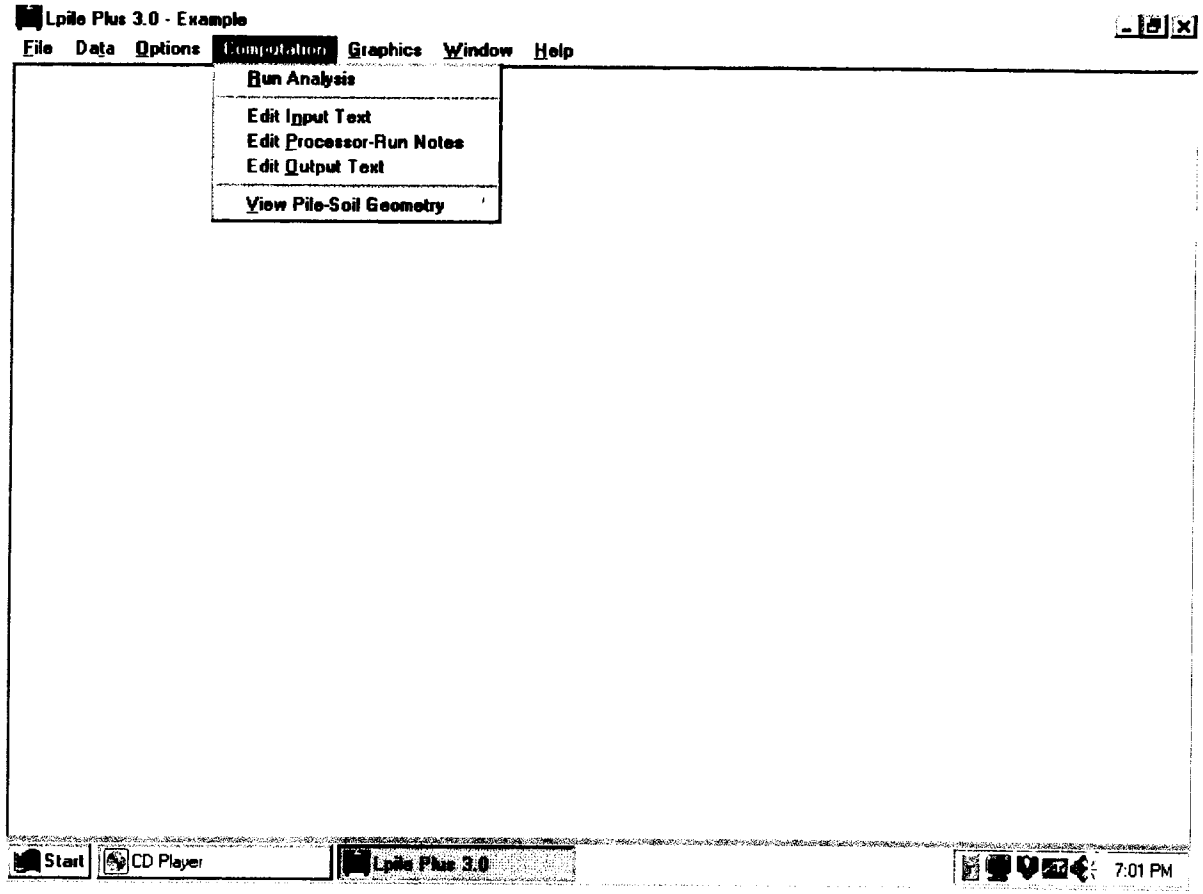


Figure B-8. LPILE Computation Menu

The first option under the Options menu allows the user to select either English or SI units. The Analysis Type option allows the user to choose one of four types of analyses:

- Computations of pile response with user-specified elastic modulus and moment of inertia, EI (analysis type used in this thesis)
- Computations of ultimate bending moment of cross-section
- Computations of ultimate bending moment and pile response using nonlinear elastic modulus and moment of inertia, EI
- Computations of ultimate bending moment and pile response with user-specified elastic modulus and moment of inertia, EI

The next three selections on the Options menu are used to activate the options on the Data menu mentioned earlier. The following two selections on the Options menu can provide the user with a plot of pile length vs. pile-head deflection as well as values for a foundation stiffness matrix. There are also options to print only a summary output table or to specify the increments along the pile for which data are to be printed. The option to print the p-y curves used in the analysis at specified depths is also provided. The final menu choice is Control Options which allows the user to specify the following:

- Maximum number of iterations (recommend 100)
- Convergence tolerance (1/10,000 inches default)
- Deflection considered to be excessive (recommend 10 times diameter)
- Text editor used to review output (c:\windows\notepad.exe default)

- The Run Analysis option under the Computation menu is selected only after input data are entered and saved.
- The Edit Input Text option allows the user to edit the input data file in a text editor.
- The Edit Processor – Run Notes option allows the user to edit the notes created by LPILE during analysis.
- The Edit Output Text option allows the user to edit the output data file in a text editor.
- The View Pile-Soil Geometry option shows the user a schematic of the pile and soil layers.

The following pages show the Graphics menu as well as graphical results obtained by choosing one of the first five options in the Graphics menu.

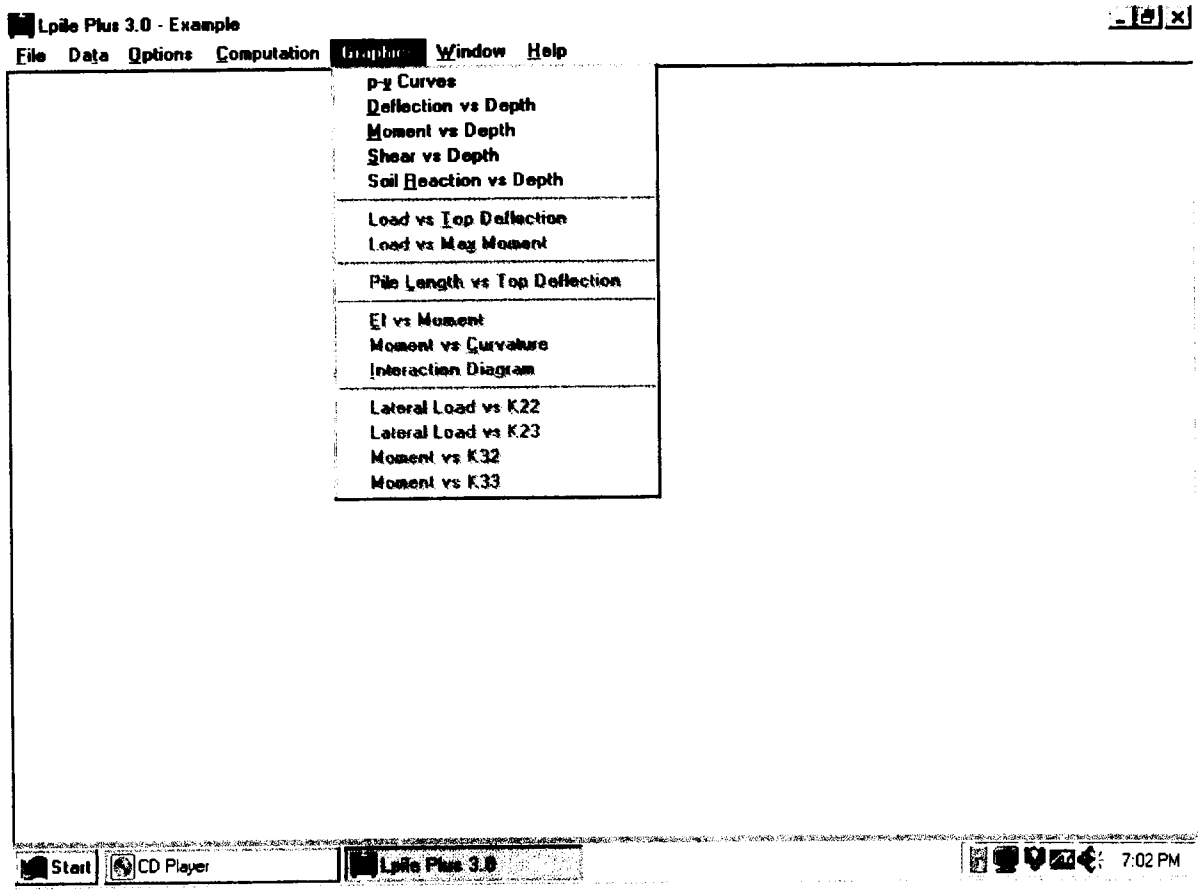


Figure B-9. LPILE Graphics Menu

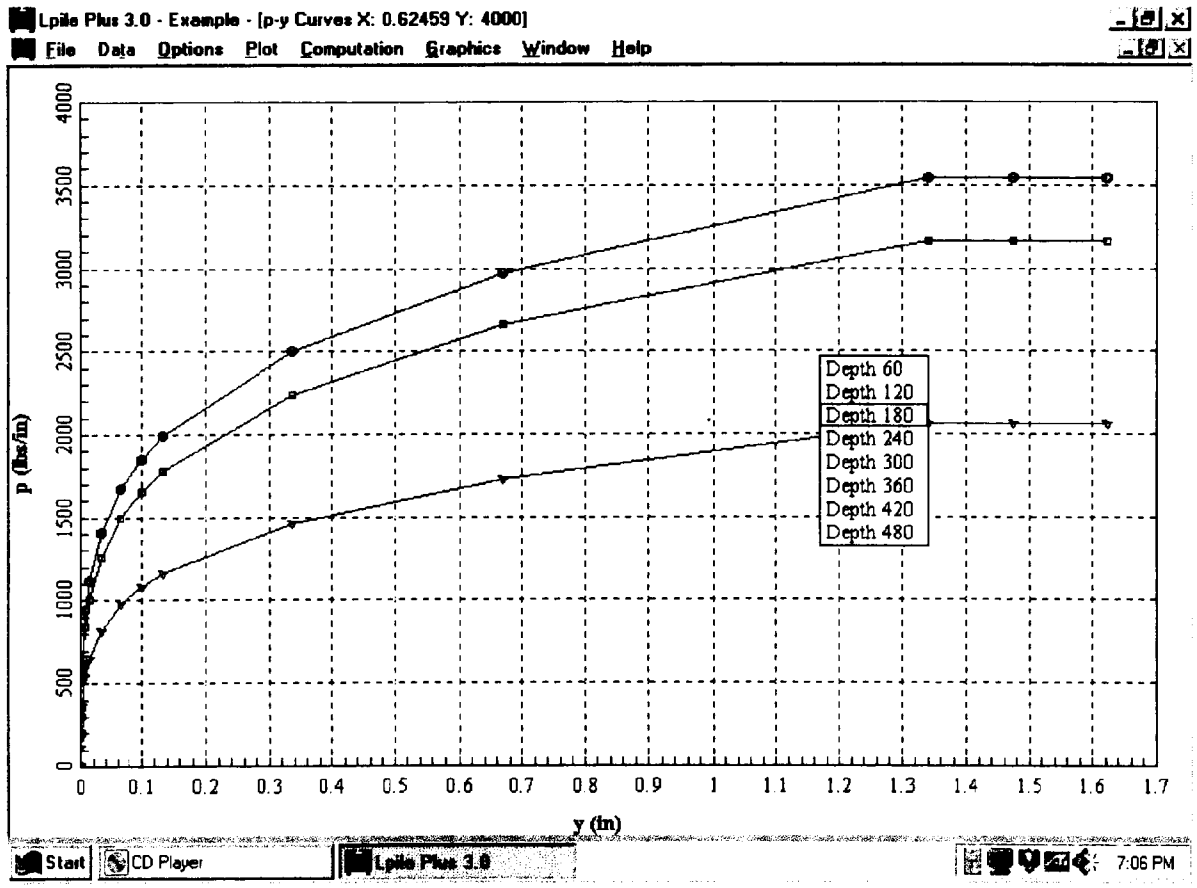


Figure B-10. LPILE p-y curves for stiff clay with no free water

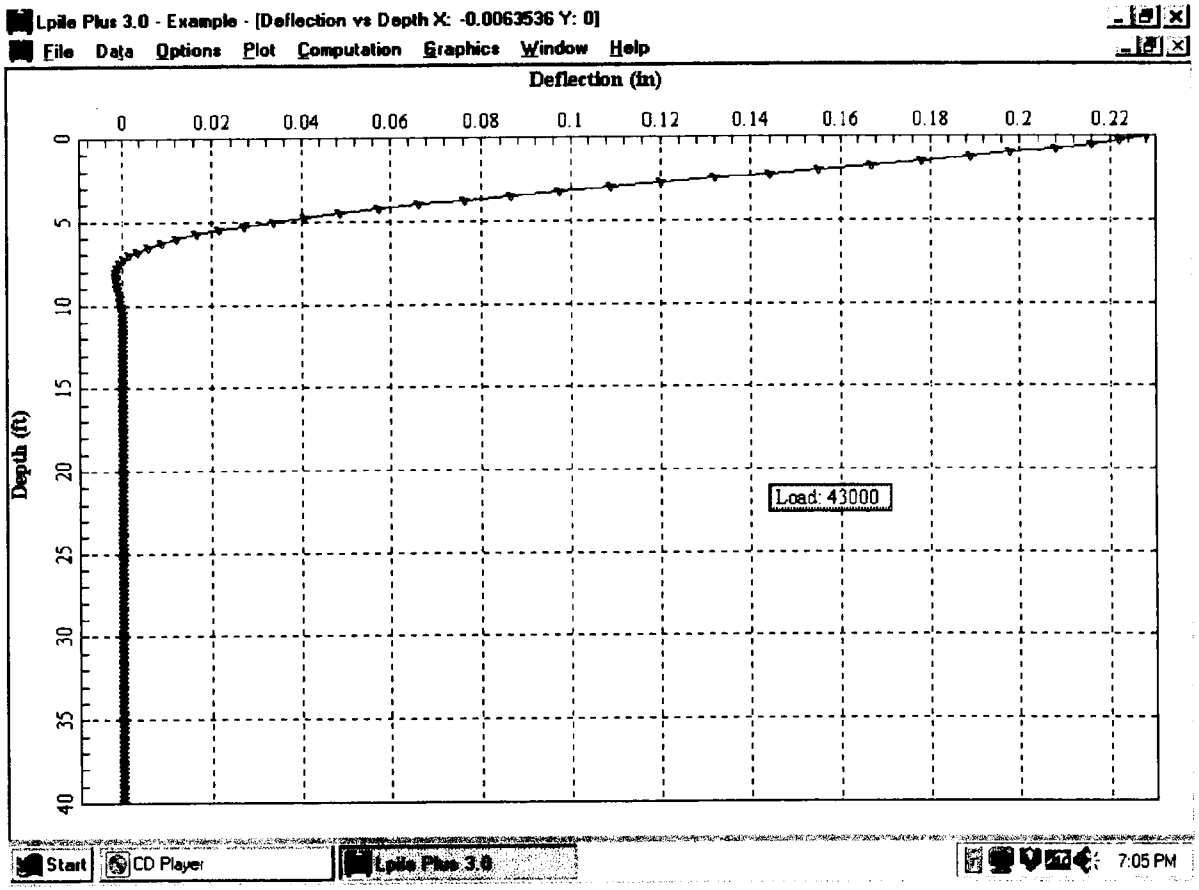


Figure B-11. LPILE Plot of Deflection vs. Depth

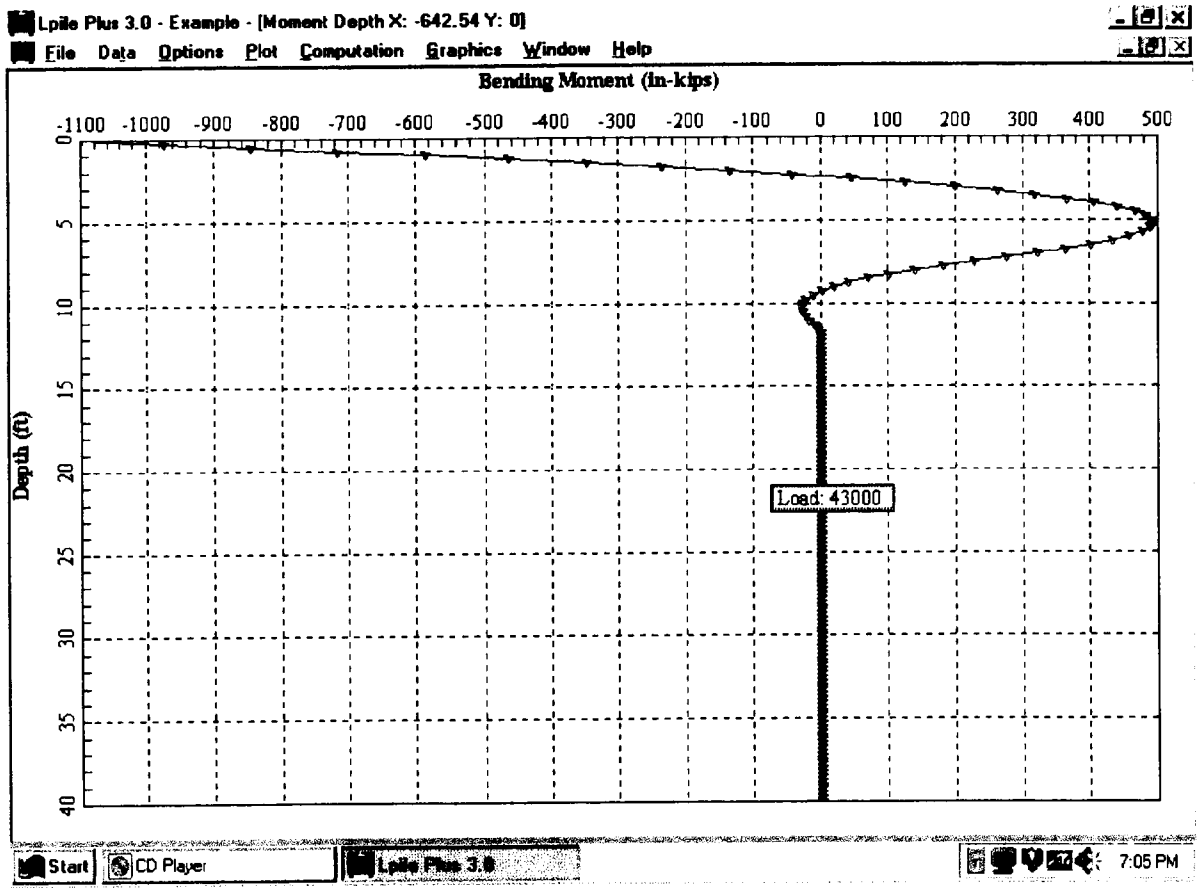


Figure B-12. LPILE Plot of Moment vs. Depth

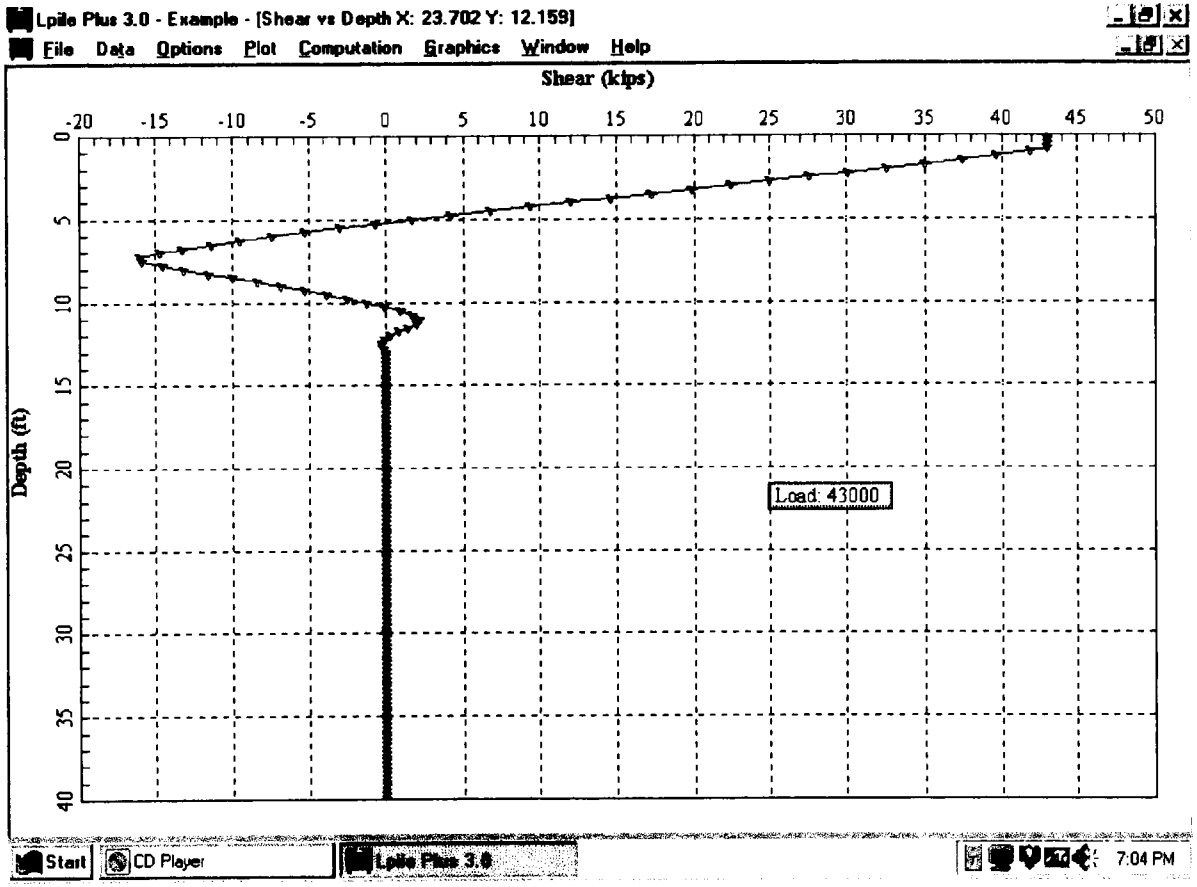


Figure B-13. LPILE Plot of Shear vs. Depth

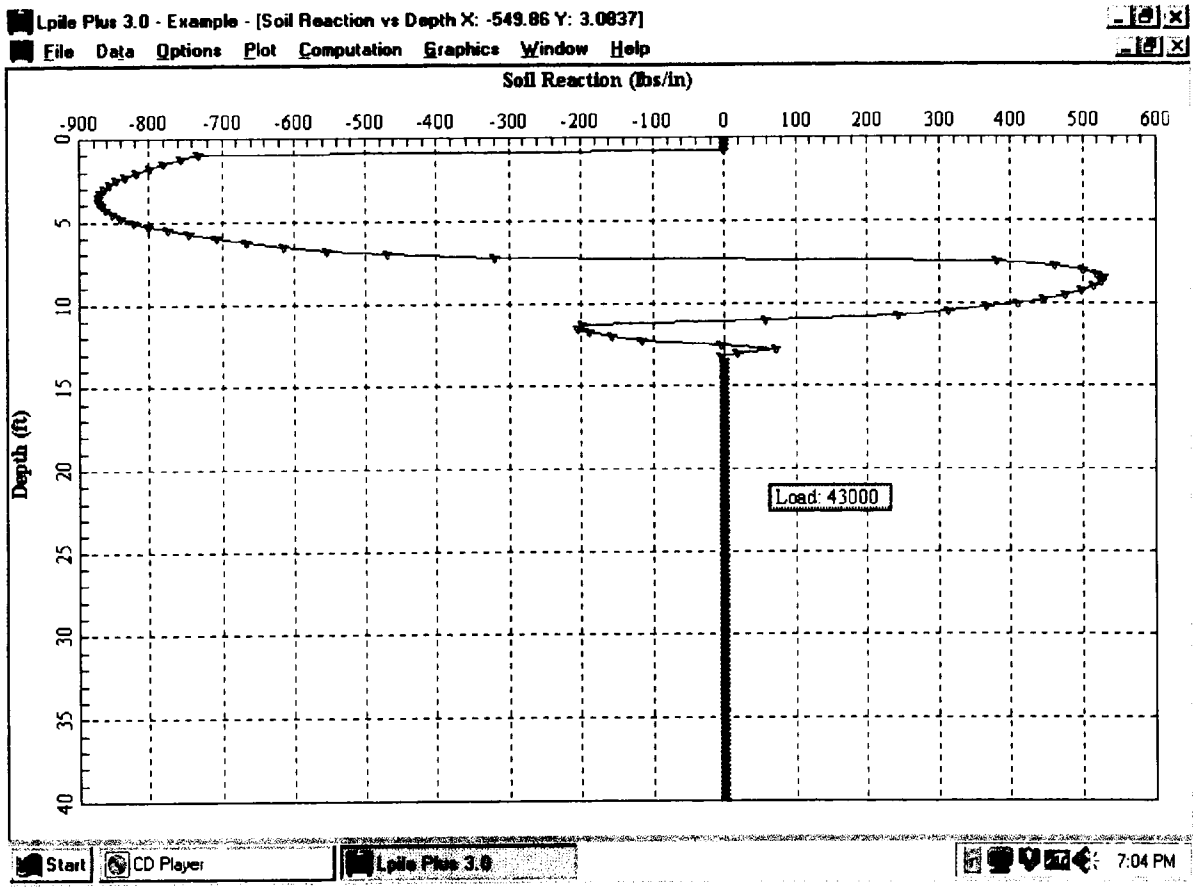


Figure B-14. LPILE Plot of Soil Reaction vs. Depth

The remaining graphics options are inactive under default conditions. The Load vs. Top Deflection and Load vs. Max. Moment option is only available if there are several loading conditions with varying values of shear and constant values of the other loading conditions. The Pile Length vs. Top Deflection option is activated by the corresponding selection under the Options menu. The next group of options is only available if the Analysis Type is set as the computation of ultimate bending moment of cross-section; also, for the Interaction Diagram option, the user must input several different load cases for the axial load. The remaining selections are only available if a foundation stiffness matrix is generated.

The following pages contain a copy of the LPILE output file for the example problem.

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UT Civil Engineering

PROGRAM LPILE plus Version 3.0
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Example Problem

UNITS--ENGLISH UNITS

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

THE LOADING IS STATIC

PILE GEOMETRY AND PROPERTIES

PILE LENGTH = 480.00 IN
2 POINTS

X	DIAMETER	MOMENT OF INERTIA	AREA	MODULUS OF ELASTICITY
IN	IN	IN**4	IN**2	LBS/IN**2
.00	11.150	.210E+03	.124E+02	.290E+08
480.00	11.150	.210E+03	.124E+02	.290E+08

SOILS INFORMATION

X AT THE GROUND SURFACE = 12.00 IN

SLOPE ANGLE AT THE GROUND SURFACE = .00 DEG.

1 LAYER(S) OF SOIL

LAYER 1
THE SOIL IS A STIFF CLAY WITH NO FREE WATER
X AT THE TOP OF THE LAYER = 12.00 IN
X AT THE BOTTOM OF THE LAYER = 480.00 IN
MODULUS OF SUBGRADE REACTION = .200E+04 LBS/IN**3

DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
2 POINTS

X,IN	WEIGHT,LBS/IN**3
12.00	.69E-01
480.00	.69E-01

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
2 POINTS

X,IN	C,LBS/IN**2	PHI,DEGREES	E50
12.00	.352E+02	.000	.300E-02
480.00	.352E+02	.000	.300E-02

BOUNDARY AND LOADING CONDITIONS

LOADING NUMBER 1

BOUNDARY-CONDITION CODE = 1
LATERAL LOAD AT THE PILE HEAD = .430E+05 LBS
MOMENT AT THE PILE HEAD = -.110E+07 IN-LBS
AXIAL LOAD AT THE PILE HEAD = .650E+05 LBS

FINITE-DIFFERENCE PARAMETERS
NUMBER OF PILE INCREMENTS = 160
DEFLECTION TOLERANCE ON DETERMINATION OF CLOSURE = .100E-03 IN
MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS = 100
MAXIMUM ALLOWABLE DEFLECTION = .11E+03 IN

OUTPUT CODES
KOUTPT = 1
KPYOP = 1
INC = 1

DEPTH IN	DIAM IN	C LBS/IN**2	CAVG LBS/IN**3	GAMMA AVG LBS/IN**3	E50
48.00	11.150	.35E+02	.35E+02	.69E-01	.300E-02

Y IN	P LBS/IN
.000E+00	.000E+00
.134E-04	.116E+03
.669E-04	.173E+03
.134E-03	.206E+03
.669E-03	.308E+03
.134E-02	.367E+03
.669E-02	.548E+03
.134E-01	.652E+03
.335E-01	.820E+03
.669E-01	.975E+03
.100E+00	.108E+04
.134E+00	.116E+04
.335E+00	.146E+04
.669E+00	.173E+04
.134E+01	.206E+04
.151E+01	.206E+04
.167E+01	.206E+04

DEPTH	DIAM	C	CAVG	GAMMA AVG	E50
IN	IN	LBS/IN**2	LBS/IN**3	LBS/IN**3	
468.00	11.150	.35E+02	.35E+02	.69E-01	.300E-02

Y	P
IN	LBS/IN
.000E+00	.000E+00
.134E-04	.199E+03
.669E-04	.297E+03
.134E-03	.354E+03
.669E-03	.529E+03
.134E-02	.629E+03
.669E-02	.940E+03
.134E-01	.112E+04
.335E-01	.141E+04
.669E-01	.167E+04
.100E+00	.185E+04
.134E+00	.199E+04
.335E+00	.250E+04
.669E+00	.297E+04
.134E+01	.354E+04
.151E+01	.354E+04
.167E+01	.354E+04

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

****

****
* COMPUTE LOAD-DISTRIBUTION AND LOAD-DEFLECTION *
* CURVES FOR LATERAL LOADING *
****

LOADING NUMBER 1

BOUNDARY CONDITION CODE	=	1
LATERAL LOAD AT THE PILE HEAD	=	.430E+05 LBS
MOMENT AT THE PILE HEAD	=	-.110E+07 IN-LBS
AXIAL LOAD AT THE PILE HEAD	=	.650E+05 LBS

X	DEFLECTION	MOMENT	SHEAR	SLOPE	TOTAL STRESS	FLEXURAL RIGIDITY	SOIL REACTION
IN	IN	LBS-IN	LBS	RAD.	LBS/IN**2	LBS-IN**2	LBS/IN
.0	.228E+00	-.110E+07	.430E+05	-.151E-02	.346E+05	.609E+10	.000E+00
3.0	.222E+00	-.975E+06	.430E+05	-.202E-02	.311E+05	.609E+10	.000E+00
6.0	.216E+00	-.845E+06	.430E+05	-.247E-02	.277E+05	.609E+10	.000E+00
9.0	.208E+00	-.716E+06	.430E+05	-.285E-02	.242E+05	.609E+10	.000E+00
12.0	.198E+00	-.586E+06	.419E+05	-.317E-02	.208E+05	.609E+10	-.732E+03
15.0	.189E+00	-.463E+06	.397E+05	-.343E-02	.175E+05	.609E+10	-.756E+03
18.0	.178E+00	-.347E+06	.374E+05	-.363E-02	.144E+05	.609E+10	-.779E+03
21.0	.167E+00	-.237E+06	.350E+05	-.377E-02	.115E+05	.609E+10	-.799E+03
24.0	.155E+00	-.135E+06	.326E+05	-.387E-02	.883E+04	.609E+10	-.817E+03
27.0	.144E+00	-.404E+05	.301E+05	-.391E-02	.632E+04	.609E+10	-.833E+03
30.0	.132E+00	.469E+05	.276E+05	-.391E-02	.649E+04	.609E+10	-.846E+03
33.0	.120E+00	.127E+06	.250E+05	-.386E-02	.860E+04	.609E+10	-.857E+03
36.0	.109E+00	.199E+06	.224E+05	-.378E-02	.105E+05	.609E+10	-.865E+03
39.0	.974E-01	.263E+06	.198E+05	-.367E-02	.122E+05	.609E+10	-.870E+03
42.0	.866E-01	.319E+06	.172E+05	-.353E-02	.137E+05	.609E+10	-.873E+03
45.0	.762E-01	.368E+06	.146E+05	-.336E-02	.150E+05	.609E+10	-.873E+03
48.0	.664E-01	.408E+06	.120E+05	-.317E-02	.161E+05	.609E+10	-.869E+03
51.0	.572E-01	.441E+06	.940E+04	-.296E-02	.169E+05	.609E+10	-.863E+03
54.0	.487E-01	.466E+06	.683E+04	-.274E-02	.176E+05	.609E+10	-.853E+03
57.0	.408E-01	.483E+06	.429E+04	-.250E-02	.181E+05	.609E+10	-.839E+03
60.0	.337E-01	.492E+06	.180E+04	-.226E-02	.183E+05	.609E+10	-.822E+03
63.0	.272E-01	.494E+06	-.633E+03	-.202E-02	.184E+05	.609E+10	-.800E+03
66.0	.215E-01	.489E+06	-.299E+04	-.178E-02	.182E+05	.609E+10	-.774E+03
69.0	.166E-01	.477E+06	-.527E+04	-.154E-02	.179E+05	.609E+10	-.744E+03
72.0	.123E-01	.458E+06	-.745E+04	-.131E-02	.174E+05	.609E+10	-.708E+03
75.0	.874E-02	.433E+06	-.951E+04	-.109E-02	.167E+05	.609E+10	-.665E+03
78.0	.579E-02	.402E+06	-.114E+05	-.883E-03	.159E+05	.609E+10	-.615E+03
81.0	.344E-02	.365E+06	-.132E+05	-.694E-03	.149E+05	.609E+10	-.553E+03
84.0	.163E-02	.323E+06	-.147E+05	-.525E-03	.138E+05	.609E+10	-.470E+03
87.0	.291E-03	.277E+06	-.159E+05	-.377E-03	.126E+05	.609E+10	-.320E+03
90.0	-.635E-03	.228E+06	-.158E+05	-.253E-03	.113E+05	.609E+10	.381E+03
93.0	-.123E-02	.182E+06	-.145E+05	-.152E-03	.101E+05	.609E+10	.462E+03
96.0	-.155E-02	.140E+06	-.131E+05	-.725E-04	.897E+04	.609E+10	.500E+03
99.0	-.166E-02	.103E+06	-.116E+05	-.125E-04	.798E+04	.609E+10	.520E+03
102.0	-.162E-02	.709E+05	-.100E+05	.304E-04	.713E+04	.609E+10	.528E+03
105.0	-.148E-02	.433E+05	-.842E+04	.586E-04	.639E+04	.609E+10	.526E+03
108.0	-.127E-02	.204E+05	-.686E+04	.743E-04	.578E+04	.609E+10	.516E+03
111.0	-.103E-02	.214E+04	-.533E+04	.798E-04	.530E+04	.609E+10	.499E+03
114.0	-.791E-03	-.116E+05	-.387E+04	.775E-04	.555E+04	.609E+10	.476E+03
117.0	-.567E-03	-.211E+05	-.249E+04	.694E-04	.580E+04	.609E+10	.446E+03
120.0	-.375E-03	-.266E+05	-.121E+04	.577E-04	.595E+04	.609E+10	.409E+03
123.0	-.221E-03	-.284E+05	-.481E+02	.441E-04	.600E+04	.609E+10	.365E+03
126.0	-.110E-03	-.269E+05	.966E+03	.305E-04	.596E+04	.609E+10	.312E+03
129.0	-.383E-04	-.226E+05	.180E+04	.183E-04	.584E+04	.609E+10	.243E+03
132.0	-.841E-07	-.161E+05	.225E+04	.877E-05	.567E+04	.609E+10	.575E+02
135.0	.143E-04	-.911E+04	.204E+04	.255E-05	.548E+04	.609E+10	-.197E+03
138.0	.152E-04	-.387E+04	.144E+04	-.646E-06	.534E+04	.609E+10	-.203E+03
141.0	.104E-04	-.463E+03	.855E+03	-.171E-05	.525E+04	.609E+10	-.188E+03
144.0	.494E-05	.126E+04	.338E+03	-.152E-05	.528E+04	.609E+10	-.157E+03
147.0	.131E-05	.157E+04	-.680E+02	-.824E-06	.528E+04	.609E+10	-.114E+03
150.0	.149E-09	.848E+03	-.247E+03	-.229E-06	.526E+04	.609E+10	-.535E+01
153.0	-.609E-07	.825E+02	-.148E+03	-.258E-10	.524E+04	.609E+10	.713E+02
156.0	-.613E-11	-.412E+02	-.137E+02	.101E-07	.524E+04	.609E+10	.183E+02
159.0	.119E-12	-.431E-02	.686E+01	.102E-11	.524E+04	.609E+10	-.457E+01
162.0	.478E-17	.803E-04	.718E-03	-.198E-13	.524E+04	.609E+10	-.497E-03
165.0	-.858E-19	.335E-08	-.134E-04	-.796E-18	.524E+04	.609E+10	.893E-05
168.0	-.370E-23	-.581E-10	-.558E-09	.143E-19	.524E+04	.609E+10	.385E-09

171.0	.621E-25	-.259E-14	.968E-11	.617E-24	.524E+04	.609E+10	-.645E-11
174.0	.286E-29	.420E-16	.432E-15	-.103E-25	.524E+04	.609E+10	-.297E-15
177.0	-.449E-31	.199E-20	-.700E-17	-.476E-30	.524E+04	.609E+10	.466E-17
180.0	-.220E-35	-.304E-22	-.332E-21	.748E-32	.524E+04	.609E+10	.228E-21
183.0	.324E-37	-.153E-26	.506E-23	.366E-36	.524E+04	.609E+10	-.337E-23
186.0	.168E-41	.219E-28	.255E-27	-.541E-38	.524E+04	.609E+10	-.175E-27
189.0	-.234E-43	.117E-32	-.366E-29	-.280E-42	.524E+04	.609E+10	.244E-29
192.0	-.128E-47	-.159E-34	-.195E-33	.391E-44	.524E+04	.609E+10	.133E-33
195.0	.170E-49	-.892E-39	.264E-35	.214E-48	.524E+04	.609E+10	-.176E-35
198.0	.977E-54	.115E-40	.149E-39	-.283E-50	.524E+04	.609E+10	-.102E-39
201.0	-.123E-55	.678E-45	-.191E-41	-.163E-54	.524E+04	.609E+10	.127E-41
204.0	-.742E-60	-.829E-47	-.113E-45	.204E-56	.524E+04	.609E+10	.772E-46
207.0	.886E-62	-.514E-51	.138E-47	.124E-60	.524E+04	.609E+10	-.921E-48
210.0	.562E-66	.599E-53	.857E-52	-.148E-62	.524E+04	.609E+10	-.585E-52
213.0	-.640E-68	.389E-57	-.999E-54	-.937E-67	.524E+04	.609E+10	.666E-54
216.0	-.425E-72	-.433E-59	-.648E-58	.107E-68	.524E+04	.609E+10	.442E-58
219.0	.463E-74	-.294E-63	.722E-60	.708E-73	.524E+04	.609E+10	-.481E-60
222.0	.321E-78	.313E-65	.490E-64	-.772E-75	.524E+04	.609E+10	-.333E-64
225.0	-.335E-80	.222E-69	-.522E-66	-.535E-79	.524E+04	.609E+10	.348E-66
228.0	-.242E-84	-.226E-71	-.369E-70	.558E-81	.524E+04	.609E+10	.251E-70
231.0	.242E-86	-.167E-75	.377E-72	.403E-85	.524E+04	.609E+10	-.252E-72
234.0	.182E-90	.164E-77	.278E-76	-.403E-87	.524E+04	.609E+10	-.189E-76
237.0	-.175E-92	.125E-81	-.273E-78	-.303E-91	.524E+04	.609E+10	.182E-78
240.0	-.136E-96	-.118E-83	-.209E-82	.291E-93	.524E+04	.609E+10	.142E-82
243.0	.126E-98	-.940E-88	.197E-84	.227E-97	.524E+04	.609E+10	-.131E-84
246.0	.102-102	.855E-90	.157E-88	-.211E-99	.524E+04	.609E+10	-.106E-88
249.0	-.914-105	.705E-94	-.143E-90	-.171-103	.524E+04	.609E+10	.950E-91
252.0	-.766-109	-.618E-96	-.117E-94	.152-105	.524E+04	.609E+10	.797E-95
255.0	.661-111	-.527-100	.103E-96	.128-109	.524E+04	.609E+10	-.687E-97
258.0	.573-115	.447-102	.879-101	-.110-111	.524E+04	.609E+10	-.596-101
261.0	-.477-117	.394-106	-.745-103	-.955-116	.524E+04	.609E+10	.496-103
264.0	-.428-121	-.323-108	-.657-107	.796-118	.524E+04	.609E+10	.445-107
267.0	.345-123	-.294-112	.538-109	.714-122	.524E+04	.609E+10	-.359-109
270.0	.320-127	.233-114	.491-113	-.575-124	.524E+04	.609E+10	-.332-113
273.0	-.249-129	.220-118	-.389-115	-.533-128	.524E+04	.609E+10	.259-115
276.0	-.238-133	-.169-120	-.366-119	.416-130	.524E+04	.609E+10	.248-119
279.0	.180-135	-.164-124	.281-121	.397-134	.524E+04	.609E+10	-.187-121
282.0	.178-139	.122-126	.273-125	-.301-136	.524E+04	.609E+10	-.185-125
285.0	-.130-141	.122-130	-.203-127	-.296-140	.524E+04	.609E+10	.136-127
288.0	-.132-145	-.882-133	-.203-131	.217-142	.524E+04	.609E+10	.137-131
291.0	.942-148	-.907-137	.147-133	.220-146	.524E+04	.609E+10	-.979-134
294.0	.983-152	.637-139	.151-137	-.157-148	.524E+04	.609E+10	-.102-137
297.0	-.681-154	.674-143	-.106-139	-.164-152	.524E+04	.609E+10	.708-140
300.0	-.730-158	-.461-145	-.112-143	.113-154	.524E+04	.609E+10	.759-144
303.0	.492-160	-.501-149	.768-146	.122-158	.524E+04	.609E+10	-.512-146
306.0	.542-164	.333-151	.835-150	-.820-161	.524E+04	.609E+10	-.564-150
309.0	-.356-166	.372-155	-.555-152	-.904-165	.524E+04	.609E+10	.370-152
312.0	-.402-170	-.241-157	-.620-156	.593-167	.524E+04	.609E+10	.418-156
315.0	.257-172	-.276-161	.401-158	.671-171	.524E+04	.609E+10	-.267-158
318.0	.298-176	.174-163	.460-162	-.428-173	.524E+04	.609E+10	-.310-162
321.0	-.186-178	.204-167	-.290-164	-.497-177	.524E+04	.609E+10	.193-164
324.0	-.221-182	-.126-169	-.341-168	.310-179	.524E+04	.609E+10	.230-168
327.0	.134-184	-.151-173	.209-170	.369-183	.524E+04	.609E+10	-.140-170
330.0	.164-188	.908-176	.252-174	-.224-185	.524E+04	.609E+10	-.170-174
333.0	-.971-191	.112-179	-.151-176	-.273-189	.524E+04	.609E+10	.101-176
336.0	-.121-194	-.657-182	-.187-180	.162-191	.524E+04	.609E+10	.126-180
339.0	.701-197	-.830-186	.109-182	.202-195	.524E+04	.609E+10	-.729-183
342.0	.897-201	.475-188	.138-186	-.117-197	.524E+04	.609E+10	-.933-187
345.0	-.507-203	.614-192	-.791-189	-.150-201	.524E+04	.609E+10	.527-189
348.0	-.663-207	-.343-194	-.102-192	.845-204	.524E+04	.609E+10	.690-193

351.0	.366-209	-.454-198	.572-195	.111-207	.524E+04	.609E+10	-.381-195
354.0	.490-213	.248-200	.756-199	-.611-210	.524E+04	.609E+10	-.510-199
357.0	-.265-215	.335-204	-.413-201	-.817-214	.524E+04	.609E+10	.275-201
360.0	-.362-219	-.179-206	-.559-205	.441-216	.524E+04	.609E+10	.377-205
363.0	.191-221	-.248-210	.299-207	.604-220	.524E+04	.609E+10	-.199-207
366.0	.267-225	.129-212	.413-211	-.319-222	.524E+04	.609E+10	-.278-211
369.0	-.138-227	.183-216	-.216-213	-.446-226	.524E+04	.609E+10	.144-213
372.0	-.197-231	-.935-219	-.305-217	.230-228	.524E+04	.609E+10	.205-217
375.0	.999-234	-.135-222	.156-219	.329-232	.524E+04	.609E+10	-.104-219
378.0	.146-237	.676-225	.225-223	-.167-234	.524E+04	.609E+10	-.151-223
381.0	-.722-240	.995-229	-.113-225	-.243-238	.524E+04	.609E+10	.751-226
384.0	-.107-243	-.489-231	-.166-229	.120-240	.524E+04	.609E+10	.112-229
387.0	.522-246	-.734-235	.814-232	.179-244	.524E+04	.609E+10	-.543-232
390.0	.791-250	.353-237	.122-235	-.870-247	.524E+04	.609E+10	-.823-236
393.0	-.377-252	.541-241	-.588-238	-.132-250	.524E+04	.609E+10	.392-238
396.0	-.583-256	-.255-243	-.901-242	.629-253	.524E+04	.609E+10	.606-242
399.0	.273-258	-.398-247	.425-244	.972-257	.524E+04	.609E+10	-.283-244
402.0	.430-262	.184-249	.664-248	-.454-259	.524E+04	.609E+10	-.447-248
405.0	-.197-264	.293-253	-.307-250	-.716-263	.524E+04	.609E+10	.205-250
408.0	-.316-268	-.133-255	-.489-254	.328-265	.524E+04	.609E+10	.329-254
411.0	.142-270	-.216-259	.222-256	.527-269	.524E+04	.609E+10	-.148-256
414.0	.233-274	.963-262	.360-260	-.237-271	.524E+04	.609E+10	-.242-260
417.0	-.103-276	.159-265	-.160-262	-.388-275	.524E+04	.609E+10	.107-262
420.0	-.171-280	-.696-268	-.265-266	.171-277	.524E+04	.609E+10	.178-266
423.0	.743-283	-.117-271	.116-268	.286-281	.524E+04	.609E+10	-.773-269
426.0	.126-286	.503-274	.195-272	-.124-283	.524E+04	.609E+10	-.131-272
429.0	-.537-289	.860-278	-.838-275	-.210-287	.524E+04	.609E+10	.558-275
432.0	-.927-293	-.363-280	-.143-278	.895-290	.524E+04	.609E+10	.964-279
435.0	.388-295	-.633-284	.605-281	.155-293	.524E+04	.609E+10	-.403-281
438.0	.682-299	.262-286	.105-284	-.647-296	.524E+04	.609E+10	-.709-285
441.0	-.280-301	.465-290	-.437-287	-.114-299	.524E+04	.609E+10	.292-287
444.0	-.501-305	-.190-292	-.775-291	.467-302	.524E+04	.609E+10	.521-291
447.0	.203-307	-.342-296	.316-293	.835-306	.524E+04	.609E+10	-.211-293
450.0	.368-311	.137-298	.569-297	-.338-308	.524E+04	.609E+10	-.383-297
453.0	-.146-313	.251-302	-.228-299	-.613-312	.524E+04	.609E+10	.152-299
456.0	-.270-317	-.990-305	-.418-303	.244-314	.524E+04	.609E+10	.281-303
459.0	.106-319	-.184-308	.165-305	.451-318	.524E+04	.609E+10	-.110-305
462.0	.000E+00	.716-311	.307-309	-.176-320	.524E+04	.609E+10	.000E+00
465.0	.000E+00	.000E+00	-.119-311	.000E+00	.524E+04	.609E+10	.000E+00
468.0	.000E+00	.000E+00	.000E+00	.000E+00	.524E+04	.609E+10	.000E+00
471.0	.000E+00	.000E+00	.000E+00	.000E+00	.524E+04	.609E+10	.000E+00
474.0	.000E+00	.000E+00	.000E+00	.000E+00	.524E+04	.609E+10	.000E+00
477.0	.000E+00	.000E+00	.000E+00	.000E+00	.524E+04	.609E+10	.000E+00
480.0	.000E+00	.000E+00	.000E+00	.000E+00	.524E+04	.609E+10	.000E+00

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = -.151E-06 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = .320E-07 LBS

OUTPUT SUMMARY

PILE-HEAD DEFLECTION	=	.228E+00 IN
COMPUTED SLOPE AT PILE HEAD	=	-.151E-02
MAXIMUM BENDING MOMENT	=	-.110E+07 LBS-IN
MAXIMUM SHEAR FORCE	=	.430E+05 LBS
NO. OF ITERATIONS	=	21
NO. OF ZERO DEFLECTION POINTS	=	54

S U M M A R Y T A B L E
****

BOUNDARY CONDITION BC1	BOUNDARY CONDITION BC2	AXIAL LOAD LBS	PILE HEAD DEFLECTION IN	MAX. MOMENT IN-LBS	MAX. SHEAR LBS
.4300E+05	-.1104E+07	.6500E+05	.2277E+00	-.1104E+07	.4300E+05

APPENDIX C

LIMITED SENSITIVITY STUDY

In this sensitivity study, the three variables are the unit weight (γ), the average undrained shear strength (c), and the strain corresponding to one-half the average undrained shear strength (ε_{50}). The example problem from Appendix B is used, and these three values are doubled and halved. The resulting moment curves are shown on the following pages.

This limited sensitivity study shows that the unit weight has a negligible effect on the pile response; however, the average undrained shear strength and the strain corresponding to one-half the average undrained shear strength have a considerable effect when they are doubled and halved. When the shear strength is increased, the maximum moment is decreased and the moment diagram is shifted up. When the strain corresponding to one-half the shear strength is increased, the maximum moment is increased and the moment diagram is shifted down. The fact that increasing the shear strength and the strain at one-half the shear strength had opposite effects is to be expected. By increasing the shear strength only, the soil becomes "stiffer," and inversely by increasing only the strain at one-half the shear strength, the soil becomes "less stiff."

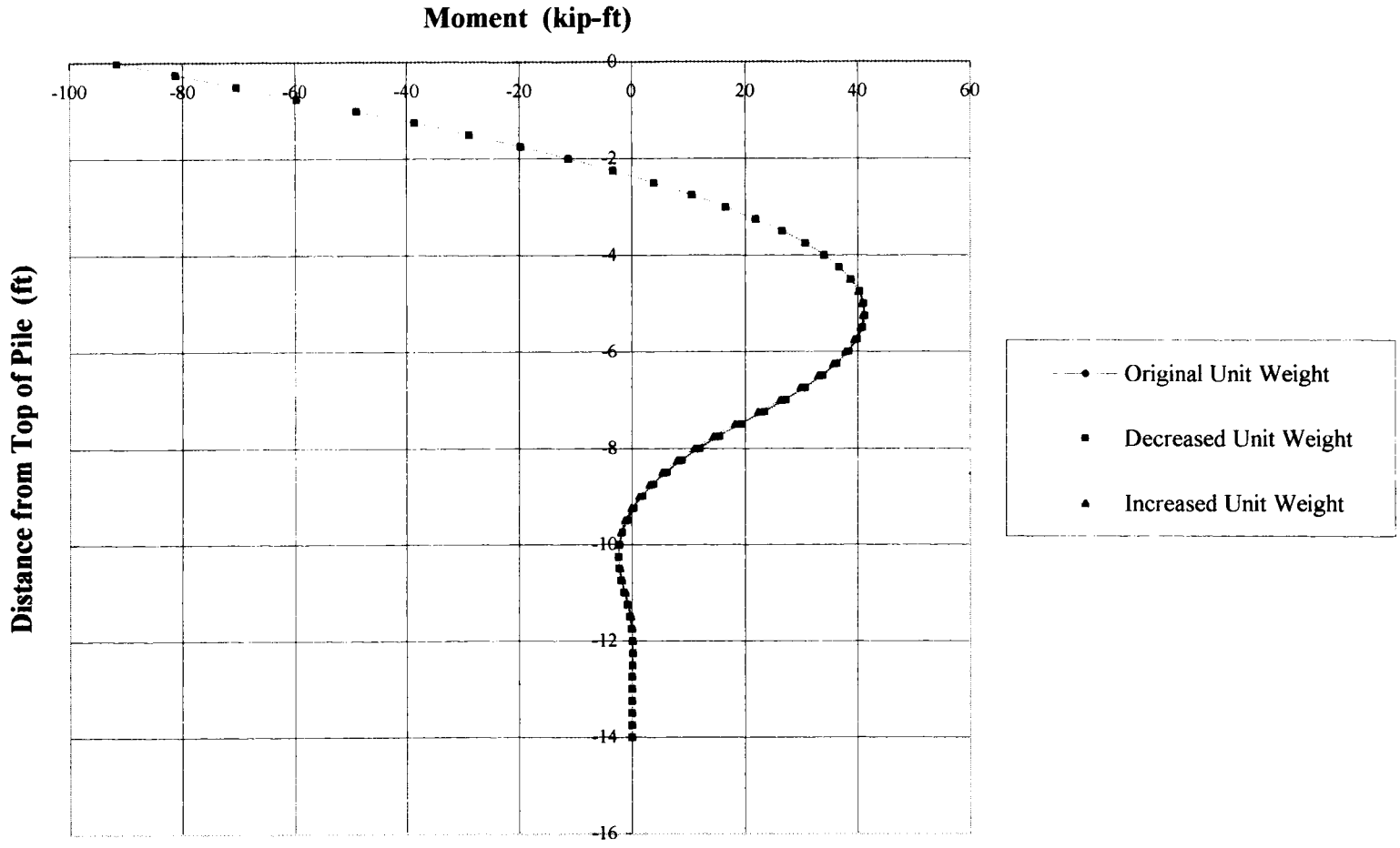


Figure C-1. Moment vs. Distance from Top of Pile with variations in unit weight

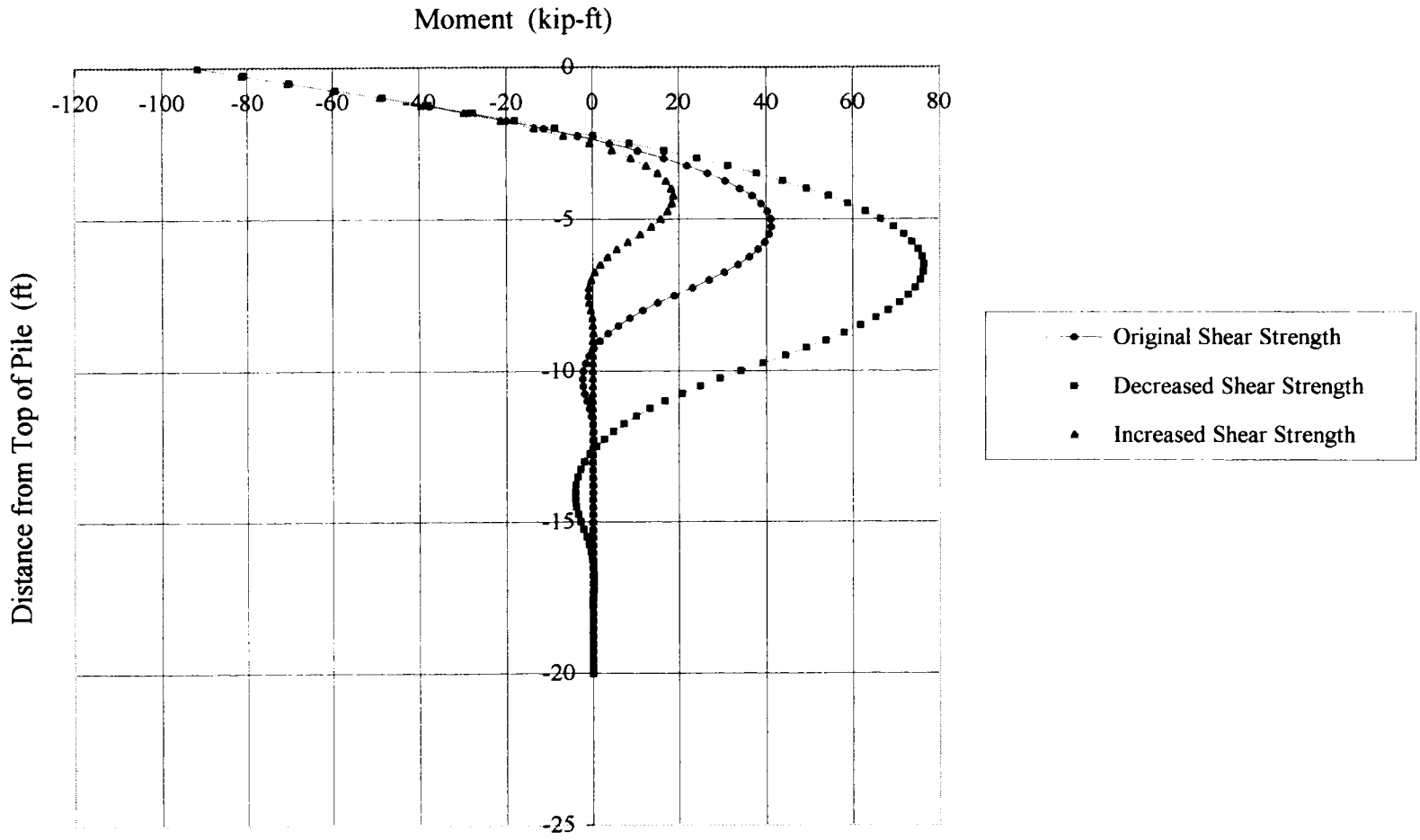


Figure C-2. Moment vs. Distance from Top of Pile with variations in average undrained shear strength

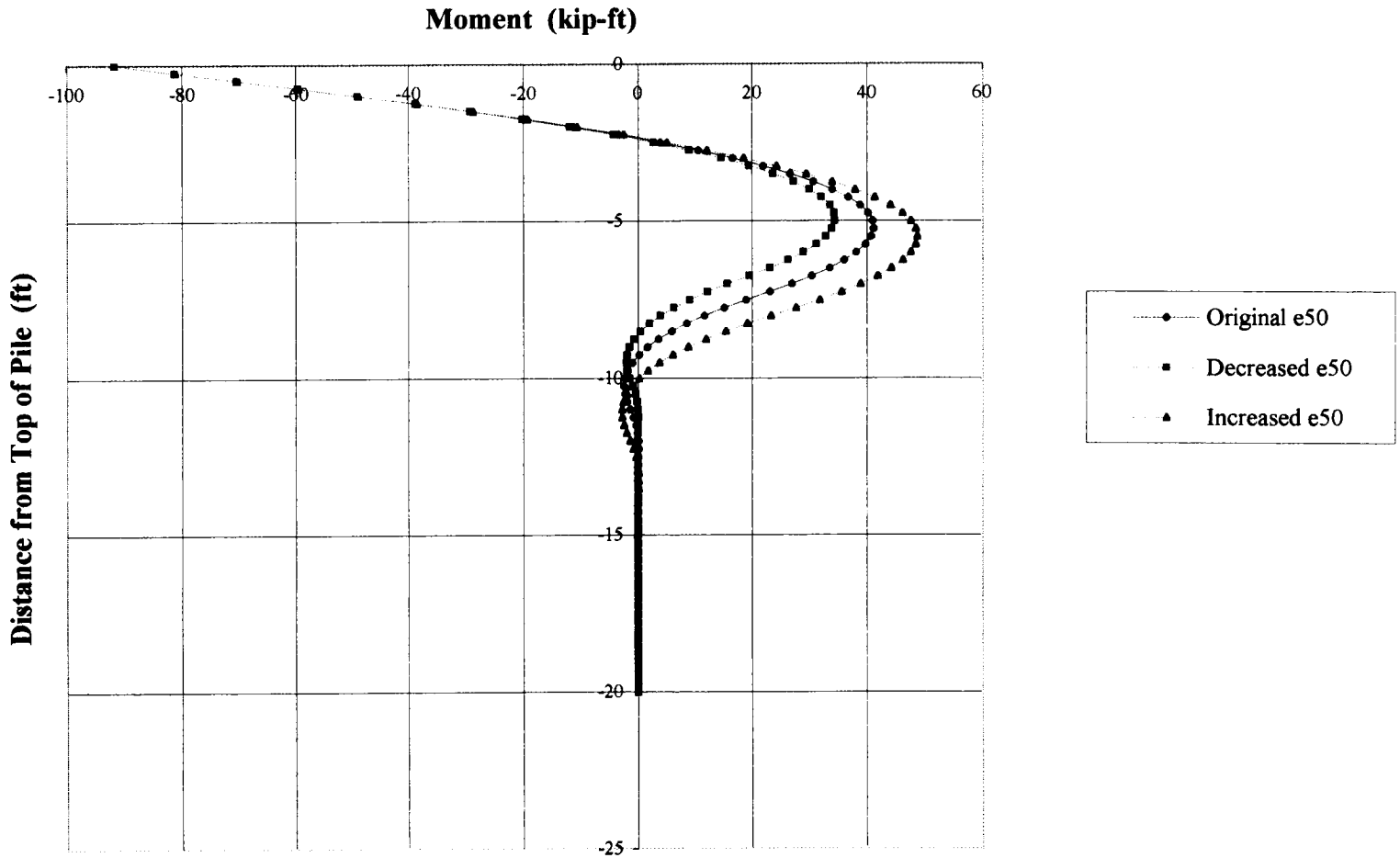


Figure C-3. Moment vs. Distance from Top of Pile with variations in strain corresponding to one-half the average undrained shear strength

VITA

Christopher Allen Myers was born in Aberdeen, Washington on September 17, 1973 and was raised in Blaine, Tennessee. After graduating from Rutledge High School in May, 1991, he attended The University of Tennessee, Knoxville. He received a Bachelor of Science degree in Civil Engineering from The University of Tennessee in May, 1997. He began graduate school the same month and received his Master of Science degree in Civil Engineering in December, 1998.