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BEHAVIOR CHARACTERISTICS OF DRIVEN H-PILES WHEN SUBJECTED TO LATERAL LOADS

A Thesis

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Master of Science

Degree

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Eric Michael Bonanno

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ABSTRACT

The Tennessee Department of Transportation (TDOT) has routinely designed and constructed jointless bridges with integral abutments for many years. The particular concern with jointless bridges is the effects of thermal expansion and contraction on piles supporting integral abutments. The University of Tennessee, Knoxville, with funding from TDOT, is involved in a research project to investigate the behavior of laterally loaded piles supporting integral abutments.

The full scale lateral load tests in this research project will involve four single piles supporting a simulated integral abutment. Tests on two of those abutments are complete, and those results form the basis of this thesis. The simulated abutments were cast on each of the steel H-piles, which were instrumented with weldable strain gages and pressure sensors along its length. To simulate the effects of thermal loading, the abutments were slowly subjected to lateral loads. The testing program consisted of repeated field tests that achieved ground level deflections of one-half inch and one inch, as well as tests involving cyclic loading and loading to a point where possible failure warranted the termination of the testing. The deflections and loads were monitored during testing so that the load, deflection, and moment were known at the ground level.

The behavior of the piles was predicted from elastic theory. A best fit sixth order polynomial was used for the moment equation. This equation was differentiated to obtain a shear and pressure equation, and integrated to obtain an equation for the deflected

shape. Although some inaccuracies in the results were apparent, this analysis method predicted results that were within a reasonable degree of accuracy.

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CHAPTER 1

INTRODUCTION

The idea of jointless bridges evolved from concerns that expansion joints are problematic. The utilization of continuous construction eliminates the high costs of installation and maintenance of expansion joints. With the introduction of the Hardy Cross Method of moment distribution in 1930, continuous bridge design became practical (1). Through the growth and change of continuous bridge design the development of integral abutments was established. Using integral abutments with continuous bridges completes the idea of a truly jointless bridge.

1.2 HISTORY

The concept of jointless bridges has been used since the early 1930's. The Ohio Department of Transportation (ODOT) was one of the first agencies to routinely use continuous construction of multiple span bridges under the direction of J.R. Burkey (2). The first bridge designed by the ODOT was an open spandrel reinforced concrete rib arch structure with continuous decks on flexible columns. Ohio continued developing continuous bridges by using riveted field splices in steel stringer bridges. The first field butt-welded splice was used in 1934, and high strength bolts were used as early as the late 1950's (3). For the last 65 years the Ohio department of transportation has been utilizing continuous construction for all types of bridges, from reinforced concrete and steel to prestressed concrete I-beams and box beam bridges.

Before Cross' method of moment distribution allowed the analysis of continuous beams and frames to be simplified, designers were designing multiple span bridges as a series of simple span bridges. As continuous bridge design progressed, designers started using integral abutments. The development of these integral bridges was mainly based on engineering discretion, observations and experimentation.

Although the method of moment distribution enabled the design of continuous spans, the hand calculations were tedious and very time consuming. With the advent of the computer, the utilization of jointless bridges with integral abutments became the preference of many state transportation departments. The Federal Highway Administration (FHWA) reported, in 1987, that 28 states and the District Construction Office of FHWA Region 15 were using jointless bridges (4). Most states design jointless bridges for short to medium span bridges, with a few states designing bridges up to 800 feet. The design criteria varies from state to state due to lack of any formal design guidelines. A list of these states and their design practices are given in Table 1 (5).

1.3 TDOT PRACTICES

One of the agencies that has adopted jointless bridge with integral abutments as standard practice is the Tennessee Department of Transportation (TDOT). TDOT current design practices are that no intermediate expansion joints are used except where absolutely essential. Currently, jointless bridges are being designed for lengths up to 400 feet for steel and up to 800 feet for both concrete and prestressed construction (5). Tennessee has designed and built a dual 29 span bridge, over half a mile in length, with

Table 1. Summary of Current Practice of Agencies Using Integral Abutments.

State	Design Considerations		Design Assumptions & Details				
	Piling Stresses Due to Lateral Movement Are Calculated	Criteria for Maximum Length for Bridges With Integral Abutments	Pile Head	Pile Cap	Approach Slab	Backfill	Comments
Alaska	Only for long bridges.	On the basis of experience Steel: ≤ 300 ft Concrete: ≤ 400 ft Prestressed: ≤ 416 ft	Hinge	-	No	Granular Material	Bridges with integral abutments may be constructed with spread footings or pilings. As longer bridges without expansion joints are found to be with out problems, the length limit has increased to 400 ft for concrete bridges
Arizona	No	On the basis of experience Steel: ≤ 253 ft Concrete: ≤ 330 ft Prestressed: ≤ 404 ft	Hinge	No	Tied to abutment with dowels & moves back & forth with superstructure	Cohesive material	
California	Piles are driven into pre-drilled holes & stresses due to lateral movement are neglected.	On the basis of experience Steel: ≤ 240 ft Concrete: ≤ 260 ft Prestressed: ≤ 150 ft	Parially Restrained	-	-	Pervious	
Colorado	No	On the basis of experience Steel: ≤ 200 ft Concrete: ≤ 400 ft Prestressed: ≤ 400 ft	Hinge	No	For bridge lenght > 200 ft, use approach slab	Granular	No problem in skew; use predrilled oversized hole
Connecticut	No	On the basis of experience Steel: ≤ 200 ft Concrete: - Prestressed: -	Fixed	-	-	-	As of May 1983, only one integral abutment has been designed & constructed. The design of this 245 ft long two-span continuous bridge was beased on information received from South Dakota DOT
Georgia	No	Based on total anticipated lateral movement of ≤ 2 in. Steel: ≤ 300 ft Concrete: ≤ 600 ft Prestressed: -	Free translation, free rotation, roller	No	Expansion joint between approach slab & bridge slab	Roadway fill	Integral abutments have been used only at sites where steel H-piles are suitable. Steel H-piles are placed so that they bend about their weak axis
Idaho	Only for those that involve some unique feature that would warrant such calculations.	Based on FHWA guidelines & the state's experience Steel: ≤ 200 ft Concrete: ≤ 400 ft Prestressed: ≤ 400 ft	Hinge	Rigid pile cap	Expansion joint specified between rigid pavement & approach slab; no special treatment specified for flexible pavement	Free draining granular material	Assume that passive earth pressure at abutments tends to restrain movement & reduce deflections from calculated values. Skewed 3-span steel grider bridge with integral abutments was built; rotational forces from lateral earth wall caused failure in pier anchor bolts on exterior girder
Indiana	No	Steel: - Concrete: ≤ 150 ft Prestressed: -	Hinge	Embed piles only 1 ft into cap	20 ft approach slab integrally attached to bridge	Select granular fill	Only vertical piles are used with integral abutments. When bridge skew > 30°, length limit for concrete bridges is ≤ 100 ft. Integral abutments have been used for many years with no adverse experience. On longer bridges, the integral connection is eliminated, substituting a neoprene bearing pad or expansive device, use alternating vertical & battered piles in the cap & still neglect lateral forces on the piles.
Iowa	Yes	Based on allowable bending stress of 55% of yield plus 30% overstress. Moment in pile found by a rigid frame analysis considering relative stiffness of the superstructure & the piling. Assume piles to be 10.5 ft & neglect soil resistance. Analysis showed that allowable pile deflection was about 0.325 in Steel: - Concrete: ≤ 265 ft Prestressed: ≤ 265 ft	Fixed	Neglect	Neglect	Roadway fill	Conservative design

Source: Wolde-Tinsae, Amde M., and Greimann, Lowell F., "General Design Details for Integral Abutment Bridges." Civil Engineering Practice, Vol. 3, No. 2. Fall, 1988.

Table 1 (continued)

State	Design Considerations		Design Assumptions & Details				
	Piling Stresses Due to Lateral Movement Are Calculated	Criteria for Maximum Length for Bridges With Integral Abutments	Pile Head	Pile Cap	Approach Slab	Backfill	Comments
Kansas	No	Based on experience Steel: ≤ 250-300 ft Concrete: ≤ 500 ft Prestressed: -	Hinge	Pile caps not used	Use slab support at backwall & pavement rests on slab with 30 ft from end of wearing surface	Backfill compaction has settlement just off end of bridge	Have used integral abutments for cast-in-place bridge structures for many years & have encountered no difficulties. Expect to increase length limits in the future
Kentucky	No	Steel: - Concrete: ≤ 300 ft Prestressed: ≤ 300 ft	Fixed or partially restrained	-	No special treatment with flexible pavement	Special granular backfill specified	Piles are placed in holes prebored for a distance of 8 ft below bottom of pile cap
Missouri	No	Based on experience of Missouri & other states (mainly Tennessee) Steel: ≤ 400 ft Concrete: ≤ 400 ft Prestressed: ≤ 500 ft	Hinge	On extreme skews (± 40°), use shear key on bottom of pile cap to prevent lateral movement of pile cap	-	Roadway fill	Required a minimum of 15 ft pile lenght to permit flexure of pile
Montana	No	Based on experience & engineering judgement Steel: ≤ 300 ft Concrete: ≤ 350 ft Prestressed: ≤ 300 ft	Hinge	-	Not fixed to abutment	Granular Material	≤ 30° skew
North Dakota	No	Steel: ≤ 350 ft	Fixed	Abutment wall is pile cap & is reinforced to resist bending below super-structure	Assume approach slab has no effect	Select granular fill	≤ 30° skew
Nebraska	No	Based primarily on past experience & recommendations from other agencies Steel: ≤ 200 ft Concrete: ≤ 300 ft	Hinge	-	-	Select granular fill	≤ 15° skew Procedures for determining piling numbers are same as for conventional abuments. Piling are rotated to provide bending about weak axis. Presently, only steel H-piles are used in integral abutments & also substantial anchorage between girder & abutment are provided. Wings on integral abutments are not attached to the abutment in order to reduce resistance to rotation. This is accomplished by using a bond breaker between the abutment & wing, & designing the wing as a stand alone structure
New Mexico	No	-	Parially Restrained or fixed	-	Used on some bridges & not on others	Do not use specified backfill anymore	Have built bridges with 15° skew; skew angle neglected
New York	No	Steel: ≤ 300 ft Concrete: - Prestressed: ≤ 400 ft	-	-	Approach slab should be 20 ft long max & its end should be parallel to the skew. Construction joint provided between approach & bridge slabs	Granular fill behind backwall & wingwalls	New York has tentative integral abutment bridge design guidelines that list the design parameters that must be satisfied
Ohio	No	Based on experience & engineering judgement Steel: ≤ 300 ft Concrete: ≤ 300 ft Prestressed: ≤ 300 ft	Hinge	Pile cast in pile cap 2 ft	Tie approach slab to abutment	Granular Material	Oil country pipelines not used in integral abutments because they are stiffer tha H-piles about weak axis. Integral abutment bridges built only with zero skews

Source: Wolde-Tinsae, Amde M., and Greimann, Lowell F., "General Design Details for Integral Abutment Bridges." Civil Engineering Practice, Vol. 3, No. 2. Fall, 1988.

Table 1 (continued)

State	Design Considerations		Design Assumptions & Details				
	Piling Stresses Due to Lateral Movement Are Calculated	Criteria for Maximum Length for Bridges With Integral Abutments	Pile Head	Pile Cap	Approach Slab	Backfill	Comments
Oklahoma	No	Based on allowable lateral movement of 0.5 in. Steel: ≤ 300 ft Concrete: ≤ 300 ft Prestressed: ≤ 300 ft	Partially restrained	-	-	-	Integral abutments only with zero skew
Oregon	No	Based on engineering judgement. Length varies depending on location in state Steel: - Concrete: ≤ 350 ft Prestressed: ≤ 350 ft	Hinge	Pile cast in pile cap 1 ft	Approach slab to pile cap	Granular	
South Dakota	Yes	-	Fixed	-	Tied to bridge to prevent erosion of shoulder	Granular	
Tennessee	No	Based on experience Steel: ≤ 400 ft Concrete: ≤ 800 ft Prestressed: ≤ 800 ft	Hinge	-	Construction joint between abutment backwall & approach slab	Granular	No bridge deck expansion joints are to be provided unless absolutely necessary
Utah	No	Steel: ≤ 300 ft Concrete: - Prestressed: ≤ 300 ft	Hinge	-	Expansion joint between approach & bridge slabs	Granualr	Steel piles used primarily through granular material over bedrock
Virginia	No	Steel: ≤ 242 ft Concrete: - Prestressed: ≤ 454 ft	Hinge or fixed	Uniform width & parallel to bridge skew	No approach slab	1.5 ft porous backfill with 0.5 in. dia. pile underdrain	Max skew 10°; relatively small movement at each abutment (± 0.75 in)
Vermont	No	Steel: ≤ 150 ft Concrete: - Prestressed: -	Partially restrained or fixed	Rigid pile cap	Approach slab anchored to abutment	No special treatment	≤ 30° skew
Washington	No	Mainly based on past experience Steel: - Concrete: ≤ 400 ft Prestressed: ≤ 400 ft	Hinged	Designed as cross beam on simple supports	Approach slab attached to abutment with allowance for expansion	Granular backfill, earth pressure applied normal to abutment	
Wisconsin	No	Steel: ≤ 200 ft Concrete: ≤ 300 ft Prestressed: ≤ 300 ft	Fixed	Designed as reinfoced continuous beam over pilings	Designed for vertical load only	Granular	≤ 30° skew for slabs; 15° skew for prestressed or steel griders
Wyoming	No	Based on various studies, reports, etc. Steel: ≤ 300 ft Concrete: ≤ 500 ft Prestressed: ≤ 500 ft	Plastic hinge	Assumed to be a mass attached to end grider	-	Granular	
FHWA, Region 15	Yes	Steel: - Concrete: ≤ 270 ft Prestressed: ≤ 300 ft	Hinge or partially restrained	Pile cast in pile cap 1 ft	-	Pervious	

Source: Wolde-Tinsae, Amde M., and Greimann, Lowell F., "General Design Details for Integral Abutment Bridges." Civil Engineering Practice, Vol. 3, No. 2. Fall, 1988.

expansion joints only at the abutments (4). As the lengths of bridges get longer and longer, the effects of lateral movements need to be addressed.

The analysis of laterally loaded piles is difficult since there is an uncertainty of the soil-pile interaction. This uncertainty is based on the fact that the deflection of the pile is a function of the soil response, and on the other hand, the soil response is a function of the pile deflection. Since the soil response is a non-linear function of the pile deflection, the rules of static equilibrium cannot be used to determine the response of the system. Tennessee's approach for determining the pile response is the use of the computer analysis program "COM624P - Laterally Loaded Pile Analysis Program For The Microcomputer, Version 2.0" (1). Since an integral abutment is subjected to both axial and lateral loads, COM624P obtains a solution by using a differential equation for the deflection of a beam-column. A group of plots of soil resistance (p) versus pile deflection (y) is produced for a given pile from experimental equations and the input of soil properties. The pile properties and loading situations need to be entered also. Through an iterative process, COM624P will reach a solution. The moment, deflection and pressure versus depth plots are produced from the program's output (6).

Thermal effects on integral abutments are among the main concerns of TDOT. Each bridge design begins with the calculation of the thermal forces acting on the bridge. A temperature range and the thermal coefficient of expansion is chosen based on the construction material of the structure. For concrete structures, a 25 to 95° F temperature range, 6.0×10^{-6} /°F coefficient of thermal expansion and 0.505 inches per 100 feet of total movement is used. Steel structures use a temperature range of 0 to 120° F, a

coefficient of expansion of 6.5 x 10^{-6} /°F and a total movement of 0.936 inches per 100 feet. The total movement is applied equally at the two abutments of the structure. Abutments with less than two inches of total movement and not restrained against movement are constructed integrally at the abutment. If the abutment is restrained against movement and rotation, an expansion joint is required for total movements greater than 0.25 inches. For abutments with less than 0.25 inches of total movement, the abutment is constructed integrally regardless of the support conditions (5).

1.4 PURPOSE

Due to unpredictability in the behavior and the complexity of the design of jointless bridges with integral abutments, TDOT is sponsoring a research project with the University of Tennessee, Knoxville to study the behavior of laterally loaded piles supporting integral abutments. The effects of thermally induced stresses to both the pile-abutment interface and the interaction between the pile and soil are the two main areas of interest in this research of integral abutments.

The field research involves the testing of four simulated integral abutment systems. The first two of these abutment systems are to be tested in virgin soil, while the last two systems are to be tested in engineered fill material. Each system involves a driven HP 10 x 42 pile, that is instrumented, with an abutment cast on top. The system is then subjected to a slowly applied lateral load to simulate thermal loading. A total of six tests are conducted in each direction; three of these tests are to a ground level deflection of one-half inch and three of these tests are to a ground level deflection of one inch.

Testing has been completed for the first two systems and testing of the third system is in progress.

The integrity of the abutment when subjected to lateral loads is a very important concern in the design of integral abutments. When designing an integral system, TDOT uses COM624P to determine the displacement that occurs when an HP 10 x 42 pile, with fixed head conditions, reaches plastic rotation. The plastic moment is then calculated and used for all load combinations that have a greater displacement than the displacement that corresponds with the plastic moment. The bearing capacity of the concrete is then calculated, and the appropriate concrete strength (f'_c) is then determined for the abutment (1). In the field research, cracking at the pile-abutment interface has occurred but does not seem to have affected the load carrying capacity. However this cracking could affect the weatherability of the abutment and cause the integral abutment to deteriorate.

Another key element in the design of integral abutments is the pile-soil interaction. By predicting the deflected shape of a pile, moments can be calculated along the depth of the pile. The moment versus depth plot is then used to determine the unbraced length of the pile. This unbraced length is defined as the distance between the first two points of zero moment on the moment versus depth curve. TDOT uses COM624P to predict the deflected shape as well as to plot the moment versus depth curve. COM624P uses the soil properties, pile properties and expected thermal movement to generate these plots. An interaction diagram is then used to determine the capacity equations for combined axial load from the abutment and bending load due to

thermal expansion. The unbraced column is compared to various factored load conditions to verify that it has adequate strength (6).

The purpose of this thesis is to determine the response of the pile-soil interface due to lateral loads. From the measured strains at a particular lateral displacement, moments were calculated and plotted versus their depth. A moment equation was then determined by using a best fit sixth order polynomial. The load distribution and deflected shape were predicted from this equation using elastic theory. Static equilibrium conditions were used to determine the accuracy of the predictions.

CHAPTER 2

PREVIOUS STUDIES

Most of the recent research regarding laterally load piles with integral abutments has been from either laboratory tests or computer modeling. Some research involving full scale field tests of laterally loaded piles, similar to that currently being performed by the University of Tennessee, has also been reported. This research is reviewed in the following section.

2.1 CALIFORNIA STATE

Research involving full scale field tests of laterally load piles was performed in northern California (7). The intent of this research was to determine lateral load-deflection behavior of vertical piles, compare theoretical values with test results, and to determine experimental values for the soil moduli. Three different bridge sites were investigated in this research.

Free head pile tests were performed at all three sites, while a fixed head pile test was only performed at one of the sites. Two of the test sites used 10.75 inch outside diameter steel pipe piles, and the other test site used steel H-piles. The piles were considered to be isolated and would not be influenced by adjacent piles. All the piles were driven into pre-drilled eighteen inch diameter holes that were twelve to twenty feet deep and backfilled with pea gravel. These holes were used to reduce friction on the pile. The embankments consisted of twelve to twenty feet of compacted fill in the upper

portion and in-situ soil conditions in the lower portion. The soil characteristics ranged from silty-sand to clay.

The lateral deflections that occurred at the ground surface were measured by deflection transducers. Electrical resistance strain gages were used to measure strains along the portion of the pile embedded in pea gravel. Bending moments were then calculated from the recorded strains. The slope at the top of the pile was measured using an electronic level. The instrumentation was the same for all the test piles.

The free head tests were performed on the piles before construction of the abutments. Each test involved two piles by jacking horizontally between them. The first test allowed the load to vary while a constant deflection was maintained between the two piles. Incremental deflections of ½, 1, 1½, and 2 inches were held for two hours for each increment. The second test held the load constant and allowed the deflection to vary. Load increments of 5, 10, 15, 20, and 25 kips were held constant for a minimum of two hours or until the pile movement was less then 0.010 inches per hour. The fixed head test was performed after the construction of the superstructure. An applied load from the effects of post-tensioning and the dead load of the bridge was used. The total post tensioning force of the concrete structure was 9,200 kips. The lateral deflections and the strains were recorded after post-tensioning, but before the falsework was removed. The strains were also measured after the falsework was removed.

The test results from both the free head and the fixed head were used in determining the effective length and experimental values for the soil moduli. It was determined that a fixed head pile was twice as stiff as a free head pile at a deflection of

0.25 inches. The depth that effectively transfers the lateral load from the pile to the soil with no substantial lateral deflection, and alternatively, no substantial soil pressure, is known as the effective length. The effective length was found to vary from 8 to 12 feet for lateral loads of 9 to 16 kips. The theoretical solutions were based on the theory of a beam on an elastic foundation. This theory uses a continuous flexural member, with a stiffness of EI, supported by independent springs, with a stiffness of k, spaced infinitely close along the length of the member (8). The stiffness of the springs was taken as the soil moduli, which were determined to vary from 10 to 65 lb/in^3. The test results compare reasonably well with the theoretical solutions.

2.2 IOWA STATE

In 1987, Iowa State University performed tests on friction H-piles to determine if lateral loads from thermal expansion and contraction influence the vertical load capacity of the piles in integral abutments (9). The main focus of this research was to develop a reasonable method of designing integral abutments. The test results were also used to verify results from a finite element modeling program. The full scale tests involved two free head piles and three loading conditions.

The tests were performed on two HP 10 x 42 piles that were 50 feet long and embedded to a depth of 40 feet. The soil on the test site consisted of weathered and unweathered glacial till that ranged from stiff sandy, silty clay to hard silt. Each pile was instrumented with electrical resistance strain gages placed one inch from the edge of the inside flange spaced 12 inches apart. The gages were protected with waterproofing and a

sheet metal conduit. Direct current displacement transducers were used to measure the displacements of the pile, and load cells were used to measure the applied loads. A data acquisition system recorded the time, loads, displacements, and pile strains for each test.

The first loading condition was an axial load test. The axial load test was used to determine soil characteristics involving skin friction, vertical displacement and bearing capacity. An axial load was applied in 20 kip increments and was held until a settlement rate of 0.01 in/hr was satisfied. The ultimate load that could be applied before a bearing capacity failure occurred was determined to be a load of 280 kips.

The second loading condition was a lateral load test. The displacement was held constant while the lateral load was allowed to vary. For this test, the orientation of the pile was such that bending occurred about the weak axis. Lateral displacements at the ground level were applied in 0.10 inch increments until a maximum displacement of two inches was reached. It was determined that a lateral displacement of 2.06 inches at the ground surface occurred at a load of 25 kips applied 20.5 inches above the ground. The relationships between lateral resistance and lateral displacement (p-y curves) were developed from the results of the lateral load test. These experimental curves compared reasonably well with theoretical p-y curves.

A combined axial and lateral load was used for the third condition. The combined load test involved applying an axial load to a laterally displaced pile. The orientation of the pile in this test was such that bending occurred about the strong axis. First, a lateral displacement of 1.93 inches at the ground surface was applied with a lateral load of 19.9 kips. The ultimate axial load of 280 kips was then applied to the pile. After the axial

load was applied, the lateral displacement increased to 2.40 inches, while the lateral load decreased to 14.7 kips.

Two alternate design methods were recommended for determining the capacity of the pile as a structural member. The first method considers the pile as an equivalent cantilever beam with a lateral end deflection. This end displacement generates elastic moments that neglect any plastic redistribution. Since the ductility of the pile material is neglected, failure is assumed to occur when the elastic yield of the pile material is reached. The second method relies on the pile cross section to be ductile and capable of plastic redistribution. To accommodate plastic hinge rotation, the steel section must be compact and be braced to prevent any type of buckling.

These test results were used to verify the finite element modeling techniques of the computer program IAB2D. IAB2D allows the pile to be modeled as a two-dimensional nonlinear beam element and the soil to be modeled as nonlinear springs. The measured strains were converted into bending moments and plotted versus depth and compared to the moment versus depth plots from the computer program. The moment diagrams from IAB2D displayed slightly higher moment values than those developed from the test data. In comparing the load-deflection curves, the computer program results matched those generated from the field testing relatively well.

CHAPTER 3

TEST PROGRAM

3.1 INTRODUCTION

The University of Tennessee established a full scale testing program to study the effects of thermal movements in integral abutments. A total of four piles are to be tested for this research. Two piles were tested in undistributed soil and the other two will be tested in engineered fill. The testing on the first two piles has been completed, and testing has started on the third pile. Preparations for testing of the fourth pile have also begun. The field tests are located at the construction site of the Dutchtown Road interchange on Pellissippi Parkway, approximately fifteen miles west of the University of Tennessee, Knoxville. The test setup, instrumentation, and test regimen are discussed in the following sections.

3.2 TEST SETUP

The test setup involves a pile, an integral abutment, load frame, and reaction beam. All four piles were steel HP 10 x 42 piles forty feet in length. The piles were driven 38 feet deep, leaving one foot to be embedded into the abutment and one foot exposed for instrumentation. Once a pile was driven, the abutment was cast, using Class A concrete ($f'_c \geq 3000$ psi), so that bending would occur about the strong axis. Reinforcing steel was placed in the abutment in accordance with a typical integral abutment designed by the Tennessee Department of Transportation. To provide a typical

abutment loading condition, concrete ballast was placed on the abutment to produce an axial load of approximately 65 kips. The simulated thermal loading was applied by using a hydraulic jack and a Dywidag post-tensioning threaded bar. The Dywidag bar was placed in a three inch diameter pipe that was cast into the abutment slab and which extended approximately twenty feet to the load frame. A plate and nut were placed on both ends of the bar; one end allowed the bar to pull the abutment, and at the load frame end, the load was applied by jacking between the load frame and plate. A reaction beam was used to obtain a rotational stiffness similar to that of a typical integral abutment. This was accomplished by allowing the abutment slab to bear on a roller support positioned on the reaction beam. The test setup is shown in Figure 1.

3.3 INSTRUMENTATION

The instrumentation of the piles was completed at the university before the piles were taken to the site and driven. Each pile was instrumented with weldable strain gages and pressure sensors. One strain gage was placed on the inside of each flange, approximately three inches from each edge. The use of four strain gages eliminates any torsional and biaxial bending effects that might occur. The pressure sensors were placed in holes cut in the center of the flange face that extended into the web approximately 2.5 inches. This allowed the face of the sensors to be flush with the face of the flange. In order to accommodate testing in both directions, sensors were placed on both sides of the pile. To minimize bending detected by the sensors, RTV-108 was used to cushion the space between the sensor and pile.

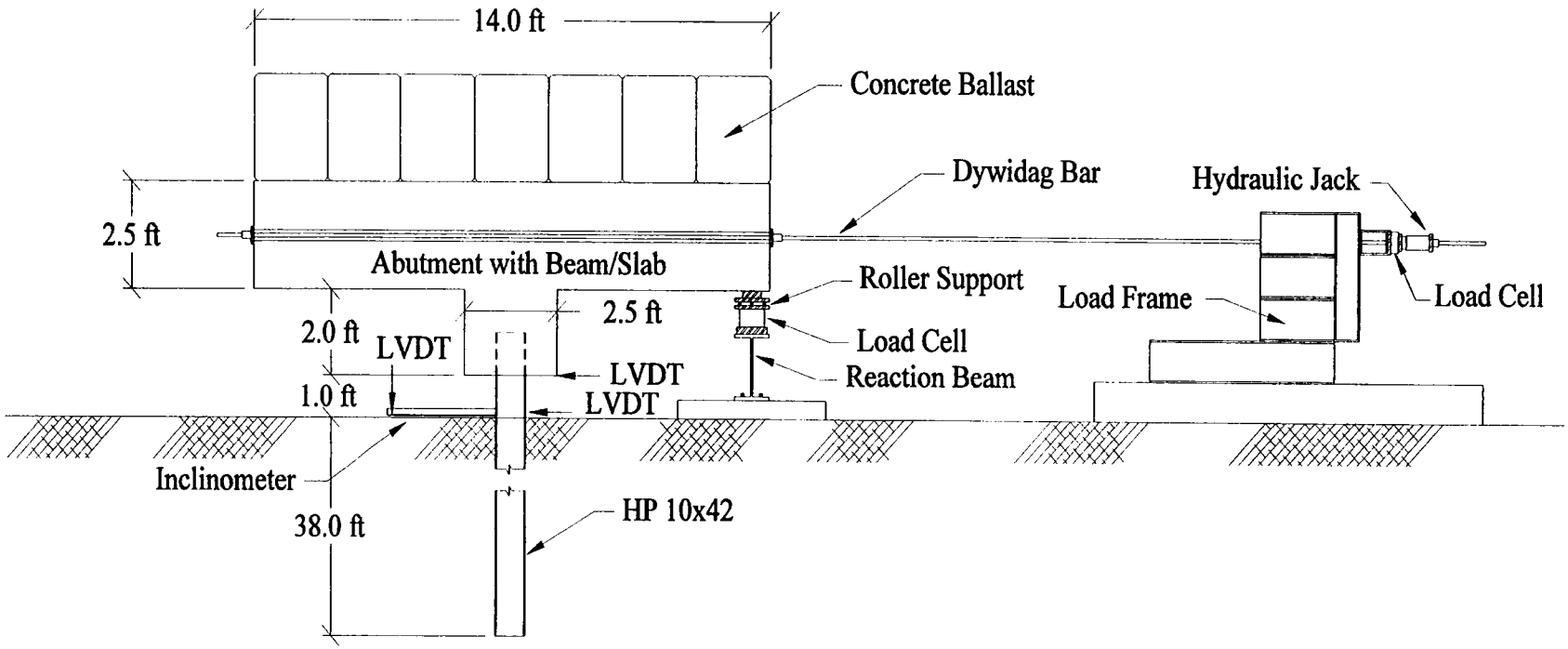


Figure 1. Test Set-up for Field Tests.

The first pile had strain gages placed every eighteen inches starting at the ground surface and continuing to a depth of 19.5 feet, for a total of fourteen stations. The pressure sensors were also spaced at eighteen inches. The pressure sensors started at a depth of 5'-3" and ended at a depth of 12'-9"on one side of the pile and went from a depth of 6'-9" to 14'-3" on the other side. The instrumentation changed slightly for the last three piles. There were two extra strain gage stations added, one at a depth of nine inches and the other at a depth of 27 inches. The pressure sensors started at a depth of 5'-3" and continued to a depth of 12'-9" on both sides of the pile. The locations of the strain gages and pressure sensors are shown in Figure 2. The addition of two strain gages on the outside of each flange above the web just below the pile-abutment interface occurred late in the testing of the second pile. These gages were added before testing began on the third pile.

After all the strain gages and pressure sensors were installed, the lead wires were labeled according to the flange they were on, A, B, C, or D, and the depth of the station, beginning with 1 at the ground surface. The wires were then taped to the fillet of the pile and waterproofed with a brushed on rubber coating. A sheet metal shield was then welded into place diagonally between the web and the flange to protect the instrumentation. The first pile used eighteen gage galvanized sheet metal, and sixteen gage ungalvanized sheet metal was used on the other three piles. To ensure that the sheet metal shield stayed attached to the pile during driving, a one-quarter inch thick triangular steel plate was welded to the pile at the bottom of the sheet metal. The gages and sensors

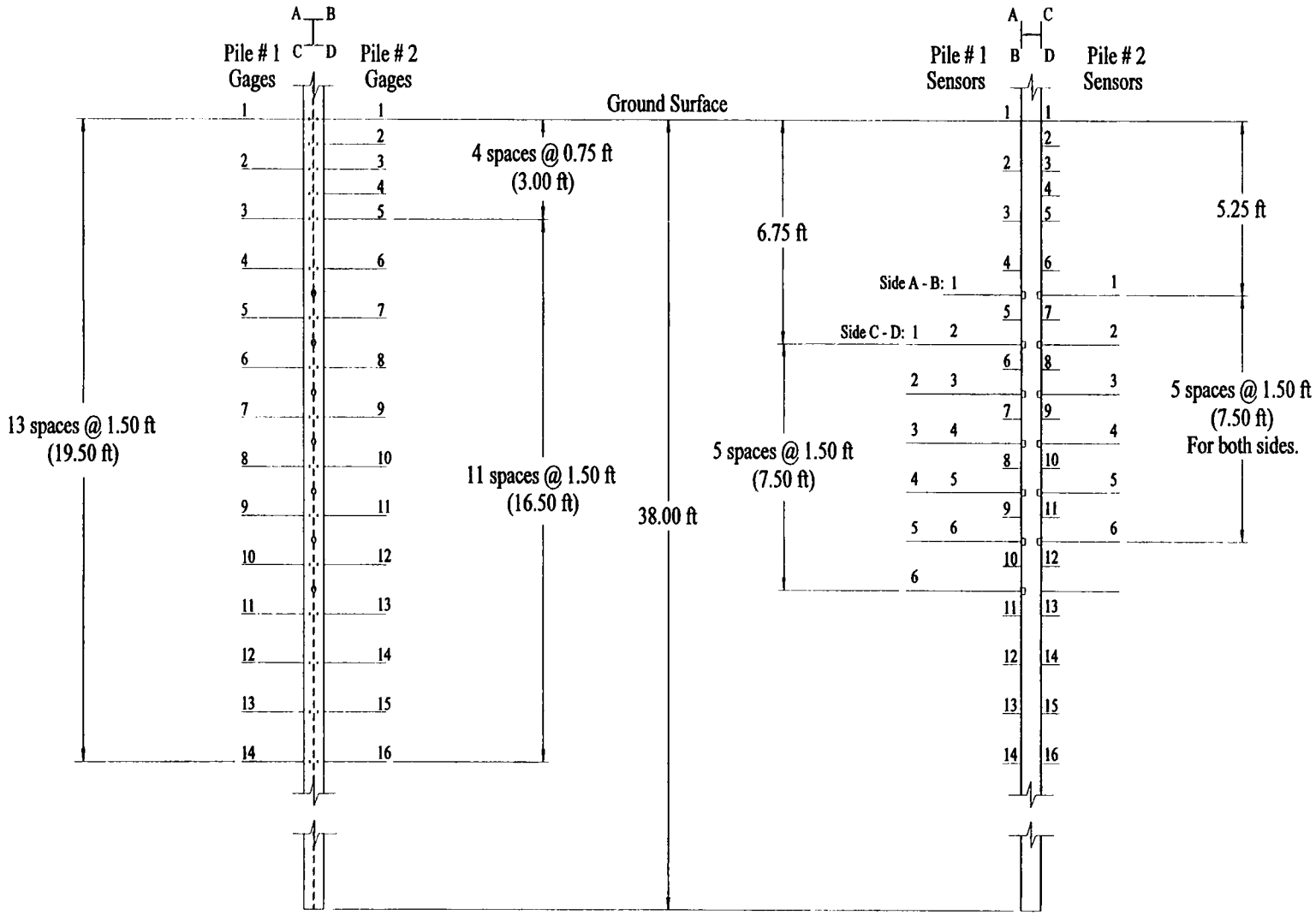


Figure 2. Location of Gages and Sensors.

were partially protected from the vibrations, that occur during the driving process, by injecting insulating foam into the space between the pile and the sheet metal.

The lateral deflections of the pile were measured by three Linear Variable Differential Transformers (LVDTs). One LVDT was placed at the ground surface to measure the pile deflection. The second LVDT measured the abutment deflection just above the pile-abutment interface. In order to estimate the rotation of the pile at the ground surface, the third LVDT measured the vertical deflection of the inclinometer. Two 100 kip load cells were used to measure the lateral load and the reaction between the abutment slab and reaction beam. The lateral load was measured at the load frame by placing the load cell in series with the load frame, hydraulic jack, and the plate on the Dywidag bar. The reaction was measured by placing the load cell between the reaction beam and the roller support. The locations of the LVDTs and load cells are also shown in Figure 1.

3.4 TEST REGIMEN

In typical bridges, thermal expansion and contraction occur every day. As the temperature rises during the day, thermal expansion gradually increases until reaching the high temperature. Thermal contraction begins as the temperature starts to decrease. This loading cycle was an important aspect in developing the loading regimen for the field tests of simulated integral abutments.

Twelve tests were run on each pile, six in each direction. For each direction, the first three tests obtained a ground level pile deflection of one-half inch, while the next

three tests obtained a ground level pile deflection of one inch. After these twelve tests were completed, other tests including cyclic loading and loading to a deflection where potential failure of the abutment forced the termination of the testing.

To simulate the thermal expansion in a typical bridge, the loading was designed to be slow and gradual. The loading was controlled by deflection, meaning that the load was applied until the pile reached a certain deflection at the ground surface. The loading time for each increment was approximately ten minutes and then held for the remainder of the time period. After the maximum deflection was attained, the load was held for approximately ten minutes and then released slowly. The one-half inch and one inch test loading regimens are shown in Figures 3 and 4, respectively.

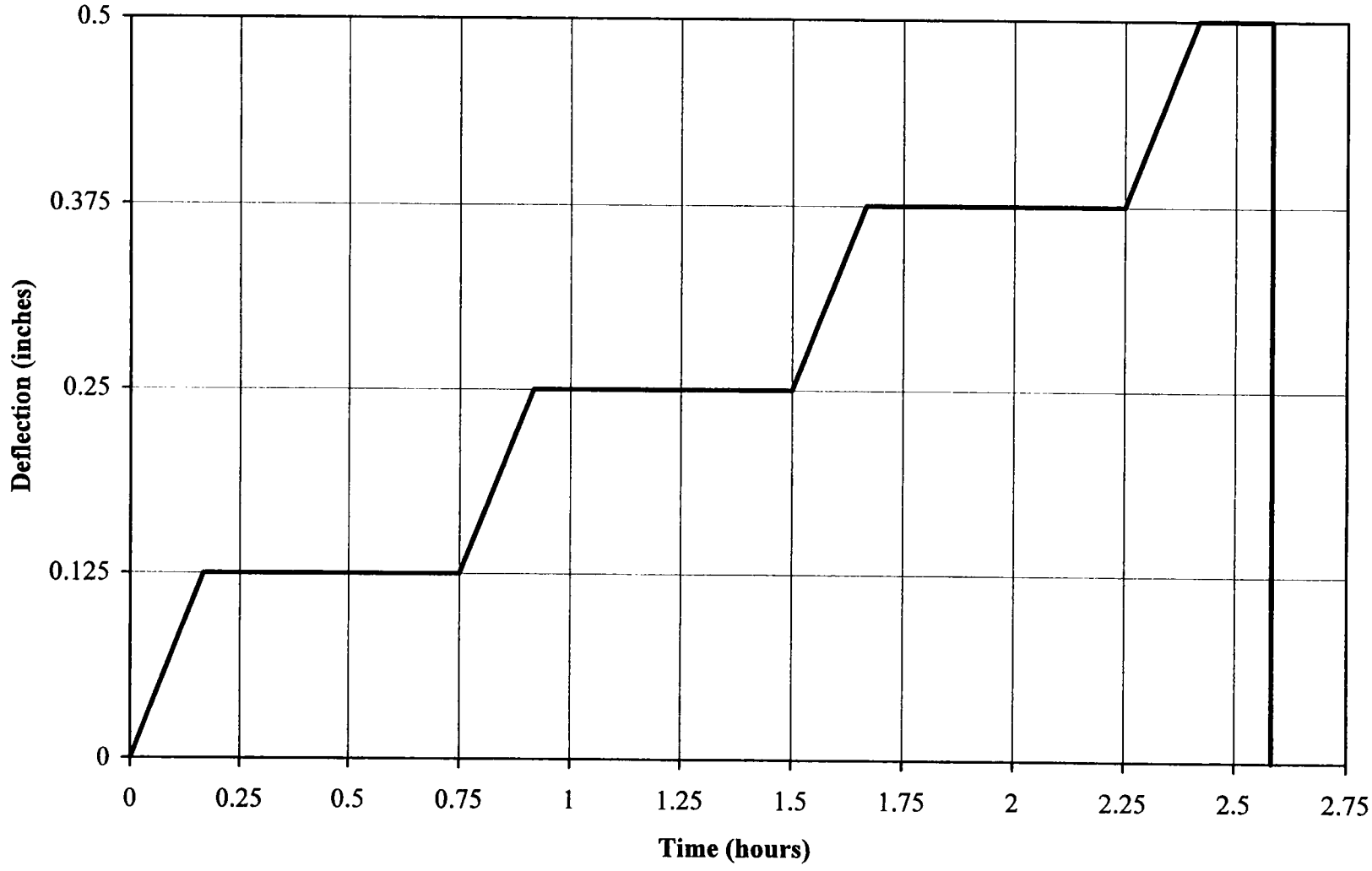


Figure 3. Loading Regimen for One-Half Inch Test.

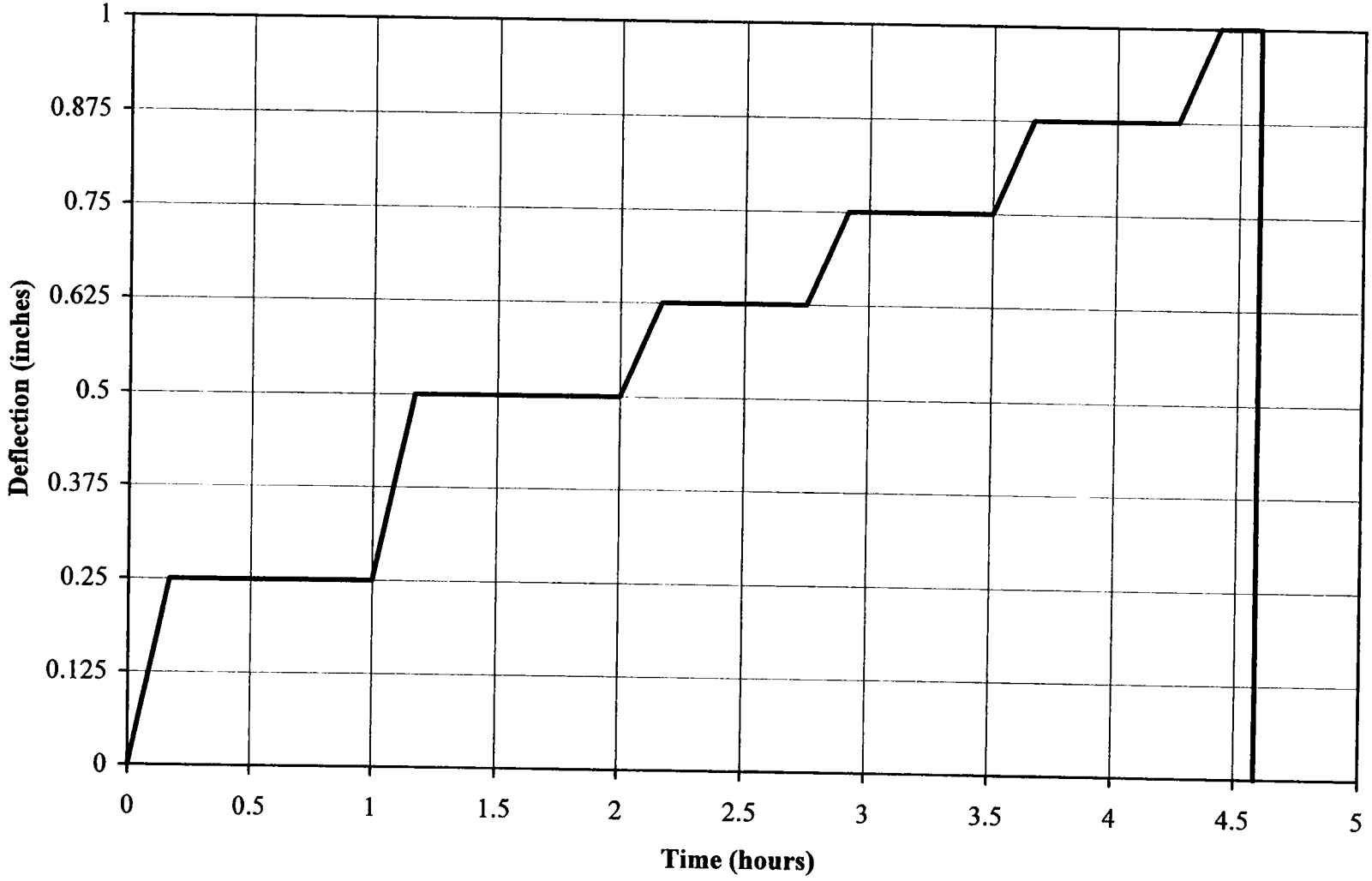


Figure 4. Loading Regimen for One Inch Test.

CHAPTER 4

DATA ACQUISITION

4.1 INTRODUCTION

As discussed in the previous chapter, there were a great number of gages, sensors, and instruments that required monitoring during a test. The data that were collected from the instrumentation were accumulated by a data acquisition system. The system used in this research was the Optim MEGADAC 3000 Series. This data were then sorted and reduced in order to determine the behavior of the pile.

4.2 DATA ACQUISITION SYSTEM

The MEGADAC system combined with the Test Control Software (TCS) allowed the data to be acquired from the instrumentation. This system had a capacity of 72 channels, which limits the amount of gages, sensors, and instruments that can be monitored during a test. To connect the instrumentation to the MEGADAC system, two types of analog input modules were used. Each analog input module had a capacity of eight channels, one of which required the use of a signal conditioning module. The system had a sampling rate of ½ Hz. for each field test; i.e. the data from the instruments was recorded every two seconds. This system was discussed in detail in an earlier thesis by Arico (10).

4.3 DATA INTERPRETATION

In order to study the behavior of piles supporting integral abutments, the data must be translated into a usable form. The data from the strain gages had to be converted from strains to bending moments. Data obtained from the pressure sensors, load cells, and LVDTs was measured in units that did not have to be converted. The pressure sensor data were questionable, and were not used in this thesis. The most valuable data were acquired from the strain gages, load cells, and LVDTs.

The majority of the data that were collected was from the strain gages. In order to account for any torsion or weak axis bending, the four gages at each station were averaged. The following relationships were then applied:

$$\sigma = \frac{My}{I} \tag{4.1}$$

and

$$\sigma = E\varepsilon \tag{4.2}$$

where σ = stress (ksi)

M = bending moment due to the lateral load (kip-in)

y = distance from the centroid of the pile to the strain gage (in)

I = moment of inertia of the pile (in^4)

E = modulus of elasticity of the pile (ksi)

ε = measured strain (in/in)

Equations 4.1 and 4.2 were then combined to derive the following equation:

$$M = \frac{EI\varepsilon}{y} \tag{4.3}$$

where ε = average strain for the station,

M, y, E, and I were defined previously.

This equation was then used to calculate the bending moments from the measured strains for each station. The moment of inertia of the H-pile was modified to account for the protective sheet metal that was added to the pile. A value of 223 in^4 was used on the first pile, while the value for the other three piles was 226 in^4. The modulus of elasticity was also modified since the weldable strain gages consistently read lower than the calculated strains in laboratory tests. By increasing the modulus from a value of 29,000 ksi to a value of 33,000 ksi, the low readings from the weldable strain gages were corrected. Figure 5 shows an example of converting the measured strains to the bending moment for a station.

The procedure described previously assumes that all four strain gages at a station were functioning properly. Since that was not always the case, an alternate method for determining a particular strain value was developed for the situation that one of the four strain gages was not functioning properly. This method uses the following relationships:

$$\varepsilon_A = + \varepsilon_{axial} + \varepsilon_{xx} - \varepsilon_{yy} \tag{4.4}$$

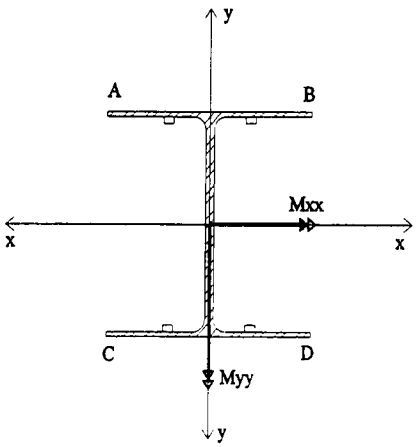
$$\varepsilon_B = + \varepsilon_{axial} + \varepsilon_{xx} + \varepsilon_{yy} \tag{4.5}$$

$$\varepsilon_C = + \varepsilon_{axial} - \varepsilon_{xx} - \varepsilon_{yy} \tag{4.6}$$

$$\varepsilon_D = + \varepsilon_{axial} - \varepsilon_{xx} + \varepsilon_{yy} \tag{4.7}$$

Time (sec)	Avg. Pile Defl. (in)	Measured Strains				Avg. Strain (Microstrain)	Moment (kip-ft)	
		A1	B1	C1	D1			
200	0.127	92.711	100.978	-65.547	-66.728	-81.491	-11.281	
202	0.127	93.892	99.797	-63.185	-63.185	-80.015	-11.077	
204	0.127	93.892	100.387	-65.547	-65.547	-81.343	-11.260	**Avg. Moment = -11.145 kip-ft**
206	0.127	92.711	99.797	-63.185	-63.185	-79.719	-11.036	
208	0.127	92.120	100.387	-65.547	-65.547	-80.900	-11.199	
210	0.127	92.120	99.797	-63.185	-63.185	-79.572	-11.015	
3840	0.323	274.589	294.076	-181.288	-183.059	-233.253	-32.289	
3842	0.323	275.180	293.485	-182.469	-184.831	-233.991	-32.392	
3844	0.323	274.589	294.666	-181.878	-183.650	-233.696	-32.351	**Avg. Moment = -32.313 kip-ft**
3846	0.323	274.589	293.485	-182.469	-183.059	-233.401	-32.310	
3848	0.323	274.589	294.666	-181.288	-183.650	-233.548	-32.330	
3850	0.323	274.589	292.895	-180.697	-182.469	-232.663	-32.208	
6330	0.429	367.890	389.149	-238.568	-235.615	-307.806	-42.610	
6332	0.429	367.300	386.787	-237.977	-234.434	-306.625	-42.446	
6334	0.429	367.890	389.149	-238.568	-236.206	-307.953	-42.630	**Avg. Moment = -42.545 kip-ft**
6336	0.429	366.709	386.787	-237.977	-235.025	-306.625	-42.446	
6338	0.429	367.890	389.149	-239.158	-236.206	-308.101	-42.651	
6340	0.429	367.300	386.787	-237.977	-235.615	-306.920	-42.487	
7360	0.508	422.217	441.114	-271.636	-266.912	-350.470	-48.516	
7362	0.508	421.627	441.114	-271.636	-266.912	-350.322	-48.496	
7364	0.508	421.627	442.295	-272.227	-266.912	-350.765	-48.557	**Avg. Moment = -48.492 kip-ft**
7366	0.508	421.627	439.933	-271.046	-266.322	-349.732	-48.414	
7368	0.508	421.627	442.295	-272.227	-266.912	-350.765	-48.557	
7370	0.508	421.036	440.523	-271.046	-266.322	-349.732	-48.414	

Figure 5. The Conversion of Mesured Strains to Bending Moments for Station One of a One-Half Inch Test.



and

$$\varepsilon_A + \varepsilon_D = \varepsilon_B + \varepsilon_C \qquad (4.8)$$

where $\varepsilon_{A, B, C, D}$ = measured strain of the flange

ε_{axial} = axial strain due to axial load

ε_{xx} = strain due to bending about the x axis

ε_{yy} = strain due to bending about the y axis

Using equation 4.8 the strain on any flange can be estimated if the strain was known on the other three flanges. It was determined that if more that one gage at a particular station was not functioning, the data for that station would be omitted from the analysis.

The bending moments at each station were calculated for each increment of loading during each test. The bending moments were then plotted versus depth. The analysis of the pile is based on these moment versus depth curves.

CHAPTER 5

ANALYSIS

5.1 INTRODUCTION

In this research, the moment versus depth curves are calculated from the measured strains. These curves are then fitted with an equation that was used to predict the behavior of the pile. By using elastic theory, the shear and pressure diagrams along with the deflected shape were determined from the moment equation. A check of static equilibrium was used to verify the results.

5.2 DATA

Once the moment diagrams were plotted, an equation was developed for each diagram. It was determined that a sixth order polynomial best fit the measured moment diagrams. Other multiple order polynomials and a combination of multiple order polynomial equations were also tried, but the sixth order polynomial equation produced a consistently reliable analysis. Figure 6 shows a moment versus depth plot with its best fit sixth order polynomial equation. The tests for both directions of the first pile and the tests for one direction of the second pile were considered for the purpose of this thesis. A total of sixteen tests were analyzed; nine one-half inch tests and seven one inch tests. These tests are described in Table 2. All the analyzed results are presented in Appendices B through Q.

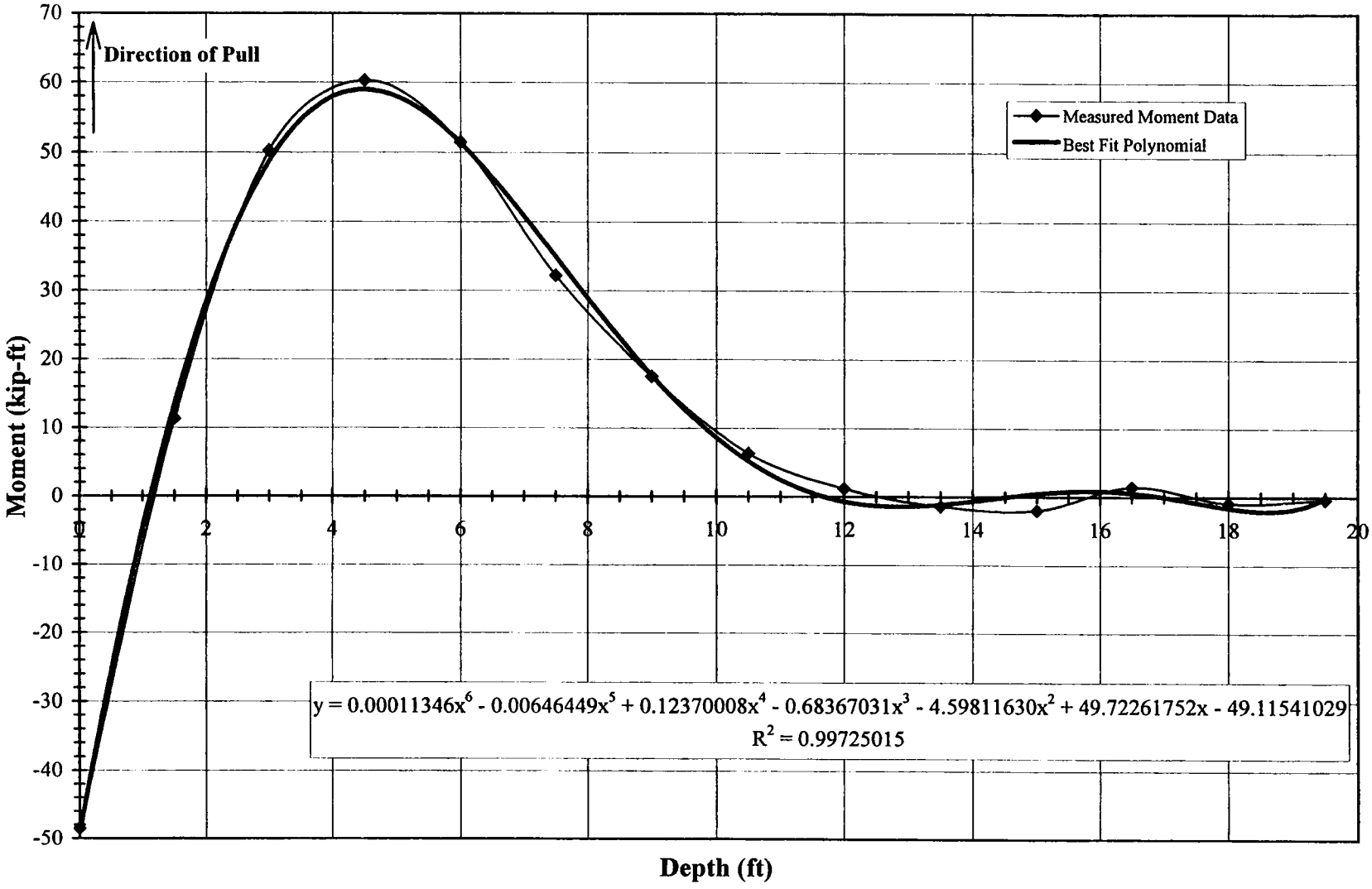


Figure 6. Moment Versus Depth Diagram With Sixth Order Equation.

Table 2. Selected Field Test Results.

Test Date	Test Description	Pile Deflection at Ground Surface
July 17, 1997	Pile 1, Third 1/2" Test Pulling East	0.508 inches
July 24, 1997	Pile 1, First 1" Test Pulling East	0.541 inches
July 24, 1997	Pile 1, First 1" Test Pulling East	1.016 inches
July 30, 1997	Pile 1, Third 1" Test Pulling East	0.564 inches
July 30, 1997	Pile 1, Third 1" Test Pulling East	1.021 inches
August 15, 1997	Pile 1, First 1/2" Test Pulling West	0.510 inches
August 22, 1997	Pile 1, Third 1/2" Test Pulling West	0.514 inches
August 25, 1997	Pile 1, First 1" Test Pulling West	0.513 inches
August 25, 1997	Pile 1, First 1" Test Pulling West	1.034 inches
August 26, 1997	Pile 1, Second 1" Test Pulling West	0.989 inches
September 5, 1997	Pile 1, Third 1" Test Pulling West	1.021 inches
January 30, 1998	Pile 2, Second 1/2" Test Pulling East	0.499 inches
February 13, 1998	Pile 2, First 1" Test Pulling East	0.512 inches
February 13, 1998	Pile 2, First 1" Test Pulling East	1.002 inches
February 24, 1998	Pile 2, Third 1" Test Pulling East	0.506 inches
February 24, 1998	Pile 2, Third 1" Test Pulling East	1.008 inches

5.3 ELASTIC THEORY

The behavior of piles supporting integral abutments was determined by using elastic theory. Based on this theory the shear, pressure, rotation, and deflected shape were determined from the moment equation. The shear equation was determined by differentiating the moment equation once as illustrated in the following mathematical relationship:

$$V = \frac{dM}{dx} \tag{5.1}$$

where dM/dx = first derivative of the moment with respect to x

V = shear (kips)

The pressure equation was determined by differentiating the shear equation once as follows:

$$P = \frac{dV}{dx} \tag{5.2}$$

where dV/dx = first derivative of the shear with respect to x

P = lateral pressure (kips/ft)

In order to determine the rotation and deflected shape equations, the moment equation was integrated. The rotation equation was determined by integrating the moment equation as follows:

$$\Theta = \int \frac{M}{EI}\, dx \tag{5.3}$$

where Θ = rotation (radians)

$\int M\ dx$ = integral of the moment

E = modulus of elasticity (ksi)

I = moment of inertia of the cross section (inches^4)

This integration produces a constant of integration. The equation for the deflected shape was then determined by integrating the rotation equation as follows:

$$\Delta = \int \Theta\ dx \tag{5.4}$$

where Δ = deflection (inches)

$\int \Theta\ dx$ = integral of the rotation

A second constant of integration resulted from this integration. These constants are discussed later in this chapter. An example of the equations produced from these relationships are shown in Figure 7.

5.4 SHEAR AND PRESSURE CURVES

From the relationships described in the previous section, the shear and pressure diagrams are easily produced. The fifth order shear equation was evaluated at each station and then plotted versus depth. The fifth order equation produces unreliable results at depths greater than eighteen feet; therefore, the shear at 19.5 feet was set at zero, since the strain readings were approximately zero at these depths. A shear diagram is shown in Figure 8.

The pressure diagram was produced from the resulting fourth order equation. The pressure equation was evaluated at each station and then plotted. Two areas were of interest on the pressure diagram. The first area was at the ground surface. In some of the tests, the equation produced a negative pressure at the ground surface. Since this negative pressure was not physically possible, a correction was made by determining the first zero root of the pressure equation and assuming the pressure was equal to zero until reaching the depth of the root. This correction was justified by assuming there was no soil confinement at the ground surface and that the soil was deformed by previous tests. The other area of interest that was examined was at the deeper stations. The pressure equation approached infinity at depths greater than sixteen feet and had to be adjusted. To correct for this inaccuracy, the last zero root of the pressure equation was determined and the

Deflection Equation:

From 0' to 19.5'

$y = 4.758E\text{-}10\ x^8 + -3.614E\text{-}08\ x^7 + 9.682E\text{-}07\ x^6 + -8.027E\text{-}06\ x^5$

$+ -8.998E\text{-}05\ x^4 + 0.001945932\ x^3 + -0.005766506\ x^2 + (C_1)\ x$

$+ 0.508\ (C_2)$

Rotation Equation:

From 0' to 19.5'

$y = 3.172E\text{-}10\ x^7 + -2.108E\text{-}08\ x^6 + 4.841E\text{-}07\ x^5 + -3.344E\text{-}06\ x^4$

$+ -2.999E\text{-}05\ x^3 + 0.000486483\ x^2 + -0.000961084\ x + (C_1)$

Moment Equation:

From 0' to 19.5'

$\mathbf{y = 0.00011346\ x^6 + -0.00646449\ x^5 + 0.12370008\ x^4 + -0.68367031\ x^3}$

$\mathbf{+ -4.59811630\ x^2 + 49.72261752\ x + -49.11541029}$

Shear Equation:

From 0' to 19.5'

$y = 0.00068076\ x^5 + -0.03232245\ x^4 + 0.49480032\ x^3 + -2.05101093\ x^2$

$+ -9.19623260\ x + 49.72261752$

Pressure Equation:

From 0' to 19.5'

$y = 0.0034038\ x^4 + -0.1292898\ x^3 + 1.48440096\ x^2 + -4.10202186\ x$

$+ -9.19623260$

Figure 7. Equations for the Deflected Shape, Shear, and Pressure Diagrams.

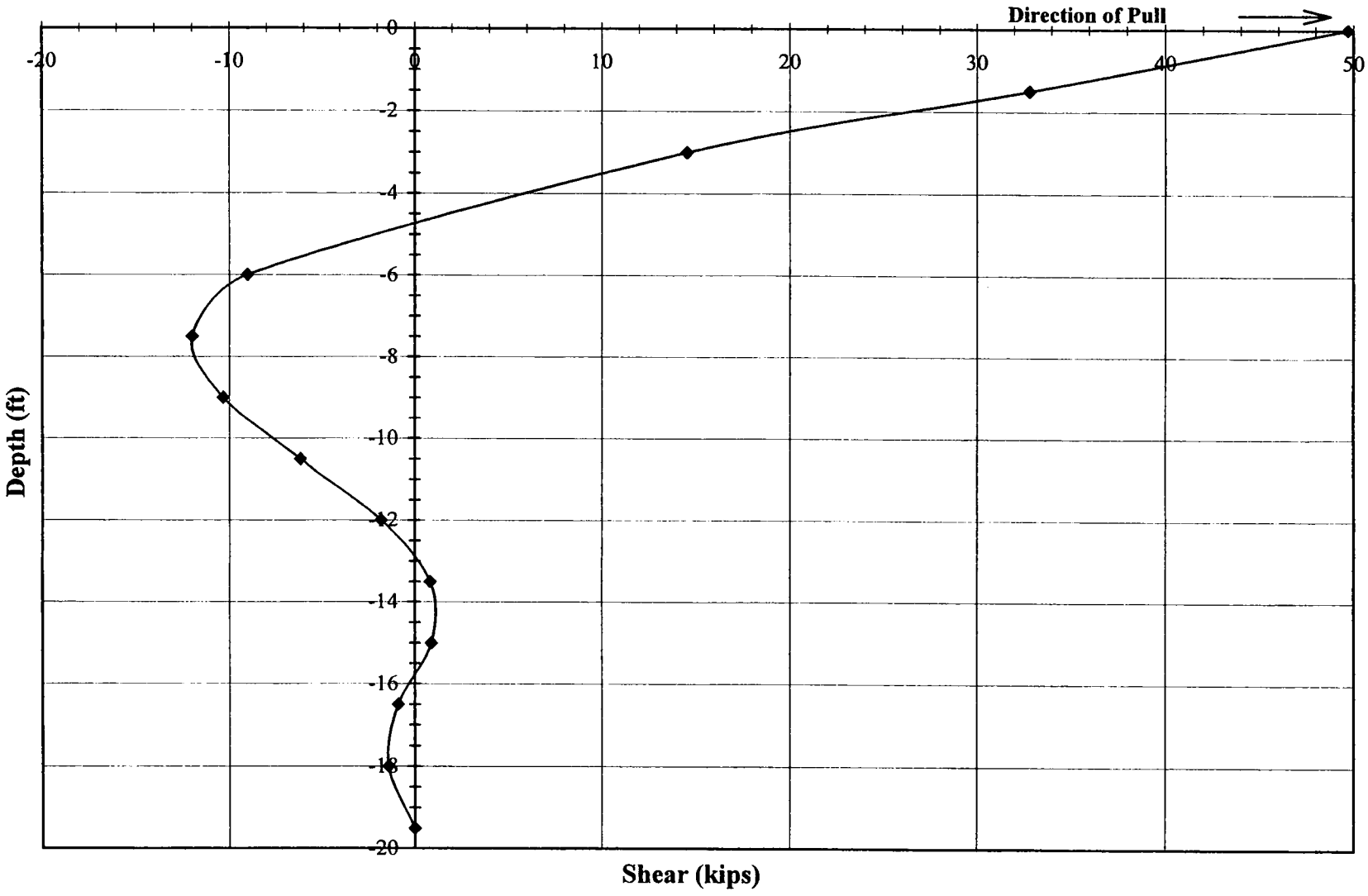


Figure 8. Shear Versus Depth Diagram.

pressures of the stations below that root were assumed to equal zero. The justification for this correction was that the strain readings were approximately zero at these depths. A pressure diagram is shown in Figure 9.

5.5 STATIC EQUILIBRIUM

In order to verify the analyzed results, several checks were made using static equilibrium. The first check was made by calculating the applied moment and comparing it to the measured moment at the ground surface. The applied moment was calculated by summing the moments about the center of the pile at the ground surface. This calculation is shown in the following equation:

$$M_{GRND} = R(x) - P(y) - W(e) \qquad (5.5)$$

where M_{GRND} = applied moment at the ground (kip-feet)

R = measured load at the reaction beam (kips)

x = distance from centerline of pile to centerline of reaction (feet)

P = measured load at the load frame (kips)

y = distance from centerline of load to ground surface (feet)

W = weight of the abutment and ballast (65 kips)

e = deflection of the abutment (Δ_{abut}) minus the deflection of the pile (Δ_{pile}) (feet)

The comparisons of the moments are shown in Table 3. The next check was made by comparing the measured load at the load frame to the calculated shear at the ground

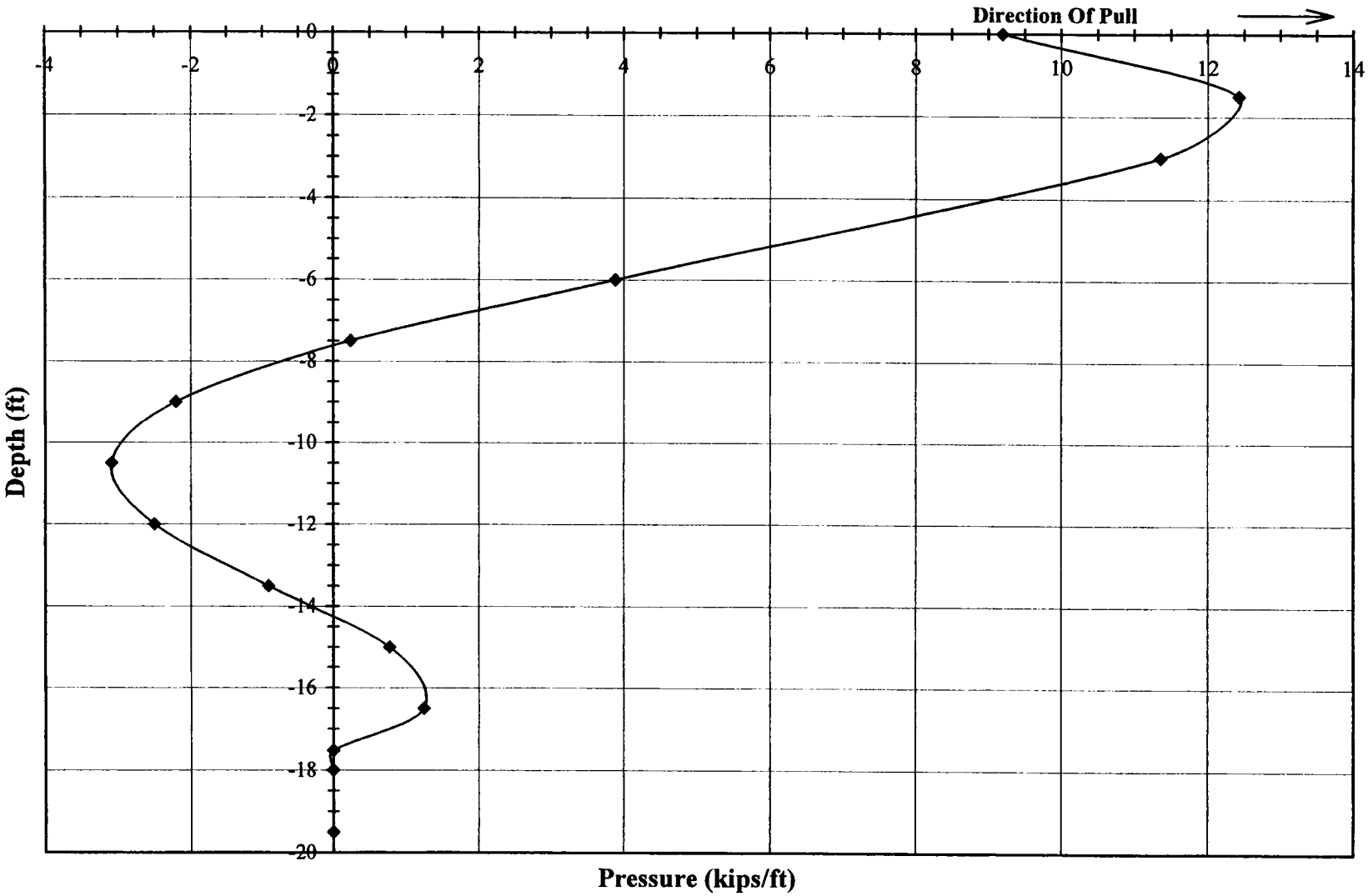


Figure 9. Lateral Pressure Versus Depth Diagram.

Table 3. Comparison of Calculated and Measured Moments.

Test Date	Test Description	Pile Deflection at Ground Surface	Calculated Moment (kip-ft)	Measured Moment (kip-ft)	$\frac{M_{calc}}{M_{meas}}$
July 17, 1997	Pile 1, Third 1/2" Test Pulling East	0.508 inches	50.837	48.492	1.048
July 24, 1997	Pile 1, First 1" Test Pulling East	0.541 inches	39.600	41.479	0.955
July 24, 1997	Pile 1, First 1" Test Pulling East	1.016 inches	51.885	60.379	0.859
July 30, 1997	Pile 1, Third 1" Test Pulling East	0.564 inches	34.865	38.124	0.915
July 30, 1997	Pile 1, Third 1" Test Pulling East	1.021 inches	56.092	59.102	0.949
August 15, 1997	Pile 1, First 1/2" Test Pulling West	0.510 inches	29.904	32.354	0.924
August 22, 1997	Pile 1, Third 1/2" Test Pulling West	0.514 inches	31.164	32.180	0.968
August 25, 1997	Pile 1, First 1" Test Pulling West	0.513 inches	29.915	31.605	0.947
August 25, 1997	Pile 1, First 1" Test Pulling West	1.034 inches	46.247	49.960	0.926
August 26, 1997	Pile 1, Second 1" Test Pulling West	0.989 inches	48.129	50.655	0.950
September 5, 1997	Pile 1, Third 1" Test Pulling West	1.021 inches	46.763	53.958	0.867
January 30, 1998	Pile 2, Second 1/2" Test Pulling East	0.499 inches	52.226	52.966	0.986
February 13, 1998	Pile 2, First 1" Test Pulling East	0.512 inches	50.154	56.300	0.891
February 13, 1998	Pile 2, First 1" Test Pulling East	1.002 inches	67.860	76.601	0.886
February 24, 1998	Pile 2, Third 1" Test Pulling East	0.506 inches	36.146	36.093	1.001
February 24, 1998	Pile 2, Third 1" Test Pulling East	1.008 inches	72.803	69.411	1.049

surface. These values are compared in Table 4. The last check was performed by integrating the fourth order pressure equation to convert the nonuniform pressure distribution into total loads. These loads were then totaled and compared to the applied load at the load frame. These comparisons are shown in Table 5.

5.6 DEFLECTED SHAPE

The deflected shape and rotation can be derived by integrating the moment equation. These relationships were shown earlier in this chapter. The constants of integration that are produced have to be determined in order to plot the deflected shape. These constants were calculated using the known boundary conditions at the ground surface.

The first constant of integration was the rotation of the pile at the ground surface. An inclinometer was used to calculate the rotation. As the pile rotated, an LVDT measured the vertical displacement of the inclinometer. The rotation was determined by taking the inverse tangent of the vertical displacement divided by the distance from the centerline of the pile to the LVDT. This procedure was not implemented until late in the testing of the second pile, and the data are considered to be questionable until more data is analyzed.

The second constant of integration was the deflection of the pile at the ground surface. This constant was known since there was a LVDT measuring the pile deflection at the ground surface. In order to determine the rotation of the pile at the ground surface, another boundary condition was chosen. The displacement of the pile was assumed to be

Table 4. Comparison of Calculated and Measured Shears.

Test Date	Test Description	Pile Deflection at Ground Surface	Calculated Shear (kips)	Measured Shear (kips)	$\frac{V_{calc}}{V_{meas}}$
July 17, 1997	Pile 1, Third 1/2" Test Pulling East	0.508 inches	49.723	44.22	1.124
July 24, 1997	Pile 1, First 1" Test Pulling East	0.541 inches	34.606	38.16	0.907
July 24, 1997	Pile 1, First 1" Test Pulling East	1.016 inches	67.533	65.20	1.036
July 30, 1997	Pile 1, Third 1" Test Pulling East	0.564 inches	16.498	24.02	0.687
July 30, 1997	Pile 1, Third 1" Test Pulling East	1.021 inches	61.860	60.54	1.022
August 15, 1997	Pile 1, First 1/2" Test Pulling West	0.510 inches	38.232	37.97	1.007
August 22, 1997	Pile 1, Third 1/2" Test Pulling West	0.514 inches	44.535	45.26	0.984
August 25, 1997	Pile 1, First 1" Test Pulling West	0.513 inches	48.314	45.56	1.060
August 25, 1997	Pile 1, First 1" Test Pulling West	1.034 inches	69.501	71.92	0.966
August 26, 1997	Pile 1, Second 1" Test Pulling West	0.989 inches	63.995	66.04	0.969
September 5, 1997	Pile 1, Third 1" Test Pulling West	1.021 inches	61.537	64.12	0.960
January 30, 1998	Pile 2, Second 1/2" Test Pulling East	0.499 inches	55.865	44.87	1.245
February 13, 1998	Pile 2, First 1" Test Pulling East	0.512 inches	57.475	44.40	1.294
February 13, 1998	Pile 2, First 1" Test Pulling East	1.002 inches	81.134	63.88	1.270
February 24, 1998	Pile 2, Third 1" Test Pulling East	0.506 inches	10.575	18.43	0.574
February 24, 1998	Pile 2, Third 1" Test Pulling East	1.008 inches	73.633	55.44	1.328

Table 5. Comparison of Calculated and Applied Loads.

Test Date	Test Description	Pile Deflection at Ground Surface	Calculated Load (kips)	Applied Load (kips)	$\frac{P_{calc}}{P_{applied}}$
July 17, 1997	Pile 1, Third 1/2" Test Pulling East	0.508 inches	51.392	44.22	1.162
July 24, 1997	Pile 1, First 1" Test Pulling East	0.541 inches	37.108	38.16	0.972
July 24, 1997	Pile 1, First 1" Test Pulling East	1.016 inches	68.645	65.20	1.053
July 30, 1997	Pile 1, Third 1" Test Pulling East	0.564 inches	27.533	24.02	1.146
July 30, 1997	Pile 1, Third 1" Test Pulling East	1.021 inches	63.376	60.54	1.047
August 15, 1997	Pile 1, First 1/2" Test Pulling West	0.510 inches	39.138	37.97	1.031
August 22, 1997	Pile 1, Third 1/2" Test Pulling West	0.514 inches	46.280	45.26	1.023
August 25, 1997	Pile 1, First 1" Test Pulling West	0.513 inches	48.589	45.56	1.066
August 25, 1997	Pile 1, First 1" Test Pulling West	1.034 inches	70.111	71.92	0.975
August 26, 1997	Pile 1, Second 1" Test Pulling West	0.989 inches	65.415	66.04	0.991
September 5, 1997	Pile 1, Third 1" Test Pulling West	1.021 inches	62.222	64.12	0.970
January 30, 1998	Pile 2, Second 1/2" Test Pulling East	0.499 inches	57.645	44.87	1.285
February 13, 1998	Pile 2, First 1" Test Pulling East	0.512 inches	58.277	44.40	1.313
February 13, 1998	Pile 2, First 1" Test Pulling East	1.002 inches	81.189	63.88	1.271
February 24, 1998	Pile 2, Third 1" Test Pulling East	0.506 inches	23.853	18.43	1.294
February 24, 1998	Pile 2, Third 1" Test Pulling East	1.008 inches	75.067	55.44	1.354

zero at the depth where the lateral pressure first equals zero. Using this boundary condition allowed the calculation of the rotation at the ground surface. Once the constants of integration were determined the deflected shape was plotted. Figure 10 shows a plotted deflected shape.

5.7 OTHER ANALYSIS

The load versus pile deflection and the load versus reaction were plotted for each direction. These plots indicate there was a reduction in stiffness as the testing progressed. The load versus reaction plots indicate there was a linear relationship between the load and reaction. The moment diagrams were also compared for each direction. These plots indicate that the moments vary from test to test and that the pile-soil interaction typically loses stiffness with repeated cycles. All of these figures are presented in Appendix A.

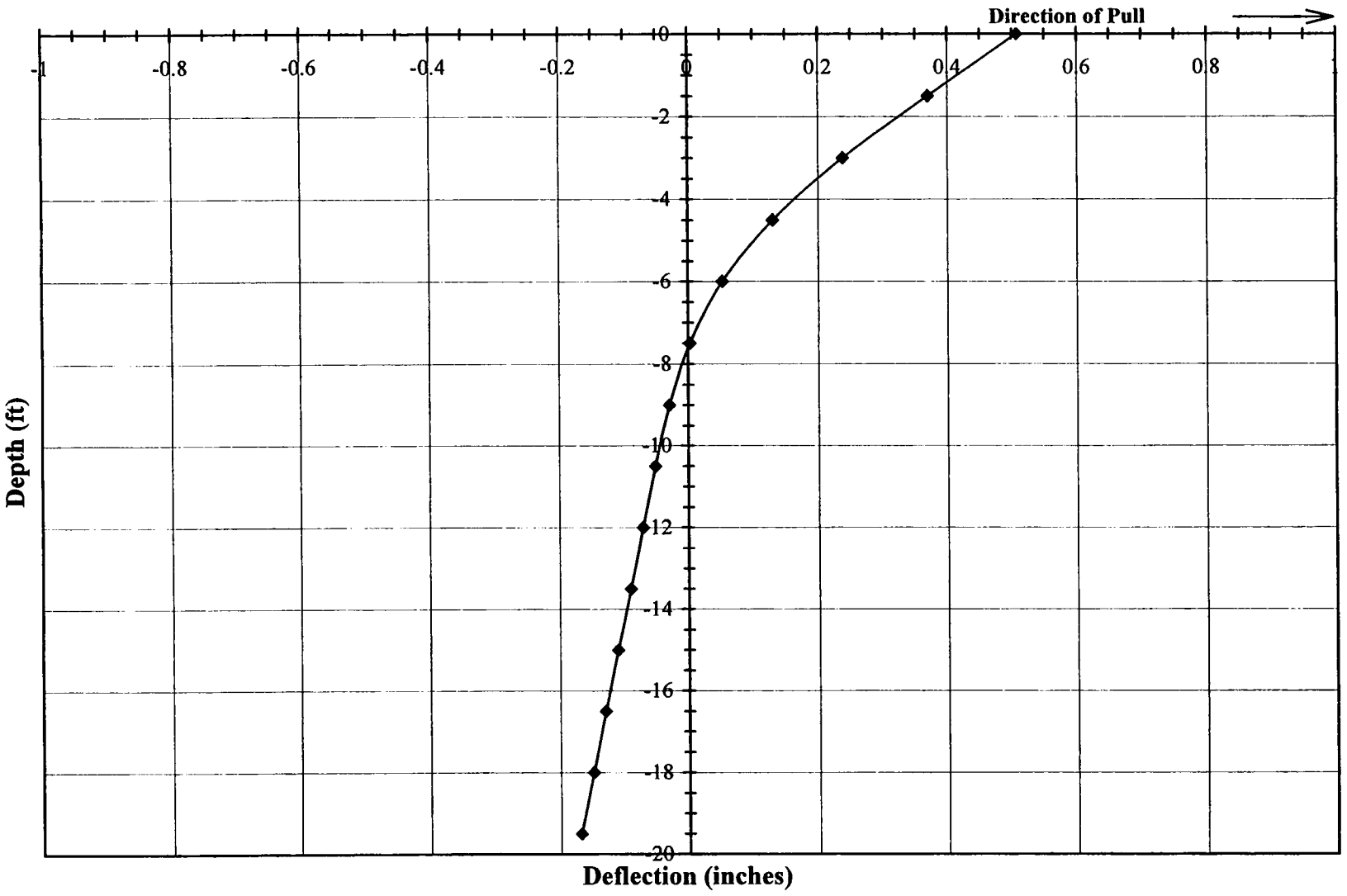


Figure 10. Pile Deflection Versus Depth Diagram (Deflected Shape).

CHAPTER 6

DISCUSSION OF RESULTS

The moment versus depth curves were fitted with an equation that was used to predict the behavior of the pile. This equation was evaluated by using elastic theory to produce shear, pressure and deflected shape diagrams. By using a simple check of static equilibrium, the results were verified. This analysis procedure produced reasonable results for the shear and pressure diagrams and erratic results for the lower portion of the deflected shape. It was of particular interest to the Tennessee Department of Transportation to know the depth where the pressure diagram first crosses zero. Based on the results in the Appendices, it was determined that the pressure diagram first crossed zero at an average depth of eight feet.

It was determined that additional instrumentation was needed to produce more accurate moment equations. As the tests proceeded, some of the strain gages were determined to be inadequate. If more than one strain gage per station was inadequate, the data for that station was omitted from the analysis. In some of the tests, as many as four stations were omitted. The number of valid data points was also limited on the second pile due to the added stations in the first three feet of the pile. The strains at the lower stations were not recorded since there was not enough channels in the data acquisition system to record every strain gage.

The deflected shape plots produced some anticipated and unanticipated results. These plots indicated that the pile had significant displacements at depths where the pile

displacement was expected to be zero. The upper portion of the deflected shape plots were reasonable for both shape and magnitude. There were a few factors that contributed to the unanticipated results of the lower portion of the plots. The first of these factors was that the rotation at the ground surface was not measured until late in the testing of the second pile. Since this was a boundary condition used in determining the deflection equation, an alternate boundary condition had to be used to determine the pile rotation at ground surface. The first point of zero lateral pressure was assumed to have zero deflection and was used as the new boundary condition. The other factor was the accuracy of the fitted moment equation as discussed previously.

Even with these inadequacies, the analysis still produced acceptable results. The measured moments at the ground surface were within 15 percent of the moments calculated by using static equilibrium. For the first pile, the calculated shear values were within 13 percent of the measured values with one exception and the calculated load values were within 16 percent of the measured values. The lack of valid data points for the second pile produced significant variation, 24 to 42 percent of the measured values, in the calculated values of the shear and load.

Further testing in the current research should result in a more accurate analysis of the test data. A more accurate analysis can be achieved by using more valid strain data and by further analyzing the inclinometer data to determine the pile rotation. These results help provide a better understanding of the behavior of piles supporting integral abutments.

CHAPTER 7

CONCLUSIONS

The procedure employed for the determination of lateral pressure versus depth of pile produced reasonable results in most cases analyzed. Uncertainty in strain gage readings at some stations due to an inoperative gage (of the four gages at a station) led to corresponding uncertainty in moments and lateral pressures. However, the overall results of the pressure versus depth curves appear to be reasonable. The first point of zero pressure consistently turned out to be approximately eight feet below the ground surface for the two piles tested in virgin soil.

The lateral displacement versus depth curve obtained from integration does not appear reasonable. As the pressure is the result of lateral displacement of the pile, one would expect that the pressure versus depth and lateral displacement versus depth curves would be at least roughly the same shape. While the pressure versus depth curves appear to be reasonable, the lateral displacement versus depth curve bears little resemblance to them and is not very believable. Perhaps the data from the inclinometer in future tests will lead to better results.

LIST OF REFERENCES

LIST OF REFERENCES

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APPENDICES

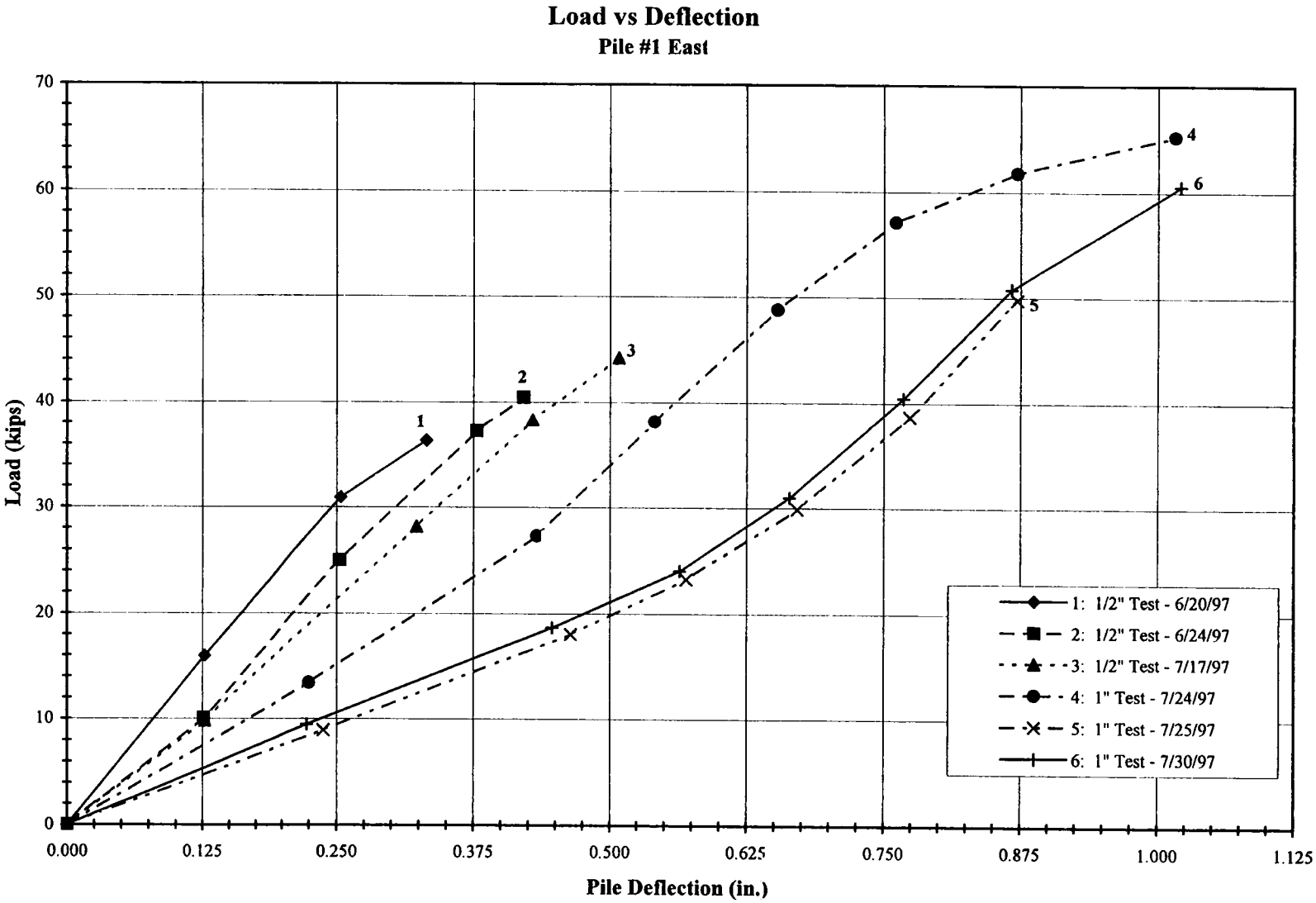
APPENDIX A

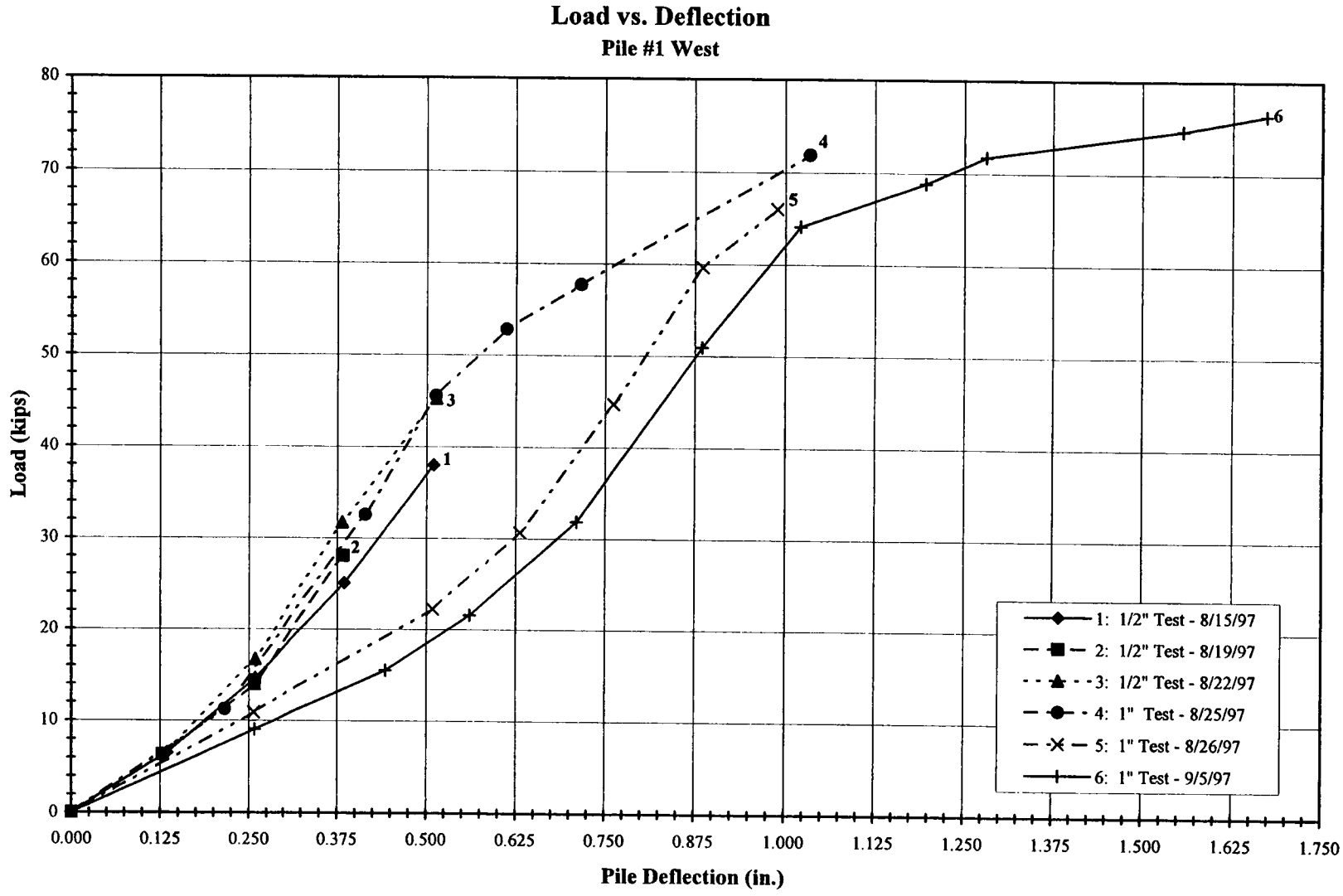
PLOTS OF

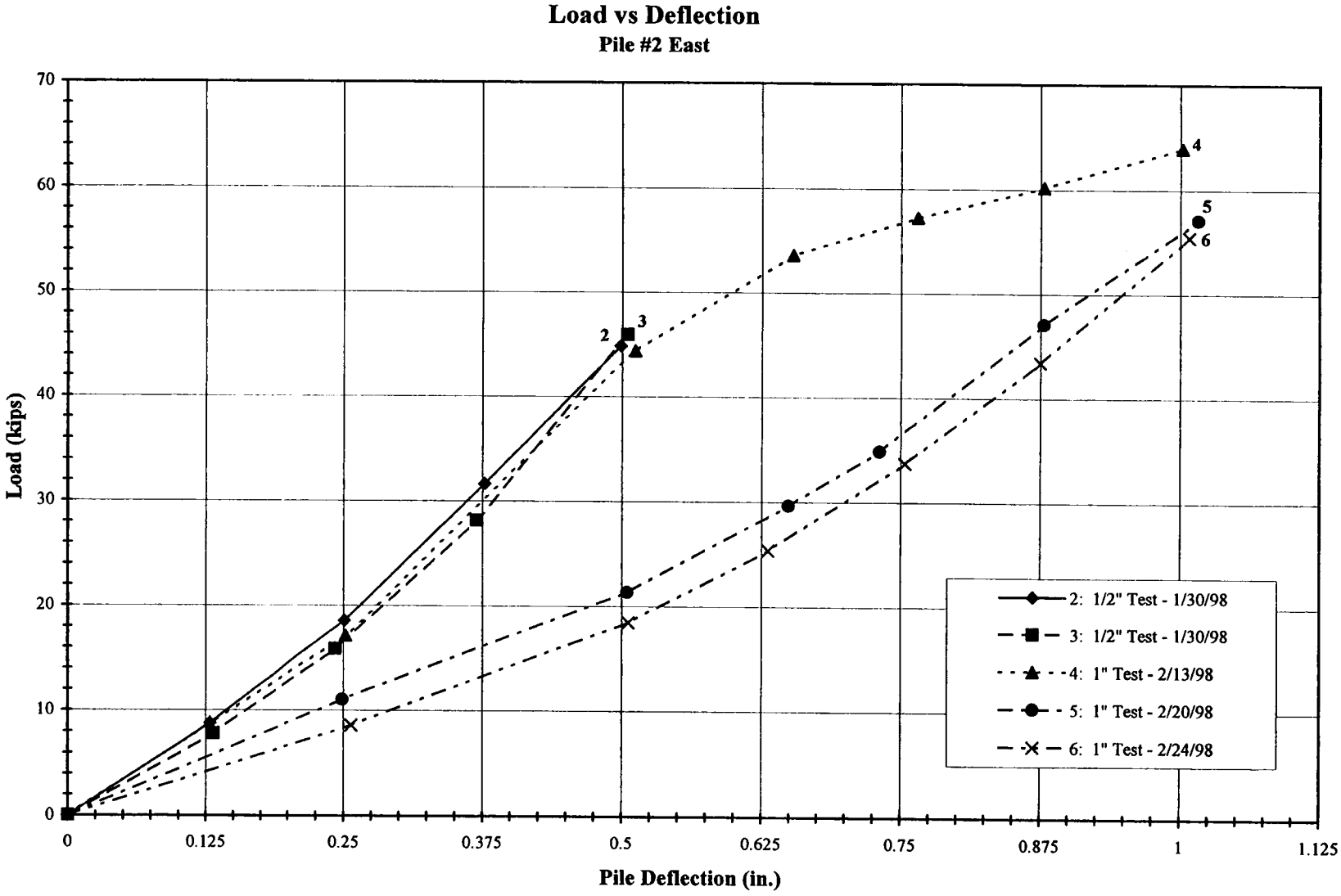
LOAD vs. DEFLECTION

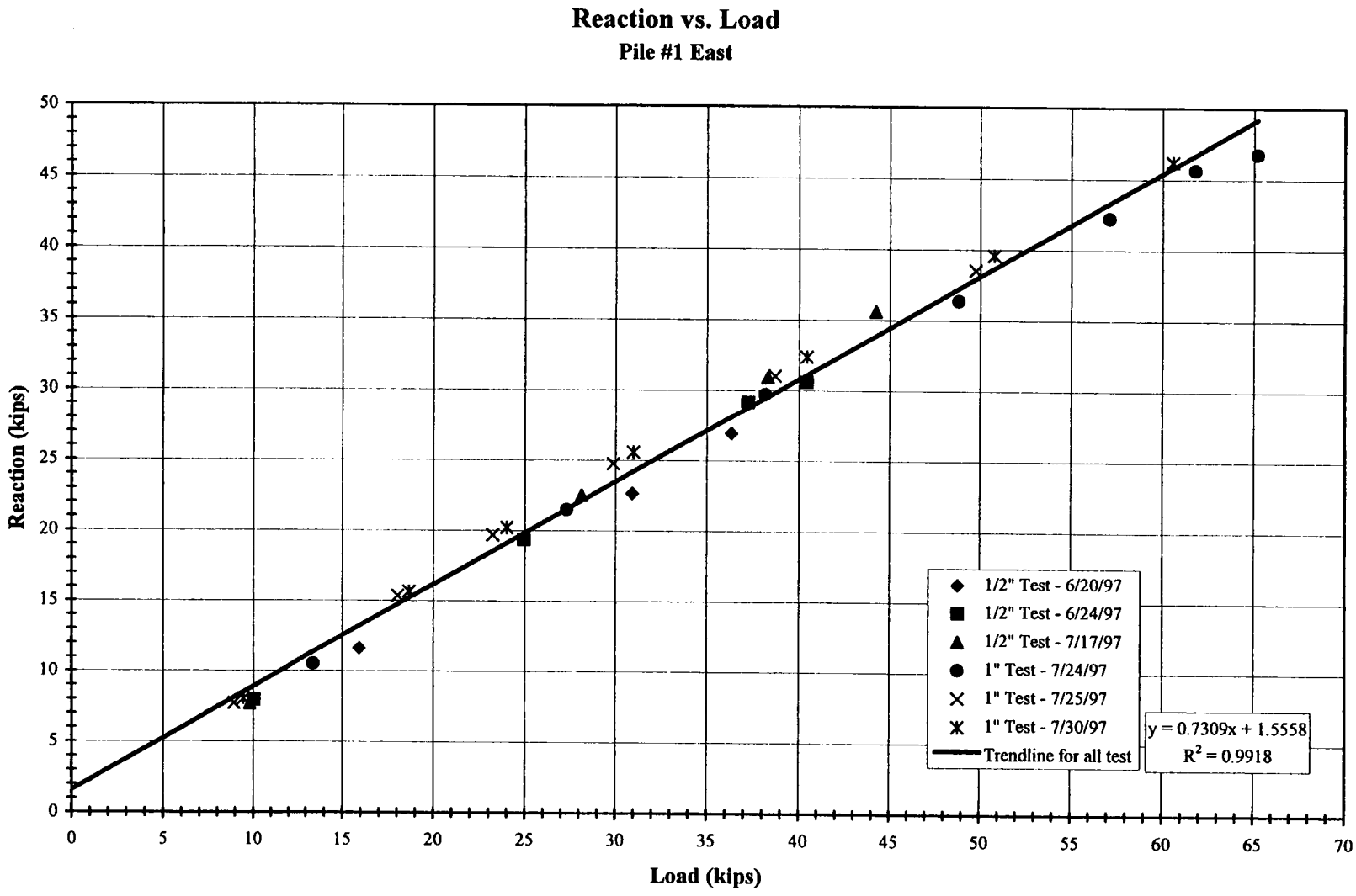
LOAD vs. REACTION

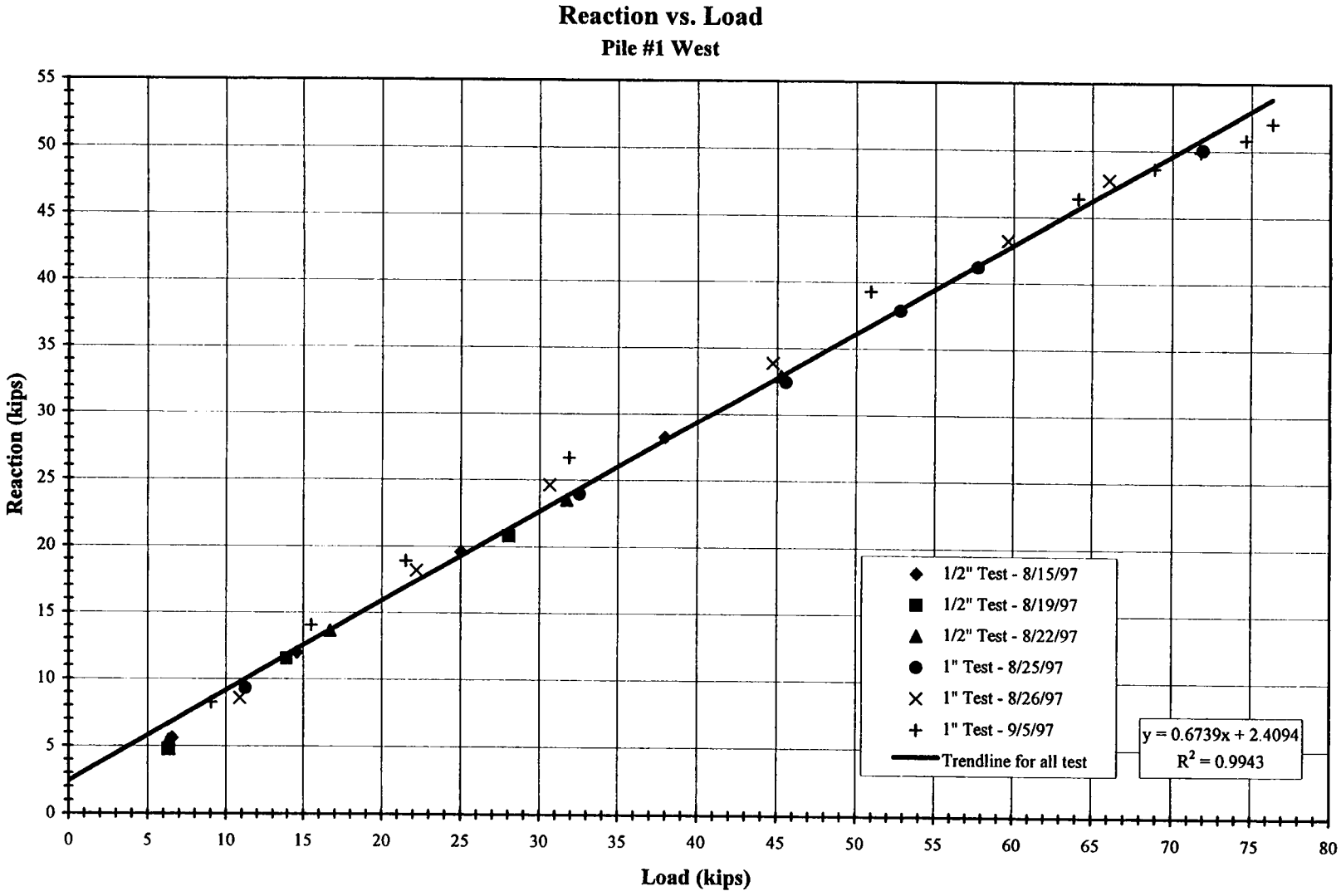
MOMENT vs. DEPTH COMPARISONS

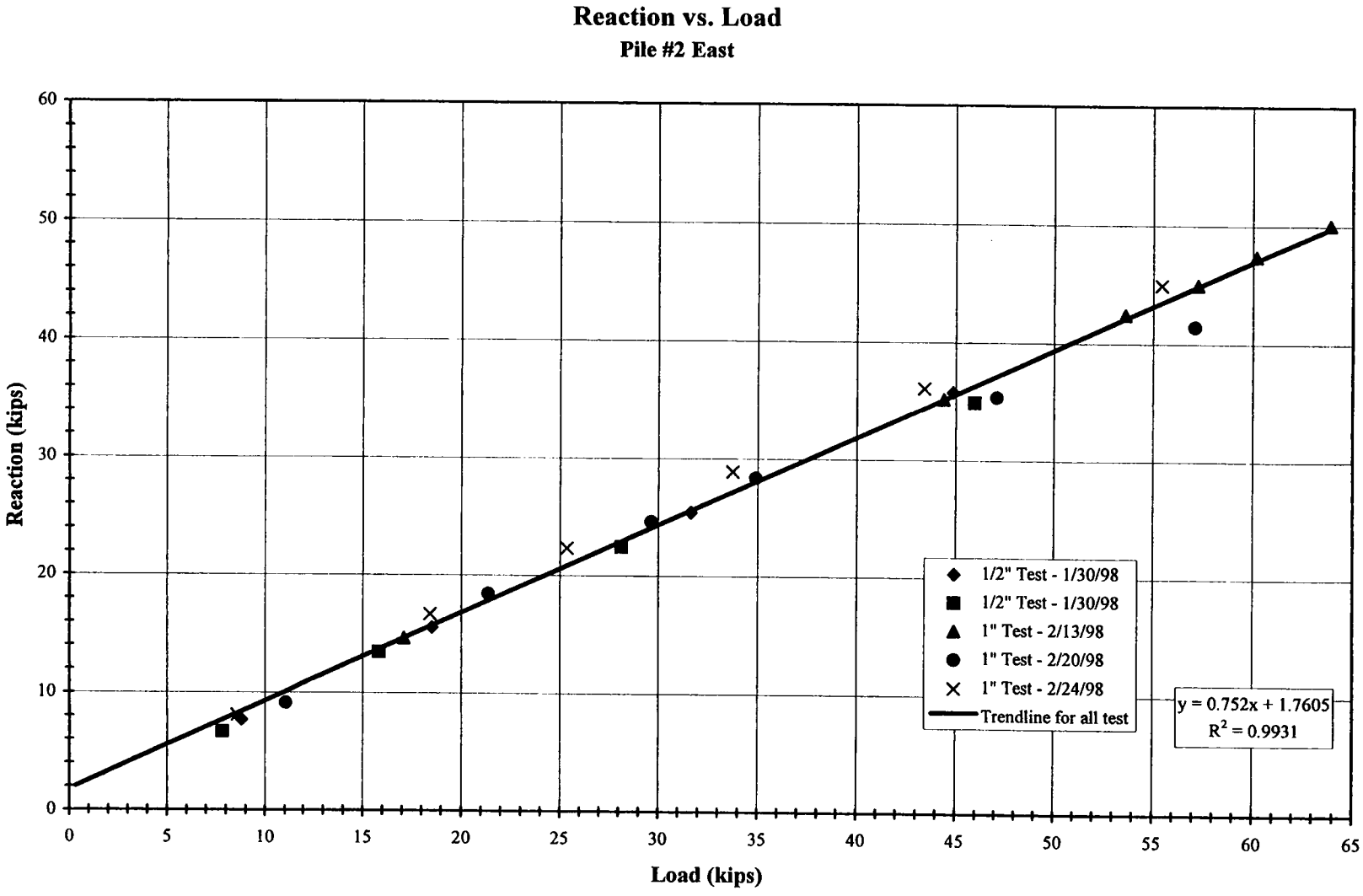

Load vs Deflection
Pile #1 East
Load (kips)
Pile Deflection (in.)
0
10
20
30
40
50
60
70
0.000
0.125
0.250
0.375
0.500
0.625
0.750
0.875
1.000
1.125
1
2
3
4
5
6
1: 1/2" Test - 6/20/97
2: 1/2" Test - 6/24/97
3: 1/2" Test - 7/17/97
4: 1" Test - 7/24/97
5: 1" Test - 7/25/97
6: 1" Test - 7/30/97


Load vs. Deflection
Pile #1 West
Load (kips)
Pile Deflection (in.)
0
10
20
30
40
50
60
70
80
0.000
0.125
0.250
0.375
0.500
0.625
0.750
0.875
1.000
1.125
1.250
1.375
1.500
1.625
1.750
1
2
3
4
5
6
1: 1/2" Test - 8/15/97
2: 1/2" Test - 8/19/97
3: 1/2" Test - 8/22/97
4: 1" Test - 8/25/97
5: 1" Test - 8/26/97
6: 1" Test - 9/5/97

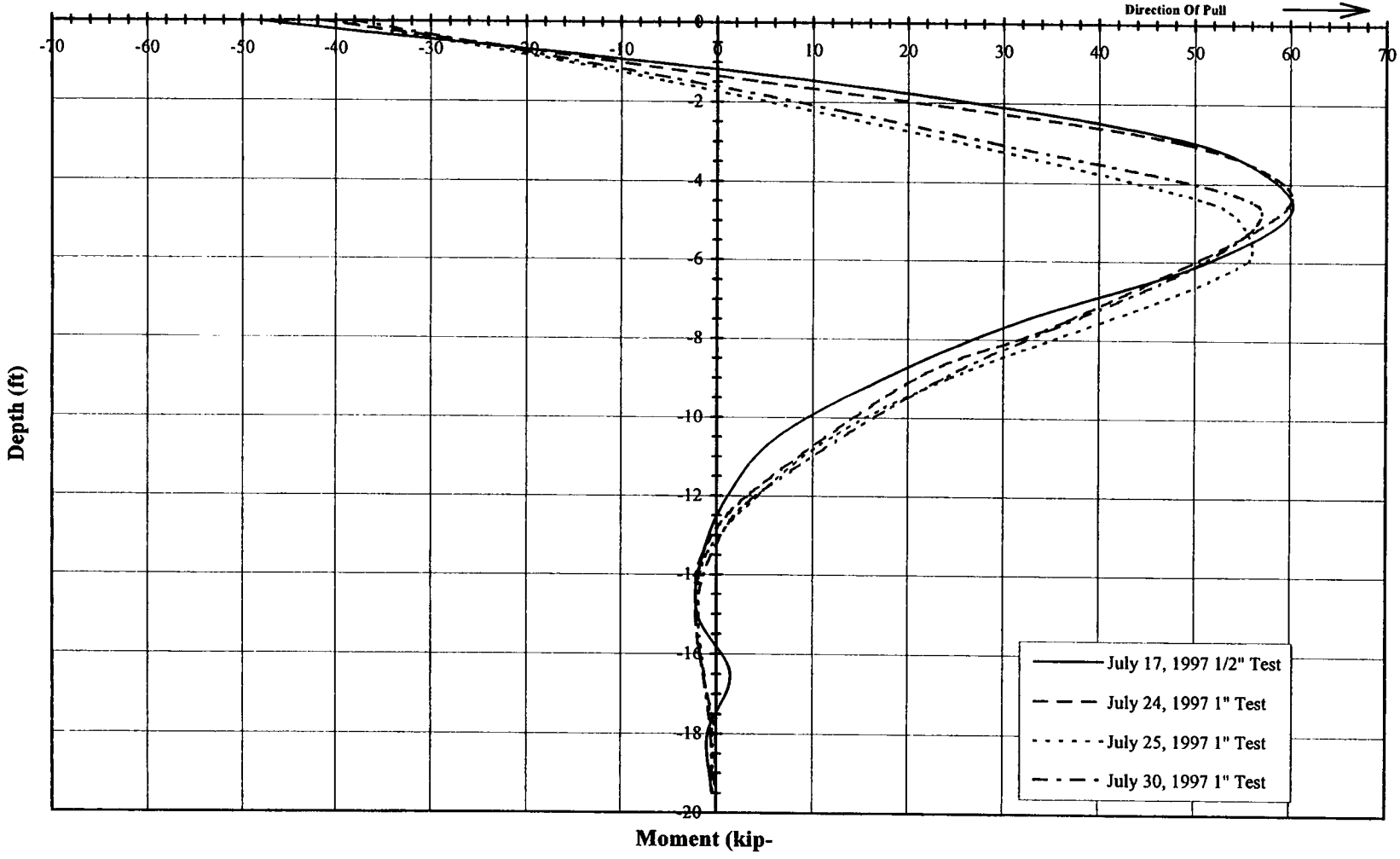

Load vs Deflection
Pile #2 East
Load (kips)
Pile Deflection (in.)
0
10
20
30
40
50
60
70
0
0.125
0.25
0.375
0.5
0.625
0.75
0.875
1
1.125
2
3
4
5
6
2: 1/2" Test - 1/30/98
3: 1/2" Test - 1/30/98
4: 1" Test - 2/13/98
5: 1" Test - 2/20/98
6: 1" Test - 2/24/98

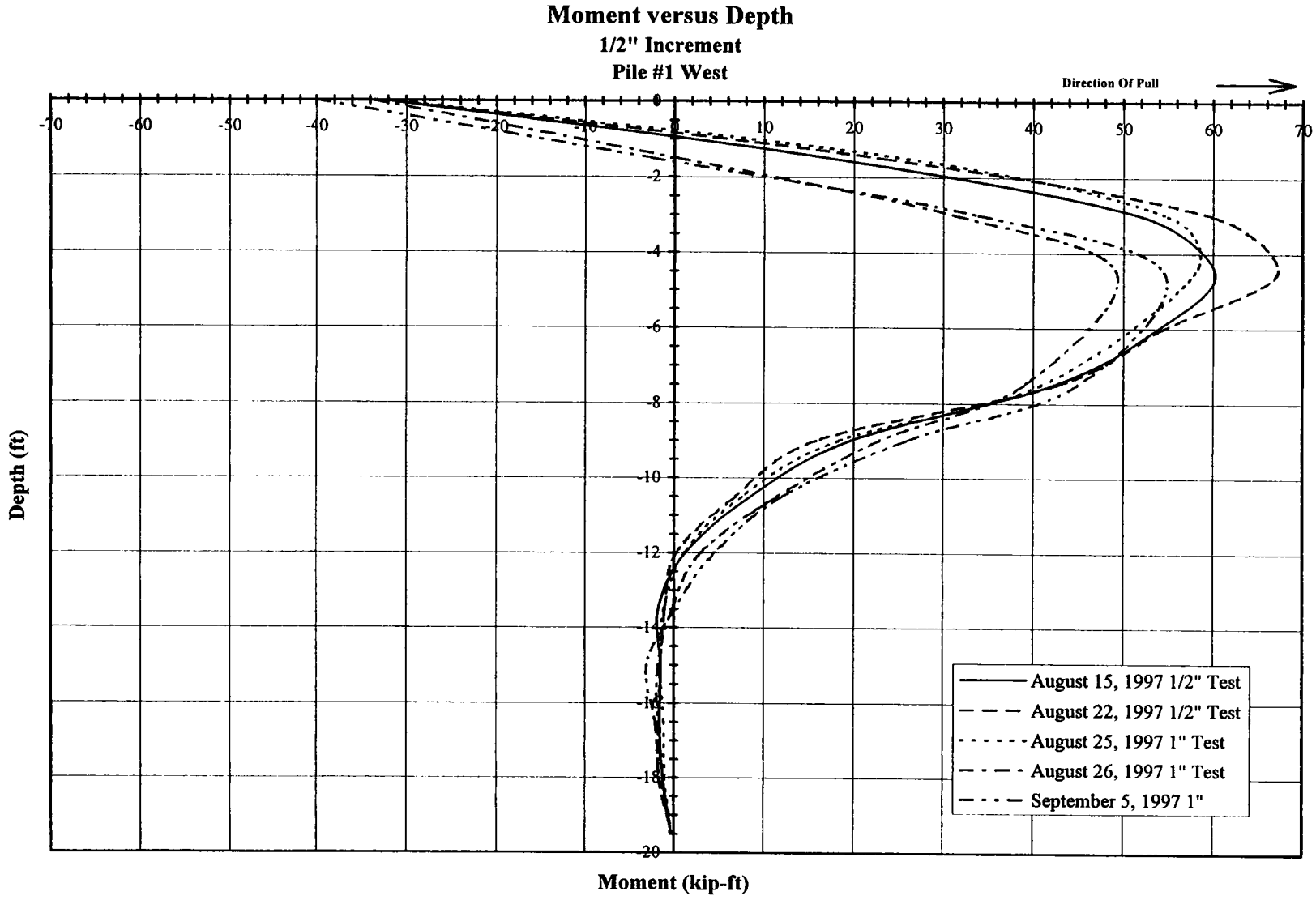

Reaction vs. Load
Pile #1 East
Reaction (kips)
Load (kips)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
1/2" Test - 6/20/97
1/2" Test - 6/24/97
1/2" Test - 7/17/97
1" Test - 7/24/97
1" Test - 7/25/97
1" Test - 7/30/97
Trendline for all test
y = 0.7309x + 1.5558
R2 = 0.9918

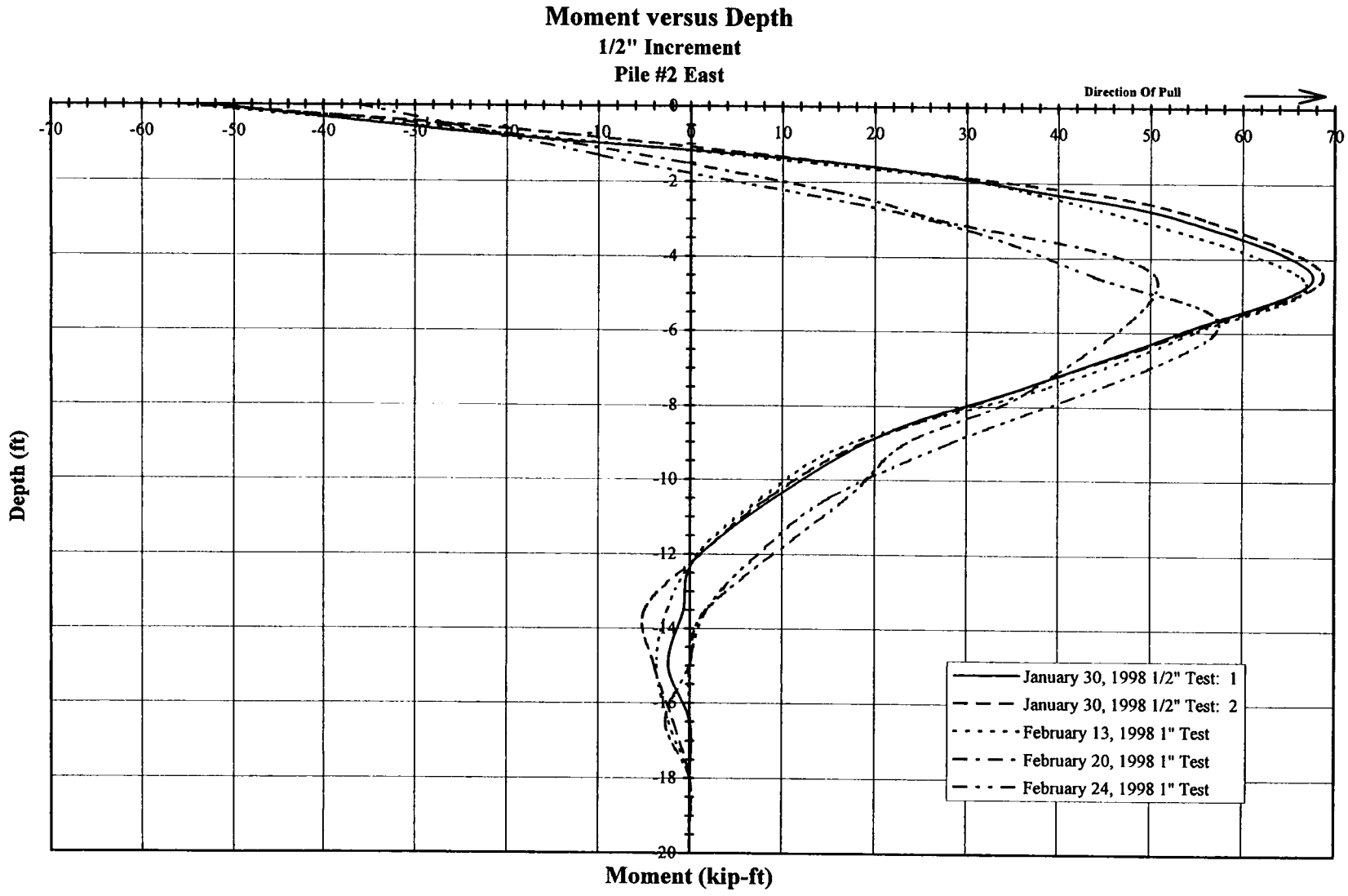

Reaction vs. Load
Pile #1 West
Reaction (kips)
Load (kips)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
1/2" Test - 8/15/97
1/2" Test - 8/19/97
1/2" Test - 8/22/97
1" Test - 8/25/97
1" Test - 8/26/97
1" Test - 9/5/97
Trendline for all test
y = 0.6739x + 2.4094
R² = 0.9943


Reaction vs. Load
Pile #2 East
Reaction (kips)
Load (kips)
0
10
20
30
40
50
60
0
5
10
15
20
25
30
35
40
45
50
55
60
65
1/2" Test - 1/30/98
1/2" Test - 1/30/98
1" Test - 2/13/98
1" Test - 2/20/98
1" Test - 2/24/98
Trendline for all test
y = 0.752x + 1.7605
R2 = 0.9931

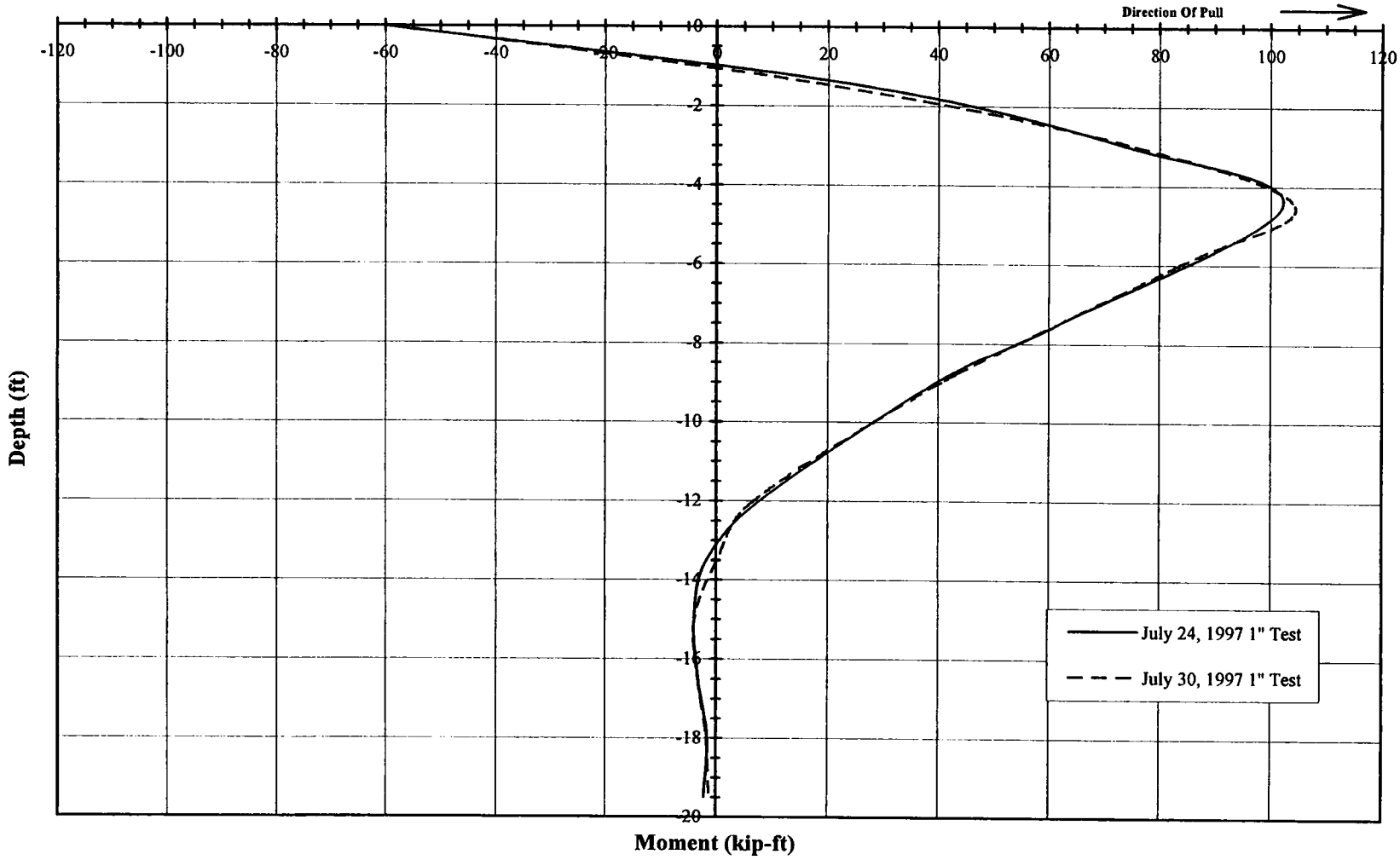
Moment versus Depth
1/2" Increment
Pile #1 East

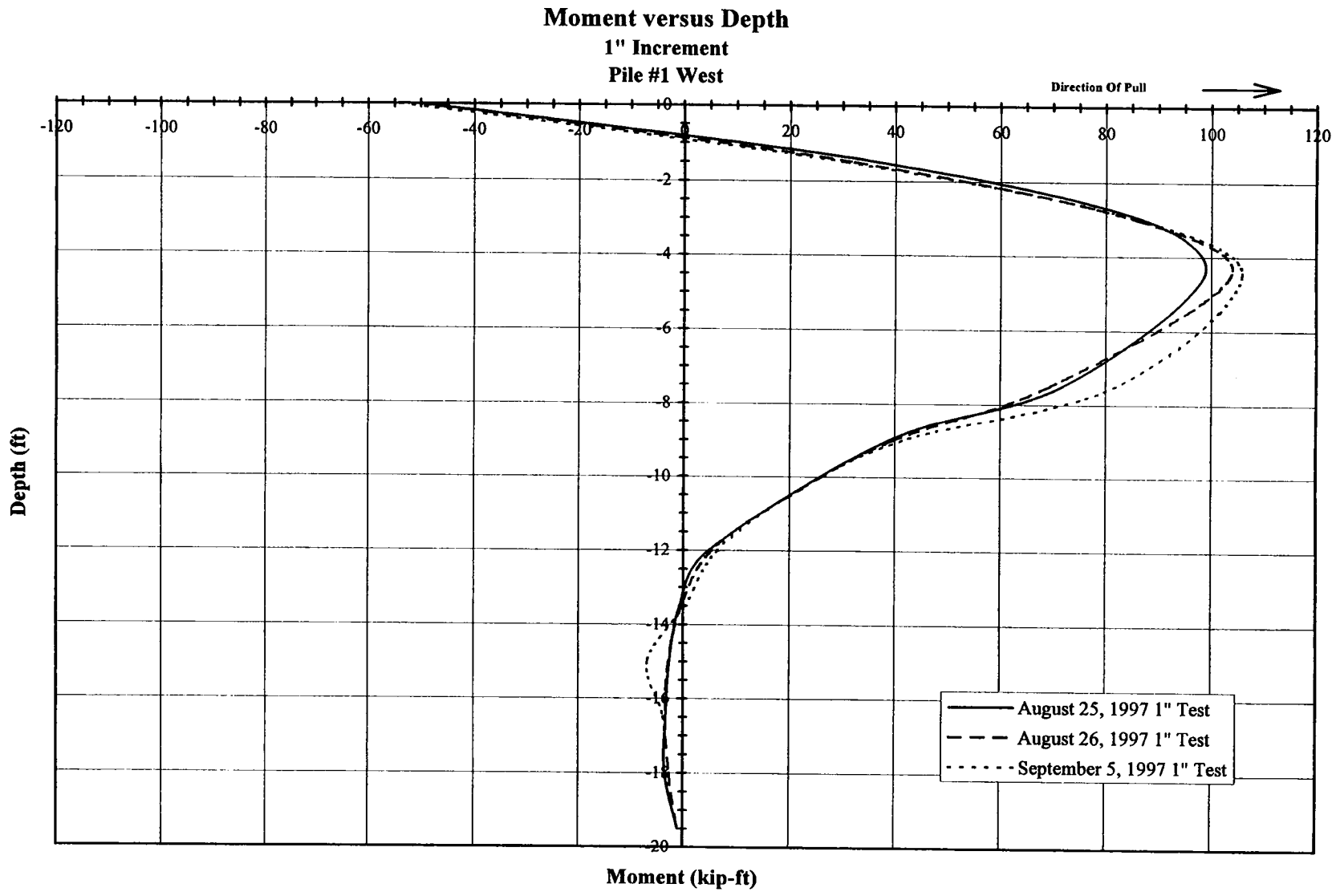


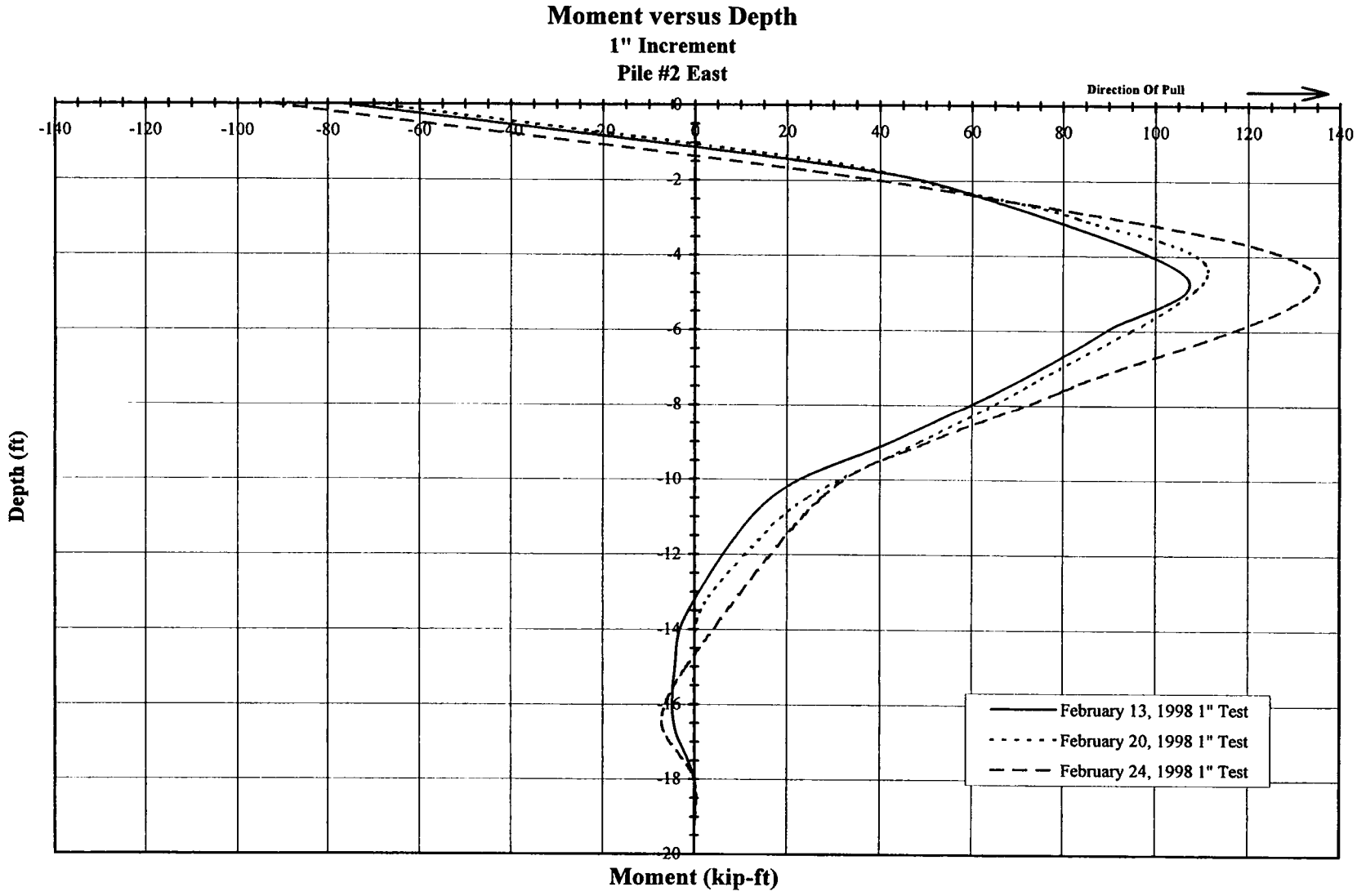

Moment versus Depth
1/2" Increment
Pile #1 West
Direction Of Pull
-70
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
70
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Moment (kip-ft)
August 15, 1997 1/2" Test
August 22, 1997 1/2" Test
August 25, 1997 1" Test
August 26, 1997 1" Test
September 5, 1997 1"


Moment versus Depth
1/2" Increment
Pile #2 East
Direction Of Pull
-70
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
70
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Moment (kip-ft)
January 30, 1998 1/2" Test: 1
January 30, 1998 1/2" Test: 2
February 13, 1998 1" Test
February 20, 1998 1" Test
February 24, 1998 1" Test

Moment versus Depth
1" Increment
Pile #1 East




Moment versus Depth
1" Increment
Pile #1 West
Direction Of Pull
-120
-100
-80
-60
-40
-20
0
20
40
60
80
100
120
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Moment (kip-ft)
August 25, 1997 1" Test
August 26, 1997 1" Test
September 5, 1997 1" Test


Moment versus Depth
1" Increment
Pile #2 East
Direction Of Pull
-140
-120
-100
-80
-60
-40
-20
0
20
40
60
80
100
120
140
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Moment (kip-ft)
February 13, 1998 1" Test
February 20, 1998 1" Test
February 24, 1998 1" Test

APPENDIX B

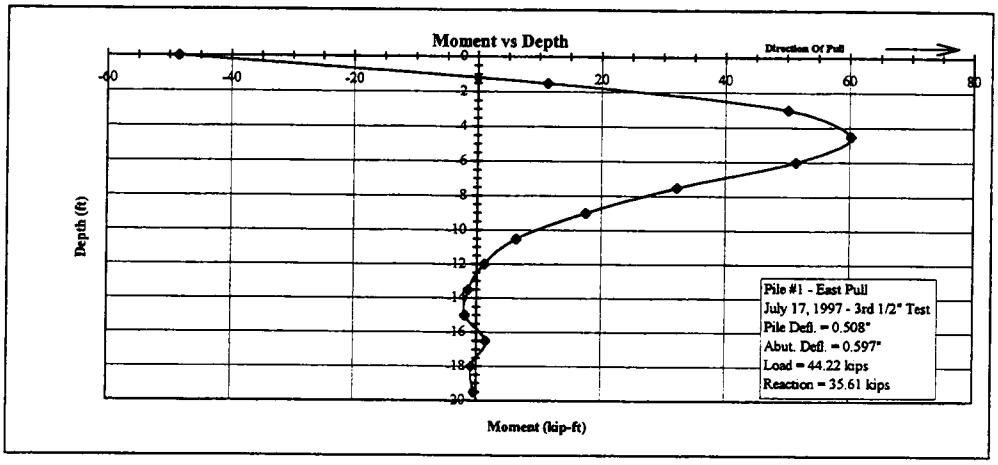
ONE-HALF INCH INCREMENT

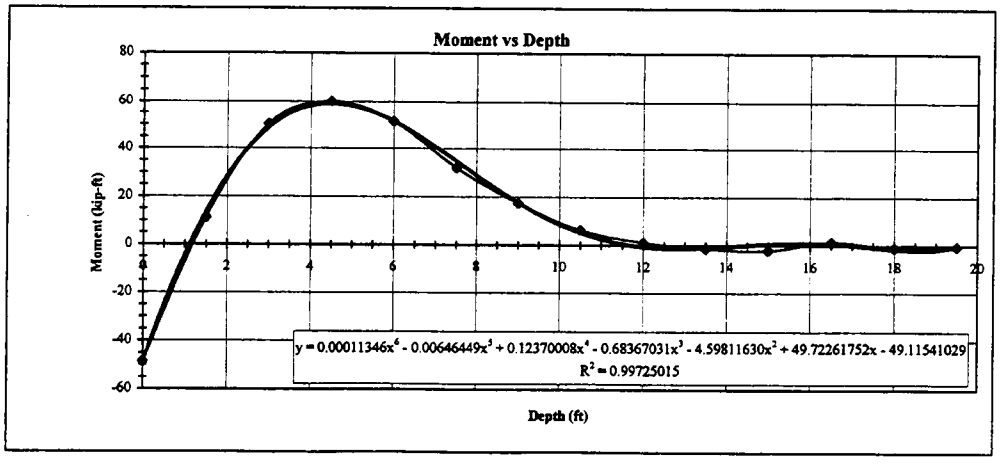
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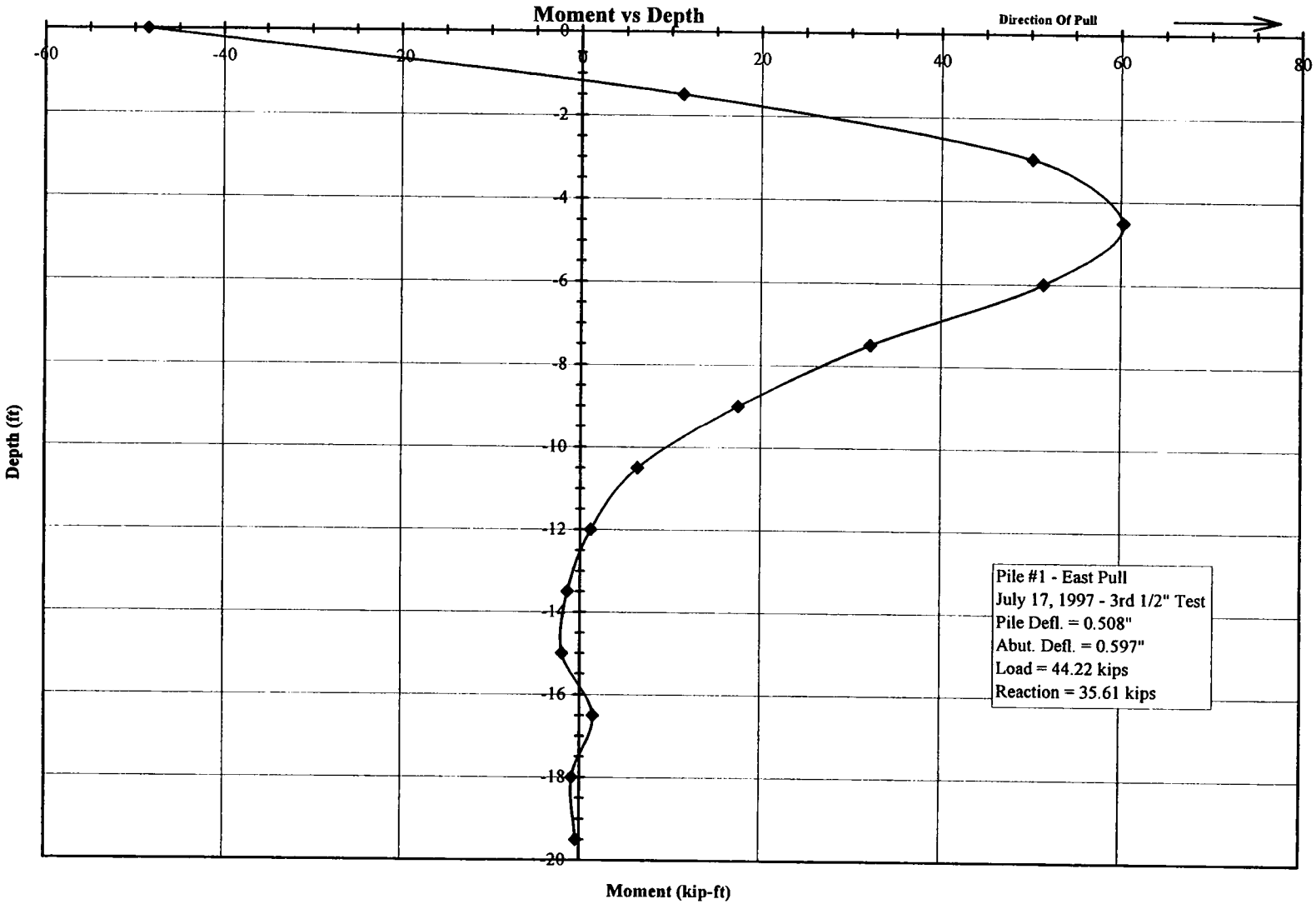
Moments

Date	07/17/97	
Time	7360 - 7370	sec
Load	44.22	kips
Reaction	35.61	kips
Abut. Defl.	0.597	inches
Pile Defl.	0.508	inches

Moment (kip-ft)	Depth (ft)	Station	Depth (ft)
-48.492	0	1	0
11.281	-1.5	2	1.5
50.198	-3	3	3
60.291	-4.5	4	4.5
51.449	-6	5	6
32.195	-7.5	6	7.5
17.534	-9	7	9
6.335	-10.5	8	10.5
1.169	-12	9	12
-1.404	-13.5	10	13.5
-1.984	-15	11	15
1.469	-16.5	12	16.5
-0.896	-18	13	18
-0.415	-19.5	14	19.5






Moment vs Depth
Direction Of Pull
-60
-40
-20
0
20
40
60
80
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Moment (kip-ft)
Pile #1 - East Pull
July 17, 1997 - 3rd 1/2" Test
Pile Defl. = 0.508"
Abut. Defl. = 0.597"
Load = 44.22 kips
Reaction = 35.61 kips

Equations

Date	07/17/97	
Time	7360 - 7370	sec
Load	44.22	kips
Reaction	35.61	kips
Abut. Defl.	0.597	inches
Pile Defl.	0.508	inches $= C_2$
Modulus of Elasticity (E) =	33000	ksi
Moment of Inertia (I) =	223	in^4

Deflection Equation:
From 0' to 19.5'

$$y = 4.75751E\text{-}10\,x^8 + -3.61418E\text{-}08\,x^7 + 9.68219E\text{-}07\,x^6 + -8.02679E\text{-}06\,x^5 + -8.99754E\text{-}05\,x^4 + 0.001945932\,x^3 + -0.005766506\,x^2 + (C_1)\,x + 0.508\,(C_2)$$

Slope Equation:
From 0' to 19.5'

$$y = 3.17167E\text{-}10\,x^7 + -2.10827E\text{-}08\,x^6 + 4.8411E\text{-}07\,x^5 + -3.34449E\text{-}06\,x^4 + -2.99918E\text{-}05\,x^3 + 0.000486483\,x^2 + -0.000961084\,x + (C_1)$$

Moment Equation:
From 0' to 19.5'

$$\mathbf{y = 0.00011346\,x^6 + -0.00646449\,x^5 + 0.12370008\,x^4 + -0.68367031\,x^3 + -4.59811630\,x^2 + 49.72261752\,x + -49.11541029}$$

Shear Equation:
From 0' to 19.5'

$$y = 0.00068076\,x^5 + -0.03232245\,x^4 + 0.49480032\,x^3 + -2.05101093\,x^2 + -9.19623260\,x + 49.72261752$$

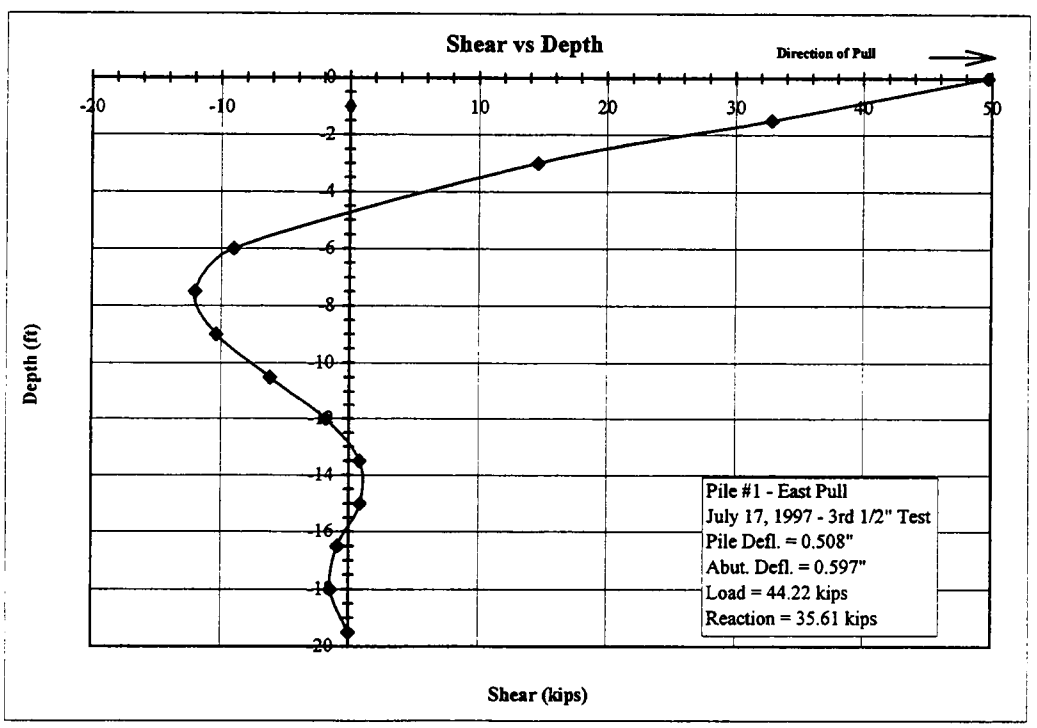
Pressure Equation:
From 0' to 19.5'

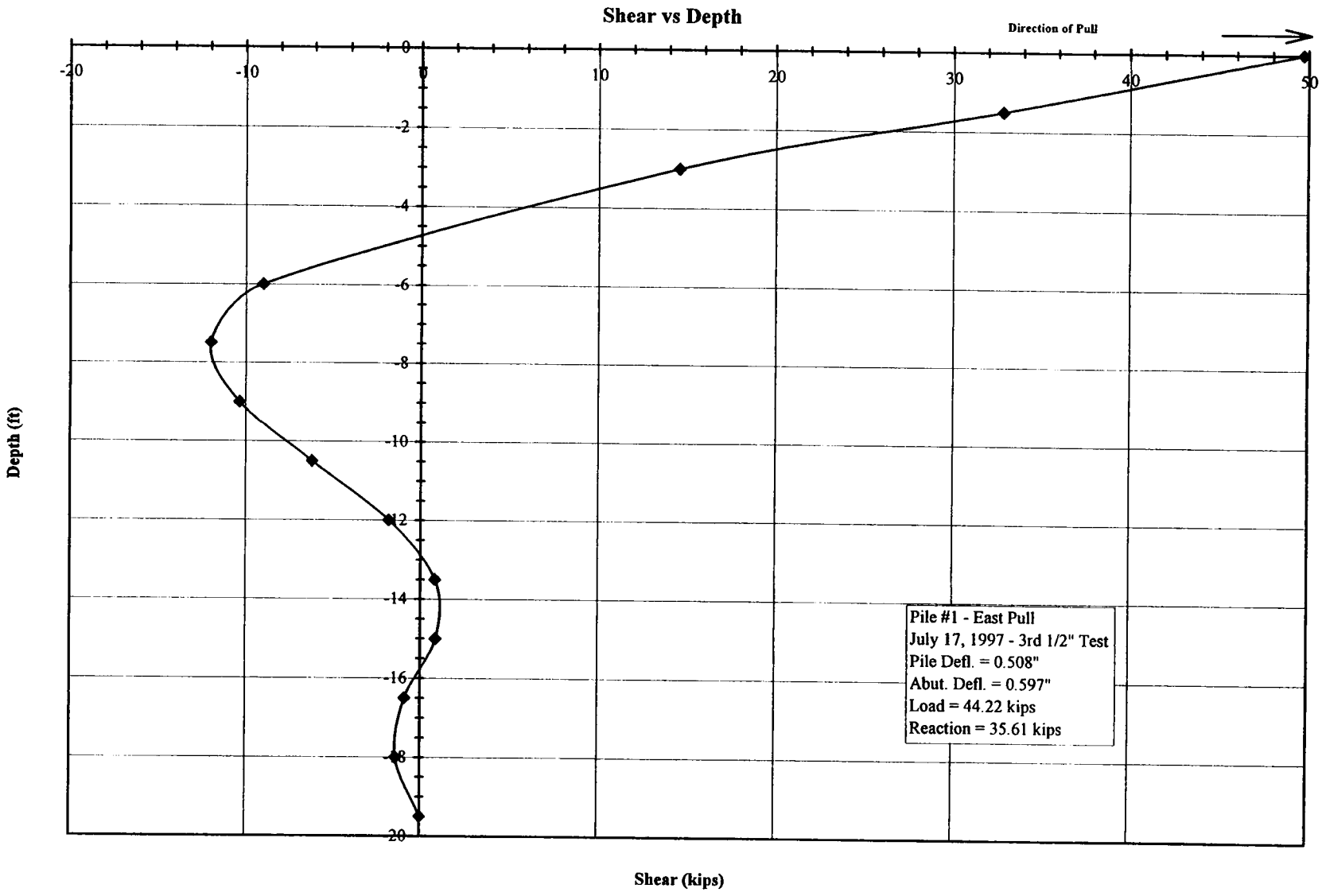
$$y = 0.0034038\,x^4 + -0.1292898\,x^3 + 1.48440096\,x^2 + -4.10202186\,x + -9.19623260$$

Shear

Date	07/17/97	
Time	7360 - 7370	sec
Load	44.22	kips
Reaction	35.61	kips
Abut. Def	0.597	inches
Pile Defl.	0.508	inches

Shear (kips)	Depth (ft)	Station	Depth (ft)
49.723	0	1	0
32.825	-1.5	2	1.5
14.582	-3	3	3
-9.011	-6	5	6
-11.990	-7.5	6	7.5
-10.335	-9	7	9
-6.166	-10.5	8	10.5
-1.806	-12	9	12
0.834	-13.5	10	13.5
0.881	-15	11	15
-0.884	-16.5	12	16.5
-1.401	-18	13	18
0.000	-19.5	14	19.5

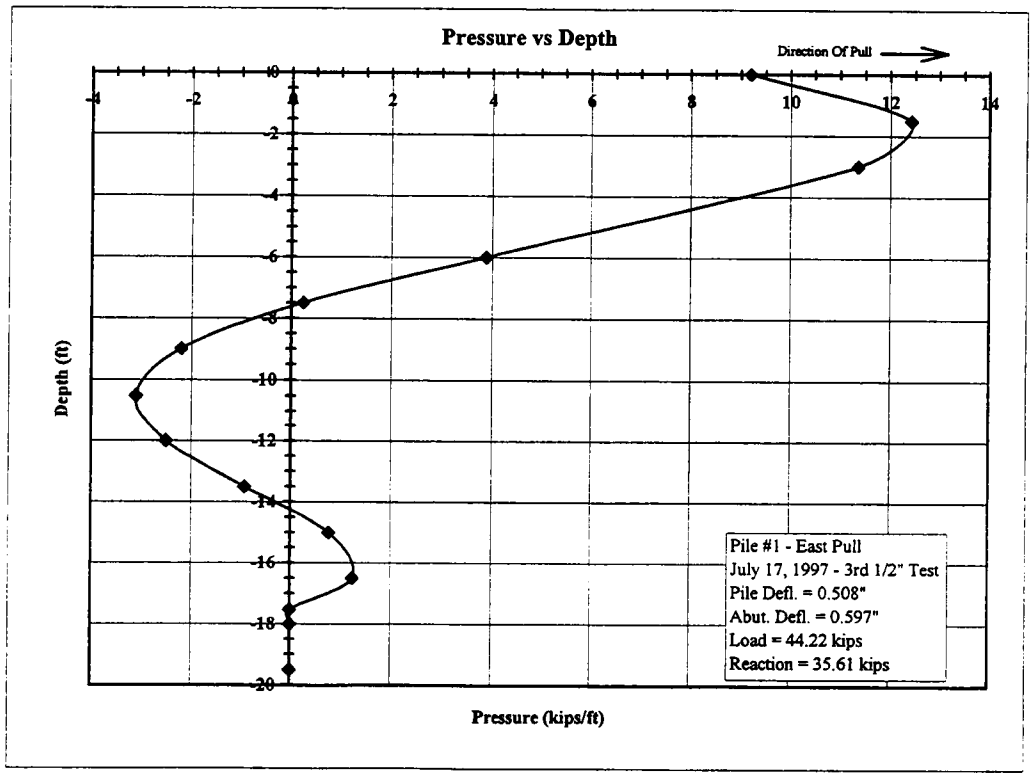


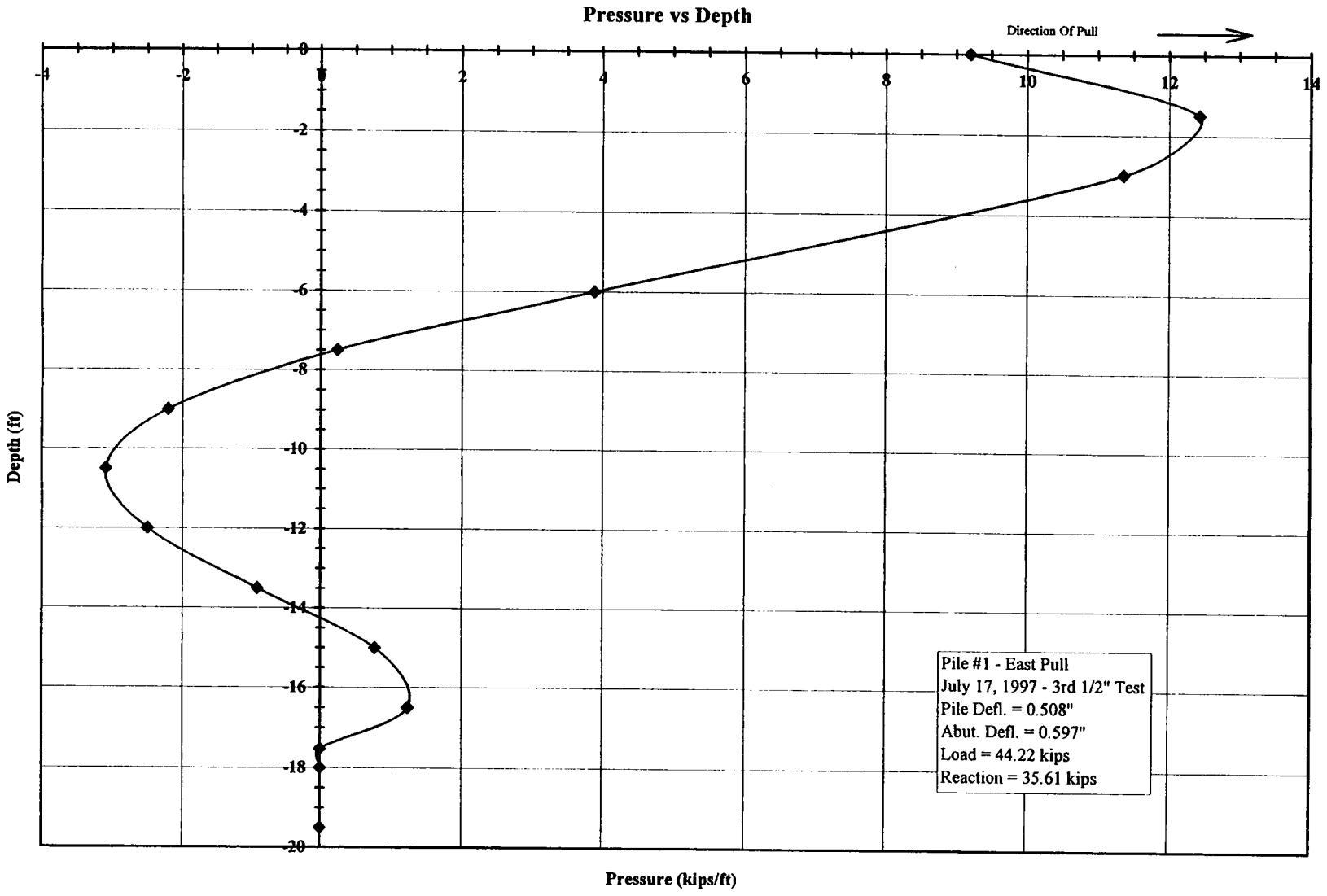

Shear vs Depth
Direction of Pull
-20
-10
0
10
20
30
40
50
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Shear (kips)
Pile #1 - East Pull
July 17, 1997 - 3rd 1/2" Test
Pile Defl. = 0.508"
Abut. Defl. = 0.597"
Load = 44.22 kips
Reaction = 35.61 kips

Pressure

Date	07/17/97	
Time	7360 - 7370	sec
Load	44.22	kips
Reaction	35.61	kips
Abut. Defl.	0.597	inches
Pile Defl.	0.508	inches

Pressure (kips/ft)	Depth (ft)	Station	Depth (ft)
9.196	0	1	0
12.428	-1.5	2	1.5
11.358	-3	3	3
3.885	-6	5	6
0.238	-7.5	6	7.5
-2.202	-9	7	9
-3.092	-10.5	8	10.5
-2.502	-12	9	12
-0.915	-13.5	10	13.5
0.772	-15	11	15
1.248	-16.5	12	16.5
0.000	-17.5277		17.5277
0.000	-18	13	18
0.000	-19.5	14	19.5




Pressure vs Depth
Direction Of Pull
Depth (ft)
Pressure (kips/ft)
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-4
-2
2
4
6
8
10
12
14
Pile #1 - East Pull
July 17, 1997 - 3rd 1/2" Test
Pile Defl. = 0.508"
Abut. Defl. = 0.597"
Load = 44.22 kips
Reaction = 35.61 kips

Statics

Date	07/17/97	
Time	7360 - 7370	sec
Load	44.22	kips
Reaction	35.61	kips
Abut. Defl.	0.597	inches
Pile Defl.	0.508	inches

Pressure Equation

$y =$ 0.0034038 $x^4 +$ -0.1292898 $x^3 +$ 1.48440096 $x^2 +$ -4.10202186 $x +$ -9.19623260

Roots for pressure equation

x = -1.4194 x = 7.61572 x = 14.2599 x = 17.5277

Integrate to check statics:

From 0 to 7.61572

= 0.000680760 $x^5 +$ -0.032322450 $x^4 +$ 0.494800320 $x^3 +$ -2.051010930 $x^2 +$ -9.19623260 x

= -61.72642832 - 0 = **-61.72643 kips**

From 7.61572 to 14.2599

= 0.000680760 $x^5 +$ -0.032322450 $x^4 +$ 0.494800320 $x^3 +$ -2.051010930 $x^2 +$ -9.19623260 x

= -48.54380799 - -61.72642832 = **13.18262 kips**

From 14.2599 to 17.5277

= 0.000680760 $x^5 +$ -0.032322450 $x^4 +$ 0.494800320 $x^3 +$ -2.051010930 $x^2 +$ -9.19623260 x

= -51.39154245 - -48.54380799 = **-2.84773 kips**

From Pressure	=	**-51.392**	**kips**
Measured	=	**44.22**	**kips**
% difference	=	***16.22***	**%**

Moment by Statics

M_{GRND} = (Reaction)(x) - (Load)(y) - (Weight)(e)

x =	6.667	ft	(distance from centerline of the pile to centerline of the reaction)
y =	4.2083	ft	(distance from centerline of the load to the ground surface)
e =	0.089	in	($e = \Delta_{abut} - \Delta_{pile}$)
Weight =	65.0	kips	

M_{GRND} =	237.412	-	186.093	-	0.482	=	**50.837**	**kip-ft**
					Measured	=	**48.492**	**kip-ft**
					% difference	=	***4.84***	**%**

Deflection

Date	07/17/97	
Time	7360 - 7370	sec
Load	44.22	kips
Reaction	35.61	kips
Abut. Defl.	0.597	inches
Pile Defl.	0.508	inches

@ x = 0

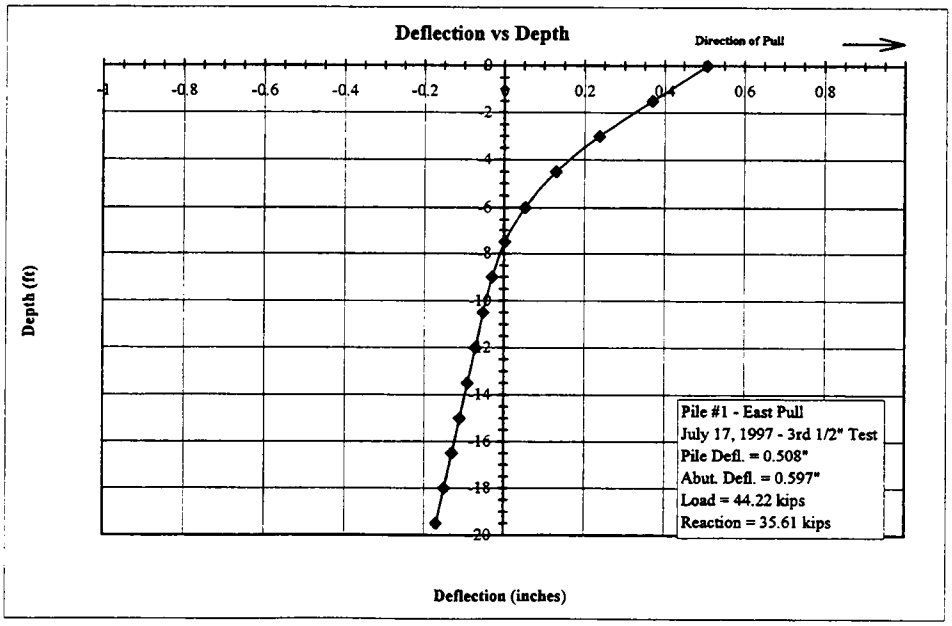
y = 0.000 + 0 C_1 + C_2 = 0.508

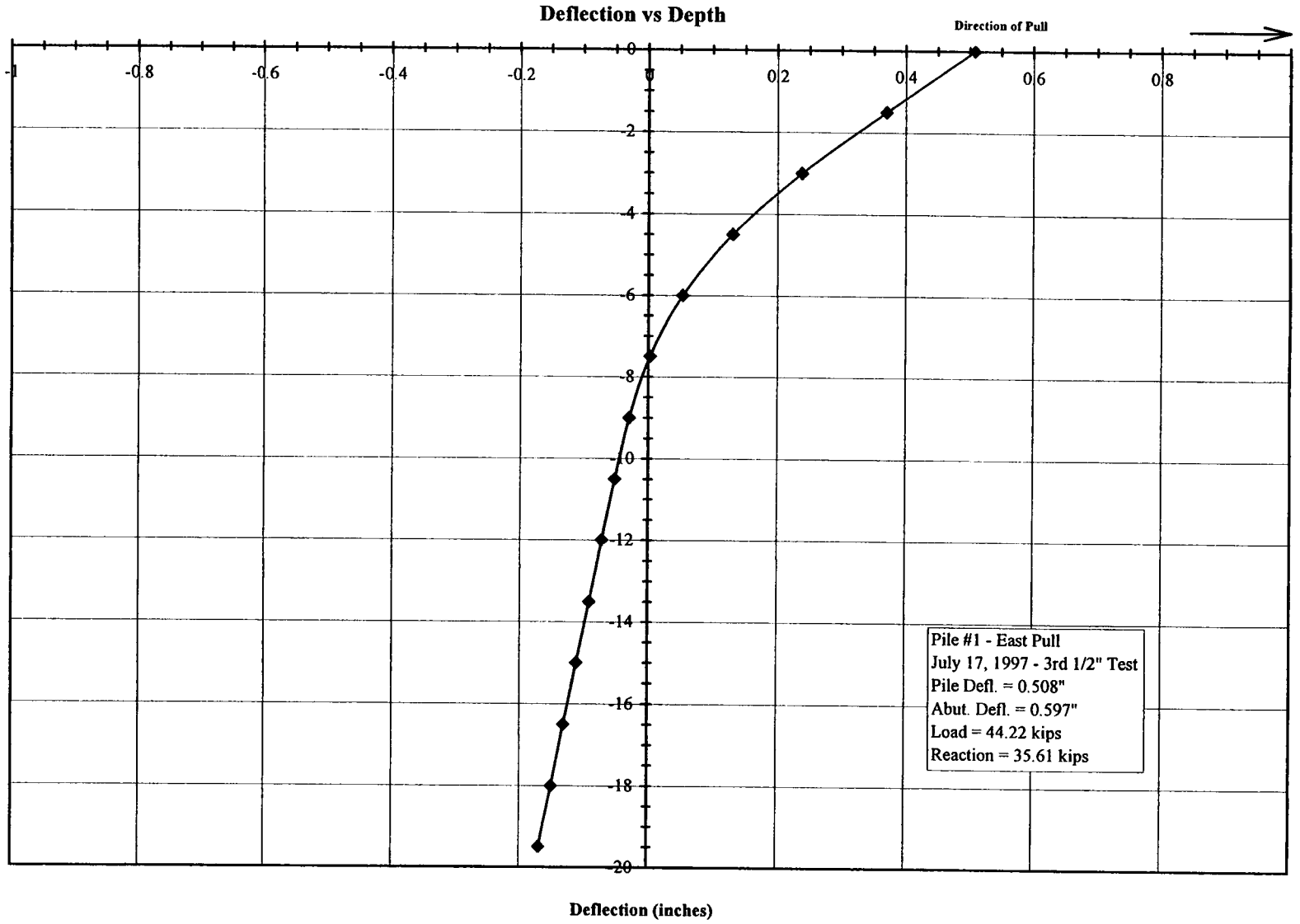
@ x = 7.61572

y = 0.157 + 7.61572 C_1 + 0.508 = 0

C_1 = -0.0873646 = -0.0072804 radians

Deflection (in)	Depth (ft)	Station	Depth (ft)
0.508	0	1	0
0.370	-1.5	2	1.5
0.238	-3	3	3
0.130	-4.5	4	4.5
0.053	-6	5	6
0.003	-7.5	6	7.5
-0.029	-9	7	9
-0.051	-10.5	8	10.5
-0.071	-12	9	12
-0.090	-13.5	10	13.5
-0.110	-15	11	15
-0.130	-16.5	12	16.5
-0.149	-18	13	18
-0.168	-19.5	14	19.5




Deflection vs Depth
Direction of Pull
-1
-0.8
-0.6
-0.4
-0.2
0
0.2
0.4
0.6
0.8
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Deflection (inches)
Pile #1 - East Pull
July 17, 1997 - 3rd 1/2" Test
Pile Defl. = 0.508"
Abut. Defl. = 0.597"
Load = 44.22 kips
Reaction = 35.61 kips

APPENDIX C

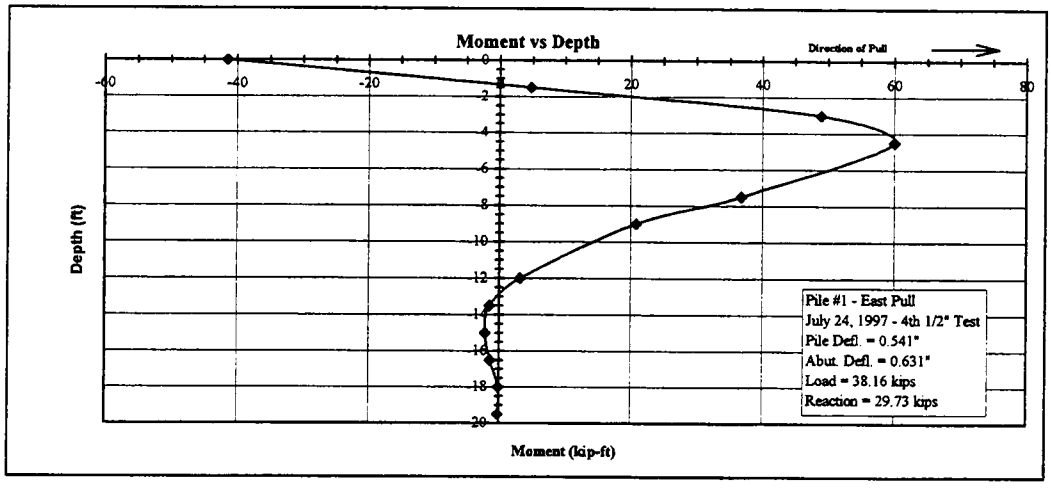
ONE-HALF INCH INCREMENT

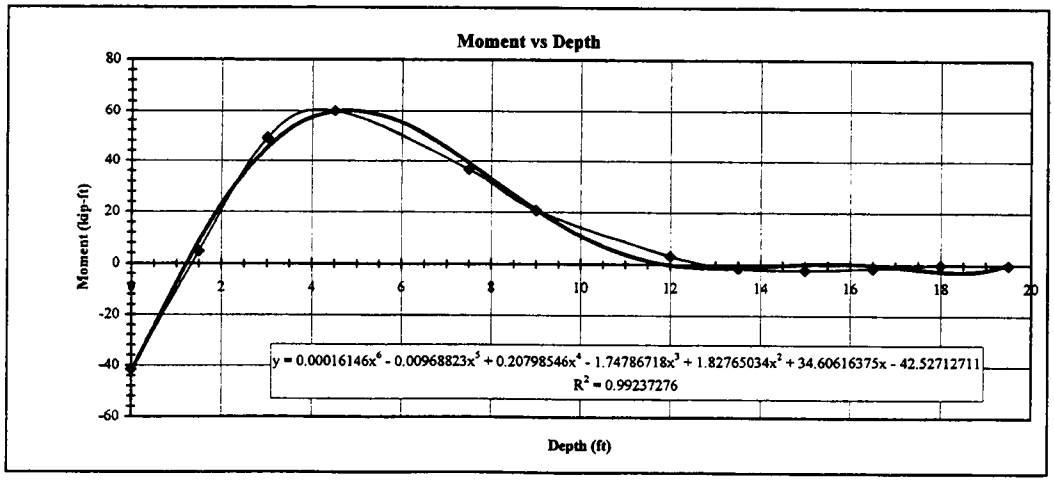
FOR JULY 24, 1997

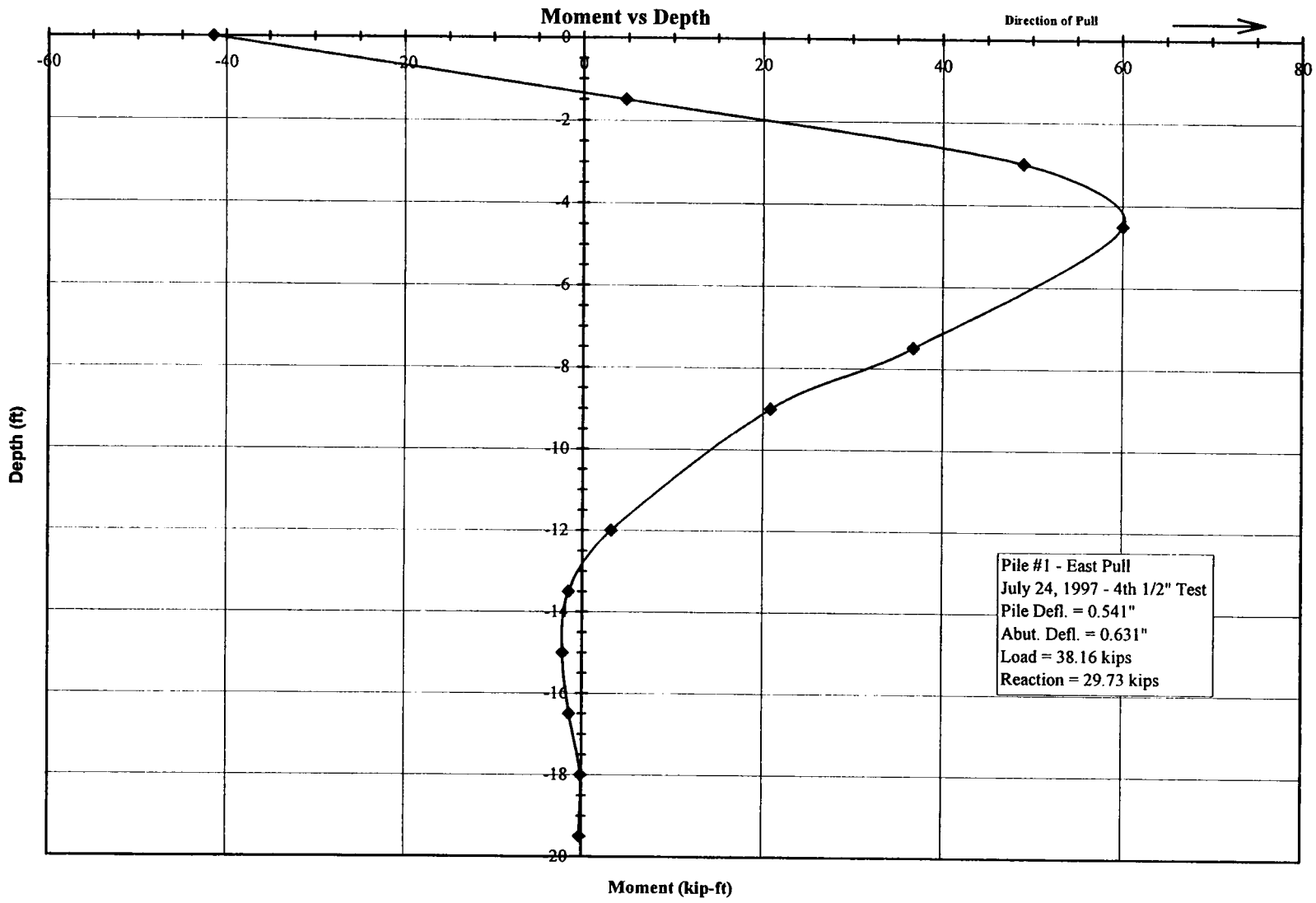
Moments

Date	07/24/98	
Time	8102 - 8112	sec
Load	38.16	kips
Reaction	29.73	kips
Abut. Defl.	0.631	inches
Pile Defl.	0.541	inches

Moment (kip-ft)	Depth (ft)	Station	Depth (ft)
-41.479	0	1	0
4.731	-1.5	2	1.5
48.955	-3	3	3
60.042	-4.5	4	4.5
36.750	-7.5	6	7.5
20.845	-9	7	9
3.186	-12	9	12
-1.463	-13.5	10	13.5
-2.146	-15	11	15
-1.375	-16.5	12	16.5
-0.093	-18	13	18
-0.203	-19.5	14	19.5






Moment vs Depth
Direction of Pull
-60
-40
-20
0
20
40
60
80
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Moment (kip-ft)
Pile #1 - East Pull
July 24, 1997 - 4th 1/2" Test
Pile Defl. = 0.541"
Abut. Defl. = 0.631"
Load = 38.16 kips
Reaction = 29.73 kips

Equations

Date	07/24/98		
Time	8102 - 8112	sec	
Load	38.16	kips	
Reaction	29.73	kips	
Abut. Defl.	0.631	inches	
Pile Defl.	0.541	inches	$= C_2$

Modulus of Elasticity (E) =	33000	ksi
Moment of Inertia (I) =	223	in^4

Deflection Equation:
From 0' to 19.5'

$$y = 6.77021E\text{-}10\ x^8 + -5.41652E\text{-}08\ x^7 + 1.62793E\text{-}06\ x^6 + -2.05212E\text{-}05\ x^5 + 3.57632E\text{-}05\ x^4 + 0.001354338\ x^3 + -0.004992993\ x^2 + (C_1)\ x + 0.541\ (C_2)$$

Slope Equation:
From 0' to 19.5'

$$y = 4.51347E\text{-}10\ x^7 + -3.15963E\text{-}08\ x^6 + 8.13967E\text{-}07\ x^5 + -8.55051E\text{-}06\ x^4 + 1.19211E\text{-}05\ x^3 + 0.000338585\ x^2 + -0.000832166\ x + (C_1)$$

Moment Equation:
From 0' to 19.5'

$$\mathbf{y = 0.00016146\ x^6 + -0.00968823\ x^5 + 0.20798546\ x^4 + -1.74786718\ x^3 + 1.82765034\ x^2 + 34.60616375\ x + -42.52712711}$$

Shear Equation:
From 0' to 19.5'

$$y = 0.00096876\ x^5 + -0.04844115\ x^4 + 0.83194184\ x^3 + -5.24360154\ x^2 + 3.65530068\ x + 34.60616375$$

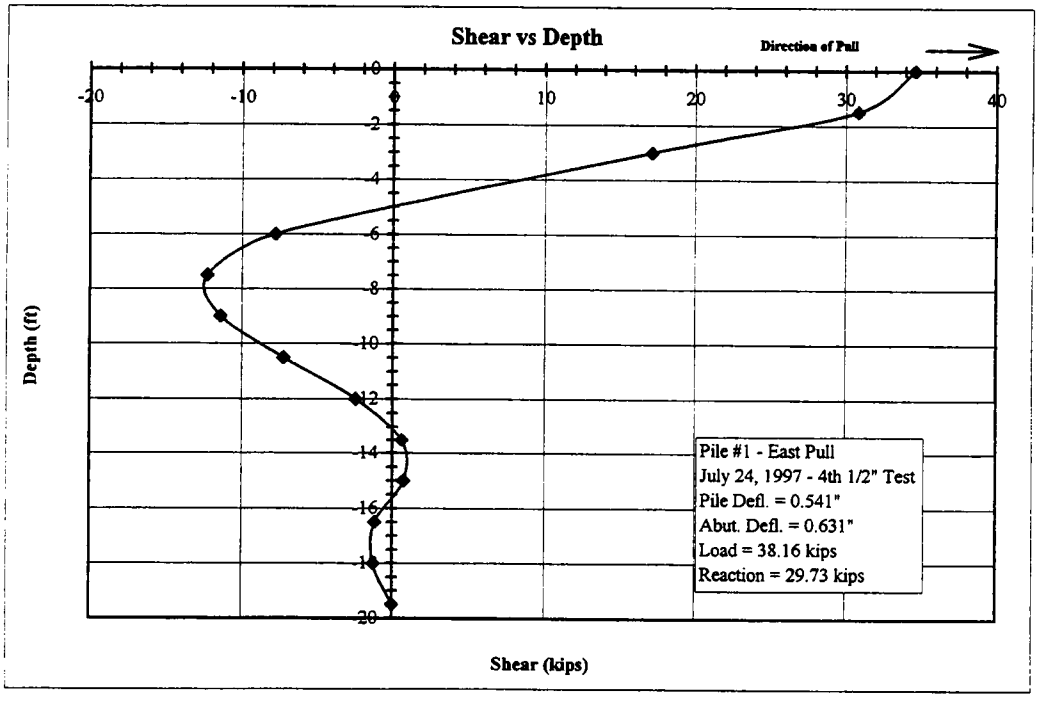
Pressure Equation:
From 0' to 19.5'

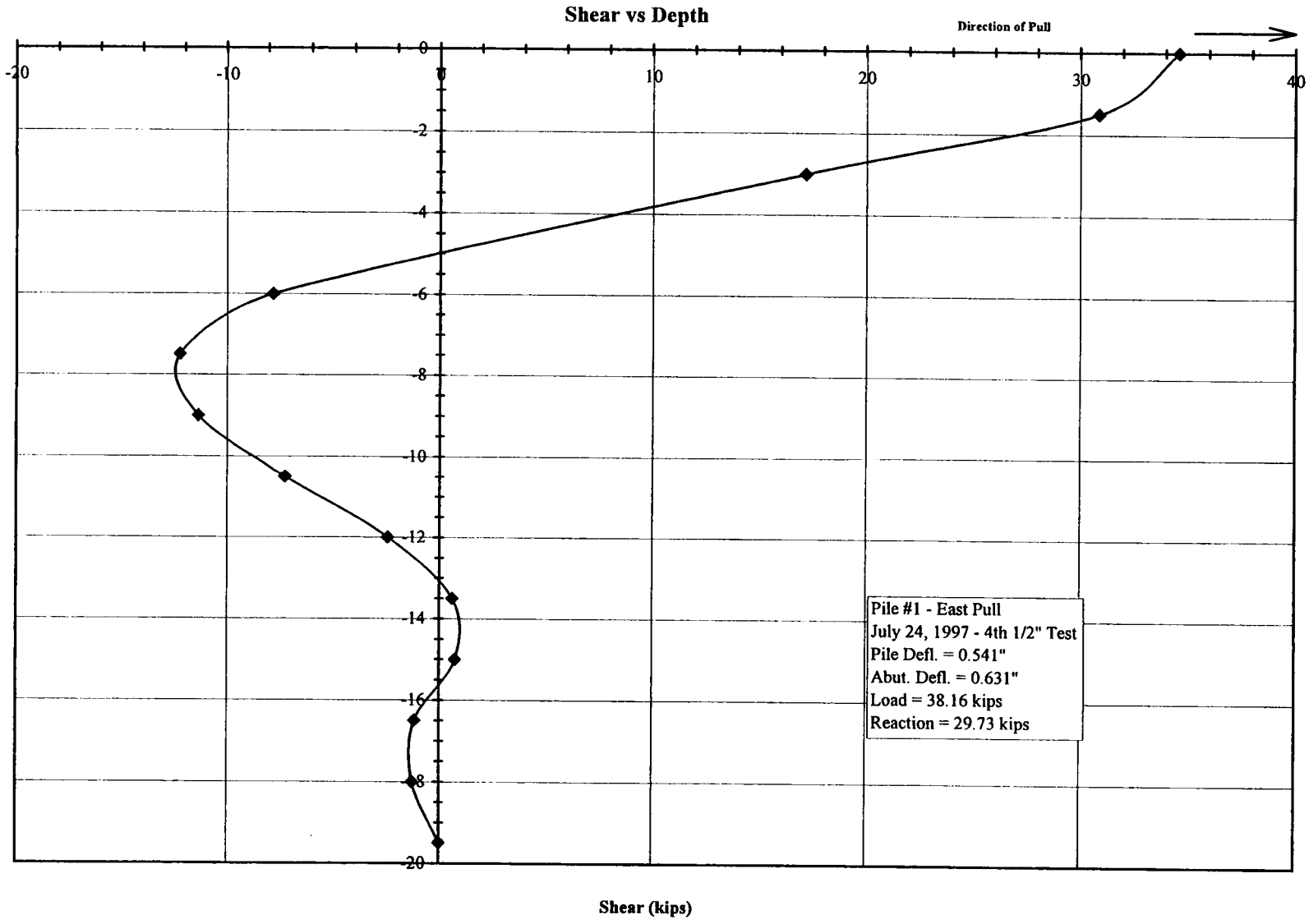
$$y = 0.0048438\ x^4 + -0.1937646\ x^3 + 2.49582552\ x^2 + -10.48720308\ x + 3.65530068$$

Shear

Date	07/24/98	
Time	8102 - 8112	sec
Load	38.16	kips
Reaction	29.73	kips
Abut. Defl.	0.631	inches
Pile Defl.	0.541	inches

Shear (kips)	Depth (ft)	Station	Depth (ft)
34.606	0	1	0
30.861	-1.5	2	1.5
17.154	-3	3	3
-7.779	-6	5	6
-12.238	-7.5	6	7.5
-11.360	-9	7	9
-7.208	-10.5	8	10.5
-2.431	-12	9	12
0.615	-13.5	10	13.5
0.748	-15	11	15
-1.151	-16.5	12	16.5
-1.261	-18	13	18
0.000	-19.5	14	19.5

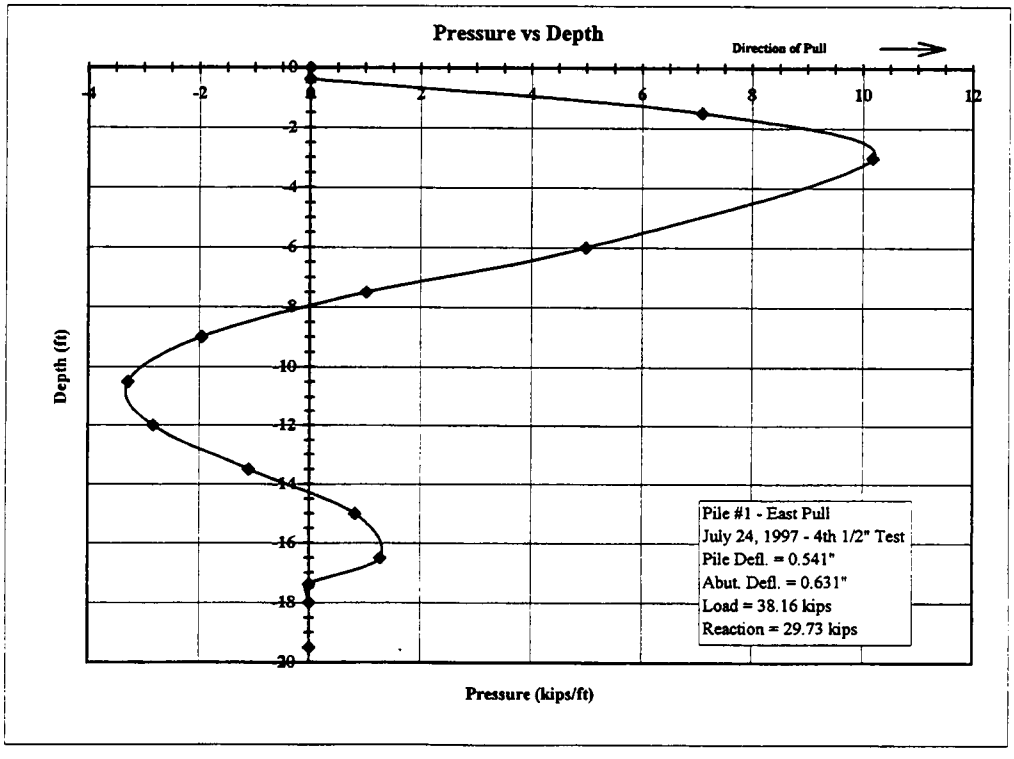


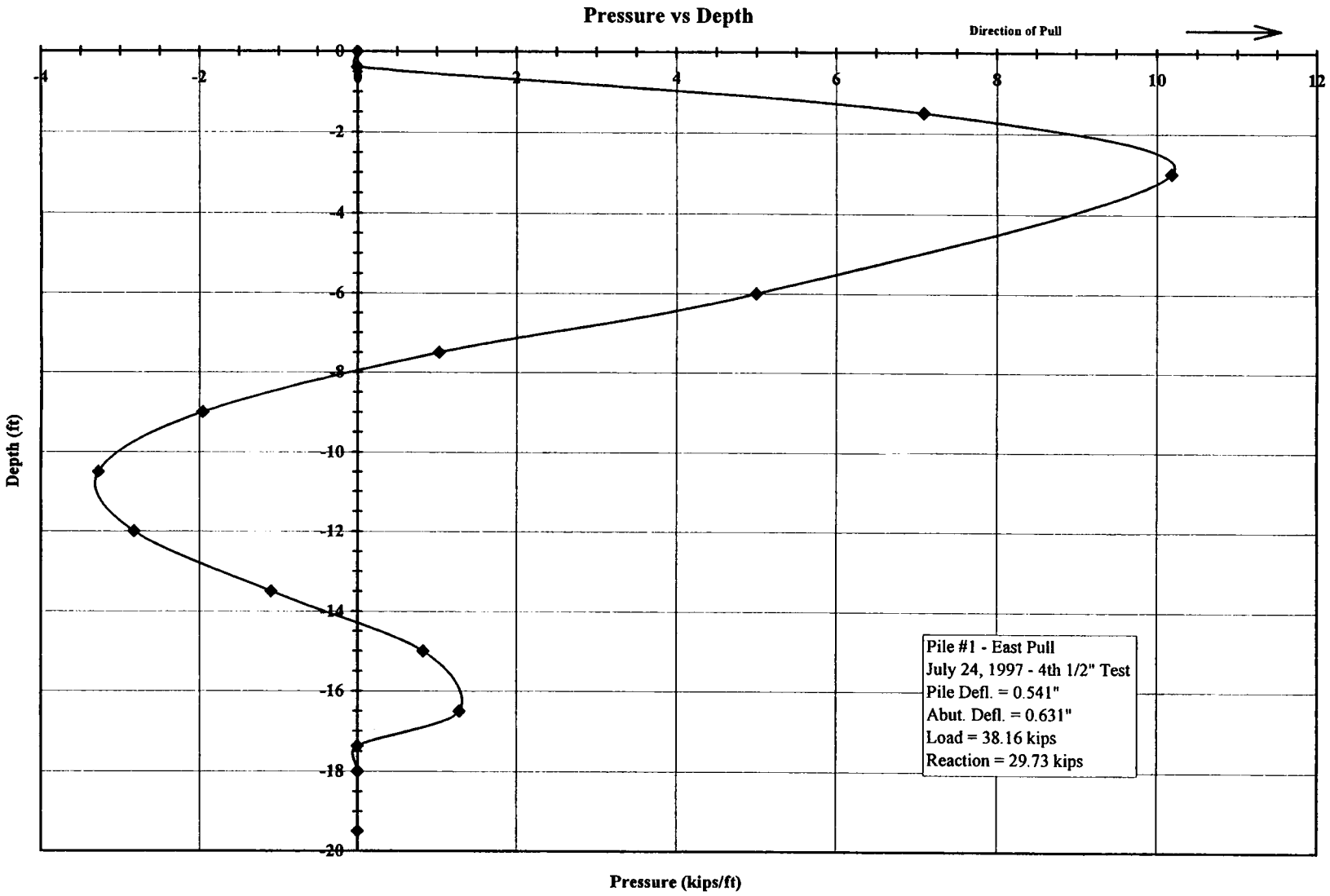

Shear vs Depth
Direction of Pull
-20
-10
0
10
20
30
40
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Shear (kips)
Pile #1 - East Pull
July 24, 1997 - 4th 1/2" Test
Pile Defl. = 0.541"
Abut. Defl. = 0.631"
Load = 38.16 kips
Reaction = 29.73 kips

Pressure

Date	07/24/98	
Time	8102 - 8112	sec
Load	38.16	kips
Reaction	29.73	kips
Abut. Defl.	0.631	inches
Pile Defl.	0.541	inches

Pressure (kips/ft)	Depth (ft)	Station	Depth (ft)
0.000	0	1	0
0.000	-0.38231		0.38231
7.089	-1.5	2	1.5
10.183	-3	3	3
4.994	-6	5	6
1.027	-7.5	6	7.5
-1.958	-9	7	9
-3.274	-10.5	8	10.5
-2.824	-12	9	12
-1.096	-13.5	10	13.5
0.830	-15	11	15
1.287	-16.5	12	16.5
0.000	-17.3761		17.3761
0.000	-18	13	18
0.000	-19.5	14	19.5




Pressure vs Depth
Direction of Pull
Depth (ft)
Pressure (kips/ft)
-4
-2
0
2
4
6
8
10
12
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Pile #1 - East Pull
July 24, 1997 - 4th 1/2" Test
Pile Defl. = 0.541"
Abut. Defl. = 0.631"
Load = 38.16 kips
Reaction = 29.73 kips

Statics

Date	07/24/98	
Time	8102 - 8112	sec
Load	38.16	kips
Reaction	29.73	kips
Abut. Defl.	0.631	inches
Pile Defl.	0.541	inches

Load by Statics

Pressure Equation

$y = 0.00484380\,x^4 + -0.19376460\,x^3 + 2.495825520\,x^2 + -10.48720308\,x + 3.65530068$

Roots for pressure equation

x = 0.38231 x = 7.94346 x = 14.30074 x = 17.3761

Integrate to check statics:

From 0 to 0.38231 (Assume to be zero due to previous pulls and no confinement at ground surface)

From 0.38231 to 7.94346

$= -0.000968760\,x^5 + 0.048441150\,x^4 + -0.831941840\,x^3 + 5.243601540\,x^2 + -3.65530068\,x$

= 47.06798625 - -0.676105712 = **47.74409 kips**

From 7.94346 to 14.30074

$= -0.000968760\,x^5 + 0.048441150\,x^4 + -0.831941840\,x^3 + 5.243601540\,x^2 + -3.65530068\,x$

= 33.55553794 - 47.06798625 = **-13.51245 kips**

From 14.30074 to 17.3761

$= -0.000968760\,x^5 + 0.048441150\,x^4 + -0.831941840\,x^3 + 5.243601540\,x^2 + -3.65530068\,x$

= 36.43192478 - 33.55553794 = **2.87639 kips**

From Pressure	=	**37.108**	**kips**
Measured	=	**38.16**	**kips**
% difference	=	***2.76***	**%**

Moment by Statics

M_{GRND} = (Reaction)(x) - (Load)(y) - (Weight)(e)

x =	6.75	ft	(distance from centerline of the pile to centerline of the reaction)
y =	4.2083	ft	(distance from centerline of the load to the ground surface)
e =	0.090	in	($e = \Delta_{abut} - \Delta_{pile}$)
Weight =	65.0	kips	

M_{GRND} =	200.678	-	160.590	-	0.488	=	**39.600** **kip-ft**
					Measured	=	**41.479** **kip-ft**
					% difference	=	***4.53*** **%**

Deflections

Date	07/24/97	
Time	8102 - 8112	sec
Load	38.16	kips
Reaction	29.73	kips
Abut. Defl.	0.631	inches
Pile Defl.	0.541	inches

@ x = 0

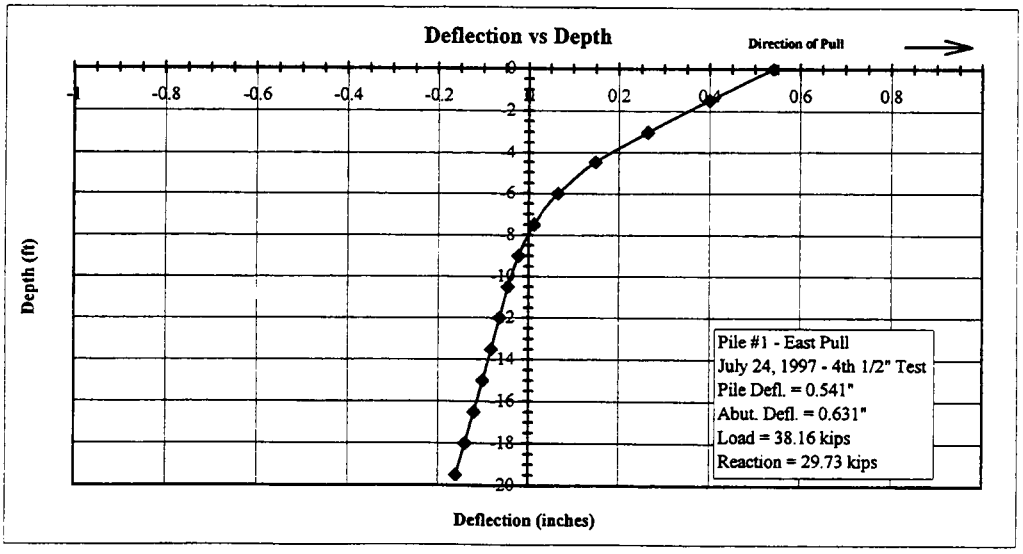
y = 0.000 + 0 C_1 + C_2 = 0.541

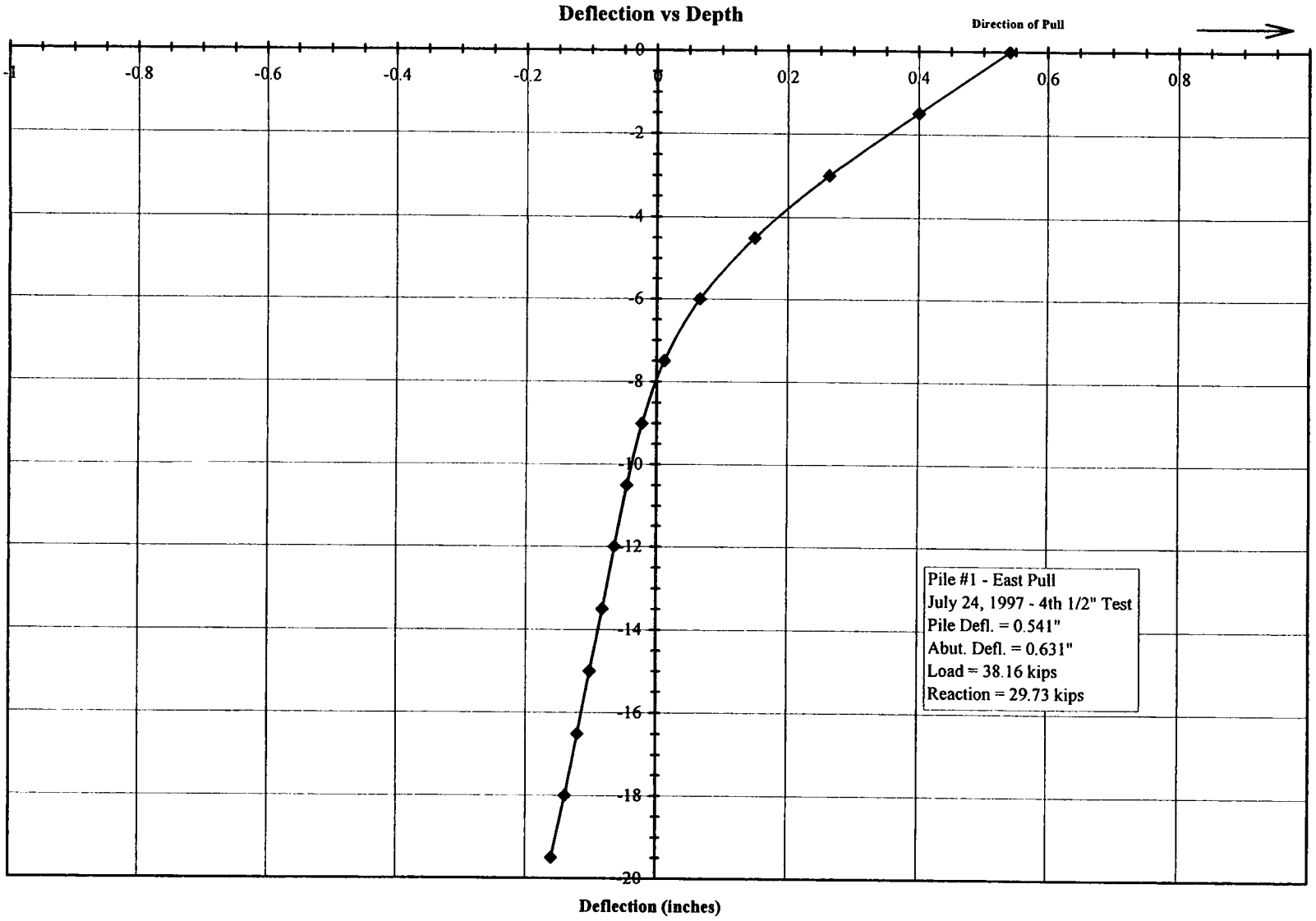
@ x = 7.94346

y = 0.169 + 7.94346 C_1 + 0.541 = 0

C_1 = -0.0893520 = -0.0074460 radians

Deflection (in)	Depth (ft)	Station	Depth (ft)
0.541	0	1	0
0.400	-1.5	2	1.5
0.264	-3	3	3
0.150	-4.5	4	4.5
0.066	-6	5	6
0.012	-7.5	6	7.5
-0.022	-9	7	9
-0.045	-10.5	8	10.5
-0.063	-12	9	12
-0.082	-13.5	10	13.5
-0.101	-15	11	15
-0.120	-16.5	12	16.5
-0.139	-18	13	18
-0.160	-19.5	14	19.5




Deflection vs Depth
Direction of Pull
-1
-0.8
-0.6
-0.4
-0.2
0
0.2
0.4
0.6
0.8
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Deflection (inches)
Pile #1 - East Pull
July 24, 1997 - 4th 1/2" Test
Pile Defl. = 0.541"
Abut. Defl. = 0.631"
Load = 38.16 kips
Reaction = 29.73 kips

APPENDIX D

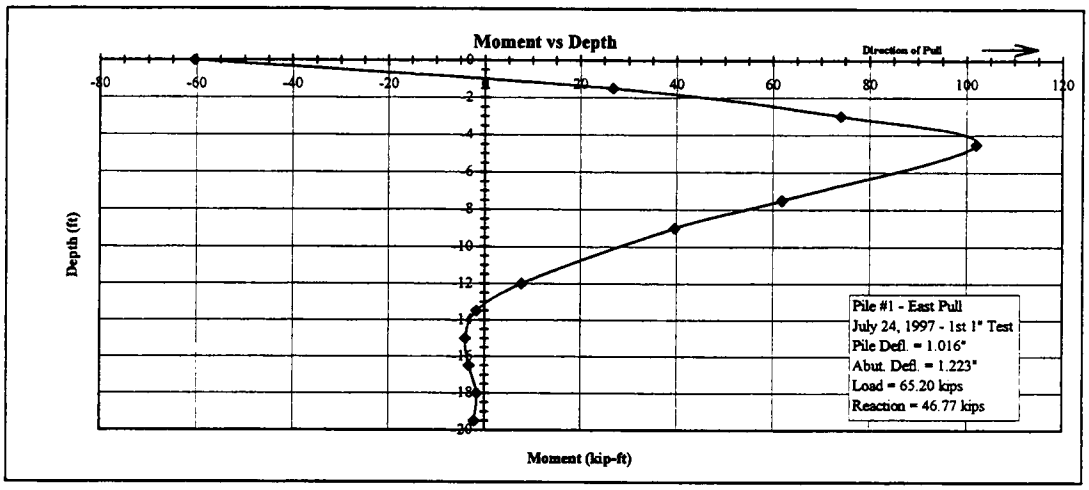
ONE INCH INCREMENT

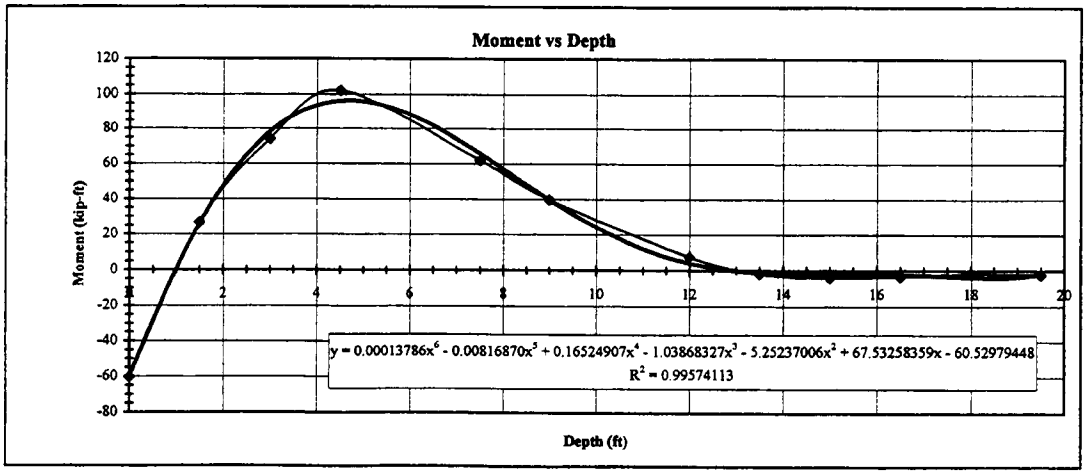
FOR JULY 24, 1997

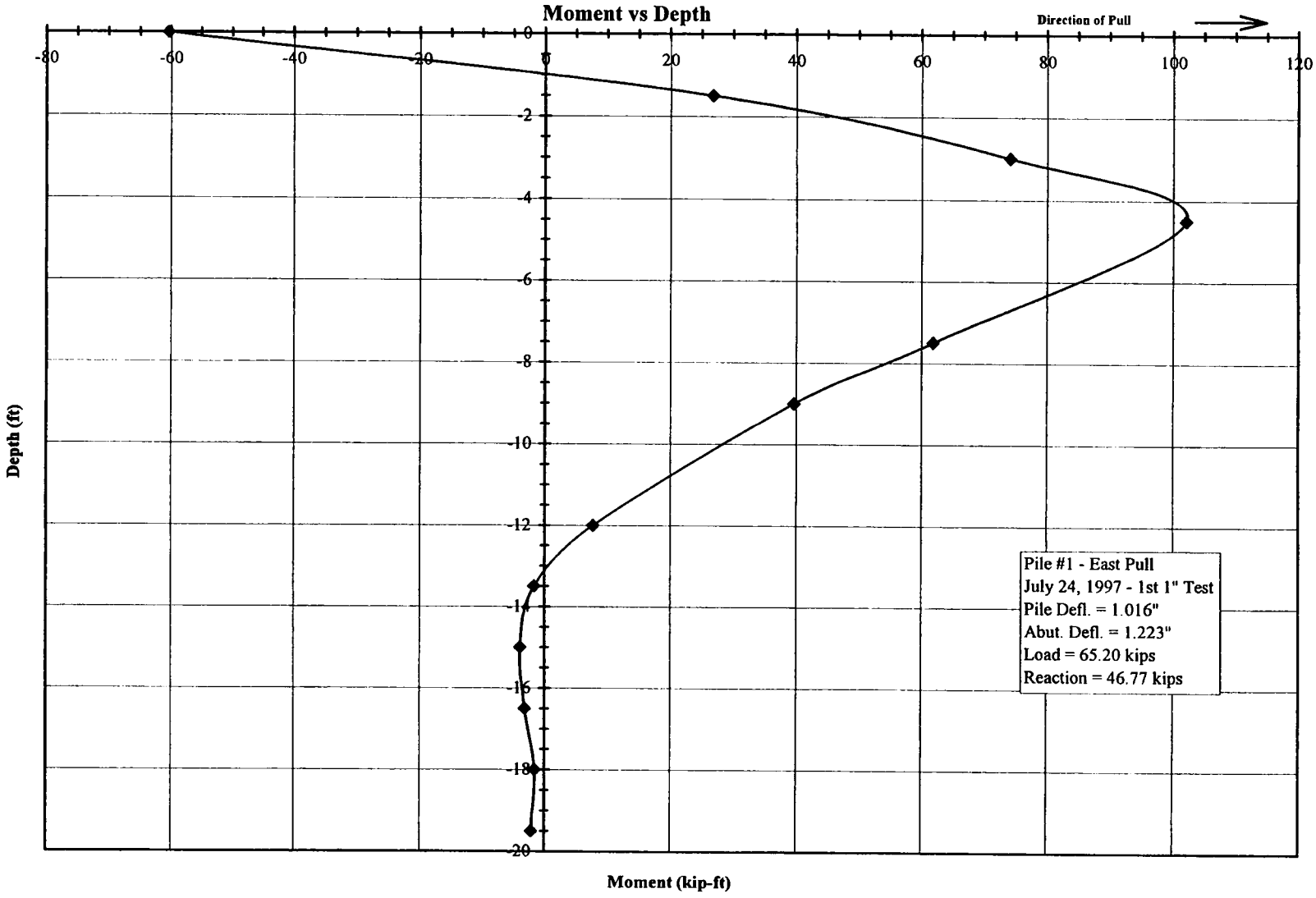
Moments

Date	07/24/97	
Time	19272 - 19282	sec
Load	65.20	kips
Reaction	46.77	kips
Abut. Defl.	1.223	inches
Pile Defl.	1.016	inches

Moment (kip-ft)	Depth (ft)	Station	Depth (ft)
-60.379	0	1	0
26.778	-1.5	2	1.5
74.142	-3	3	3
102.083	-4.5	4	4.5
61.881	-7.5	6	7.5
39.667	-9	7	9
7.749	-12	9	12
-1.655	-13.5	10	13.5
-3.887	-15	11	15
-3.145	-16.5	12	16.5
-1.548	-18	13	18
-2.115	-19.5	14	19.5






Moment vs Depth
Direction of Pull
-80
-60
-40
-20
0
20
40
60
80
100
120
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Moment (kip-ft)
Pile #1 - East Pull
July 24, 1997 - 1st 1" Test
Pile Defl. = 1.016"
Abut. Defl. = 1.223"
Load = 65.20 kips
Reaction = 46.77 kips

Equations

Date	07/24/97		
Time	19272 - 19282	sec	
Load	65.20	kips	
Reaction	46.77	kips	
Abut. Defl.	1.223	inches	
Pile Defl.	1.016	inches	$= C_2$

Modulus of Elasticity (E) =	33000	ksi
Moment of Inertia (I) =	223	in^4

Deflection Equation:
From 0' to 19.5'

$$y = 5.78063E\text{-}10\ x^8 + -4.56697E\text{-}08\ x^7 + 1.29343E\text{-}06\ x^6 + -1.21949E\text{-}05\ x^5$$
$$+ -0.000102778\ x^4 + 0.002642938\ x^3 + -0.007106637\ x^2 + (C_1)\ x$$
$$+ 1.016\ (C_2)$$

Slope Equation:
From 0' to 19.5'

$$y = 3.85375E\text{-}10\ x^7 + -2.66407E\text{-}08\ x^6 + 6.46715E\text{-}07\ x^5 + -5.08121E\text{-}06\ x^4$$
$$+ -3.42592E\text{-}05\ x^3 + 0.000660735\ x^2 + -0.00118444\ x + (C_1)$$

Moment Equation:
From 0' to 19.5'

$$\mathbf{y = 0.00013786\ x^6 + -0.00816870\ x^5 + 0.16524907\ x^4 + -1.03868327\ x^3}$$
$$\mathbf{+ -5.25237006\ x^2 + 67.53258359\ x + -60.52979448}$$

Shear Equation:
From 0' to 19.5'

$$y = 0.00082716\ x^5 + -0.04084350\ x^4 + 0.66099628\ x^3 + -3.11604981\ x^2$$
$$+ -10.50474012\ x + 67.53258359$$

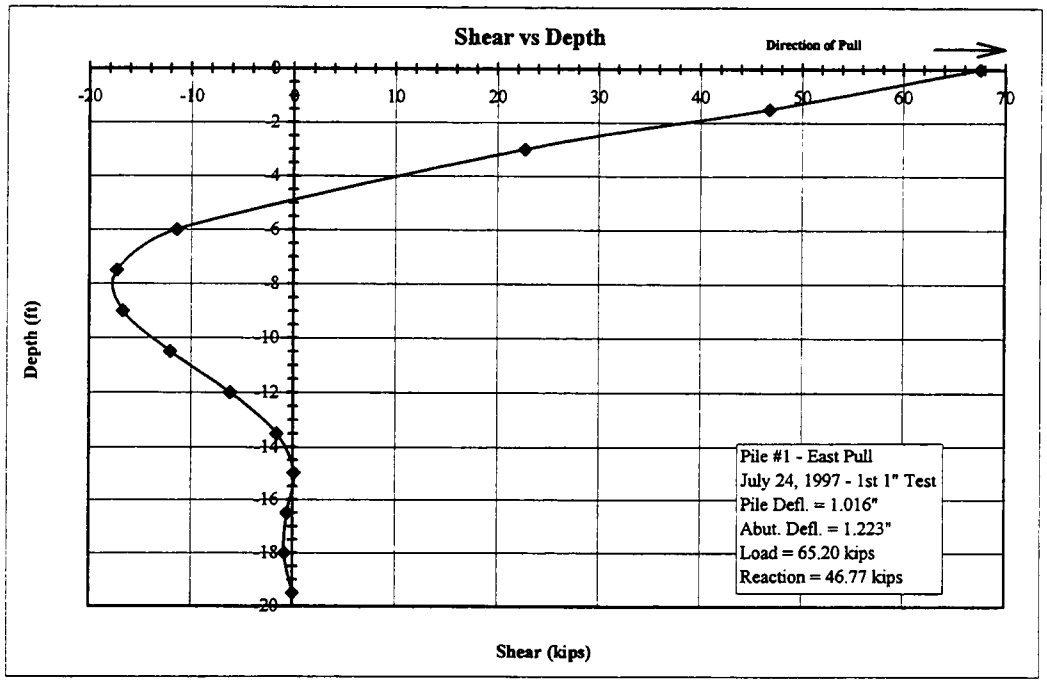
Pressure Equation:
From 0' to 19.5'

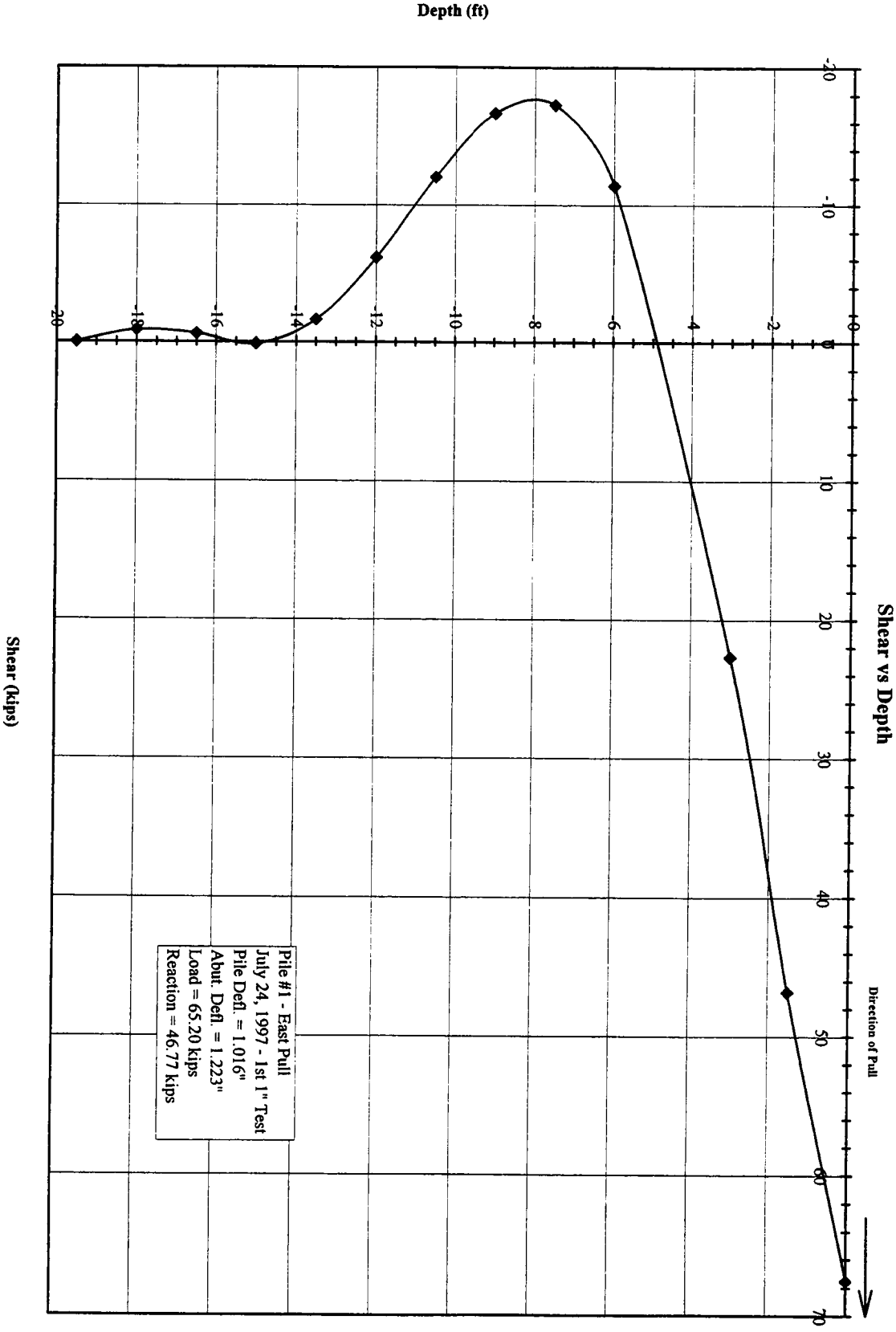
$$y = 0.0041358\ x^4 + -0.163374\ x^3 + 1.98298884\ x^2 + -6.23209962\ x$$
$$+ -10.50474012$$

Shear

Date	07/24/97	
Time	19272 - 19282	sec
Load	65.20	kips
Reaction	46.77	kips
Abut. Def	1.223	inches
Pile Defl.	1.016	inches

Shear (kips)	Depth (ft)	Station	Depth (ft)
67.533	0	1	0
46.795	-1.5	2	1.5
22.713	-3	3	3
-11.400	-6	5	6
-17.275	-7.5	6	7.5
-16.675	-9	7	9
-12.012	-10.5	8	10.5
-6.141	-12	9	12
-1.601	-13.5	10	13.5
0.135	-15	11	15
-0.584	-16.5	12	16.5
-0.835	-18	13	18
0.000	-19.5	14	19.5

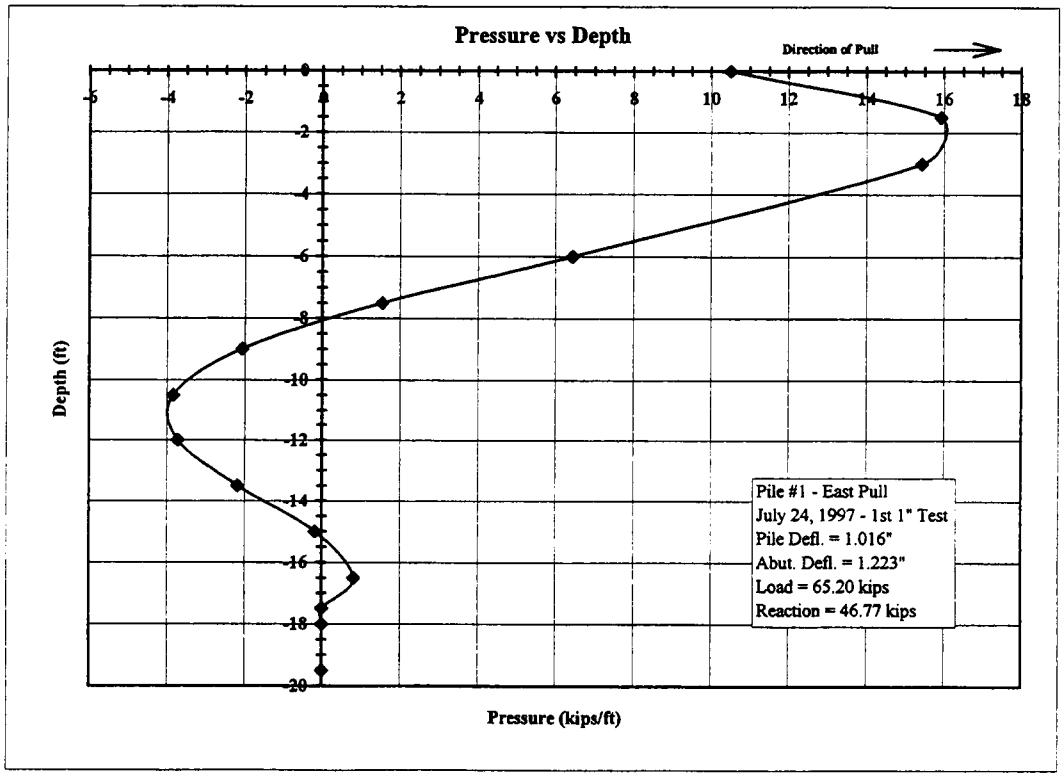


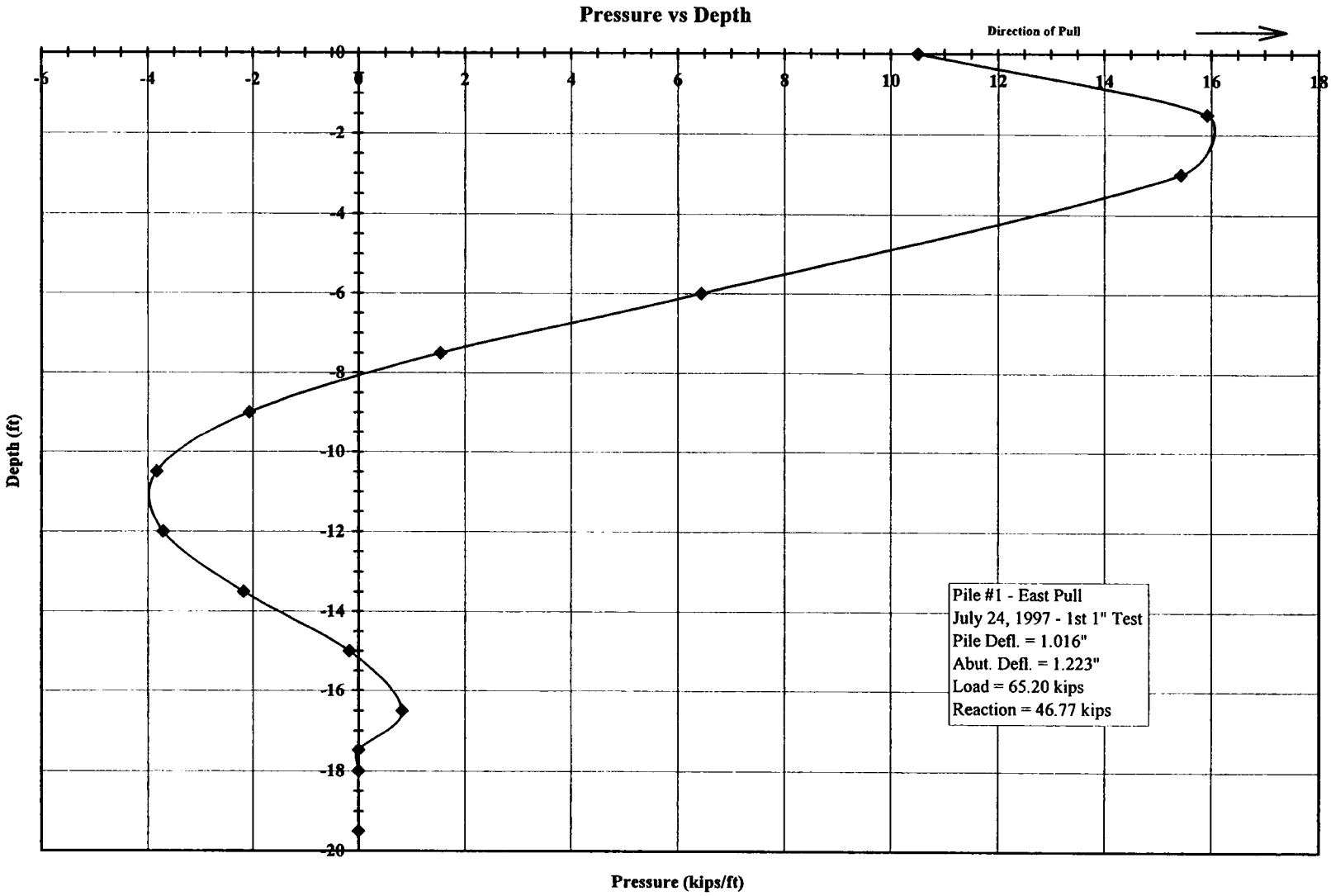

Depth (ft)
-20
-10
0
10
20
30
40
50
60
70
-20
-18
-16
-14
-12
-10
-8
-6
-4
-2
0
Shear vs Depth
Direction of Pull
Shear (kips)
Pile #1 - East Pull
July 24, 1997 - 1st 1" Test
Pile Defl. = 1.016"
Abut. Defl. = 1.223"
Load = 65.20 kips
Reaction = 46.77 kips

Pressure

Date	07/24/97	
Time	19272 - 19282	sec
Load	65.20	kips
Reaction	46.77	kips
Abut. Defl.	1.223	inches
Pile Defl.	1.016	inches

Pressure (kips/ft)	Depth (ft)	Station	Depth (ft)
10.505	0	1	0
15.922	-1.5	2	1.5
15.430	-3	3	3
6.439	-6	5	6
1.540	-7.5	6	7.5
-2.064	-9	7	9
-3.828	-10.5	8	10.5
-3.710	-12	9	12
-2.171	-13.5	10	13.5
-0.174	-15	11	15
0.816	-16.5	12	16.5
0.000	-17.48009		17.48009
0.000	-18	13	18
0.000	-19.5	14	19.5




Pressure vs Depth
Direction of Pull
Depth (ft)
Pressure (kips/ft)
Pile #1 - East Pull
July 24, 1997 - 1st 1" Test
Pile Defl. = 1.016"
Abut. Defl. = 1.223"
Load = 65.20 kips
Reaction = 46.77 kips

Statics

Date	07/24/97	
Time	19272 - 19282	sec
Load	65.20	kips
Reaction	46.77	kips
Abut. Defl.	1.223	inches
Pile Defl.	1.016	inches

Load by Statics

Pressure Equation

$y =$ 0.00413580 $x^4 +$ -0.16337400 $x^3 +$ 1.982988840 $x^2 +$ -6.23209962 $x +$ -10.50474012

Roots for pressure equation

x = -1.18973 x = 8.06113 x = 15.15090 x = 17.48009

Integrate to check statics:

From 0 to 8.06113

= -0.000827160 $x^5 +$ 0.040843500 $x^4 +$ -0.660996280 $x^3 +$ 3.116049810 $x^2 +$ 10.50474012 x

= 85.23041117 - 0 = **85.23041 kips**

From 8.06113 to 15.15090

= -0.000827160 $x^5 +$ 0.040843500 $x^4 +$ -0.660996280 $x^3 +$ 3.116049810 $x^2 +$ 10.50474012 x

= 67.38444114 - 85.23041117 = **-17.84597 kips**

From 15.15090 to 17.48009

= -0.000827160 $x^5 +$ 0.040843500 $x^4 +$ -0.660996280 $x^3 +$ 3.116049810 $x^2 +$ 10.50474012 x

= 68.64536446 - 67.38444114 = **1.26092 kips**

From Pressure	=	**68.645 kips**
Measured	=	**65.20 kips**
% difference	=	***5.28*** **%**

Moment by Statics

M_{GRND} = (Reaction)(x) - (Load)(y) - (Weight)(e)

x =	7	ft	(distance from centerline of the pile to centerline of the reaction)
y =	4.2083	ft	(distance from centerline of the load to the ground surface)
e =	0.207	in	($e = \Delta_{abut} - \Delta_{pile}$)
Weight =	65.0	kips	

M_{GRND} = 327.390 - 274.383 - 1.121 = **51.885 kip-ft**

Measured = 60.379 kip-ft

% difference = *14.07* %

Deflections

Date	07/24/97	
Time	19272 - 19282	sec
Load	65.20	kips
Reaction	46.77	kips
Abut. Defl.	1.223	inches
Pile Defl.	1.016	inches

@ x = 0

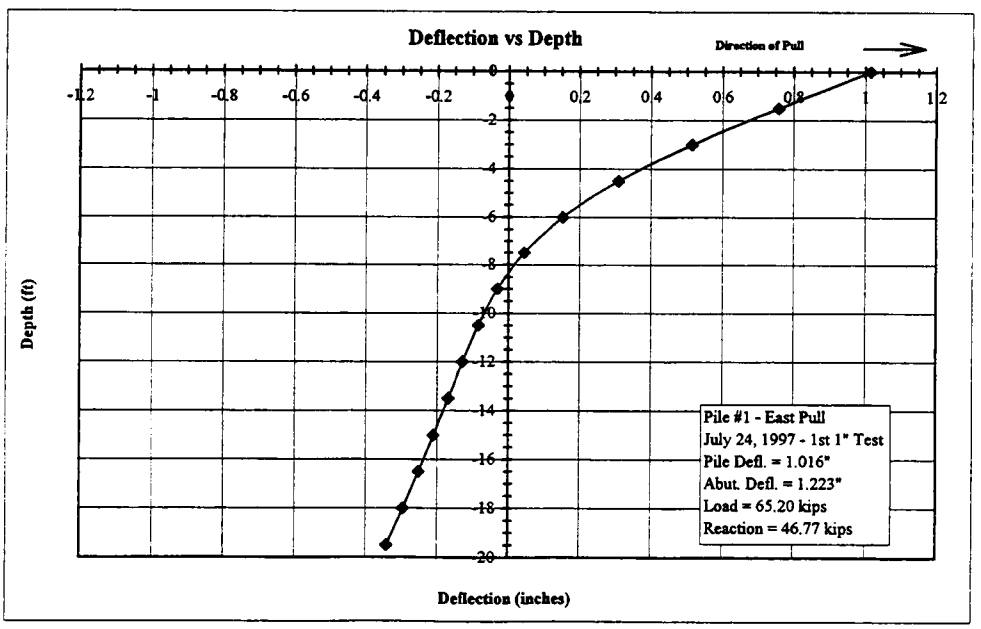
y = 0.000 + 0 C_1 + C_2 = 1.016

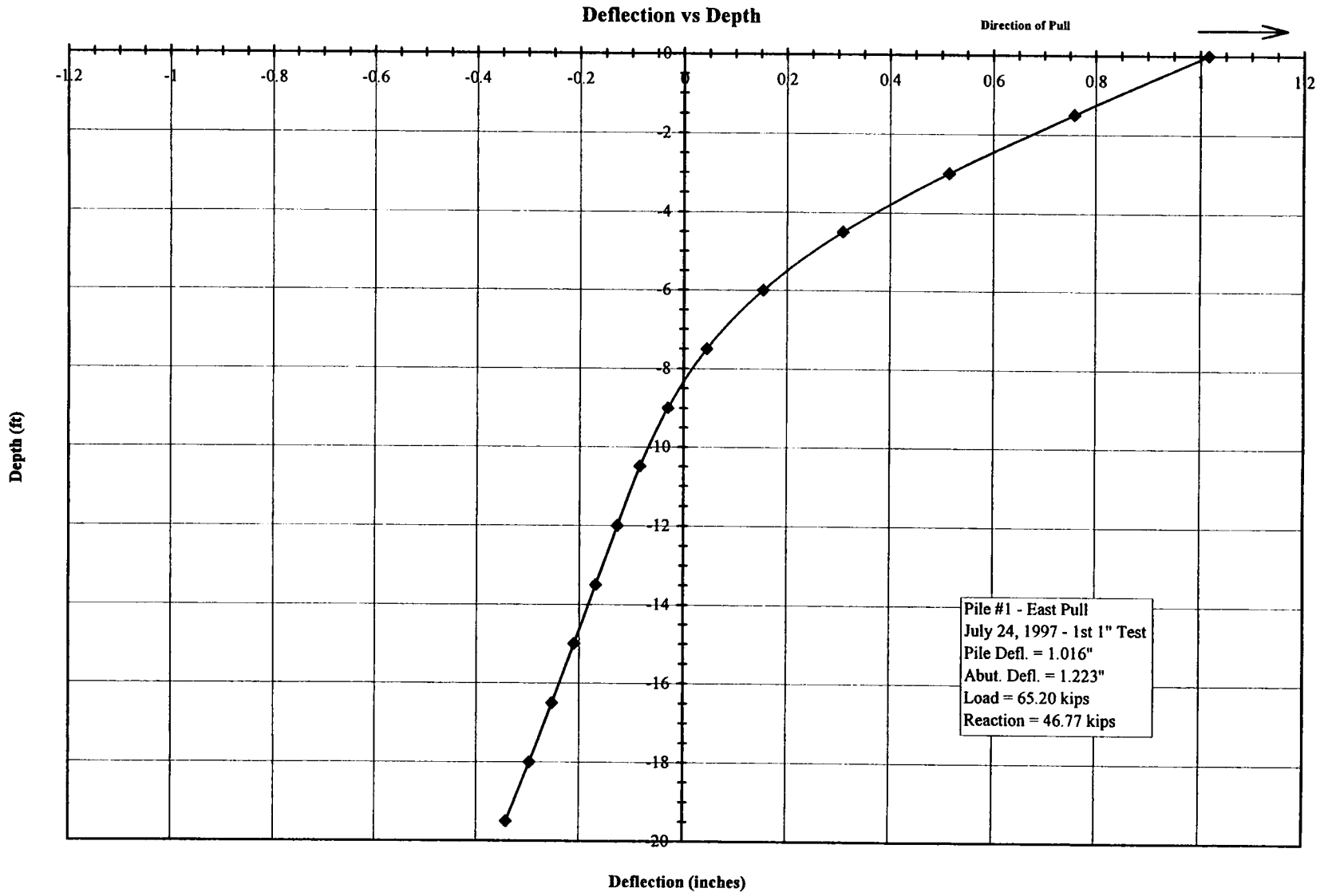
@ x = 7.94346

y = 0.324 + 8.06113 C_1 + 1.016 = 0

C_1 = -0.1662743 = -0.0138562 radians

Deflection (in)	Depth (ft)	Station	Depth (ft)
1.016	0	1	0
0.759	-1.5	2	1.5
0.514	-3	3	3
0.309	-4.5	4	4.5
0.154	-6	5	6
0.045	-7.5	6	7.5
-0.030	-9	7	9
-0.083	-10.5	8	10.5
-0.127	-12	9	12
-0.168	-13.5	10	13.5
-0.209	-15	11	15
-0.252	-16.5	12	16.5
-0.295	-18	13	18
-0.341	-19.5	14	19.5




Deflection vs Depth
Direction of Pull
-1.2
-1
-0.8
-0.6
-0.4
-0.2
0
0.2
0.4
0.6
0.8
1
1.2
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Deflection (inches)
Pile #1 - East Pull
July 24, 1997 - 1st 1" Test
Pile Defl. = 1.016"
Abut. Defl. = 1.223"
Load = 65.20 kips
Reaction = 46.77 kips

APPENDIX E

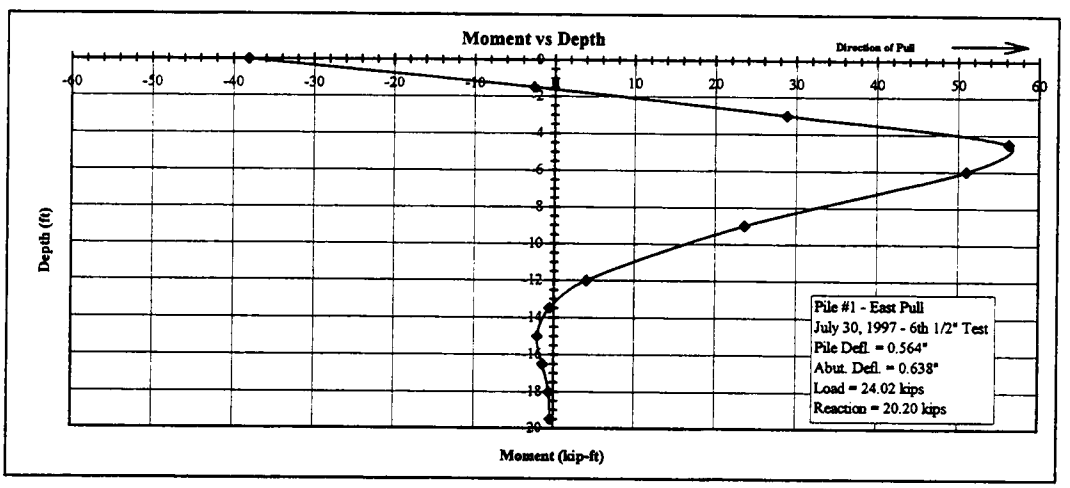
ONE-HALF INCH INCREMENT

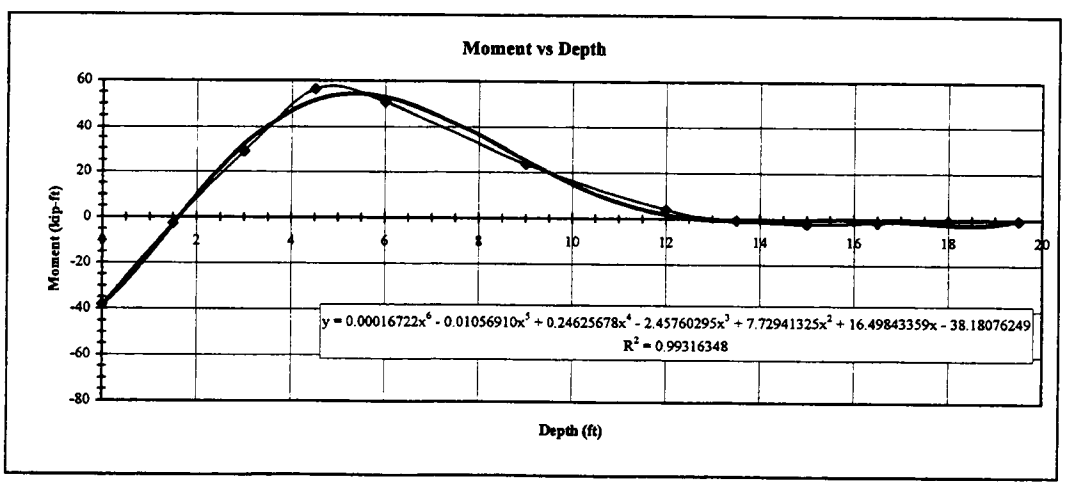
FOR JULY 30, 1997

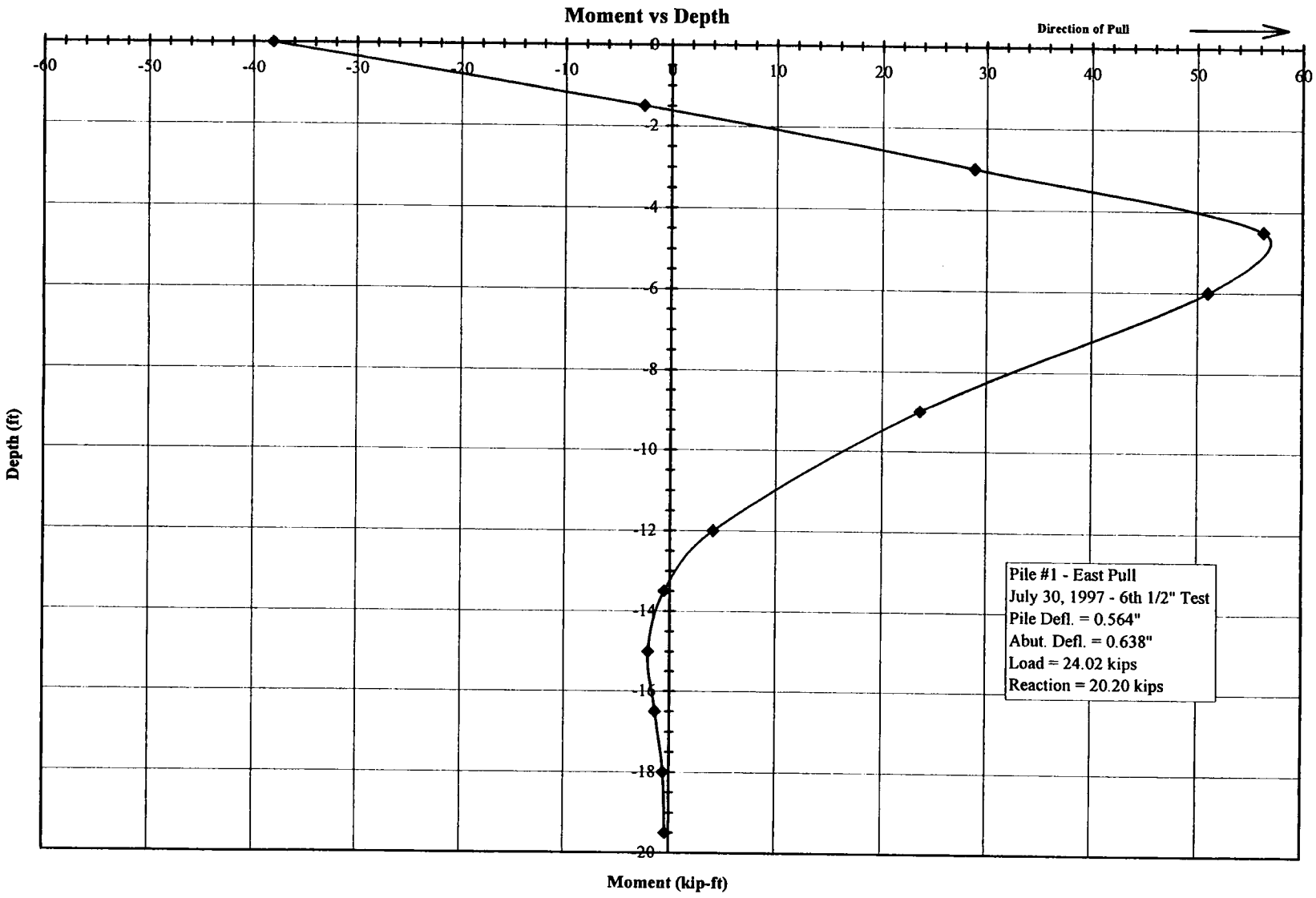
Moments

Date	07/30/97	
Time	6592 - 6602	sec
Load	24.02	kips
Reaction	20.20	kips
Abut. Defl.	0.638	inches
Pile Defl.	0.564	inches

Moment (kip-ft)	Depth (ft)	Station	Depth (ft)
-38.124	0	1	0
-2.594	-1.5	2	1.5
28.858	-3	3	3
56.311	-4.5	4	4.5
51.043	-6	5	6
23.659	-9	7	9
4.080	-12	9	12
-0.525	-13.5	10	13.5
-2.029	-15	11	15
-1.353	-16.5	12	16.5
-0.574	-18	13	18
-0.381	-19.5	14	19.5






Moment vs Depth
Direction of Pull
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Moment (kip-ft)
Pile #1 - East Pull
July 30, 1997 - 6th 1/2" Test
Pile Defl. = 0.564"
Abut. Defl. = 0.638"
Load = 24.02 kips
Reaction = 20.20 kips

Equations

Date	07/30/98		
Time	6592 - 6602	sec	
Load	24.02	kips	
Reaction	20.20	kips	
Abut. Defl.	0.638	inches	
Pile Defl.	0.564	inches	$= C_2$

Modulus of Elasticity (E) =	33000	ksi
Moment of Inertia (I) =	223	in^4

Deflection Equation:
From 0' to 19.5'

$$y = 7.01173E\text{-}10\ x^8 + -5.909E\text{-}08\ x^7 + 1.92749E\text{-}06\ x^6 + -2.8854E\text{-}05\ x^5 + 0.000151248\ x^4 + 0.000645679\ x^3 + -0.004482699\ x^2 + (C_1)\ x + 0.564\ (C_2)$$

Slope Equation:
From 0' to 19.5'

$$y = 4.67449E\text{-}10\ x^7 + -3.44691E\text{-}08\ x^6 + 9.63744E\text{-}07\ x^5 + -1.20225E\text{-}05\ x^4 + 5.04161E\text{-}05\ x^3 + 0.00016142\ x^2 + -0.000747116\ x + (C_1)$$

Moment Equation:
From 0' to 19.5'

$$\mathbf{y = 0.00016722\ x^6 + -0.01056910\ x^5 + 0.24625678\ x^4 + -2.45760295\ x^3 + 7.72941325\ x^2 + 16.49843359\ x + -38.18076249}$$

Shear Equation:
From 0' to 19.5'

$$y = 0.00100332\ x^5 + -0.05284550\ x^4 + 0.98502712\ x^3 + -7.37280885\ x^2 + 15.45882650\ x + 16.49843359$$

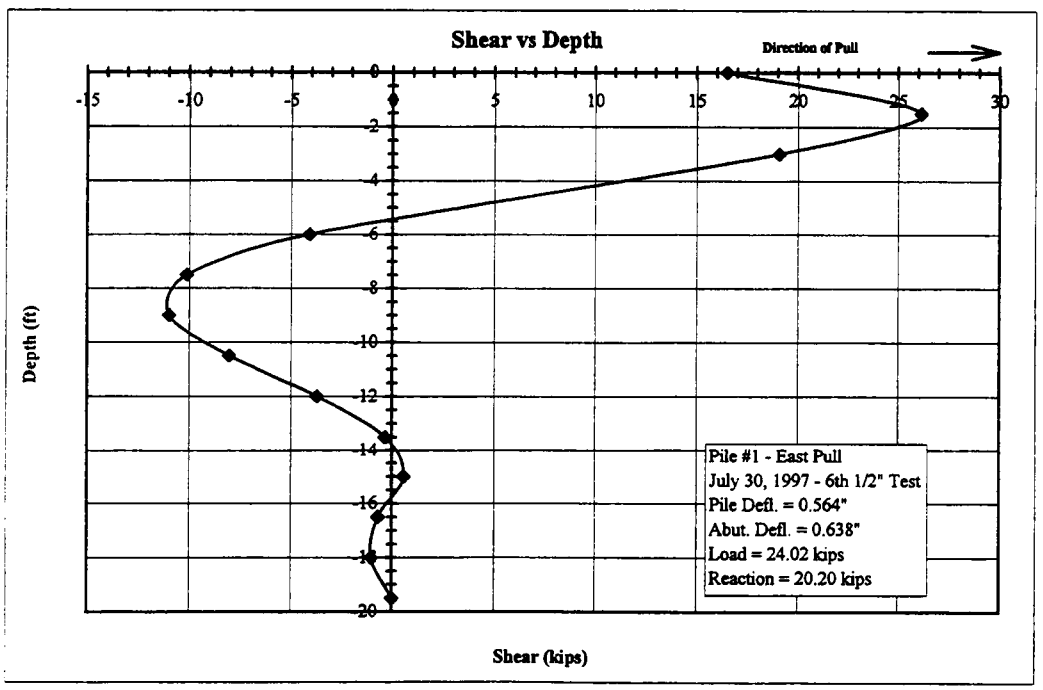
Pressure Equation:
From 0' to 19.5'

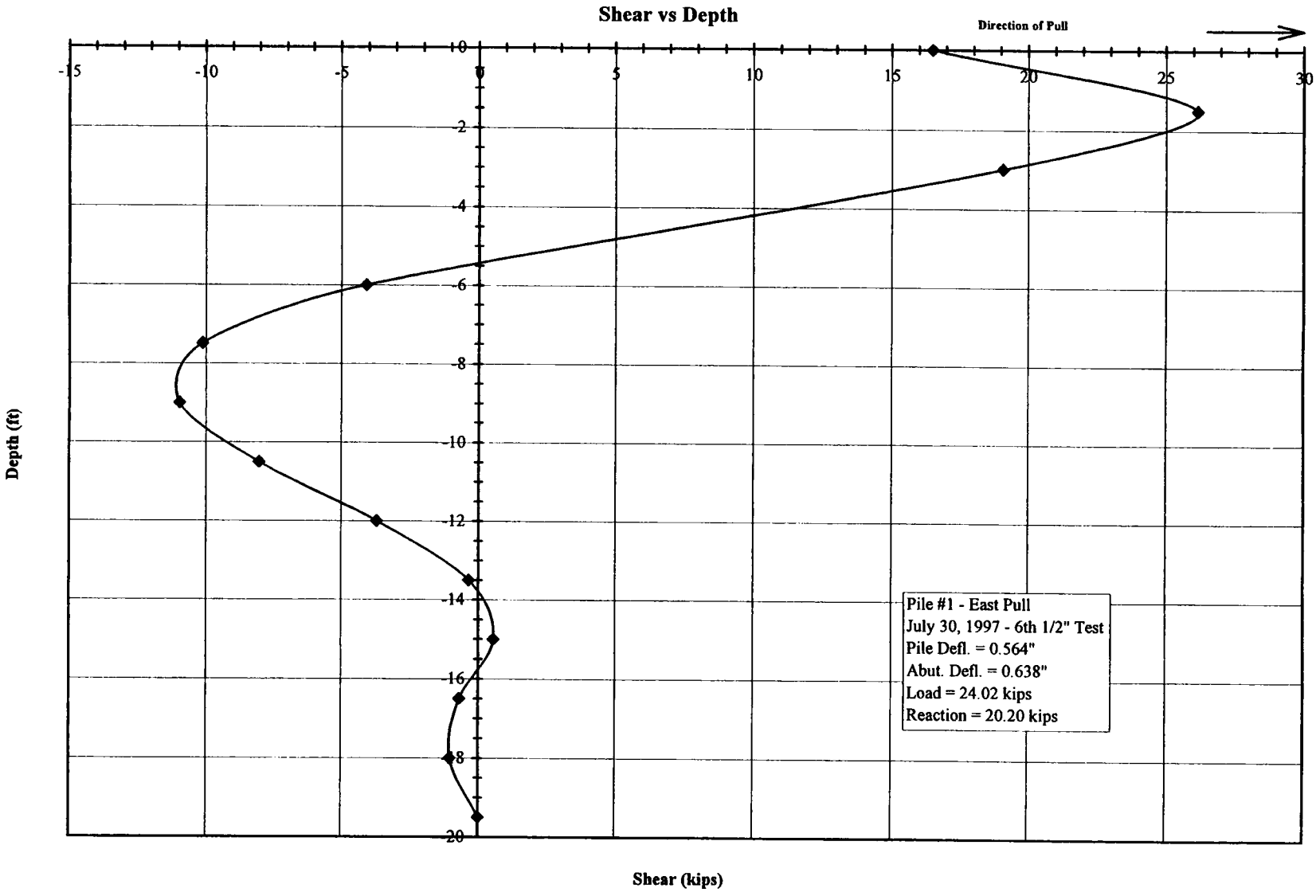
$$y = 0.0050166\ x^4 + -0.211382\ x^3 + 2.95508136\ x^2 + -14.7456177\ x + 15.45882650$$

Shear

Date	07/30/98	
Time	6592 - 6602	sec
Load	24.02	kips
Reaction	20.20	kips
Abut. Defl.	0.638	inches
Pile Defl.	0.564	inches

Shear (kips)	Depth (ft)	Station	Depth (ft)
16.498	0	1	0
26.162	-1.5	2	1.5
19.079	-3	3	3
-4.090	-6	5	6
-10.120	-7.5	6	7.5
-10.959	-9	7	9
-8.033	-10.5	8	10.5
-3.699	-12	9	12
-0.340	-13.5	10	13.5
0.558	-15	11	15
-0.684	-16.5	12	16.5
-1.022	-18	13	18
0.000	-19.5	14	19.5

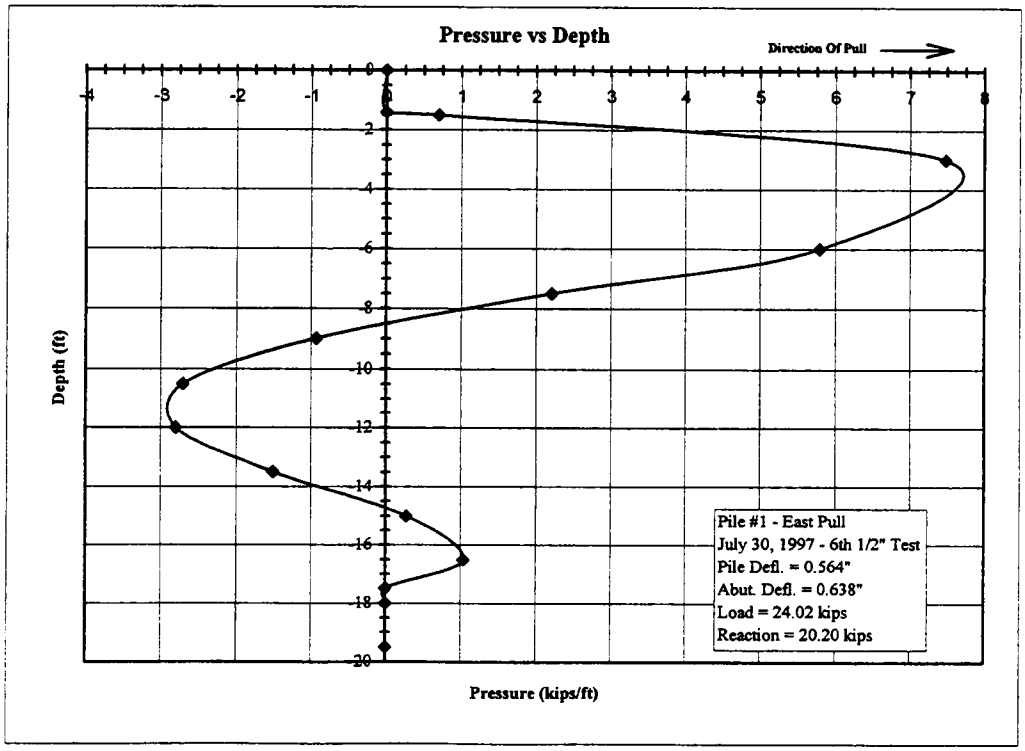


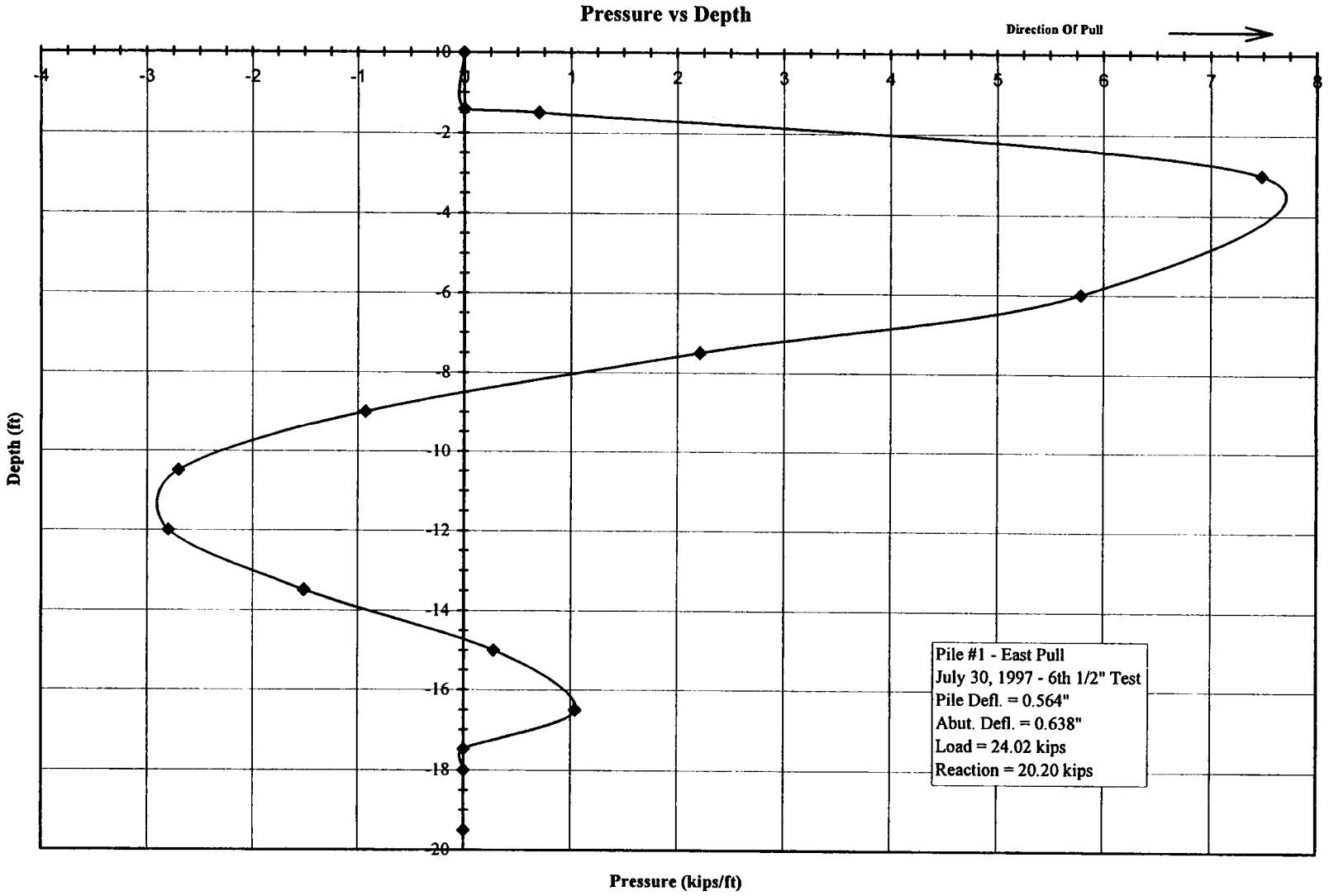

Shear vs Depth
Direction of Pull
-15
-10
-5
0
5
10
15
20
25
30
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Shear (kips)
Pile #1 - East Pull
July 30, 1997 - 6th 1/2" Test
Pile Defl. = 0.564"
Abut. Defl. = 0.638"
Load = 24.02 kips
Reaction = 20.20 kips

Pressure

Date	07/30/98	
Time	6592 - 6602	sec
Load	24.02	kips
Reaction	20.20	kips
Abut. Defl.	0.638	inches
Pile Defl.	0.564	inches

Pressure (kips/ft)	Depth (ft)	Station	Depth (ft)
0.000	0	1	0
0.000	-1.40604		1.40604
0.699	-1.5	2	1.5
7.483	-3	3	3
5.789	-6	5	6
2.214	-7.5	6	7.5
-0.926	-9	7	9
-2.704	-10.5	8	10.5
-2.799	-12	9	12
-1.504	-13.5	10	13.5
0.281	-15	11	15
1.047	-16.5	12	16.5
0.000	-17.4845		17.4845
0.000	-18	13	18
0.000	-19.5	14	19.5




Pressure vs Depth
Direction Of Pull
Depth (ft)
Pressure (kips/ft)
-4
-3
-2
-1
0
1
2
3
4
5
6
7
8
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Pile #1 - East Pull
July 30, 1997 - 6th 1/2" Test
Pile Defl. = 0.564"
Abut. Defl. = 0.638"
Load = 24.02 kips
Reaction = 20.20 kips

Statics

Date	07/30/98	
Time	6592 - 6602	sec
Load	24.02	kips
Reaction	20.20	kips
Abut. Defl.	0.638	inches
Pile Defl.	0.564	inches

Pressure Equation

$y = 0.0050166\,x^4 + -0.2113820\,x^3 + 2.95508136\,x^2 + -14.7456177\,x + 15.45882650$

Roots for pressure equation

x = 1.40604 x = 8.50115 x = 14.7448 x = 17.4845

Integrate to check statics:

From 0 to 1.40604 (Assumed to be zero due to previous pulls and no confinement at ground surface)

From 1.40604 to 8.50115

$= 0.001003320\,x^5 + -0.052845500\,x^4 + 0.985027120\,x^3 + -7.372808850\,x^2 + 15.45882650\,x$

$= -27.69460685 - 9.69709025 =$ **-37.39170 kips**

From 8.50115 to 14.7448

$= 0.001003320\,x^5 + -0.052845500\,x^4 + 0.985027120\,x^3 + -7.372808850\,x^2 + 15.45882650\,x$

$= -15.90401631 - -27.69460685 =$ **11.79059 kips**

From 14.7448 to 17.4845

$= 0.001003320\,x^5 + -0.052845500\,x^4 + 0.985027120\,x^3 + -7.372808850\,x^2 + 15.45882650\,x$

$= -17.83592856 - -15.90401631 =$ **-1.93191 kips**

From Pressure	**-27.533**	**kips**
Measured	**24.02**	**kips**
% difference	***14.63***	**%**

Moment by Statics

M_{GRND} = (Reaction)(x) - (Load)(y) - (Weight)(e)

x =	6.75	ft	(distance from centerline of the pile to centerline of the reaction)
y =	4.2083	ft	(distance from centerline of the load to the ground surface)
e =	0.074	in	($e = \Delta_{abut} - \Delta_{pile}$)
Weight =	65.0	kips	

$M_{GRND} = 136.350 - 101.084 - 0.401 =$ **34.865 kip-ft**

Measured	=	**38.124**	**kip-ft**
% difference	=	***8.55***	**%**

Deflections

Date	07/30/97	
Time	6592 - 6602	sec
Load	24.02	kips
Reaction	20.20	kips
Abut. Defl.	0.638	inches
Pile Defl.	0.564	inches

@ x = 0

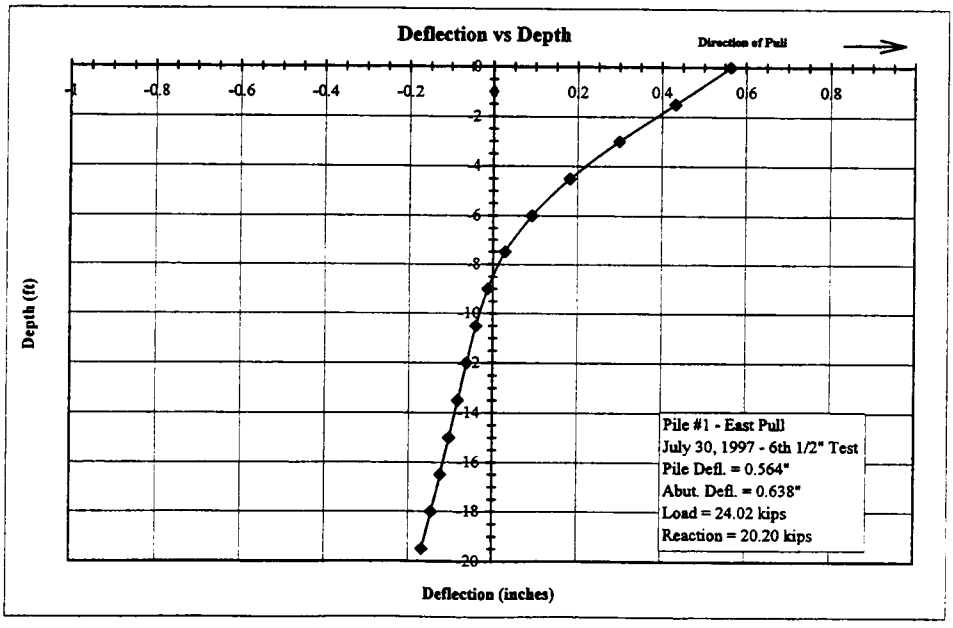
y = 0.000 + 0 C_1 + C_2 = 0.564

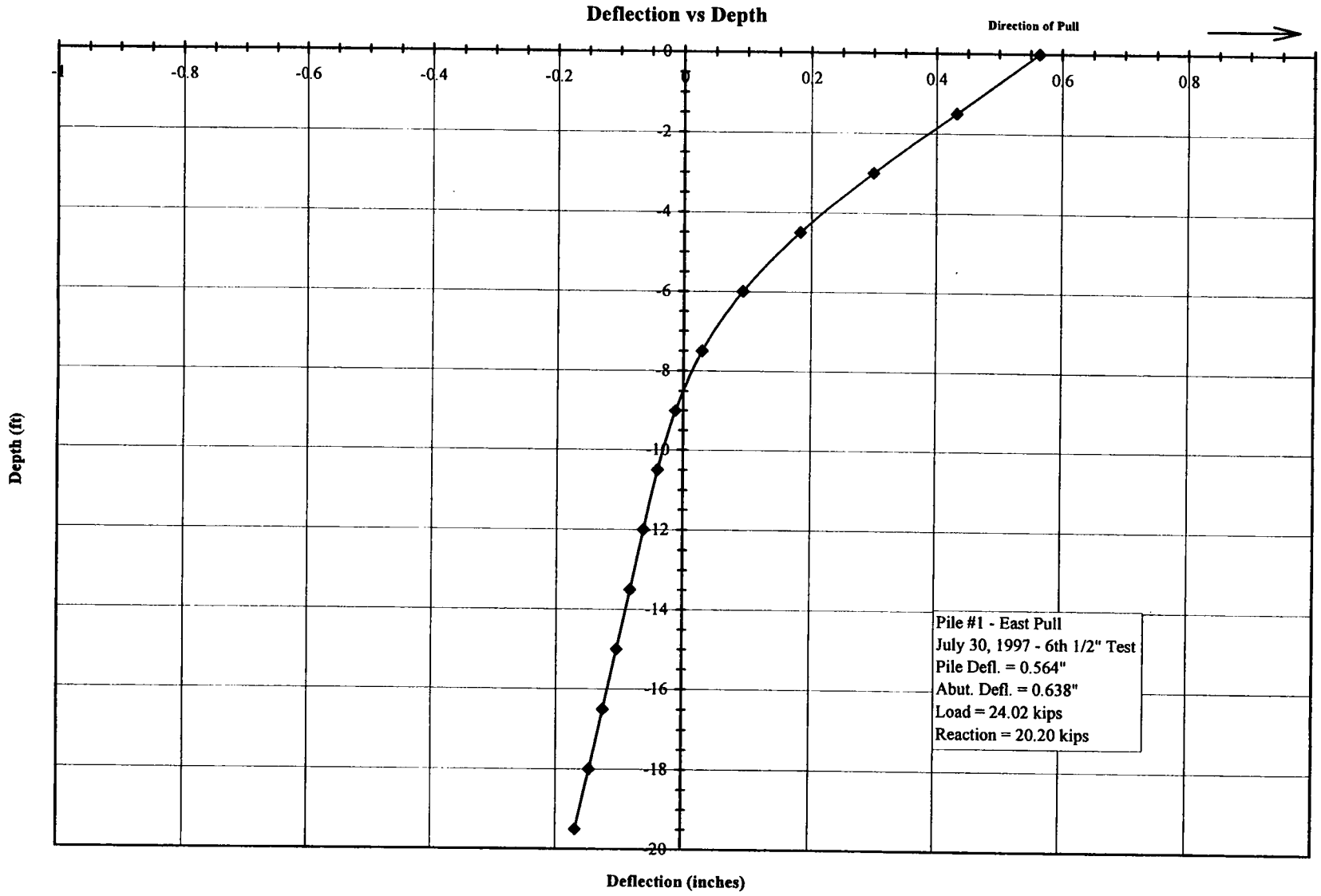
@ x = 8.50115

y = 0.139 + 8.50115 C_1 + 0.564 = 0

C_1 = -0.0826482 = -0.0068873 radians

Deflection (in)	Depth (ft)	Station	Depth (ft)
0.564	0	1	0
0.433	-1.5	2	1.5
0.300	-3	3	3
0.183	-4.5	4	4.5
0.092	-6	5	6
0.029	-7.5	6	7.5
-0.012	-9	7	9
-0.040	-10.5	8	10.5
-0.061	-12	9	12
-0.082	-13.5	10	13.5
-0.103	-15	11	15
-0.124	-16.5	12	16.5
-0.146	-18	13	18
-0.168	-19.5	14	19.5




Deflection vs Depth
Direction of Pull
-1
-0.8
-0.6
-0.4
-0.2
0
0.2
0.4
0.6
0.8
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Deflection (inches)
Pile #1 - East Pull
July 30, 1997 - 6th 1/2" Test
Pile Defl. = 0.564"
Abut. Defl. = 0.638"
Load = 24.02 kips
Reaction = 20.20 kips

APPENDIX F

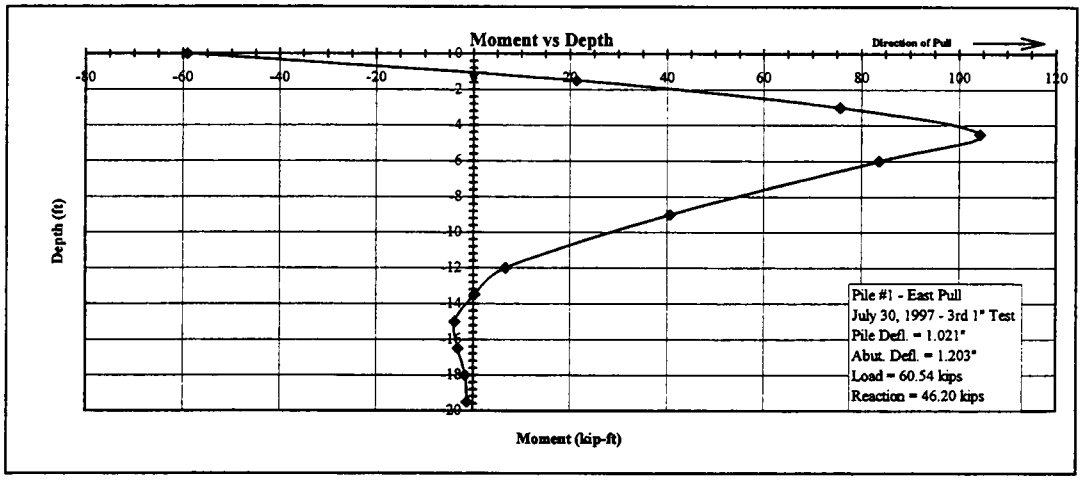
ONE INCH INCREMENT

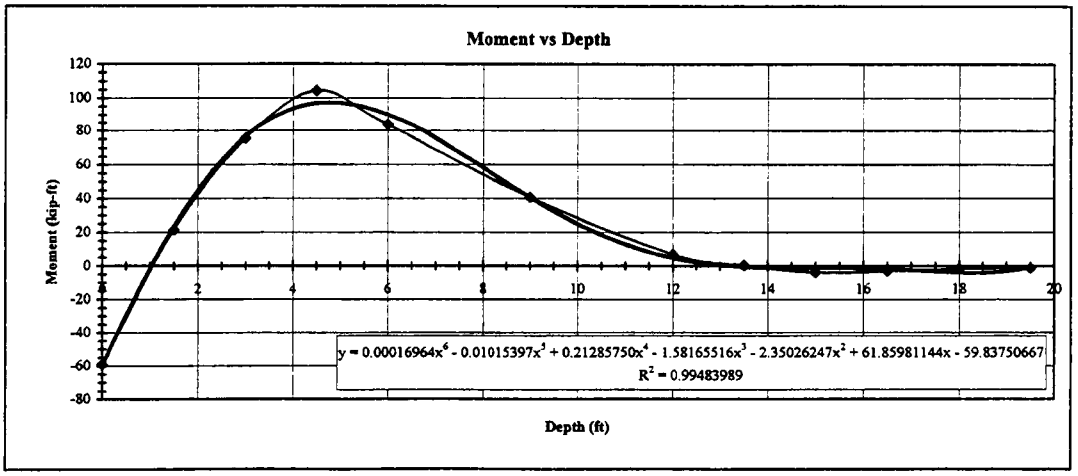
FOR JULY 30, 1997

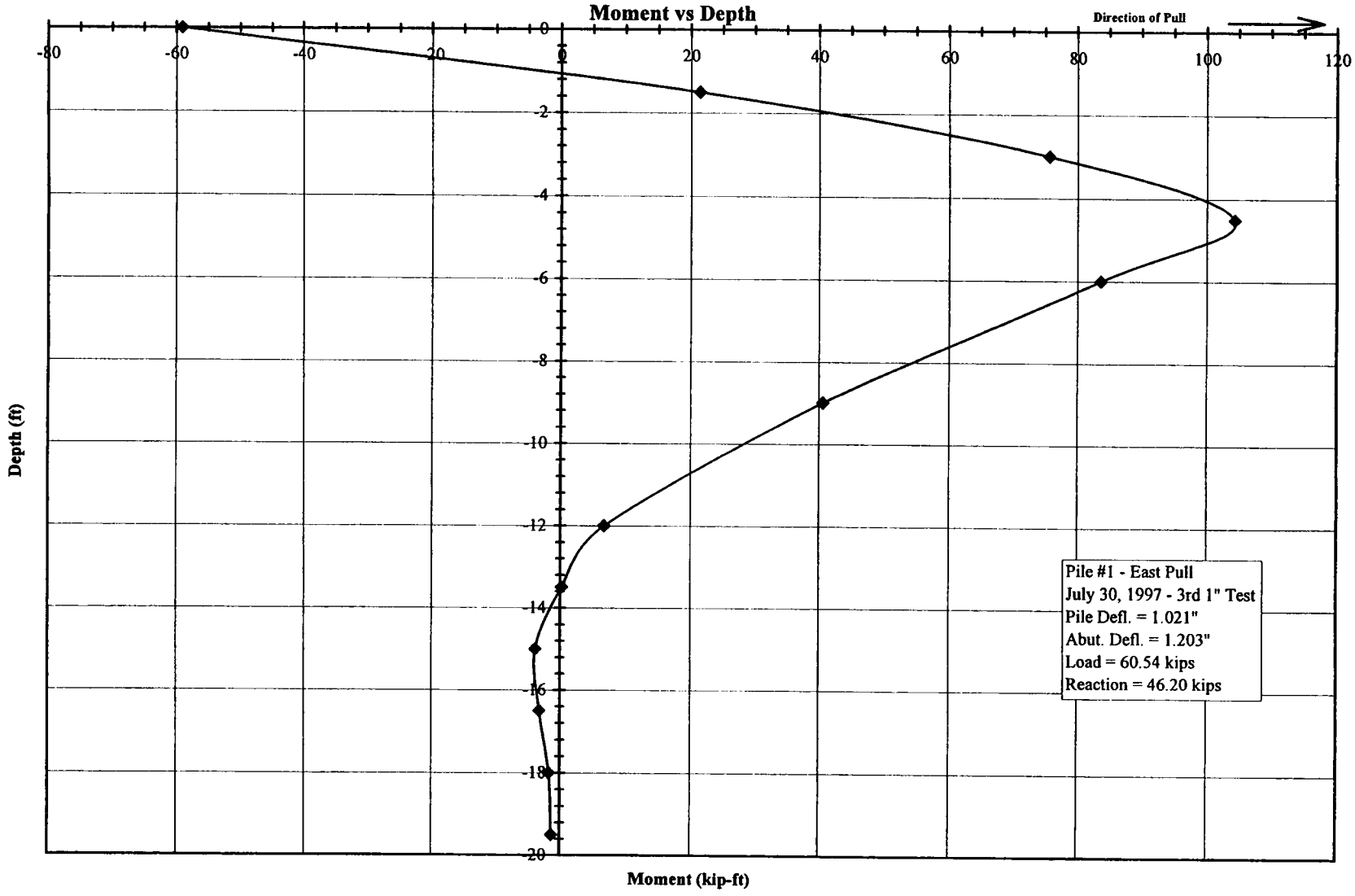
Moments

Date	07/30/97	
Time	14772 - 14782	sec
Load	60.54	kips
Reaction	46.20	kips
Abut. Defl.	1.203	inches
Pile Defl.	1.021	inches

Moment (kip-ft)	Depth (ft)	Station	Depth (ft)
-59.102	0	1	0
21.375	-1.5	2	1.5
75.649	-3	3	3
104.344	-4.5	4	4.5
83.644	-6	5	6
40.624	-9	7	9
6.777	-12	9	12
0.247	-13.5	10	13.5
-3.801	-15	11	15
-3.169	-16.5	12	16.5
-1.665	-18	13	18
-1.260	-19.5	14	19.5






Moment vs Depth
Direction of Pull
-80
-60
-40
-20
0
20
40
60
80
100
120
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Moment (kip-ft)
Pile #1 - East Pull
July 30, 1997 - 3rd 1" Test
Pile Defl. = 1.021"
Abut. Defl. = 1.203"
Load = 60.54 kips
Reaction = 46.20 kips

Equations

Date	07/30/98		
Time	14772 - 14782	sec	
Load	60.54	kips	
Reaction	46.20	kips	
Abut. Defl.	1.203	inches	
Pile Defl.	1.021	inches	$= C_2$

Modulus of Elasticity (E) =	33000	ksi
Moment of Inertia (I) =	223	in^4

Deflection Equation:
From 0' to 19.5'

$$y = 7.1132E\text{-}10\ x^8 + -5.6769E\text{-}08\ x^7 + 1.66607E\text{-}06\ x^6 + -1.85698E\text{-}05\ x^5 + -4.59896E\text{-}05\ x^4 + 0.00242093\ x^3 + -0.007025357\ x^2 + (C_1)\ x + 1.021\ (C_2)$$

Slope Equation:
From 0' to 19.5'

$$y = 4.74213E\text{-}10\ x^7 + -3.31153E\text{-}08\ x^6 + 8.33034E\text{-}07\ x^5 + -7.73741E\text{-}06\ x^4 + -1.53299E\text{-}05\ x^3 + 0.000605233\ x^2 + -0.001170893\ x + (C_1)$$

Moment Equation:
From 0' to 19.5'

$$\mathbf{y = 0.00016964\ x^6 + -0.01015397\ x^5 + 0.21285750\ x^4 + -1.58165516\ x^3 + -2.35026247\ x^2 + 61.85981144\ x + -59.83750667}$$

Shear Equation:
From 0' to 19.5'

$$y = 0.00101784\ x^5 + -0.05076985\ x^4 + 0.85143000\ x^3 + -4.74496548\ x^2 + -4.70052494\ x + 61.85981144$$

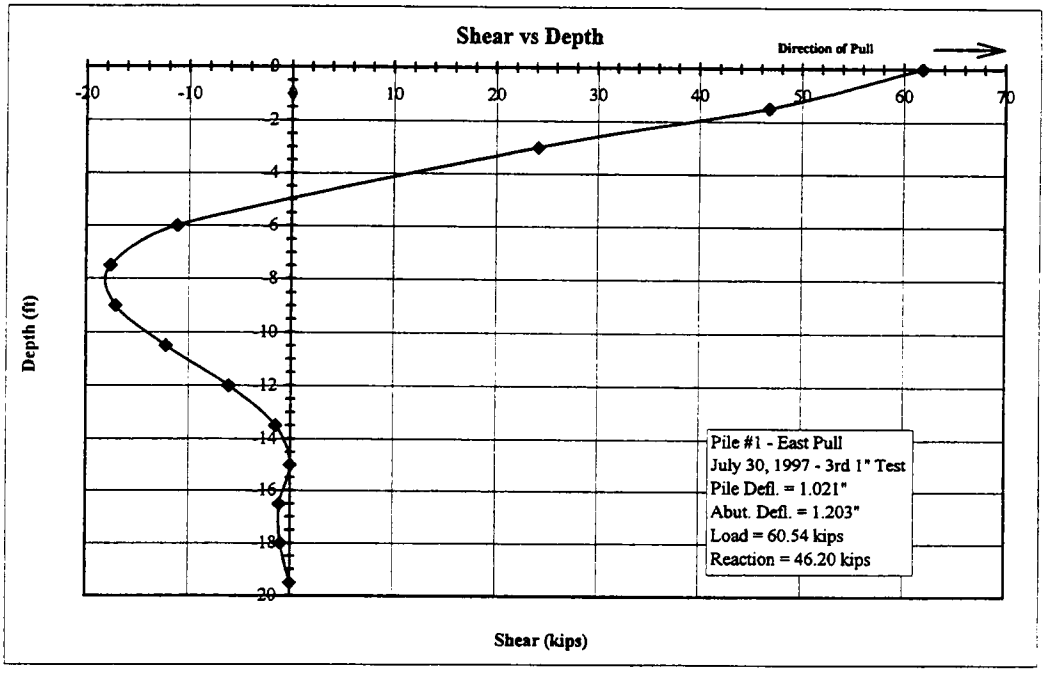
Pressure Equation:
From 0' to 19.5'

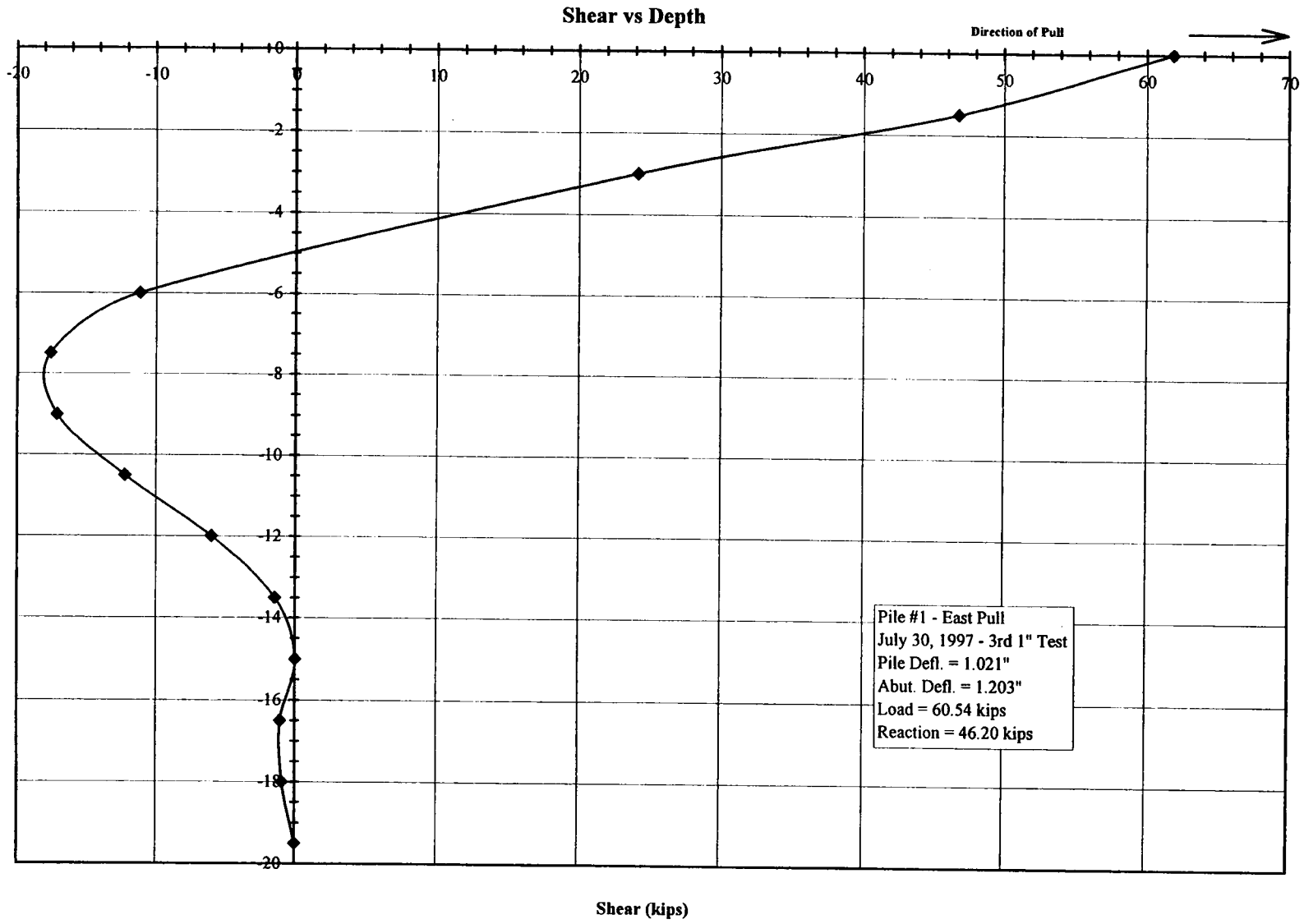
$$y = 0.0050892\ x^4 + -0.2030794\ x^3 + 2.55429\ x^2 + -9.48993096\ x + -4.70052494$$

Shear

Date	07/30/98	
Time	14772 - 14782	sec
Load	60.54	kips
Reaction	46.20	kips
Abut. Defl.	1.203	inches
Pile Defl.	1.021	inches

Shear (kips)	Depth (ft)	Station	Depth (ft)
61.860	0	1	0
46.757	-1.5	2	1.5
24.177	-3	3	3
-11.136	-6	5	6
-17.587	-7.5	6	7.5
-17.093	-9	7	9
-12.197	-10.5	8	10.5
-6.043	-12	9	12
-1.451	-13.5	10	13.5
0.010	-15	11	15
-1.051	-16.5	12	16.5
-0.917	-18	13	18
0.000	-19.5	14	19.5

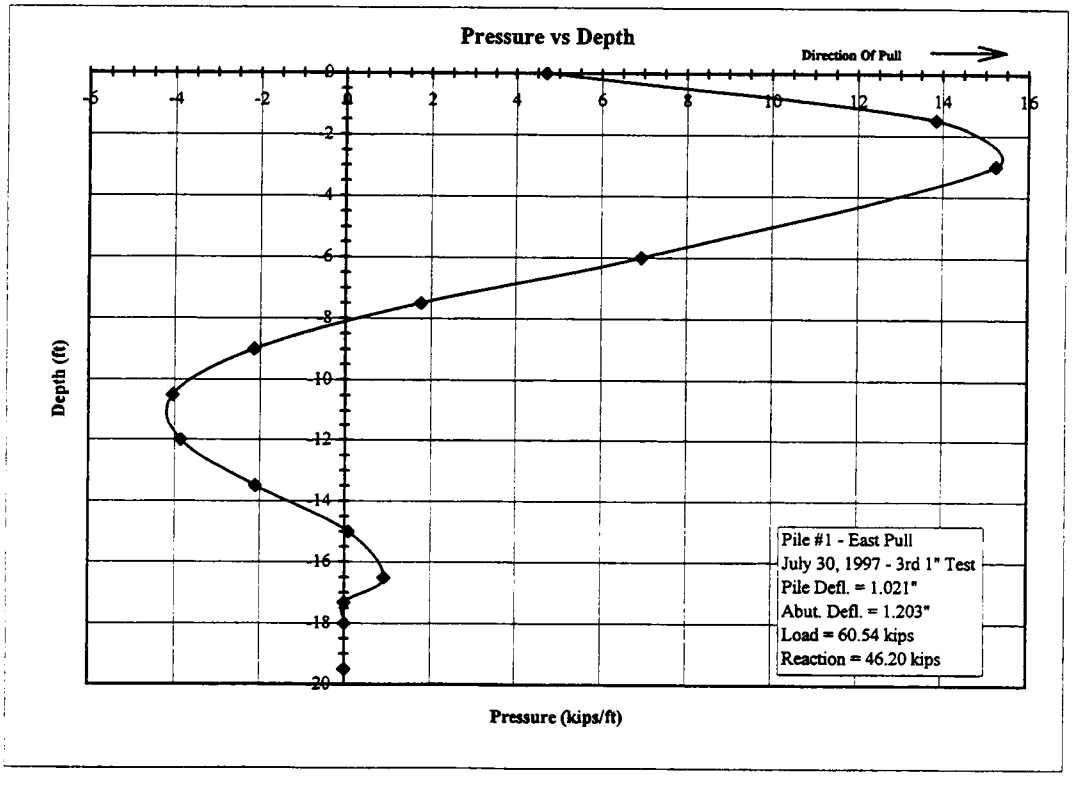


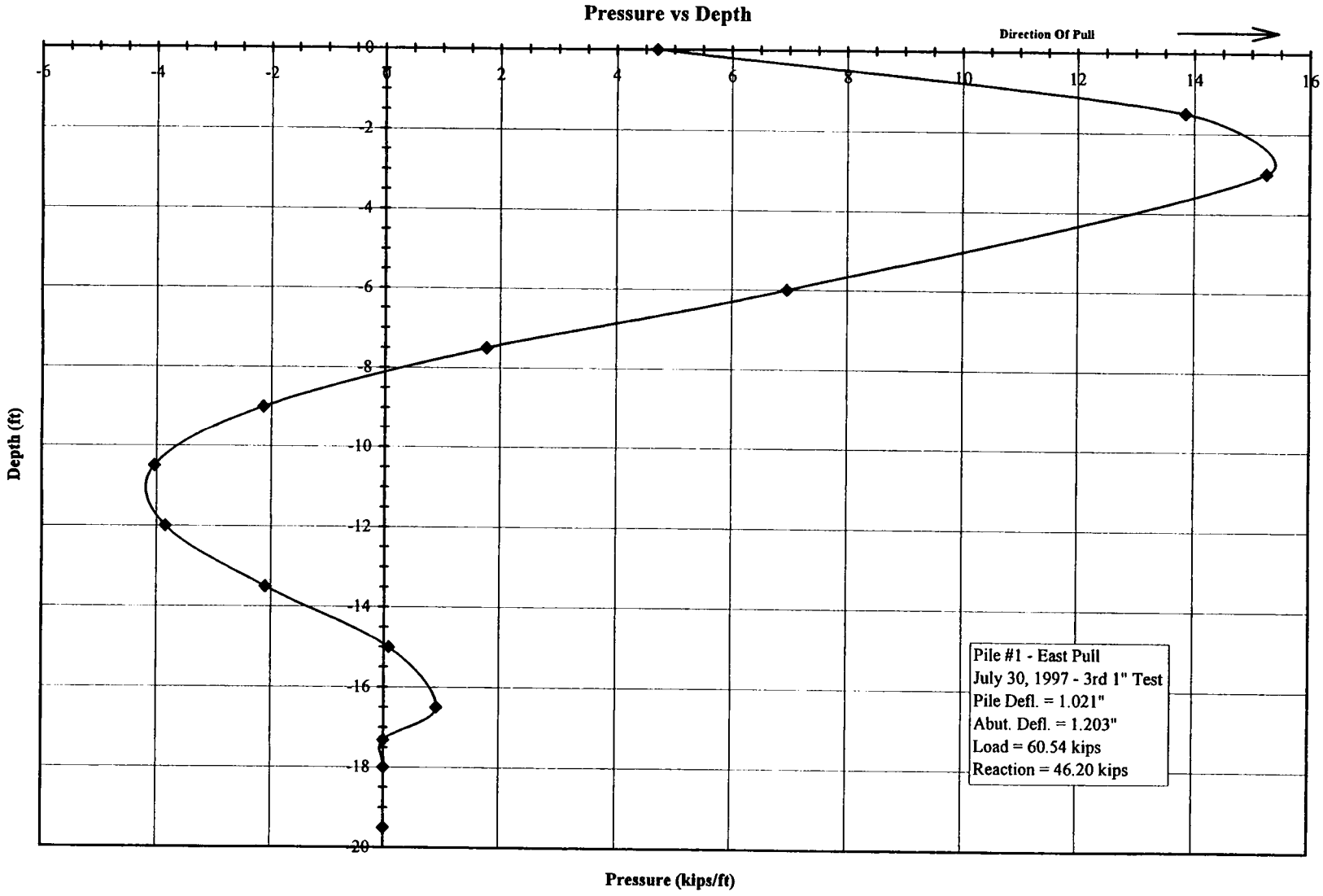

Shear vs Depth
Direction of Pull
-20
-10
0
10
20
30
40
50
60
70
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Shear (kips)
Pile #1 - East Pull
July 30, 1997 - 3rd 1" Test
Pile Defl. = 1.021"
Abut. Defl. = 1.203"
Load = 60.54 kips
Reaction = 46.20 kips

Pressure

Date	07/30/98	
Time	14772 - 14782	sec
Load	60.54	kips
Reaction	46.20	kips
Abut. Defl.	1.203	inches
Pile Defl.	1.021	inches

Pressure (kips/ft)	Depth (ft)	Station	Depth (ft)
4.701	0	1	0
13.848	-1.5	2	1.5
15.253	-3	3	3
6.955	-6	5	6
1.768	-7.5	6	7.5
-2.133	-9	7	9
-4.035	-10.5	8	10.5
-3.847	-12	9	12
-2.091	-13.5	10	13.5
0.086	-15	11	15
0.925	-16.5	12	16.5
0.000	-17.31421		17.31421
0.000	-18	13	18
0.000	-19.5	14	19.5




Pressure vs Depth
Direction Of Pull
Depth (ft)
Pressure (kips/ft)
Pile #1 - East Pull
July 30, 1997 - 3rd 1" Test
Pile Defl. = 1.021"
Abut. Defl. = 1.203"
Load = 60.54 kips
Reaction = 46.20 kips

Statics

Date	07/30/98	
Time	14772 - 14782	sec
Load	60.54	kips
Reaction	46.20	kips
Abut. Defl.	1.203	inches
Pile Defl.	1.021	inches

Pressure Equation

$y = 0.0050892\,x^4 + -0.2030794\,x^3 + 2.55429\,x^2 + -9.48993096\,x + -4.70052494$

Roots for pressure equation

x = -0.44109 x = 8.09977 x = 14.9311 x = 17.3142

Integrate to check statics:

From 0 to 8.09977

$= 0.001017840\,x^5 + -0.050769850\,x^4 + 0.851430000\,x^3 + -4.744965480\,x^2 + -4.70052494\,x$

= -79.96429707 - 0 = **-79.96430 kips**

From 8.09977 to 14.9311

$= 0.001017840\,x^5 + -0.050769850\,x^4 + 0.851430000\,x^3 + -4.744965480\,x^2 + -4.70052494\,x$

= -61.84726947 - -79.96429707 = **18.11703 kips**

From 14.9311 to 17.3142

$= 0.001017840\,x^5 + -0.050769850\,x^4 + 0.851430000\,x^3 + -4.744965480\,x^2 + -4.70052494\,x$

= -63.37607173 - -61.84726947 = **-1.52880 kips**

From Pressure	=	**-63.376 kips**
Measured	=	**60.54 kips**
% difference	=	***4.68*** **%**

Moment by Statics

M_{GRND} = (Reaction)(x) - (Load)(y) - (Weight)(e)

x =	6.75	ft	(distance from centerline of the pile to centerline of the reaction)
y =	4.2083	ft	(distance from centerline of the load to the ground surface)
e =	0.182	in	($e = D_{abut} - D_{pile}$)
Weight =	65.0	kips	

M_{GRND} = 311.850 - 254.773 - 0.986 = **56.092 kip-ft**

Measured	=	**59.102**	**kip-ft**
% difference	=	***5.09***	**%**

Deflections

Date	07/30/97	
Time	14772 - 14782	sec
Load	60.54	kips
Reaction	46.20	kips
Abut. Defl.	1.203	inches
Pile Defl.	1.021	inches

@ x = 0

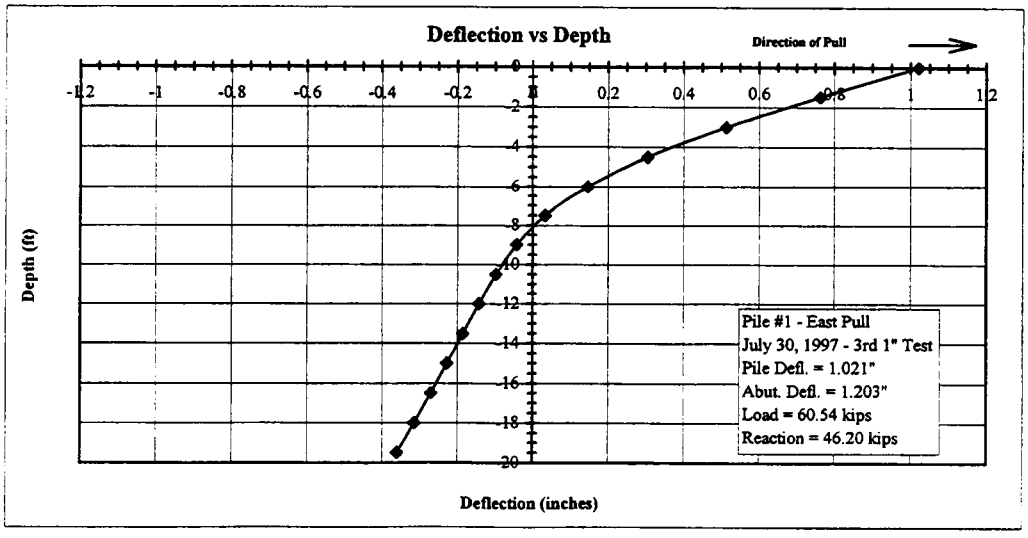
y = 0.000 + 0 C_1 + C_2 = 1.021

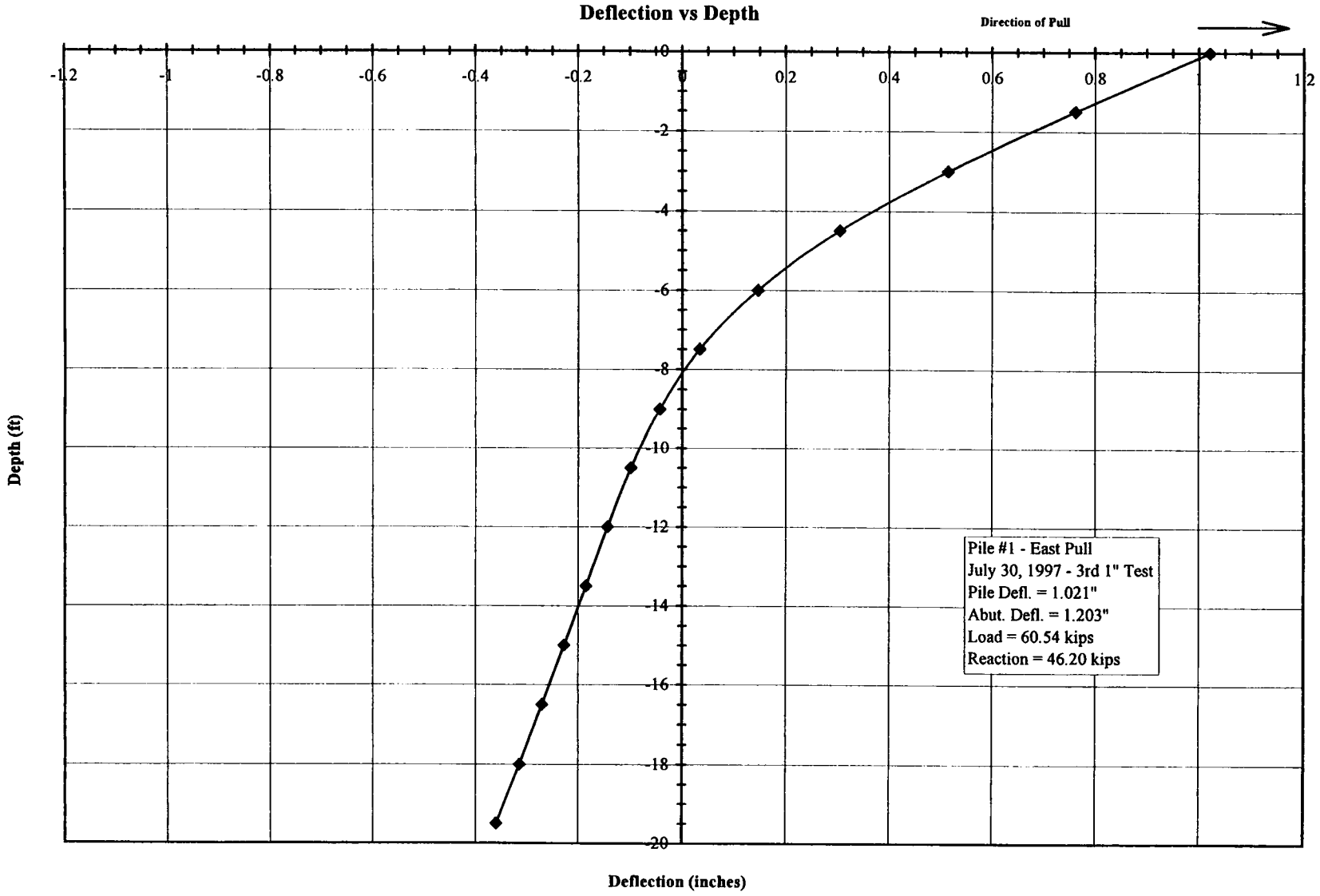
@ x = 8.09977

y = 0.334 + 8.09977 C_1 + 1.021 = 0

C_1 = -0.1672913 = -0.0139409 radians

Deflection (in)	Depth (ft)	Station	Depth (ft)
1.021	0	1	0
0.762	-1.5	2	1.5
0.514	-3	3	3
0.305	-4.5	4	4.5
0.146	-6	5	6
0.034	-7.5	6	7.5
-0.043	-9	7	9
-0.098	-10.5	8	10.5
-0.143	-12	9	12
-0.185	-13.5	10	13.5
-0.227	-15	11	15
-0.270	-16.5	12	16.5
-0.314	-18	13	18
-0.360	-19.5	14	19.5




Deflection vs Depth
Direction of Pull
-1.2
-1
-0.8
-0.6
-0.4
-0.2
0
0.2
0.4
0.6
0.8
1
1.2
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Deflection (inches)
Pile #1 - East Pull
July 30, 1997 - 3rd 1" Test
Pile Defl. = 1.021"
Abut. Defl. = 1.203"
Load = 60.54 kips
Reaction = 46.20 kips

APPENDIX G

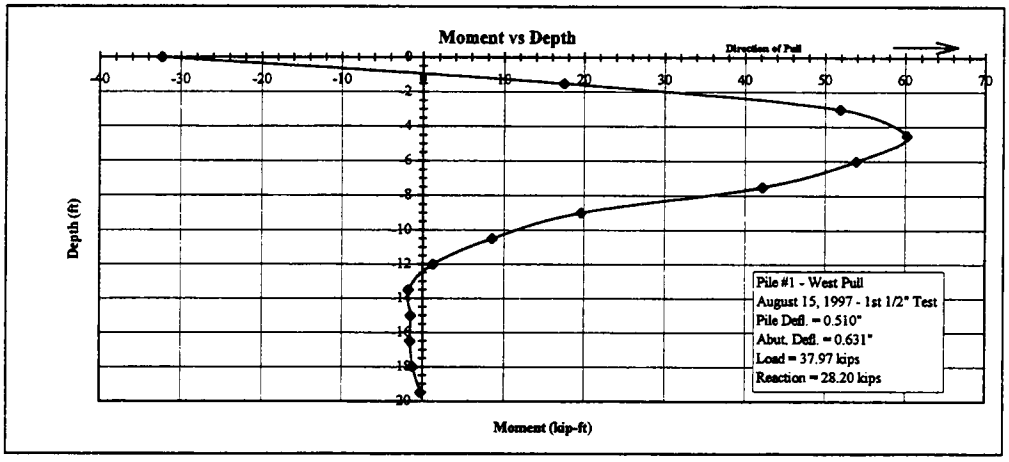
ONE-HALF INCH INCREMENT

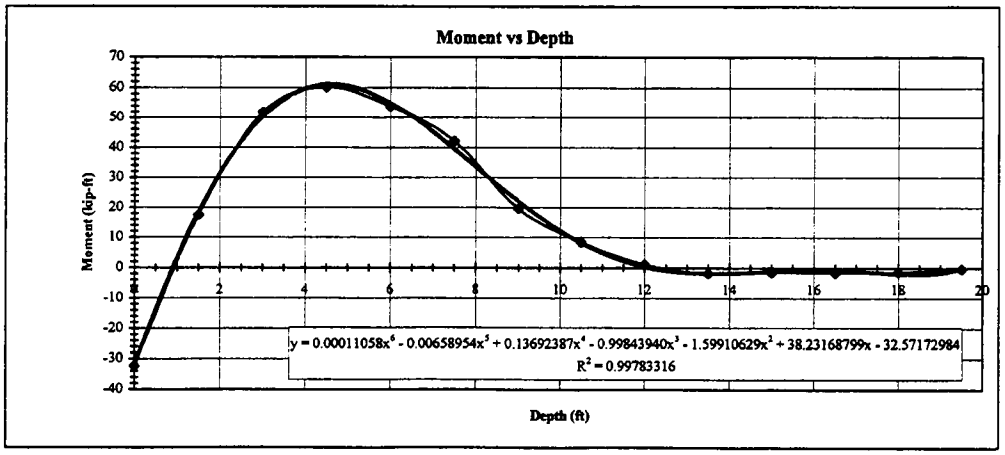
FOR AUGUST 15, 1997

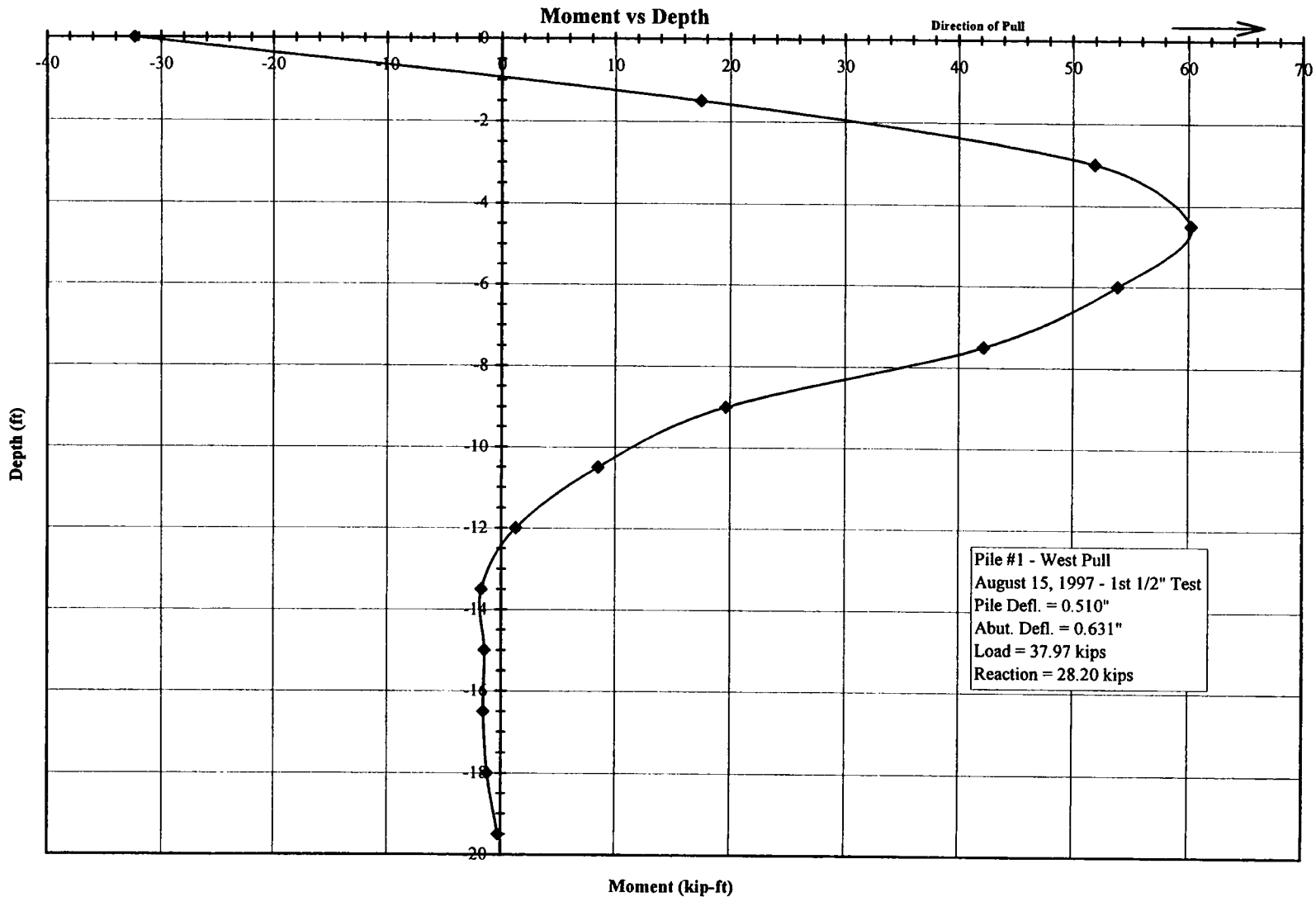
Moments

Date	08/15/97	
Time	8890-8900	sec
Load	37.97	kips
Reaction	28.20	kips
Abut. Defl.	0.631	inches
Pile Defl.	0.510	inches

Moment (kip-ft)	Depth (ft)	Station	Depth (ft)
-32.354	0	1	0
17.480	-1.5	2	1.5
51.901	-3	3	3
60.241	-4.5	4	4.5
53.891	-6	5	6
42.214	-7.5	6	7.5
19.650	-9	7	9
8.530	-10.5	8	10.5
1.277	-12	9	12
-1.761	-13.5	10	13.5
-1.466	-15	11	15
-1.562	-16.5	12	16.5
-1.218	-18	13	18
-0.254	-19.5	14	19.5






Moment vs Depth
Direction of Pull
-40
-30
-20
-10
0
10
20
30
40
50
60
70
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Moment (kip-ft)
Pile #1 - West Pull
August 15, 1997 - 1st 1/2" Test
Pile Defl. = 0.510"
Abut. Defl. = 0.631"
Load = 37.97 kips
Reaction = 28.20 kips

Equations

Date	08/15/97		
Time	8890-8900	sec	
Load	37.97	kips	
Reaction	28.20	kips	
Abut. Defl.	0.631	inches	
Pile Defl.	0.510	inches	$= C_2$

Modulus of Elasticity (E) =	33000	ksi
Moment of Inertia (I) =	223	in^4

Deflection Equation:
From 0' to 19.5'

$$y = 4.63675E\text{-}10\ x^8 + -3.68409E\text{-}08\ x^7 + 1.07172E\text{-}06\ x^6 + -1.17224E\text{-}05\ x^5 + -3.1291E\text{-}05\ x^4 + 0.001496226\ x^3 + -0.003824157\ x^2 + (C_1)\ x + 0.510\ (C_2)$$

Slope Equation:
From 0' to 19.5'

$$y = 3.09117E\text{-}10\ x^7 + -2.14906E\text{-}08\ x^6 + 5.35862E\text{-}07\ x^5 + -4.88433E\text{-}06\ x^4 + -1.043E\text{-}05\ x^3 + 0.000374056\ x^2 + -0.00063736\ x + (C_1)$$

Moment Equation:
From 0' to 19.5'

$$\mathbf{y = 0.00011058\ x^6 + -0.00658954\ x^5 + 0.13692387\ x^4 + -0.99843940\ x^3 + -1.59910629\ x^2 + 38.23168799\ x + -32.57172984}$$

Shear Equation:
From 0' to 19.5'

$$y = 0.00066348\ x^5 + -0.03294770\ x^4 + 0.54769548\ x^3 + -2.99531820\ x^2 + -3.19821258\ x + 38.23168799$$

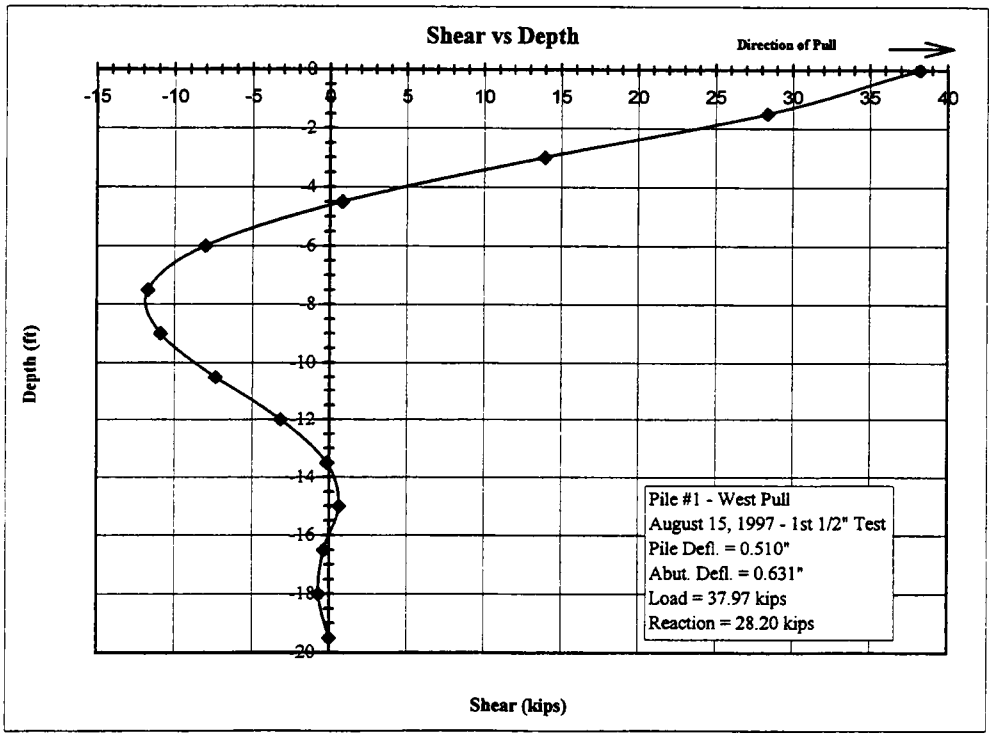
Pressure Equation:
From 0' to 19.5'

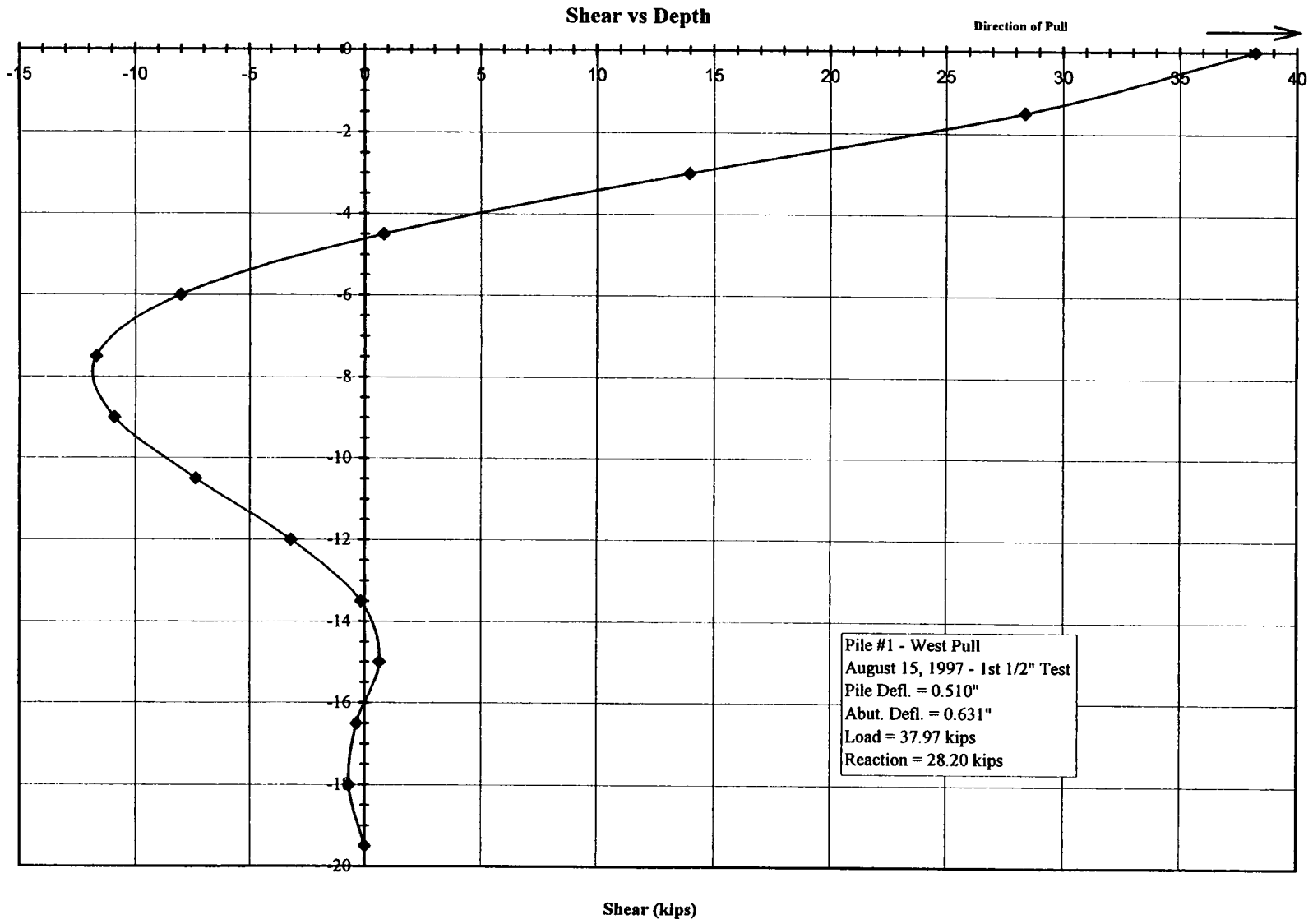
$$y = 0.0033174\ x^4 + -0.1317908\ x^3 + 1.64308644\ x^2 + -5.9906364\ x + -3.19821258$$

Shear

Date	08/15/97	
Time	8890-8900	sec
Load	37.97	kips
Reaction	28.20	kips
Abut. Def	0.631	inches
Pile Defl.	0.510	inches

Shear (kips)	Depth (ft)	Station	Depth (ft)
38.232	0	1	0
28.382	-1.5	2	1.5
13.959	-3	3	3
0.807	-4.5	4	4.5
-8.028	-6	5	6
-11.686	-7.5	6	7.5
-10.895	-9	7	9
-7.360	-10.5	8	10.5
-3.163	-12	9	12
-0.158	-13.5	10	13.5
0.637	-15	11	15
-0.360	-16.5	12	16.5
-0.686	-18	13	18
0.000	-19.5	14	19.5

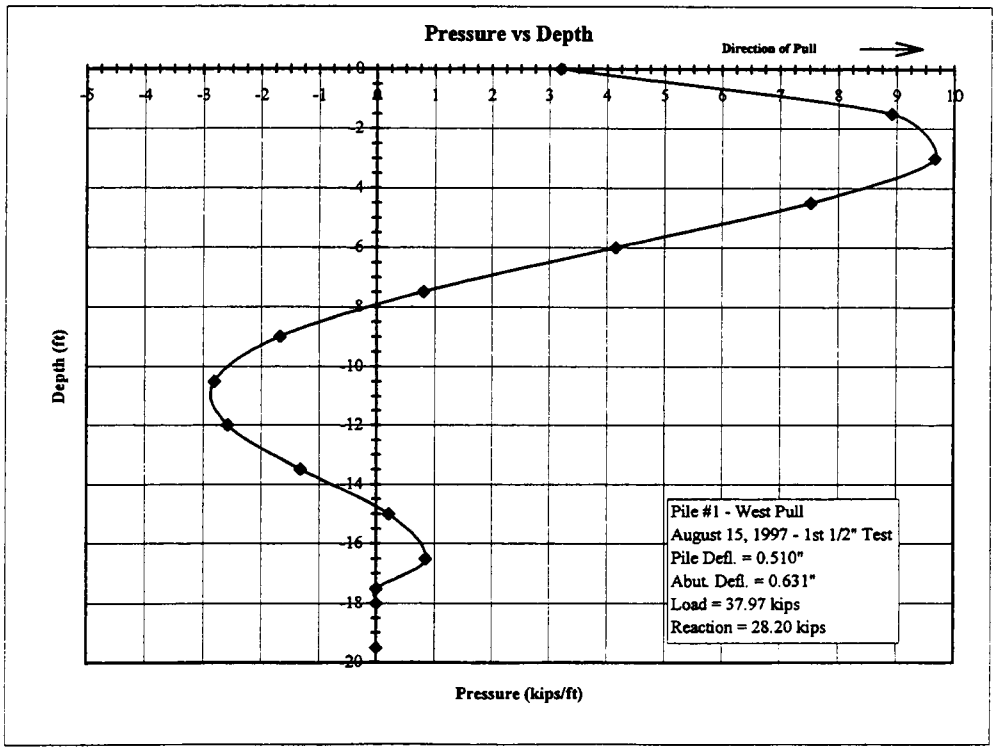


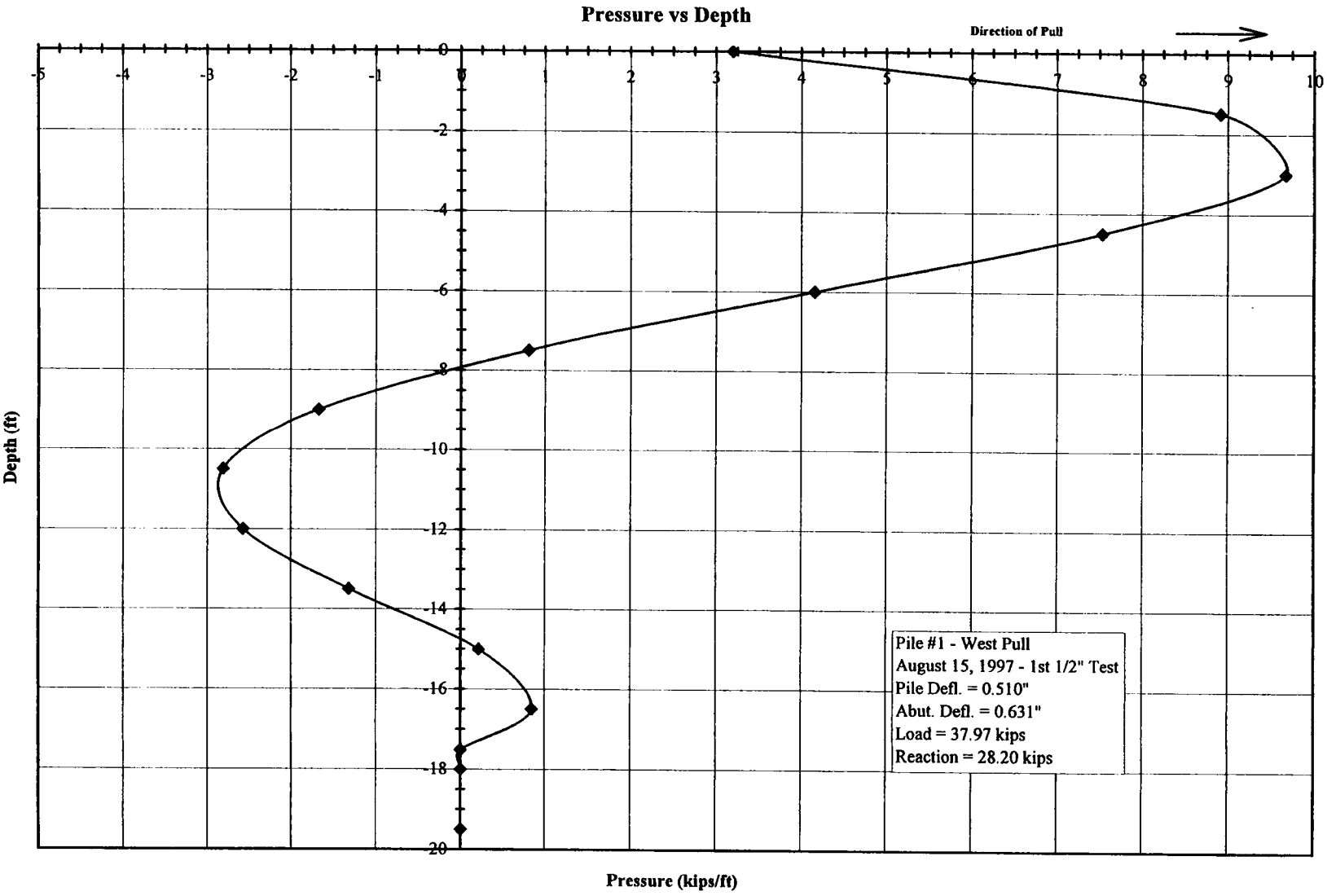

Shear vs Depth
Direction of Pull
-15
-10
-5
0
5
10
15
20
25
30
35
40
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Shear (kips)
Pile #1 - West Pull
August 15, 1997 - 1st 1/2" Test
Pile Defl. = 0.510"
Abut. Defl. = 0.631"
Load = 37.97 kips
Reaction = 28.20 kips

Pressure

Date	08/15/97	
Time	8890-8900	sec
Load	37.97	kips
Reaction	28.20	kips
Abut. Defl.	0.631	inches
Pile Defl.	0.510	inches

Pressure (kips/ft)	Depth (ft)	Station	Depth (ft)
3.198	0	1	0
8.915	-1.5	2	1.5
9.672	-3	3	3
7.533	-4.5	4	4.5
4.158	-6	5	6
0.807	-7.5	6	7.5
-1.666	-9	7	9
-2.809	-10.5	8	10.5
-2.574	-12	9	12
-1.314	-13.5	10	13.5
0.214	-15	11	15
0.848	-16.5	12	16.5
0.000	-17.51256		17.51256
0.000	-18	13	18
0.000	-19.5	14	19.5




Pressure vs Depth
Direction of Pull
Depth (ft)
Pressure (kips/ft)
Pile #1 - West Pull
August 15, 1997 - 1st 1/2" Test
Pile Defl. = 0.510"
Abut. Defl. = 0.631"
Load = 37.97 kips
Reaction = 28.20 kips

Statics

Time	8890-8900	sec
Load	37.97	kips
Reaction	28.20	kips
Abut. Defl.	0.631	inches
Pile Defl.	0.510	inches

Load by Statics

Pressure Equation

$y = 0.00331740\,x^4 + -0.13179080\,x^3 + 1.643086440\,x^2 + -5.99063640\,x + -3.19821258$

Roots for pressure equation

x = -0.47076 x = 7.91960 x = 14.76574 x = 17.51256

Integrate to check statics:

From 0 to 7.9196

$= -0.000663480\,x^5 + 0.032947700\,x^4 + -0.547695480\,x^3 + 2.995318200\,x^2 + 3.19821258\,x$

= 50.08471277 - 0 = **50.08471 kips**

From 7.9196 to 14.76574

$= -0.000663480\,x^5 + 0.032947700\,x^4 + -0.547695480\,x^3 + 2.995318200\,x^2 + 3.19821258\,x$

= 37.56931473 - 50.08471277 = **-12.51540 kips**

From 14.76574 to 17.5126

$= -0.000663480\,x^5 + 0.032947700\,x^4 + -0.547695480\,x^3 + 2.995318200\,x^2 + 3.19821258\,x$

= 39.13804792 - 37.56931473 = **1.56873 kips**

From Pressure	**39.138**	**kips**
Measured	**37.97**	**kips**
% difference	***3.08***	**%**

Moment by Statics

M_{GRND} = (Reaction)(x) - (Load)(y) - (Weight)(e)

x =	6.75	ft	(distance from centerline of the pile to centerline of the reaction)
y =	4.2083	ft	(distance from centerline of the load to the ground surface)
e =	0.121	in	($e = \Delta_{abut} - \Delta_{pile}$)
Weight =	65.0	kips	

M_{GRND} = 190.350 - 159.790 - 0.655 = **29.904 kip-ft**

Measured = 32.354 kip-ft

% difference = *7.57* %

Deflections

Date	08/15/97	
Time	8890-8900	sec
Load	37.97	kips
Reaction	28.20	kips
Abut. Defl.	0.631	inches
Pile Defl.	0.510	inches

@ x = 0

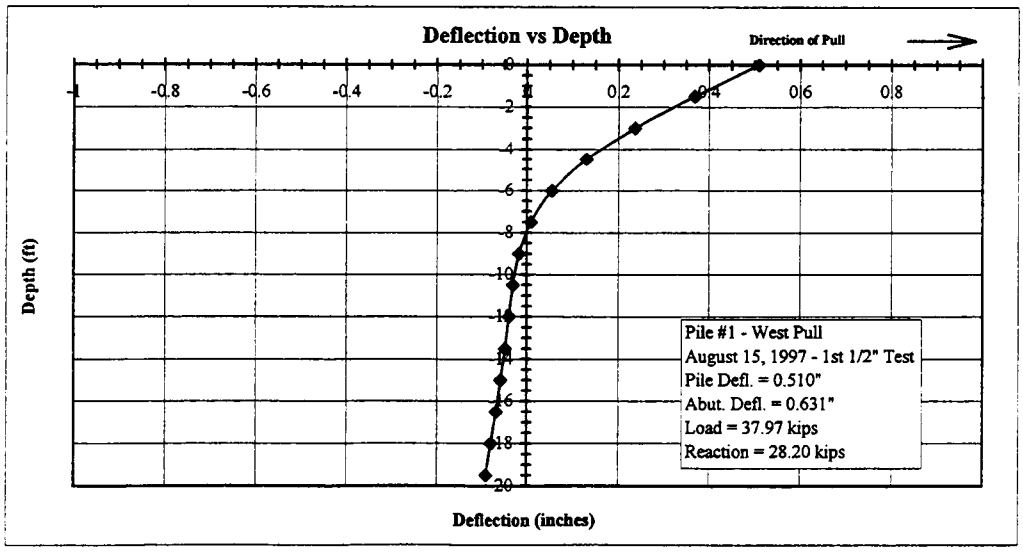
y = 0.000 + 0 C_1 + C_2 = 0.510

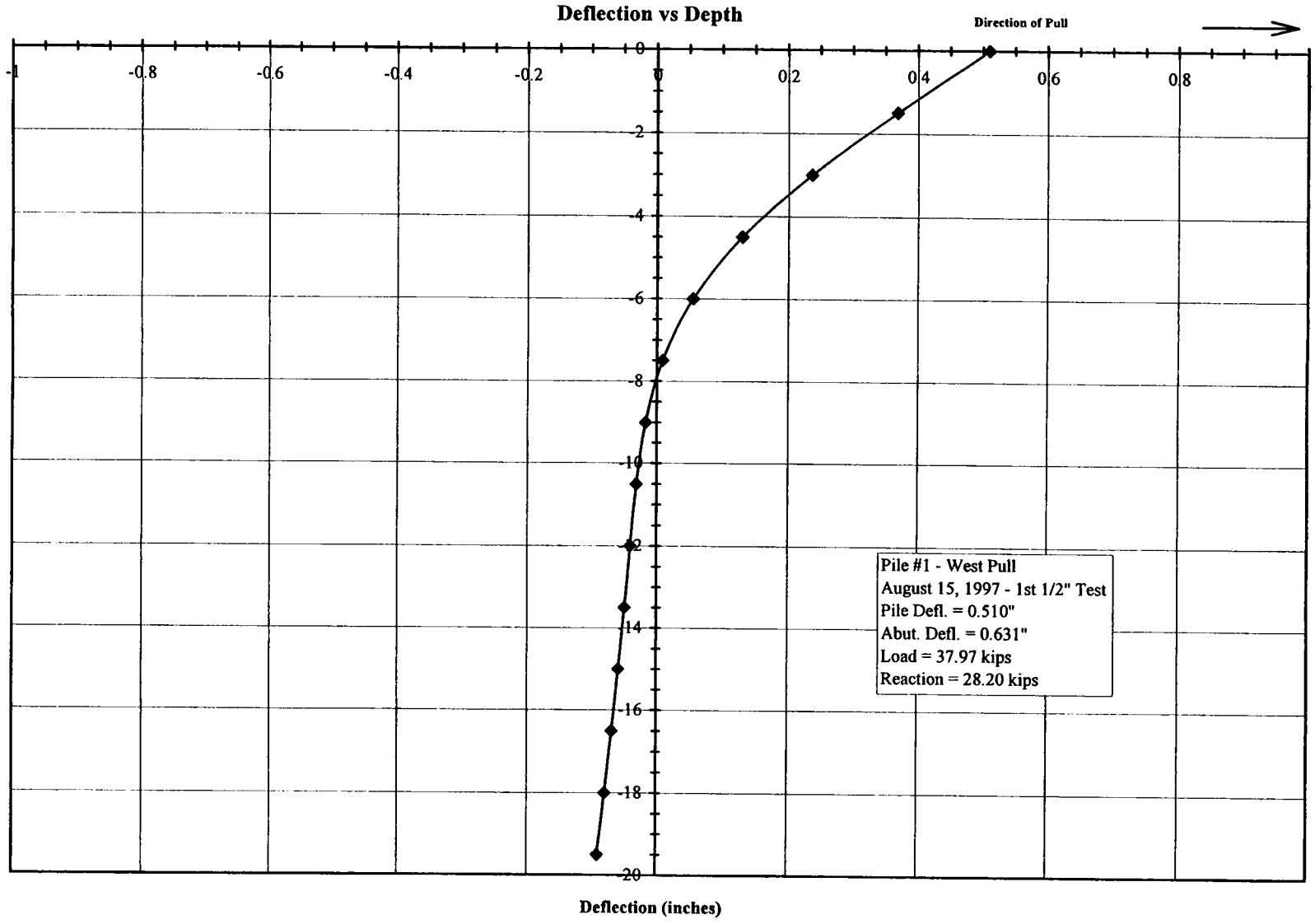
@ x = 7.91960

y = 0.215 + 7.9196 C_1 + 0.510 = 0

C_1 = -0.0915032 = -0.0076253 radians

Deflection (in)	Depth (ft)	Station	Depth (ft)
0.510	0	1	0
0.369	-1.5	2	1.5
0.237	-3	3	3
0.130	-4.5	4	4.5
0.055	-6	5	6
0.009	-7.5	6	7.5
-0.017	-9	7	9
-0.030	-10.5	8	10.5
-0.040	-12	9	12
-0.048	-13.5	10	13.5
-0.058	-15	11	15
-0.068	-16.5	12	16.5
-0.079	-18	13	18
-0.090	-19.5	14	19.5




Deflection vs Depth
Direction of Pull
-1
-0.8
-0.6
-0.4
-0.2
0
0.2
0.4
0.6
0.8
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Deflection (inches)
Pile #1 - West Pull
August 15, 1997 - 1st 1/2" Test
Pile Defl. = 0.510"
Abut. Defl. = 0.631"
Load = 37.97 kips
Reaction = 28.20 kips

APPENDIX H

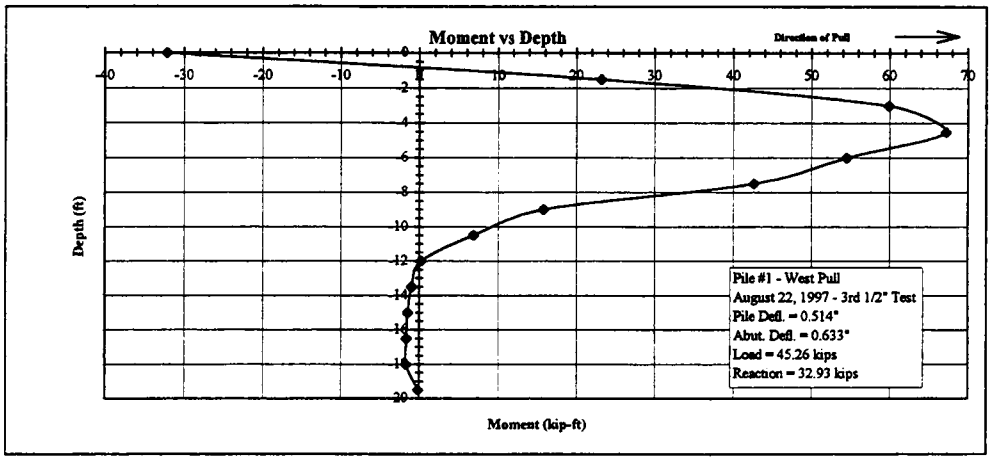
ONE-HALF INCH INCREMENT

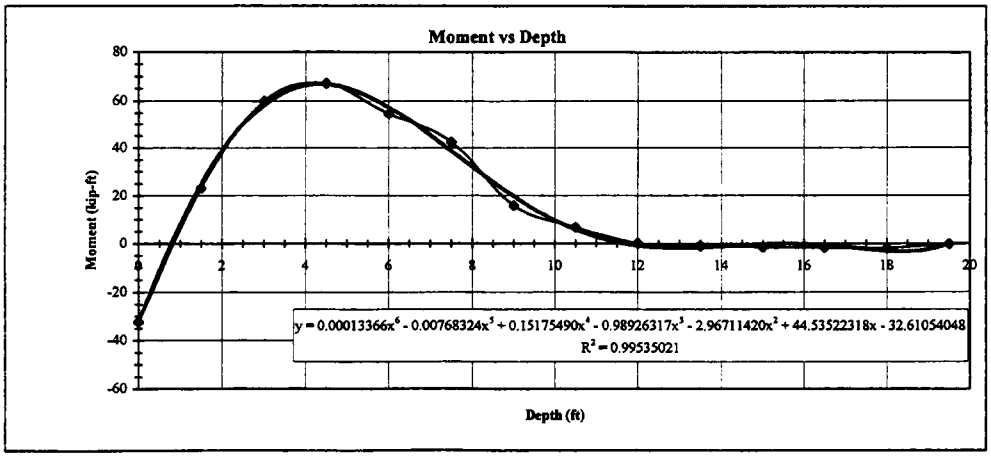
FOR AUGUST 22, 1997

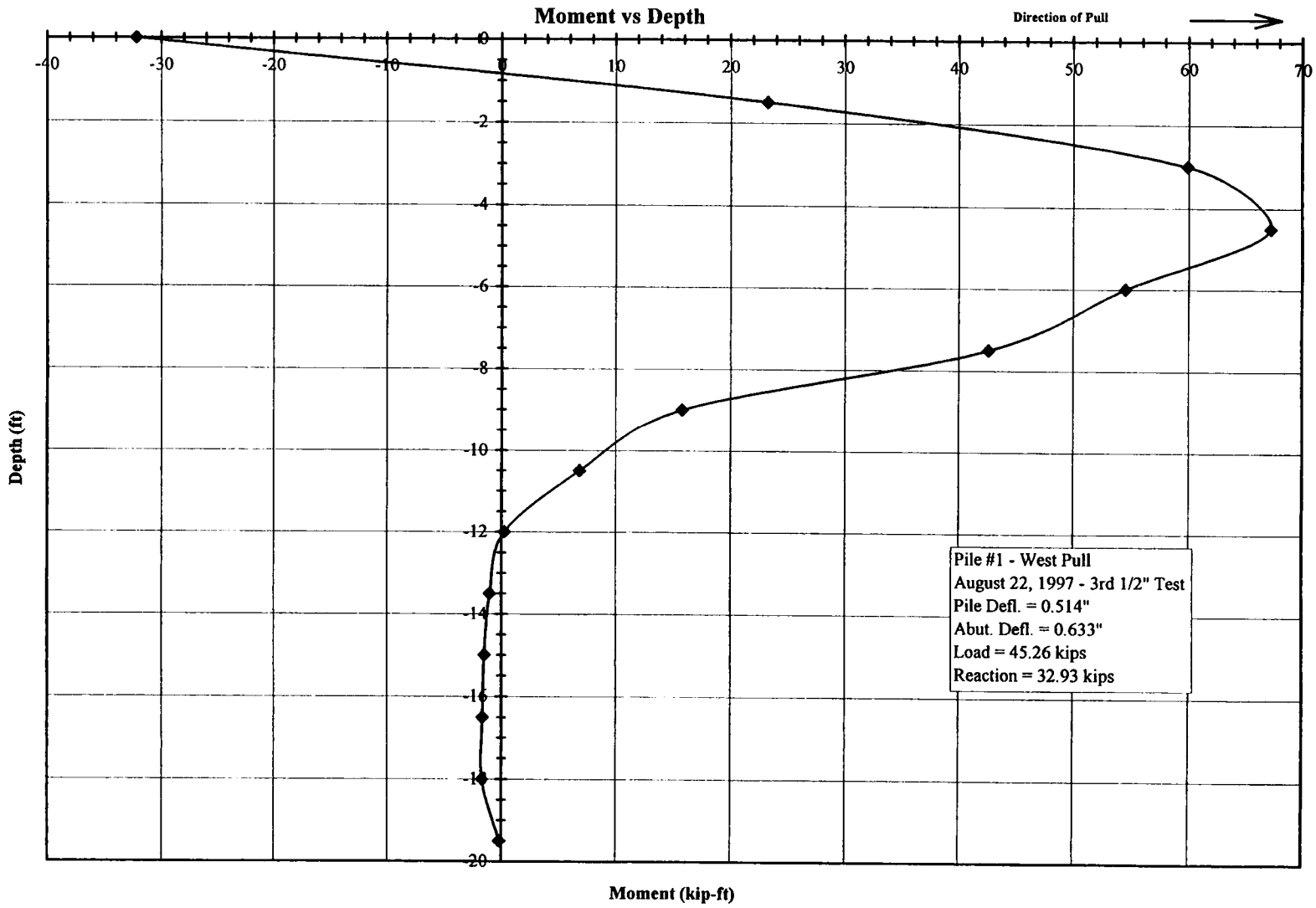
Moments

Date	08/22/97	
Time	8300-8310	sec
Load	45.26	kips
Reaction	32.93	kips
Abut. Defl.	0.633	inches
Pile Defl.	0.514	inches

Moment (kip-ft)	Depth (ft)	Station	Depth (ft)
-32.180	0	1	0
23.229	-1.5	2	1.5
59.918	-3	3	3
67.239	-4.5	4	4.5
54.511	-6	5	6
42.632	-7.5	6	7.5
15.825	-9	7	9
6.847	-10.5	8	10.5
0.250	-12	9	12
-1.013	-13.5	10	13.5
-1.490	-15	11	15
-1.648	-16.5	12	16.5
-1.706	-18	13	18
-0.182	-19.5	14	19.5






Moment vs Depth
Direction of Pull
-40
-30
-20
-10
0
10
20
30
40
50
60
70
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Moment (kip-ft)
Pile #1 - West Pull
August 22, 1997 - 3rd 1/2" Test
Pile Defl. = 0.514"
Abut. Defl. = 0.633"
Load = 45.26 kips
Reaction = 32.93 kips

Equations

Date	08/22/97	
Time	8300-8310	sec
Load	45.26	kips
Reaction	32.93	kips
Abut. Defl.	0.633	inches
Pile Defl.	0.514	inches $= C_2$

Modulus of Elasticity (E) =	33000	ksi
Moment of Inertia (I) =	223	in^4

Deflection Equation:
From 0' to 19.5'

$$y = 5.60452E\text{-}10\ x^8 + -4.29556E\text{-}08\ x^7 + 1.18781E\text{-}06\ x^6 + -1.16147E\text{-}05\ x^5 + -5.80601E\text{-}05\ x^4 + 0.001742919\ x^3 + -0.003828714\ x^2 + (C_1)\ x + 0.514\ (C_2)$$

Slope Equation:
From 0' to 19.5'

$$y = 3.73635E\text{-}10\ x^7 + -2.50574E\text{-}08\ x^6 + 5.93904E\text{-}07\ x^5 + -4.83944E\text{-}06\ x^4 + -1.93534E\text{-}05\ x^3 + 0.00043573\ x^2 + -0.000638119\ x + (C_1)$$

Moment Equation:
From 0' to 19.5'

$$\mathbf{y = 0.00013366\ x^6 + -0.00768324\ x^5 + 0.15175490\ x^4 + -0.98926317\ x^3 + -2.96711420\ x^2 + 44.53522318\ x + -32.61054048}$$

Shear Equation:
From 0' to 19.5'

$$y = 0.00080196\ x^5 + -0.03841620\ x^4 + 0.60701960\ x^3 + -2.96778951\ x^2 + -5.93422840\ x + 44.53522318$$

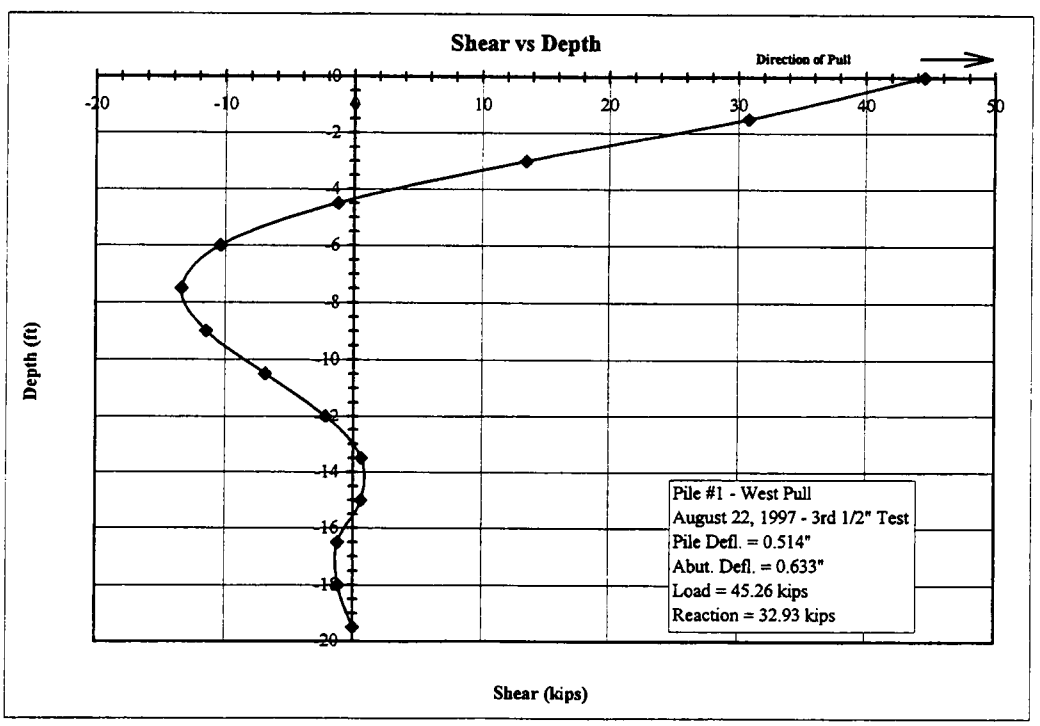
Pressure Equation:
From 0' to 19.5'

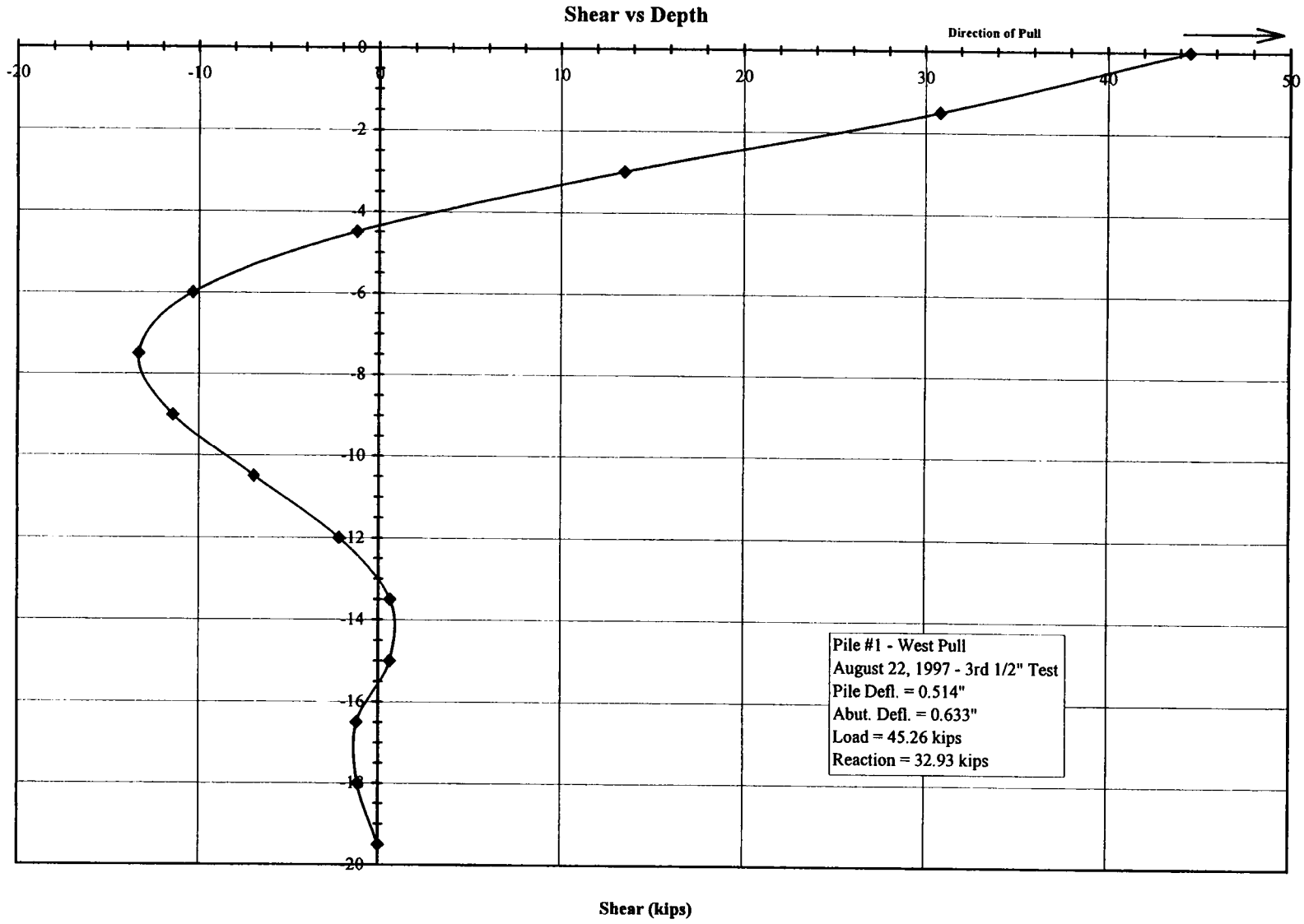
$$y = 0.0040098\ x^4 + -0.1536648\ x^3 + 1.8210588\ x^2 + -5.93557902\ x + -5.93422840$$

Shear

Date	08/22/97	
Time	8300-8310	sec
Load	45.26	kips
Reaction	32.93	kips
Abut. Defl.	0.633	inches
Pile Defl.	0.514	inches

Shear (kips)	Depth (ft)	Station	Depth (ft)
44.535	0	1	0
30.817	-1.5	2	1.5
13.495	-3	3	3
-1.225	-4.5	4	4.5
-10.346	-6	5	6
-13.344	-7.5	6	7.5
-11.440	-9	7	9
-6.871	-10.5	8	10.5
-2.152	-12	9	12
0.644	-13.5	10	13.5
0.629	-15	11	15
-1.182	-16.5	12	16.5
-1.127	-18	13	18
0.000	-19.5	14	19.5

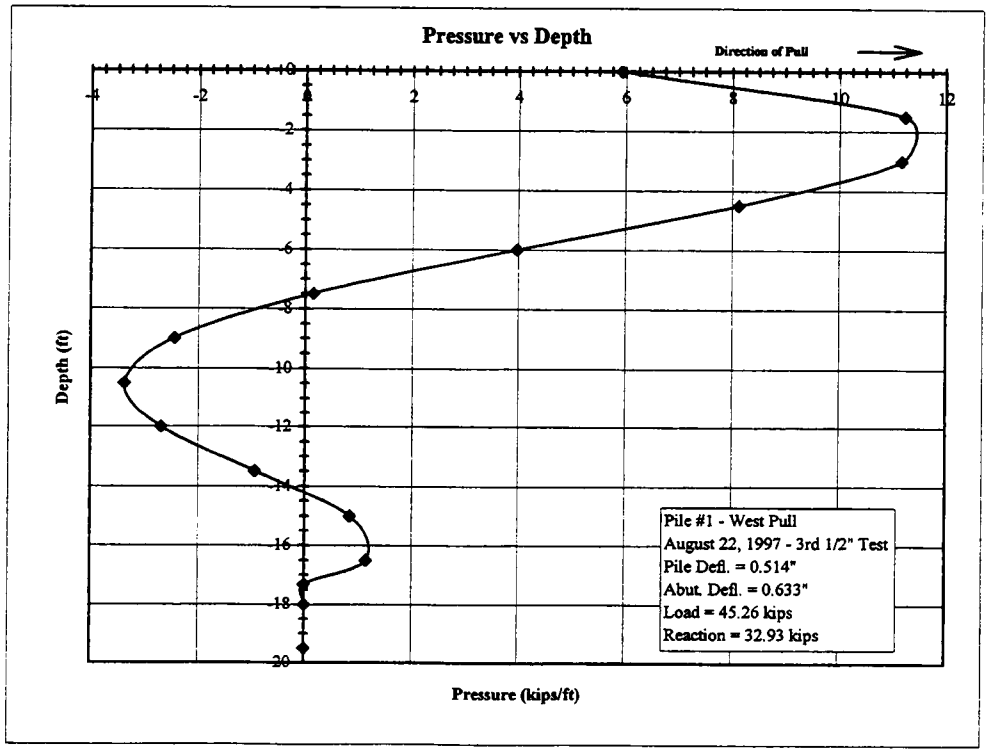


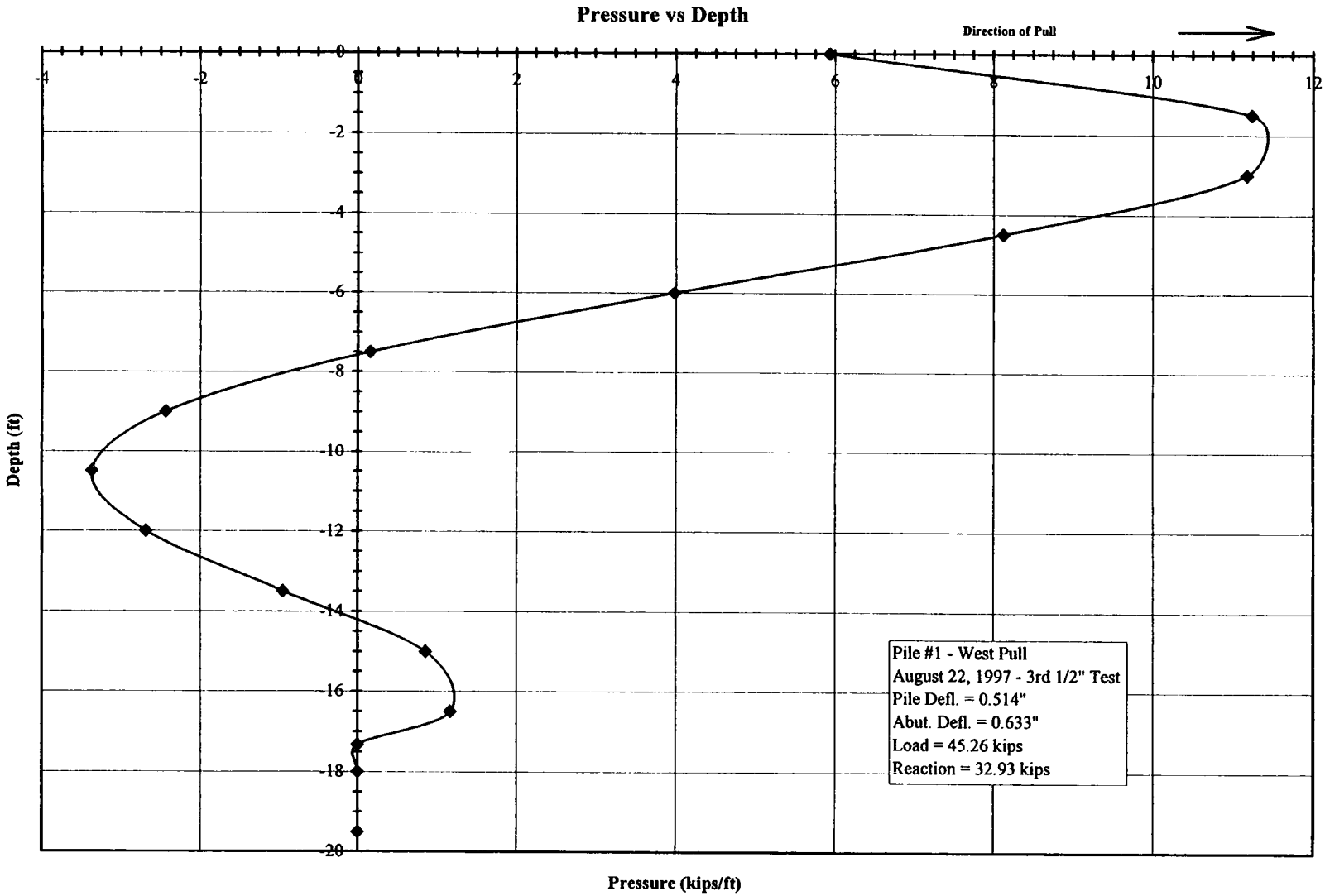

Shear vs Depth
Direction of Pull
-20
-10
0
10
20
30
40
50
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Shear (kips)
Pile #1 - West Pull
August 22, 1997 - 3rd 1/2" Test
Pile Defl. = 0.514"
Abut. Defl. = 0.633"
Load = 45.26 kips
Reaction = 32.93 kips

Pressure

Date	08/22/97	
Time	8300-8310	sec
Load	45.26	kips
Reaction	32.93	kips
Abut. Defl.	0.633	inches
Pile Defl.	0.514	inches

Pressure (kips/ft)	Depth (ft)	Station	Depth (ft)
5.934	0	1	0
11.239	-1.5	2	1.5
11.176	-3	3	3
8.126	-4.5	4	4.5
3.984	-6	5	6
0.157	-7.5	6	7.5
-2.438	-9	7	9
-3.367	-10.5	8	10.5
-2.686	-12	9	12
-0.936	-13.5	10	13.5
0.852	-15	11	15
1.163	-16.5	12	16.5
0.000	-17.32395		17.32395
0.000	-18	13	18
0.000	-19.5	14	19.5




Pressure vs Depth
Direction of Pull
Depth (ft)
Pressure (kips/ft)
Pile #1 - West Pull
August 22, 1997 - 3rd 1/2" Test
Pile Defl. = 0.514"
Abut. Defl. = 0.633"
Load = 45.26 kips
Reaction = 32.93 kips

Statics

Date	08/22/97	
Time	8300-8310	sec
Load	45.26	kips
Reaction	32.93	kips
Abut. Defl.	0.633	inches
Pile Defl.	0.514	inches

Load by Statics

Pressure Equation

$y =$ 0.00400980 $x^4 +$ -0.15366480 $x^3 +$ 1.821058800 $x^2 +$ -5.93557902 $x +$ -5.9342284

Roots for pressure equation

x = -0.79343 x = 7.57132 x = 14.22047 x = 17.32395

Integrate to check statics:

From 0 to 7.57132

= -0.000801960 $x^5 +$ 0.038416200 $x^4 +$ -0.607019600 $x^3 +$ 2.967789510 $x^2 +$ 5.9342284 x

= 57.88441731 - 0 = **57.88442 kips**

From 7.57132 to 14.22047

= -0.000801960 $x^5 +$ 0.038416200 $x^4 +$ -0.607019600 $x^3 +$ 2.967789510 $x^2 +$ 5.9342284 x

= 43.55694794 - 57.88441731 = **-14.32747 kips**

From 14.22047 to 17.3240

= -0.000801960 $x^5 +$ 0.038416200 $x^4 +$ -0.607019600 $x^3 +$ 2.967789510 $x^2 +$ 5.9342284 x

= 46.28043269 - 43.55694794 = **2.72348 kips**

From Pressure	**=**	**46.280 kips**
Measured	**=**	**45.26 kips**
% difference	**=**	***2.25*** **%**

Moment by Statics

M_{GRND} = (Reaction)(x) - (Load)(y) - (Weight)(e)

x =	6.75	ft	(distance from centerline of the pile to centerline of the reaction)
y =	4.2083	ft	(distance from centerline of the load to the ground surface)
e =	0.119	in	($e = \Delta_{abut} - \Delta_{pile}$)
Weight =	65.0	kips	

M_{GRND} = 222.278 - 190.469 - 0.645 = **31.164 kip-ft**

Measured	**=**	**32.180**	**kip-ft**
% difference	**=**	***3.16***	**%**

Deflections

Date	08/22/97	
Time	8300-8310	sec
Load	45.26	kips
Reaction	32.93	kips
Abut. Defl.	0.633	inches
Pile Defl.	0.514	inches

@ x = 0

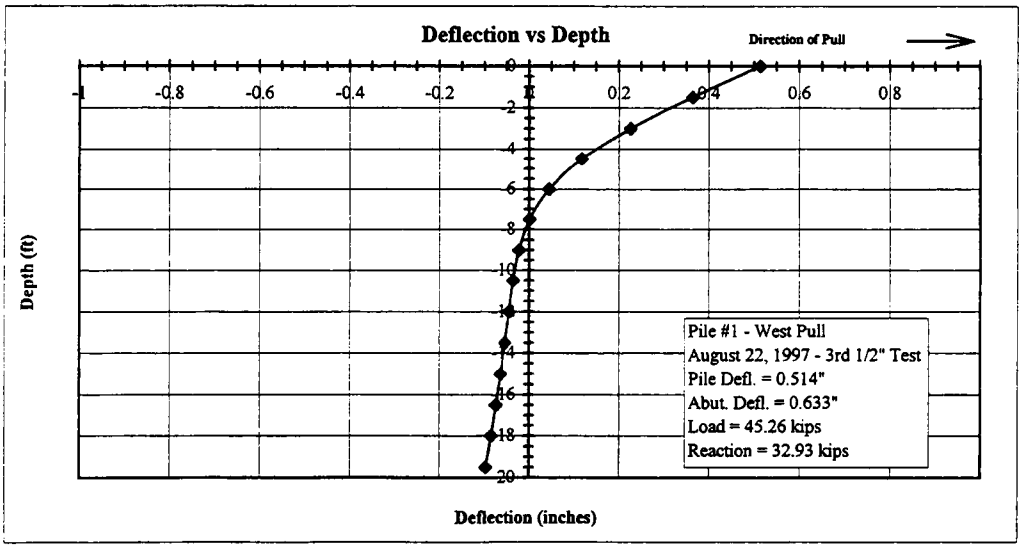
y = 0.000 + 0 C_1 + C_2 = 0.514

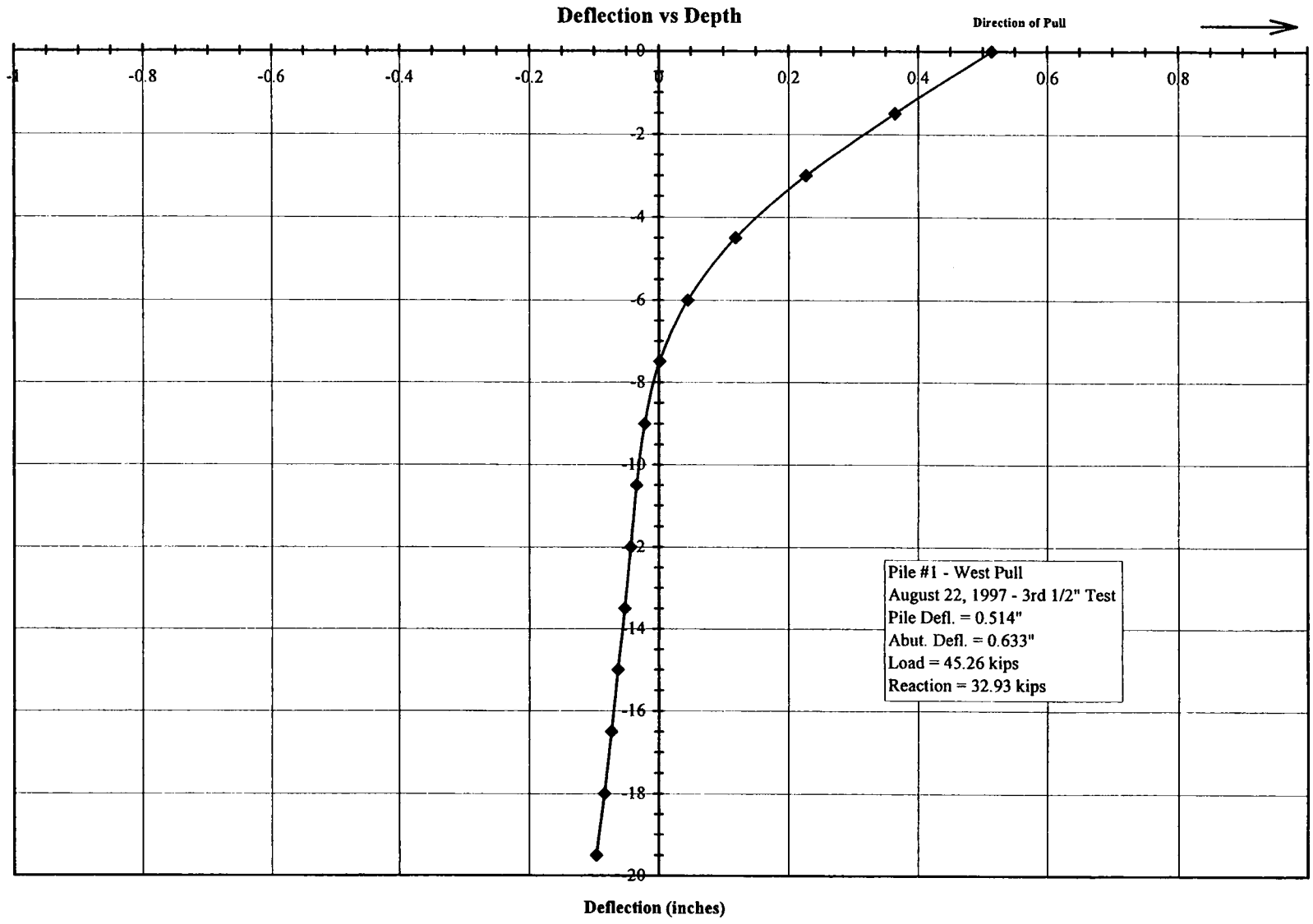
@ x = 7.57132

y = 0.226 + 7.57132 C_1 + 0.514 = 0

C_1 = -0.0977056 = -0.0081421 radians

Deflection (in)	Depth (ft)	Station	Depth (ft)
0.514	0	1	0
0.364	-1.5	2	1.5
0.227	-3	3	3
0.119	-4.5	4	4.5
0.045	-6	5	6
0.001	-7.5	6	7.5
-0.022	-9	7	9
-0.034	-10.5	8	10.5
-0.043	-12	9	12
-0.053	-13.5	10	13.5
-0.063	-15	11	15
-0.073	-16.5	12	16.5
-0.084	-18	13	18
-0.096	-19.5	14	19.5




Deflection vs Depth
Direction of Pull
-1
-0.8
-0.6
-0.4
-0.2
0
0.2
0.4
0.6
0.8
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Deflection (inches)
Pile #1 - West Pull
August 22, 1997 - 3rd 1/2" Test
Pile Defl. = 0.514"
Abut. Defl. = 0.633"
Load = 45.26 kips
Reaction = 32.93 kips

APPENDIX I

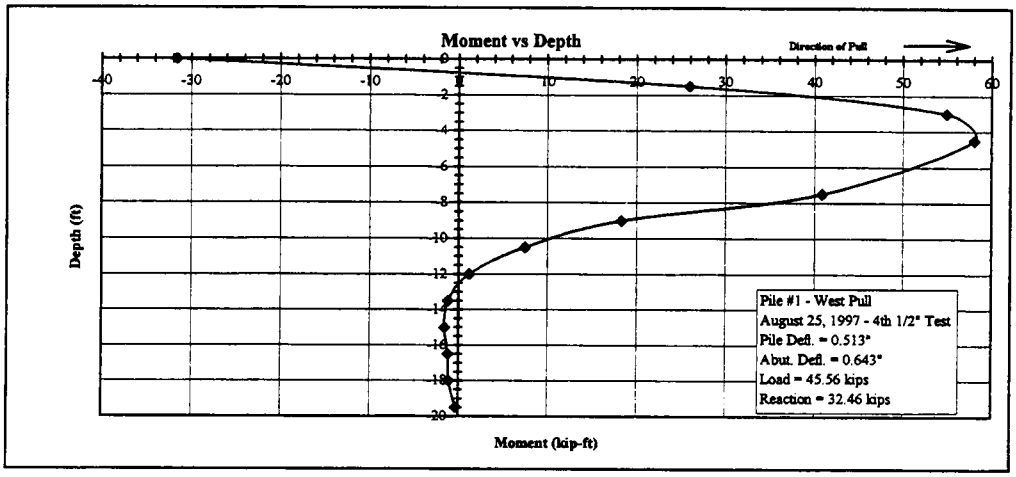
ONE-HALF INCH INCREMENT

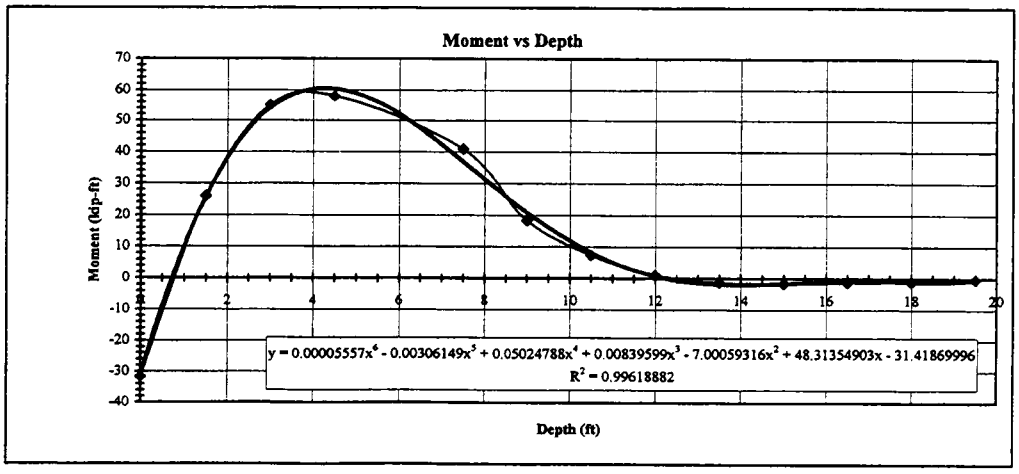
FOR AUGUST 25, 1997

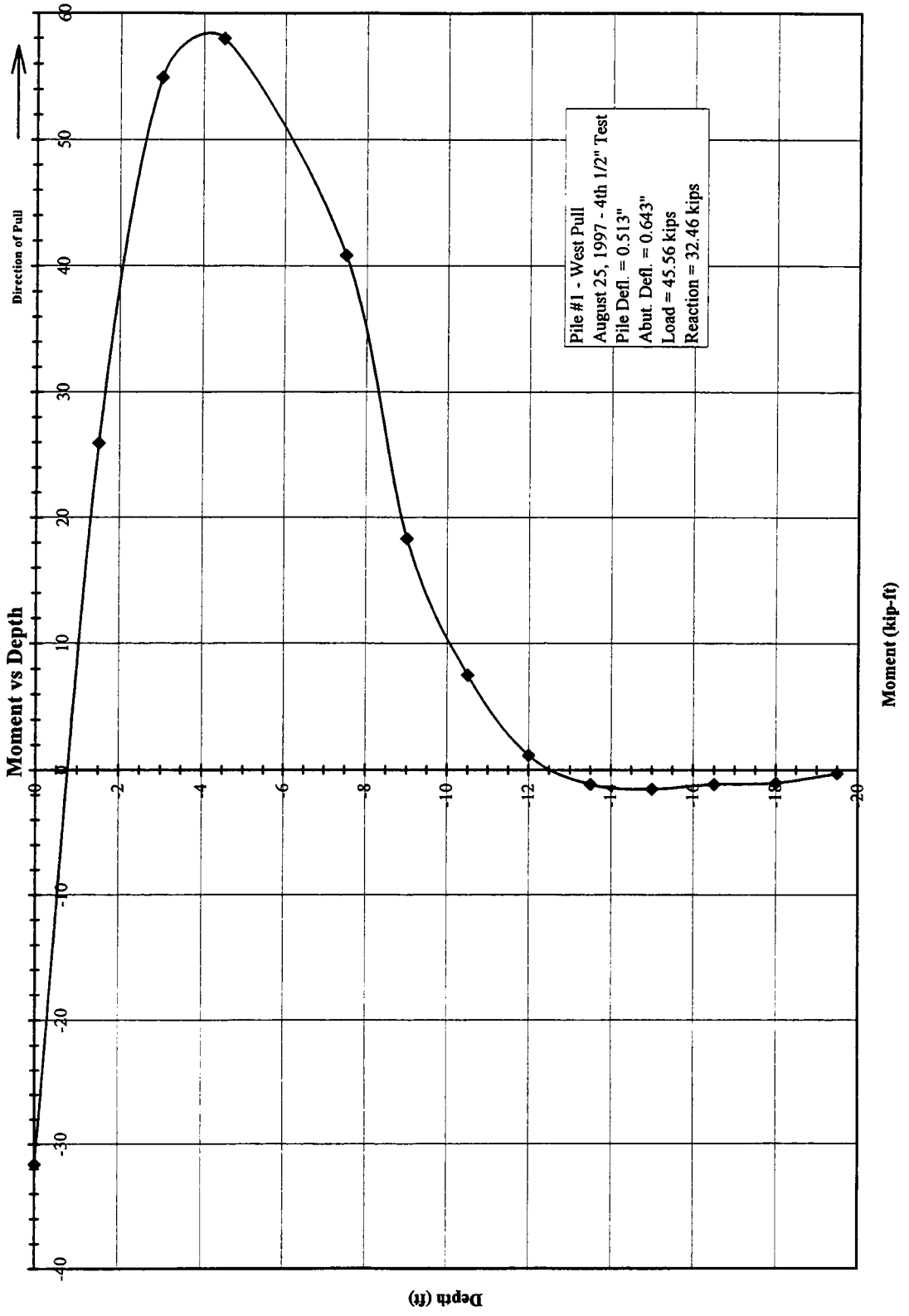
Moments

Date	08/25/97	
Time	7850-7860	sec
Load	45.56	kips
Reaction	32.46	kips
Abut. Defl.	0.643	inches
Pile Defl.	0.513	inches

Moment (kip-ft)	Depth (ft)	Station	Depth (ft)
-31.605	0	1	0
25.937	-1.5	2	1.5
54.943	-3	3	3
58.006	-4.5	4	4.5
40.866	-7.5	6	7.5
18.325	-9	7	9
7.527	-10.5	8	10.5
1.181	-12	9	12
-1.140	-13.5	10	13.5
-1.521	-15	11	15
-1.140	-16.5	12	16.5
-1.027	-18	13	18
-0.264	-19.5	14	19.5






Moment vs Depth
Direction of Pull
Pile #1 - West Pull
August 25, 1997 - 4th 1/2" Test
Pile Defl. = 0.513"
Abut. Defl. = 0.643"
Load = 45.56 kips
Reaction = 32.46 kips
Moment (kip-ft)
Depth (ft)
60
50
40
30
20
10
0
-10
-20
-30
-40
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20

Equations

Date	08/25/97	
Time	7850-7860	sec
Load	45.56	kips
Reaction	32.46	kips
Abut. Defl.	0.643	inches
Pile Defl.	0.513	inches $= C_2$
Modulus of Elasticity (E) =	33000	ksi
Moment of Inertia (I) =	223	in^4

Deflection Equation:
From 0' to 19.5'

$$y = 2.33011E\text{-}10\,x^8 + -1.71162E\text{-}08\,x^7 + 3.93298E\text{-}07\,x^6 + 9.8575E\text{-}08\,x^5 + -0.000136987\,x^4 + 0.001890787\,x^3 + -0.003688783\,x^2 + (C_1)\,x + 0.513\,(C_2)$$

Slope Equation:
From 0' to 19.5'

$$y = 1.55341E\text{-}10\,x^7 + -9.98448E\text{-}09\,x^6 + 1.96649E\text{-}07\,x^5 + 4.10729E\text{-}08\,x^4 + -4.56622E\text{-}05\,x^3 + 0.000472697\,x^2 + -0.000614797\,x + (C_1)$$

Moment Equation:
From 0' to 19.5'

$$\mathbf{y = 0.00005557\,x^6 + -0.00306149\,x^5 + 0.05024788\,x^4 + 0.00839599\,x^3 + -7.00059316\,x^2 + 48.31354903\,x + -31.41869996}$$

Shear Equation:
From 0' to 19.5'

$$y = 0.00033342\,x^5 + -0.01530745\,x^4 + 0.20099152\,x^3 + 0.02518797\,x^2 + -14.00118632\,x + 48.31354903$$

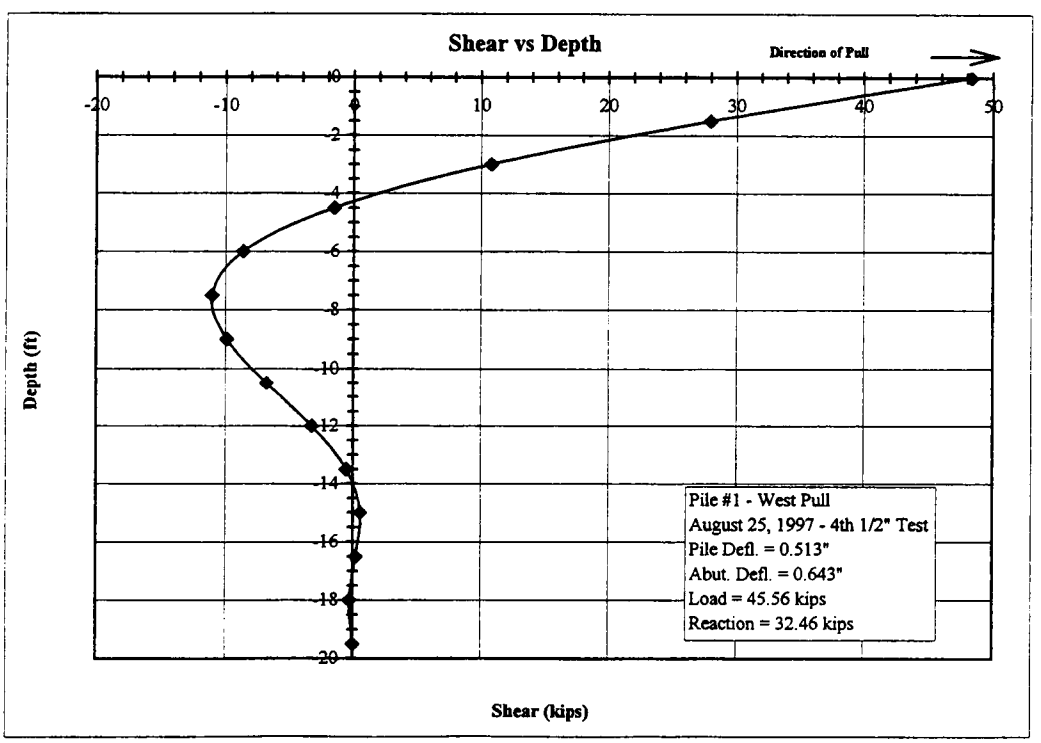
Pressure Equation:
From 0' to 19.5'

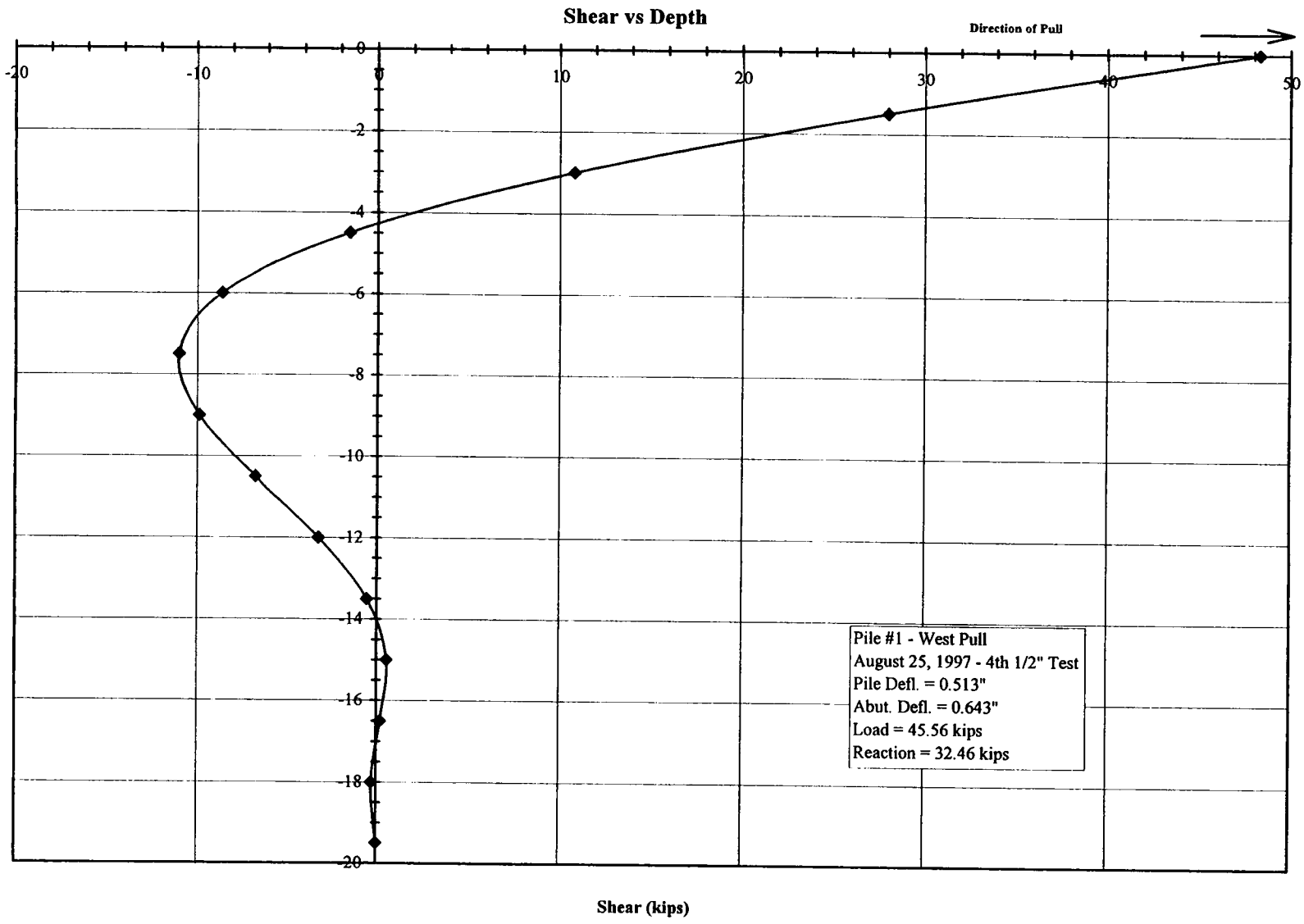
$$y = 0.0016671\,x^4 + -0.0612298\,x^3 + 0.60297456\,x^2 + 0.05037594\,x + -14.00118632$$

Shear

Date	08/25/97	
Time	7850-7860	sec
Load	45.56	kips
Reaction	32.46	kips
Abut. Defl.	0.643	inches
Pile Defl.	0.513	inches

Shear (kips)	Depth (ft)	Station	Depth (ft)
48.314	0	1	0
27.972	-1.5	2	1.5
10.805	-3	3	3
-1.528	-4.5	4	4.5
-8.618	-6	5	6
-11.007	-7.5	6	7.5
-9.878	-9	7	9
-6.758	-10.5	8	10.5
-3.210	-12	9	12
-0.529	-13.5	10	13.5
0.561	-15	11	15
0.208	-16.5	12	16.5
-0.259	-18	13	18
0.000	-19.5	14	19.5

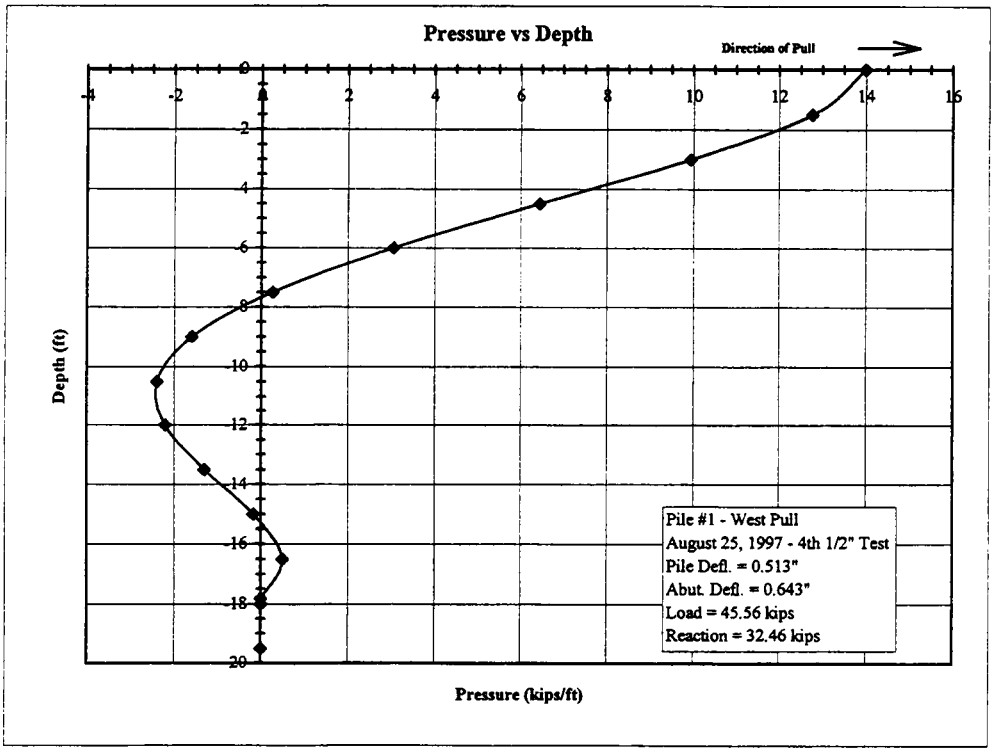


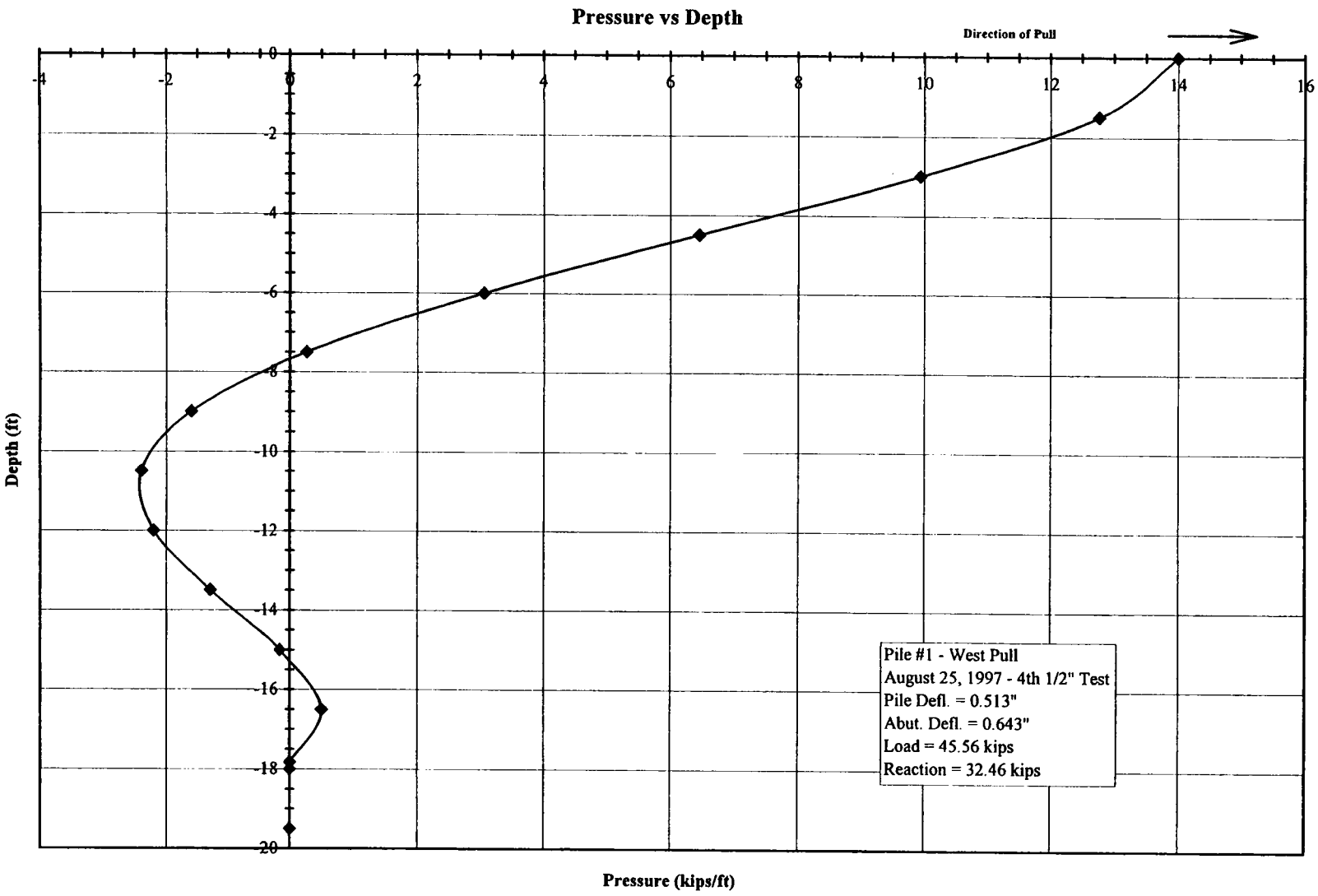

Shear vs Depth
Direction of Pull
-20
-10
0
10
20
30
40
50
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Shear (kips)
Pile #1 - West Pull
August 25, 1997 - 4th 1/2" Test
Pile Defl. = 0.513"
Abut. Defl. = 0.643"
Load = 45.56 kips
Reaction = 32.46 kips

Pressure

Date	08/25/97	
Time	7850-7860	sec
Load	45.56	kips
Reaction	32.46	kips
Abut. Defl.	0.643	inches
Pile Defl.	0.513	inches

Pressure (kips/ft)	Depth (ft)	Station	Depth (ft)
14.001	0	1	0
12.767	-1.5	2	1.5
9.941	-3	3	3
6.460	-4.5	4	4.5
3.057	-6	5	6
0.263	-7.5	6	7.5
-1.594	-9	7	9
-2.388	-10.5	8	10.5
-2.196	-12	9	12
-1.296	-13.5	10	13.5
-0.170	-15	11	15
0.497	-16.5	12	16.5
0.000	-17.82432		17.82432
0.000	-18	13	18
0.000	-19.5	14	19.5




Pressure vs Depth
Direction of Pull
Depth (ft)
Pressure (kips/ft)
Pile #1 - West Pull
August 25, 1997 - 4th 1/2" Test
Pile Defl. = 0.513"
Abut. Defl. = 0.643"
Load = 45.56 kips
Reaction = 32.46 kips

Statics

Date	08/25/97	
Time	7850-7860	sec
Load	45.56	kips
Reaction	32.46	kips
Abut. Defl.	0.643	inches
Pile Defl.	0.513	inches

Load by Statics

Pressure Equation

$y =$ 0.00166710 $x^4 +$ -0.06122980 $x^3 +$ 0.602974560 $x^2 +$ 0.05037594 $x +$ -14.00118632

Roots for pressure equation

$x =$ -4.02563 $x =$ 7.67058 $x =$ 15.25906 $x =$ 17.82432

Integrate to check statics:

From 0 to 7.67058

$=$ -0.000333420 $x^5 +$ 0.015307450 $x^4 +$ -0.200991520 $x^3 +$ -0.025187970 $x^2 +$ 14.00118632 x

$=$ 59.34250859 - 0 $=$ **59.34251 kips**

From 7.67058 to 15.25906

$=$ -0.000333420 $x^5 +$ 0.015307450 $x^4 +$ -0.200991520 $x^3 +$ -0.025187970 $x^2 +$ 14.00118632 x

$=$ 47.73126239 - 59.34250859 $=$ **-11.61125 kips**

From 15.25906 to 17.8243

$=$ -0.000333420 $x^5 +$ 0.015307450 $x^4 +$ -0.200991520 $x^3 +$ -0.025187970 $x^2 +$ 14.00118632 x

$=$ 48.58858612 - 47.73126239 $=$ **0.85732 kips**

From Pressure	**=**	**48.589 kips**
Measured	**=**	**45.56 kips**
% difference	**=**	***6.65*** **%**

Moment by Statics

M_{GRND} = (Reaction)(x) - (Load)(y) - (Weight)(e)

x =	6.85	ft	(distance from centerline of the pile to centerline of the reaction)
y =	4.2083	ft	(distance from centerline of the load to the ground surface)
e =	0.130	in	($e = \Delta_{abut} - \Delta_{pile}$)
Weight =	65.0	kips	

$M_{GRND} =$ 222.351 - 191.732 - 0.704 = **29.915 kip-ft**

Measured = 31.605 kip-ft

% difference = ***5.35*** **%**

Deflections

Date	08/25/97	
Time	7850-7860	sec
Load	45.56	kips
Reaction	32.46	kips
Abut. Defl.	0.643	inches
Pile Defl.	0.513	inches

@ x = 0

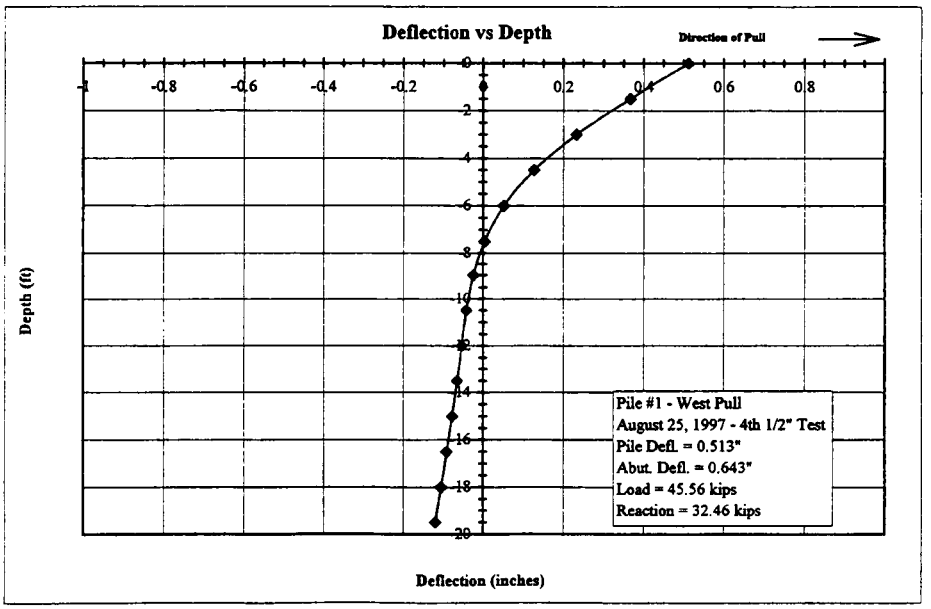
y = 0.000 + 0 C_1 + C_2 = 0.513

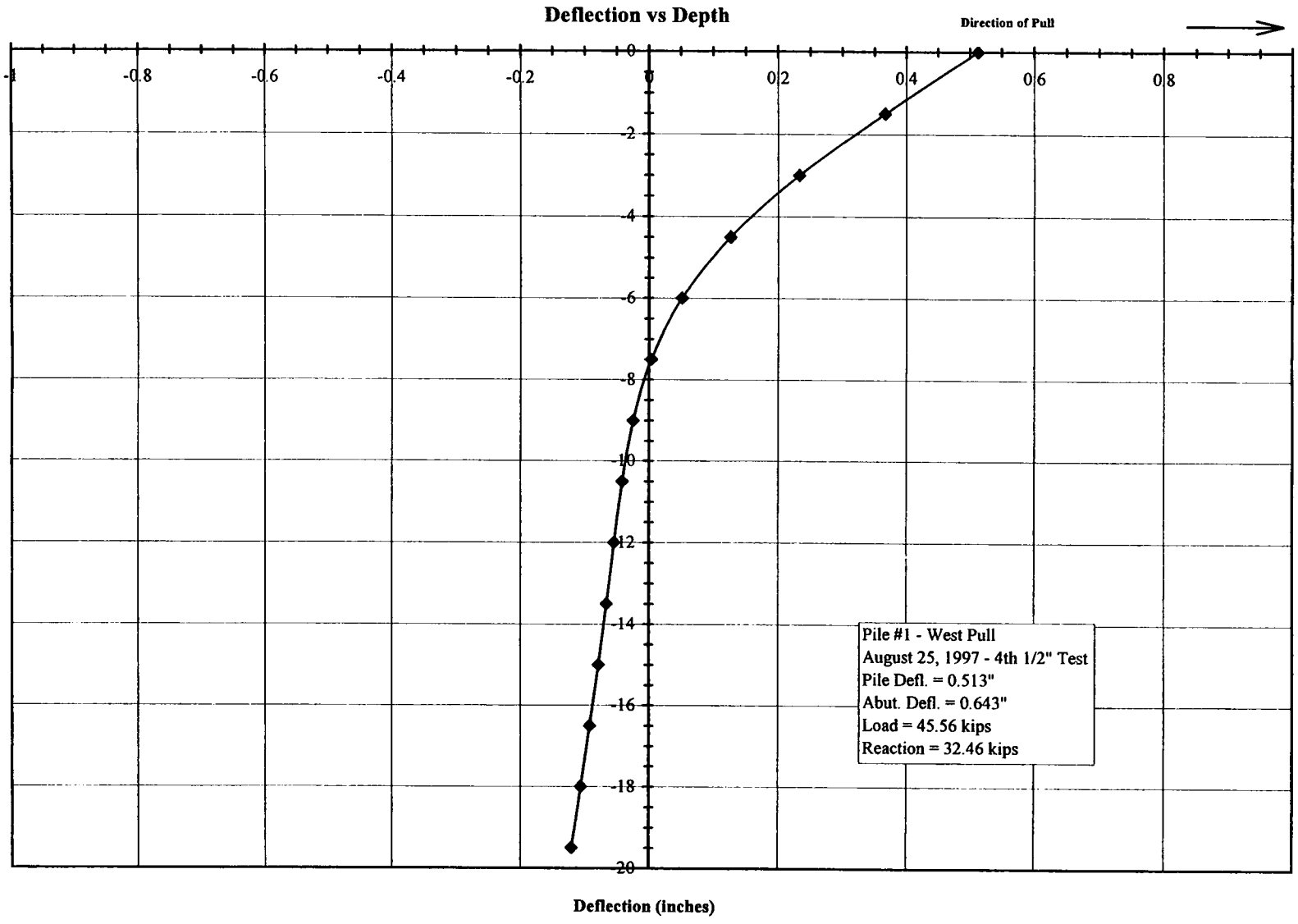
@ x = 7.67058

y = 0.221 + 7.67058 C_1 + 0.513 = 0

C_1 = -0.0956714 = -0.0079726 radians

Deflection (in)	Depth (ft)	Station	Depth (ft)
0.513	0	1	0
0.367	-1.5	2	1.5
0.233	-3	3	3
0.127	-4.5	4	4.5
0.052	-6	5	6
0.004	-7.5	6	7.5
-0.024	-9	7	9
-0.041	-10.5	8	10.5
-0.054	-12	9	12
-0.066	-13.5	10	13.5
-0.078	-15	11	15
-0.092	-16.5	12	16.5
-0.105	-18	13	18
-0.120	-19.5	14	19.5




Deflection vs Depth
Direction of Pull
-1
-0.8
-0.6
-0.4
-0.2
0
0.2
0.4
0.6
0.8
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Deflection (inches)
Pile #1 - West Pull
August 25, 1997 - 4th 1/2" Test
Pile Defl. = 0.513"
Abut. Defl. = 0.643"
Load = 45.56 kips
Reaction = 32.46 kips

APPENDIX J

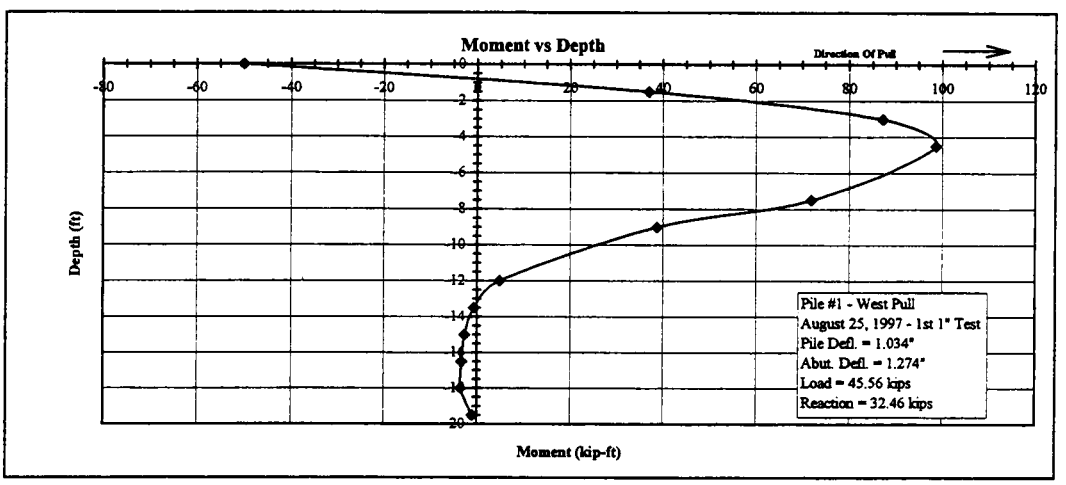
ONE INCH INCREMENT

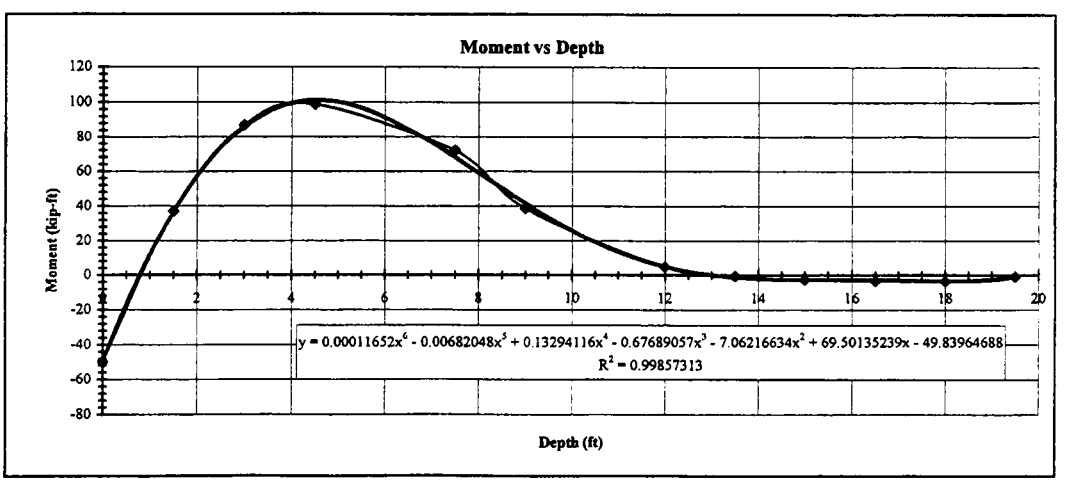
FOR AUGUST 25, 1997

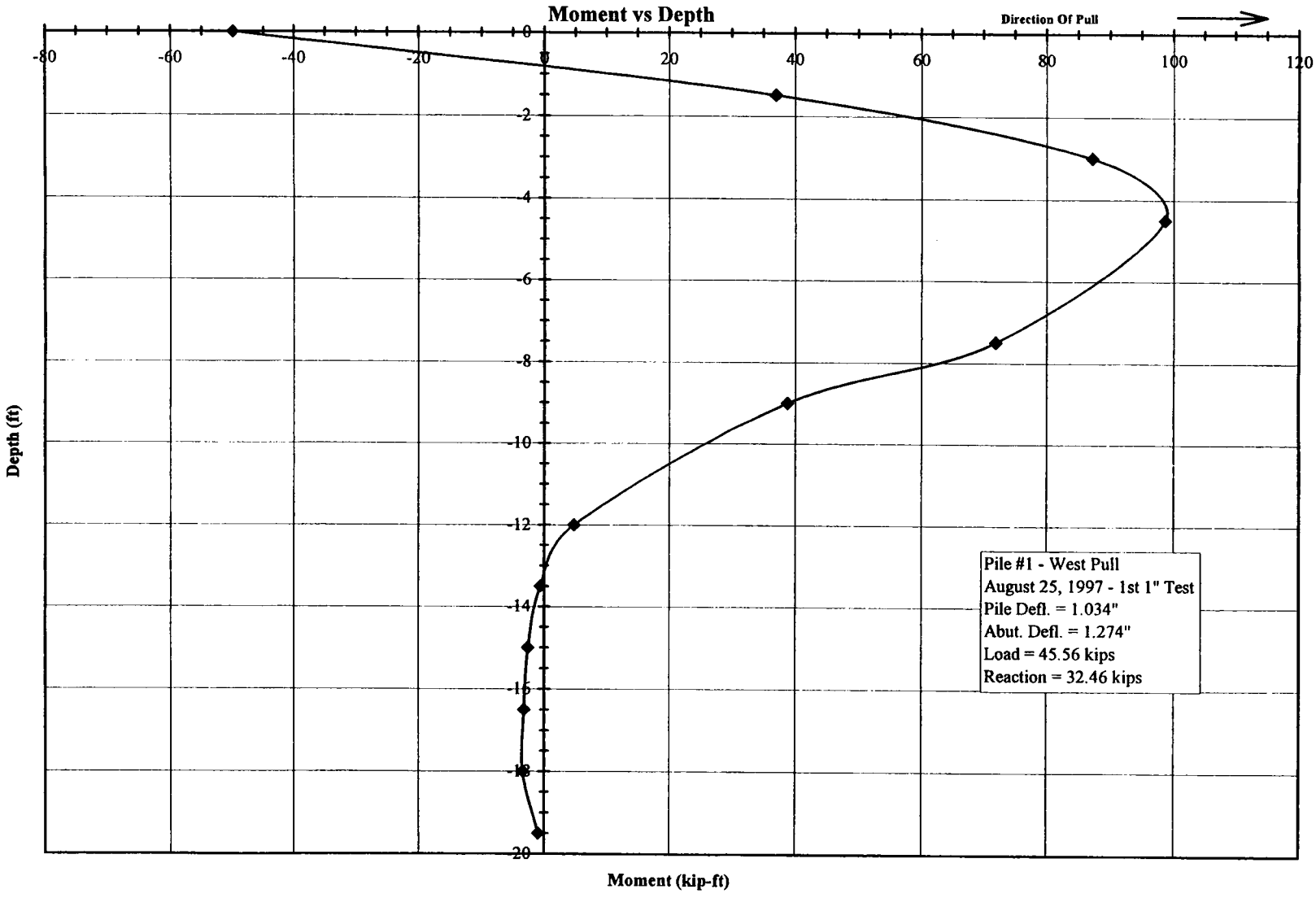
Moments

Date	08/25/97	
Time	14750-14760	sec
Load	71.92	kips
Reaction	50.03	kips
Abut. Defl.	1.274	inches
Pile Defl.	1.034	inches

Moment (kip-ft)	Depth (ft)	Station	Depth (ft)
-49.960	0	1	0
36.976	-1.5	2	1.5
87.166	-3	3	3
98.675	-4.5	4	4.5
71.929	-7.5	6	7.5
38.747	-9	7	9
4.804	-12	9	12
-0.601	-13.5	10	13.5
-2.596	-15	11	15
-3.231	-16.5	12	16.5
-3.420	-18	13	18
-0.948	-19.5	14	19.5






Moment vs Depth
Direction Of Pull
-80
-60
-40
-20
0
20
40
60
80
100
120
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Moment (kip-ft)
Pile #1 - West Pull
August 25, 1997 - 1st 1" Test
Pile Defl. = 1.034"
Abut. Defl. = 1.274"
Load = 45.56 kips
Reaction = 32.46 kips

Equations

Date	08/25/97		
Time	14750-14760	sec	
Load	71.92	kips	
Reaction	50.03	kips	
Abut. Defl.	1.274	inches	
Pile Defl.	1.034	inches	$= C_2$

Modulus of Elasticity (E) =	33000	ksi
Moment of Inertia (I) =	223	in^4

Deflection Equation:
From 0' to 19.5'

$$y = 4.88582E\text{-}10\ x^8 + -3.81321E\text{-}08\ x^7 + 1.04055E\text{-}06\ x^6 + -7.94719E\text{-}06\ x^5 + -0.000138192\ x^4 + 0.002719988\ x^3 + -0.005851536\ x^2 + (C_1)\ x + 1.034\ (C_2)$$

Slope Equation:
From 0' to 19.5'

$$y = 3.25721E\text{-}10\ x^7 + -2.22437E\text{-}08\ x^6 + 5.20275E\text{-}07\ x^5 + -3.31133E\text{-}06\ x^4 + -4.60639E\text{-}05\ x^3 + 0.000679997\ x^2 + -0.000975256\ x + (C_1)$$

Moment Equation:
From 0' to 19.5'

$$\mathbf{y = 0.00011652\ x^6 + -0.00682048\ x^5 + 0.13294116\ x^4 + -0.67689057\ x^3 + -7.06216634\ x^2 + 69.50135239\ x + -49.83964688}$$

Shear Equation:
From 0' to 19.5'

$$y = 0.00069912\ x^5 + -0.03410240\ x^4 + 0.53176464\ x^3 + -2.03067171\ x^2 + -14.12433268\ x + 69.50135239$$

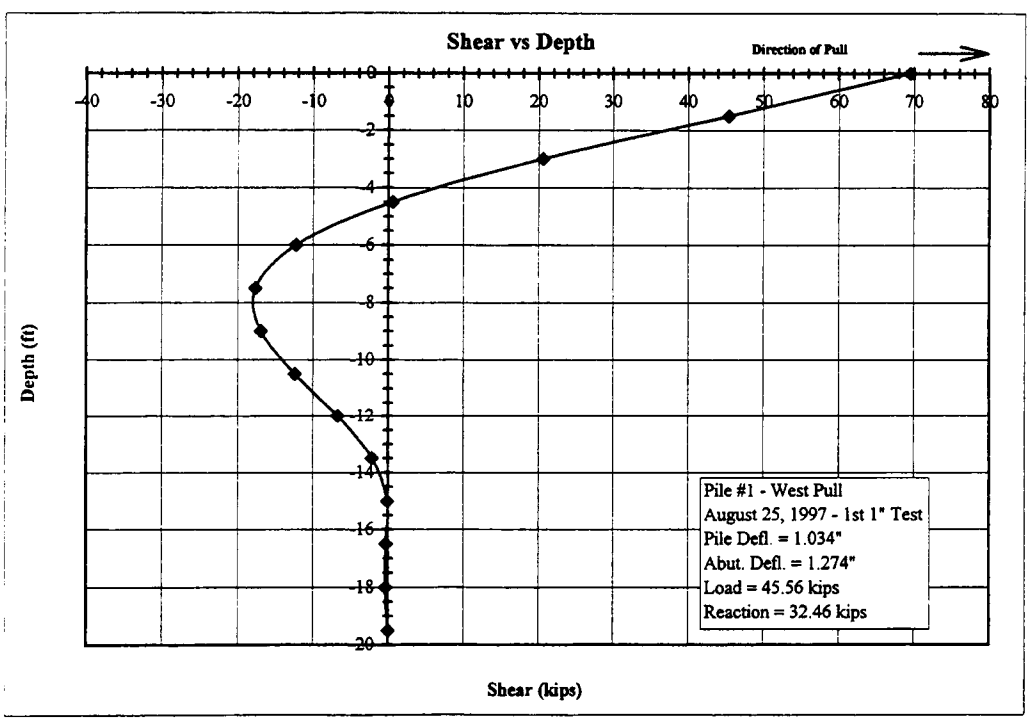
Pressure Equation:
From 0' to 19.5'

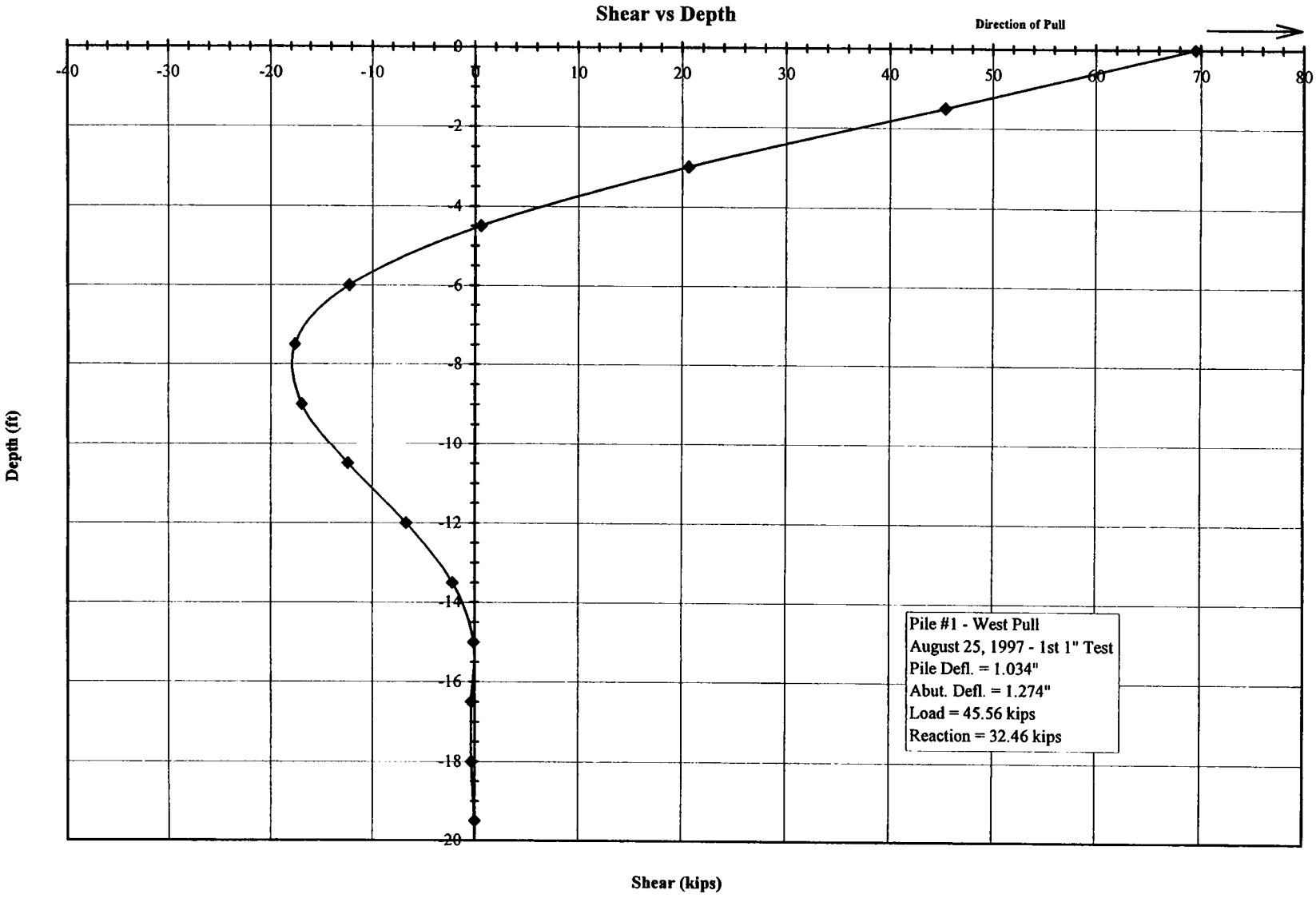
$$y = 0.0034956\ x^4 + -0.1364096\ x^3 + 1.59529392\ x^2 + -4.06134342\ x + -14.12433268$$

Shear

Date	08/25/97	
Time	14750-14760	sec
Load	71.92	kips
Reaction	50.03	kips
Abut. Defl.	1.274	inches
Pile Defl.	1.034	inches

Shear (kips)	Depth (ft)	Station	Depth (ft)
69.501	0	1	0
45.373	-1.5	2	1.5
20.618	-3	3	3
0.584	-4.5	4	4.5
-12.248	-6	5	6
-17.630	-7.5	6	7.5
-16.909	-9	7	9
-12.391	-10.5	8	10.5
-6.702	-12	9	12
-2.152	-13.5	10	13.5
-0.099	-15	11	15
-0.309	-16.5	12	16.5
-0.322	-18	13	18
0.000	-19.5	14	19.5

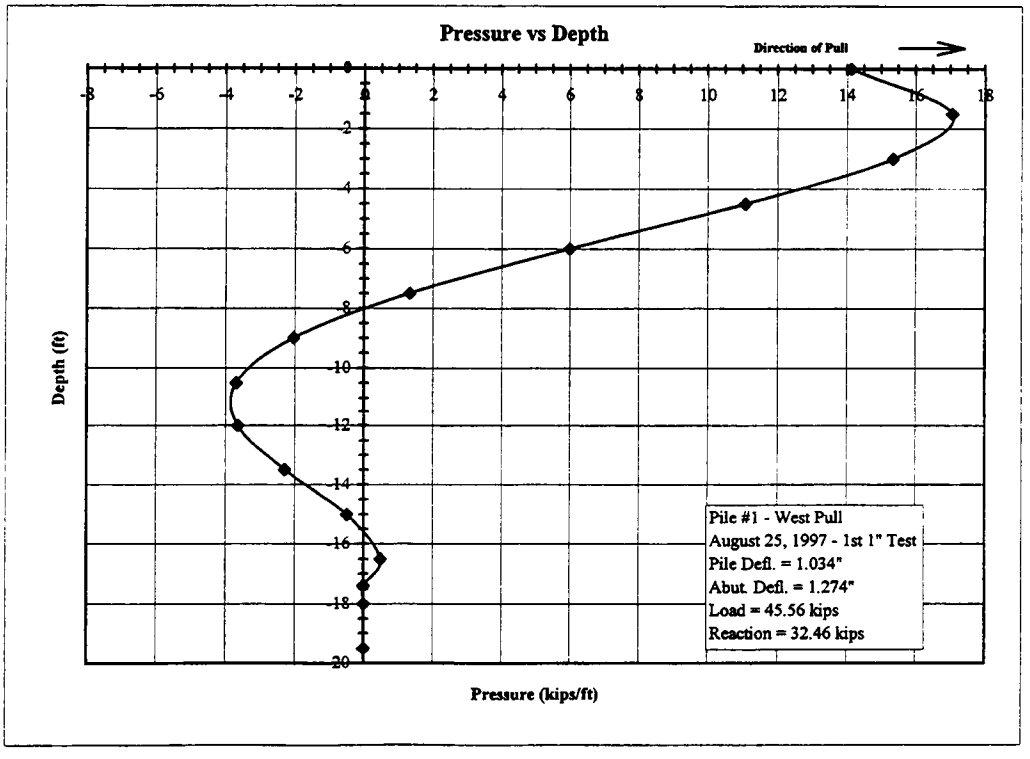


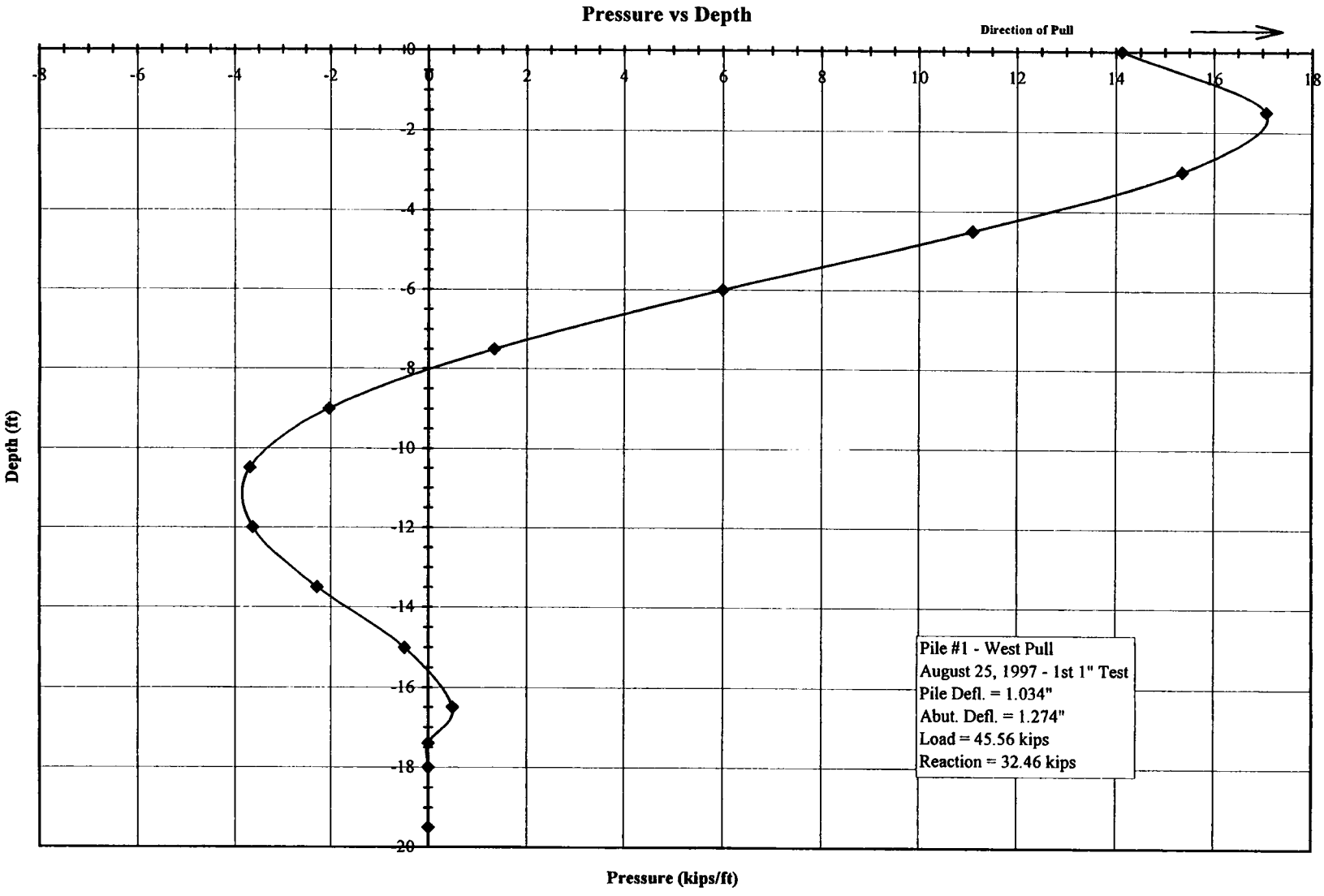

Shear vs Depth
Direction of Pull
-40
-30
-20
-10
0
10
20
30
40
50
60
70
80
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Shear (kips)
Pile #1 - West Pull
August 25, 1997 - 1st 1" Test
Pile Defl. = 1.034"
Abut. Defl. = 1.274"
Load = 45.56 kips
Reaction = 32.46 kips

Pressure

Date	08/25/97	
Time	14750-14760	sec
Load	71.92	kips
Reaction	50.03	kips
Abut. Defl.	1.274	inches
Pile Defl.	1.034	inches

Pressure (kips/ft)	Depth (ft)	Station	Depth (ft)
14.124	0	1	0
17.070	-1.5	2	1.5
15.351	-3	3	3
11.093	-4.5	4	4.5
5.996	-6	5	6
1.337	-7.5	6	7.5
-2.034	-9	7	9
-3.691	-10.5	8	10.5
-3.631	-12	9	12
-2.278	-13.5	10	13.5
-0.479	-15	11	15
0.493	-16.5	12	16.5
0.000	-17.39192		17.39192
0.000	-18	13	18
0.000	-19.5	14	19.5




Pressure vs Depth
Direction of Pull
Depth (ft)
Pressure (kips/ft)
-8
-6
-4
-2
0
2
4
6
8
10
12
14
16
18
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Pile #1 - West Pull
August 25, 1997 - 1st 1" Test
Pile Defl. = 1.034"
Abut. Defl. = 1.274"
Load = 45.56 kips
Reaction = 32.46 kips

Statics

Date	08/25/97	
Time	14750-14760	sec
Load	71.92	kips
Reaction	50.03	kips
Abut. Defl.	1.274	inches
Pile Defl.	1.034	inches

Load by Statics

Pressure Equation

$y = 0.00349560\ x^4 + -0.13640960\ x^3 + 1.595293920\ x^2 + -4.06134342\ x + -14.12433268$

Roots for pressure equation

x = -1.87141 x = 8.01649 x = 15.48623 x = 17.39192

Integrate to check statics:

From 0 to 8.01649

= -0.000699120 x^5 + 0.034102400 x^4 + -0.531764640 x^3 + 2.030671710 x^2 + 14.12433268 x

= 87.46915104 - 0 = **87.46915 kips**

From 8.01649 to 15.48623

= -0.000699120 x^5 + 0.034102400 x^4 + -0.531764640 x^3 + 2.030671710 x^2 + 14.12433268 x

= 69.48818736 - 87.46915104 = **-17.98096 kips**

From 15.48623 to 17.3919

= -0.000699120 x^5 + 0.034102400 x^4 + -0.531764640 x^3 + 2.030671710 x^2 + 14.12433268 x

= 70.11075288 - 69.48818736 = **0.62257 kips**

From Pressure	=	**70.111 kips**
Measured	=	**71.92 kips**
% difference	=	***2.52*** **%**

Moment by Statics

M_{GRND} = (Reaction)(x) - (Load)(y) - (Weight)(e)

x =	7	ft	(distance from centerline of the pile to centerline of the reaction)
y =	4.2083	ft	(distance from centerline of the load to the ground surface)
e =	0.240	in	($e = \Delta_{abut} - \Delta_{pile}$)
Weight =	65.0	kips	

M_{GRND} = 350.210 - 302.663 - 1.300 = **46.247 kip-ft**

Measured = 49.96 kip-ft

% difference = *7.43* %

Deflections

Date	08/25/97	
Time	14750-14760	sec
Load	71.92	kips
Reaction	50.03	kips
Abut. Defl.	1.274	inches
Pile Defl.	1.034	inches

@ x = 0

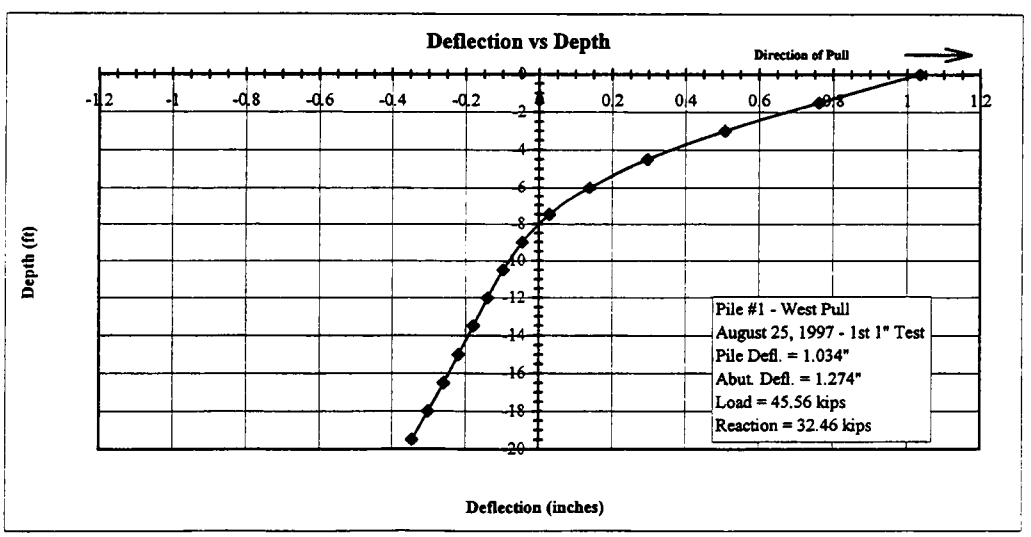
y = 0.000 + 0 C_1 + C_2 = 1.034

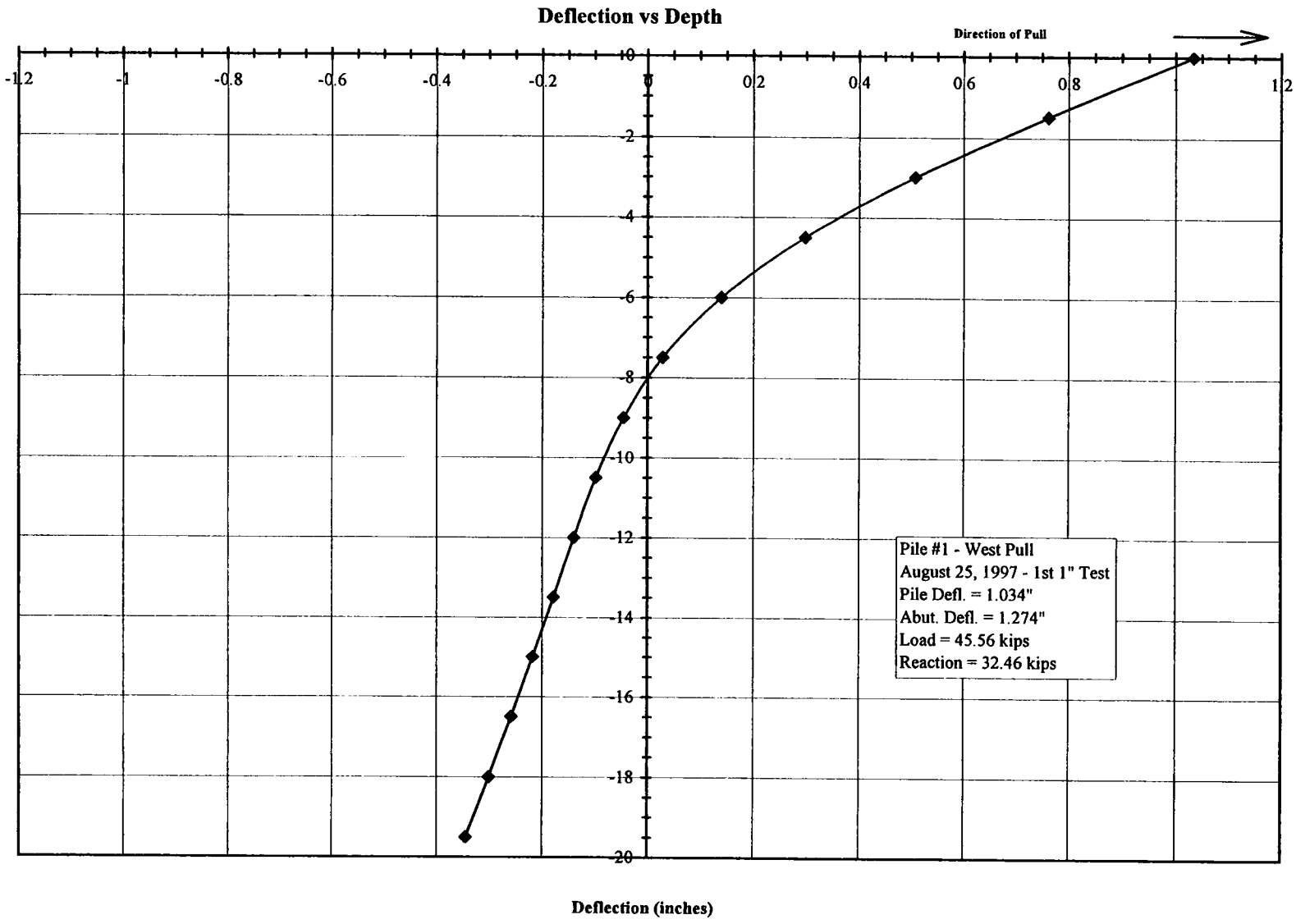
@ x = 8.01649

y = 0.395 + 8.01649 C_1 + 1.034 = 0

C_1 = -0.1782281 = -0.0148523 radians

Deflection (in)	Depth (ft)	Station	Depth (ft)
1.034	0	1	0
0.762	-1.5	2	1.5
0.508	-3	3	3
0.297	-4.5	4	4.5
0.139	-6	5	6
0.029	-7.5	6	7.5
-0.045	-9	7	9
-0.098	-10.5	8	10.5
-0.139	-12	9	12
-0.178	-13.5	10	13.5
-0.217	-15	11	15
-0.258	-16.5	12	16.5
-0.300	-18	13	18
-0.344	-19.5	14	19.5




Deflection vs Depth
Direction of Pull
-1.2
-1
-0.8
-0.6
-0.4
-0.2
0
0.2
0.4
0.6
0.8
1
1.2
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Deflection (inches)
Pile #1 - West Pull
August 25, 1997 - 1st 1" Test
Pile Defl. = 1.034"
Abut. Defl. = 1.274"
Load = 45.56 kips
Reaction = 32.46 kips

APPENDIX K

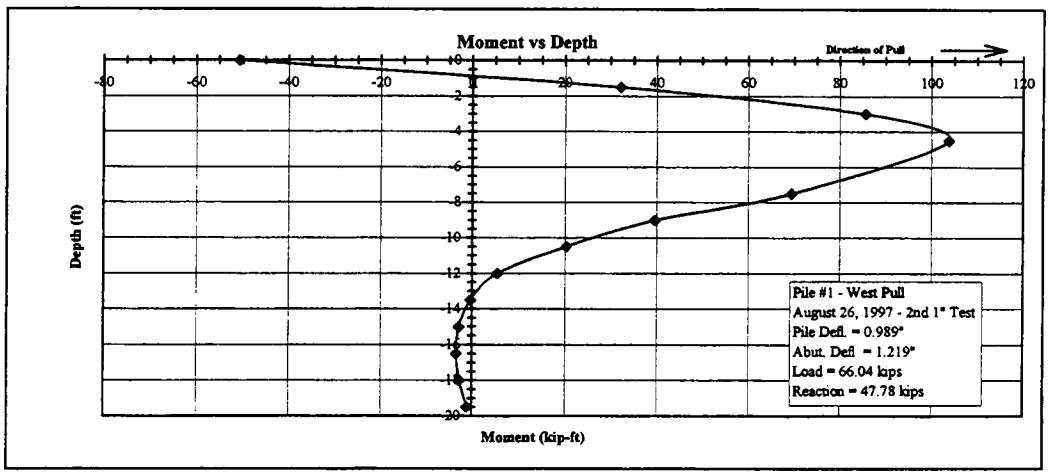
ONE INCH INCREMENT

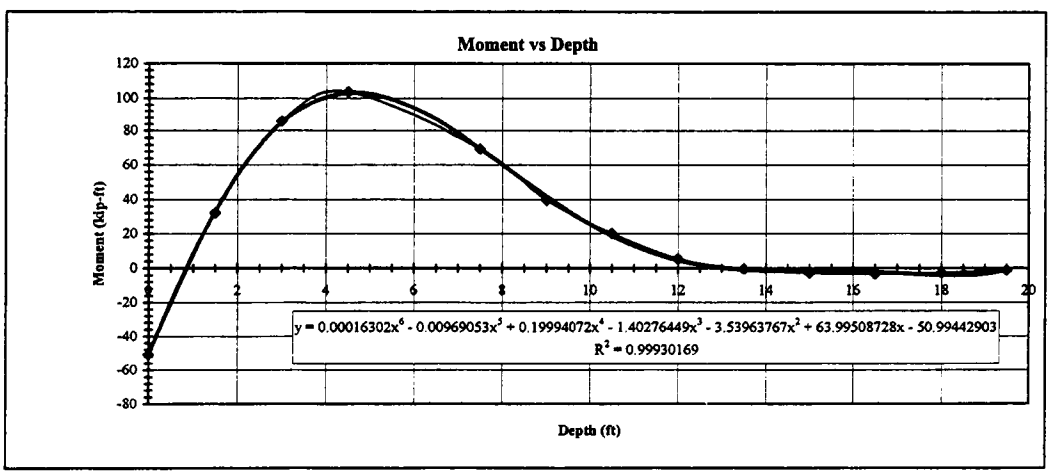
FOR AUGUST 26, 1997

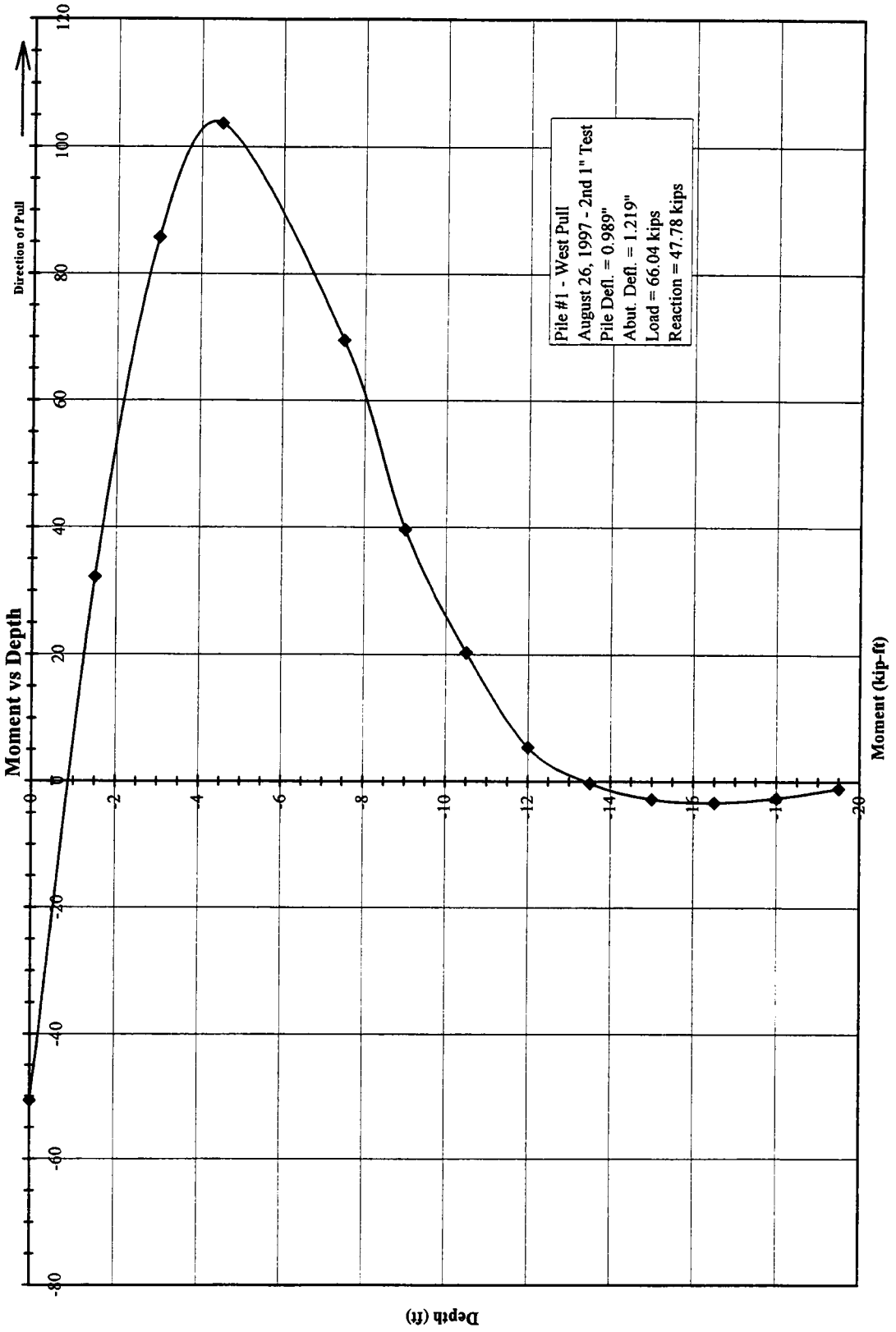
Moments

Date	08/26/97	
Time	13930-13940	sec
Load	66.04	kips
Reaction	47.78	kips
Abut. Defl.	1.219	inches
Pile Defl.	0.989	inches

Moment (kip-ft)	Depth (ft)	Station	Depth (ft)
-50.655	0	1	0
32.187	-1.5	2	1.5
85.730	-3	3	3
103.767	-4.5	4	4.5
69.545	-7.5	6	7.5
39.667	-9	7	9
20.334	-10.5	8	10.5
5.423	-12	9	12
-0.282	-13.5	10	13.5
-2.805	-15	11	15
-3.344	-16.5	12	16.5
-2.637	-18	13	18
-1.085	-19.5	14	19.5






Moment vs Depth
Direction of Pull
Pile #1 - West Pull
August 26, 1997 - 2nd 1" Test
Pile Defl. = 0.989"
Abut. Defl. = 1.219"
Load = 66.04 kips
Reaction = 47.78 kips
Moment (kip-ft)
Depth (ft)
120
100
80
60
40
20
0
-20
-40
-60
-80
-0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20

Equations

Date	08/26/97	
Time	13930-13940	sec
Load	66.04	kips
Reaction	47.78	kips
Abut. Defl.	1.219	inches
Pile Defl.	0.989	inches $= C_2$

Modulus of Elasticity (E) =	33000	ksi
Moment of Inertia (I) =	223	in^4

Deflection Equation:
From 0' to 19.5'

$$y = 6.83562E\text{-}10\ x^8 + -5.4178E\text{-}08\ x^7 + 1.56497E\text{-}06\ x^6 + -1.64695E\text{-}05\ x^5 + -6.92632E\text{-}05\ x^4 + 0.002504496\ x^3 + -0.005987116\ x^2 + (C_1)\ x + 0.989\ (C_2)$$

Slope Equation:
From 0' to 19.5'

$$y = 4.55708E\text{-}10\ x^7 + -3.16038E\text{-}08\ x^6 + 7.82483E\text{-}07\ x^5 + -6.86228E\text{-}06\ x^4 + -2.30877E\text{-}05\ x^3 + 0.000626124\ x^2 + -0.000997853\ x + (C_1)$$

Moment Equation:
From 0' to 19.5'

$$\mathbf{y = 0.00016302\ x^6 + -0.00969053\ x^5 + 0.19994072\ x^4 + -1.40276449\ x^3 + -3.53963767\ x^2 + 63.99508728\ x + -50.99442903}$$

Shear Equation:
From 0' to 19.5'

$$y = 0.00097812\ x^5 + -0.04845265\ x^4 + 0.79976288\ x^3 + -4.20829347\ x^2 + -7.07927534\ x + 63.99508728$$

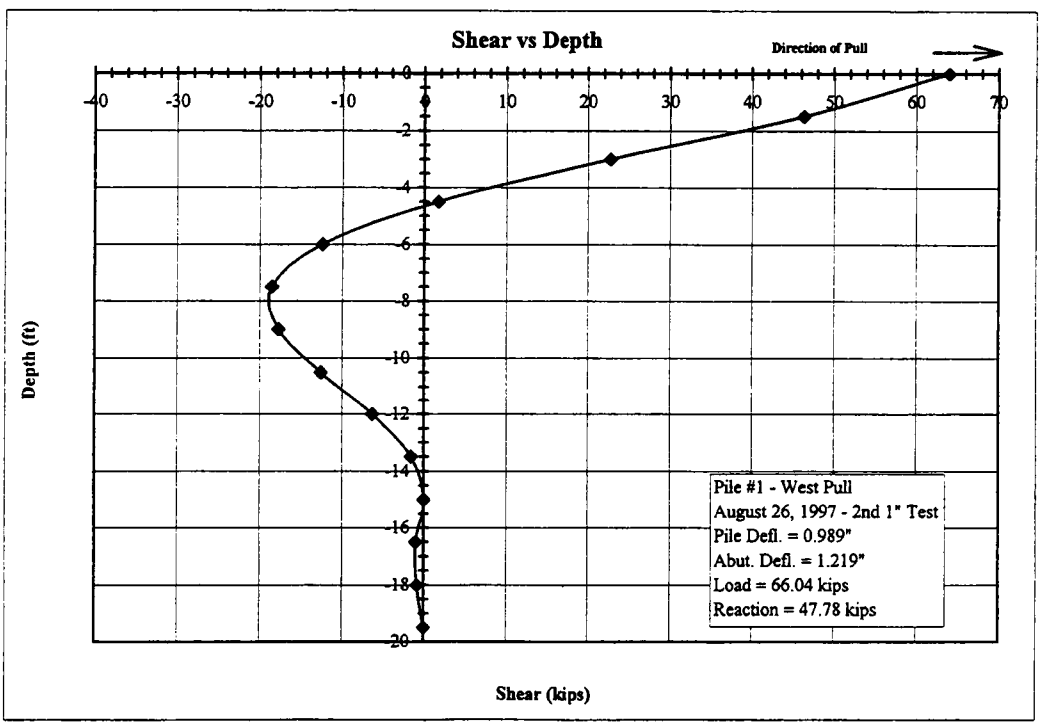
Pressure Equation:
From 0' to 19.5'

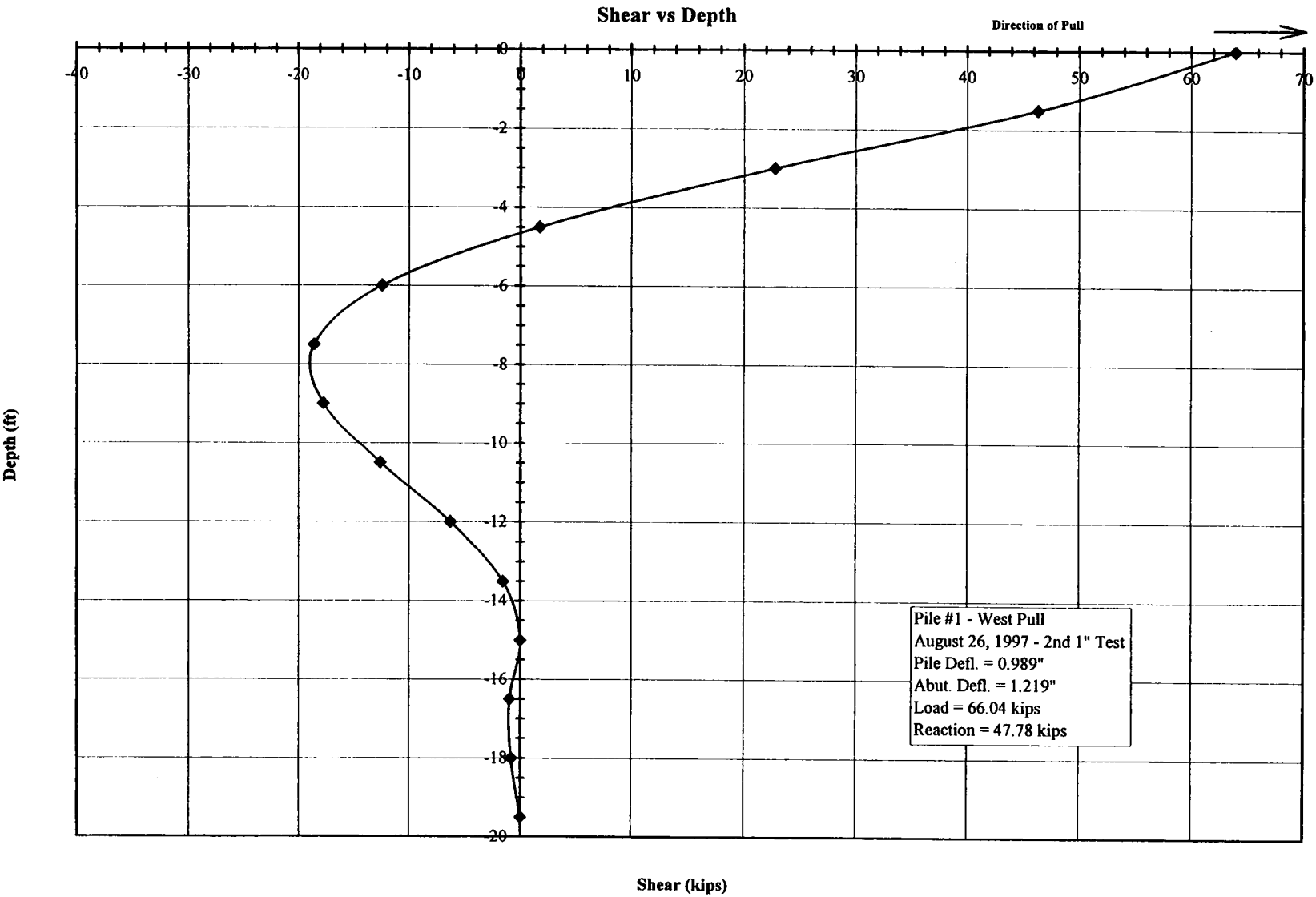
$$y = 0.0048906\ x^4 + -0.1938106\ x^3 + 2.39928864\ x^2 + -8.41658694\ x + -7.07927534$$

Shear

Date	08/26/97	
Time	13930-13940	sec
Load	66.04	kips
Reaction	47.78	kips
Abut. Defl.	1.219	inches
Pile Defl.	0.989	inches

Shear (kips)	Depth (ft)	Station	Depth (ft)
63.995	0	1	0
46.369	-1.5	2	1.5
22.789	-3	3	3
1.735	-4.5	4	4.5
-12.419	-6	5	6
-18.512	-7.5	6	7.5
-17.704	-9	7	9
-12.585	-10.5	8	10.5
-6.287	-12	9	12
-1.586	-13.5	10	13.5
-0.016	-15	11	15
-0.977	-16.5	12	16.5
-0.843	-18	13	18
0.000	-19.5	14	19.5

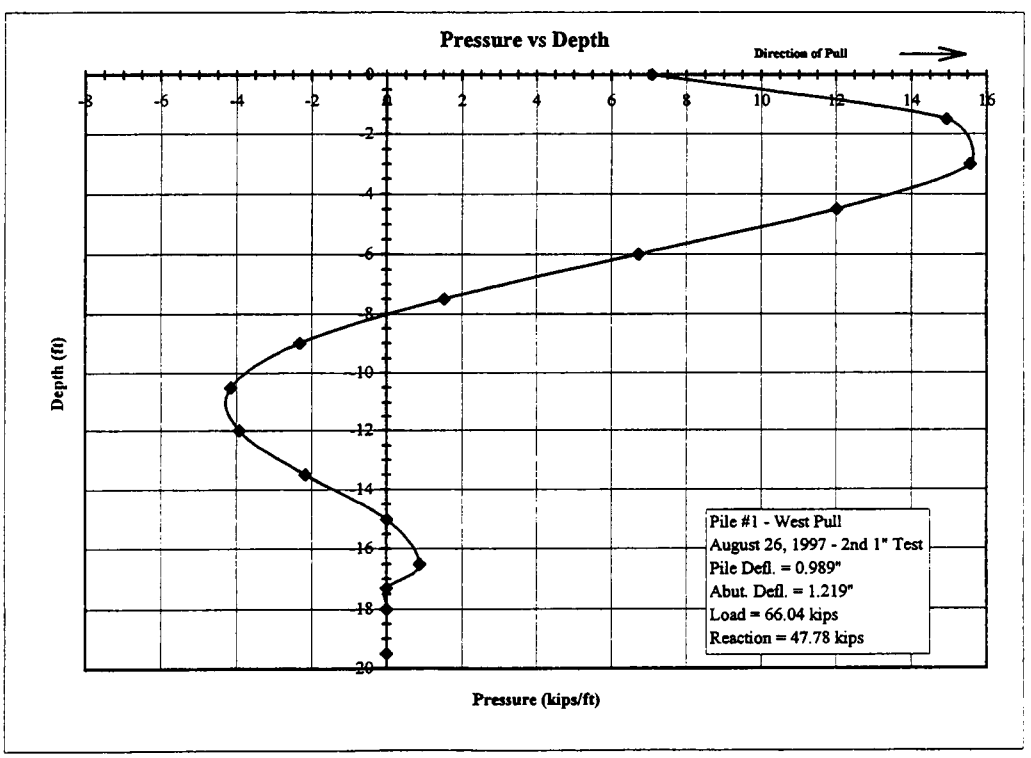


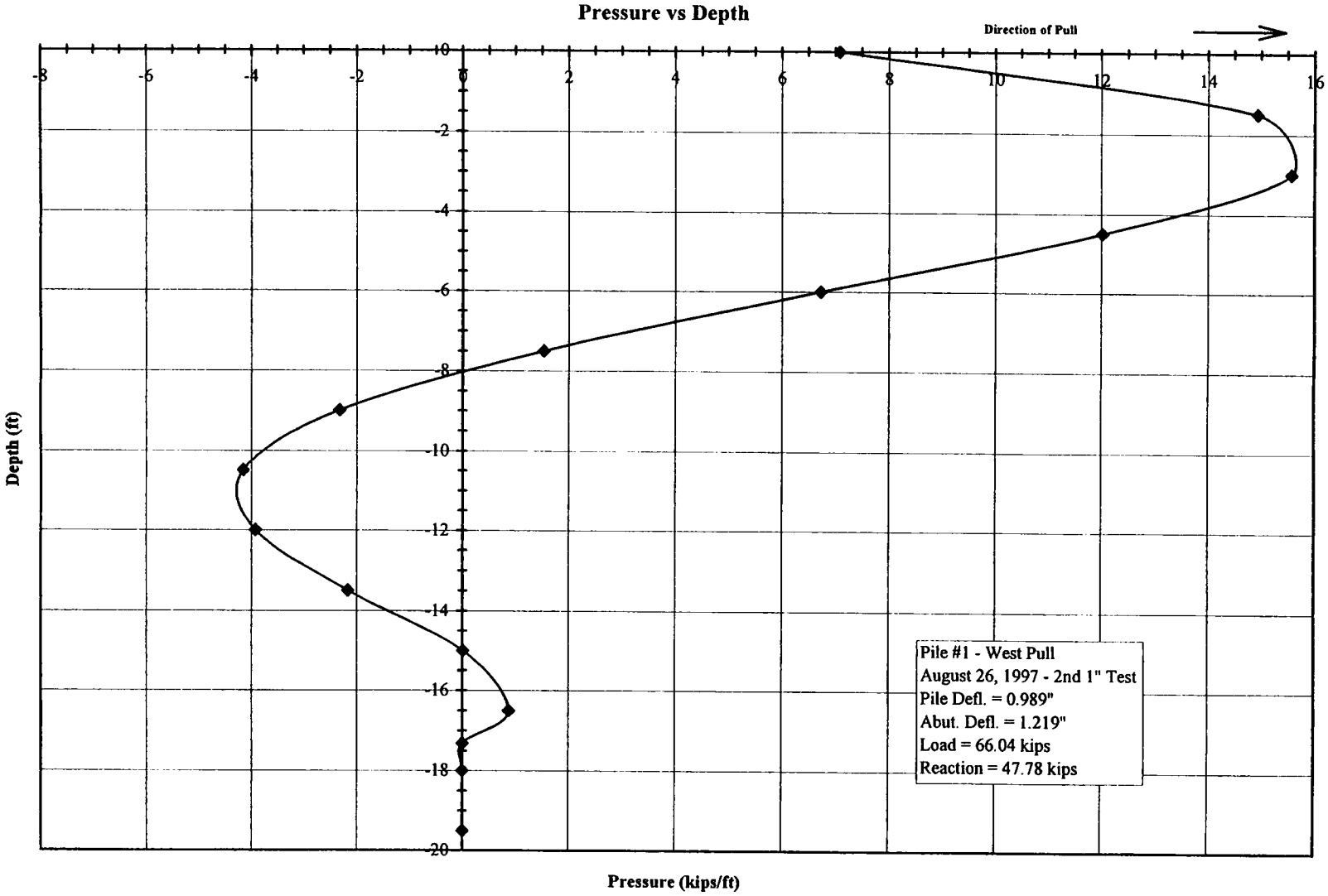

Shear vs Depth
Direction of Pull
Depth (ft)
Shear (kips)
-40
-30
-20
-10
0
10
20
30
40
50
60
70
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Pile #1 - West Pull
August 26, 1997 - 2nd 1" Test
Pile Defl. = 0.989"
Abut. Defl. = 1.219"
Load = 66.04 kips
Reaction = 47.78 kips

Pressure

Date	08/26/97	
Time	13930-13940	sec
Load	66.04	kips
Reaction	47.78	kips
Abut. Defl.	1.219	inches
Pile Defl.	0.989	inches

Pressure (kips/ft)	Depth (ft)	Station	Depth (ft)
7.079	0	1	0
14.935	-1.5	2	1.5
15.572	-3	3	3
12.024	-4.5	4	4.5
6.729	-6	5	6
1.533	-7.5	6	7.5
-2.313	-9	7	9
-4.154	-10.5	8	10.5
-3.926	-12	9	12
-2.162	-13.5	10	13.5
0.012	-15	11	15
0.876	-16.5	12	16.5
0.000	-17.31405		17.31405
0.000	-18	13	18
0.000	-19.5	14	19.5




Pressure vs Depth
Direction of Pull
Depth (ft)
Pressure (kips/ft)
-8
-6
-4
-2
0
2
4
6
8
10
12
14
16
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Pile #1 - West Pull
August 26, 1997 - 2nd 1" Test
Pile Defl. = 0.989"
Abut. Defl. = 1.219"
Load = 66.04 kips
Reaction = 47.78 kips

Statics

Date	08/26/97	
Time	13930-13940	sec
Load	66.04	kips
Reaction	47.78	kips
Abut. Defl.	1.219	inches
Pile Defl.	0.989	inches

Load by Statics

Pressure Equation

$y =$ 0.00489060 $x^4 +$ -0.19381060 $x^3 +$ 2.399288640 $x^2 +$ -8.41658694 $x +$ -7.07927534

Roots for pressure equation

x = -0.69538 x = 8.02045 x = 14.99009 x = 17.31405

Integrate to check statics:

From 0 to 8.02045

= -0.000978120 $x^5 +$ 0.048452650 $x^4 +$ -0.799762880 $x^3 +$ 4.208293470 $x^2 +$ 7.07927534 x

= 82.89798704 - 0 = **82.89799 kips**

From 8.02045 to 14.99009

= -0.000978120 $x^5 +$ 0.048452650 $x^4 +$ -0.799762880 $x^3 +$ 4.208293470 $x^2 +$ 7.07927534 x

= 64.01091118 - 82.89798704 = **-18.88708 kips**

From 14.99009 to 17.3141

= -0.000978120 $x^5 +$ 0.048452650 $x^4 +$ -0.799762880 $x^3 +$ 4.208293470 $x^2 +$ 7.07927534 x

= 65.41521768 - 64.01091118 = **1.40431 kips**

From Pressure	=	**65.415 kips**
Measured	=	**66.04 kips**
% difference	=	***0.95*** **%**

Moment by Statics

M_{GRND} = (Reaction)(x) - (Load)(y) - (Weight)(e)

x =	6.85	ft	(distance from centerline of the pile to centerline of the reaction)
y =	4.2083	ft	(distance from centerline of the load to the ground surface)
e =	0.230	in	($e = \Delta_{abut} - \Delta_{pile}$)
Weight =	65.0	kips	

M_{GRND} = 327.293 - 277.918 - 1.246 = **48.129 kip-ft**

Measured = 50.655 kip-ft

% difference = *4.99* %

Deflections

Date	08/26/97	
Time	13930-13940	sec
Load	66.04	kips
Reaction	47.78	kips
Abut. Defl.	1.219	inches
Pile Defl.	0.989	inches

@ x = 0

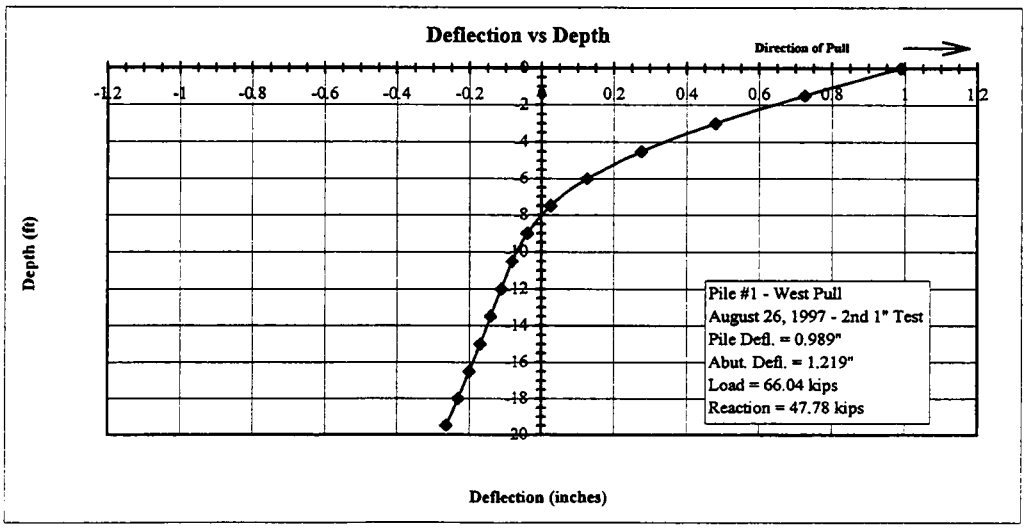
y = 0.000 + 0 C_1 + C_2 = 0.989

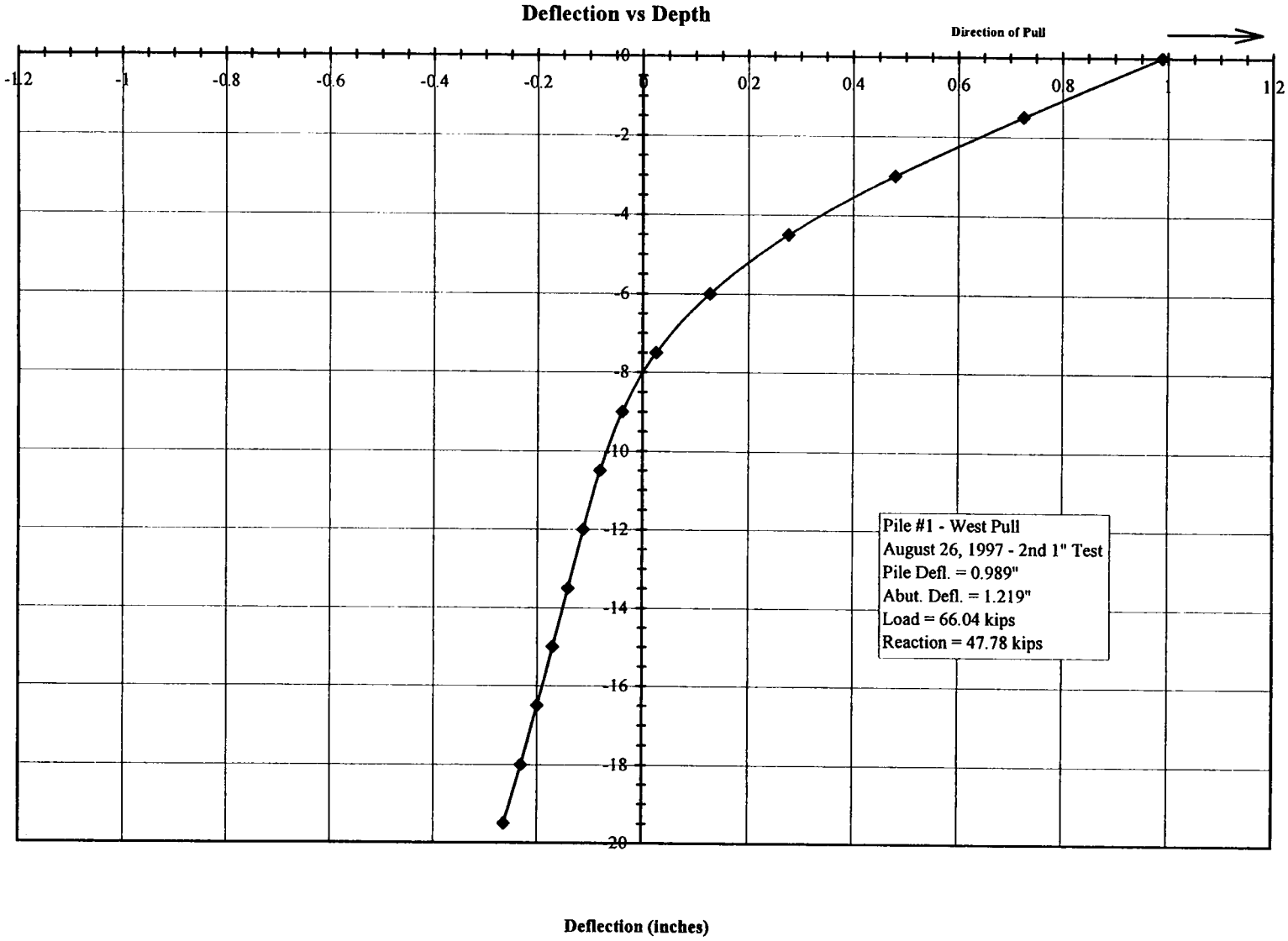
@ x = 8.02045

y = 0.386 + 8.02045 C_1 + 0.989 = 0

C_1 = -0.1714892 = -0.0142908 radians

Deflection (in)	Depth (ft)	Station	Depth (ft)
0.989	0	1	0
0.726	-1.5	2	1.5
0.480	-3	3	3
0.277	-4.5	4	4.5
0.127	-6	5	6
0.026	-7.5	6	7.5
-0.039	-9	7	9
-0.080	-10.5	8	10.5
-0.112	-12	9	12
-0.141	-13.5	10	13.5
-0.170	-15	11	15
-0.199	-16.5	12	16.5
-0.231	-18	13	18
-0.264	-19.5	14	19.5




Deflection vs Depth
Direction of Pull
-1.2
-1
-0.8
-0.6
-0.4
-0.2
0
0.2
0.4
0.6
0.8
1
1.2
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Pile #1 - West Pull
August 26, 1997 - 2nd 1" Test
Pile Defl. = 0.989"
Abut. Defl. = 1.219"
Load = 66.04 kips
Reaction = 47.78 kips
Depth (ft)
Deflection (inches)

APPENDIX L

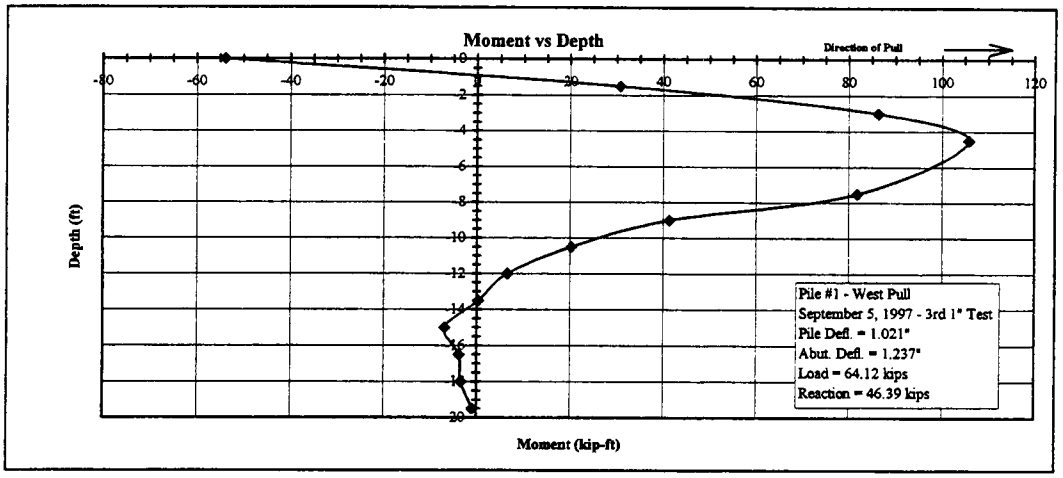
ONE INCH INCREMENT

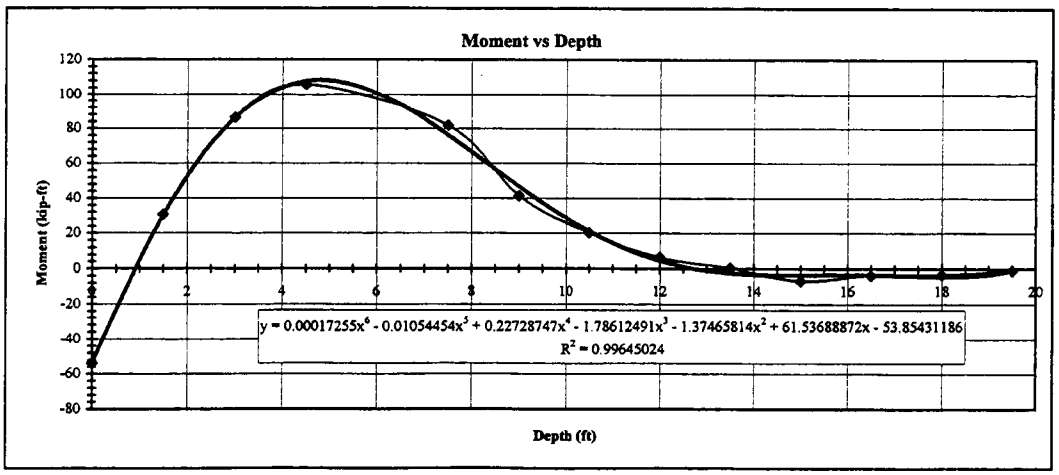
FOR SEPTEMBER 5, 1997

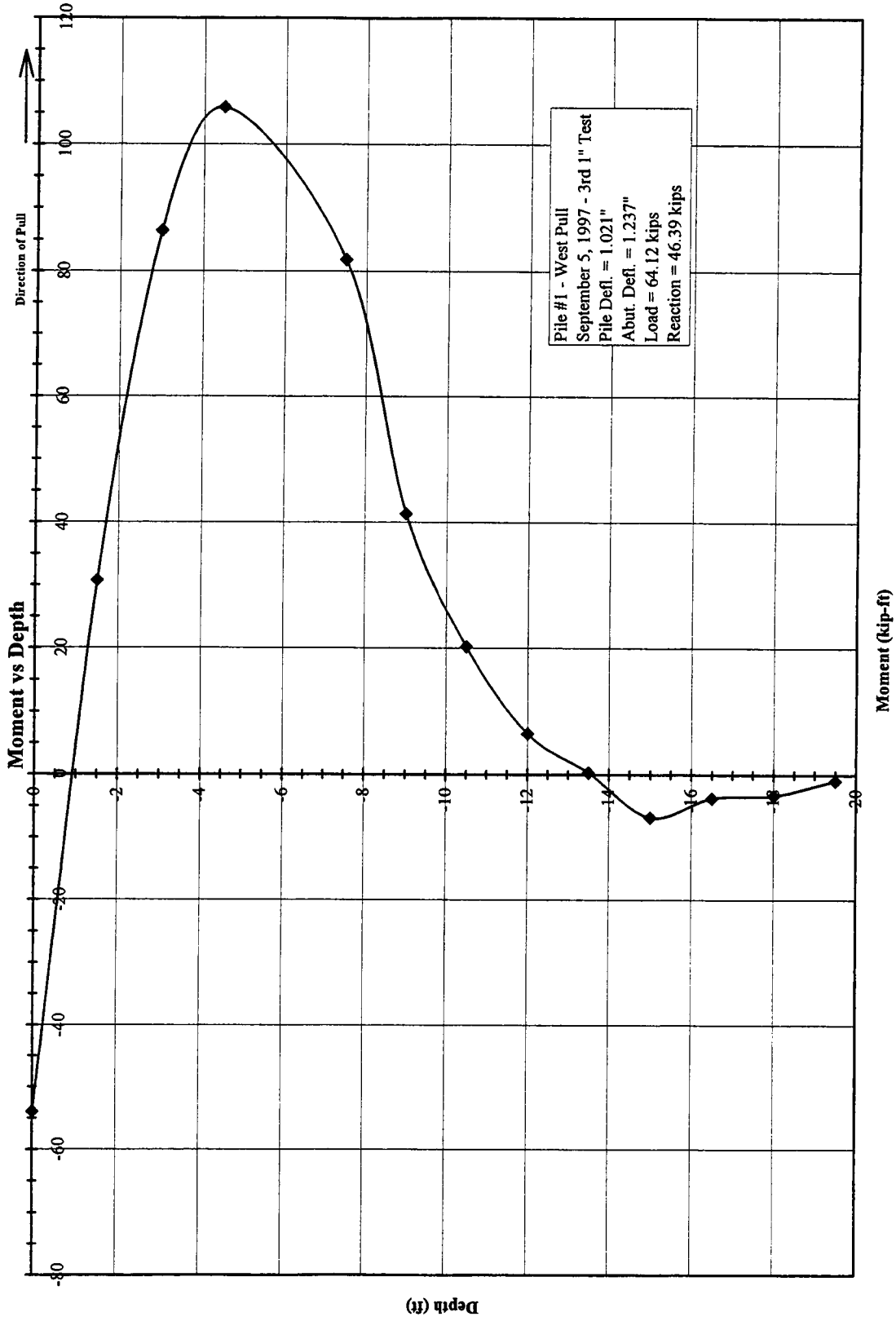
Moments

Date	09/05/97	
Time	16020-16030	sec
Load	64.12	kips
Reaction	46.39	kips
Abut. Defl.	1.237	inches
Pile Defl.	1.021	inches

Moment (kip-ft)	Depth (ft)	Station	Depth (ft)
-53.958	0	1	0
30.736	-1.5	2	1.5
86.398	-3	3	3
105.895	-4.5	4	4.5
81.898	-7.5	6	7.5
41.377	-9	7	9
20.275	-10.5	8	10.5
6.534	-12	9	12
0.371	-13.5	10	13.5
-6.798	-15	11	15
-3.732	-16.5	12	16.5
-3.286	-18	13	18
-0.893	-19.5	14	19.5






Moment vs Depth
Direction of Pull
Pile #1 - West Pull
September 5, 1997 - 3rd 1" Test
Pile Defl. = 1.021"
Abut. Defl. = 1.237"
Load = 64.12 kips
Reaction = 46.39 kips
Moment (kip-ft)
Depth (ft)
120
100
80
60
40
20
0
-20
-40
-60
-80
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20

Equations

Date	09/05/97	
Time	16020-16030	sec
Load	64.12	kips
Reaction	46.39	kips
Abut. Defl.	1.237	inches
Pile Defl.	1.021	inches $= C_2$
Modulus of Elasticity (E) =	33000	ksi
Moment of Inertia (I) =	223	in^4

Deflection Equation:
From 0' to 19.5'

$$y = 7.23522E\text{-}10\,x^8 + -5.89526E\text{-}08\,x^7 + 1.77901E\text{-}06\,x^6 + -2.09704E\text{-}05\,x^5 + -2.68991E\text{-}05\,x^4 + 0.002408292\,x^3 + -0.006322887\,x^2 + (C_1)\,x + 1.021\,(C_2)$$

Slope Equation:
From 0' to 19.5'

$$y = 4.82348E\text{-}10\,x^7 + -3.4389E\text{-}08\,x^6 + 8.89507E\text{-}07\,x^5 + -8.73767E\text{-}06\,x^4 + -8.96638E\text{-}06\,x^3 + 0.000602073\,x^2 + -0.001053815\,x + (C_1)$$

Moment Equation:
From 0' to 19.5'

$$\mathbf{y = 0.00017255\,x^6 + -0.01054454\,x^5 + 0.22728747\,x^4 + -1.78612491\,x^3 + -1.37465814\,x^2 + 61.53688872\,x + -53.85431186}$$

Shear Equation:
From 0' to 19.5'

$$y = 0.00103530\,x^5 + -0.05272270\,x^4 + 0.90914988\,x^3 + -5.35837473\,x^2 + -2.74931628\,x + 61.53688872$$

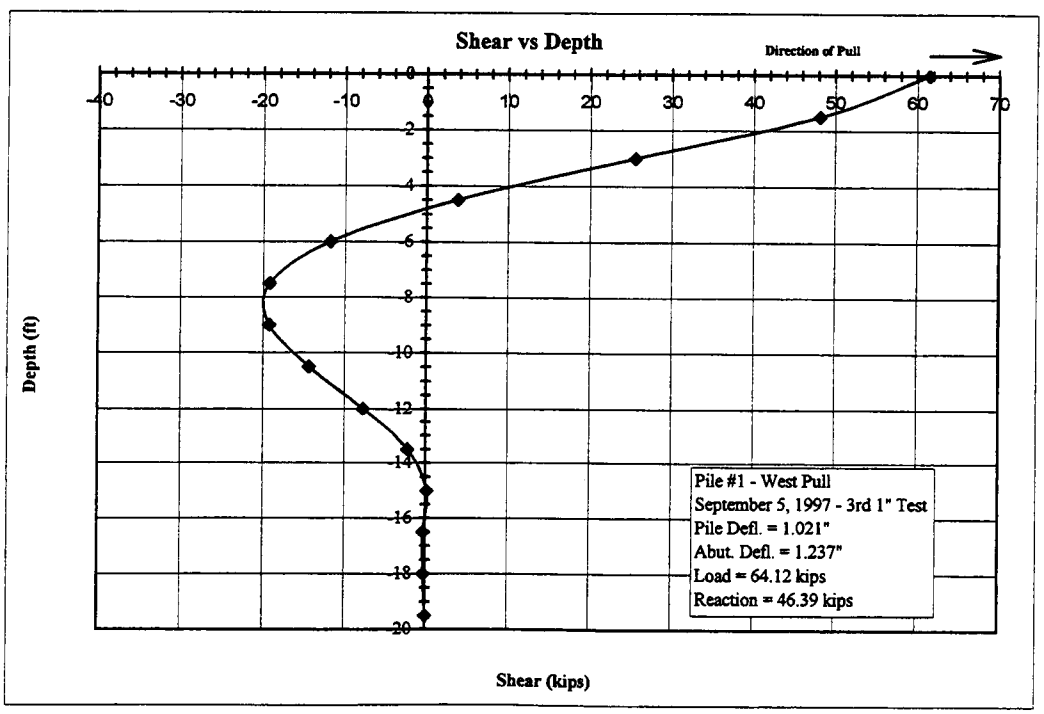
Pressure Equation:
From 0' to 19.5'

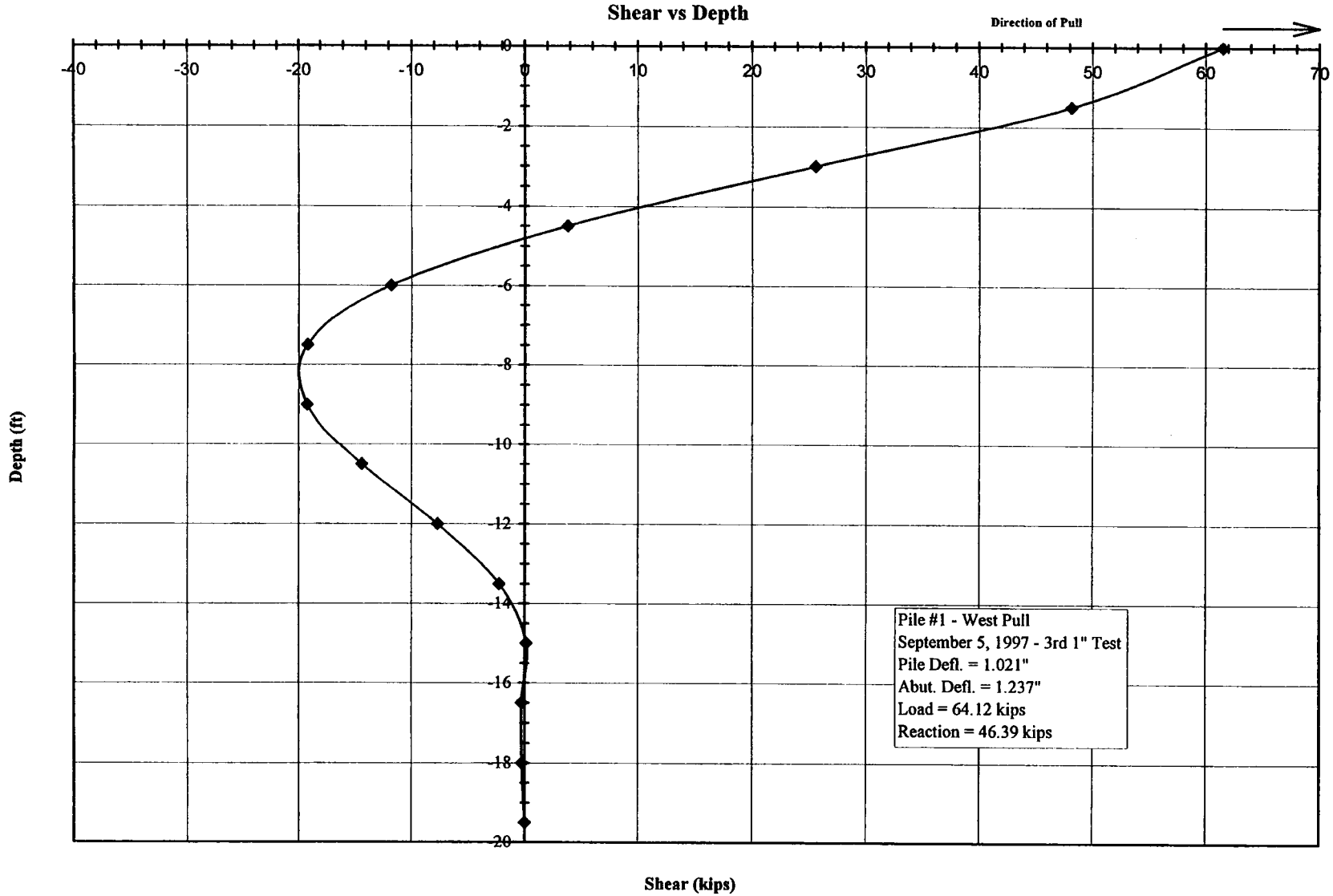
$$y = 0.0051765\,x^4 + -0.2108908\,x^3 + 2.72744964\,x^2 + -10.71674946\,x + -2.74931628$$

Shear

Date	09/05/97	
Time	16020-16030	sec
Load	64.12	kips
Reaction	46.39	kips
Abut. Defl.	1.237	inches
Pile Defl.	1.021	inches

Shear (kips)	Depth (ft)	Station	Depth (ft)
61.537	0	1	0
48.166	-1.5	2	1.5
25.592	-3	3	3
3.795	-4.5	4	4.5
-11.762	-6	5	6
-19.194	-7.5	6	7.5
-19.245	-9	7	9
-14.351	-10.5	8	10.5
-7.692	-12	9	12
-2.249	-13.5	10	13.5
0.138	-15	11	15
-0.287	-16.5	12	16.5
-0.251	-18	13	18
0.000	-19.5	14	19.5

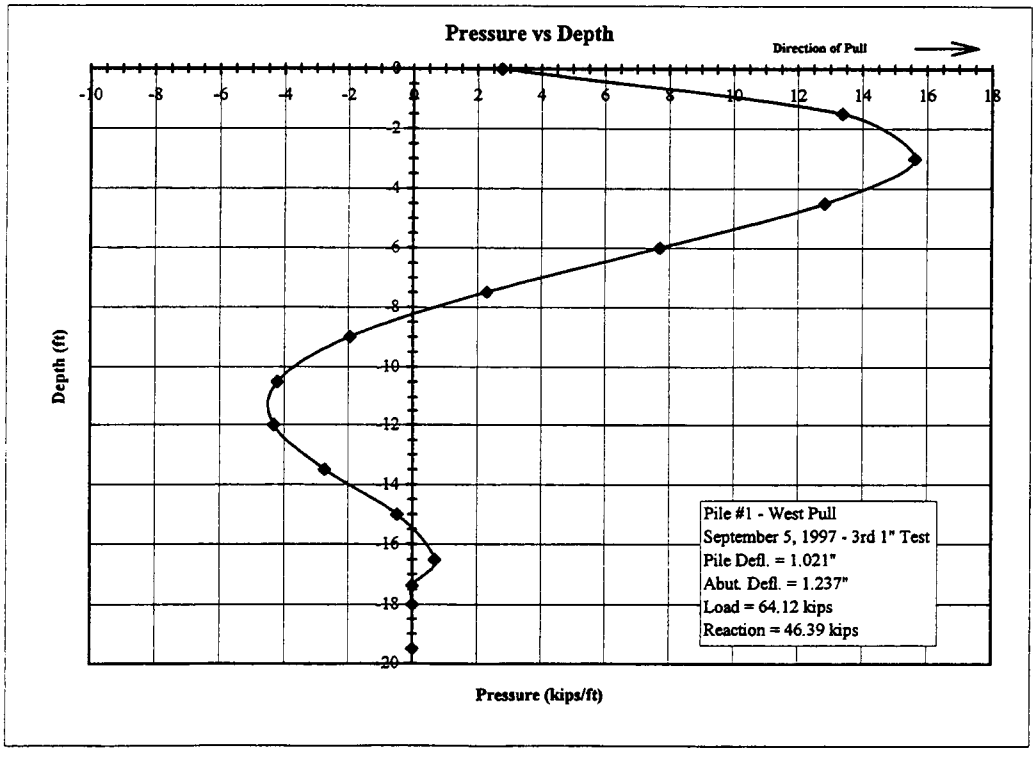


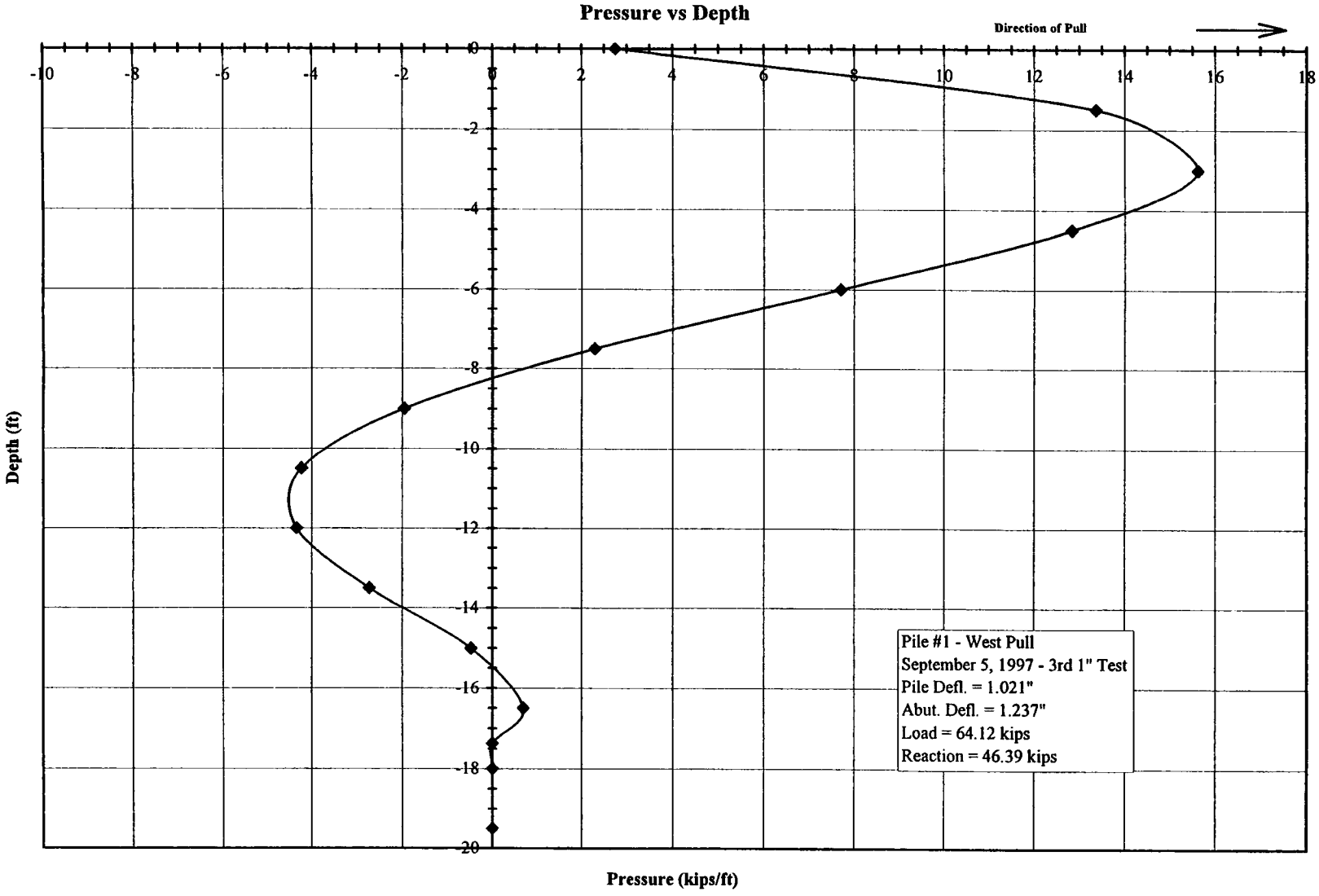

Shear vs Depth
Direction of Pull
-40
-30
-20
-10
0
10
20
30
40
50
60
70
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Shear (kips)
Pile #1 - West Pull
September 5, 1997 - 3rd 1" Test
Pile Defl. = 1.021"
Abut. Defl. = 1.237"
Load = 64.12 kips
Reaction = 46.39 kips

Pressure

Date	09/05/97	
Time	16020-16030	sec
Load	64.12	kips
Reaction	46.39	kips
Abut. Defl.	1.237	inches
Pile Defl.	1.021	inches

Pressure (kips/ft)	Depth (ft)	Station	Depth (ft)
2.749	0	1	0
13.373	-1.5	2	1.5
15.627	-3	3	3
12.839	-4.5	4	4.5
7.705	-6	5	6
2.297	-7.5	6	7.5
-1.947	-9	7	9
-4.214	-10.5	8	10.5
-4.323	-12	9	12
-2.720	-13.5	10	13.5
-0.479	-15	11	15
0.693	-16.5	12	16.5
0.000	-17.36402		17.36402
0.000	-18	13	18
0.000	-19.5	14	19.5




Pressure vs Depth
Direction of Pull
Depth (ft)
Pressure (kips/ft)
-10
-8
-6
-4
-2
0
2
4
6
8
10
12
14
16
18
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Pile #1 - West Pull
September 5, 1997 - 3rd 1" Test
Pile Defl. = 1.021"
Abut. Defl. = 1.237"
Load = 64.12 kips
Reaction = 46.39 kips

Statics

Date	09/05/97	
Time	16020-16030	sec
Load	64.12	kips
Reaction	46.39	kips
Abut. Defl.	1.237	inches
Pile Defl.	1.021	inches

Load by Statics

Pressure Equation

$y =$ 0.00517650 $x^4 +$ -0.21089080 $x^3 +$ 2.727449640 $x^2 +$ -10.71674946 $x +$ -2.74931628

Roots for pressure equation

x = -0.24143 x = 8.23732 x = 15.38013 x = 17.36402

Integrate to check statics:

From 0 to 8.23732

= -0.001035300 $x^5 +$ 0.052722700 $x^4 +$ -0.909149880 $x^3 +$ 5.358374730 $x^2 +$ 2.74931628 x

= 81.55546156 - 0 = **81.55546 kips**

From 8.23732 to 15.38013

= -0.001035300 $x^5 +$ 0.052722700 $x^4 +$ -0.909149880 $x^3 +$ 5.358374730 $x^2 +$ 2.74931628 x

= 61.31043056 - 81.55546156 = **-20.24503 kips**

From 15.38013 to 17.3640

= -0.001035300 $x^5 +$ 0.052722700 $x^4 +$ -0.909149880 $x^3 +$ 5.358374730 $x^2 +$ 2.74931628 x

= 62.22217285 - 61.31043056 = **0.91174 kips**

From Pressure	**62.222**	**kips**
Measured	**64.12**	**kips**
% difference	***2.96***	**%**

Moment by Statics

M_{GRND} = (Reaction)(x) - (Load)(y) - (Weight)(e)

x =	6.85	ft	(distance from centerline of the pile to centerline of the reaction)
y =	4.2083	ft	(distance from centerline of the load to the ground surface)
e =	0.216	in	($e = \Delta_{abut} - \Delta_{pile}$)
Weight =	65.0	kips	

M_{GRND} = 317.772 - 269.838 - 1.170 = **46.763 kip-ft**

Measured	**=**	**53.958**	**kip-ft**
% difference	**=**	***13.33***	**%**

Deflections

Date	09/05/97	
Time	16020-16030	sec
Load	64.12	kips
Reaction	46.39	kips
Abut. Defl.	1.237	inches
Pile Defl.	1.021	inches

@ x = 0

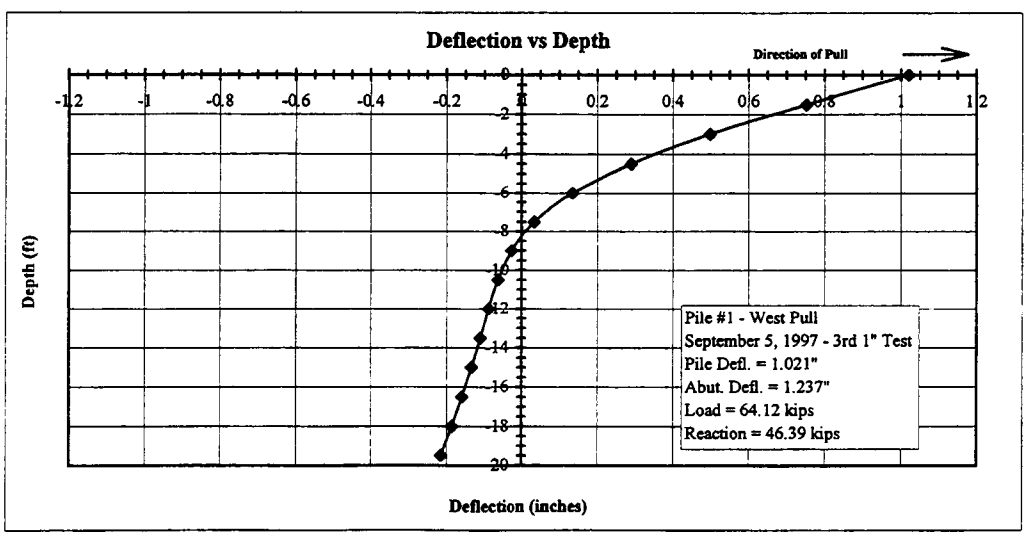
y = 0.000 + 0 C_1 + C_2 = 1.021

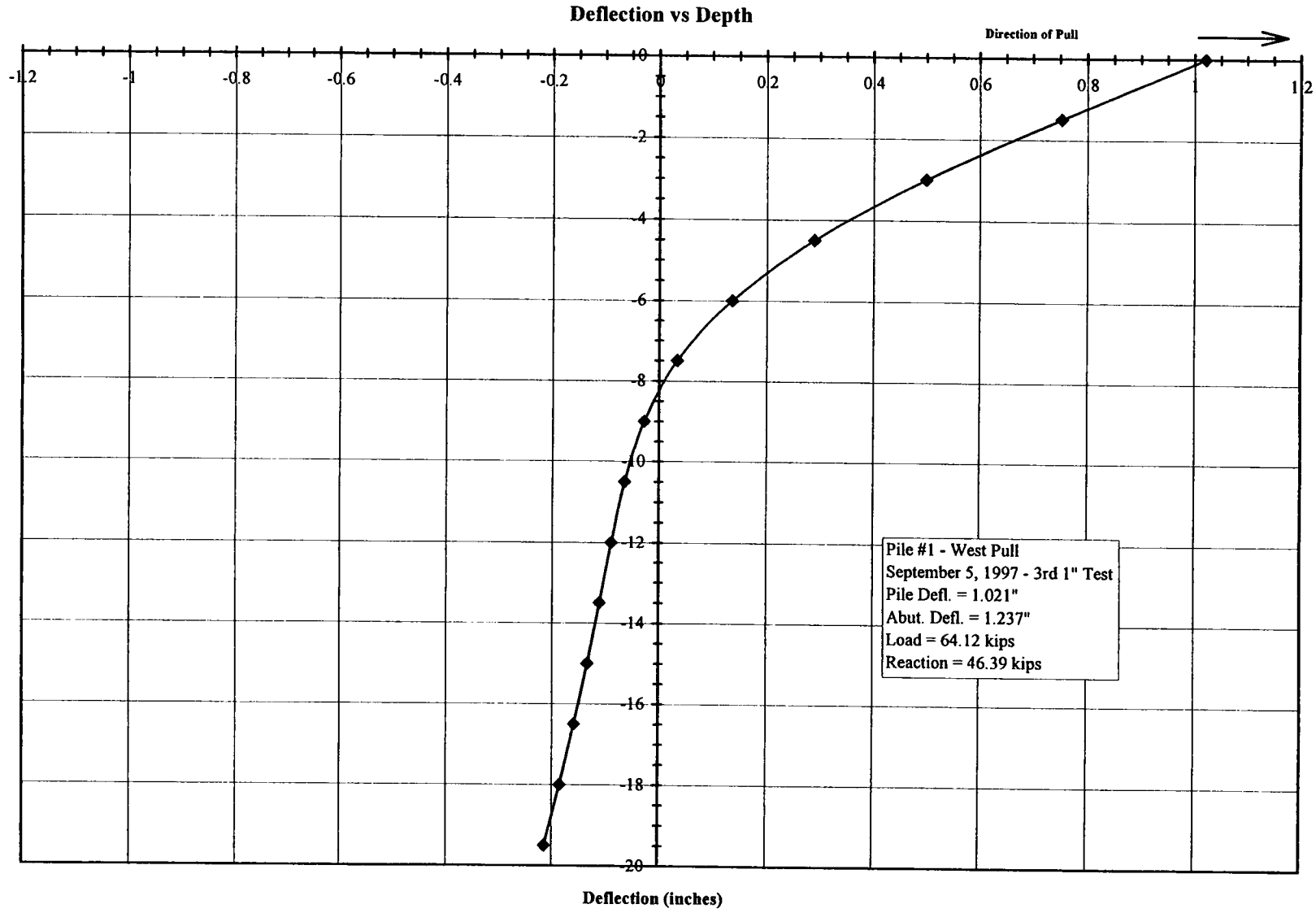
@ x = 8.23732

y = 0.417 + 8.23732 C_1 + 1.021 = 0

C_1 = -0.1746055 = -0.0145505 radians

Deflection (in)	Depth (ft)	Station	Depth (ft)
1.021	0	1	0
0.753	-1.5	2	1.5
0.499	-3	3	3
0.290	-4.5	4	4.5
0.136	-6	5	6
0.034	-7.5	6	7.5
-0.027	-9	7	9
-0.064	-10.5	8	10.5
-0.088	-12	9	12
-0.110	-13.5	10	13.5
-0.133	-15	11	15
-0.158	-16.5	12	16.5
-0.185	-18	13	18
-0.214	-19.5	14	19.5




Deflection vs Depth
Direction of Pull
-1.2
-1
-0.8
-0.6
-0.4
-0.2
0
0.2
0.4
0.6
0.8
1
1.2
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Deflection (inches)
Pile #1 - West Pull
September 5, 1997 - 3rd 1" Test
Pile Defl. = 1.021"
Abut. Defl. = 1.237"
Load = 64.12 kips
Reaction = 46.39 kips

APPENDIX M

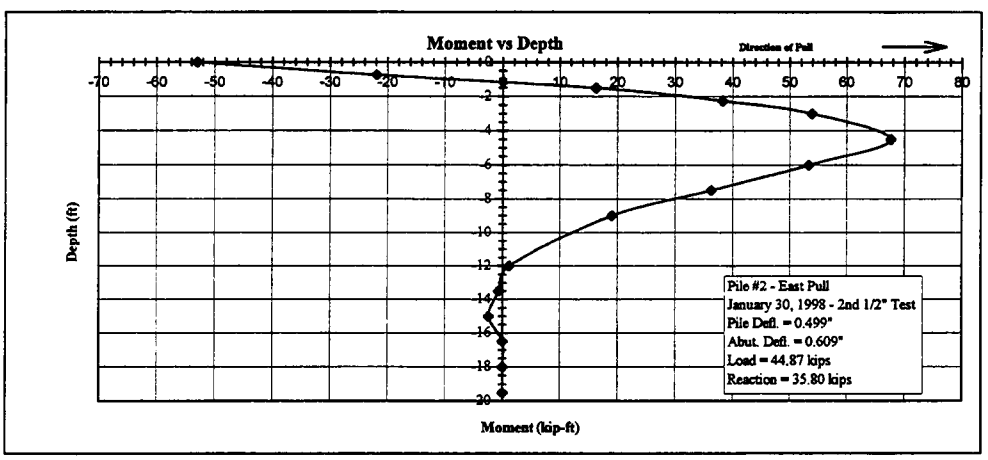
ONE-HALF INCH INCREMENT

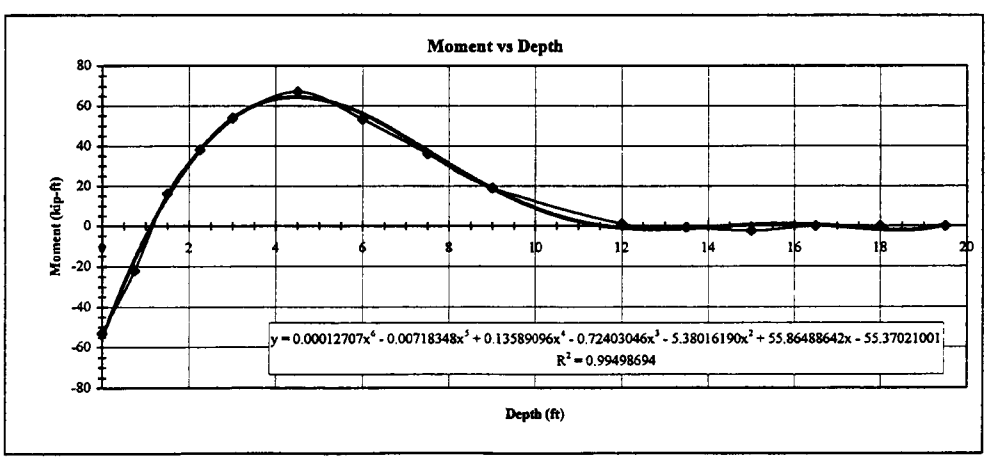
FOR JANUARY 30, 1998

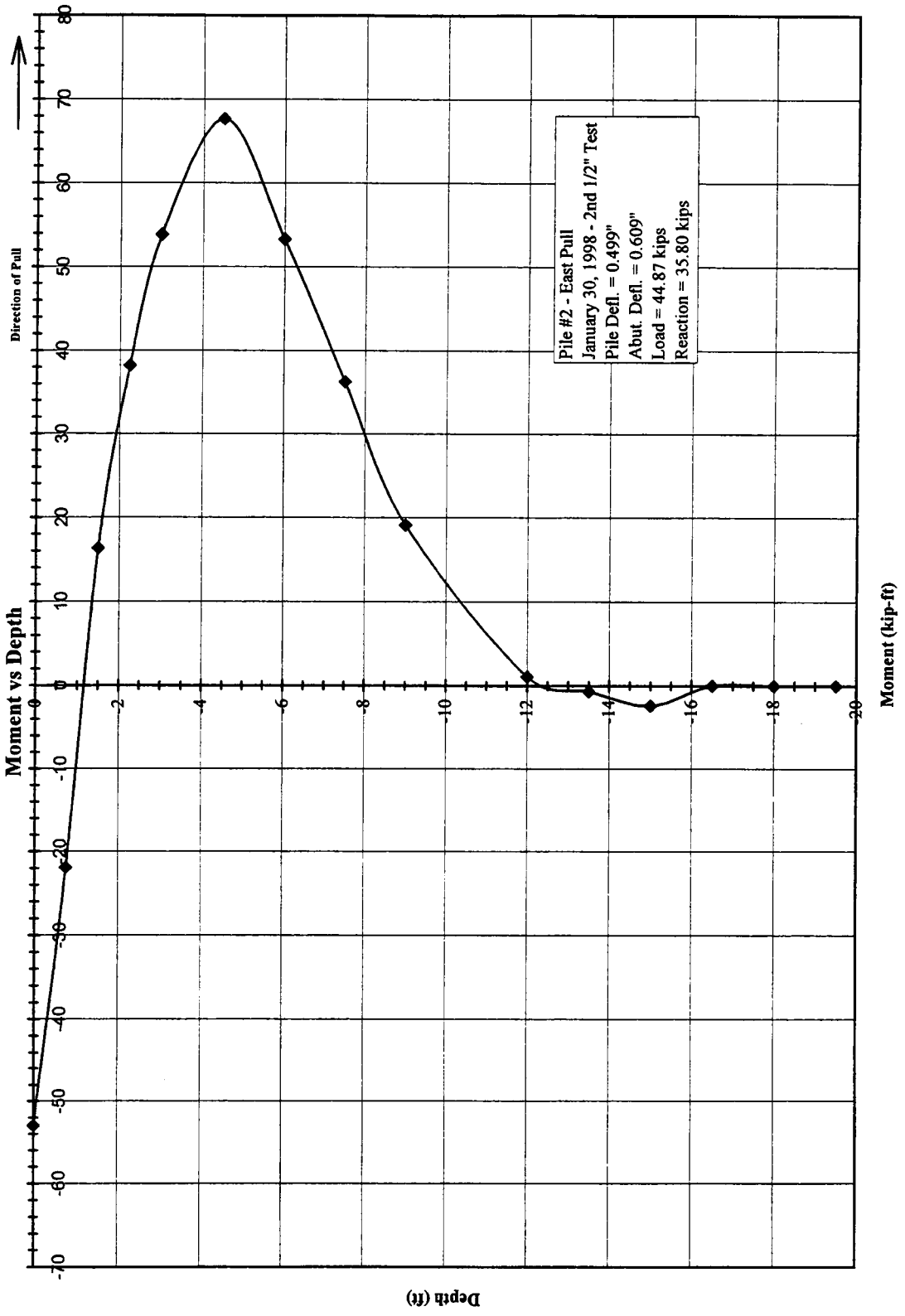
Moments

Date	01/30/98	(First Test)
Time	8410-8420	sec
Load	44.87	kips
Reaction	35.80	kips
Abut. Defl.	0.609	inches
Pile Defl.	0.499	inches

Moment (kip-ft)	Depth (ft)	Station	Depth (ft)
-52.966	0	1	0
-21.891	-0.75	2	0.75
16.343	-1.5	3	1.5
38.236	-2.25	4	2.25
53.846	-3	5	3
67.650	-4.5	6	4.5
53.311	-6	7	6
36.290	-7.5	8	7.5
19.082	-9	9	9
1.106	-12	11	12
-0.666	-13.5	12	13.5
-2.378	-15	13	15
0.000	-16.5	14	16.5
0.000	-18	15	18
0.000	-19.5	16	19.5






Moment vs Depth
Direction of Pull
Pile #2 - East Pull
January 30, 1998 - 2nd 1/2" Test
Pile Defl. = 0.499"
Abut. Defl. = 0.609"
Load = 44.87 kips
Reaction = 35.80 kips
Moment (kip-ft)
Depth (ft)
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20

Equations

Date	01/30/98	(First Test)
Time	8410-8420	sec
Load	44.87	kips
Reaction	35.80	kips
Abut. Defl.	0.609	inches
Pile Defl.	0.499	inches $= C_2$

Modulus of Elasticity (E) =	33000	ksi
Moment of Inertia (I) =	223	in^4

Deflection Equation:
From 0' to 19.5'

$$y = 5.32819E\text{-}10\ x^8 + -4.01616E\text{-}08\ x^7 + 1.06364E\text{-}06\ x^6 + -8.50064E\text{-}06\ x^5 + -0.000105278\ x^4 + 0.002186314\ x^3 + -0.006500864\ x^2 + (C_1)\ x + 0.499\ (C_2)$$

Slope Equation:
From 0' to 19.5'

$$y = 3.55213E\text{-}10\ x^7 + -2.34276E\text{-}08\ x^6 + 5.31819E\text{-}07\ x^5 + -3.54193E\text{-}06\ x^4 + -3.50928E\text{-}05\ x^3 + 0.000546579\ x^2 + -0.001083477\ x + (C_1)$$

Moment Equation:
From 0' to 19.5'

$$\mathbf{y = 0.00012707\ x^6 + -0.00718348\ x^5 + 0.13589096\ x^4 + -0.72403046\ x^3 + -5.38016190\ x^2 + 55.86488642\ x + -55.37021001}$$

Shear Equation:
From 0' to 19.5'

$$y = 0.00076242\ x^5 + -0.03591740\ x^4 + 0.54356384\ x^3 + -2.17209138\ x^2 + -10.76032380\ x + 55.86488642$$

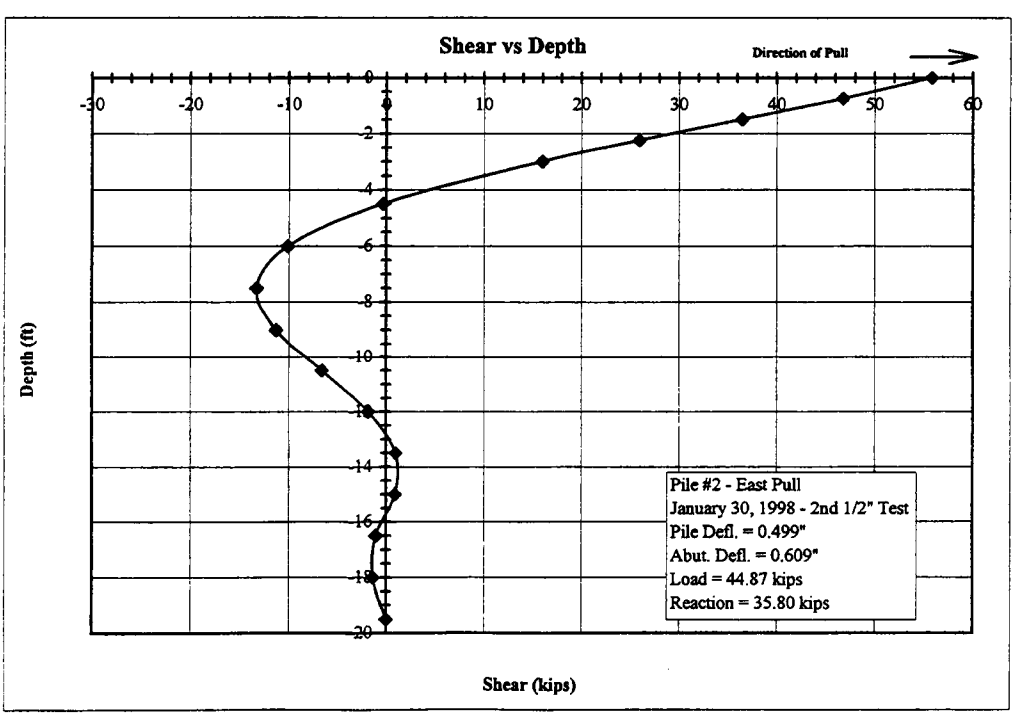
Pressure Equation:
From 0' to 19.5'

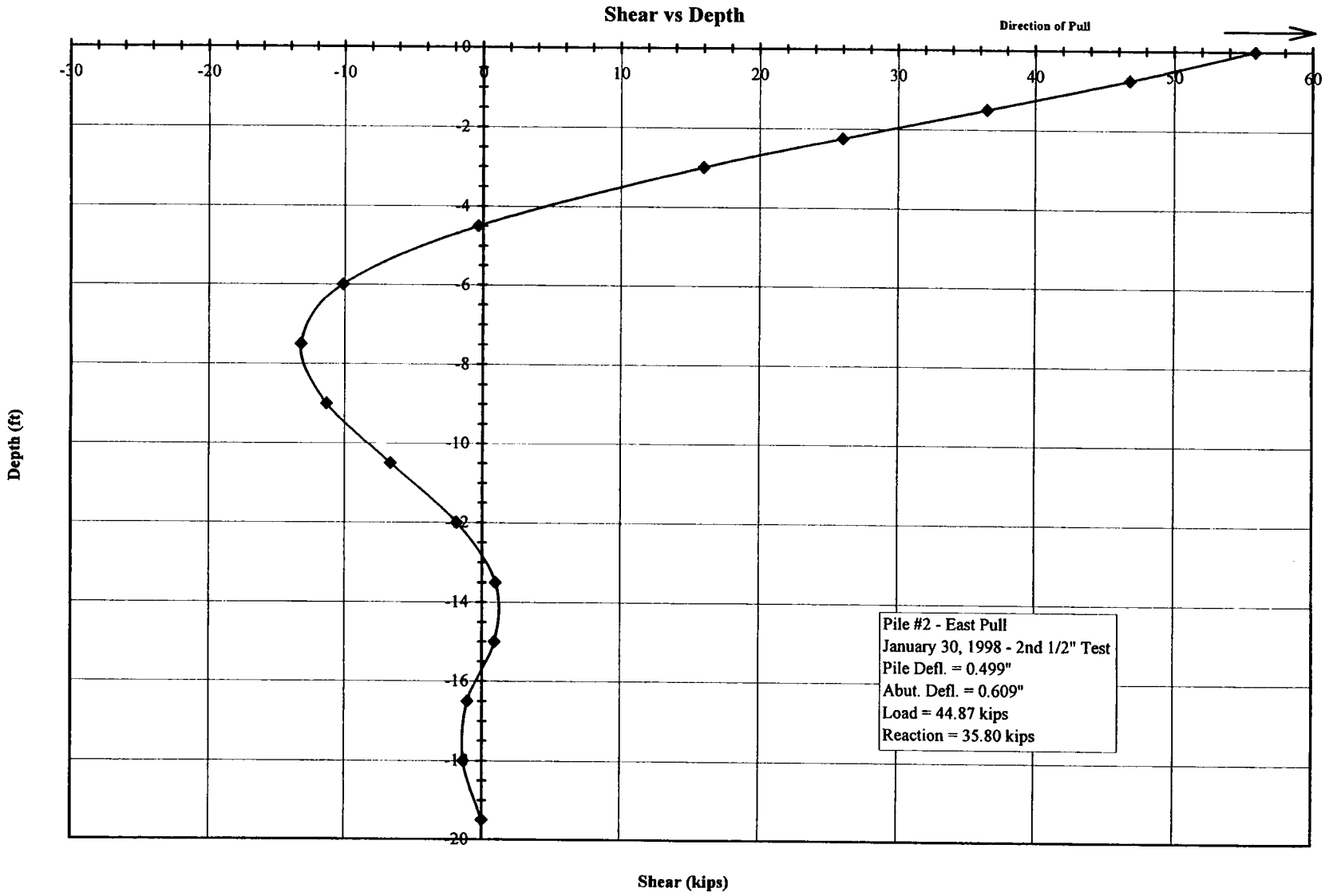
$$y = 0.0038121\ x^4 + -0.1436696\ x^3 + 1.63069152\ x^2 + -4.34418276\ x + -10.76032380$$

Shear

Date	01/30/98	(First Test)
Time	8410-8420	sec
Load	44.87	kips
Reaction	35.80	kips
Abut. Defl.	0.609	inches
Pile Defl.	0.499	inches

Shear (kips)	Depth (ft)	Station	Depth (ft)
55.865	0	1	0
46.791	-0.75	2	0.75
36.496	-1.5	3	1.5
25.973	-2.25	4	2.25
15.987	-3	5	3
-0.331	-4.5	6	4.5
-10.103	-6	7	6
-13.254	-7.5	8	7.5
-11.293	-9	9	9
-6.620	-10.5	10	10.5
-1.831	-12	11	12
0.981	-13.5	12	13.5
0.912	-15	13	15
-1.050	-16.5	14	16.5
-1.335	-18	15	18
0.000	-19.5	16	19.5

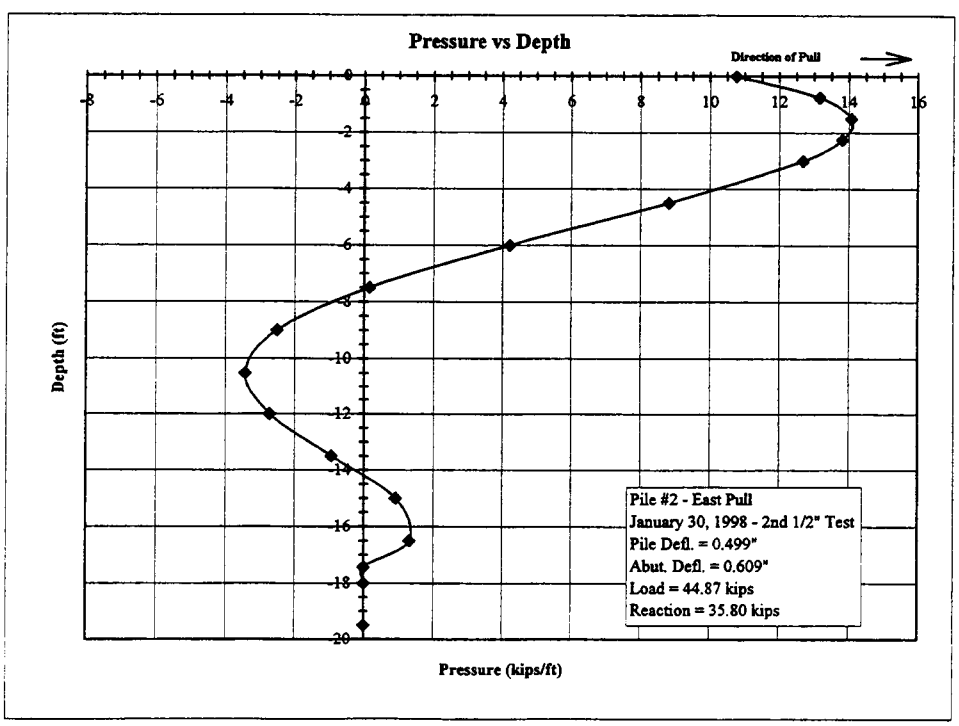


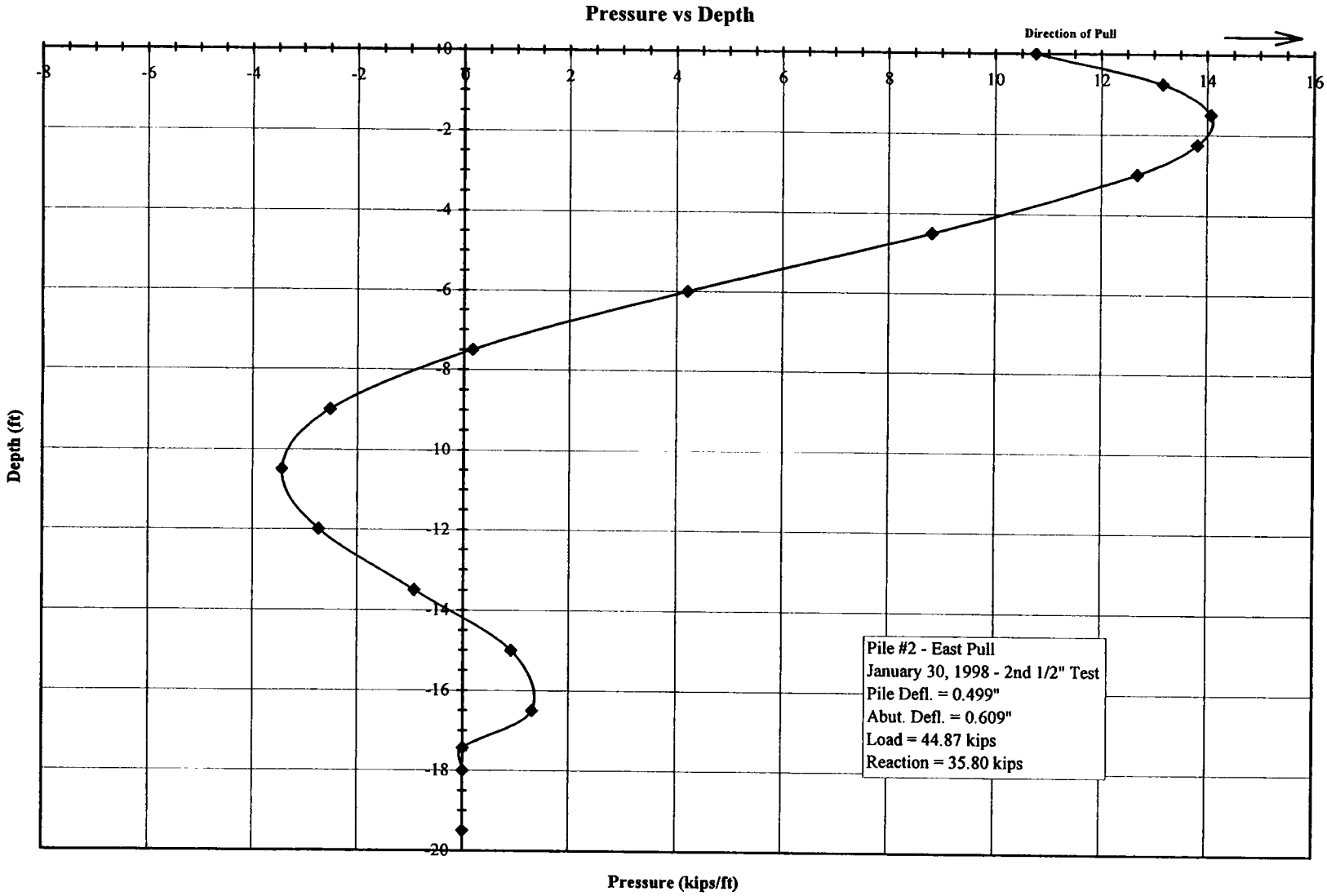

Shear vs Depth
Direction of Pull
-30
-20
-10
0
10
20
30
40
50
60
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Shear (kips)
Pile #2 - East Pull
January 30, 1998 - 2nd 1/2" Test
Pile Defl. = 0.499"
Abut. Defl. = 0.609"
Load = 44.87 kips
Reaction = 35.80 kips

Pressure

Date	01/30/98	(First Test)
Time	8410-8420	sec
Load	44.87	kips
Reaction	35.80	kips
Abut. Defl.	0.609	inches
Pile Defl.	0.499	inches

Pressure (kips/ft)	Depth (ft)	Station	Depth (ft)
10.760	0	1	0
13.161	-0.75	2	0.75
14.073	-1.5	3	1.5
13.818	-2.25	4	2.25
12.687	-3	5	3
8.816	-4.5	6	4.5
4.213	-6	7	6
0.164	-7.5	8	7.5
-2.504	-9	9	9
-3.430	-10.5	10	10.5
-2.716	-12	11	12
-0.925	-13.5	12	13.5
0.915	-15	13	15
1.312	-16.5	14	16.5
0.000	-17.42861		17.42861
0.000	-18	15	18
0.000	-19.5	16	19.5




Pressure vs Depth
Direction of Pull
Depth (ft)
Pressure (kips/ft)
Pile #2 - East Pull
January 30, 1998 - 2nd 1/2" Test
Pile Defl. = 0.499"
Abut. Defl. = 0.609"
Load = 44.87 kips
Reaction = 35.80 kips

Statics

Date	01/30/98	(First Test)
Time	8410-8420	sec
Load	44.87	kips
Reaction	35.80	kips
Abut. Defl.	0.609	inches
Pile Defl.	0.499	inches

Load by Statics

Pressure Equation

$y =$ 0.00381210 $x^4 +$ -0.14366960 $x^3 +$ 1.630691520 $x^2 +$ -4.34418276 $x +$ -10.7603238

Roots for pressure equation

x = -1.50689 x = 7.57195 x = 14.19412 x = 17.42861

Integrate to check statics:

From 0 to 7.57195

= -0.000762420 $x^5 +$ 0.035917400 $x^4 +$ -0.543563840 $x^3 +$ 2.172091380 $x^2 +$ 10.7603238 x

= 69.12477471 - 0 = **69.12477 kips**

From 7.57195 to 14.19412

= -0.000762420 $x^5 +$ 0.035917400 $x^4 +$ -0.543563840 $x^3 +$ 2.172091380 $x^2 +$ 10.7603238 x

= 54.56590584 - 69.12477471 = **-14.55887 kips**

From 14.19412 to 17.4286

= -0.000762420 $x^5 +$ 0.035917400 $x^4 +$ -0.543563840 $x^3 +$ 2.172091380 $x^2 +$ 10.7603238 x

= 57.64496705 - 54.56590584 = **3.07906 kips**

From Pressure = 57.645 kips
Measured = 44.87 kips
% difference = *28.47* %

Moment by Statics

M_{GRND} = (Reaction)(x) - (Load)(y) - (Weight)(e)

x =	6.75	ft	(distance from centerline of the pile to centerline of the reaction)
y =	4.2083	ft	(distance from centerline of the load to the ground surface)
e =	0.110	in	($e = \Delta_{abut} - \Delta_{pile}$)
Weight =	65.0	kips	

M_{GRND} = 241.650 - 188.828 - 0.596 = **52.226 kips / ft**

Measured = 52.966 kips / ft

% difference = *1.40* %

Deflections

Date	01/30/98	(First Test)
Time	8410-8420	sec
Load	44.87	kips
Reaction	35.80	kips
Abut. Defl.	0.609	inches
Pile Defl.	0.499	inches

@ x = 0

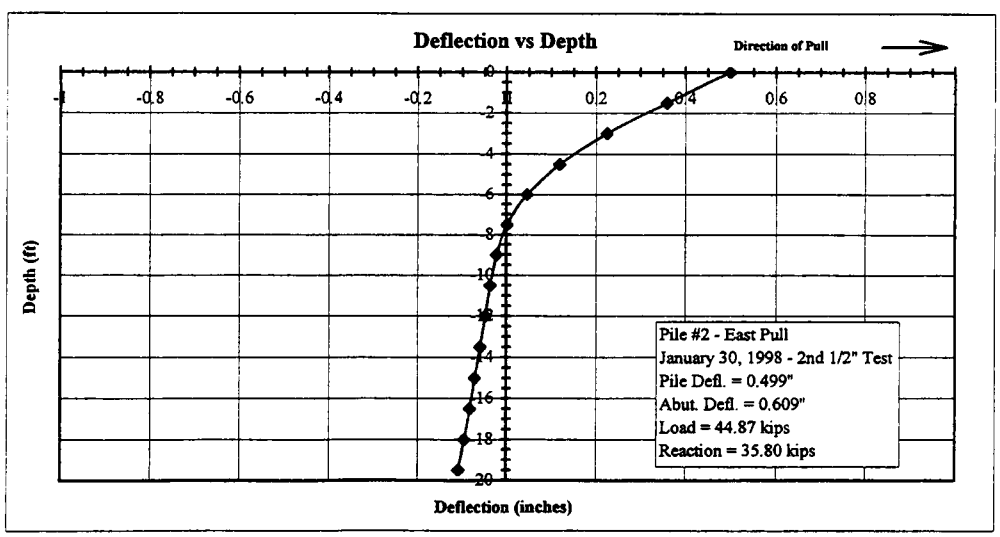
y = 0.000 + 0 C_1 + C_2 = 0.499

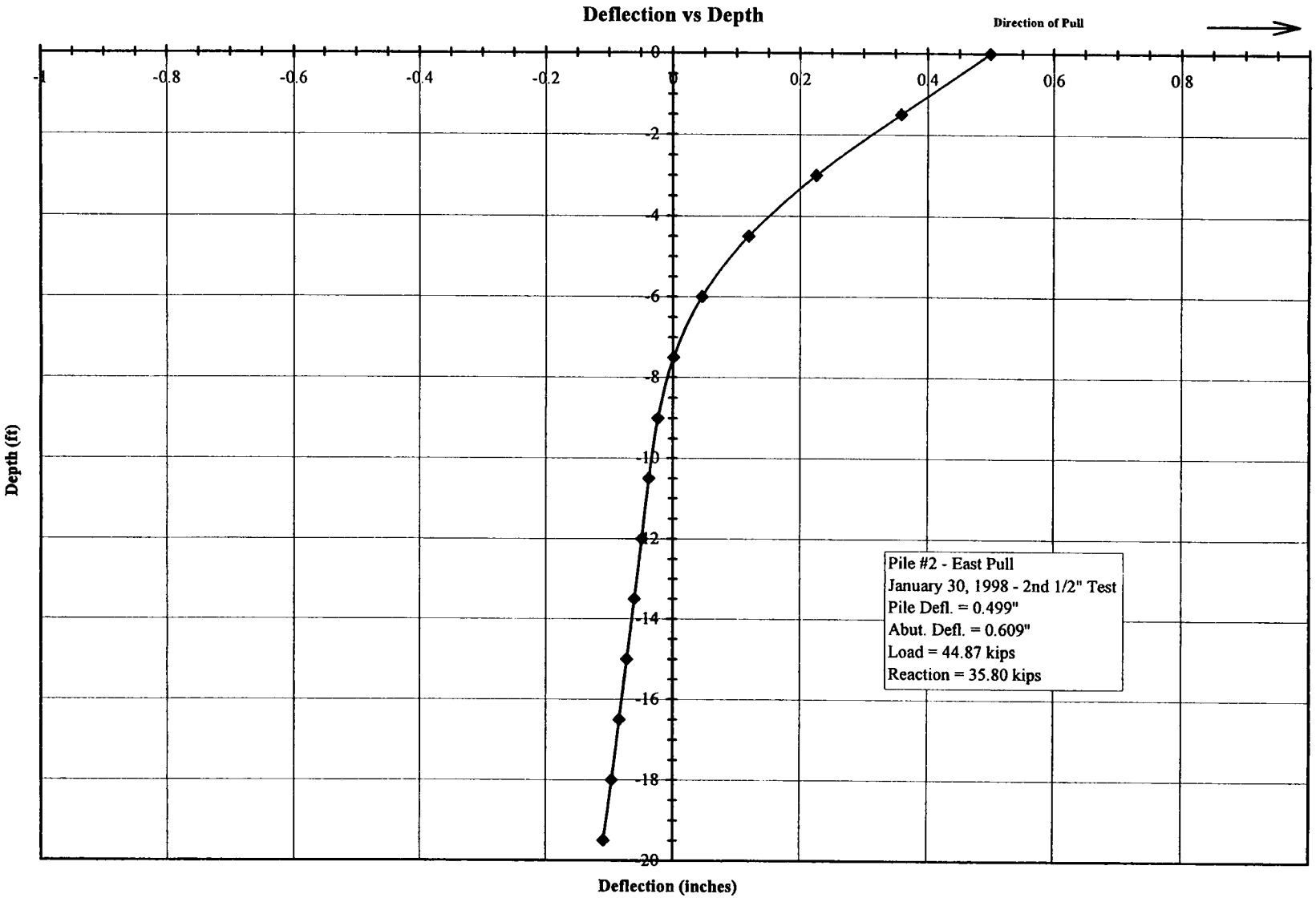
@ x = 7.57195

y = 0.168 + 7.57195 C_1 + 0.499 = 0

C_1 = -0.0880455 = -0.0073371 radians

Deflection (in)	Depth (ft)	Station	Depth (ft)
0.499	0	1	0
0.359	-1.5	2	1.5
0.225	-3	3	3
0.119	-4.5	4	4.5
0.046	-6	5	6
0.002	-7.5	6	7.5
-0.023	-9	7	9
-0.037	-10.5	8	10.5
-0.048	-12	9	12
-0.059	-13.5	10	13.5
-0.071	-15	11	15
-0.083	-16.5	12	16.5
-0.095	-18	13	18
-0.108	-19.5	14	19.5




Deflection vs Depth
Direction of Pull
-1
-0.8
-0.6
-0.4
-0.2
0
0.2
0.4
0.6
0.8
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Deflection (inches)
Pile #2 - East Pull
January 30, 1998 - 2nd 1/2" Test
Pile Defl. = 0.499"
Abut. Defl. = 0.609"
Load = 44.87 kips
Reaction = 35.80 kips

APPENDIX N

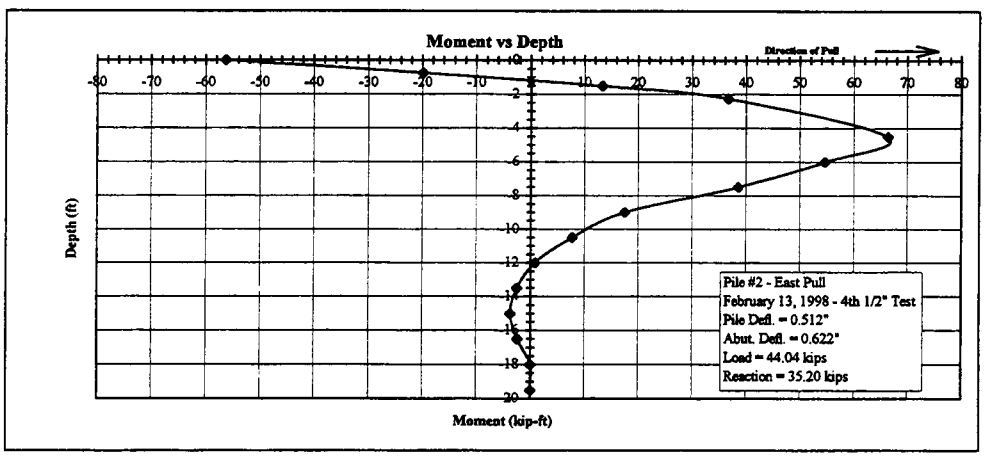
ONE-HALF INCH INCREMENT

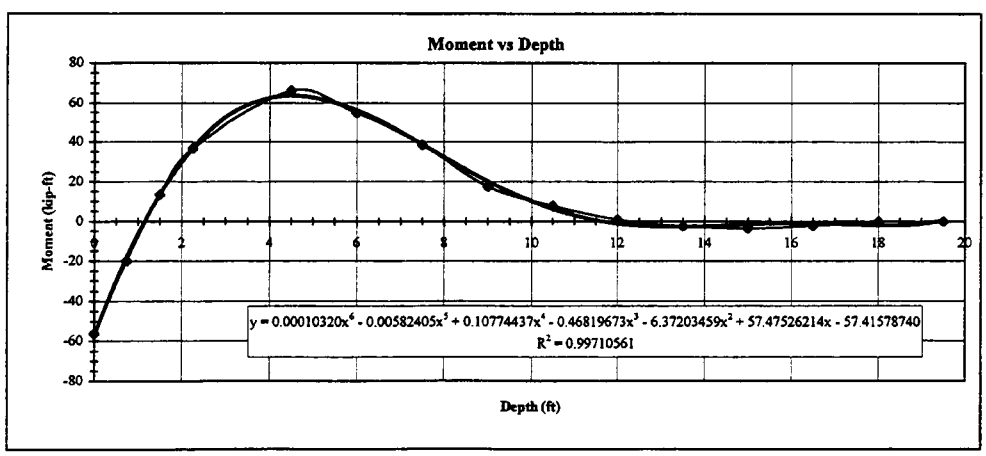
FOR FEBRUARY 13, 1998

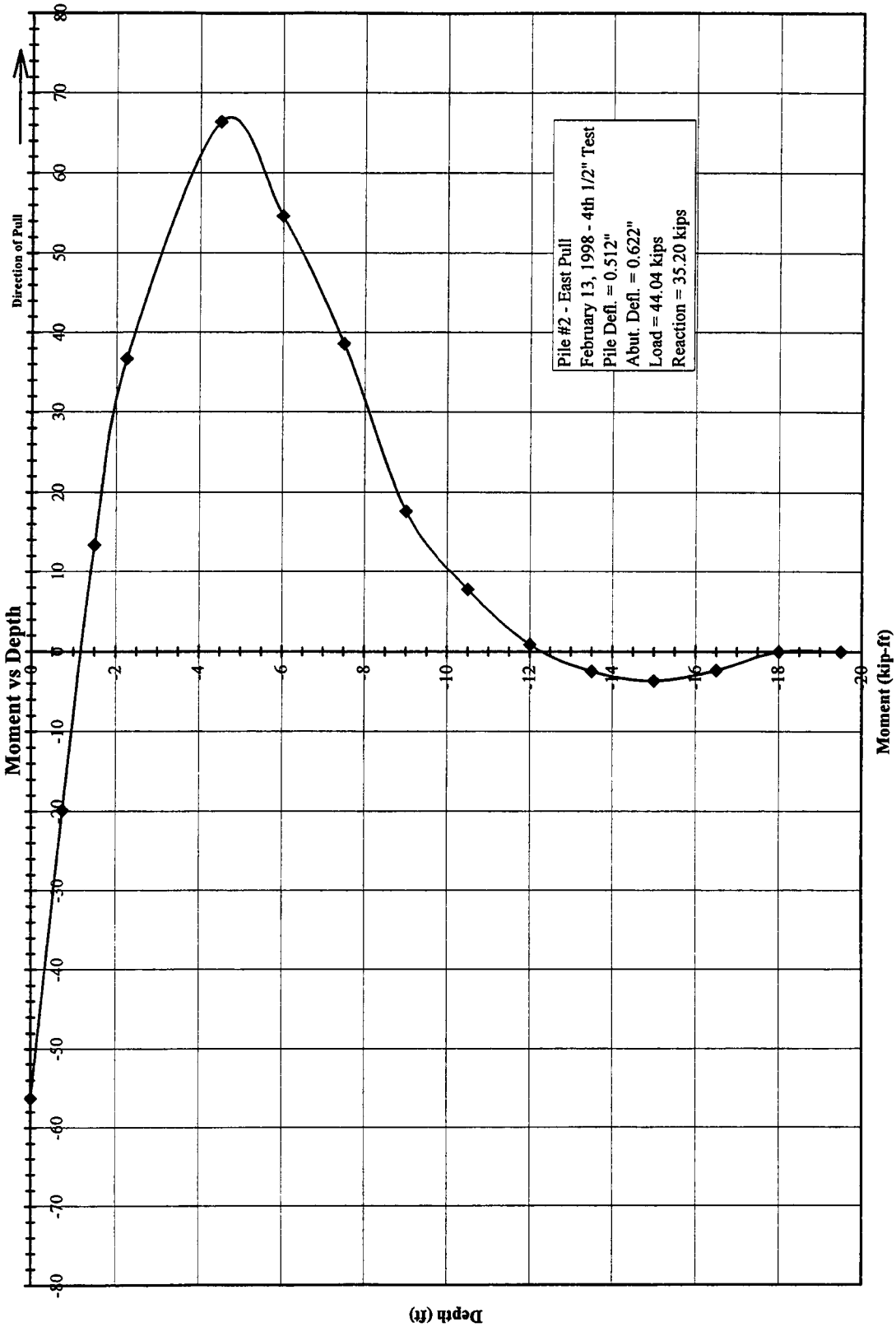
Moments

Date	02/13/98	
Time	6000-6010	sec
Load	44.40	kips
Reaction	35.20	kips
Abut. Defl.	0.622	inches
Pile Defl.	0.512	inches

Moment (kip-ft)	Depth (ft)	Station	Depth (ft)
-56.300	0	1	0
-19.858	-0.75	2	0.75
13.380	-1.5	3	1.5
36.700	-2.25	4	2.25
66.411	-4.5	6	4.5
54.685	-6	7	6
38.602	-7.5	8	7.5
17.620	-9	9	9
7.770	-10.5	10	10.5
0.859	-12	11	12
-2.484	-13.5	12	13.5
-3.650	-15	13	15
-2.356	-16.5	14	16.5
0.000	-18	15	18
0.000	-19.5	16	19.5






Moment vs Depth
Direction of Pull
Pile #2 - East Pull
February 13, 1998 - 4th 1/2" Test
Pile Defl. = 0.512"
Abut. Defl. = 0.622"
Load = 44.04 kips
Reaction = 35.20 kips
Depth (ft)
Moment (kip-ft)

Equations

Date	02/13/98		
Time	6000-6010	sec	
Load	44.40	kips	
Reaction	35.20	kips	
Abut. Defl.	0.622	inches	
Pile Defl.	0.512	inches	$= C_2$
Modulus of Elasticity (E) =	33000	ksi	
Moment of Inertia (I) =	223	in^4	

Deflection Equation:
From 0' to 19.5'

$$y = 4.3273E\text{-}10\ x^8 + -3.25612E\text{-}08\ x^7 + 8.43331E\text{-}07\ x^6 + -5.49697E\text{-}06\ x^5 + -0.000124687\ x^4 + 0.002249338\ x^3 + -0.00674103\ x^2 + (C_1)\ x + 0.512\ (C_2)$$

Slope Equation:
From 0' to 19.5'

$$y = 2.88486E\text{-}10\ x^7 + -1.8994E\text{-}08\ x^6 + 4.21666E\text{-}07\ x^5 + -2.2904E\text{-}06\ x^4 + -4.15624E\text{-}05\ x^3 + 0.000562334\ x^2 + -0.001123505\ x + (C_1)$$

Moment Equation:
From 0' to 19.5'

$$\mathbf{y = 0.00010320\ x^6 + -0.00582405\ x^5 + 0.10774437\ x^4 + -0.46819673\ x^3 + -6.37203459\ x^2 + 57.47526214\ x + -57.41578740}$$

Shear Equation:
From 0' to 19.5'

$$y = 0.00061920\ x^5 + -0.02912025\ x^4 + 0.43097748\ x^3 + -1.40459019\ x^2 + -12.74406918\ x + 57.47526214$$

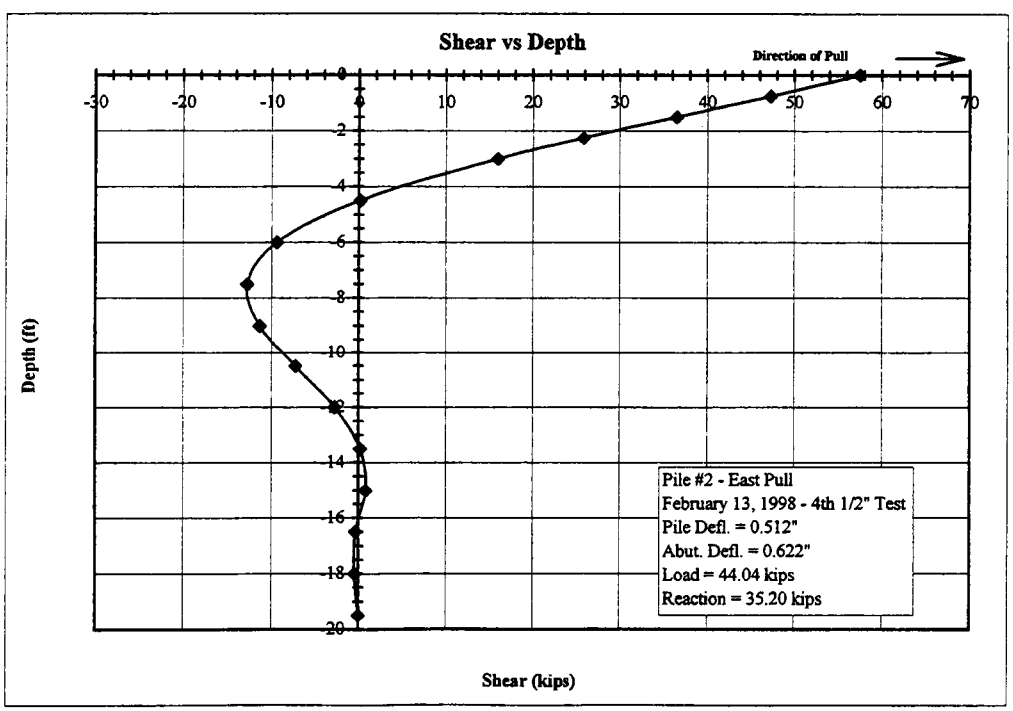
Pressure Equation:
From 0' to 19.5'

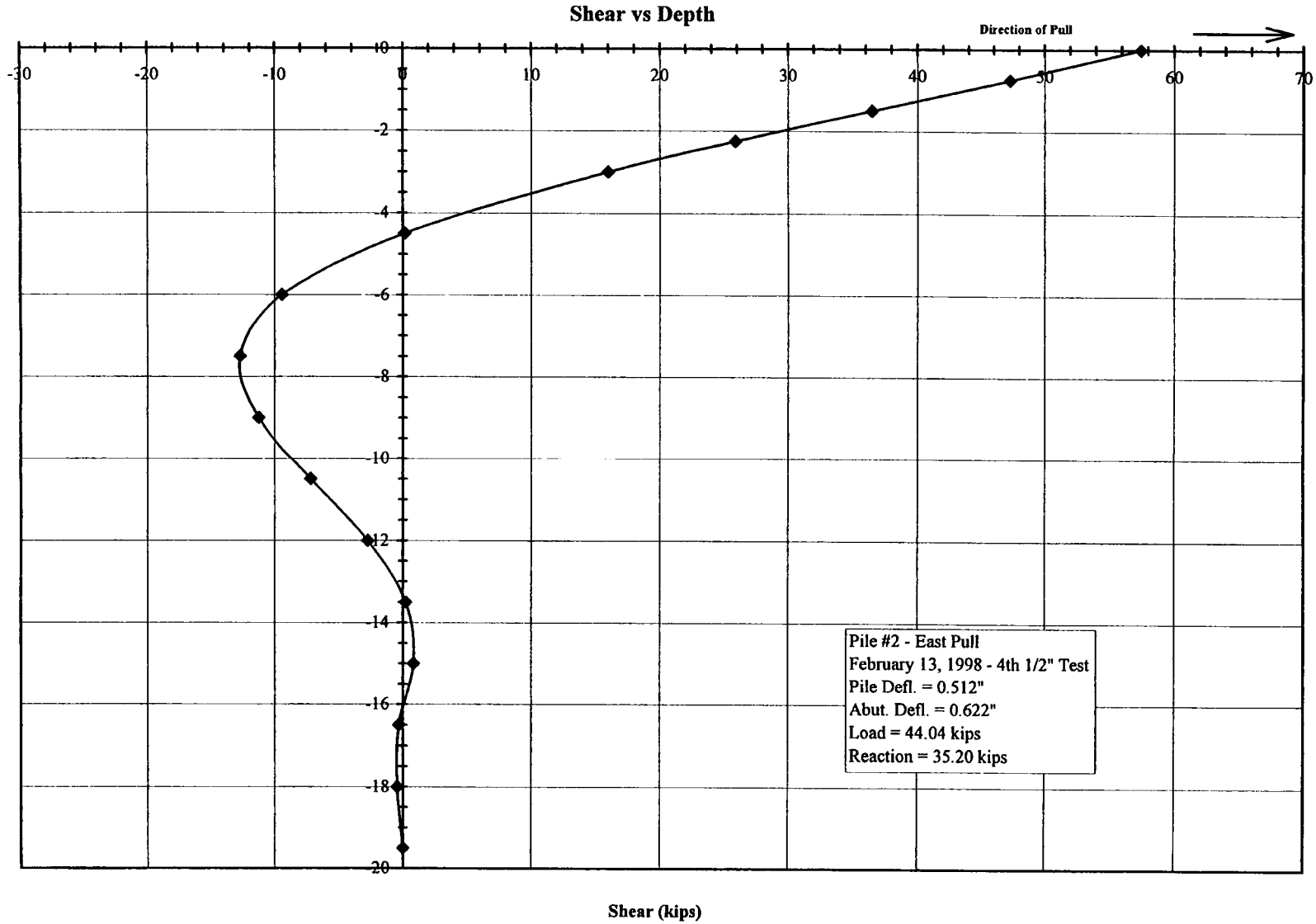
$$y = 0.003096\ x^4 + -0.116481\ x^3 + 1.29293244\ x^2 + -2.80918038\ x + -12.74406918$$

Shear

Date	02/13/98	
Time	6000-6010	sec
Load	44.40	kips
Reaction	35.20	kips
Abut. Defl.	0.622	inches
Pile Defl.	0.512	inches

Shear (kips)	Depth (ft)	Station	Depth (ft)
57.475	0	1	0
47.300	-0.75	2	0.75
36.511	-1.5	3	1.5
25.889	-2.25	4	2.25
16.030	-3	5	3
0.158	-4.5	6	4.5
-9.388	-6	7	6
-12.739	-7.5	8	7.5
-11.305	-9	9	9
-7.214	-10.5	10	10.5
-2.746	-12	11	12
0.230	-13.5	12	13.5
0.823	-15	13	15
-0.322	-16.5	14	16.5
-0.451	-18	15	18
0.000	-19.5	16	19.5

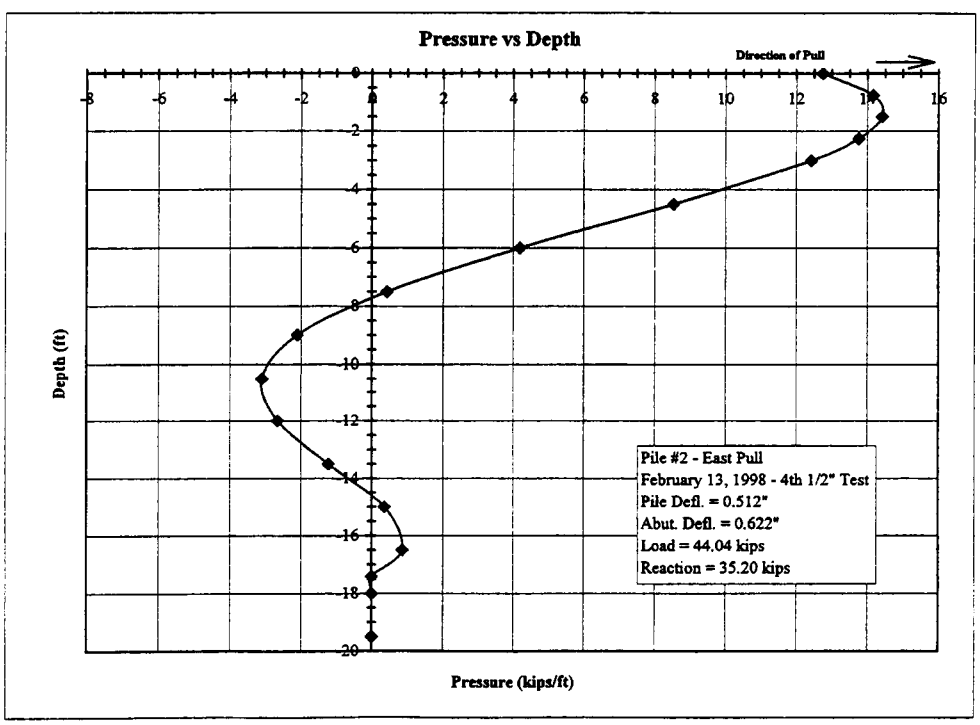


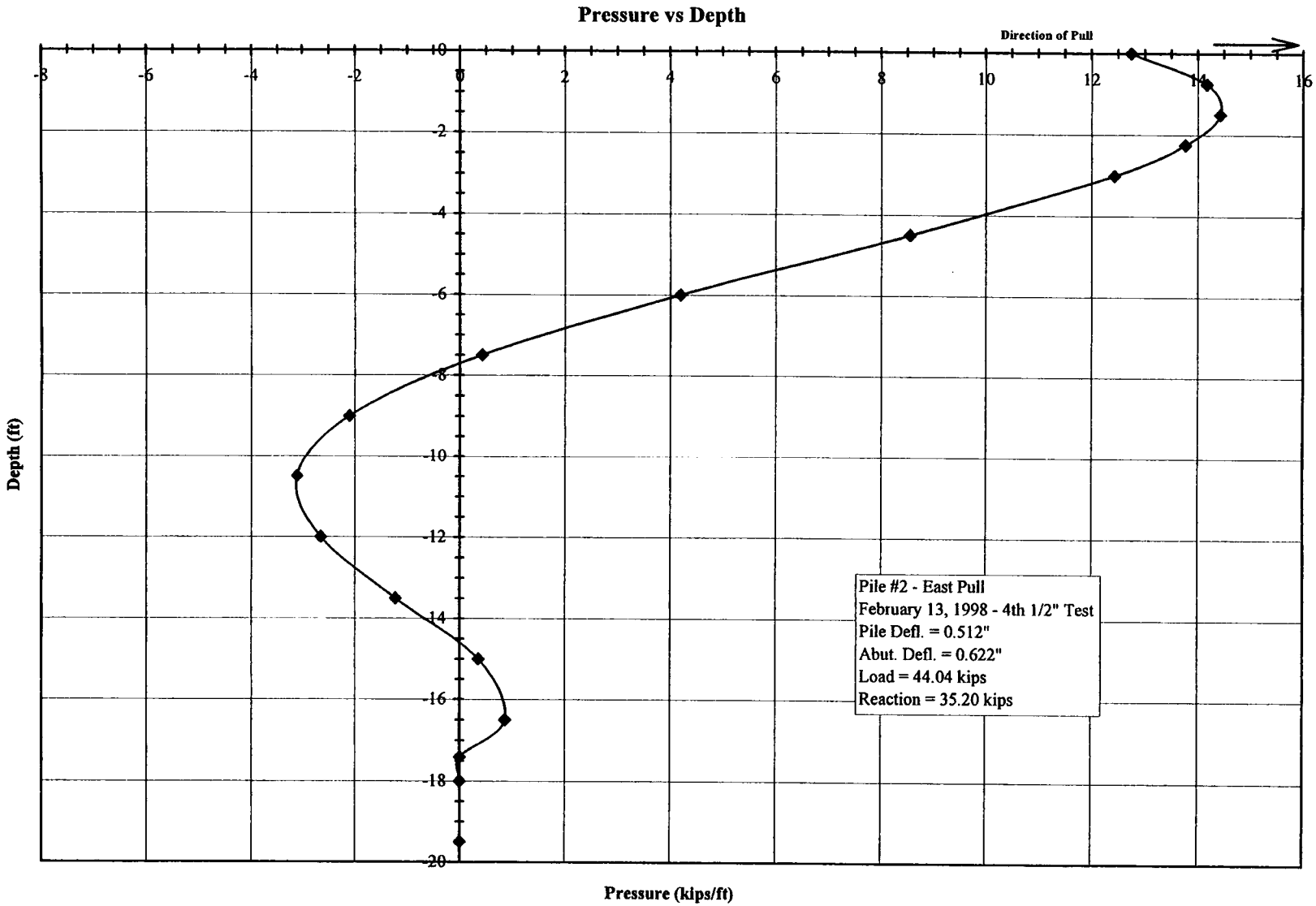

Shear vs Depth
Direction of Pull
-30
-20
-10
0
10
20
30
40
50
60
70
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Shear (kips)
Pile #2 - East Pull
February 13, 1998 - 4th 1/2" Test
Pile Defl. = 0.512"
Abut. Defl. = 0.622"
Load = 44.04 kips
Reaction = 35.20 kips

Pressure

Date	02/13/98	
Time	6000-6010	sec
Load	44.40	kips
Reaction	35.20	kips
Abut. Defl.	0.622	inches
Pile Defl.	0.512	inches

Pressure (kips/ft)	Depth (ft)	Station	Depth (ft)
12.744	0	1	0
14.172	-0.75	2	0.75
14.426	-1.5	3	1.5
13.767	-2.25	4	2.25
12.429	-3	5	3
8.548	-4.5	6	4.5
4.201	-6	7	6
0.430	-7.5	8	7.5
-2.099	-9	9	9
-3.096	-10.5	10	10.5
-2.648	-12	11	12
-1.216	-13.5	12	13.5
0.360	-15	13	15
0.866	-16.5	14	16.5
0.000	-17.40596		17.40596
0.000	-18	15	18
0.000	-19.5	16	19.5




Pressure vs Depth
Direction of Pull
-8
-6
-4
-2
0
2
4
6
8
10
12
14
16
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Pressure (kips/ft)
Pile #2 - East Pull
February 13, 1998 - 4th 1/2" Test
Pile Defl. = 0.512"
Abut. Defl. = 0.622"
Load = 44.04 kips
Reaction = 35.20 kips

Statics

Date	02/13/98	
Time	6000-6010	sec
Load	44.40	kips
Reaction	35.20	kips
Abut. Defl.	0.622	inches
Pile Defl.	0.512	inches

Load by Statics

Pressure Equation

$y =$ 0.00309600 $x^4 +$ -0.11648100 $x^3 +$ 1.292932440 $x^2 +$ -2.80918038 $x +$ -12.74406918

Roots for pressure equation

x = -2.10041 x = 7.70532 x = 14.61219 x = 17.40596

Integrate to check statics:

From 0 to 7.70532

= -0.000619200 $x^5 +$ 0.029120250 $x^4 +$ -0.430977480 $x^3 +$ 1.404590190 $x^2 +$ 12.74406918 x

= 70.25819871 - 0 = **70.25820 kips**

From 7.70532 to 14.61219

= -0.000619200 $x^5 +$ 0.029120250 $x^4 +$ -0.430977480 $x^3 +$ 1.404590190 $x^2 +$ 12.74406918 x

= 56.58075625 - 70.25819871 = **-13.67744 kips**

From 14.61219 to 17.4060

= -0.000619200 $x^5 +$ 0.029120250 $x^4 +$ -0.430977480 $x^3 +$ 1.404590190 $x^2 +$ 12.74406918 x

= 58.27716188 - 56.58075625 = **1.69641 kips**

From Pressure	=	**58.277 kips**
Measured	=	**44.4 kips**
% difference	=	***31.25*** **%**

Moment by Statics

M_{GRND} = (Reaction)(x) - (Load)(y) - (Weight)(e)

x =	6.75	ft	(distance from centerline of the pile to centerline of the reaction)
y =	4.2083	ft	(distance from centerline of the load to the ground surface)
e =	0.110	in	($e = \Delta_{abut} - \Delta_{pile}$)
Weight =	65.0	kips	

M_{GRND} = 237.600 - 186.850 - 0.596 = **50.154 kip-ft**

Measured = 56.300 kip-ft

% difference = *10.92* %

Deflections

Date	02/13/98	
Time	6000-6010	sec
Load	44.40	kips
Reaction	35.20	kips
Abut. Defl.	0.622	inches
Pile Defl.	0.512	inches

@ x = 0

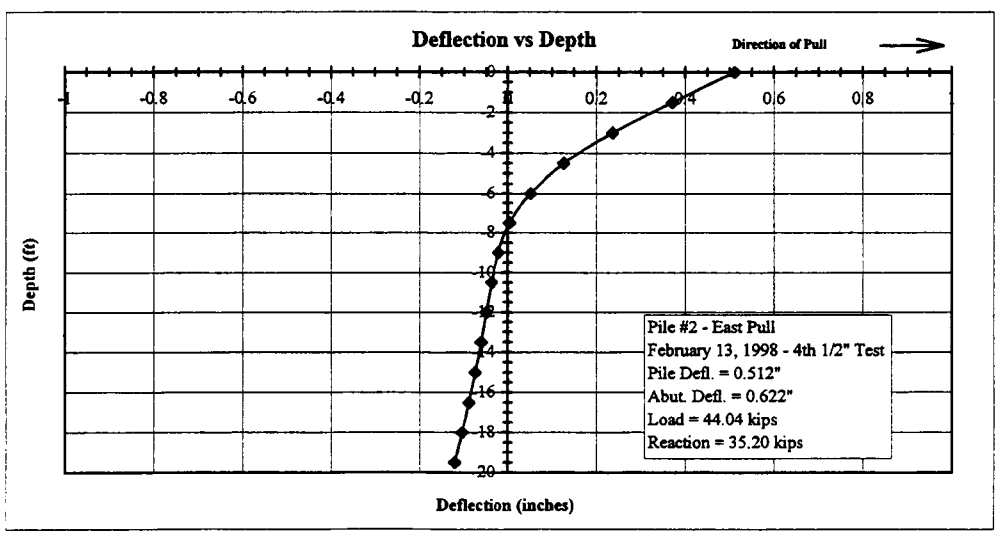
y = 0.000 + 0 C_1 + C_2 = 0.512

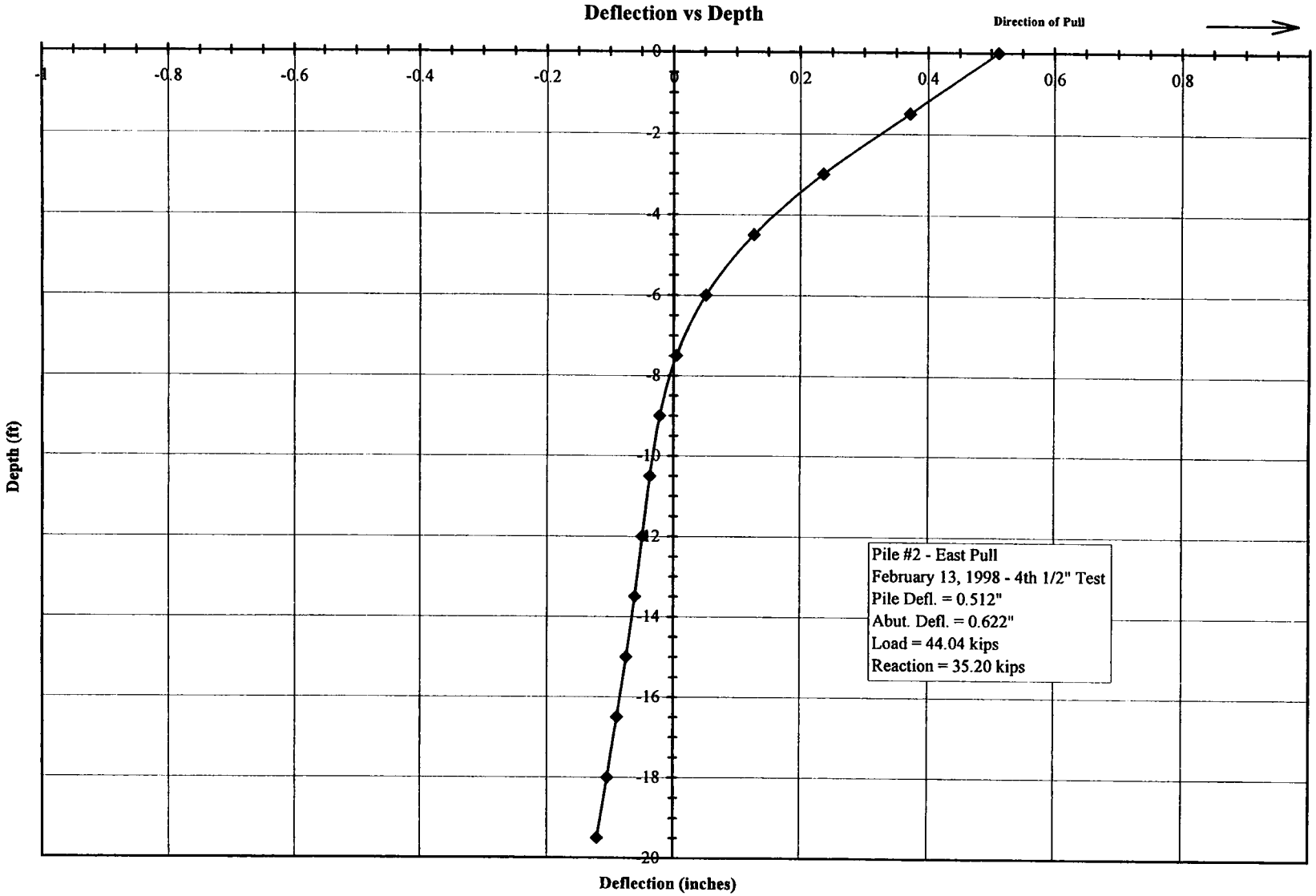
@ x = 7.70532

y = 0.169 + 7.70532 C_1 + 0.512 = 0

C_1 = -0.0884239 = -0.0073687 radians

Deflection (in)	Depth (ft)	Station	Depth (ft)
0.512	0	1	0
0.371	-1.5	2	1.5
0.236	-3	3	3
0.127	-4.5	4	4.5
0.051	-6	5	6
0.005	-7.5	6	7.5
-0.022	-9	7	9
-0.037	-10.5	8	10.5
-0.049	-12	9	12
-0.061	-13.5	10	13.5
-0.074	-15	11	15
-0.089	-16.5	12	16.5
-0.104	-18	13	18
-0.120	-19.5	14	19.5




Deflection vs Depth
Direction of Pull
-1
-0.8
-0.6
-0.4
-0.2
0
0.2
0.4
0.6
0.8
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Deflection (inches)
Pile #2 - East Pull
February 13, 1998 - 4th 1/2" Test
Pile Defl. = 0.512"
Abut. Defl. = 0.622"
Load = 44.04 kips
Reaction = 35.20 kips

APPENDIX O

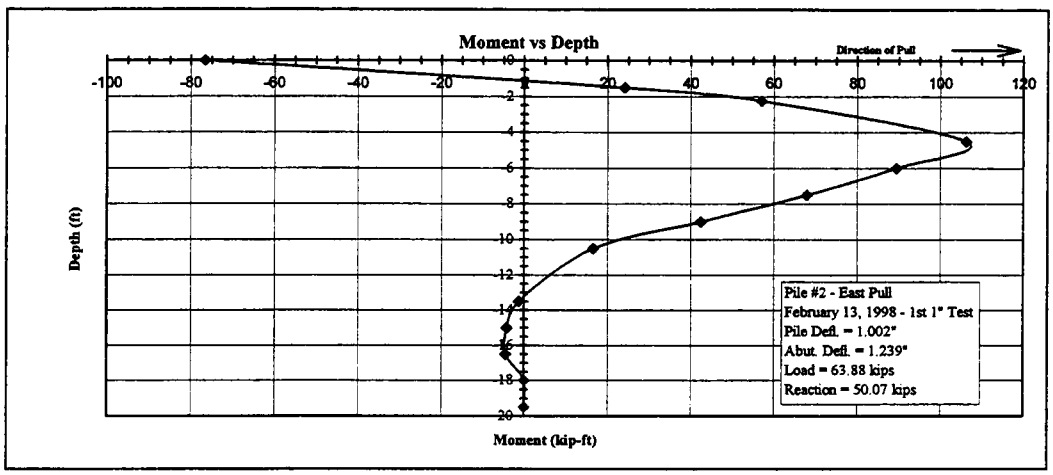
ONE INCH INCREMENT

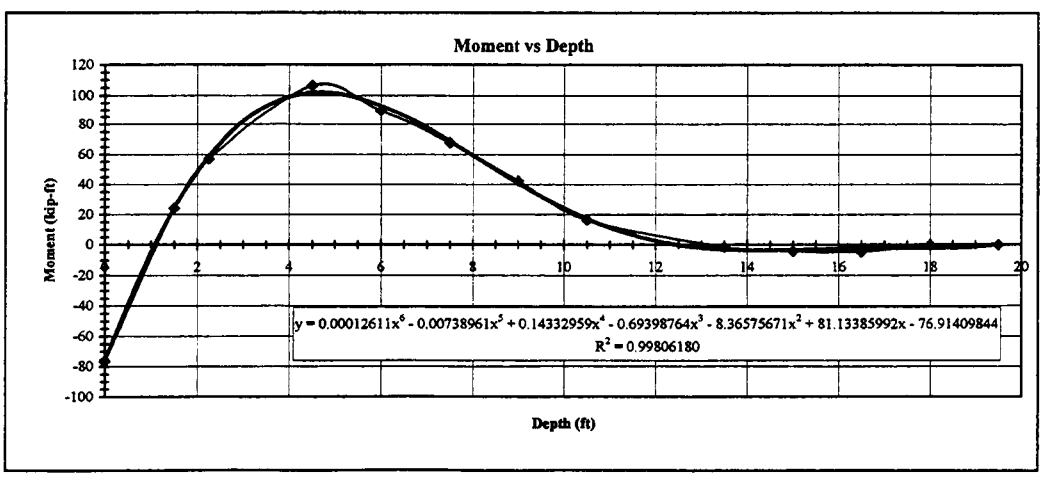
FOR FEBRUARY 13, 1998

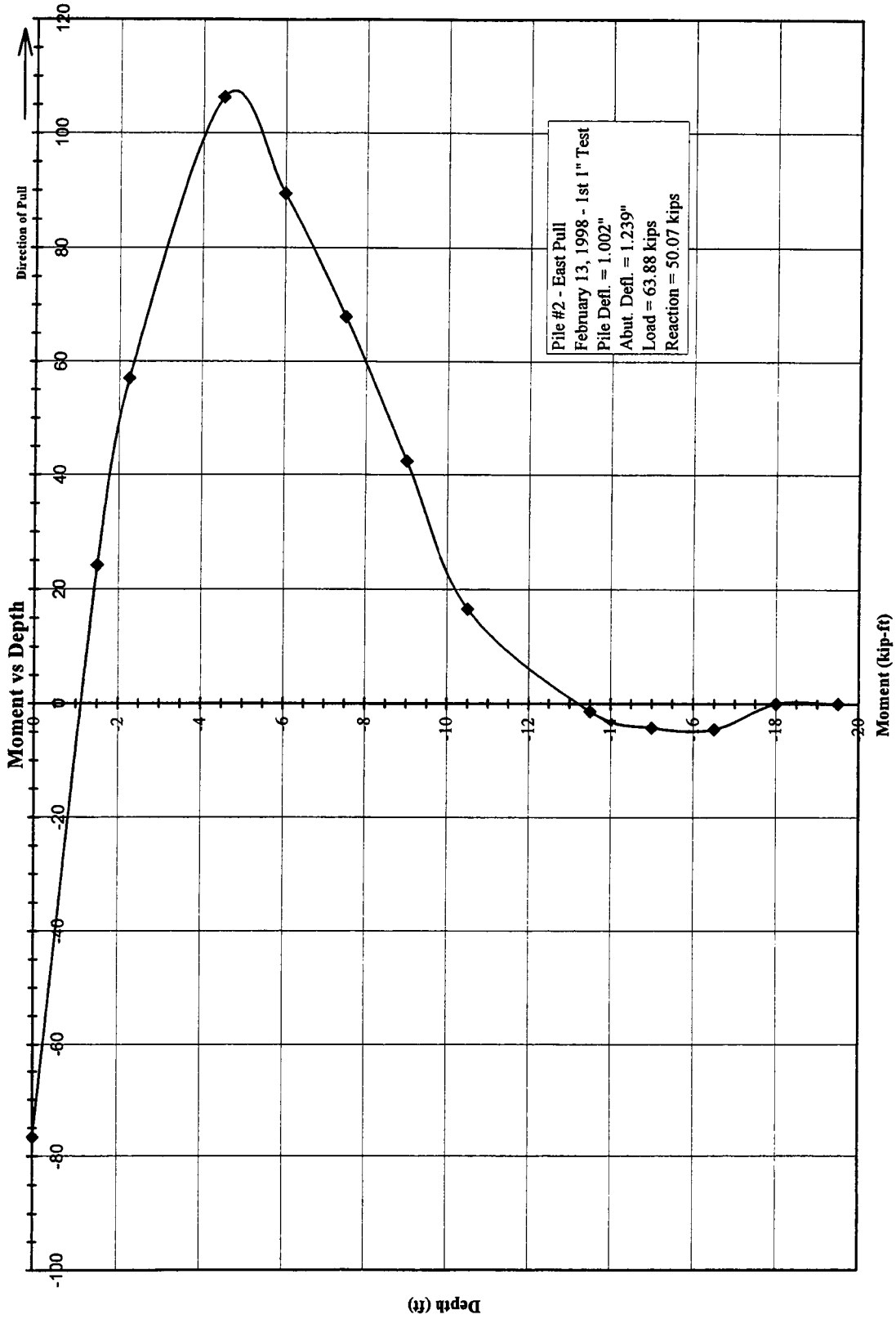
Moments

Date	02/13/98	
Time	17540-17550	sec
Load	63.88	kips
Reaction	50.07	kips
Abut. Defl.	1.239	inches
Pile Defl.	1.002	inches

Moment (kip-ft)	Depth (ft)	Station	Depth (ft)
-76.601	0	1	0
24.180	-1.5	3	1.5
57.009	-2.25	4	2.25
106.290	-4.5	6	4.5
89.438	-6	7	6
67.961	-7.5	8	7.5
42.407	-9	9	9
16.594	-10.5	10	10.5
-1.364	-13.5	12	13.5
-4.234	-15	13	15
-4.502	-16.5	14	16.5
0.000	-18	15	18
0.000	-19.5	16	19.5






Moment vs Depth
Direction of Full
Pile #2 - East Pull
February 13, 1998 - 1st 1" Test
Pile Defl. = 1.002"
Abut. Defl. = 1.239"
Load = 63.88 kips
Reaction = 50.07 kips
Moment (kip-ft)
Depth (ft)
120
100
80
60
40
20
0
-20
-40
-60
-80
-100
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20

Equations

Date	02/13/98		
Time	17540-17550	sec	
Load	63.88	kips	
Reaction	50.07	kips	
Abut. Defl.	1.239	inches	
Pile Defl.	1.002	inches	$= C_2$
Modulus of Elasticity (E) =	33000	ksi	
Moment of Inertia (I) =	223	in^4	

Deflection Equation:
From 0' to 19.5'

$$y = 5.28794E\text{-}10\ x^8 + -4.1314E\text{-}08\ x^7 + 1.12186E\text{-}06\ x^6 + -8.14792E\text{-}06\ x^5 + -0.0001637\ x^4 + 0.003175235\ x^3 + -0.009030273\ x^2 + (C_1)\ x + 1.002\ (C_2)$$

Slope Equation:
From 0' to 19.5'

$$y = 3.52529E\text{-}10\ x^7 + -2.40998E\text{-}08\ x^6 + 5.60931E\text{-}07\ x^5 + -3.39497E\text{-}06\ x^4 + -5.45667E\text{-}05\ x^3 + 0.000793809\ x^2 + -0.001505046\ x + (C_1)$$

Moment Equation:
From 0' to 19.5'

$$\mathbf{y = 0.00012611\ x^6 + -0.00738961\ x^5 + 0.14332959\ x^4 + -0.69398764\ x^3 + -8.36575671\ x^2 + 81.13385992\ x + -76.91409844}$$

Shear Equation:
From 0' to 19.5'

$$y = 0.00075666\ x^5 + -0.03694805\ x^4 + 0.57331836\ x^3 + -2.08196292\ x^2 + -16.73151342\ x + 81.13385992$$

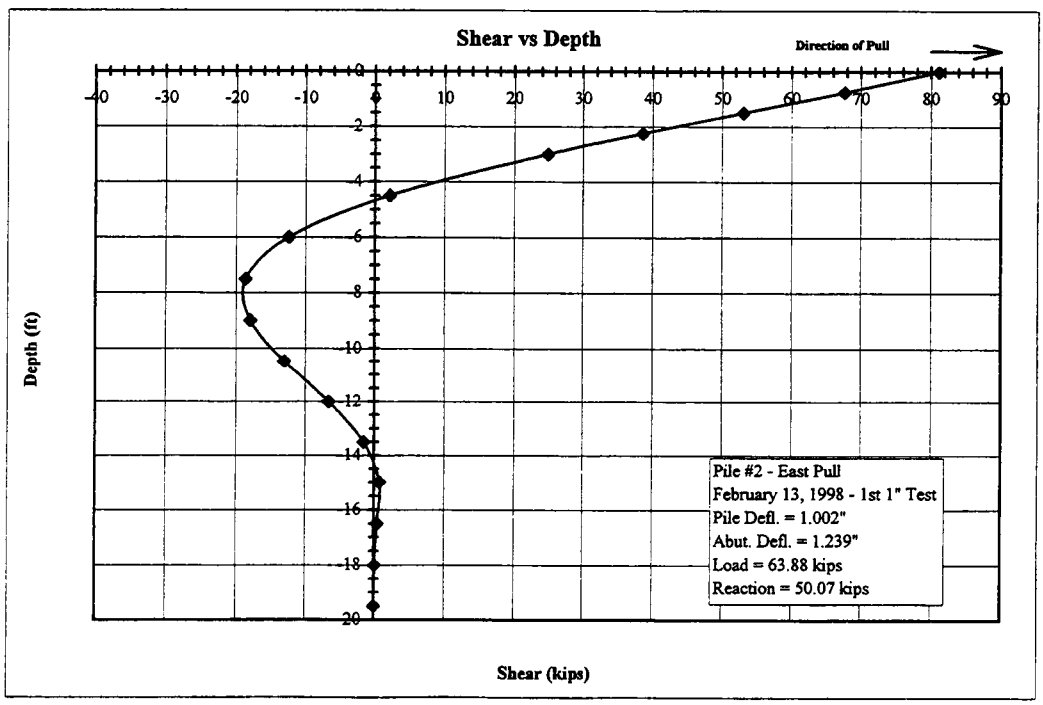
Pressure Equation:
From 0' to 19.5'

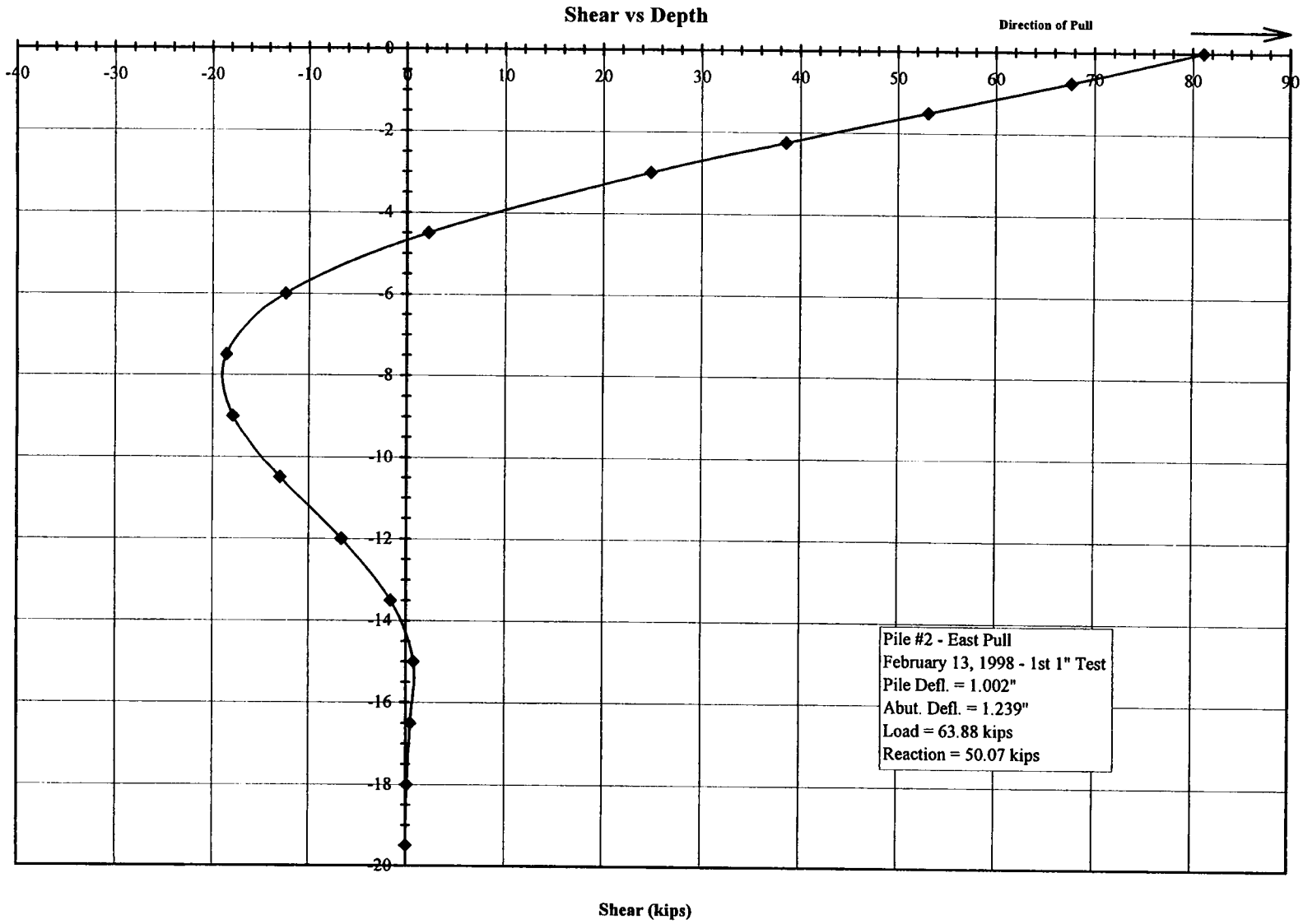
$$y = 0.0037833\ x^4 + -0.1477922\ x^3 + 1.71995508\ x^2 + -4.16392584\ x + -16.73151342$$

Shear

Date	02/13/98	
Time	17540-17550	sec
Load	63.88	kips
Reaction	50.07	kips
Abut. Defl.	1.239	inches
Pile Defl.	1.002	inches

Shear (kips)	Depth (ft)	Station	Depth (ft)
81.134	0	1	0
67.644	-0.75	2	0.75
53.106	-1.5	3	1.5
38.575	-2.25	4	2.25
24.872	-3	5	3
2.171	-4.5	6	4.5
-12.370	-6	7	6
-18.544	-7.5	8	7.5
-17.876	-9	9	9
-12.931	-10.5	10	10.5
-6.626	-12	11	12
-1.544	-13.5	12	13.5
0.763	-15	13	15
0.456	-16.5	14	16.5
0.105	-18	15	18
0.000	-19.5	16	19.5

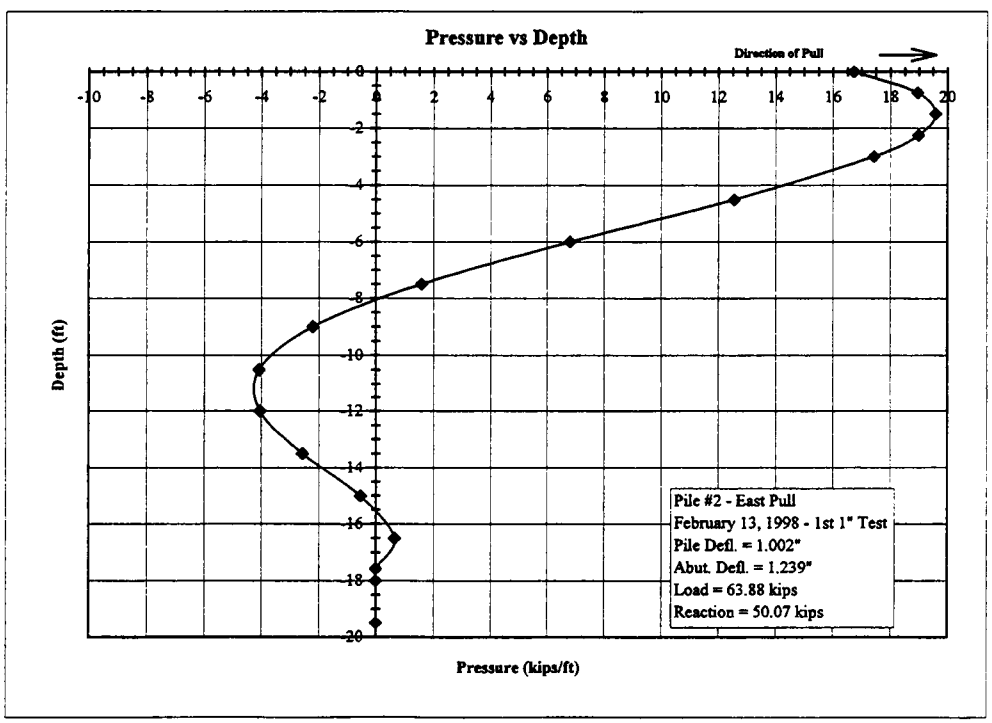


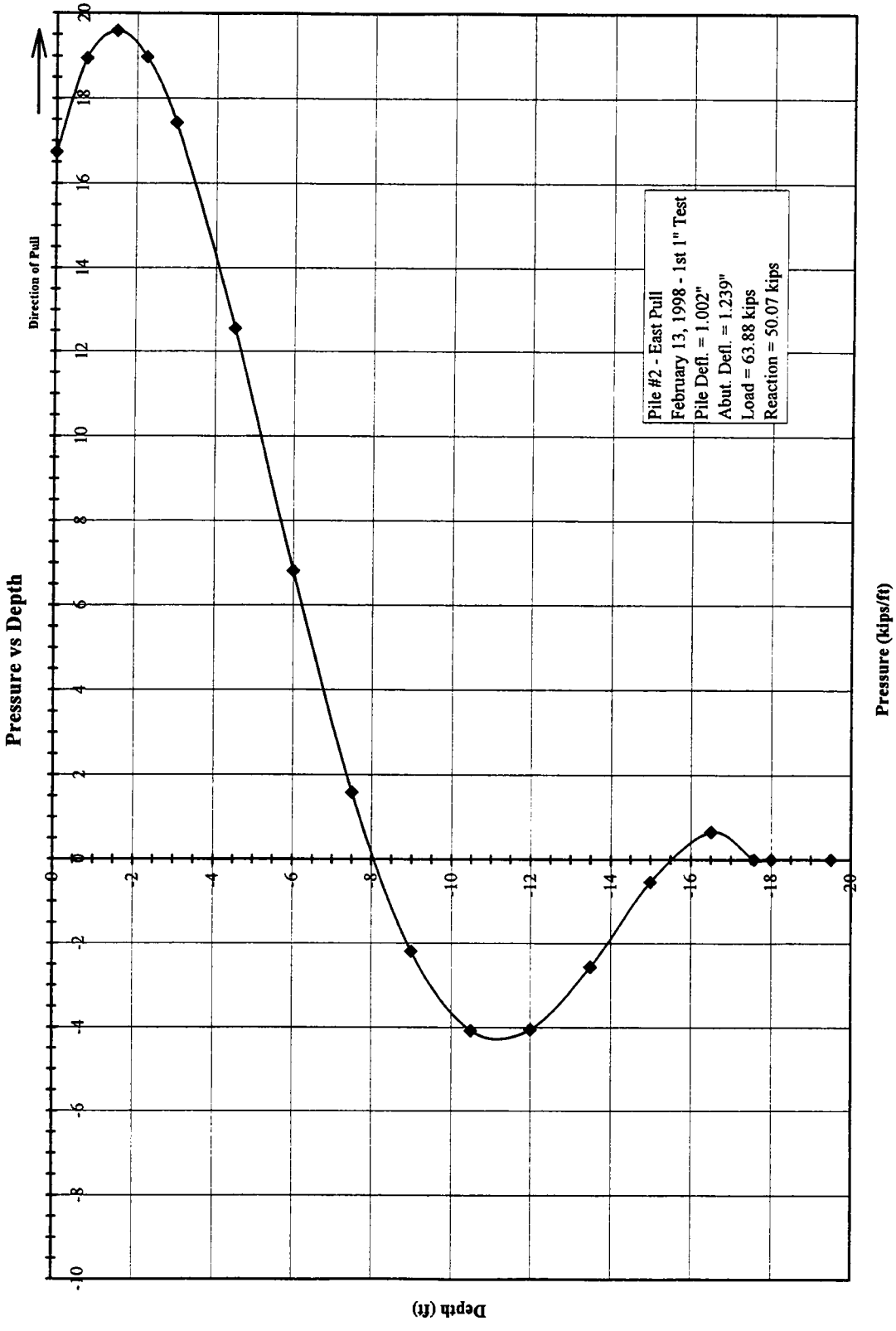

Shear vs Depth
Direction of Pull
-40
-30
-20
-10
0
10
20
30
40
50
60
70
80
90
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Shear (kips)
Pile #2 - East Pull
February 13, 1998 - 1st 1" Test
Pile Defl. = 1.002"
Abut. Defl. = 1.239"
Load = 63.88 kips
Reaction = 50.07 kips

Pressure

Date	02/13/98	
Time	17540-17550	sec
Load	63.88	kips
Reaction	50.07	kips
Abut. Defl.	1.239	inches
Pile Defl.	1.002	inches

Pressure (kips/ft)	Depth (ft)	Station	Depth (ft)
16.732	0	1	0
18.948	-0.75	2	0.75
19.587	-1.5	3	1.5
18.980	-2.25	4	2.25
17.428	-3	5	3
12.556	-4.5	6	4.5
6.817	-6	7	6
1.593	-7.5	8	7.5
-2.191	-9	9	9
-4.071	-10.5	10	10.5
-4.040	-12	11	12
-2.556	-13.5	12	13.5
-0.530	-15	13	15
0.661	-16.5	14	16.5
0.000	-17.57422		17.57422
0.000	-18	15	18
0.000	-19.5	16	19.5




Pressure vs Depth
Direction of Pull
Pile #2 - East Pull
February 13, 1998 - 1st 1" Test
Pile Defl. = 1.002"
Abut. Defl. = 1.239"
Load = 63.88 kips
Reaction = 50.07 kips
Depth (ft)
Pressure (kips/ft)

Statics

Date	02/13/98	
Time	17540-17550	sec
Load	63.88	kips
Reaction	50.07	kips
Abut. Defl.	1.239	inches
Pile Defl.	1.002	inches

Load by Statics

Pressure Equation

$y =$ 0.00378330 $x^4 +$ -0.14779220 $x^3 +$ 1.719955080 $x^2 +$ -4.16392584 $x +$ -16.73151342

Roots for pressure equation

x = -2.02162 x = 8.05133 x = 15.46043 x = 17.57422

Integrate to check statics:

From 0 to 8.05133

= -0.000756660 $x^5 +$ 0.036948050 $x^4 +$ -0.573318360 $x^3 +$ 2.081962920 $x^2 +$ 16.73151342 x

= 100.1072708 - 0 = **100.10727 kips**

From 8.05133 to 15.46043

= -0.000756660 $x^5 +$ 0.036948050 $x^4 +$ -0.573318360 $x^3 +$ 2.081962920 $x^2 +$ 16.73151342 x

= 80.25292364 - 100.1072708 = **-19.85435 kips**

From 15.46043 to 17.5742

= -0.000756660 $x^5 +$ 0.036948050 $x^4 +$ -0.573318360 $x^3 +$ 2.081962920 $x^2 +$ 16.73151342 x

= 81.18893668 - 80.25292364 = **0.93601 kips**

From Pressure	=	**81.189 kips**
Measured	=	**63.88 kips**
% difference	=	***27.10*** **%**

Moment by Statics

M_{GRND} = (Reaction)(x) - (Load)(y) - (Weight)(e)

x =	6.75	ft	(distance from centerline of the pile to centerline of the reaction)
y =	4.2083	ft	(distance from centerline of the load to the ground surface)
e =	0.237	in	($e = \Delta_{abut} - \Delta_{pile}$)
Weight =	65.0	kips	

M_{GRND} = 337.973 - 268.828 - 1.284 = **67.860 kip-ft**

Measured = 76.601 kip-ft

% difference = *11.41* %

Deflections

Date	02/13/98	
Time	17540-17550	sec
Load	63.88	kips
Reaction	50.07	kips
Abut. Defl.	1.239	inches
Pile Defl.	1.002	inches

@ x = 0

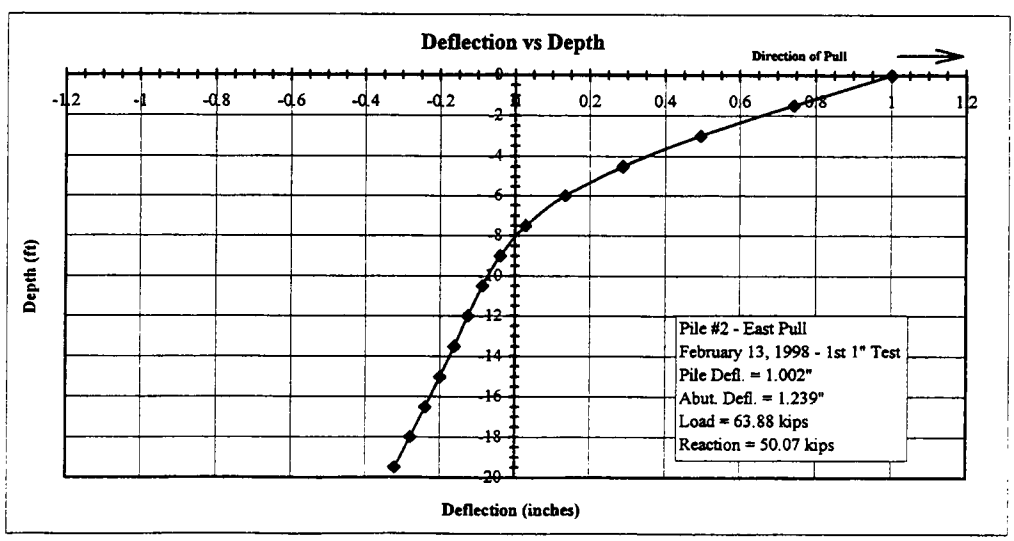
y = 0.000 + 0 C_1 + C_2 = 1.002

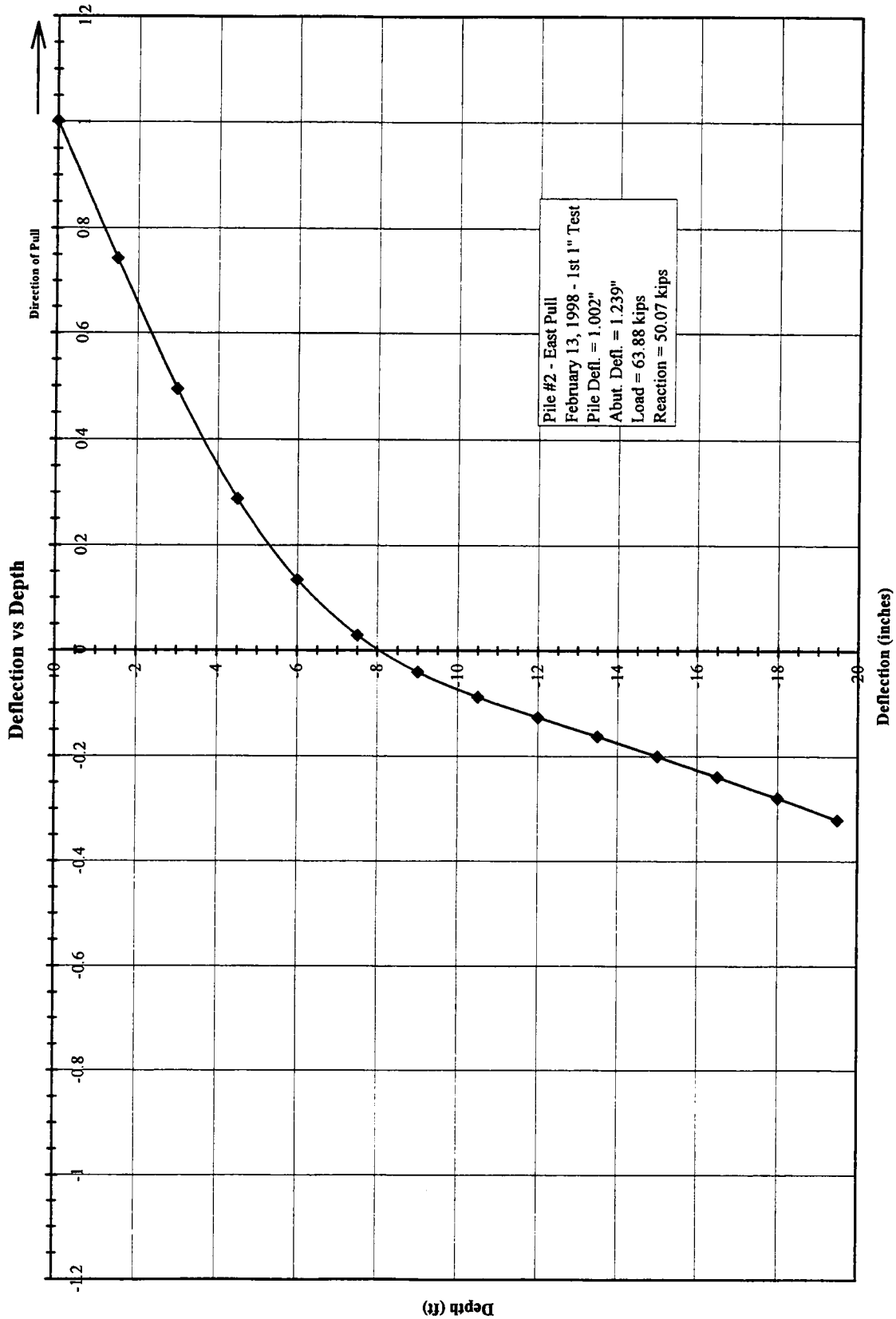
@ x = 8.05133

y = 0.333 + 8.05133 C_1 + 1.002 = 0

C_1 = -0.1657617 = -0.0138135 radians

Deflection (in)	Depth (ft)	Station	Depth (ft)
1.002	0	1	0
0.743	-1.5	2	1.5
0.495	-3	3	3
0.288	-4.5	4	4.5
0.134	-6	5	6
0.029	-7.5	6	7.5
-0.040	-9	7	9
-0.088	-10.5	8	10.5
-0.126	-12	9	12
-0.162	-13.5	10	13.5
-0.199	-15	11	15
-0.239	-16.5	12	16.5
-0.279	-18	13	18
-0.321	-19.5	14	19.5




Deflection vs Depth
Direction of Pull
Pile #2 - East Pull
February 13, 1998 - 1st 1" Test
Pile Defl. = 1.002"
Abut. Defl. = 1.239"
Load = 63.88 kips
Reaction = 50.07 kips
Deflection (inches)
Depth (ft)
1.2
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
-1.2
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20

APPENDIX P

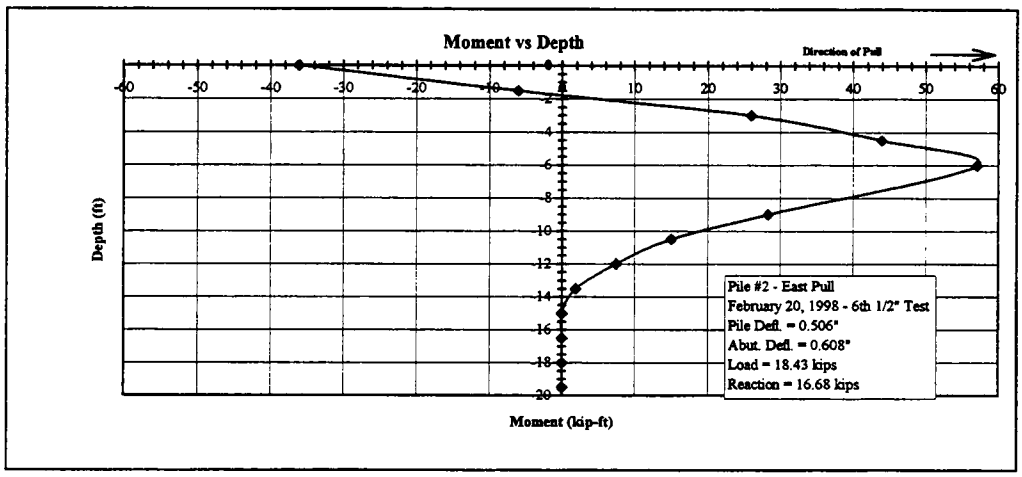
ONE-HALF INCH INCREMENT

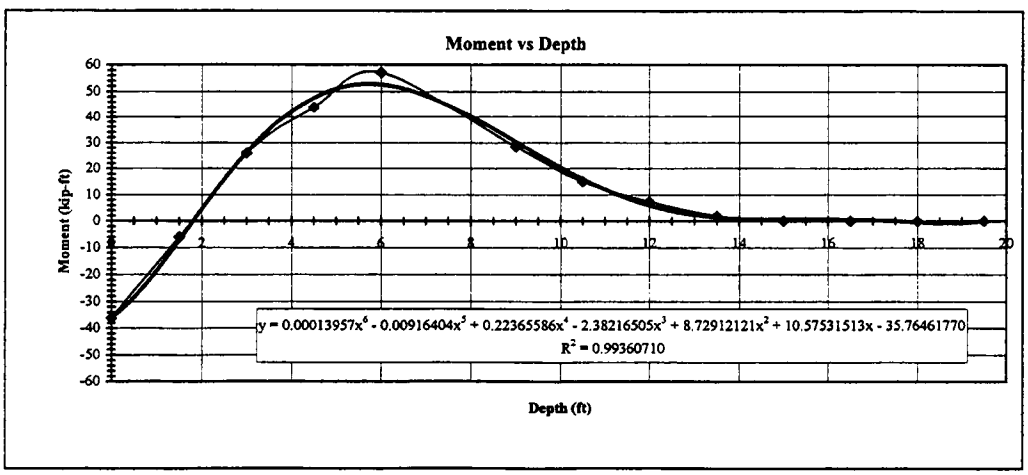
FOR FEBRUARY 24, 1998

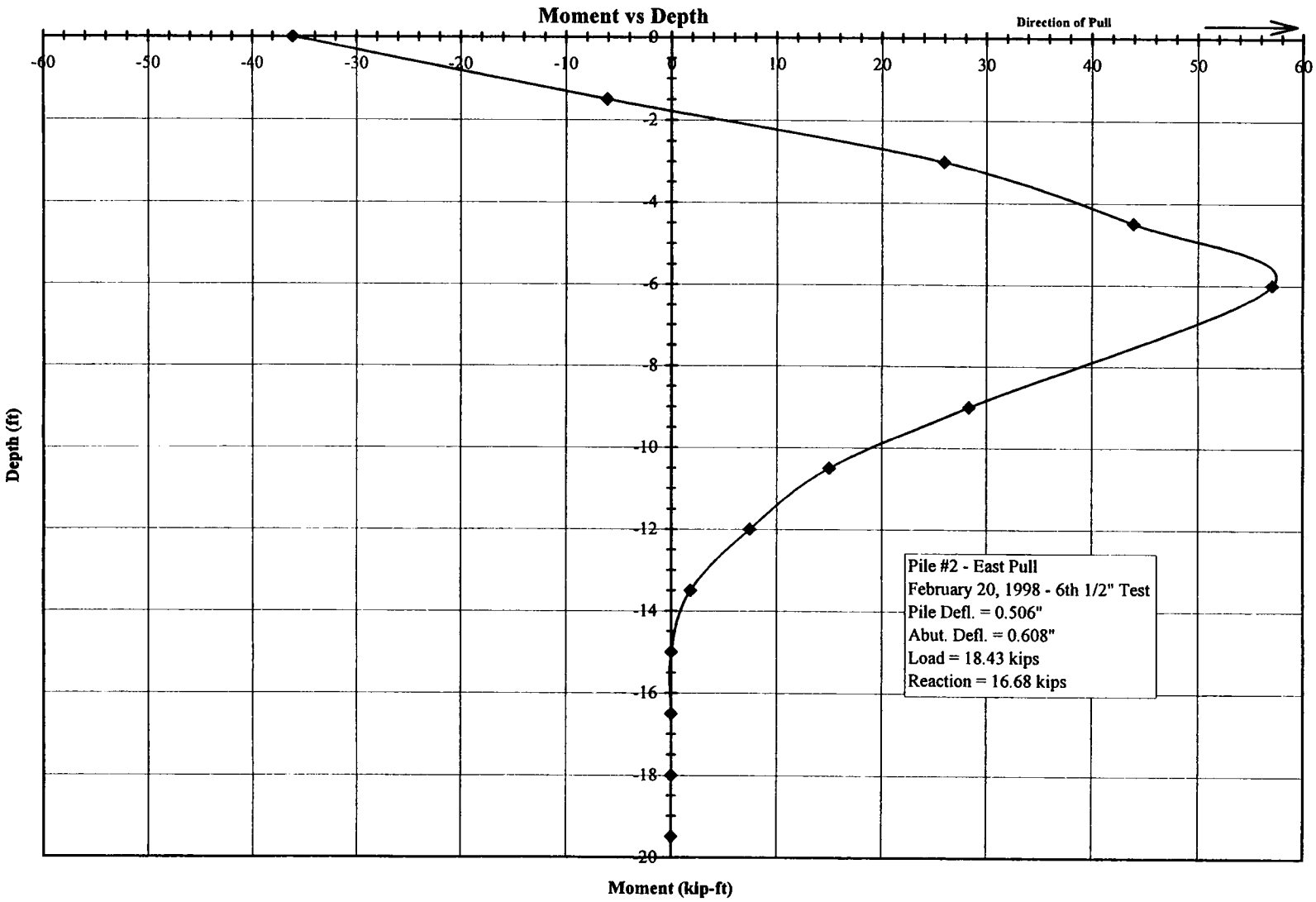
Moments

Date	02/24/98	
Time	5720-5730	sec
Load	18.43	kips
Reaction	16.68	kips
Abut. Defl.	0.608	inches
Pile Defl.	0.506	inches

Moment (kip-ft)	Depth (ft)	Station	Depth (ft)
-36.093	0	1	0
-6.087	-1.5	3	1.5
25.937	-3	5	3
43.906	-4.5	6	4.5
57.077	-6	7	6
28.325	-9	9	9
15.016	-10.5	10	10.5
7.447	-12	11	12
1.855	-13.5	12	13.5
0.000	-15	13	15
0.000	-16.5	14	16.5
0.000	-18	15	18
0.000	-19.5	16	19.5






Moment vs Depth
Direction of Pull
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Moment (kip-ft)
Pile #2 - East Pull
February 20, 1998 - 6th 1/2" Test
Pile Defl. = 0.506"
Abut. Defl. = 0.608"
Load = 18.43 kips
Reaction = 16.68 kips

Equations

Date	02/24/98	
Time	5720-5730	sec
Load	18.43	kips
Reaction	16.68	kips
Abut. Defl.	0.608	inches
Pile Defl.	0.506	inches $= C_2$
Modulus of Elasticity (E) =	33000	ksi
Moment of Inertia (I) =	223	in^4

Deflection Equation:
From 0' to 19.5'

$$y = 5.85233E\text{-}10\,x^8 + -5.12345E\text{-}08\,x^7 + 1.75059E\text{-}06\,x^6 + -2.79683E\text{-}05\,x^5 + 0.00017081\,x^4 + 0.000413873\,x^3 + -0.004199026\,x^2 + (C_1)\,x + 0.506\,(C_2)$$

Slope Equation:
From 0' to 19.5'

$$y = 3.90155E\text{-}10\,x^7 + -2.98868E\text{-}08\,x^6 + 8.75294E\text{-}07\,x^5 + -1.16535E\text{-}05\,x^4 + 5.69368E\text{-}05\,x^3 + 0.000103468\,x^2 + -0.000699838\,x + (C_1)$$

Moment Equation:
From 0' to 19.5'

$$y = \mathbf{0.00013957\,x^6 + -0.00916404\,x^5 + 0.22365586\,x^4 + -2.38216505\,x^3 + 8.72912121\,x^2 + 10.57531513\,x + -35.76461770}$$

Shear Equation:
From 0' to 19.5'

$$y = 0.00083742\,x^5 + -0.04582020\,x^4 + 0.89462344\,x^3 + -7.14649515\,x^2 + 17.45824242\,x + 10.57531513$$

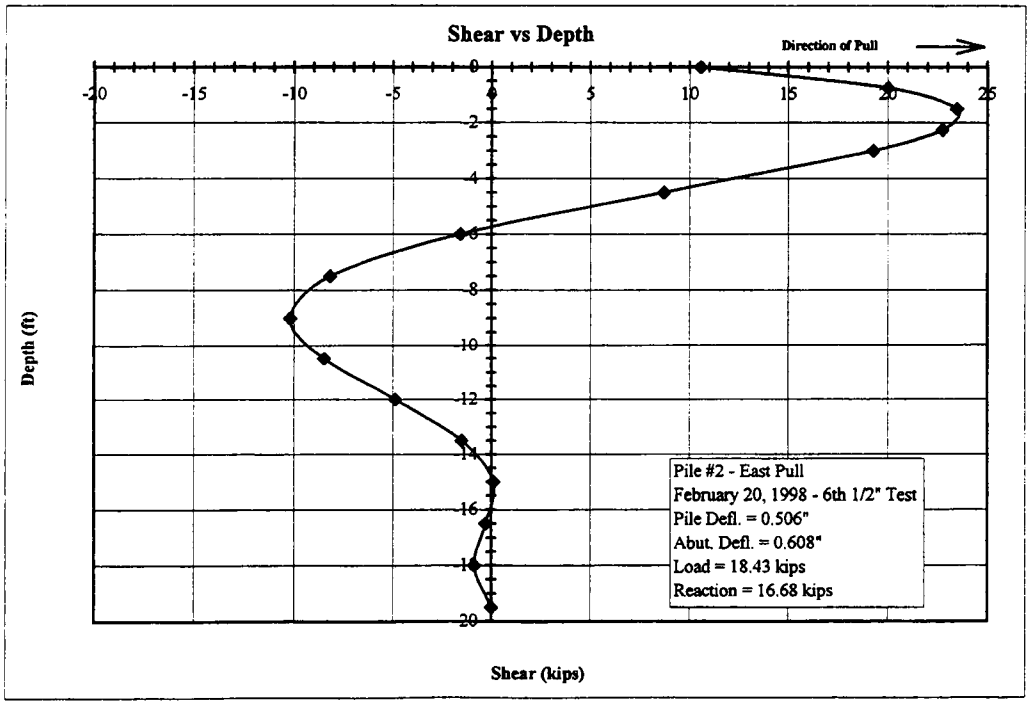
Pressure Equation:
From 0' to 19.5'

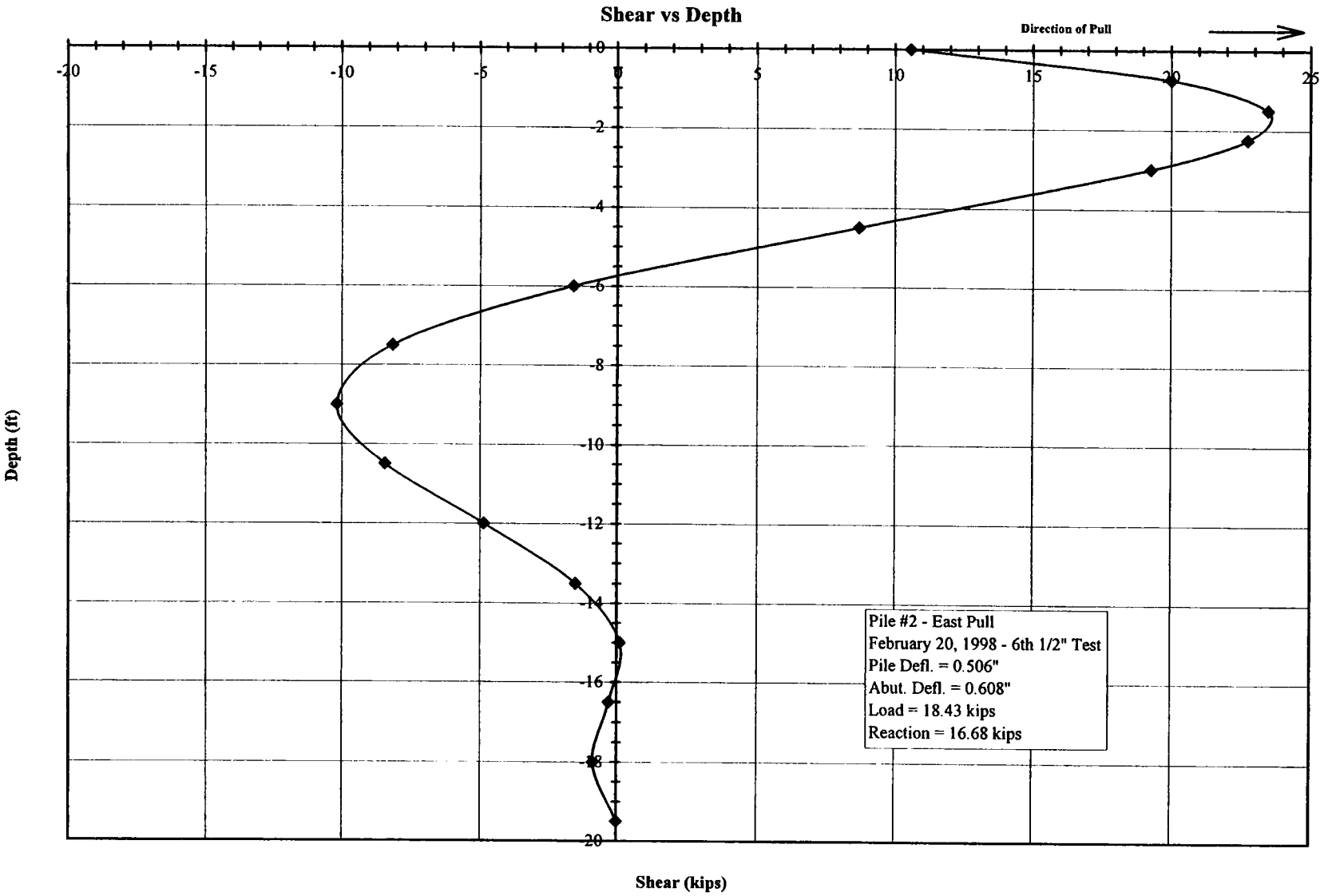
$$y = 0.0041871\,x^4 + -0.1832808\,x^3 + 2.68387032\,x^2 + -14.2929903\,x + 17.45824242$$

Shear

Date	02/24/98	
Time	5720-5730	sec
Load	18.43	kips
Reaction	16.68	kips
Abut. Defl.	0.608	inches
Pile Defl.	0.506	inches

Shear (kips)	Depth (ft)	Station	Depth (ft)
10.575	0	1	0
20.012	-0.75	2	0.75
23.477	-1.5	3	1.5
22.742	-2.25	4	2.25
19.278	-3	5	3
8.700	-4.5	6	4.5
-1.582	-6	7	6
-8.165	-7.5	8	7.5
-10.164	-9	9	9
-8.445	-10.5	10	10.5
-4.863	-12	11	12
-1.497	-13.5	12	13.5
0.110	-15	13	15
-0.284	-16.5	14	16.5
-0.856	-18	15	18
0.000	-19.5	16	19.5

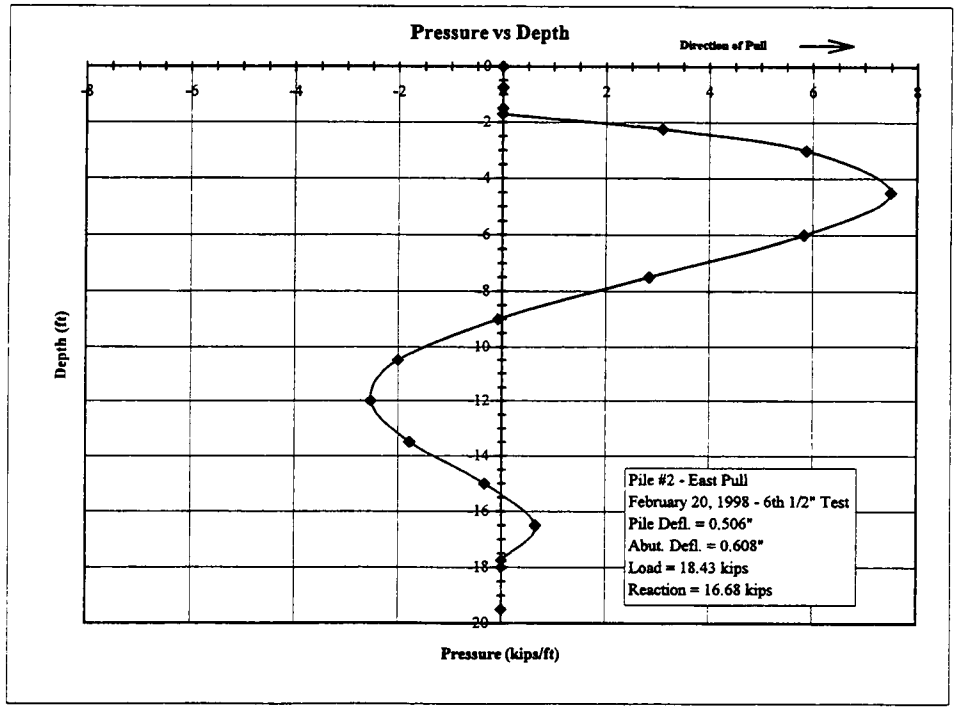


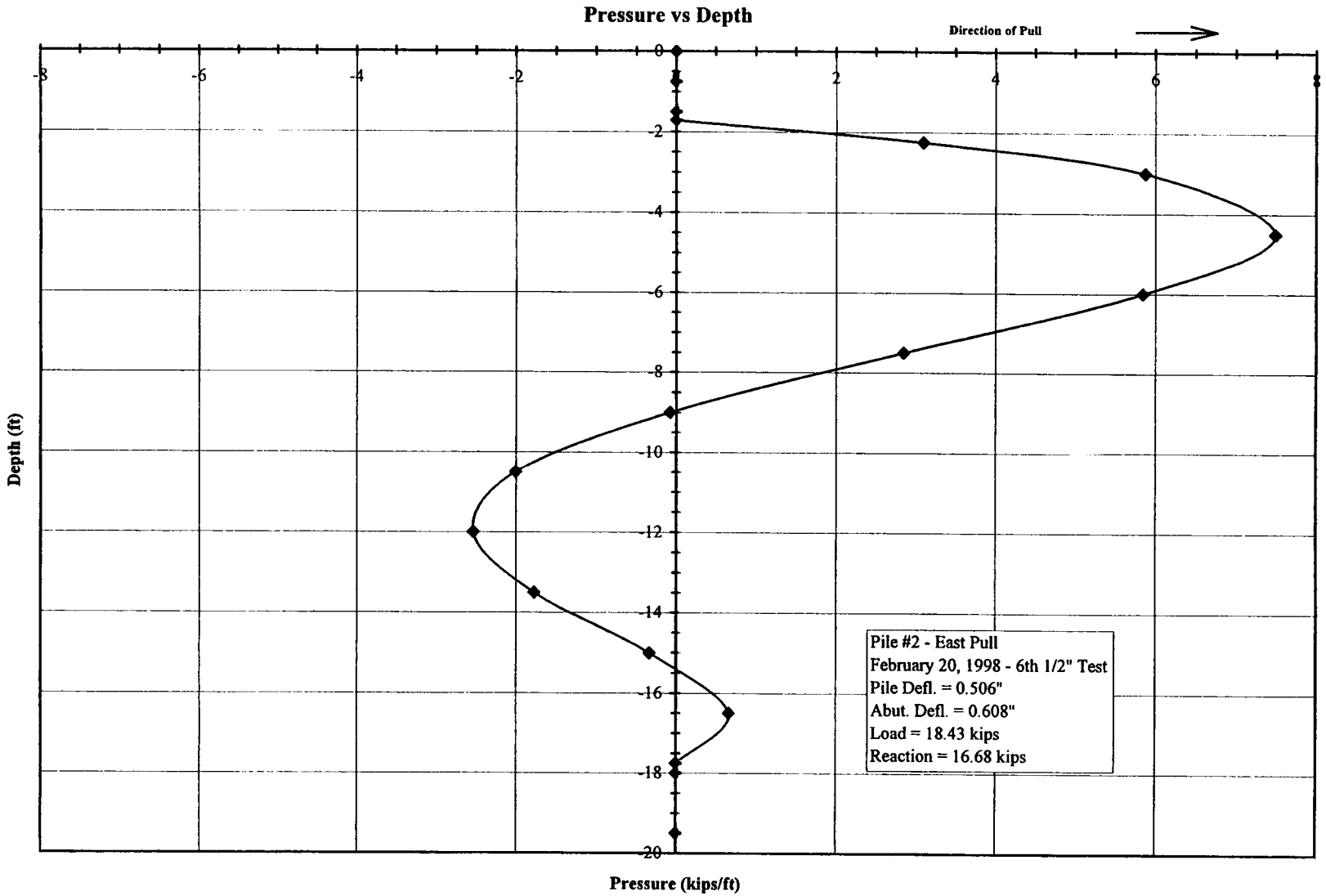

Shear vs Depth
Direction of Pull
-20
-15
-10
-5
0
5
10
15
20
25
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Shear (kips)
Pile #2 - East Pull
February 20, 1998 - 6th 1/2" Test
Pile Defl. = 0.506"
Abut. Defl. = 0.608"
Load = 18.43 kips
Reaction = 16.68 kips

Pressure

Date	02/24/98	
Time	5720-5730	sec
Load	18.43	kips
Reaction	16.68	kips
Abut. Defl.	0.608	inches
Pile Defl.	0.506	inches

Pressure (kips/ft)	Depth (ft)	Station	Depth (ft)
0.000	0	1	0
0.000	-0.75	2	0.75
0.000	-1.5	3	1.5
0.000	-1.70767		1.70767
3.094	-2.25	4	2.25
5.875	-3	5	3
7.496	-4.5	6	4.5
5.843	-6	7	6
2.845	-7.5	8	7.5
-0.075	-9	9	9
-2.003	-10.5	10	10.5
-2.534	-12	11	12
-1.774	-13.5	12	13.5
-0.333	-15	13	15
0.665	-16.5	14	16.5
0.000	-17.74834		17.74834
0.000	-18	15	18
0.000	-19.5	16	19.5




Pressure vs Depth
Direction of Pull
-8
-6
-4
-2
0
2
4
6
8
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Pressure (kips/ft)
Pile #2 - East Pull
February 20, 1998 - 6th 1/2" Test
Pile Defl. = 0.506"
Abut. Defl. = 0.608"
Load = 18.43 kips
Reaction = 16.68 kips

Statics

Date	02/24/98	
Time	5720-5730	sec
Load	18.43	kips
Reaction	16.68	kips
Abut. Defl.	0.608	inches
Pile Defl.	0.506	inches

Load by Statics

Pressure Equation

$y =$ 0.00418710 $x^4 +$ -0.18328080 $x^3 +$ 2.683870320 $x^2 +$ -14.29299030 $x +$ 17.45824242

Roots for pressure equation

x = 1.70767 x = 8.95604 x = 15.36068 x = 17.74834

Integrate to check statics:

From 0 to 1.70767 (Assume to be zero due to previous pulls and no confinement at ground surface)

From 1.70767 to 8.95604

= -0.000837420 $x^5 +$ 0.045820200 $x^4 +$ -0.894623440 $x^3 +$ 7.146495150 $x^2 +$ -17.45824242 x

= 20.74060093 - -12.37276539 = **33.11337 kips**

From 8.95604 to 15.36068

= -0.000837420 $x^5 +$ 0.045820200 $x^4 +$ -0.894623440 $x^3 +$ 7.146495150 $x^2 +$ -17.45824242 x

= 10.40633097 - 20.74060093 = **-10.33427 kips**

From 15.36068 to 17.7483

= -0.000837420 $x^5 +$ 0.045820200 $x^4 +$ -0.894623440 $x^3 +$ 7.146495150 $x^2 +$ -17.45824242 x

= 11.4806625 - 10.40633097 = **1.07433 kips**

From Pressure	**=**	**23.853 kips**
Measured	**=**	**18.43 kips**
% difference	**=**	***29.43*** **%**

Moment by Statics

M_{GRND} = (Reaction)(x) - (Load)(y) - (Weight)(e)

x =	6.85	ft	(distance from centerline of the pile to centerline of the reaction)
y =	4.2083	ft	(distance from centerline of the load to the ground surface)
e =	0.102	in	($e = \Delta_{abut} - \Delta_{pile}$)
Weight =	65.0	kips	

M_{GRND} = 114.258 - 77.560 - 0.553 = **36.146 kip-ft**

Measured = 36.093 kip-ft

% difference = *0.15* %

Deflections

Date	02/24/98	
Time	5720-5730	sec
Load	18.43	kips
Reaction	16.68	kips
Abut. Defl.	0.608	inches
Pile Defl.	0.506	inches

@ x = 0

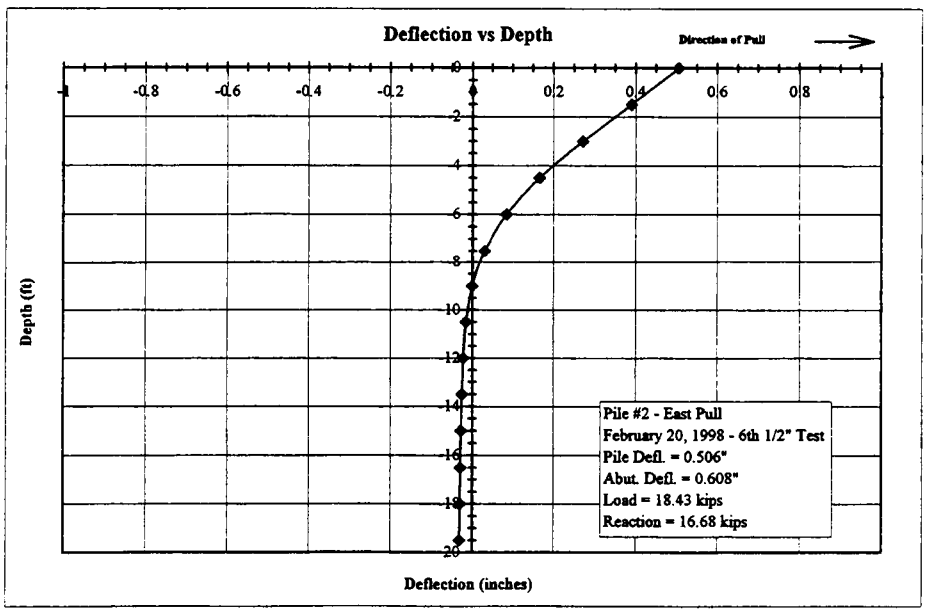
y = 0.000 + 0 C_1 + C_2 = 0.506

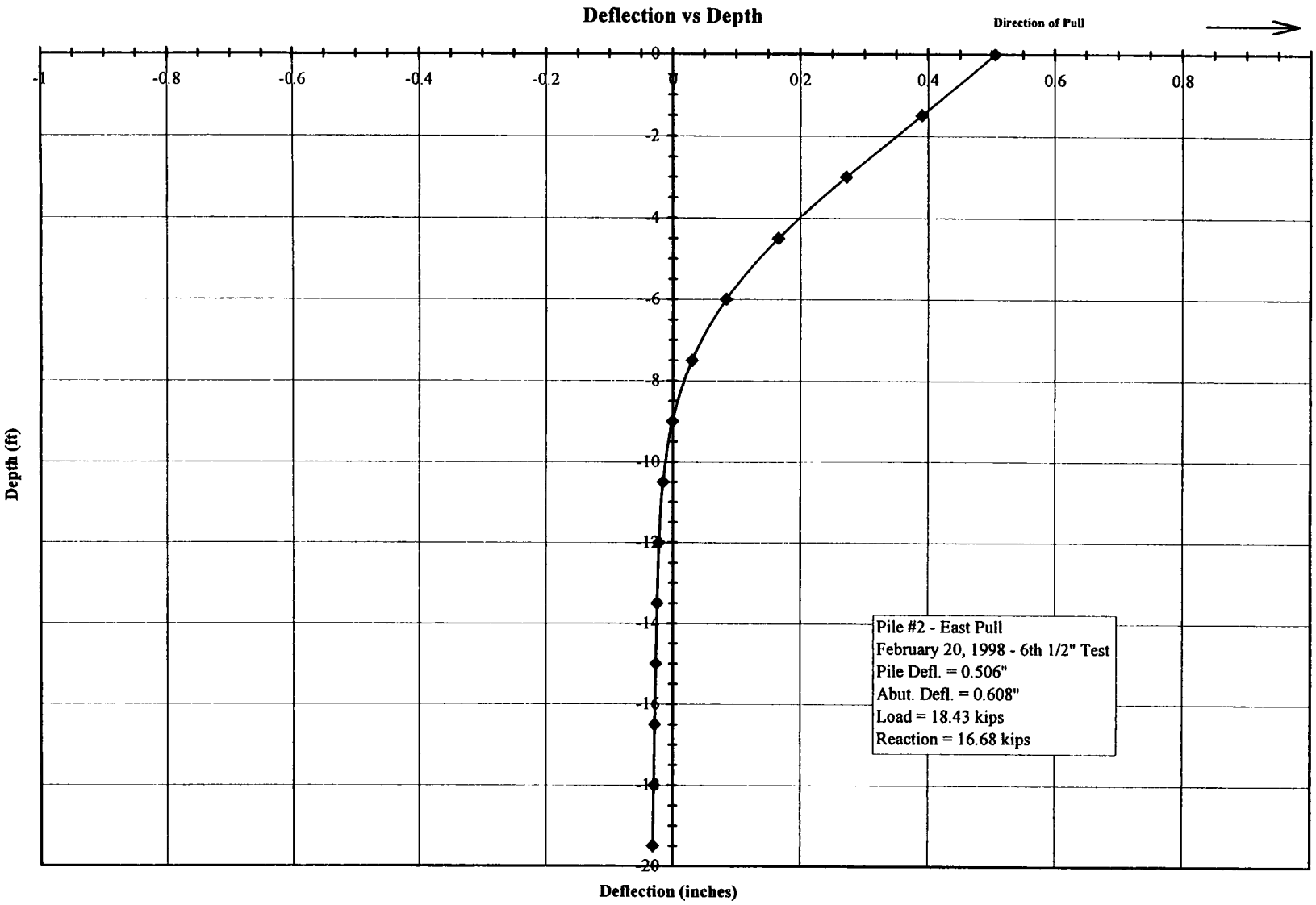
@ x = 8.95604

y = 0.139 + 8.95604 C_1 + 0.506 = 0

C_1 = -0.0719879 = -0.0059990 radians

Deflection (in)	Depth (ft)	Station	Depth (ft)
0.506	0	1	0
0.391	-1.5	2	1.5
0.272	-3	3	3
0.166	-4.5	4	4.5
0.085	-6	5	6
0.030	-7.5	6	7.5
-0.001	-9	7	9
-0.016	-10.5	8	10.5
-0.022	-12	9	12
-0.025	-13.5	10	13.5
-0.027	-15	11	15
-0.028	-16.5	12	16.5
-0.030	-18	13	18
-0.031	-19.5	14	19.5




Deflection vs Depth
Direction of Pull
-1
-0.8
-0.6
-0.4
-0.2
0
0.2
0.4
0.6
0.8
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Deflection (inches)
Pile #2 - East Pull
February 20, 1998 - 6th 1/2" Test
Pile Defl. = 0.506"
Abut. Defl. = 0.608"
Load = 18.43 kips
Reaction = 16.68 kips

APPENDIX Q

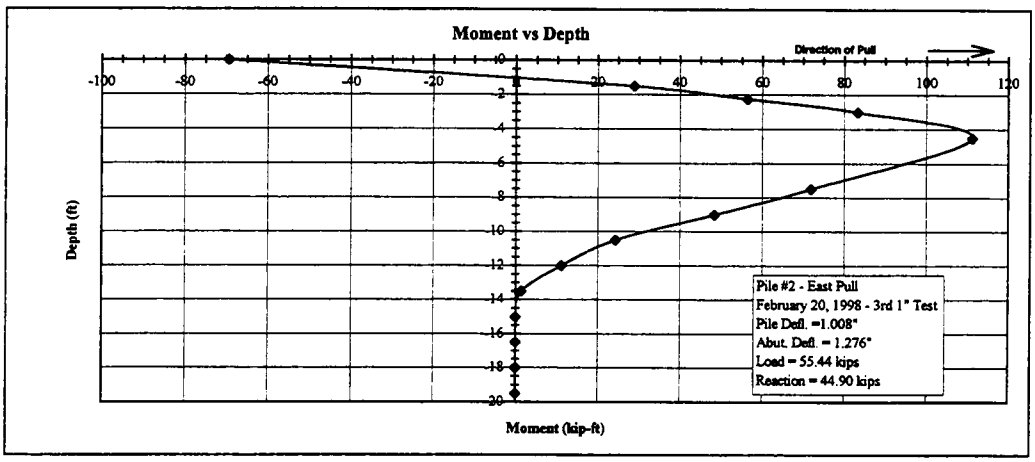
ONE INCH INCREMENT

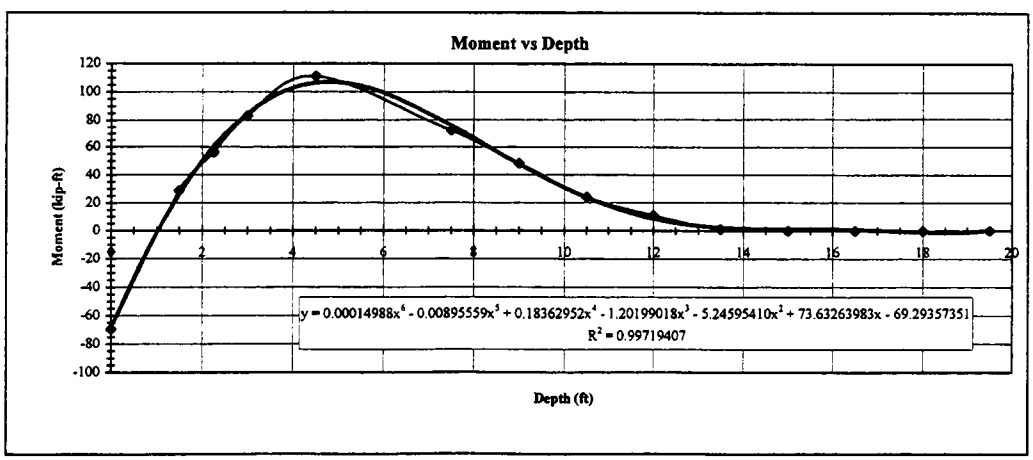
FOR FEBRUARY 24, 1998

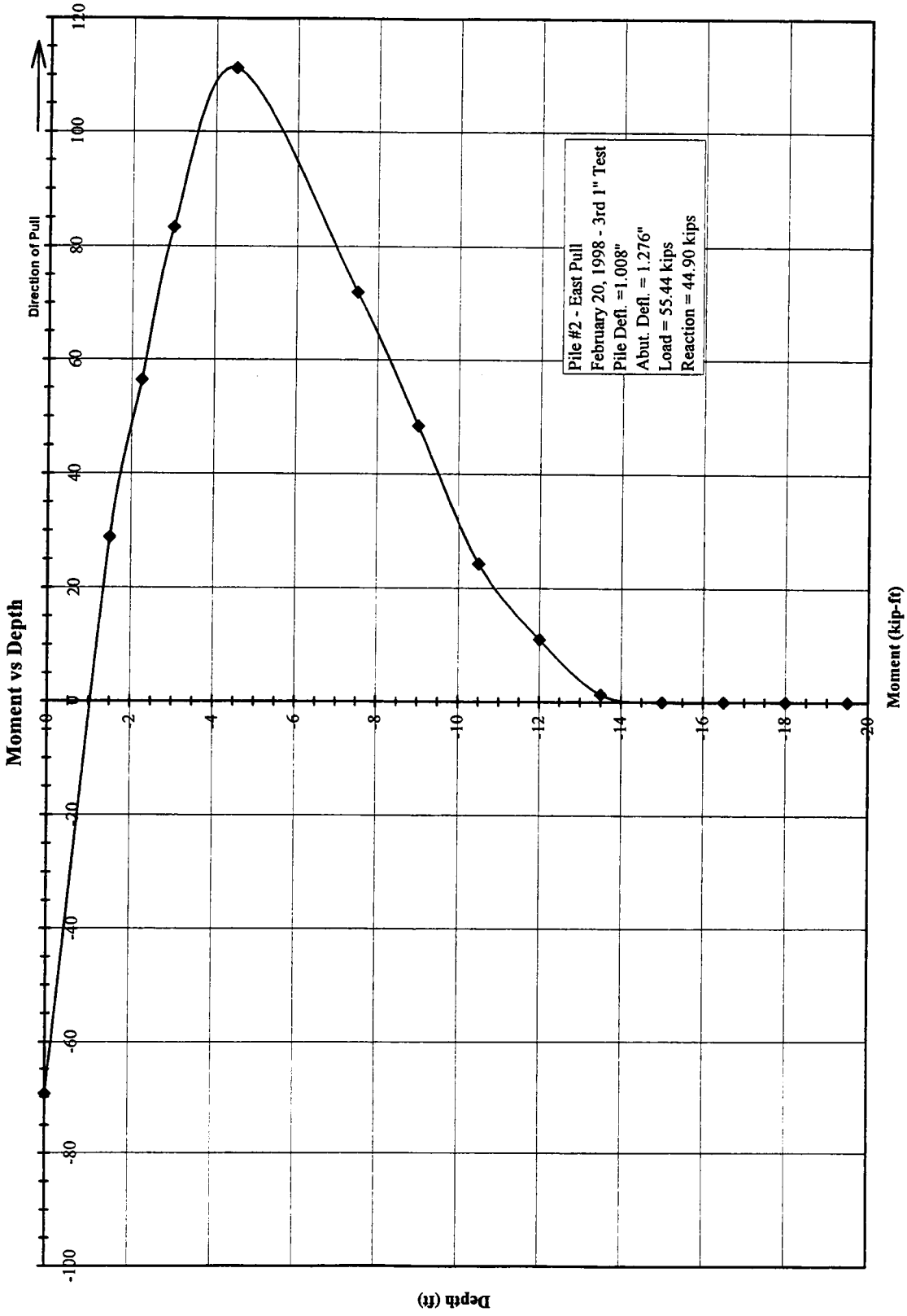
Moments

Date	02/24/98	
Time	15600-15610	sec
Load	55.44	kips
Reaction	44.90	kips
Abut. Defl.	1.276	inches
Pile Defl.	1.008	inches

Moment (kip-ft)	Depth (ft)	Station	Depth (ft)
-69.411	0	1	0
28.877	-1.5	3	1.5
56.405	-2.25	4	2.25
83.349	-3	5	3
111.255	-4.5	6	4.5
71.960	-7.5	8	7.5
48.464	-9	9	9
24.307	-10.5	10	10.5
11.039	-12	11	12
1.305	-13.5	12	13.5
0.000	-15	13	15
0.000	-16.5	14	16.5
0.000	-18	15	18
0.000	-19.5	16	19.5






Moment vs Depth
Direction of Pull
Pile #2 - East Pull
February 20, 1998 - 3rd 1" Test
Pile Defl. =1.008"
Abut. Defl. = 1.276"
Load = 55.44 kips
Reaction = 44.90 kips
Moment (kip-ft)
Depth (ft)
120
100
80
60
40
20
0
-20
-40
-60
-80
-100
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20

Equations

Date	02/24/98	
Time	15600-15610	sec
Load	55.44	kips
Reaction	44.90	kips
Abut. Defl.	1.276	inches
Pile Defl.	1.008	inches $= C_2$
Modulus of Elasticity (E) =	33000	ksi
Moment of Inertia (I) =	223	in^4

Deflection Equation:
From 0' to 19.5'

$$y = 6.28464E\text{-}10\ x^8 + -5.00691E\text{-}08\ x^7 + 1.4373E\text{-}06\ x^6 + -1.41122E\text{-}05\ x^5 + -0.000102652\ x^4 + 0.002881669\ x^3 + -0.008135568\ x^2 + (C_1)\ x + 1.008\ (C_2)$$

Slope Equation:
From 0' to 19.5'

$$y = 4.18976E\text{-}10\ x^7 + -2.9207E\text{-}08\ x^6 + 7.18648E\text{-}07\ x^5 + -5.8801E\text{-}06\ x^4 + -3.42174E\text{-}05\ x^3 + 0.000720417\ x^2 + -0.001355928\ x + (C_1)$$

Moment Equation:
From 0' to 19.5'

$$\mathbf{y = 0.00014988\ x^6 + -0.00895559\ x^5 + 0.18362952\ x^4 + -1.20199018\ x^3 + -5.24595410\ x^2 + 73.63263983\ x + -69.29357351}$$

Shear Equation:
From 0' to 19.5'

$$y = 0.00089928\ x^5 + -0.04477795\ x^4 + 0.73451808\ x^3 + -3.60597054\ x^2 + -10.49190820\ x + 73.63263983$$

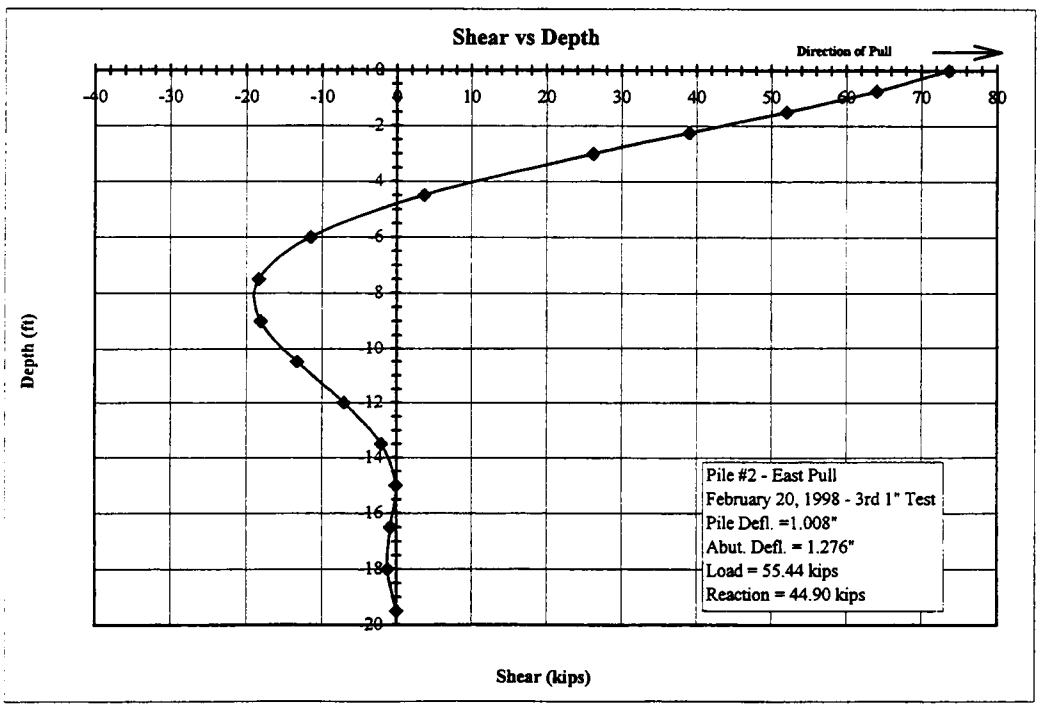
Pressure Equation:
From 0' to 19.5'

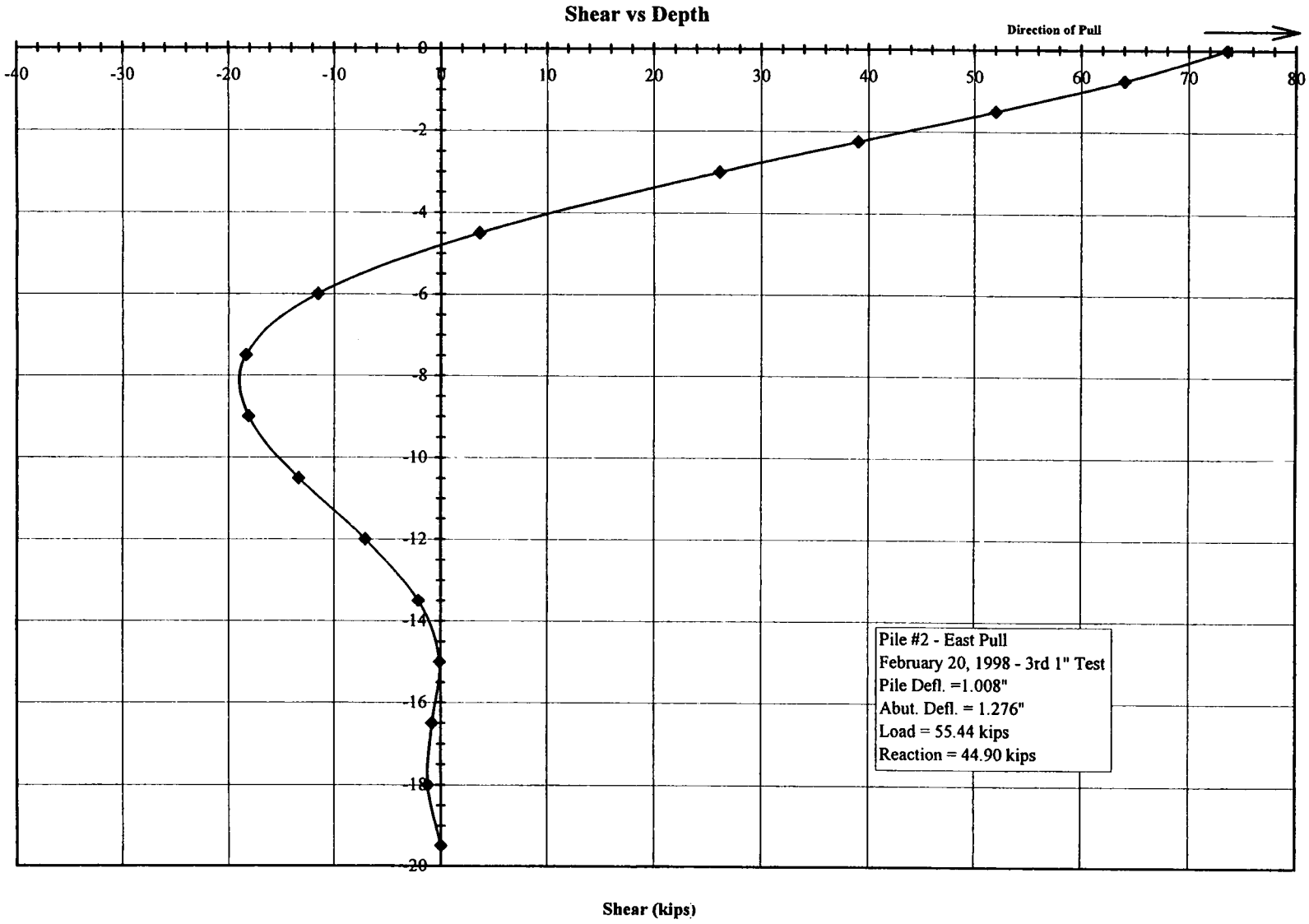
$$y = 0.0044964\ x^4 + -0.1791118\ x^3 + 2.20355424\ x^2 + -7.21194108\ x + -10.49190820$$

Shear

Date	02/24/98	
Time	15600-15610	sec
Load	55.44	kips
Reaction	44.90	kips
Abut. Defl.	1.276	inches
Pile Defl.	1.008	inches

Shear (kips)	Depth (ft)	Station	Depth (ft)
73.633	0	1	0
64.031	-0.75	2	0.75
52.040	-1.5	3	1.5
39.041	-2.25	4	2.25
26.127	-3	5	3
3.629	-4.5	6	4.5
-11.517	-6	7	6
-18.358	-7.5	8	7.5
-18.101	-9	9	9
-13.299	-10.5	10	10.5
-7.029	-12	11	12
-2.069	-13.5	12	13.5
-0.084	-15	13	15
-0.804	-16.5	14	16.5
-1.206	-18	15	18
0.000	-19.5	16	19.5

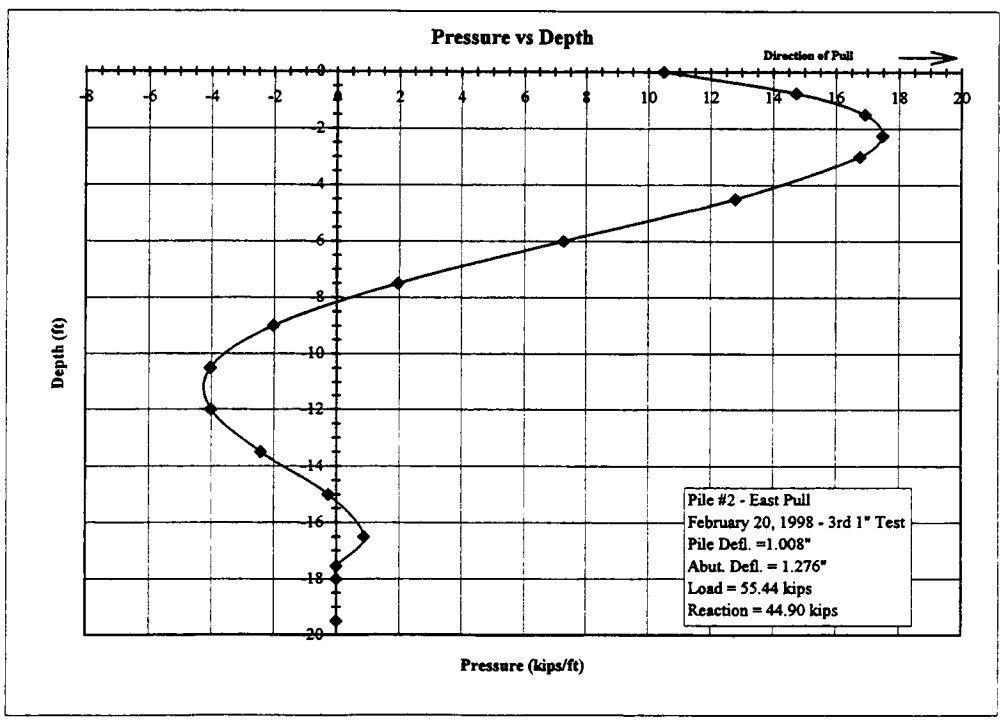


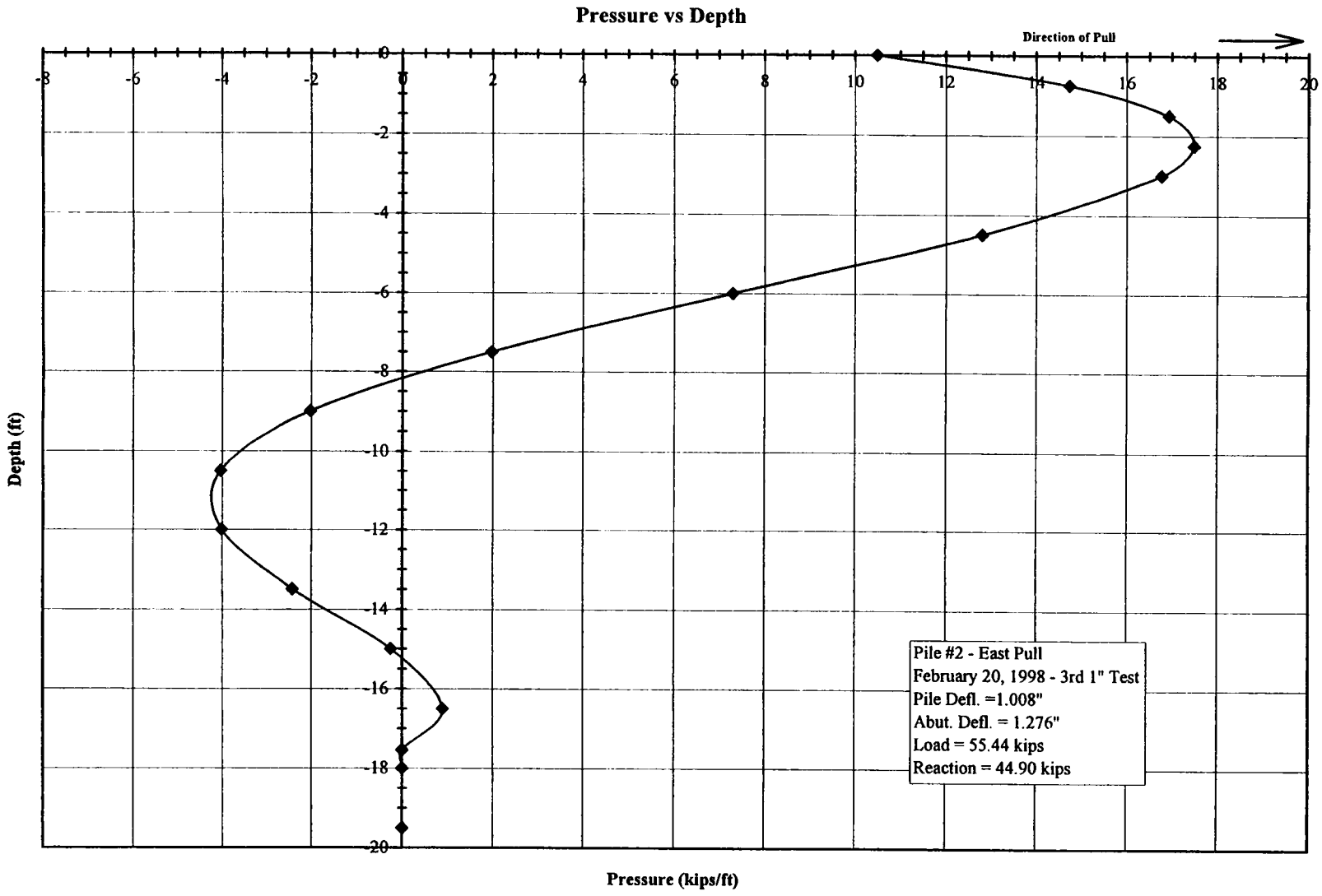

Shear vs Depth
Direction of Pull
-40
-30
-20
-10
0
10
20
30
40
50
60
70
80
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth (ft)
Shear (kips)
Pile #2 - East Pull
February 20, 1998 - 3rd 1" Test
Pile Defl. =1.008"
Abut. Defl. = 1.276"
Load = 55.44 kips
Reaction = 44.90 kips

Pressure

Date	02/24/98	
Time	15600-15610	sec
Load	55.44	kips
Reaction	44.90	kips
Abut. Defl.	1.276	inches
Pile Defl.	1.008	inches

Pressure (kips/ft)	Depth (ft)	Station	Depth (ft)
10.492	0	1	0
14.736	-0.75	2	0.75
16.934	-1.5	3	1.5
17.488	-2.25	4	2.25
16.768	-3	5	3
12.801	-4.5	6	4.5
7.296	-6	7	6
1.967	-7.5	8	7.5
-2.017	-9	9	9
-4.034	-10.5	10	10.5
-4.009	-12	11	12
-2.411	-13.5	12	13.5
-0.257	-15	13	15
0.890	-16.5	14	16.5
0.000	-17.54275		17.54275
0.000	-18	15	18
0.000	-19.5	16	19.5




Pressure vs Depth
Direction of Pull
Depth (ft)
Pressure (kips/ft)
Pile #2 - East Pull
February 20, 1998 - 3rd 1" Test
Pile Defl. =1.008"
Abut. Defl. = 1.276"
Load = 55.44 kips
Reaction = 44.90 kips

Statics

Date	02/24/98	
Time	15600-15610	sec
Load	55.44	kips
Reaction	44.90	kips
Abut. Defl.	1.276	inches
Pile Defl.	1.008	inches

Load by Statics

Pressure Equation

$y = 0.00449640\ x^4 + -0.17911180\ x^3 + 2.203554240\ x^2 + -7.21194108\ x + -10.4919082$

Roots for pressure equation

x = -1.07215 x = 8.15974 x = 15.20415 x = 17.54275

Integrate to check statics:

From 0 to 8.15974

$= -0.000899280\ x^5 + 0.044777950\ x^4 + -0.734518080\ x^3 + 3.605970540\ x^2 + 10.4919082\ x$

= 92.62252037 - 0 = **92.62252 kips**

From 8.15974 to 15.20415

$= -0.000899280\ x^5 + 0.044777950\ x^4 + -0.734518080\ x^3 + 3.605970540\ x^2 + 10.4919082\ x$

= 73.69059319 - 92.62252037 = **-18.93193 kips**

From 15.20415 to 17.5428

$= -0.000899280\ x^5 + 0.044777950\ x^4 + -0.734518080\ x^3 + 3.605970540\ x^2 + 10.4919082\ x$

= 75.06664564 - 73.69059319 = **1.37605 kips**

From Pressure = 75.067 kips
Measured = 55.44 kips
% difference = *35.40* %

Moment by Statics

M_{GRND} = (Reaction)(x) - (Load)(y) - (Weight)(e)

x =	6.85	ft	(distance from centerline of the pile to centerline of the reaction)
y =	4.2083	ft	(distance from centerline of the load to the ground surface)
e =	0.268	in	($e = \Delta_{abut} - \Delta_{pile}$)
Weight =	65.0	kips	

M_{GRND} = 307.565 - 233.310 - 1.452 = **72.803 kip-ft**

Measured = 69.411 kip-ft

% difference = *4.89* %

Deflections

Date	02/24/98	
Time	15600-15610	sec
Load	55.44	kips
Reaction	44.90	kips
Abut. Defl.	1.276	inches
Pile Defl.	1.008	inches

@ x = 0

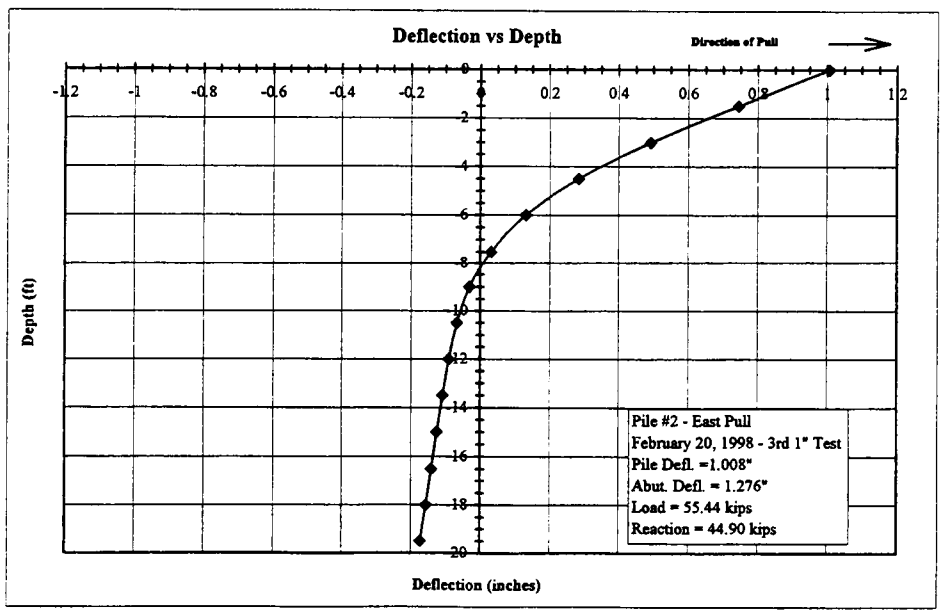
y = 0.000 + 0 C_1 + C_2 = 1.008

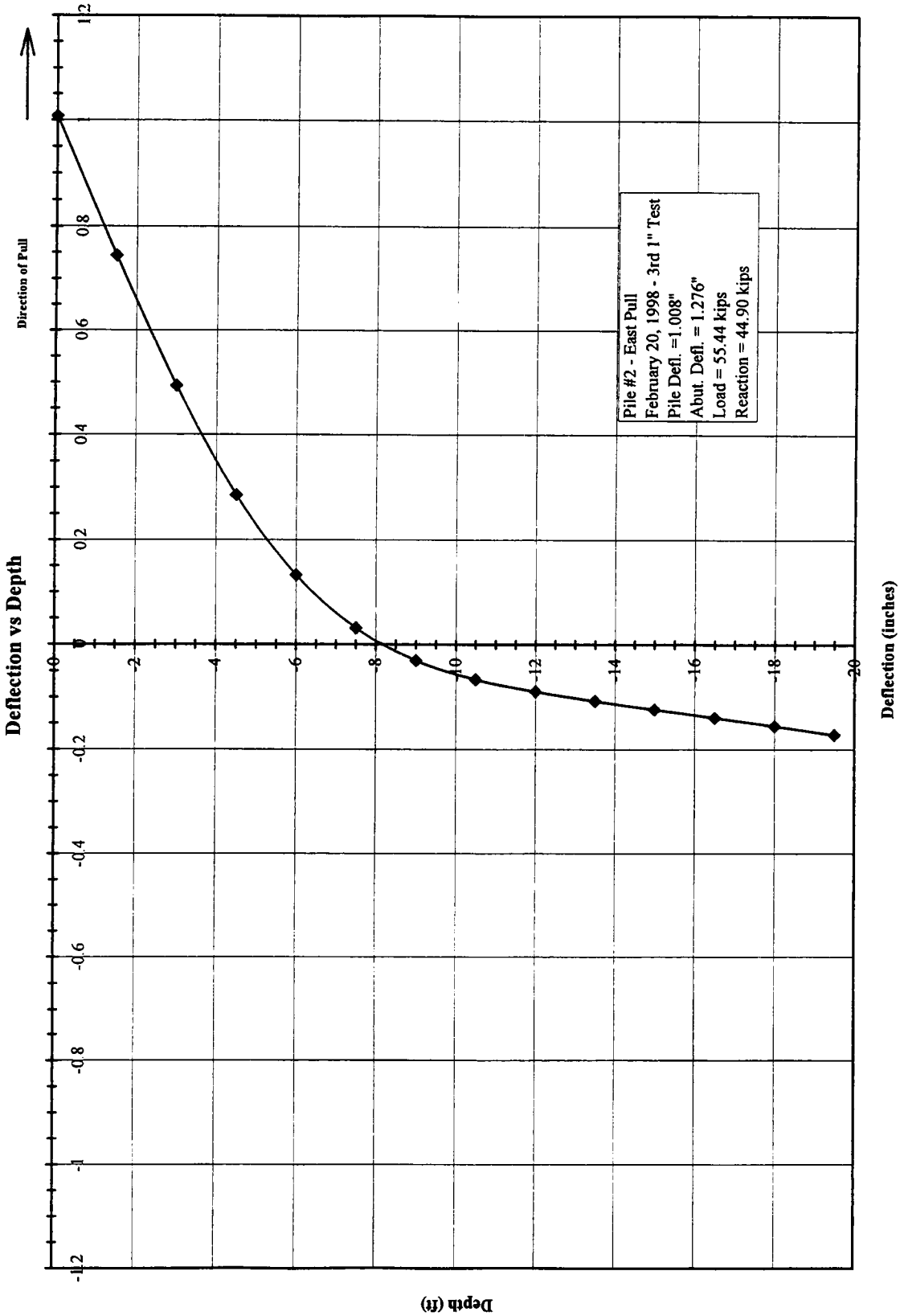
@ x = 8.15974

y = 0.374 + 8.15974 C_1 + 1.008 = 0

C_1 = -0.1694110 = -0.0141176 radians

Deflection (in)	Depth (ft)	Station	Depth (ft)
1.008	0	1	0
0.745	-1.5	2	1.5
0.494	-3	3	3
0.286	-4.5	4	4.5
0.132	-6	5	6
0.031	-7.5	6	7.5
-0.030	-9	7	9
-0.066	-10.5	8	10.5
-0.089	-12	9	12
-0.107	-13.5	10	13.5
-0.124	-15	11	15
-0.140	-16.5	12	16.5
-0.155	-18	13	18
-0.172	-19.5	14	19.5




Deflection vs Depth
Direction of Pull
Deflection (inches)
Depth (ft)
Pile #2 - East Pull
February 20, 1998 - 3rd 1" Test
Pile Defl. =1.008"
Abut. Defl. = 1.276"
Load = 55.44 kips
Reaction = 44.90 kips

VITA

Eric Michael Bonanno was born in Miami, Florida on January 26, 1974 and was raised in Athens, Tennessee. After graduating from McMinn County High School in May, 1992, he attended Cleveland State Community College, Cleveland, Tennessee. He received an Associate of Science degree from Cleveland State Community College in May, 1994. He transferred to The University of Tennessee, Knoxville in June of 1994 where in May, 1997 he received a Bachelor of Science degree in Civil Engineering. He began graduate school the same month and received his Master of Science degree in Civil Engineering in December, 1998.