5-1999

Behavior of integral abutment piles

Samer N. Al-Jamal

To the Graduate Council:

I am submitting herewith a thesis written by Samer N. Al-Jamal entitled "Behavior of integral abutment piles." I have examined the final electronic copy of this thesis for form and content and recommend that it be accepted in partial fulfillment of the requirements for the degree of Master of Science, with a major in Civil Engineering.

David W Goodpasture, Major Professor

We have read this thesis and recommend its acceptance:

Accepted for the Council:
Carolyn R. Hodges

Vice Provost and Dean of the Graduate School

(Original signatures are on file with official student records.)

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BEHAVIOR OF INTEGRAL ABUTMENT PILES

A Thesis
Presented for the
Master of Science Degree
The University of Tennessee, Knoxville

Samer N. Al-Jamal
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ABSTRACT

The Tennessee Department of Transportation (TDOT) has routinely designed and constructed jointless bridges with integral abutments for many years. Due the constant need of constructing longer jointless bridges and more accurate design approach, TDOT has sponsored a research project with University of Tennessee, Knoxville. The project involved five steel H-piles to be driven into ground with integral abutment slabs cast on top.

The first four piles were instrumented with weldable strain gages and pressure sensors, while the last pile was only instrumented with weldable strain gages. A slow lateral loading simulated the thermal movement of the bridge substructure. The testing program consisted of field tests that achieved a ground deflection of the pile of ½" and 1", and tests involved cyclic loading and loading to a point where failure warranted the determination of testing. The deflections, load, and reaction were monitored during testing.

The data were analyzed using elastic theory. Moment vs. depth curves were constructed using a six-order polynomial to best fit the moment points. The pressure vs. depth curves were obtained by differentiating the moment vs. depth equation twice. Another set of pressure vs. depth curves was constructed using the pressure sensor data. The purpose of the pressure vs. depth curves was the determination of the zero pressure point. The zero pressure points obtained from both sets of pressure vs. depth curves were analyzed and compared. Although different factors affected the accuracy of the field data, this study provided a better understanding of piles supporting integral abutments.

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CHAPTER 1

INTRODUCTION

The introduction of moment distribution method, in May 1930 by Hardy Cross (1), started a new generation of bridge construction. Bridges with more than one unit and statically indeterminate bridges became easier to design, and the use of continuous jointless bridge construction started to grow.

The design of integral-abutment bridges involves two areas, the design of integral-abutments and the piles supporting the integral abutments. The design of integral abutments has been based on judgment, experimentation, and observations. The reason behind the absence of a rational design approach was the complexity of the analysis and the many variables involved in the design of the integral abutments. More details about the design of integral abutments can be found in a recent thesis by Jay Lewis (12). Designing integral abutment piles is also not straightforward. The non-linearity of the soil-pile interaction makes the problem arduous to solve. In addition, the fact that the soil response depends on the deflection of the pile, and that the deflected shape is a function of the soil response added more complexity to the problem (1).

The introduction of a new program named, *Laterally Loaded Pile Analysis* Program (LPILE), gave promising results in predicting the behavior of laterally loaded piles. The Tennessee Department of Transportation routinely designs and constructs jointless bridges for short to medium bridge length (1). TDOT uses

COM624P, which is equivalent to LPILE, for designing integral abutment piles. This program analyzes laterally loaded piles for given soil properties and different displacements of the pile head.

Due to the current need for designing longer jointless bridges than what have been designed using the current approach, TDOT sponsored a research project with University of Tennessee, Knoxville. The purpose of this research was to investigate the behavior of laterally loaded piles supporting integral abutments. The study was most focused on the pile-abutment interaction and the soil-pile interaction. Of particular interest to TDOT was determination of the location of the zero lateral pressure below the ground level. Therefore, the piles were instrumented with pressure sensors and strain gages to indicate the depth of the zero pressure point.

This thesis focuses on analyzing the pressure data from pressure sensors and constructing pressure vs. depth curves. In addition, this thesis studies how the zero pressure point is related to the displacement of the pile head. Pressure sensor data are compared to the data obtained from the strain gages. Finally, comments are made with regard to on the reliability and accuracy of both sets of data and the circumstances and the variables that could affect field test data.

CHAPTER 2

TEST PROGRAM

2-1 Introduction

The University of Tennessee is in the process of conducting a full-scale simulation of integral bridge abutments. Four piles have been driven into the ground. Three piles have been tested, while preparations are almost finished for the fourth pile. The location for the field tests is at the construction site of the Dutchtown Road interchange on Pellissppi Parkway, fifteen miles west of The University of Tennessee, Knoxville. Test setup, instrumentation, and test regimen are briefly described in the following sections.

2-2 Test Setup

HP 10 X 42 piles forty feet into ground were used with type A concrete for the abutments and abutment slabs (minimum f_c' =3000 psi). Two feet of the total length of the pile were above the ground level for embedment and instrumentation. The total vertical load applied on the pile was 65 Kips, to simulate the typical value of axial loads on piles in continuous bridges. The thermal movement of a jointless bridge superstructure was simulated by a horizontal load applied at the abutment. The loading was applied through a dywidag post-tensioned threaded bar placed in a 3" diameter pipe cast in the abutment. A hydraulic jack was used to pull on the bar, and a reaction beam was placed at the end of the bridge abutment in the direction of the applied load to resist vertical movement of the end of the abutment slab.

2-3 Instrumentation

Fifteen stations of weldable strain gages were placed on the pile. Each station consists of four strain gages placed on the inside faces of the flanges of the cross section. Six stations of pressure sensors were placed on the pile, each station consisting of two pressure sensors placed on the opposite flanges of the cross section. Since it was assumed that the thermal movement at the top would not affect total length of the pile, instrumentation was only placed on the top 20 feet of the pile. Linear Variable Differential Transformers (LVDTs) measured the lateral deflections of the pile. One LVDT measured the deflection at the top of the pile at the ground level; another measured the deflection of the abutment. One more LVDT was added to measure the deflection of the inclinometer placed at the ground level. The purpose of the inclinometer was to measure the rotation of the top of the pile at ground level.

2-4 Test Regimen

Twelve main tests were run for each pile, six in each direction. The tests were deflection controlled. In the first three tests, the pile was pulled to a ½" deflection. Each test consists of four increments of 1/8" deflection. And each increment was held for 45 min except for the last increment, which was held for 10-15 min. In the following three tests in the same direction, the pile was pulled to 1" deflection. Each test consists of six increments; the first two increments were ¼" deflection while the last four were 1/8" deflection. The first two increments were held for one hour, the next three were held for 45 min, and the last increment was held for 10-15 min.

2-5 Data Collection System

The system used for collecting data was a MEGADAC 3108. The MEGADAC converts the signals coming from the different kinds of instruments to a user-friendly format. This was done through OPTIM's Test Control Software (TCS). TCS is an engineering software package which translates and documents the input from the instruments through the MEGADAC. The data, once documented, can be translated into different format, for example EXCEL, to be carefully studied. More details can be found in an earlier thesis by Arico(3).

CHAPTER 3

DESIGN APPROACHES FOR INTEGRAL-ABUTMENT PILES

3-1 Introduction

The design of integral abutment piles has been based upon intuition, experimentation, and observation throughout the past 50 years. The reason that a rational approach has not been developed is that soil-pile interaction is an unusual problem. This problem is complex because the pile deflected shape depends upon the soil resistance and soil response is a function of the deflection of the pile. However, a few approaches have been developed and successfully used with an acceptable conservatism.

3-2 TDOT Design Approach and LPILE

The *Laterally loaded pile Analysis Program for Microcomputer*, COM624P, or LPILE, is a computer program that solves the problem of laterally loaded piles using a set of differential equations for a beam-column to obtain the deflected shape of the pile. The deflected shape of the pile is obtained under combined axial and lateral loading. The differential equations are solved using finite differential numerical method. The general form of the differential equations is:

$$\frac{d^2M}{dx^2} + Q\frac{d^2Y}{dx^2} - \frac{dV_V}{dx} = 0 \qquad (3\text{-}1)$$

Where:

V_V = shear force in pile

M = bending moment at distance x

Q = axial load

Y = lateral deflection at distance x

Solving this equation requires the establishment of the p-y curves. The p-y curves are plots of soil response (p) vs. pile deflection (y). A given pile has a set of p-y curves for different depths. The soil resistance was determined by differentiating the moment curve twice, while the pile deflection was determined by integrating the moment curve twice.

$$\text{Deflection} = y(x) = \iint \frac{M(x)dx}{EI} \tag{3-2}$$

$$\text{Soil Resistance} = P(x) = \frac{d^2M(x)}{dx^2} \tag{3-3}$$

Where:

EI = flexural rigidity of pile

P = soil reaction per unit length of the pile

Tennessee Department of Transportation (TDOT) adopted the LPILE software for design of integral abutments (1). The first step in the design of laterally loaded piles is to establish the moment vs. depth curve and the deflected shape for a given thermal movement at the pile head. The unbraced length of the pile is taken as the distance between the two zero moment points on the moment vs. depth curve. As specified in AASHTO Standard Specifications for Highway Bridges (10), equations (3-4) and (3-5) are used to calculate the capacity of piles under combined axial and lateral loading. From the calculated pile capacity for different axial loads and bending moments, the data points can be used to plot an interaction diagram for the pile.

$$\frac{P}{0.85AsFcr} + \frac{MC}{Mu\left(1 - \dfrac{P}{AsFe}\right)} \leq 1.0 \tag{3-4}$$

$$\frac{P}{0.85AsFcr} + \frac{M}{Mp} \leq 1.0 \tag{3-5}$$

Where:

$$\text{Fcr} = Fy\left[1 - \frac{Fy}{4\Pi^2 E}\left(\frac{kLc}{r}\right)^2\right] \quad \text{,for} \left[\frac{kLc}{r}\right] > \sqrt{\frac{2\Pi^2 E}{Fy}} \tag{3-6}$$

$$\text{Fcr} = \frac{\Pi^2 E}{\left(\dfrac{KLc}{r}\right)^2} \quad \text{;for} \left[\frac{kLc}{r}\right] < \sqrt{\frac{2\Pi^2 E}{Fy}} \tag{3-7}$$

$$C = 0.6 + 0.4a \quad \geq 0.4 \tag{3-8}$$

$$F_e = \frac{\Pi^2 E}{\left(\dfrac{KLc}{r}\right)^2} \tag{3-9}$$

K = effective length factor taken as 0.875.

Lc = unbraced length of the pile.

r = pile radius of gyration in plane of interest.

F_{cr} = buckling stress.

C = equivalent moment factor.

F_e = Euler buckling stress.

M_p = plastic moment of the pile cross-section.

M_U = maximum flexural strength of the pile cross-section.

a = the ratio of the rotational moments $\left(\frac{m_1}{m_2}\right)$, whereas a is negative for double curvature and positive for single curvature.

The previous method has shown promising results for piles driven into different types of soil except for hard clay. However, where the use of hard clay is predicted, the pile need to be driven into a pre-bored hole of twice the diameter of the pile (1). Greimann et al (2) has introduced new methods for designing integral abutment piles.

3-3 Elastic and Inelastic Design Methods for Integral Abutment Piles

Abendroth, Greimann, and Ebner (2) have published several papers about designing integral abutment piles. Two main alternatives in designing piles under combined axial and lateral loading showed promising results. The first design approach is the elastic design method. This method accounts for stresses exerted by thermal movement. However, It neglects any redistribution of internal forces and assumes failure occurs when the stress in the pile reaches yielding stress at any depth. The pile is represented as a cantilever column having a fixed base at distance (L_e) from the ground level. The length of the equivalent cantilever is assumed to be the distance above the ground plus the distance embedded in soil where the lateral displacements below are 4% or smaller.

Another method in designing integral abutment piles was also presented by Greimann, Abendroth, and, Ebner. However, this alternative takes into consideration pile ductility and plastic redistribution of internal forces exerted by thermal movement of the bridge abutment. The stresses induced in the pile from lateral movement are neglected, while the strains are taken into consideration. Therefore the applied moment is only due to the displacement of the axial load of the pile. The pile must have enough moment-rotation capacity to account for these strains and rotations caused by gravity loads. The theoretical length factor, K, for fixed-headed piles and pinned-headed piles for design purposes are equal to 0.65 and 0.8, respectively (2). Based on the AASHTO specifications for calculating the capacity of the pile, interaction diagrams were constructed.

Both methods were compared to a finite element investigation. The results showed that alternative one was very conservative for small slenderness ratios, while both methods gave an acceptable conservatism for large slenderness ratios. Moreover, as the lateral movement of the pile increased, both alternatives and the finite element solution indicated decrease in strength. Alternative two specified a sufficient ductility in the pile, whereas alternative one has no requirements for ductility. Consequently, it was recommended that for longer integral-abutment bridges, alternative two provided a safer design (2).

CHAPTER 4

PRESSURE DATA

4-1 Introduction

Six stations of pressure sensors were installed for each pile. Each station consists of two pressure sensors placed on the opposite flanges of the cross section. The pressure sensors were installed in the center of the outside face of the flanges, in which they were extend 2.5 inches into the web. Thus, the pressure sensors were placed flush with the outside face of the flanges. To minimize the effect of bending on the pressure sensors, RTV-108 was used to cushion the space around perimeter of the sensors. The first station of pressure sensors was installed at 5'-3" depth below the ground level. Then, the other stations were installed below station one using a spacing of 1'-6".

The main purpose of the pressure sensors was to determine the zero pressure point below ground level. The soil around the pile could be modeled as a series of horizontal springs. The forces in the springs would be the soil response or the pressure due to the lateral movement of the pile (8). The pressure is a function of the soil horizontal stiffness and the deflection of the pile. And since the confinement and soil properties vary with depth, the horizontal stiffness is also expected to vary with depth. As a result of this modeling, zero lateral deflection of the pile would indicate zero response of the soil or zero pressure. In other words, the zero pressure point is the same as the zero deflection point. Therefore, using the same definition Greimann et al (2) used in describing the zero deflection point, the zero pressure point could be defined as the depth where the lateral displacements below are 4% or less.

4-2 Pressure vs. Depth curves

For the first pile, pressure sensors with a 5 psi capacity were used. However, most of the pressure sensors appeared not to respond. Apparently, the readings of the pressure sensors had gone off-scale. Therefore, it was decided to use pressure sensors with a capacity of 20 psi for the second pile. Unfortunately, the results were not promising. Many pressure sensors went off-scale, and others did not respond for various reasons. However, from the limited number of pressure sensors that showed "good" readings, the zero pressure point was between 8 and 10 feet below the ground level. Subsequently, pressure sensors with a 100 psi capacity were chosen to be used for piles three and four. Reasonable readings were collected from the pressure sensors for pile #3.

Both pulling and suction sides had a few pressure sensors that were reading satisfactorily. Pressure vs. depth curves were plotted for each side of the pile, as shown in Appendix A. Pressure vs. depth curves were also developed from strain gage data. The pressure curves were obtained from the strain gage data using the following steps. First, the moments for different depths were calculated using equation (4-1).

$$M = \frac{\varepsilon EI}{y} \qquad (4\text{-}1)$$

Where:

M= bending moment due lateral load (k-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)

The bending moments were then plotted vs. depth. A six-order polynomial used to fit a curve through the moment points. By differentiating the moment equation twice, the pressure equation (4-2) was developed; the results of pressure vs. depth curves obtained by using this equation can be seen in Appendix B.

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

Where:

P= lateral pressure (ksi)

x = depth below ground level

4-3 Zero Pressure Points

The two sets of pressure vs. depth curves, for both sides of the pile, obtained from pressure sensor data did not perfectly match. Driving the pile into ground appeared to have induced compression pressures of 3-4 psi in the pressure sensors. Therefore, the readings of negative pressure in some of the pressure sensors could be a release in the compression pressure that existed before performing the tests, as seen in Appendix A. The zero pressure point for the pressure sensors on the opposite side of pulling was between 7'-4" and 8'-1" for pulling West and between 8-10" and 10'-0" for pulling East, as seen in Table (4-1). Whereas, for the pressure sensors on the puling side indicated that the zero pressure point was between 8'-4" and 9'-3" for pulling West Side and 10'-6" and 11'-6" for pulling East Side, as indicated in Table (4-1). However, the pressure vs. depth curves obtained from strain gage data, Figures (B-1) to (B-16), indicated that the zero pressure point is somewhere between 10' and 11' when pulling west, as shown in Table (4-1). Of particular note, the pressure vs. depth curves, for the first 1/2" test, obtained from the pressure sensor data for both the pulling and the suction sides and from the strain gage data, indicated that the zero pressure point is within a range of 1'. This was the smallest range for the zero pressure point obtained from the pressure sensors for both sides and the strain gages for the same test. The reason behind that was probably because this was the only test where most of the strain gages and pressure sensors working properly. However for the other tests, there are numerous reasons for the diversity between the results; some of the reasons are discussed in the next section.

4-4 Result Analysis

The pressure sensors needed to have a large diameter to prevent arching of the soil around the sensor. However, due to the small width of flanges, 2.25" diameter pressure sensors were used. Also, pressure sensors with a large diameter would be sensitive to the driving action while the pile was being installed.

The pressure sensors were calibrated in the lab to indicate the effect of bending moment on their readings. As a result, the pressure sensors were found to read a pressure of 0.8-1.5 psi due to bending moment. Therefore, an inconsistency of 5% would be expected due the moment variation with depth. In other words, the pressure sensors near the maximum moment would be more affected than the pressure sensors near small or zero moment. The inconsistency in readings is a function of depth, moment, and the location of the pressure sensor with respect to the moment curve.

Since it was assumed that the zero deflection point did not vary with lateral displacement of the pile head (2), the zero pressure point was expected not to vary either, as indicated in Figures (4-1) to (4-4). However, as the tests were repeated, the location of zero pressure was not the same for the two sides of the pile. In general the depth on the pressure side was increasing slightly from 8'-6" to 9'-3". The location of the zero pressure point on the suction side was decreasing from 8 ft to 7.4 feet. The strain gage data indicated that the point of zero pressure was increasing from 7.5 feet to 11.5 feet. Therefore, the results from the pulling side pressure sensors generally agreed with the variation from the strain gage data.

The zero pressure point was assumed to be a function of flexural rigidity of the pile and horizontal stiffness of the soil. The pressure sensor data indicated that the zero pressure points varied from one test to another, as seen in Figures (4-5) and (4-6). In Figure (4-5) the pressure diagrams for the 0.5 inch displacement in each of the three 1 inch tests are shown. The points of the zero deflection varies from 8.5' to 8.7' to 9.4' for the three tests. As the tests were continued to a maximum displacement of 1 inch, the points of zero pressure varies from 8.5' to 9.0' to 9.6' for the three tests. Therefore, the zero point of pressure remains essentially the same for a specific test and the point is deeper with different test dates. This was very reasonable since the soil stiffness is expected to change due to the confinement applied at the soil in each test. As repeated tests are performed the clay soil becomes more consolidated due to the earlier deflection of the pile. There is also a gap from earlier tests that must be closed in later tests and this gap will affect the lateral pressure. In addition, since tests were performed every week, the rain or snow during the week probably had an effect on the soil stiffness. It was assumed that the pressure sensors on the pulling side provided more accurate and consistent results. On the other hand, the pressure sensors on the suction side were useful in two ways. The pressure vs. depth curves obtained from the pressure sensors on the suction side showed that the zero pressure point did not vary with lateral deflection of the pile head. In addition, the pressure vs. depth curves obtained from the pressure sensors on the suction side were used to confirm the location of the zero pressure point obtained from the pressure sensors on the pulling side. In other words, the pressure data obtained from the suction side indicated that there were pressure sensors reading compression pressure on the opposite side of pulling below the zero pressure point, which confirmed the depth of the zero pressure point obtained from the pressure sensors on the pulling side.

As shown in Figures (4-7) to (4-10), the pressure diagrams indicated that zero pressure point increased from 7.5 feet to 11.5 feet. Although the depths of the zero pressure point obtained from the strain gage data and the pressure sensor data did not perfectly match, both results indicated that the point of zero pressure increased from one test to another. The pressure vs. depth curves obtained from the strain gage data did not confirm that the zero pressure point was independent of the lateral deflection of the pile head for a certain test. However, as shown in the first three 1" tests, the depths of the zero pressure point for the 0.5 inch and 1 inch increments were within 1 ft range. Considering the interpolation of missing strain gages and the small number of the moment points used to develop the moment equation, the one foot range is not a significant variation of the zero pressure point for different deflection increments for the same test.

Table (4-1). Depths of zero pressure point for pile # 3.

PRESSURE SENSIR DATA			STRAIN GAGE DATA	
TEST	AB SIDE	CD SIDE		
West Side				
10/5/98	8'-5"	8'-0"	7'-6"	1/2" test
10/9/98	8'-6"	8'-1"	10'-0"	1/2" test
10/16/98	8'-5"	7'-5"	10'-0"	1/2" test
10/23/98	8'-6"	7'-6"	10'-6"	1" test
10/30/98	8'-9"	7'-5"	11'-4"	1" test
11/6/98	9'-3"	7'-5"	11'-6"	1" test
East Side				
2/5/99	9'-1"	10'-11"	9'-10"	1/2" test
2/26/99	9'-1"	10'-10"	9'-9"	1" test

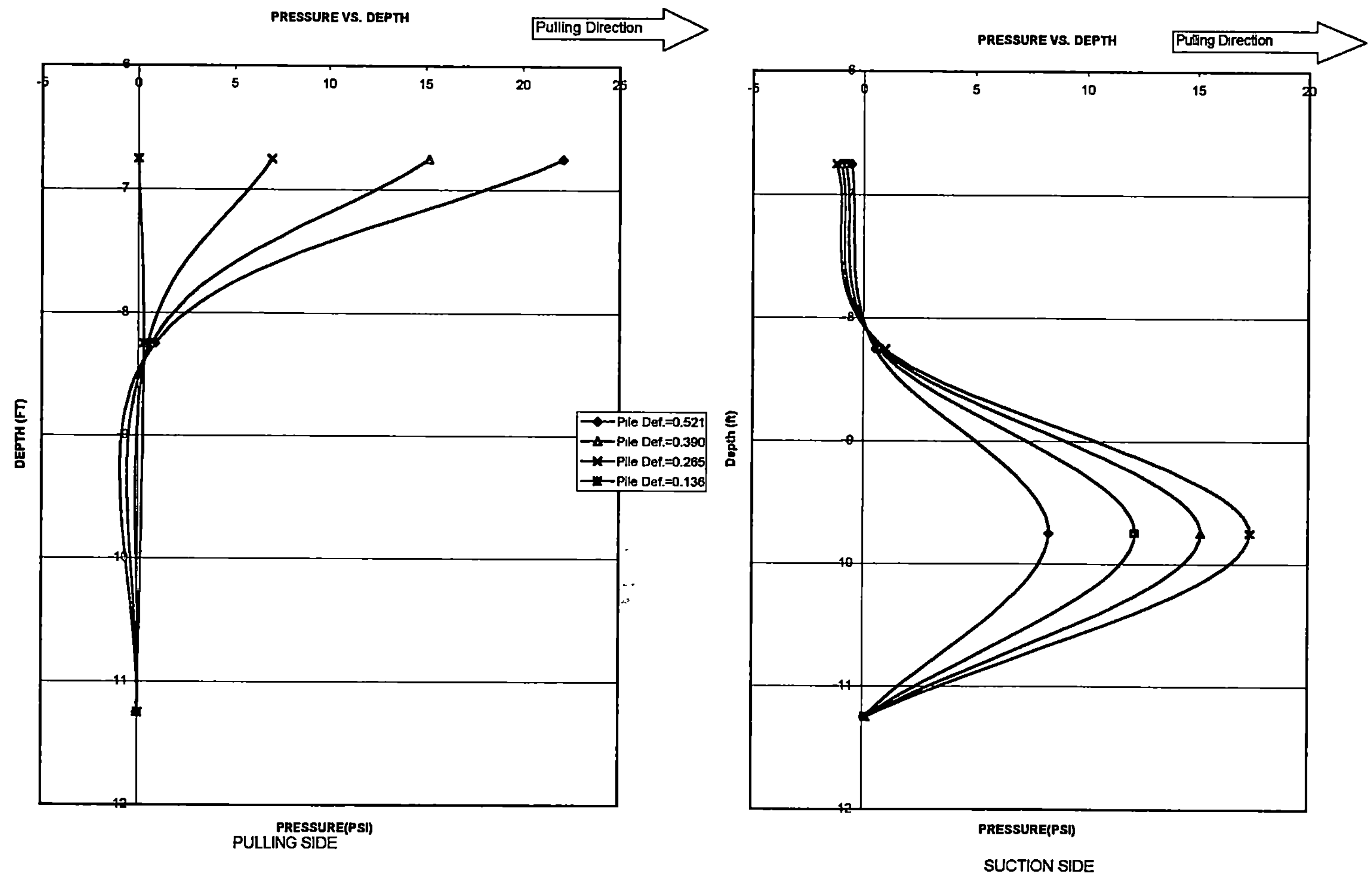
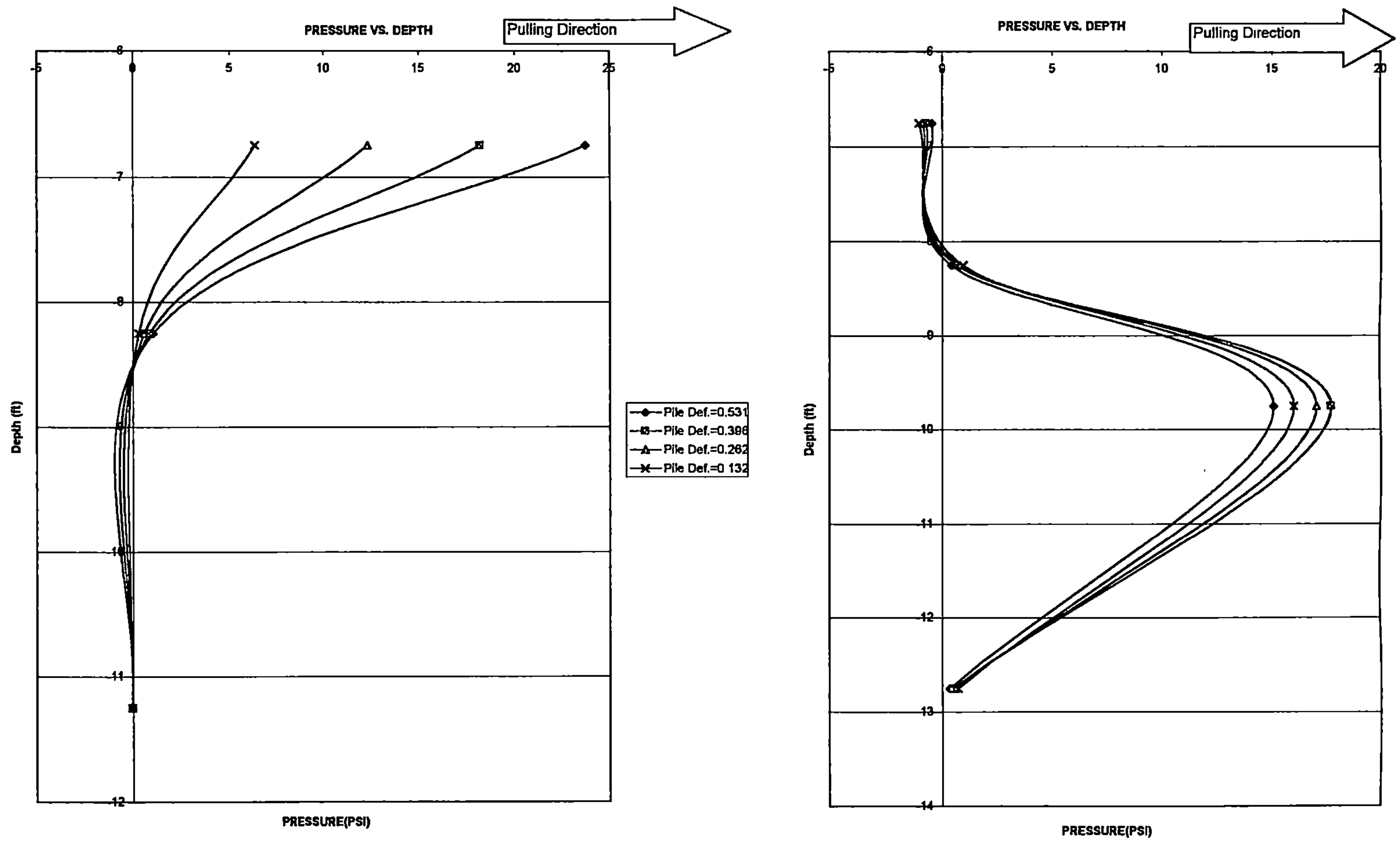


Figure (4-1). Pressure vs. depth curves obtained from pressure sensor data.
First 1/2" test, Oct, 05, 1999



PULLING SIDE

SUCTION SIDE

Figure (4-2). Pressure vs. depth curves obtained from pressure sensor data.
Second 1/2" test, Oct, 09, 1999

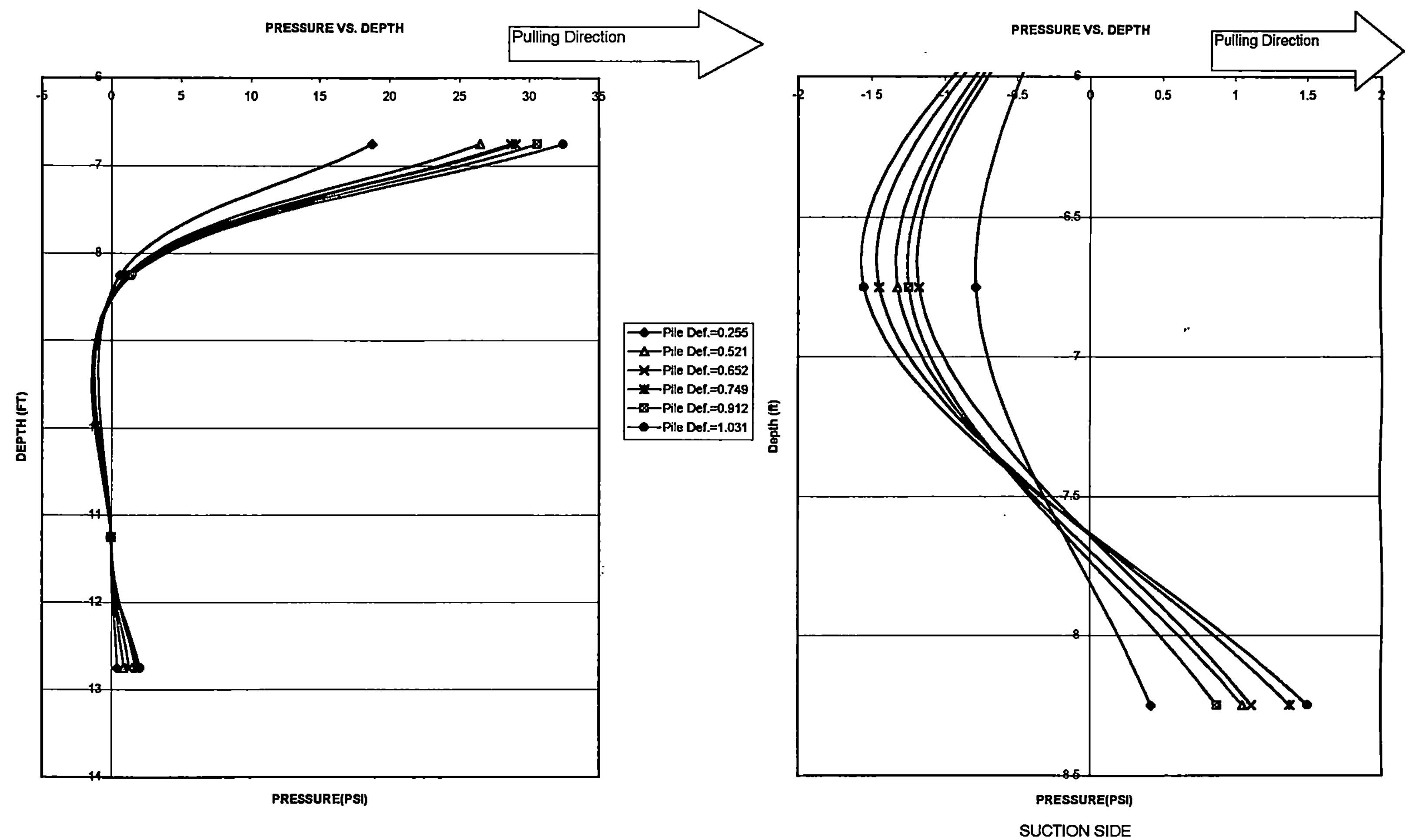


Figure (4-3). Pressure vs. depth curves obtained from pressure sensor data.
First 1" test, Oct, 23, 1999

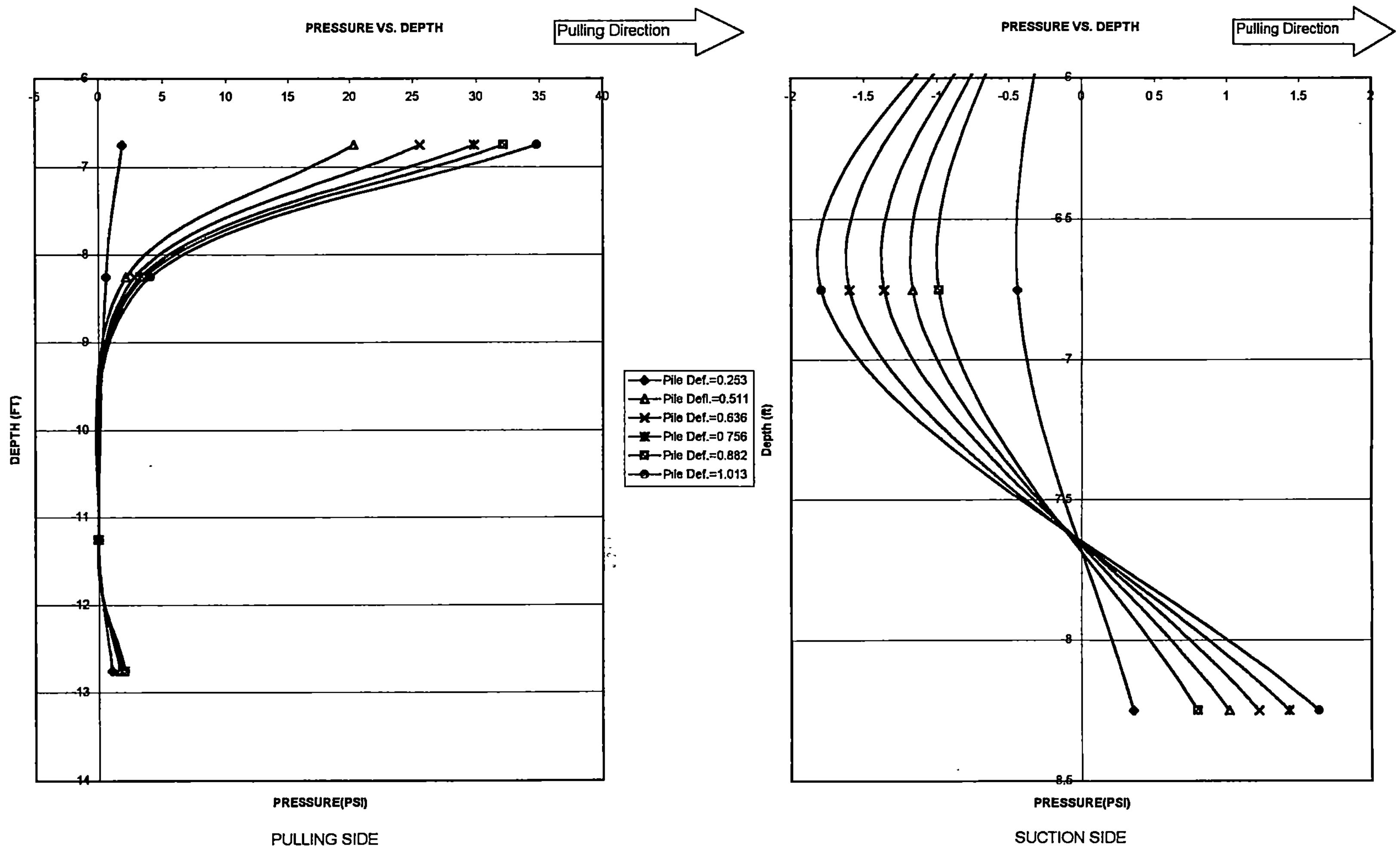


Figure (4-4). Pressure vs. depth curves obtained from pressure sensor data.
Third 1" test, Nov, 06, 1999

AB Pulling Side

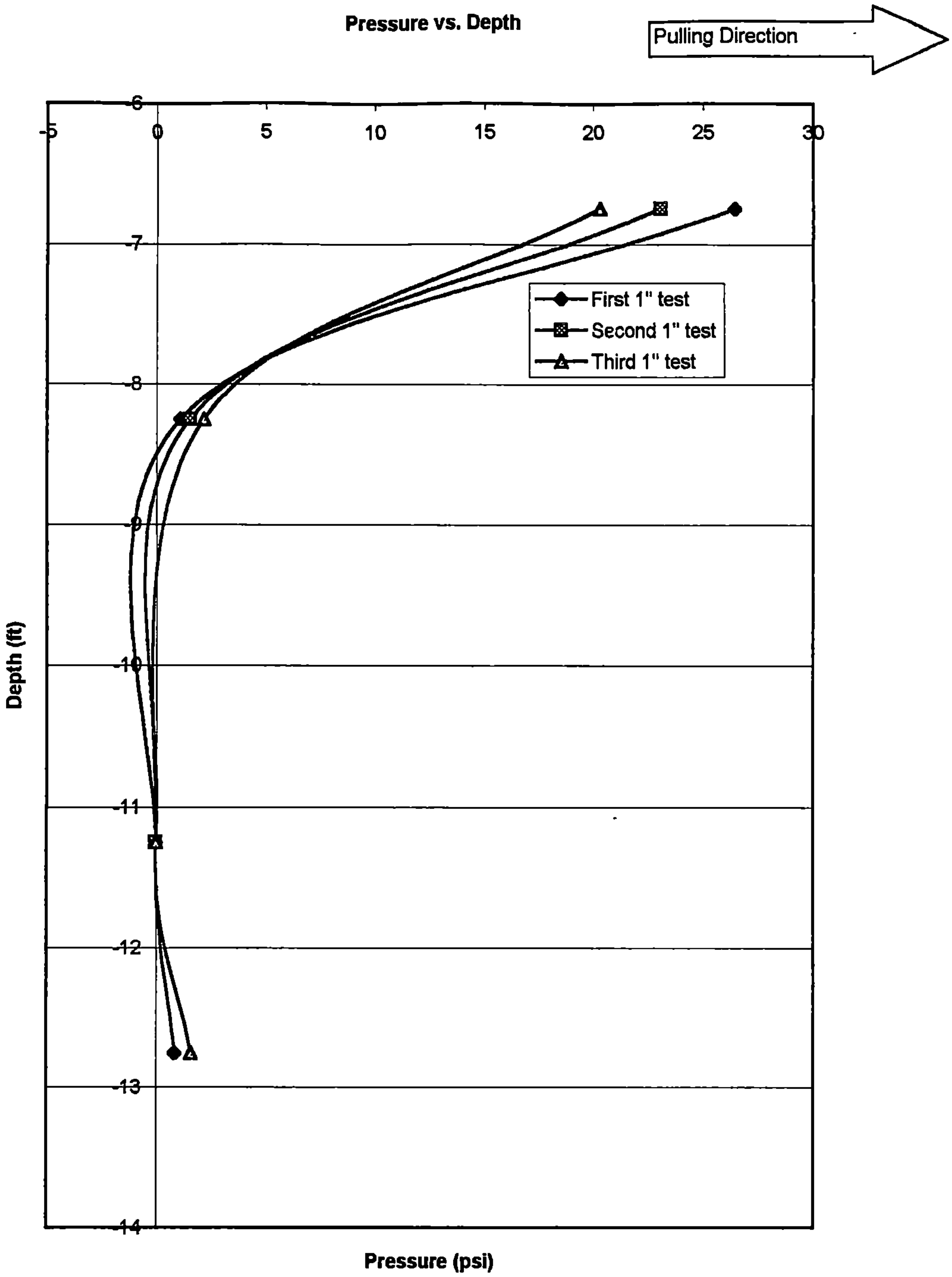


Figure (4-5), Comparison between zero pressure points for the first three 1" tests; 0.5" increment, for pile # 3, pulling west.

AB Pulling Side

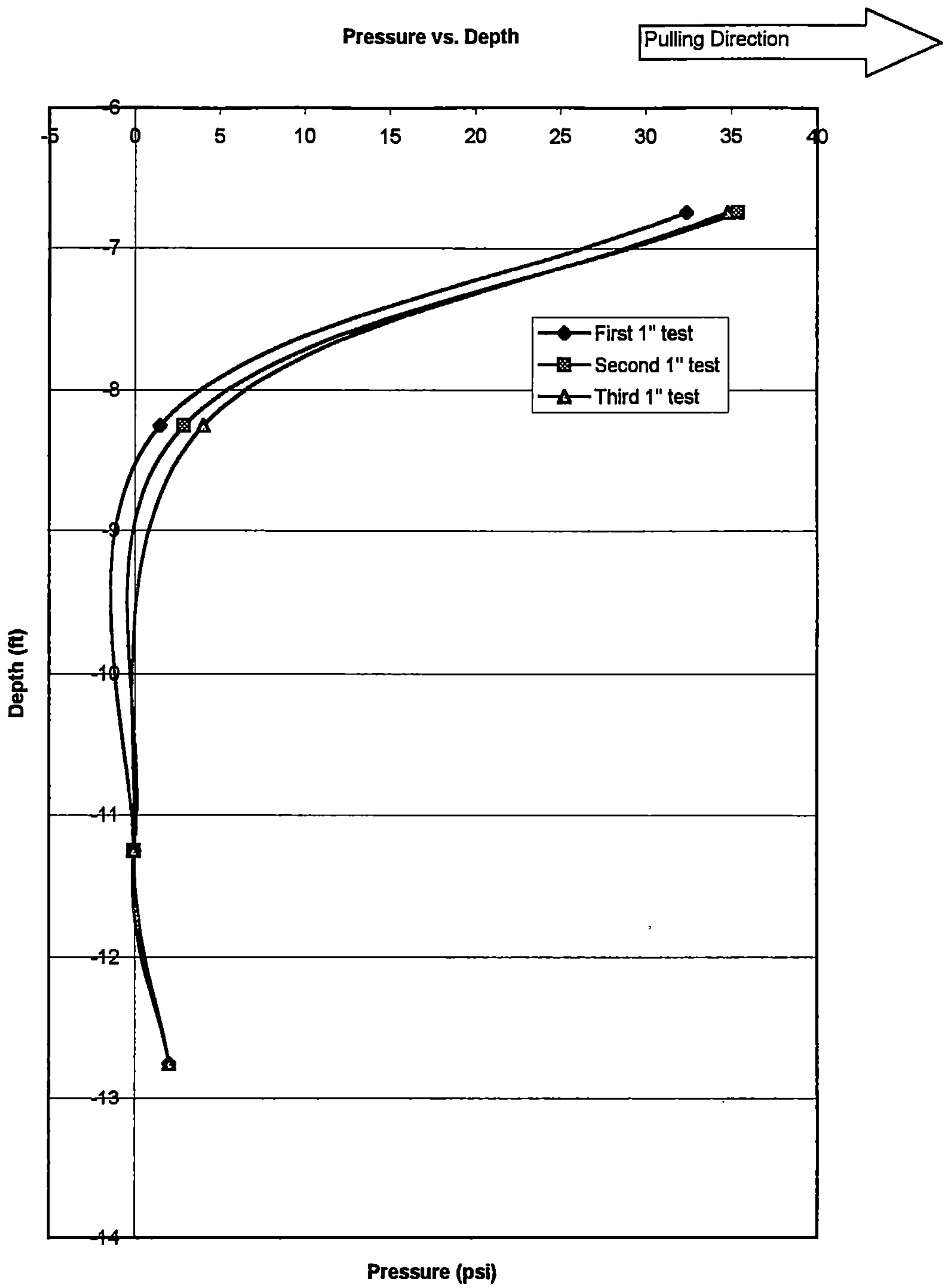


Figure (4-6), Comparison between the zero pressure points of the first 1" tests; 1" increments, for pile # 3, pulling west.

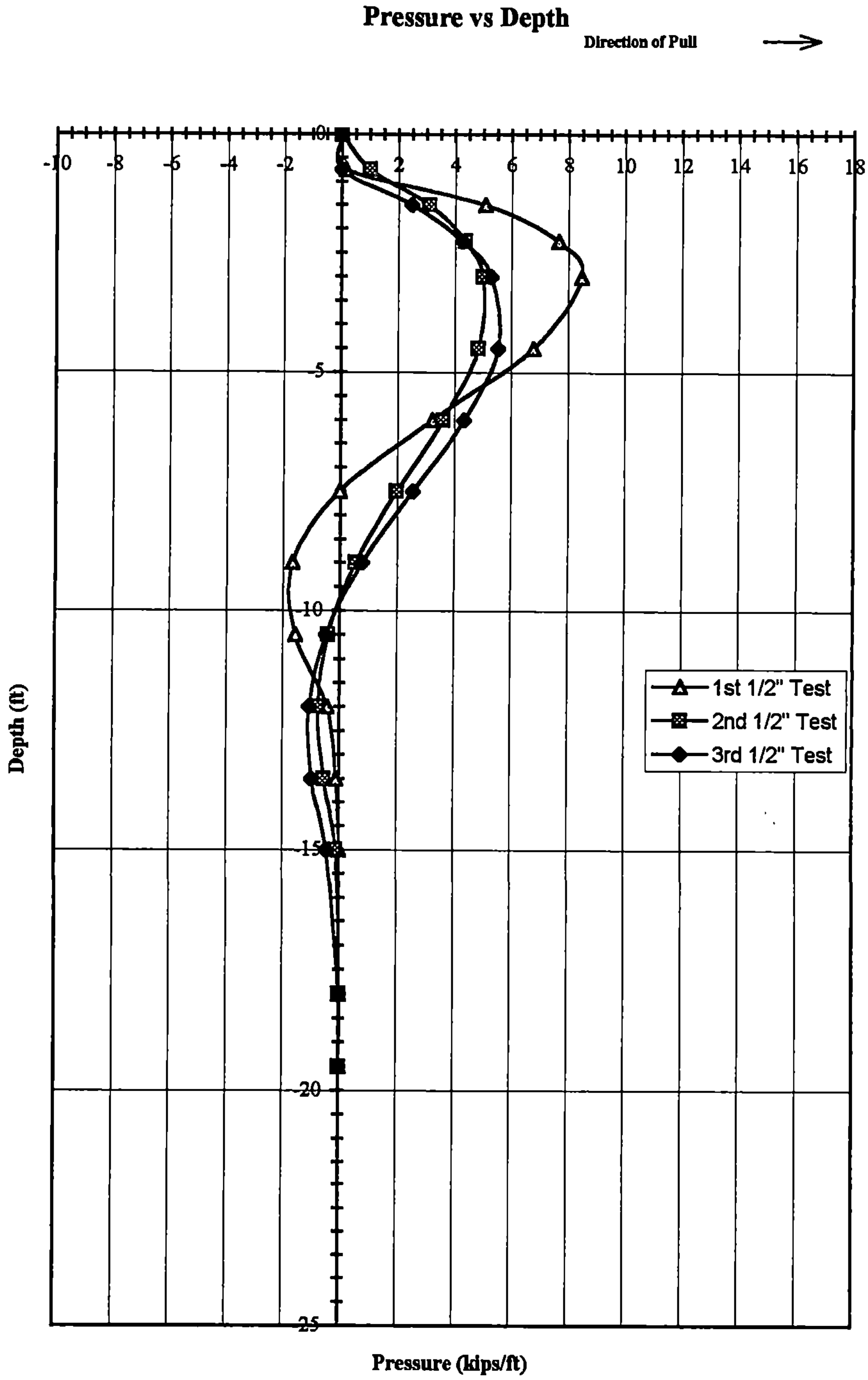


Figure (4-7). Pressure vs Depth from Strain Gage Data.
First three 1/2" tests.

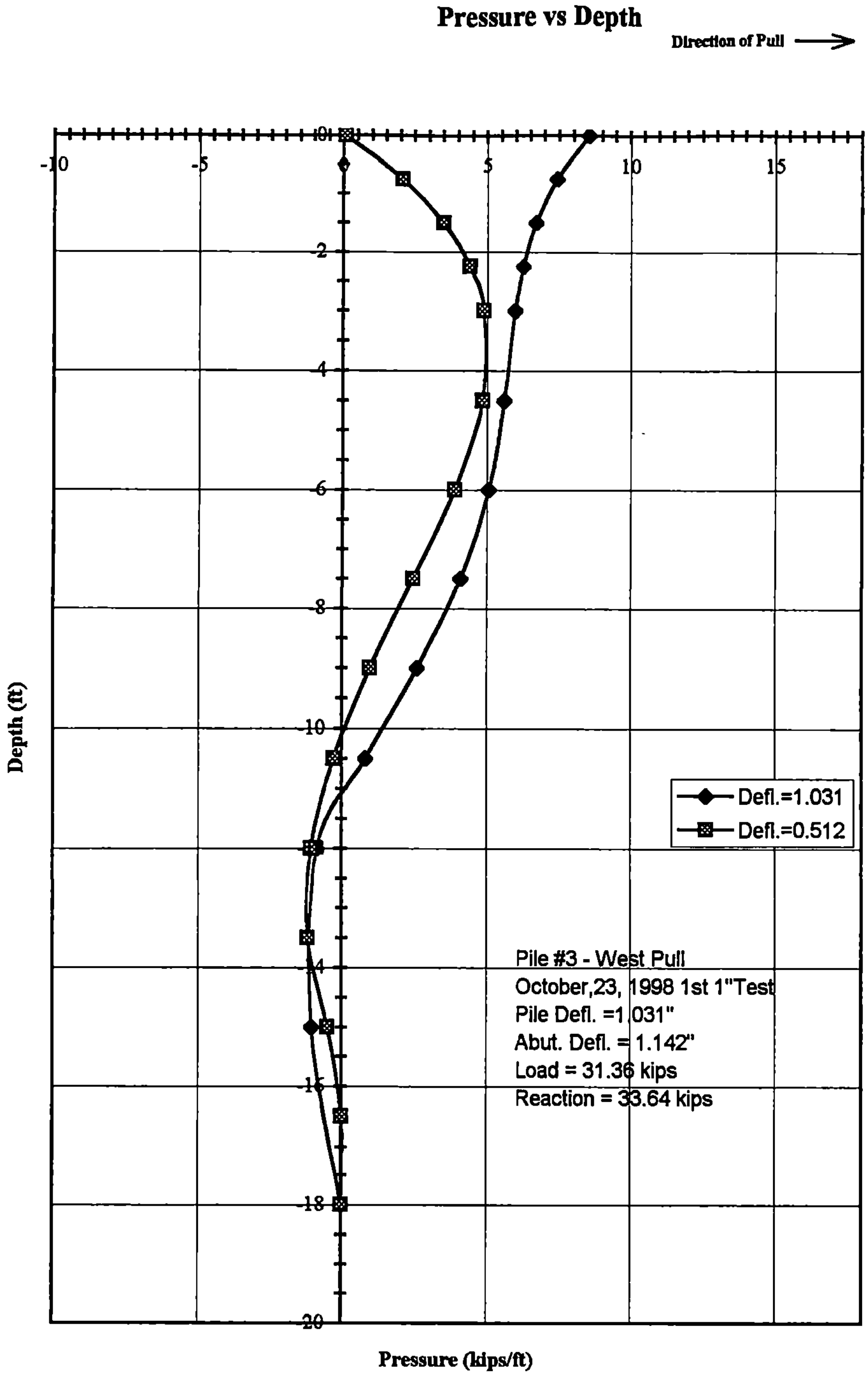


Figure (4-8). Pressure vs Depth from Strain Gage Data.
First 1" test.

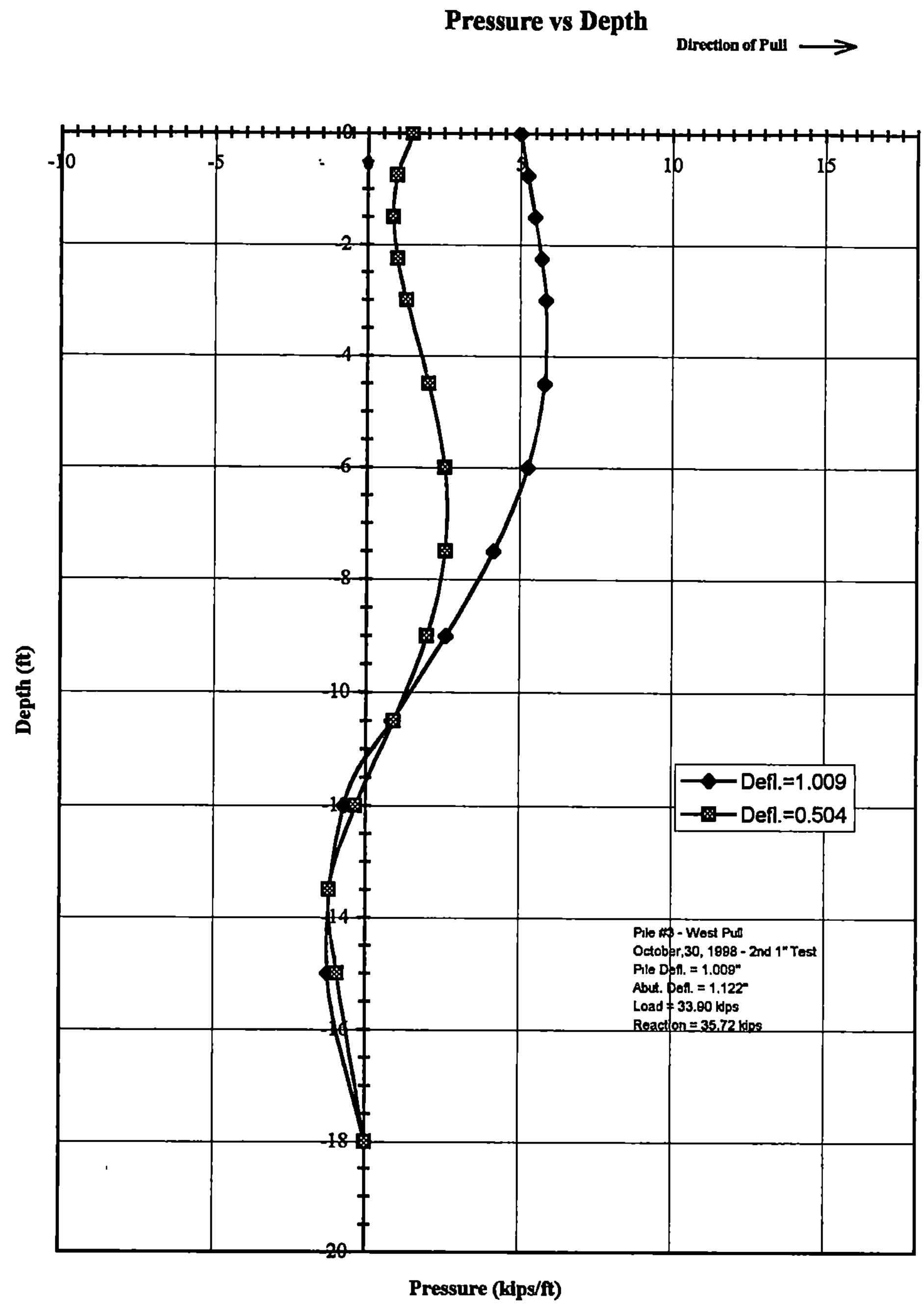


Figure (4-9). Pressure vs Depth from Strain Gage Data.
Second 1" test.

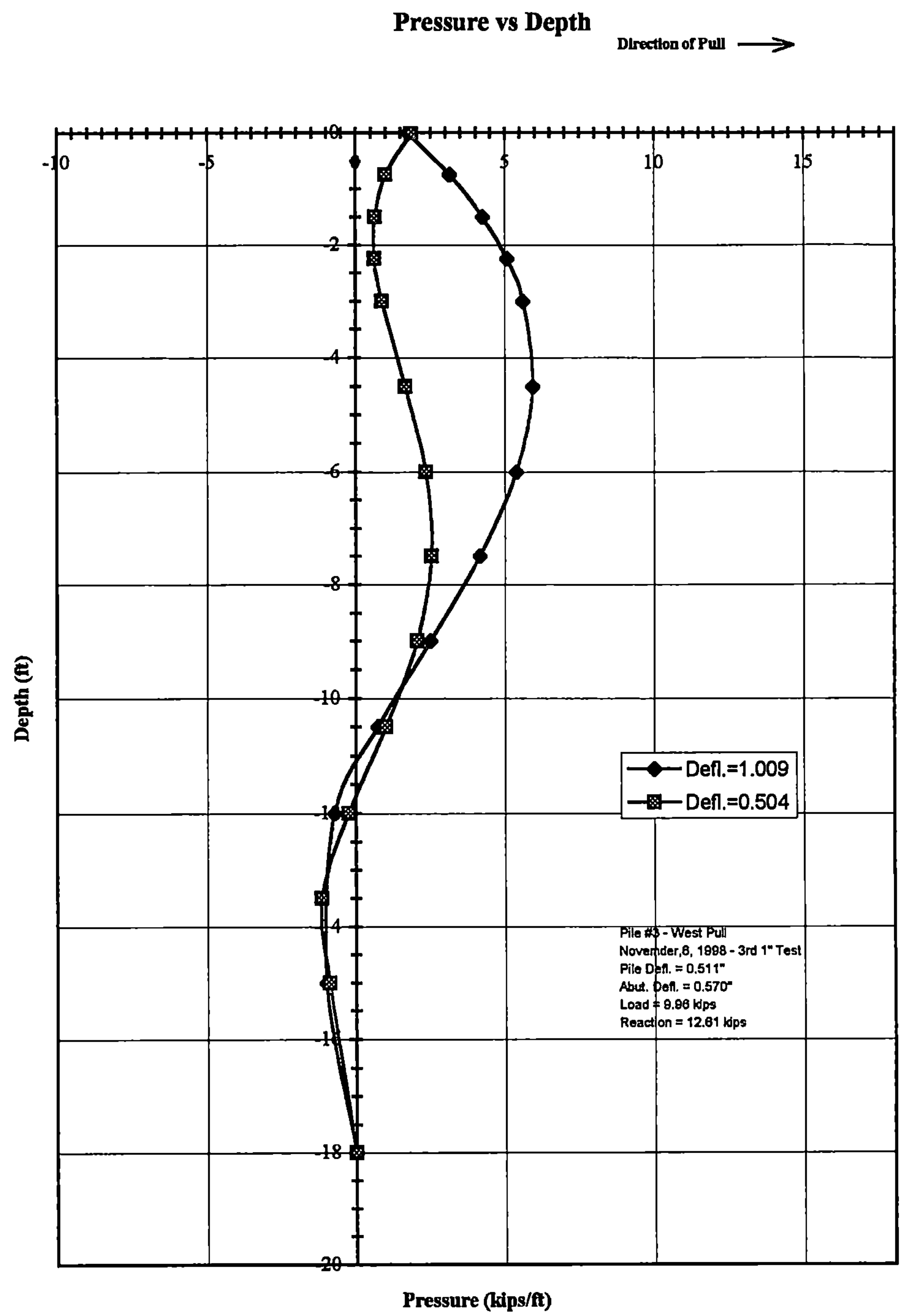


Figure (4-10). Pressure vs Depth from Strain Gage Data.
Third 1" test.

CHAPTER 5

SUMMARY AND CONCLUSION

The purpose of this research was to investigate the behavior of piles subjected to lateral loads. The strain gage data and pressure sensor data obtained from the field tests were reduced and analyzed. This thesis focused on the pressure distribution for the piles and the location of the zero lateral pressure point.

Pressure sensors with a capacity of 5 psi and 20 psi were used for the first and the second piles. Unfortunately, most of the pressure sensors went off-scale. Therefore, the pressure data collected from the pressure sensors for piles one and two were inconclusive. Pressure sensors with a capacity of 100 psi were used for the third and the fourth piles. Reasonable readings were collected for the third pile. Two sets of pressure vs. depth curves were plotted for different deflection increments for selected tests. The first set of pressure vs. depth curves was obtained from pressure sensor data. The pressure points considered in indicating the zero pressure point were only for the pressure sensors on the pulling side of the pile. Whereas, the pressure sensors on the suction side provided a rough estimation of the zero pressure point and indicated that the zero pressure point was constant for a certain test for different deflection increments of the pile head. After plotting the pressure vs. depth curves, the zero pressure point was found to be located for the west side at a depth between 8'-4" and 9'-3".

The second set of pressure vs. depth curves was obtained from the strain gage data. During testing, several strain gages were unreadable. Therefore, a few stations of strain gages had to be taken out, which affected the shape of the moment vs. depth curves. In addition, due to the limited number of points on the curve, the locations of the zero points were very sensitive to each and every point on the curve. The strain gage data were converted to bending moment points, and a six-order polynomial was used to fit these moment points. The pressure vs. depth curve was determined by differentiating the moment vs. depth equation twice. The zero pressure point for the West Side of pile #3 was found to be between 10' and 11'.

The zero pressure point was assumed to be independent of the horizontal displacement of the pile head for a certain test. As shown in Appendix (A), the pressure sensor data provided a similar results to the expected distribution of pressure for laterally loaded piles. Moreover, the zero pressure point was assumed to be a function of the length of the pile, flexural rigidity, and the horizontal stiffness of the soil. It was assumed that the zero pressure point and the zero deflection point coincide. More accurate results and better understanding will result through further testing of piles four and five. This can be achieved by using better instrumentation; such as pressure sensors and strain gages that have better resistance to severe weather, and having more data about the soil properties. The previous results of pressure data provided a better understanding of the behavior of piles supporting jointless bridges.

LIST OF REFERENCES

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APPENDICES

APPENDIX (A)

AB Pulling Side
Def.=.136"

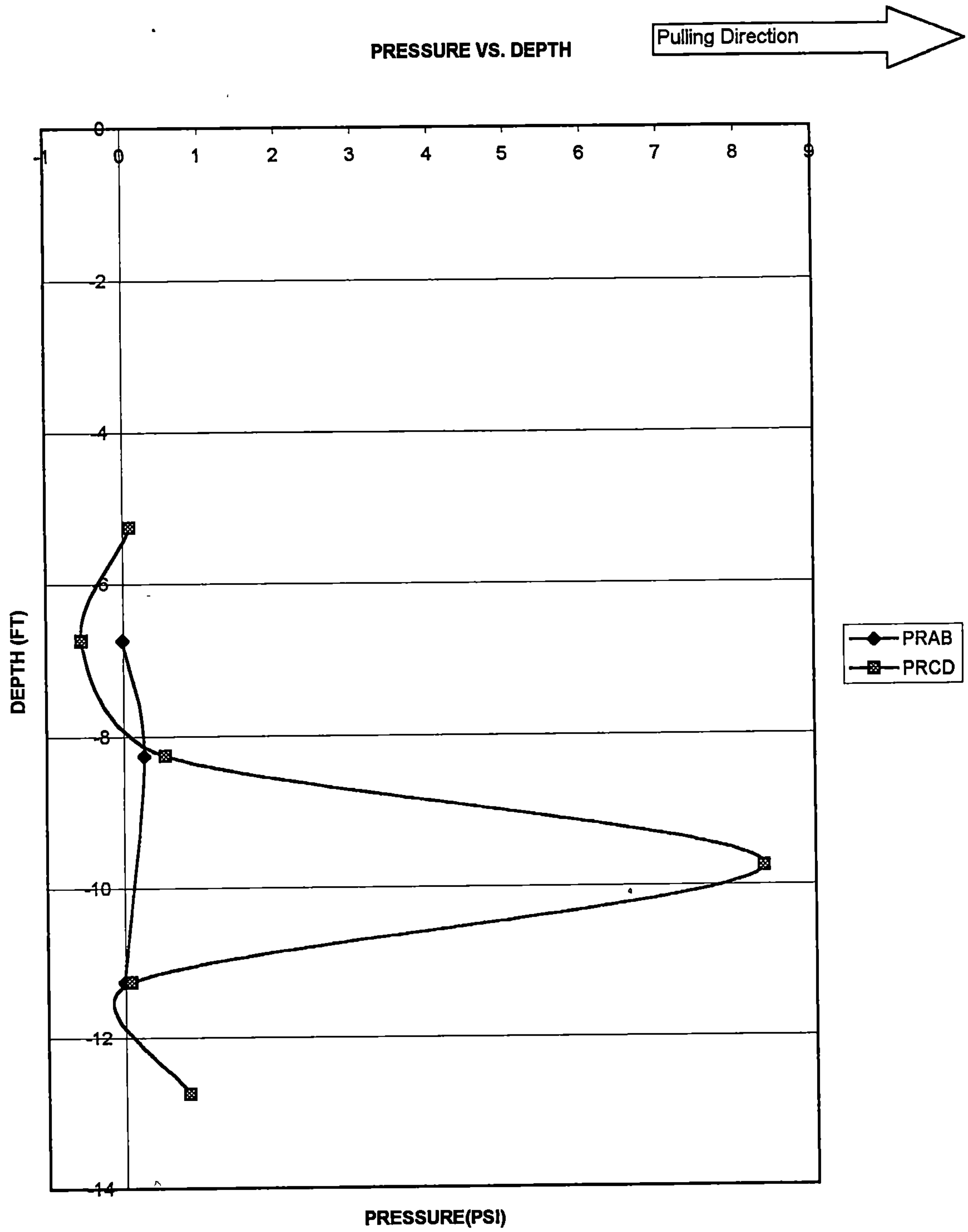

Figure A-1 Pressure vs Depth for pile # 3, pulling west. First 1/2" test, Oct,05,1999.
Deflection of the pile=.136"

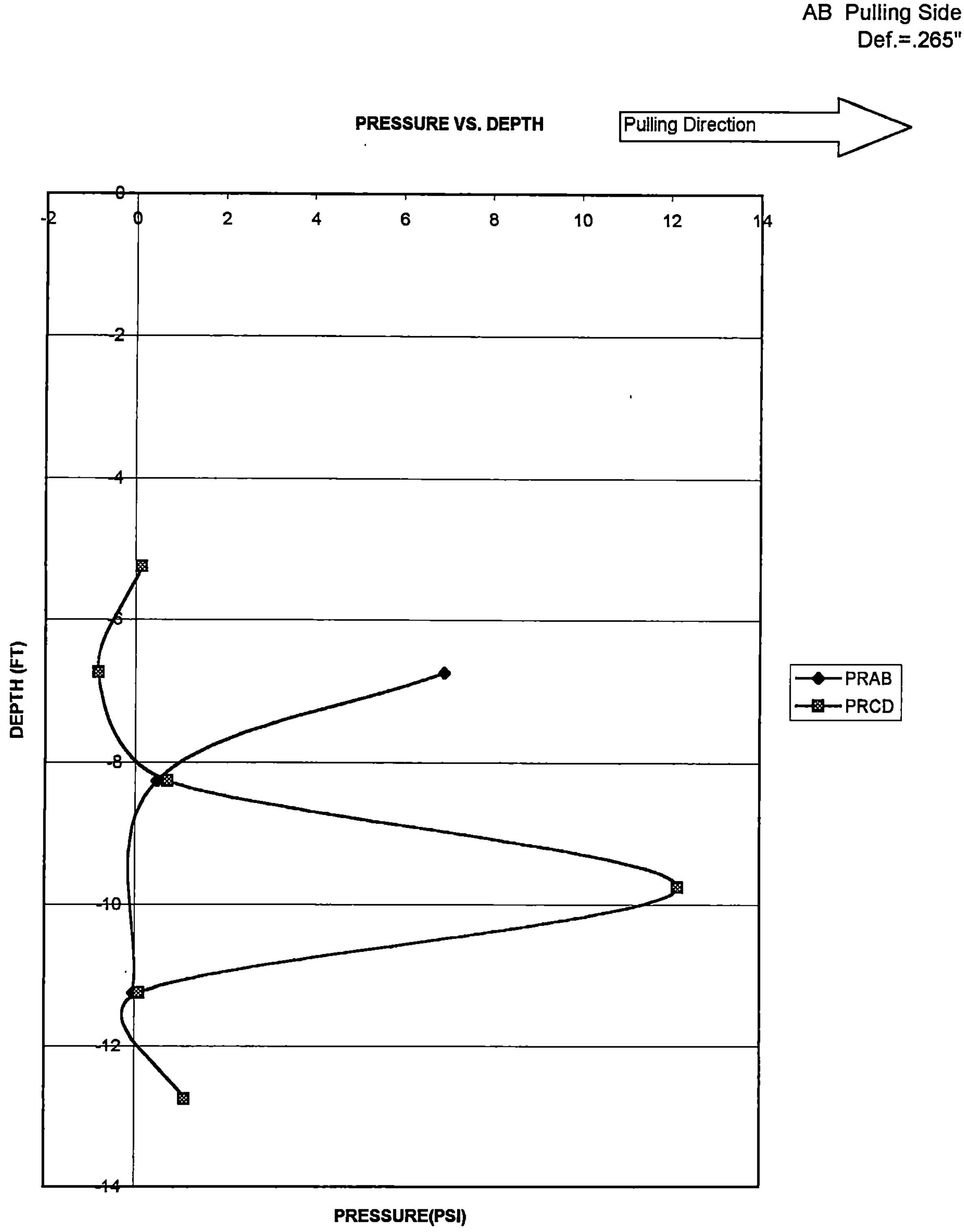

Figure A-2 Pressure vs Depth for pile # 3, pulling west. First 1/2" test, Oct,05,1999. Deflection of the pile=.265"

AB Pulling Side
Def.=.390"

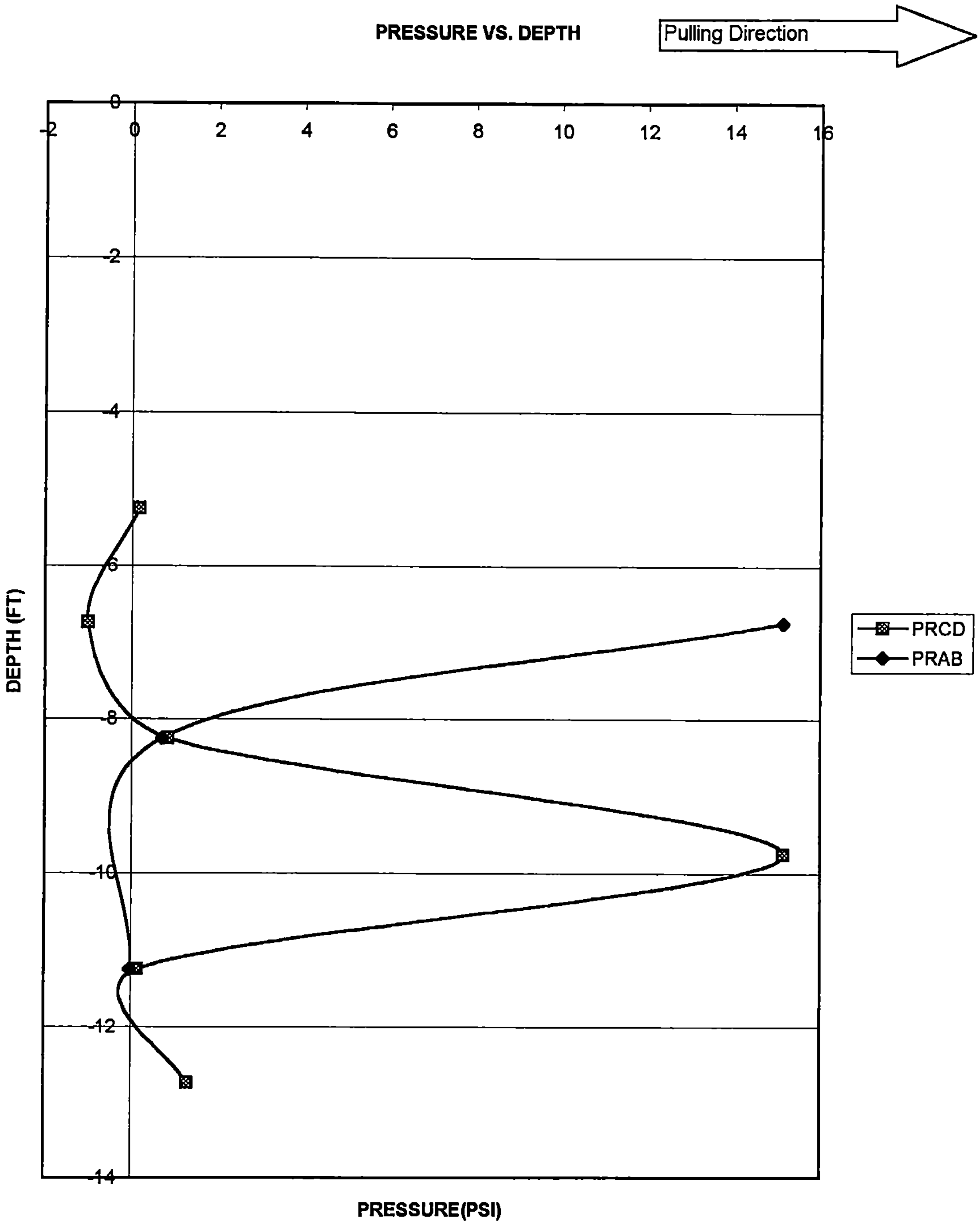

Figure A-3 Pressure vs Depth for pile # 3, pulling west. First 1/2" test, Oct,05,1999.
Deflection of the pile=.390"

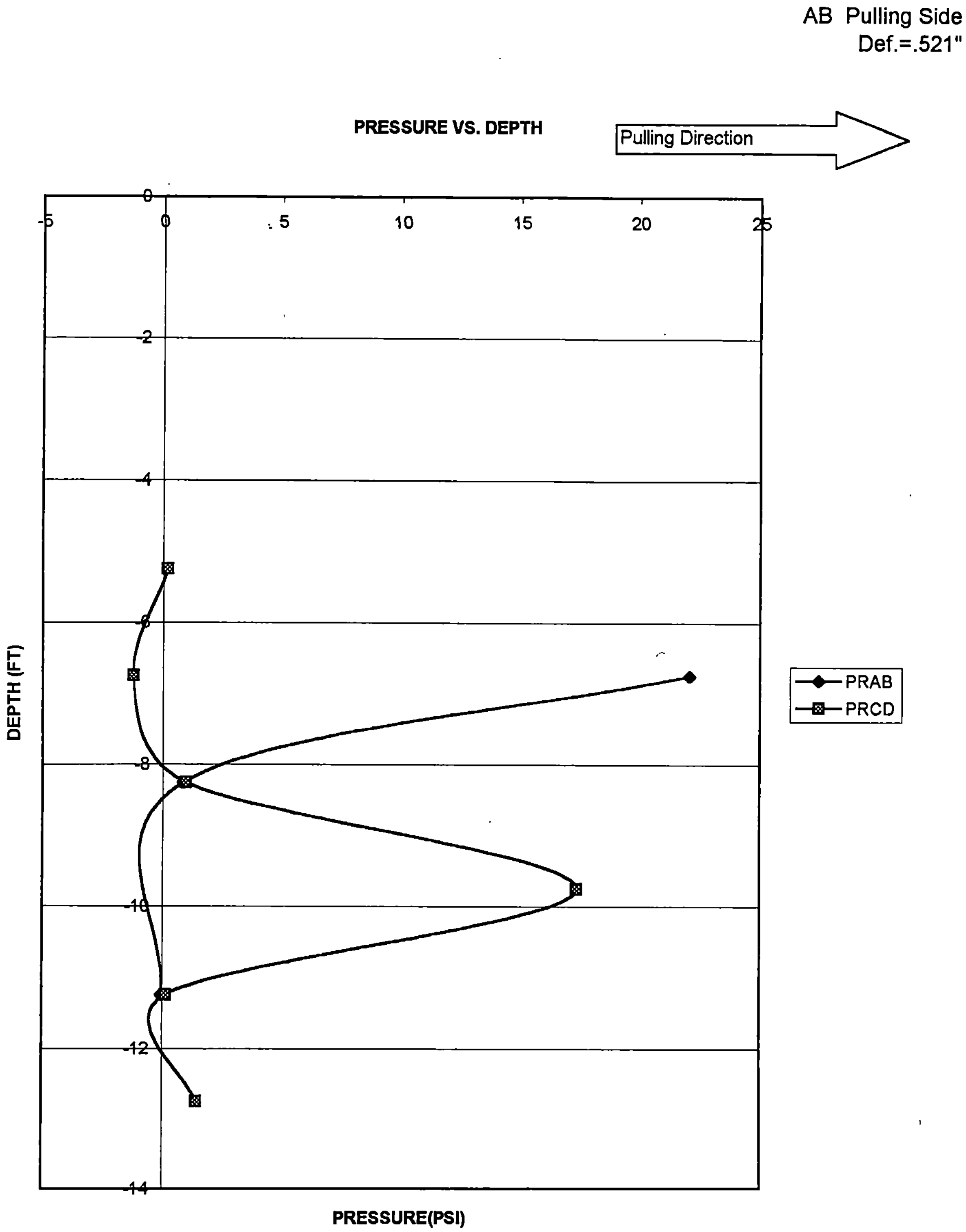

Figure A-4 Pressure vs Depth for pile # 3, pulling west. First 1/2" test, Oct,05,1999. Deflection of the pile=.521"

AB Pulling Side
Def.=PRCD

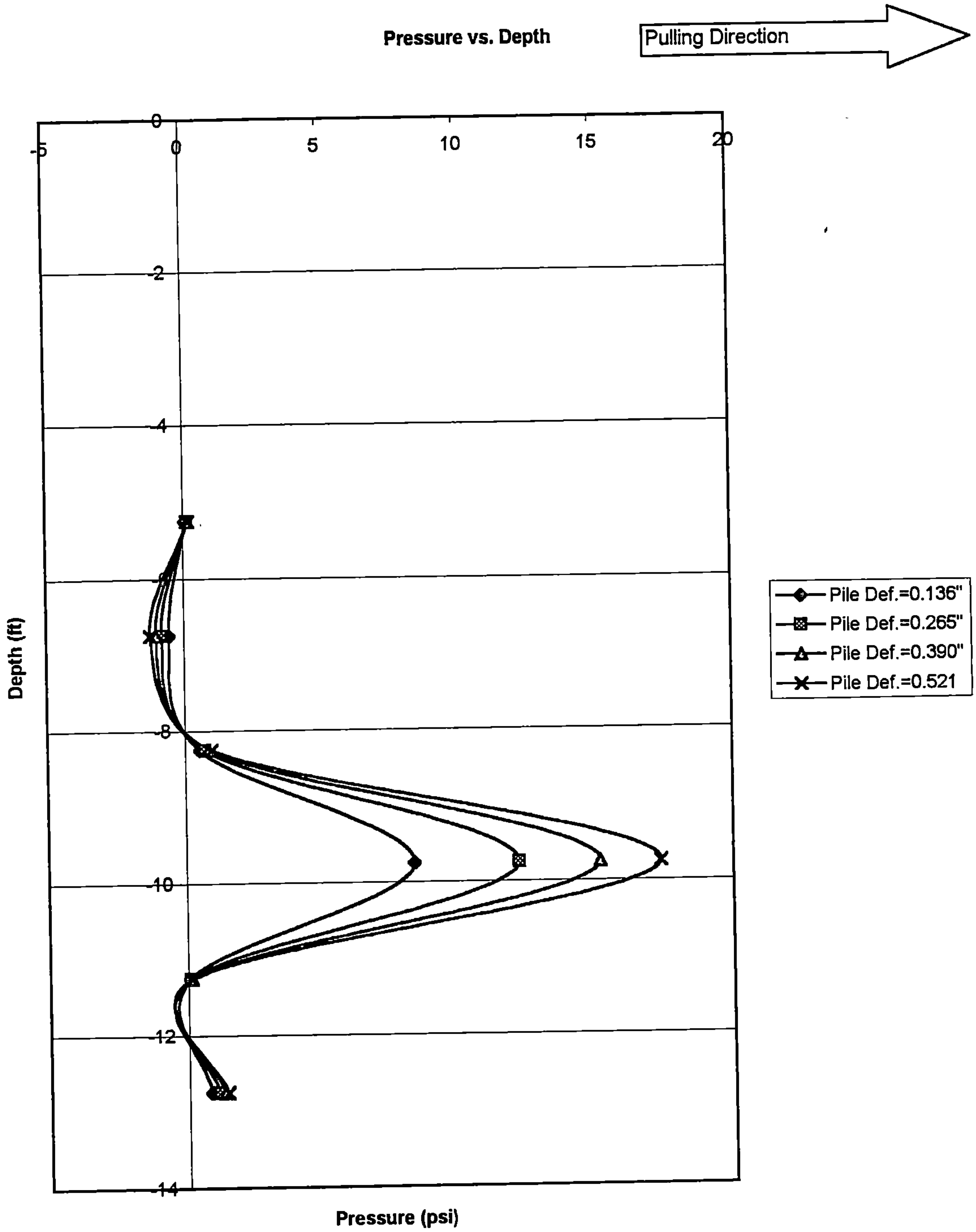

Figure A-5 Pressure vs Depth for pile # 3, pulling west. First 1/2" test, Oct,05,1999.
CD Side

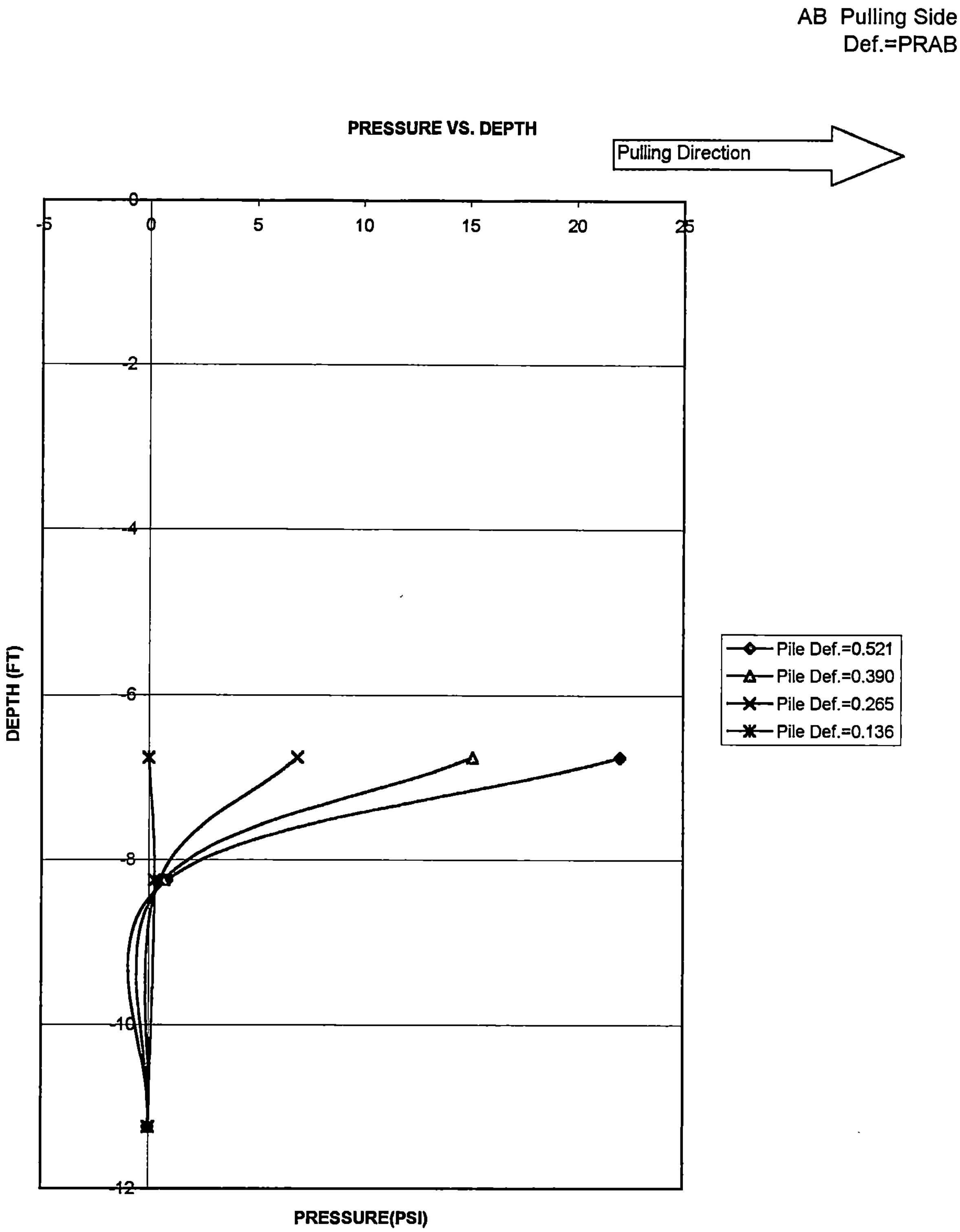

Figure A-6 Pressure vs Depth for pile # 3, pulling west. First 1/2" test, Oct,05,1999.
AB Side

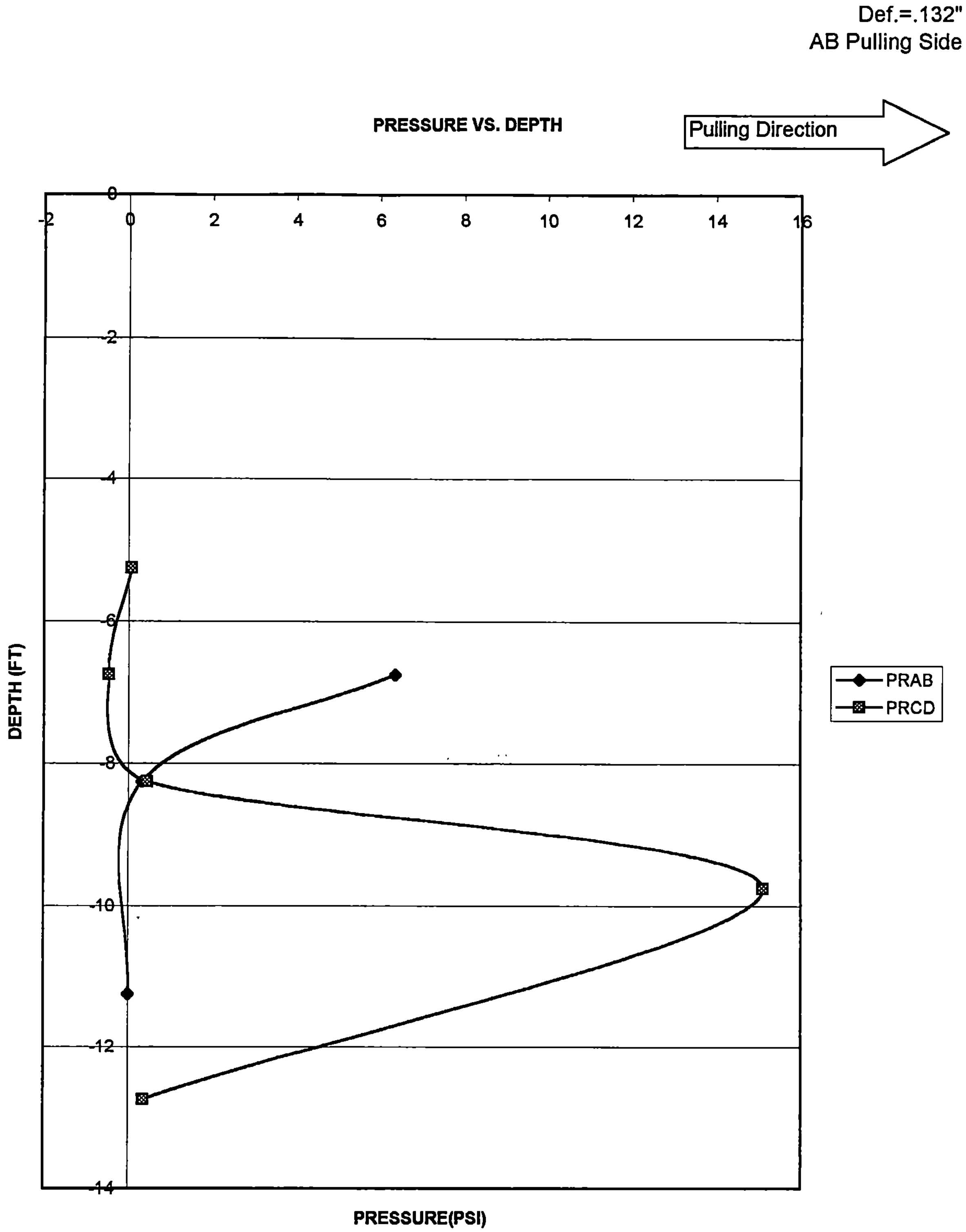

Figure A-7 Pressure vs Depth for pile # 3, pulling west. Second 1/2" test, Oct,9,1999.
Deflection of the pile=.132"

Def.=.262"
AB Pulling Side

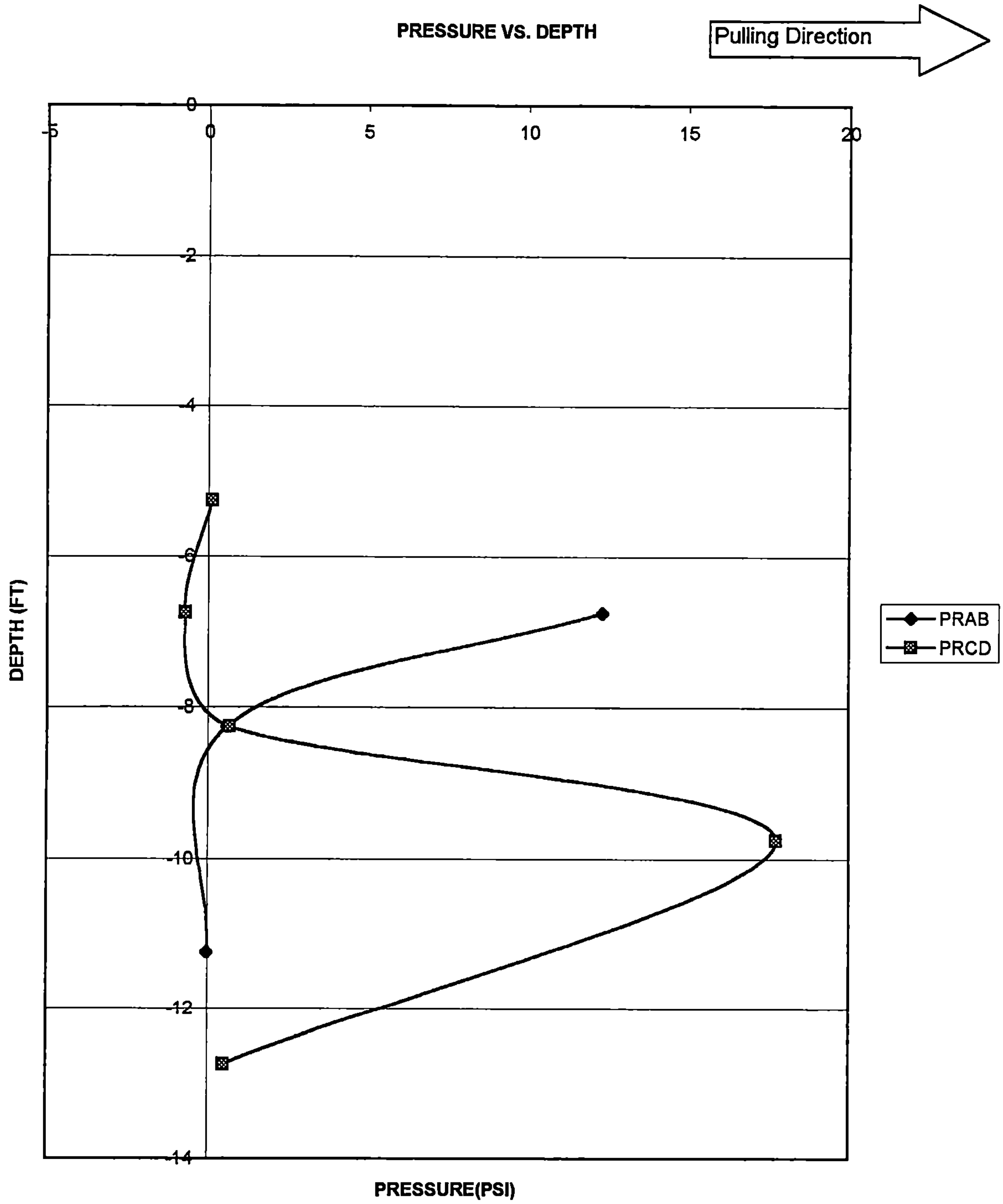

Figure A-8 Pressure vs Depth for pile # 3, pulling west. Second 1/2" test, Oct,9,1999.
Deflection of the pile=.262"

Def.=.396"
AB Pulling Side

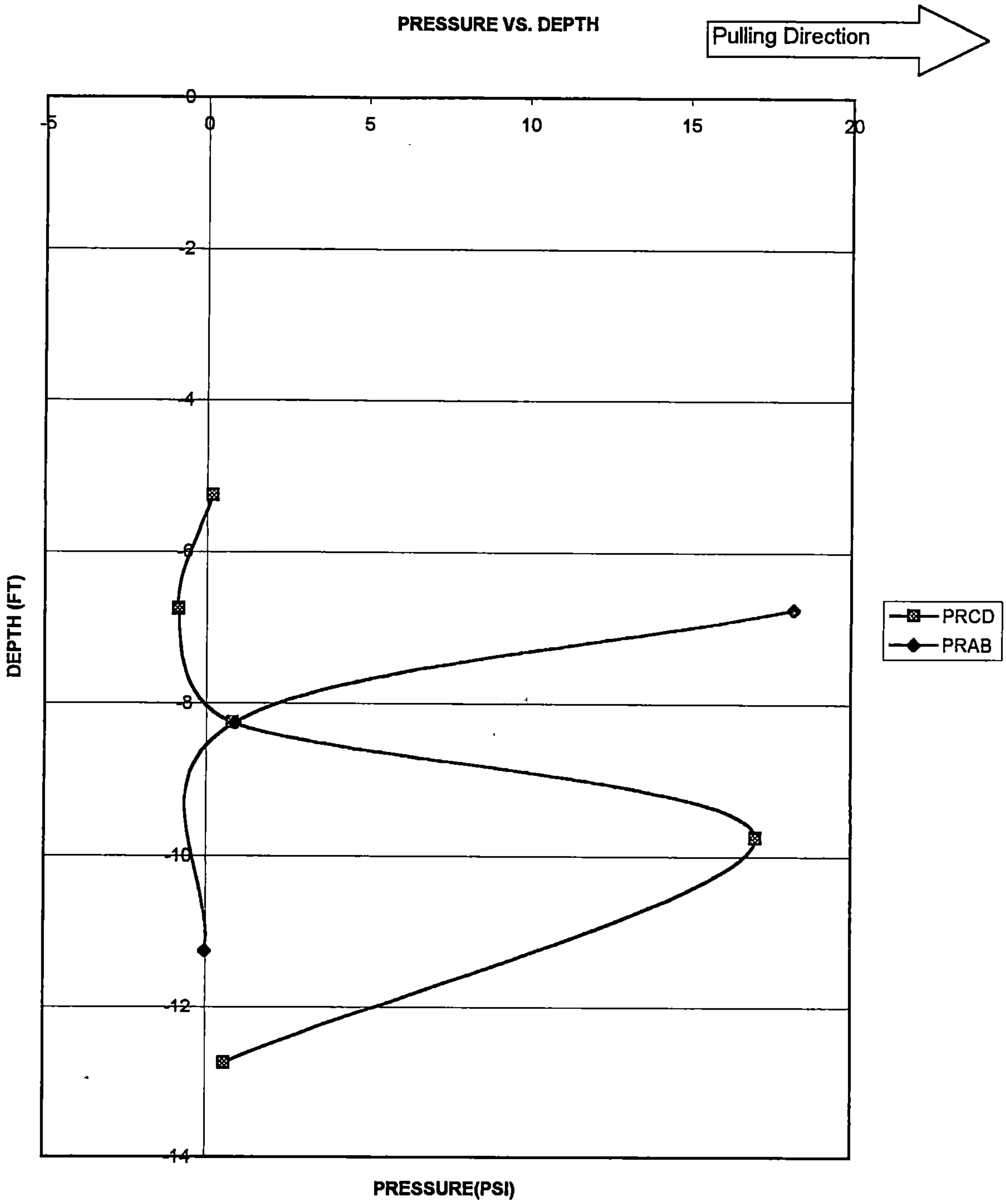

Figure A-9 Pressure vs Depth for pile # 3, pulling west. Second 1/2" test, Oct,9,1999.
Deflection of the pile=.396"

Def.=.531"
AB Pulling Side

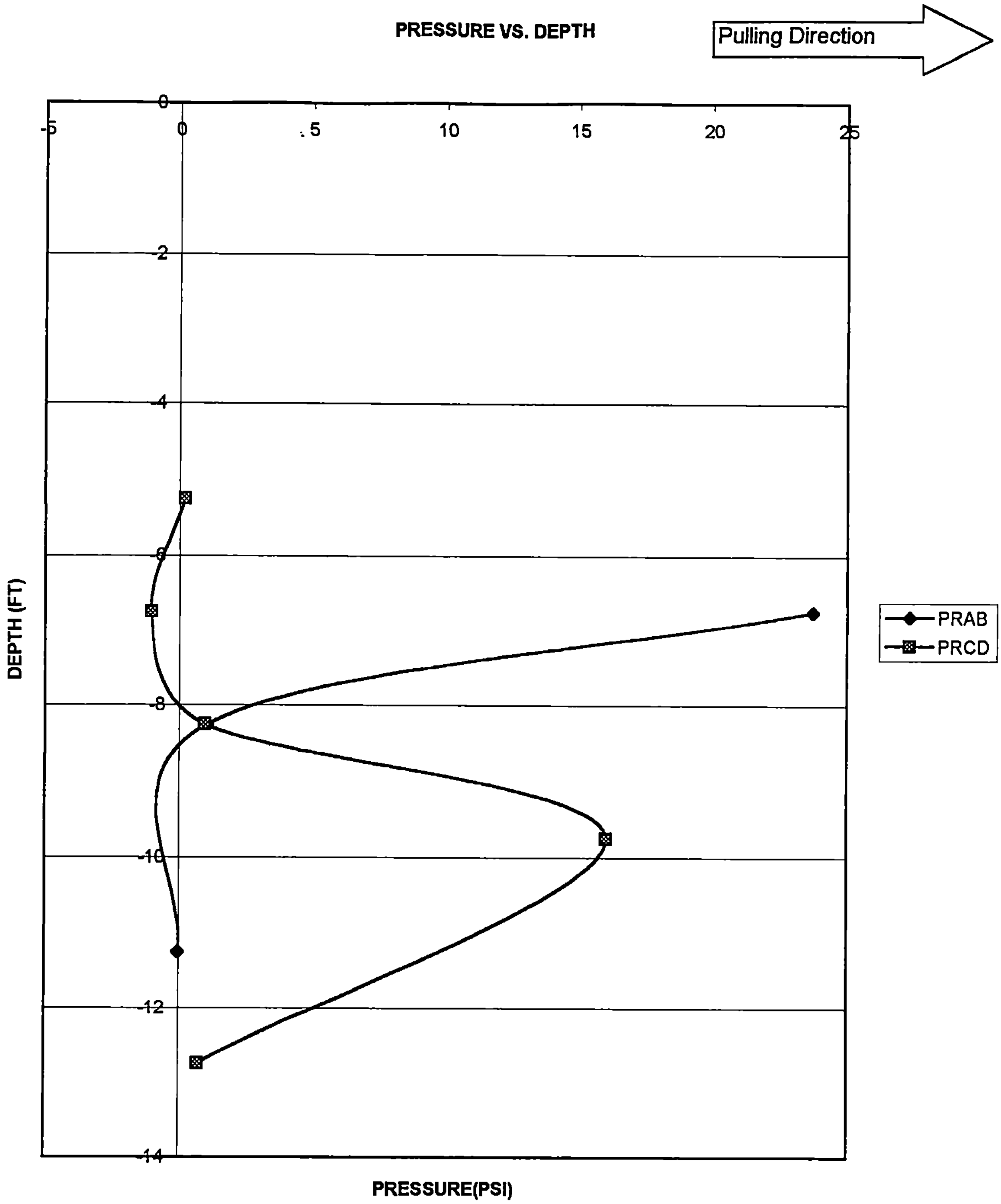

Figure A-10 Pressure vs Depth for pile # 3, pulling west. Second 1/2" test, Oct,9,1999.
Deflection of the pile=.531"

Def.=PRCD
AB Pulling Side

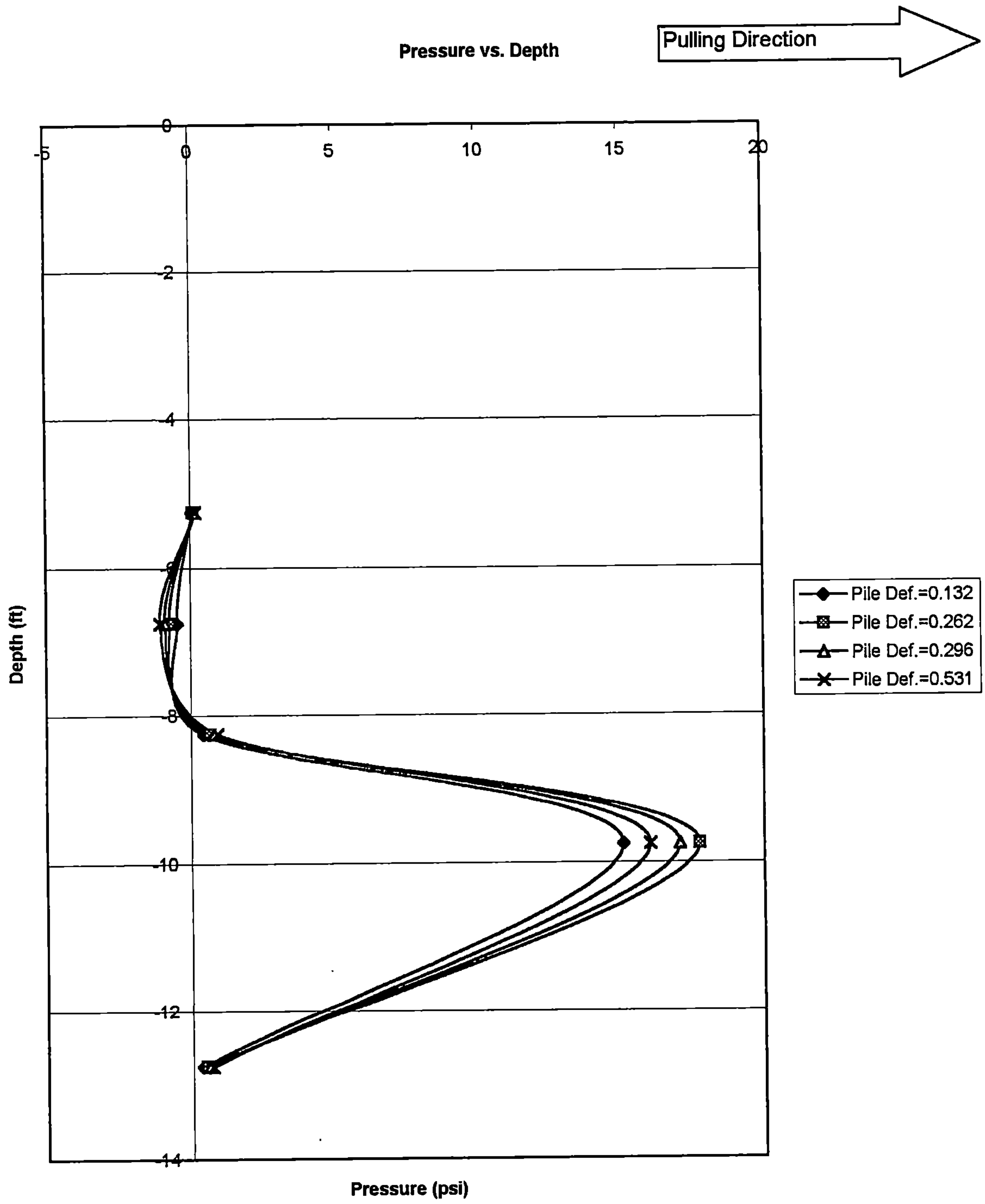

Figure A-11 Pressure vs Depth for pile # 3, pulling west. Second 1/2" test, Oct,9,1999.
CD Side

Def.=PRAB
AB Pulling Side

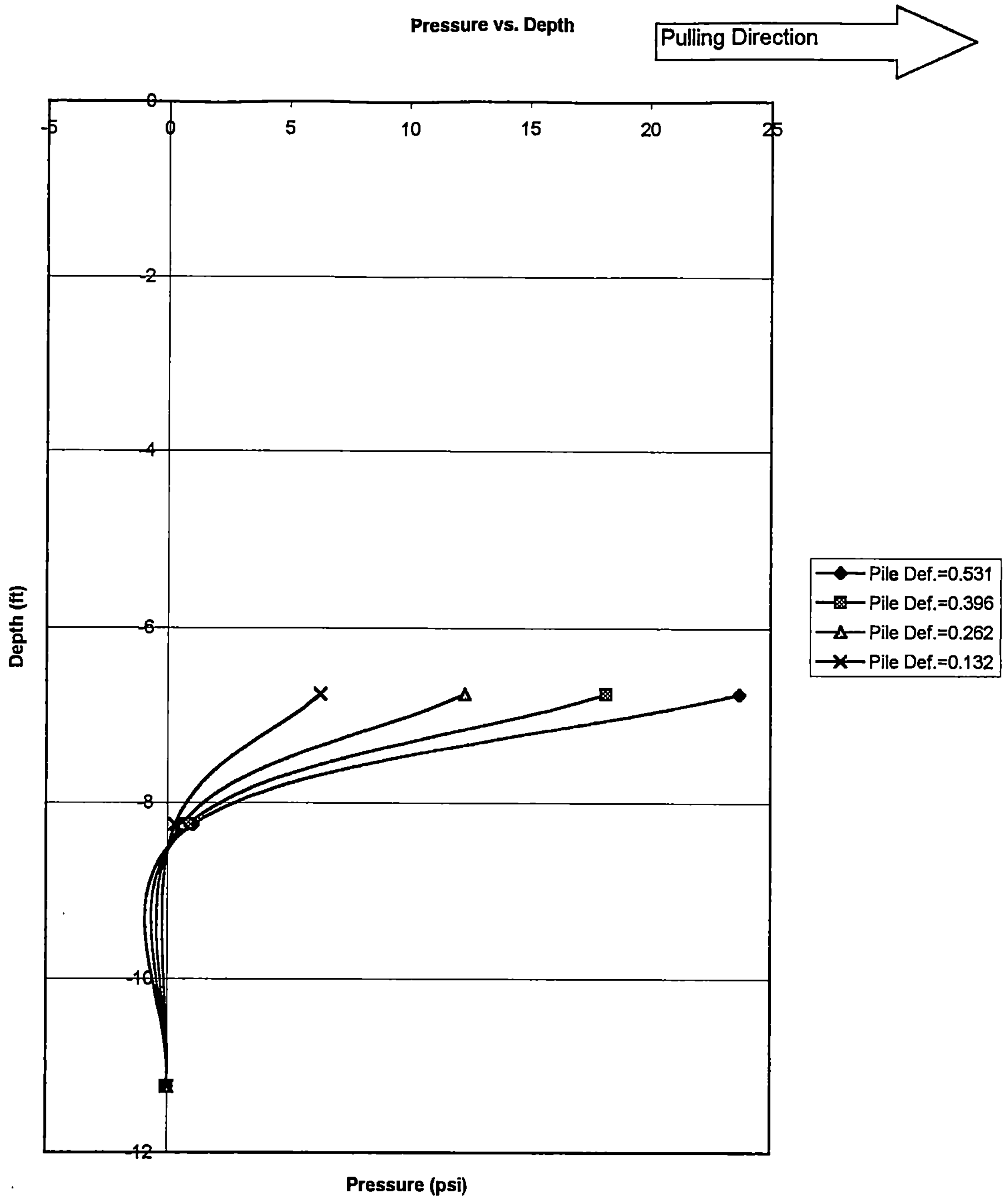

Figure A-12 Pressure vs Depth for pile # 3, pulling west. Second 1/2" test, Oct,9,1999.
AB Side

AB Pulling Side
Def.=.126"

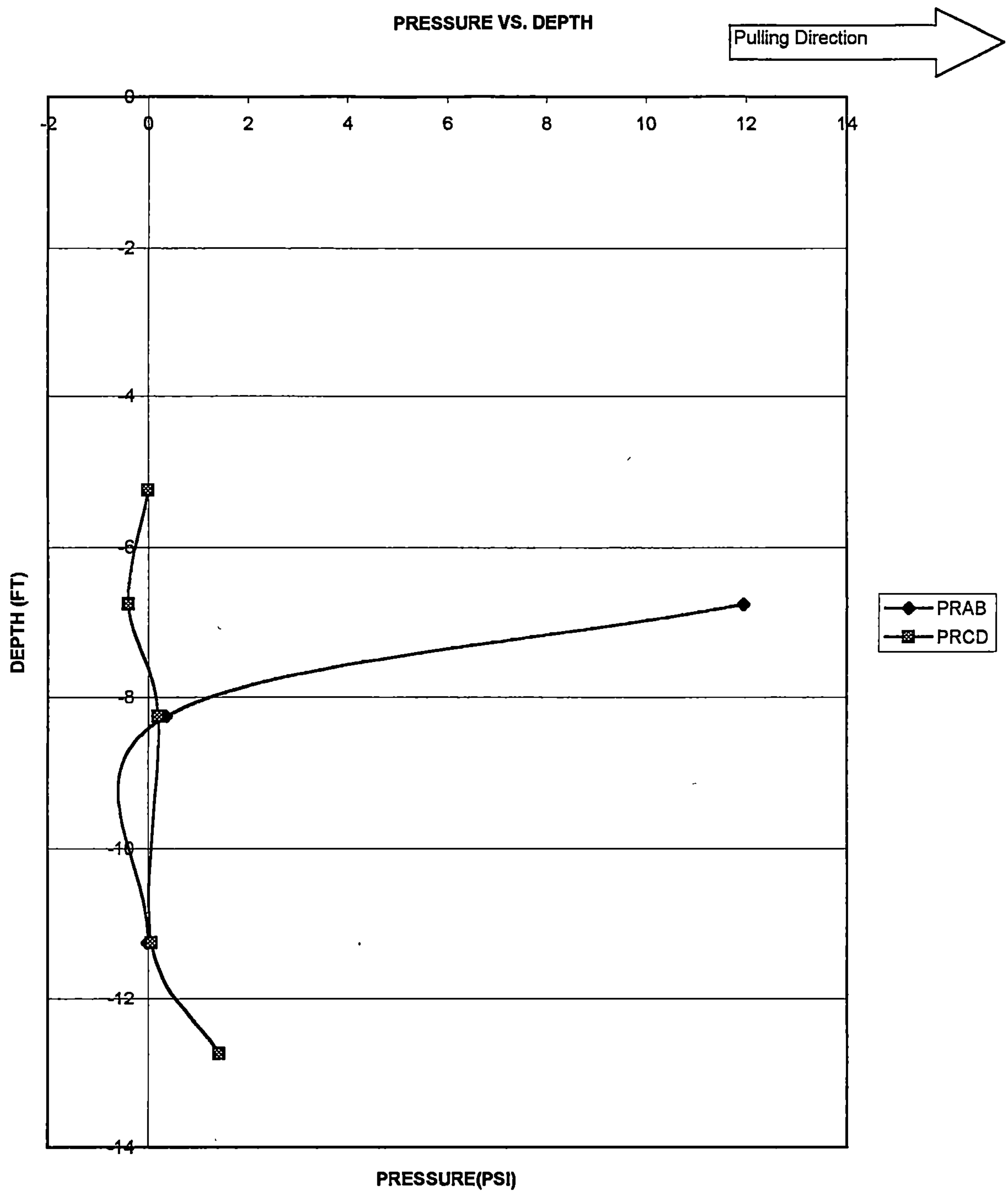

Figure A-13. Pressure vs Depth for pile # 3, pulling west. Third 1/2" test, Oct,16,1999.
Deflection of the pile=.126"

AB Pulling Side
Def.=.261"

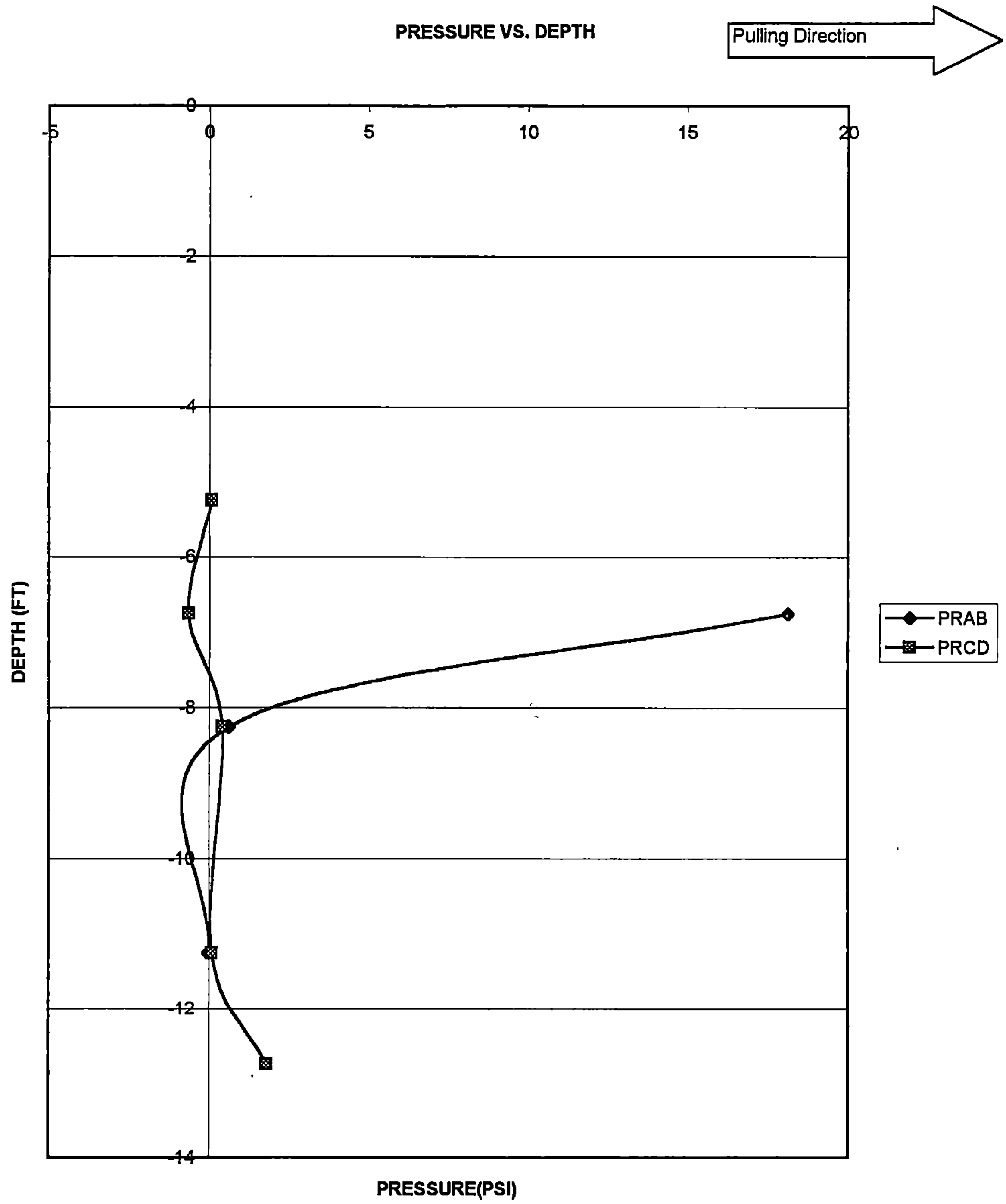

Figure A-14. Pressure vs Depth for pile # 3, pulling west. Third 1/2" test, Oct,16,1999.
Deflection of the pile=.261"

AB Pulling Side
Def.=.388"

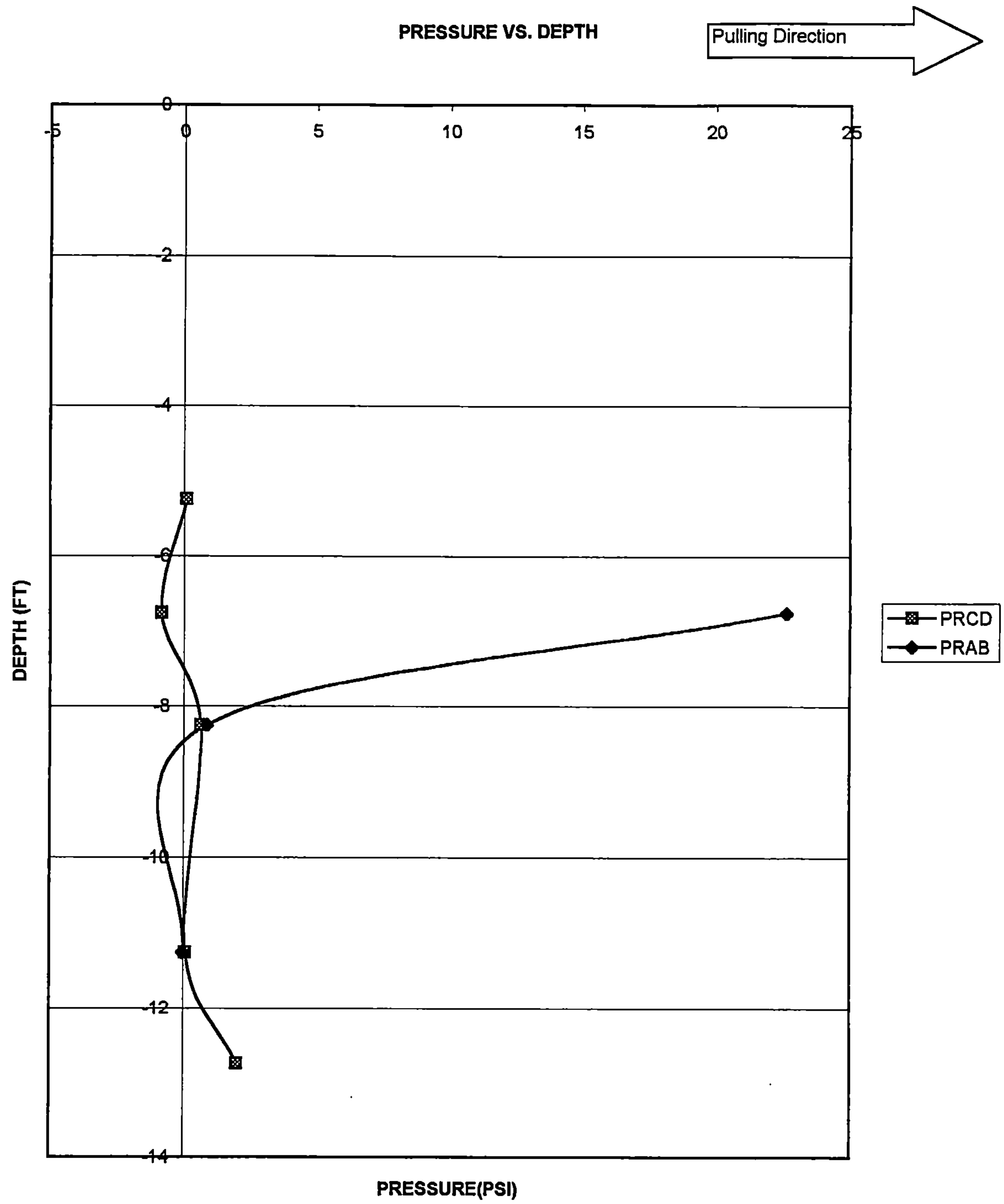

Figure A-15. Pressure vs Depth for pile # 3, pulling west. Third 1/2" test, Oct,16,1999.
Deflection of the pile=.388"

AB Pulling Side
Def.=.549"

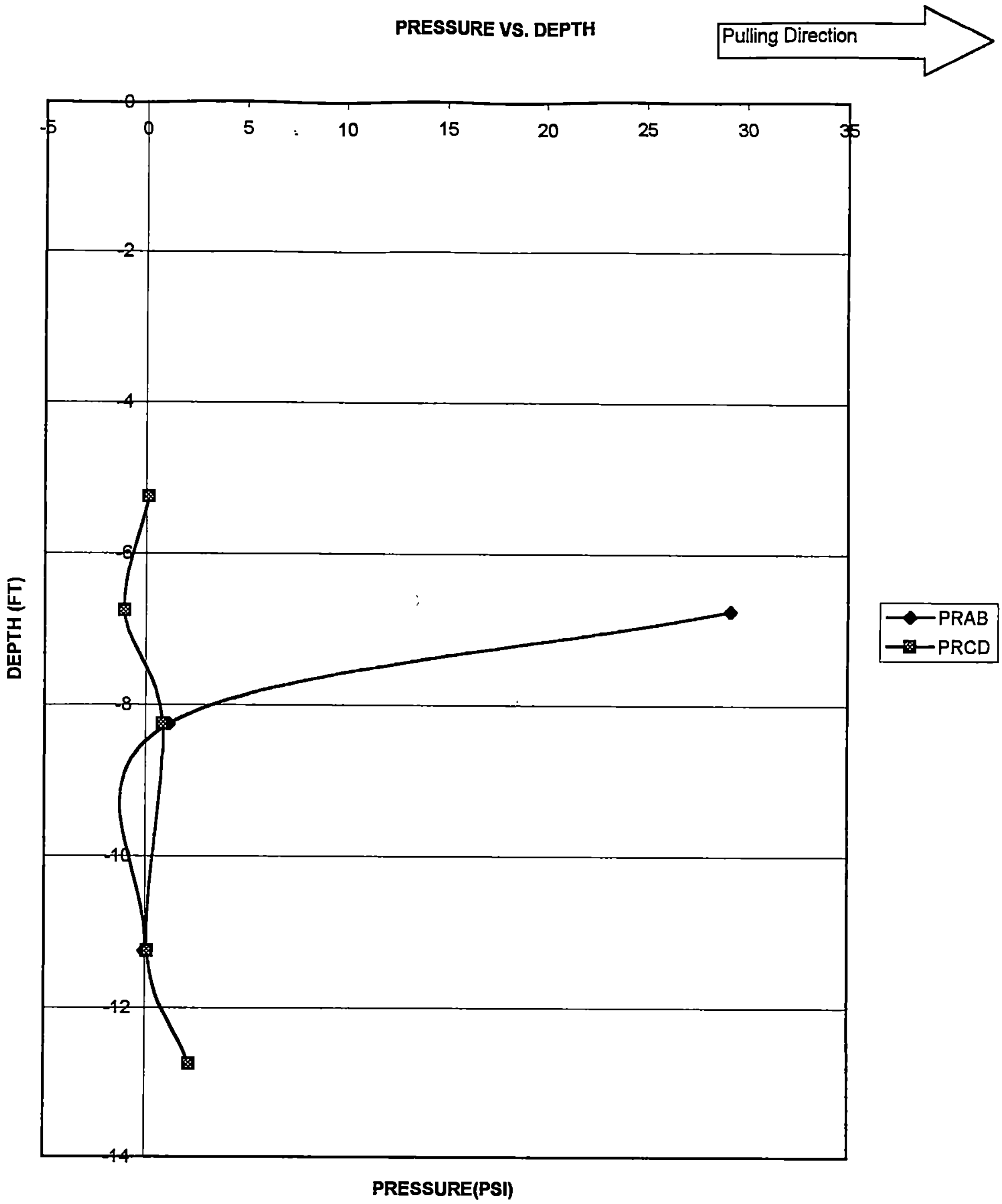

Figure A-16. Pressure vs Depth for pile # 3, pulling west. Third 1/2" test, Oct,16,1999.
Deflection of the pile=.549"

AB Pulling Side
Def.=PRCD

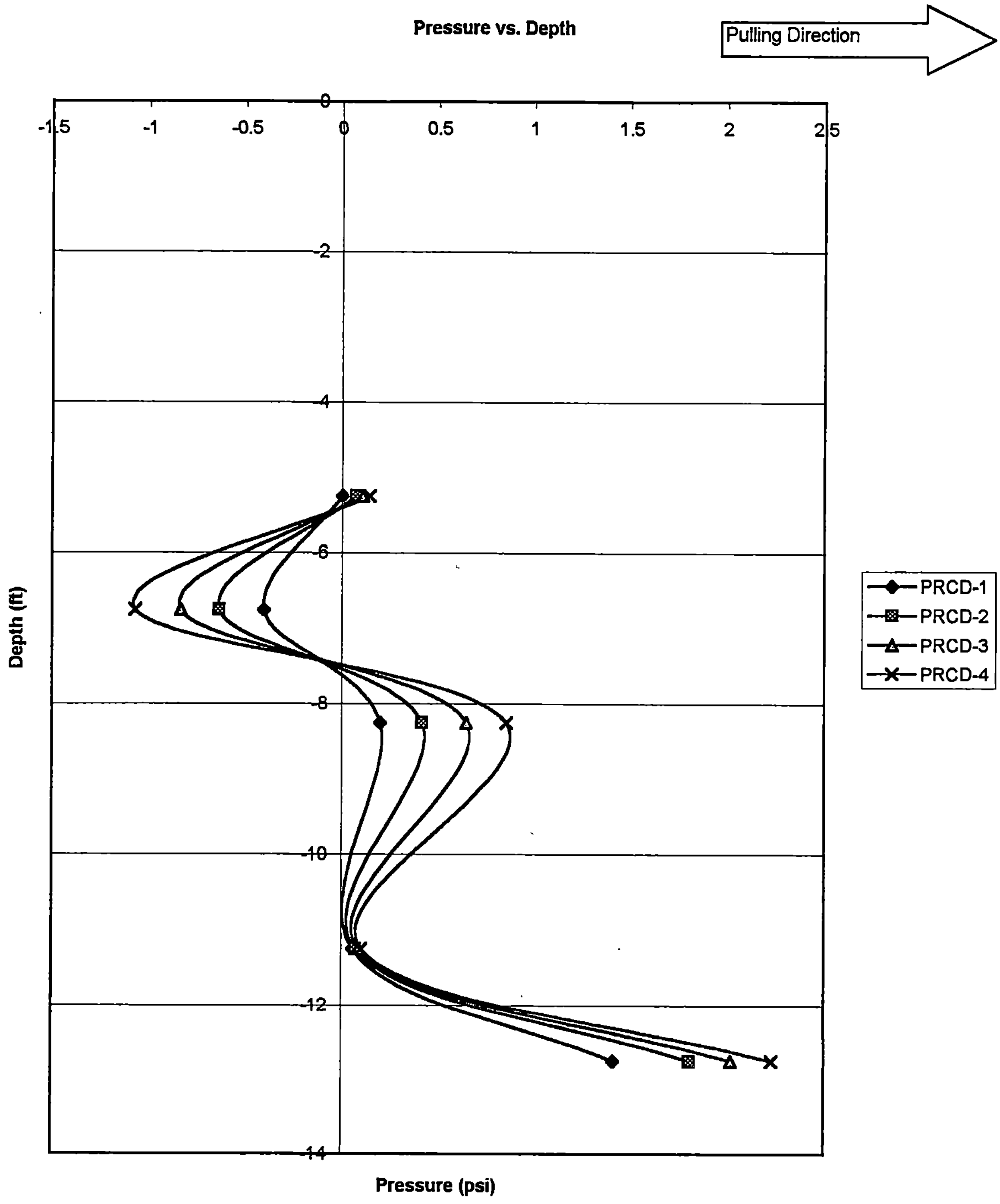

Figure A-17. Pressure vs Depth for pile # 3, pulling west. Third 1/2" test, Oct,16,1999.
CD Side

AB Pulling Side
Def.=PRAB

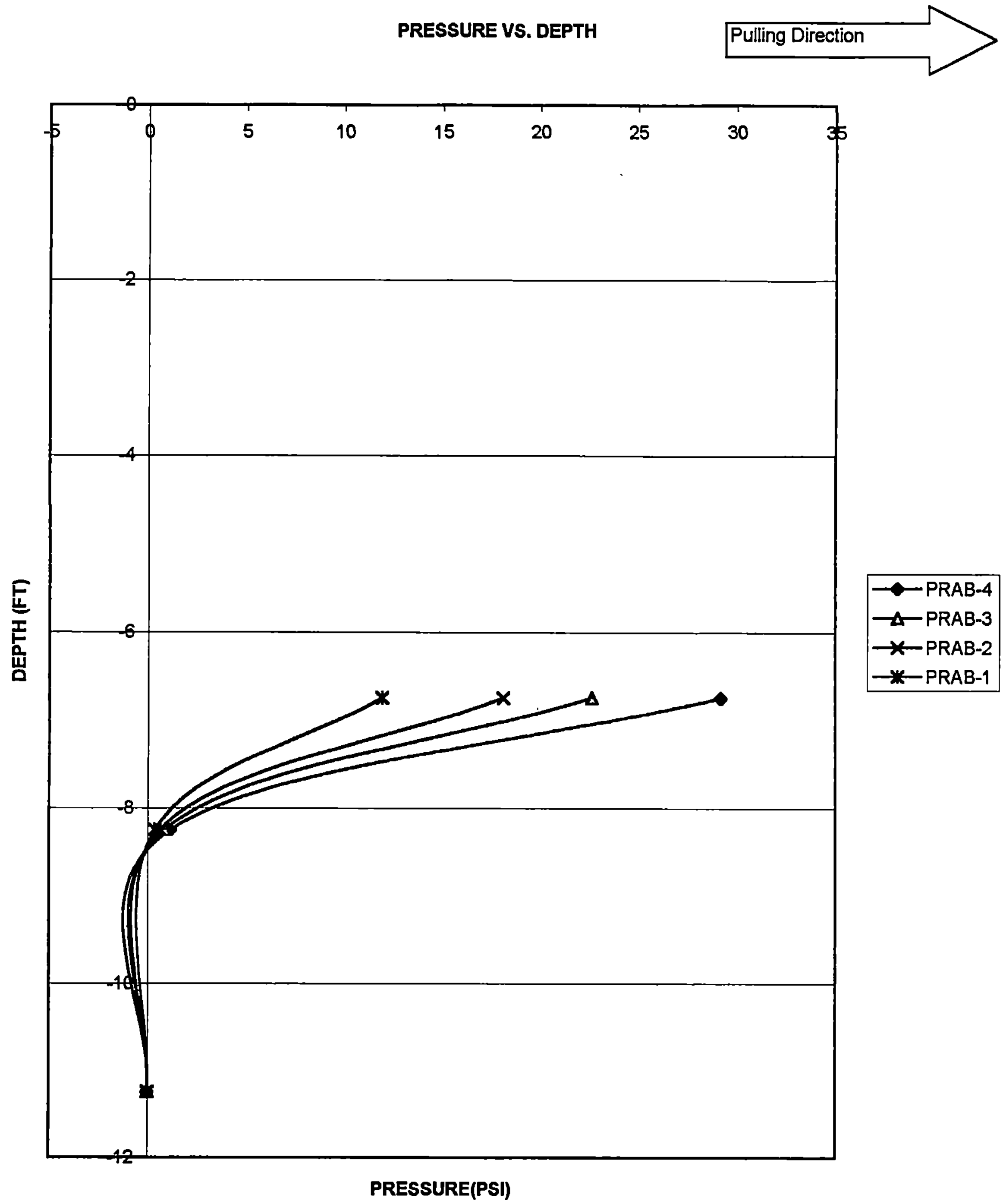

Figure A-18. Pressure vs Depth for pile # 3, pulling west. Third 1/2" test, Oct,16,1999.
AB Side

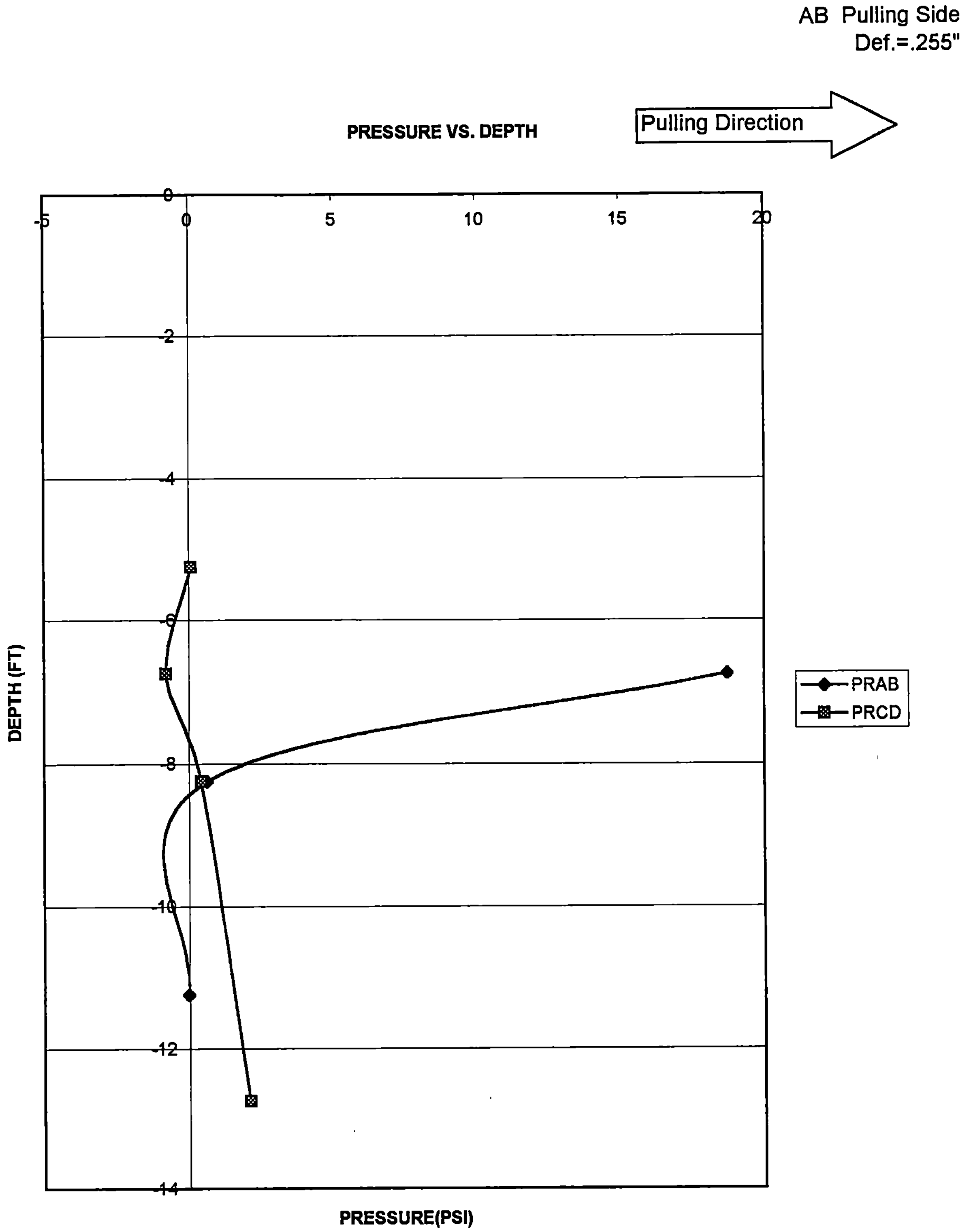

Figure A-19. Presure vs Depth for pile # 3, puling west. First 1" test, Oct,23,1999.
Deflection of the pile=.255"

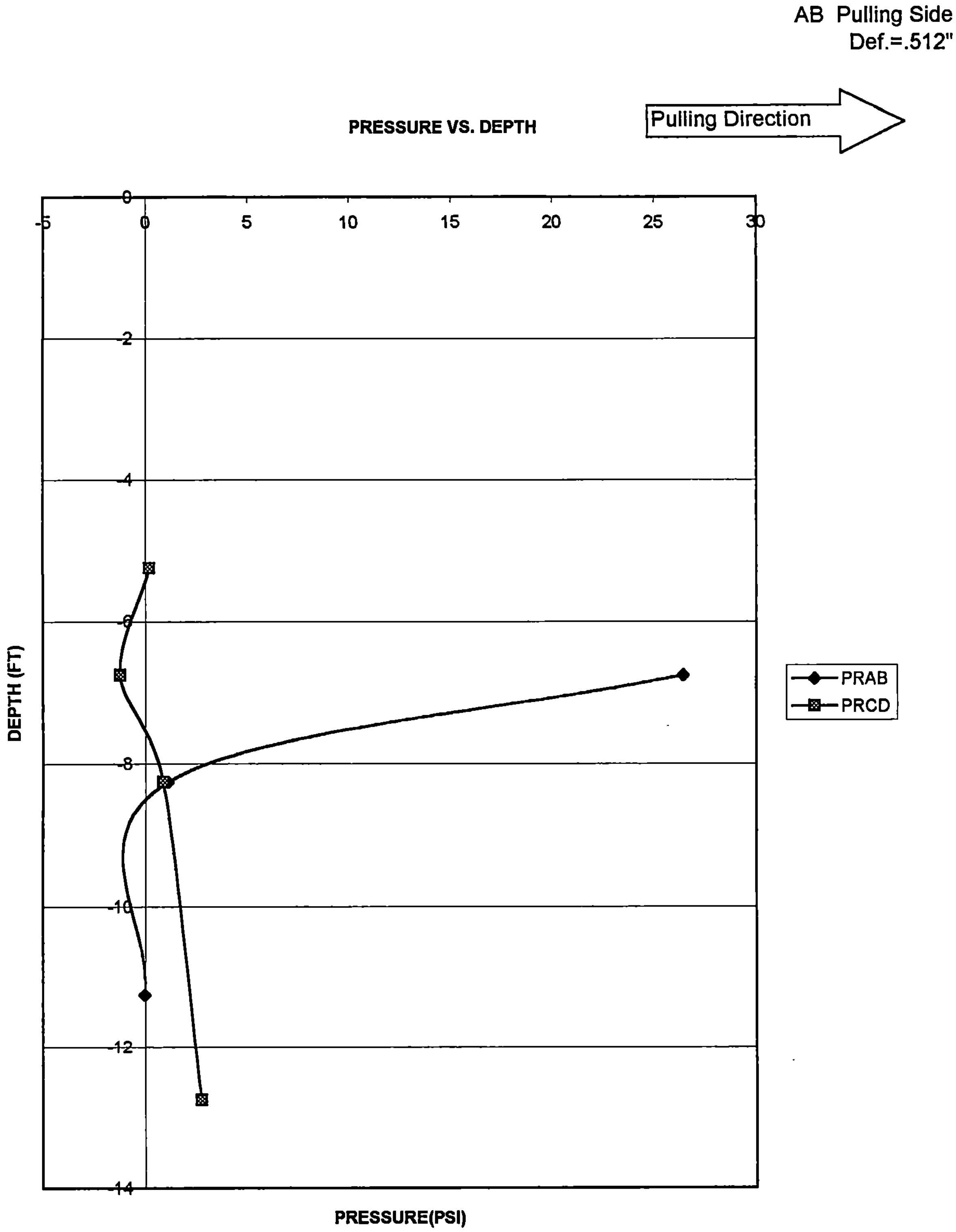

Figure A-20. Presure vs Depth for pile # 3, puling west. First 1" test, Oct,23,1999. Deflection of the pile=.512"

AB Pulling Side
Def.=.652"

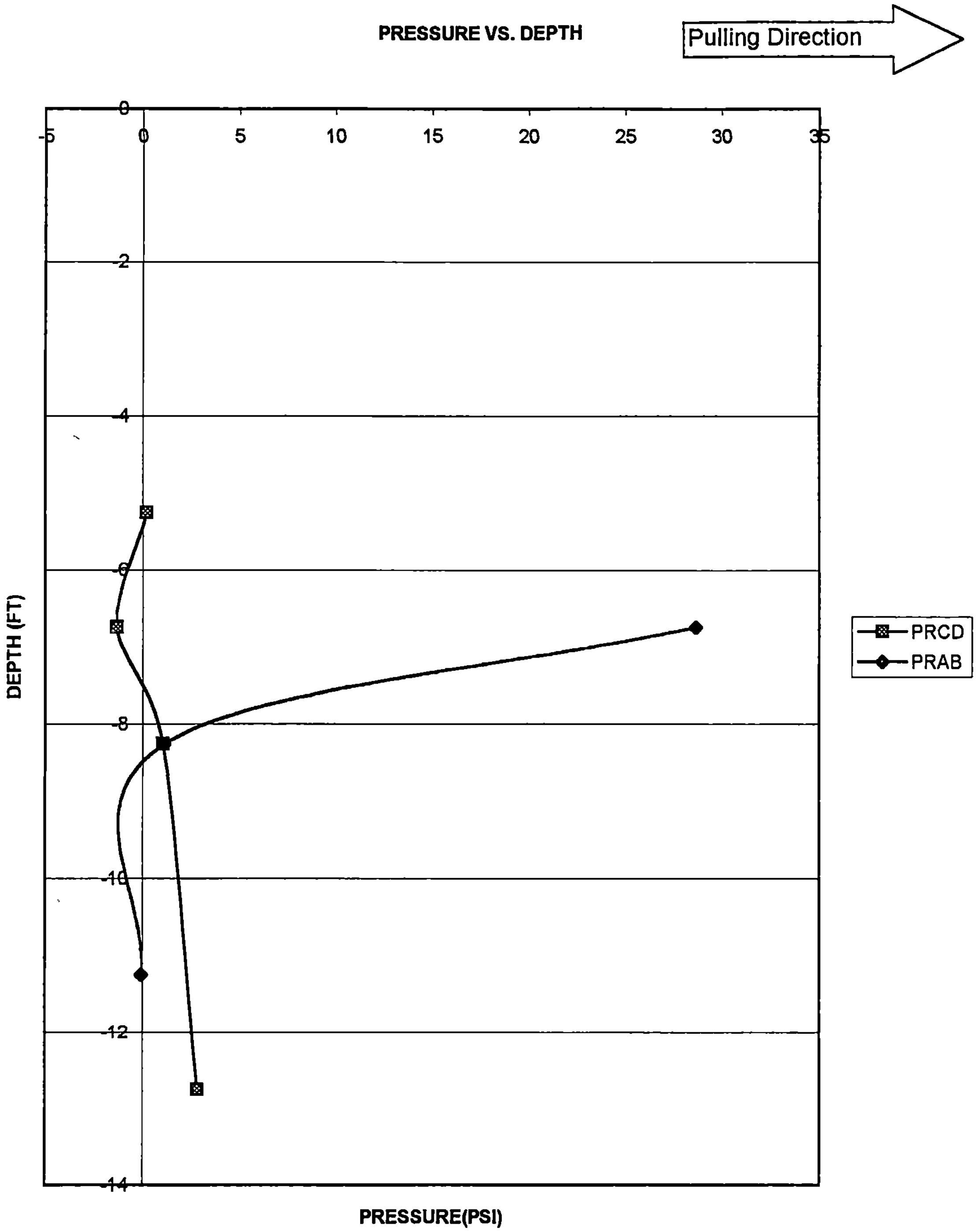

Figure A-21. Presure vs Depth for pile # 3, puling west. First 1" test, Oct,23,1999.
Deflection of the pile=.652"

AB Pulling Side
Def.=.749"

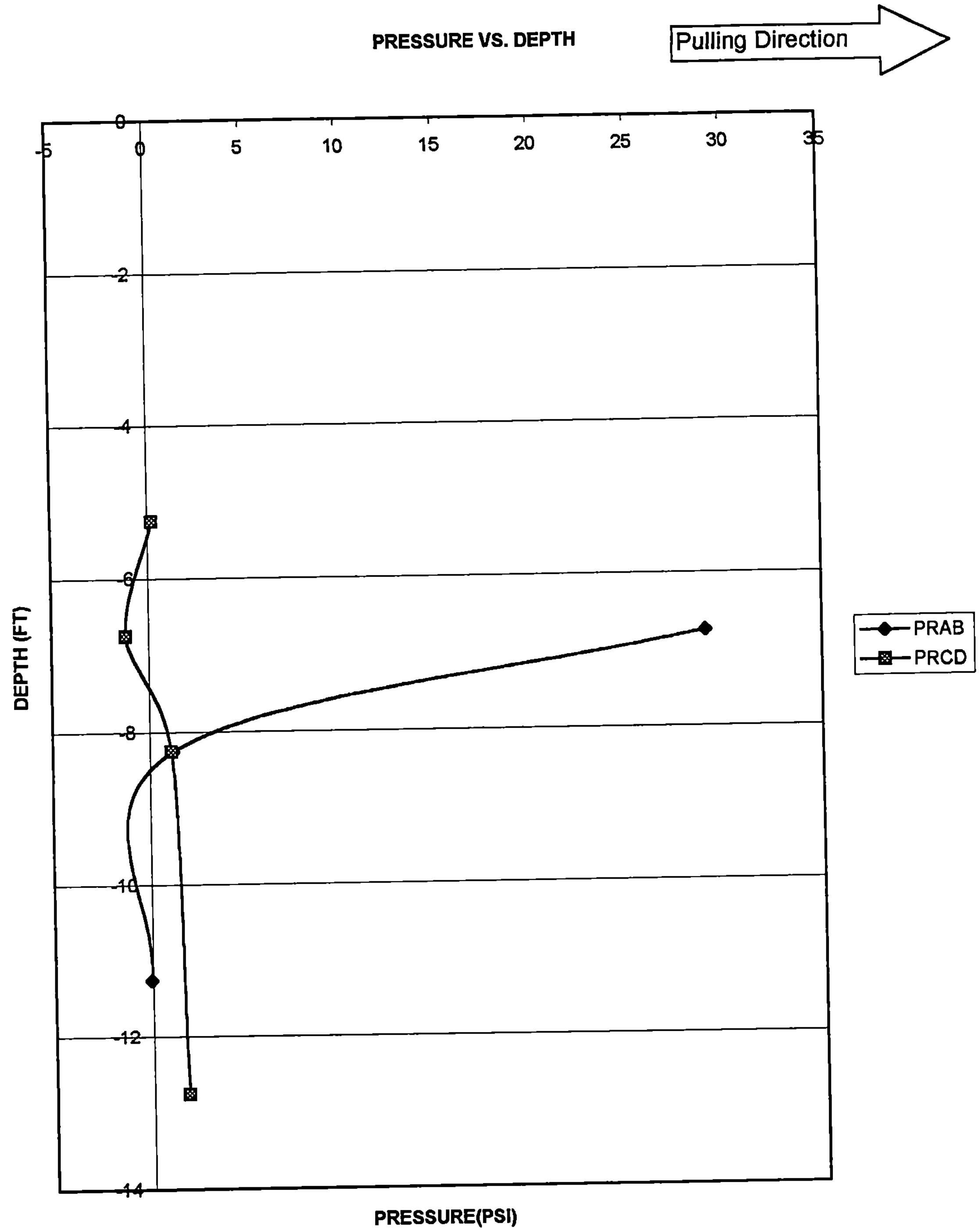

Figure A-22. Presure vs Depth for pile # 3, puling west. First 1" test, Oct,23,1999.
Deflection of the pile=.749"

AB Pulling Side
Def.=.912"

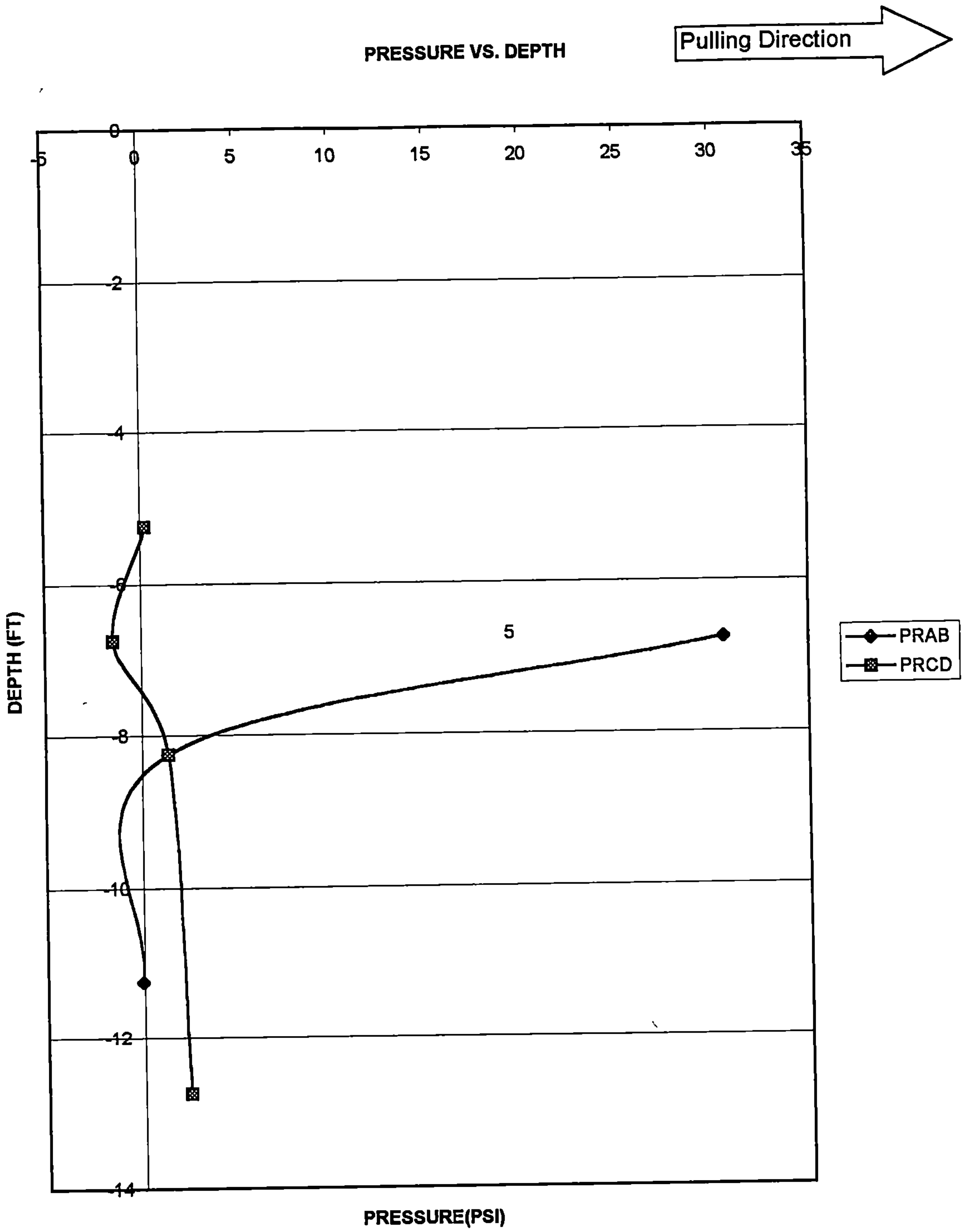

Figure A-23. Presure vs Depth for pile # 3, puling west. First 1" test, Oct,23,1999.
Deflection of the pile=.912"

AB Pulling Side
Def.=1.031"

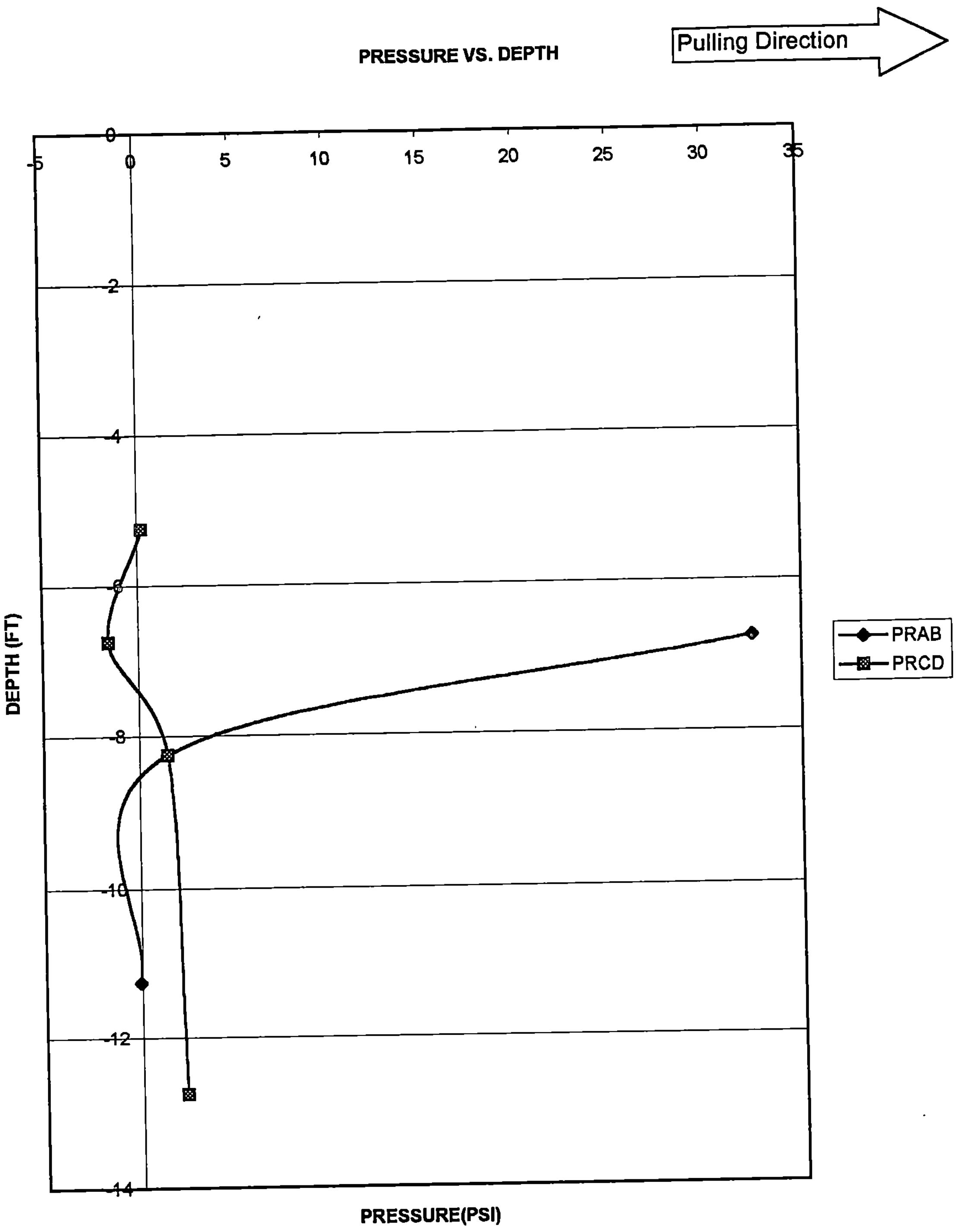

Figure A-24. Presure vs Depth for pile # 3, puling west. First 1" test, Oct,23,1999.
Deflection of the pile=1.031"

AB Pulling Side
Def.=PRAB

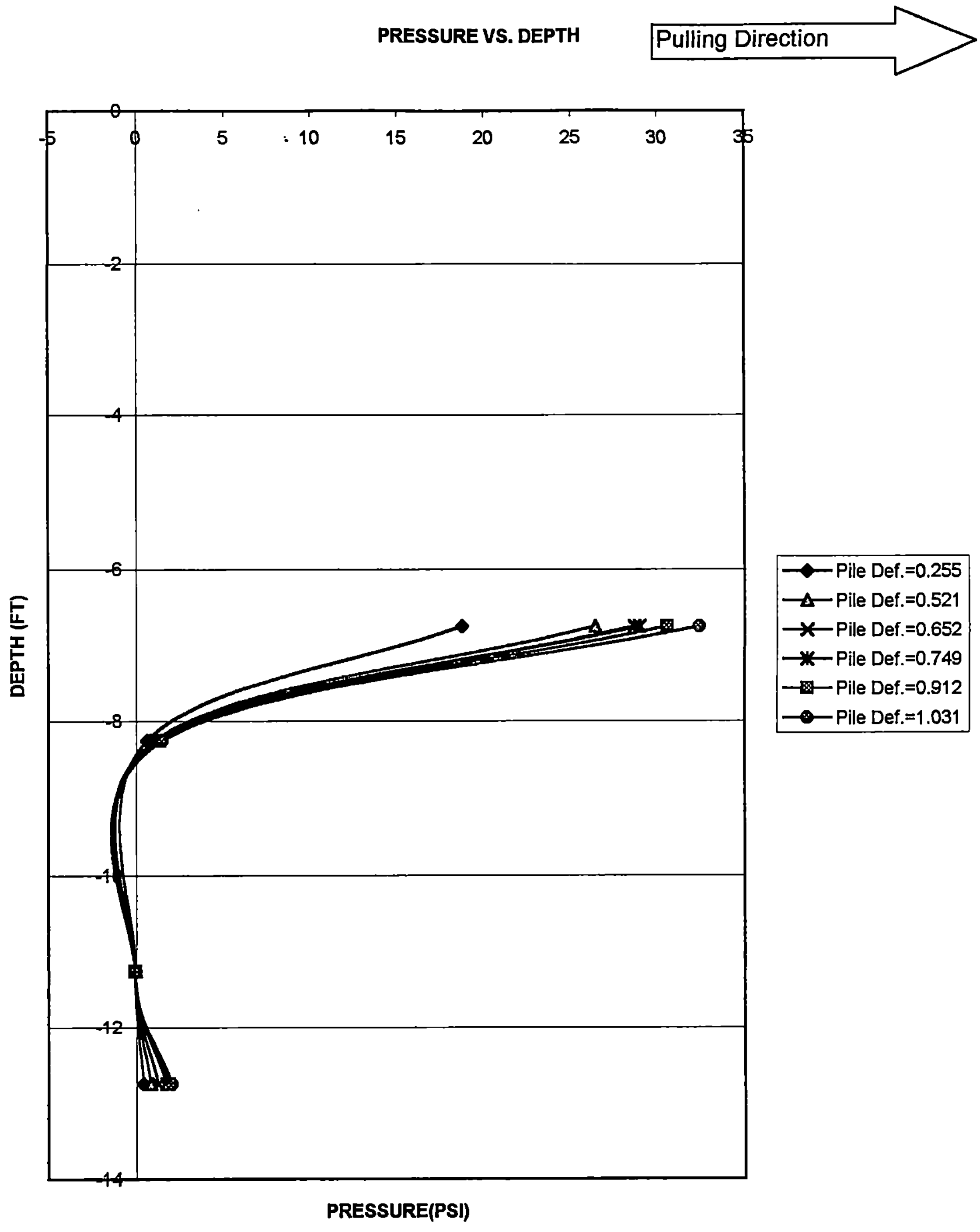

Figure A-25. Presure vs Depth for pile # 3, puling west. First 1" test, Oct,23,1999.
AB Side

AB Pulling Side
Def.=PRCD

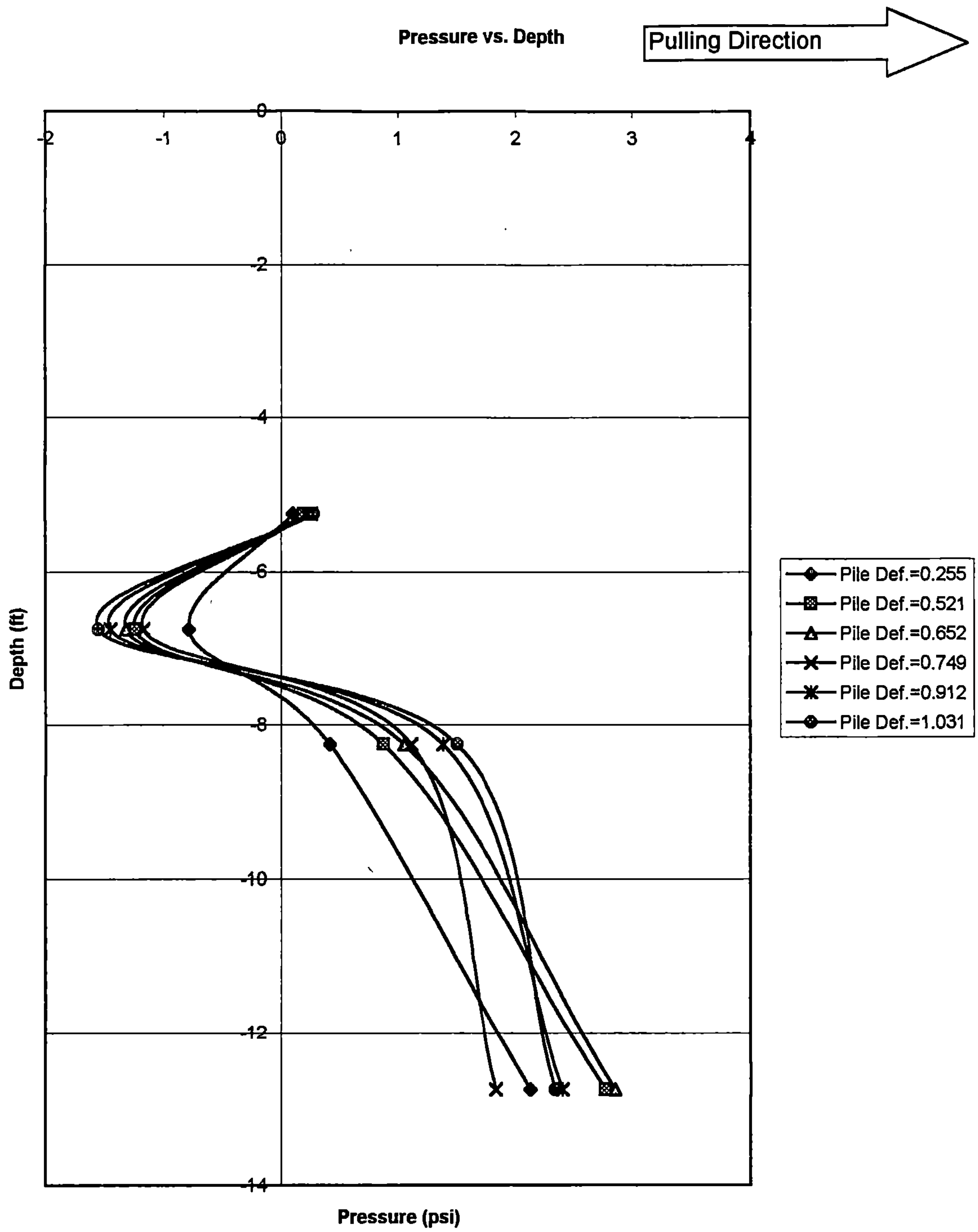

Figure A-26. Presure vs Depth for pile # 3, puling west. First 1" test, Oct,23,1999.
CD Side

AB Pulling Side
Def.=.254"

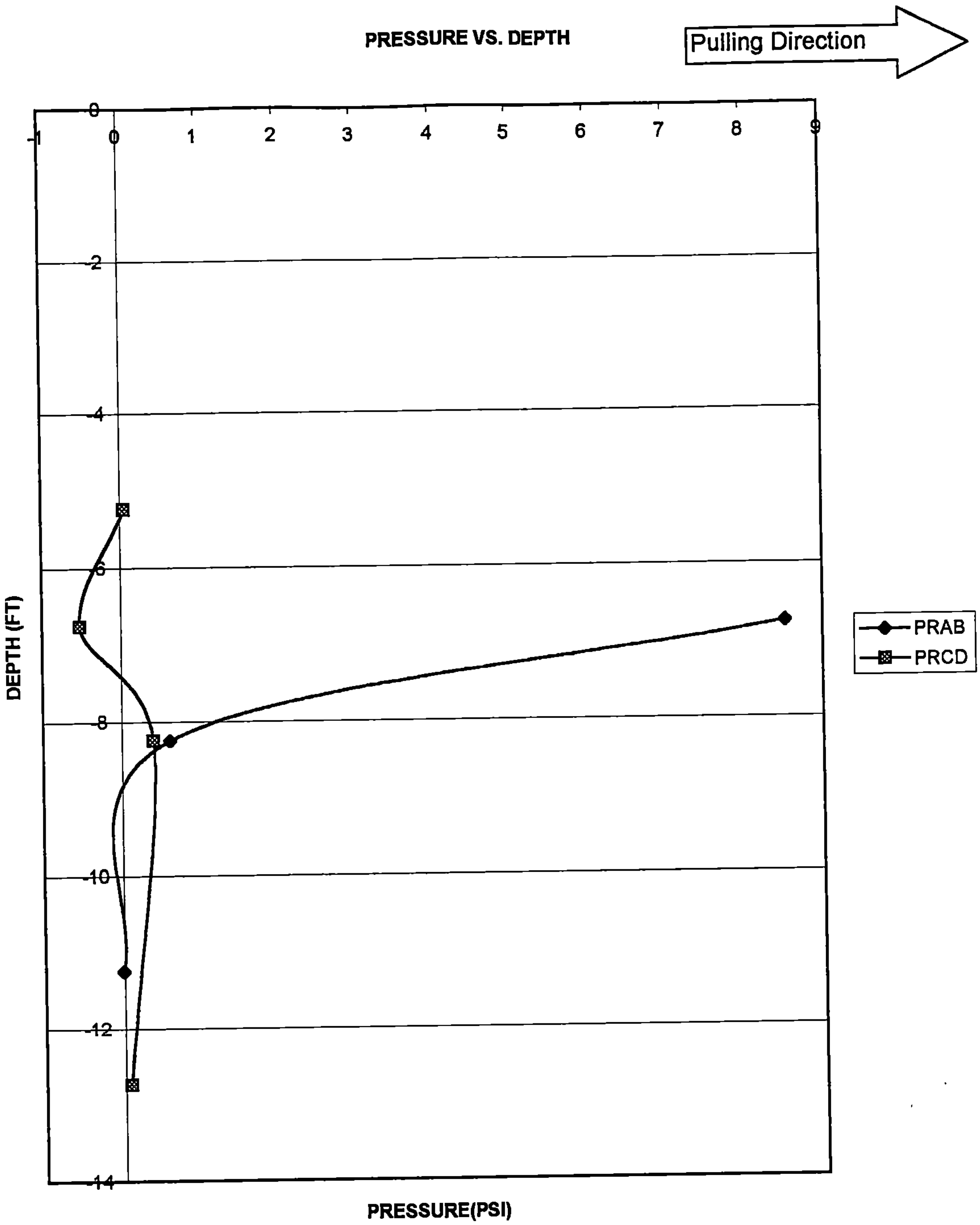

Figure A-27. Pressure vs Depth for pile # 3, pulling west. Second 1" test, Oct,30,1999.
Deflection of the pile=.254"

AB Pulling Side
Def.=.504"

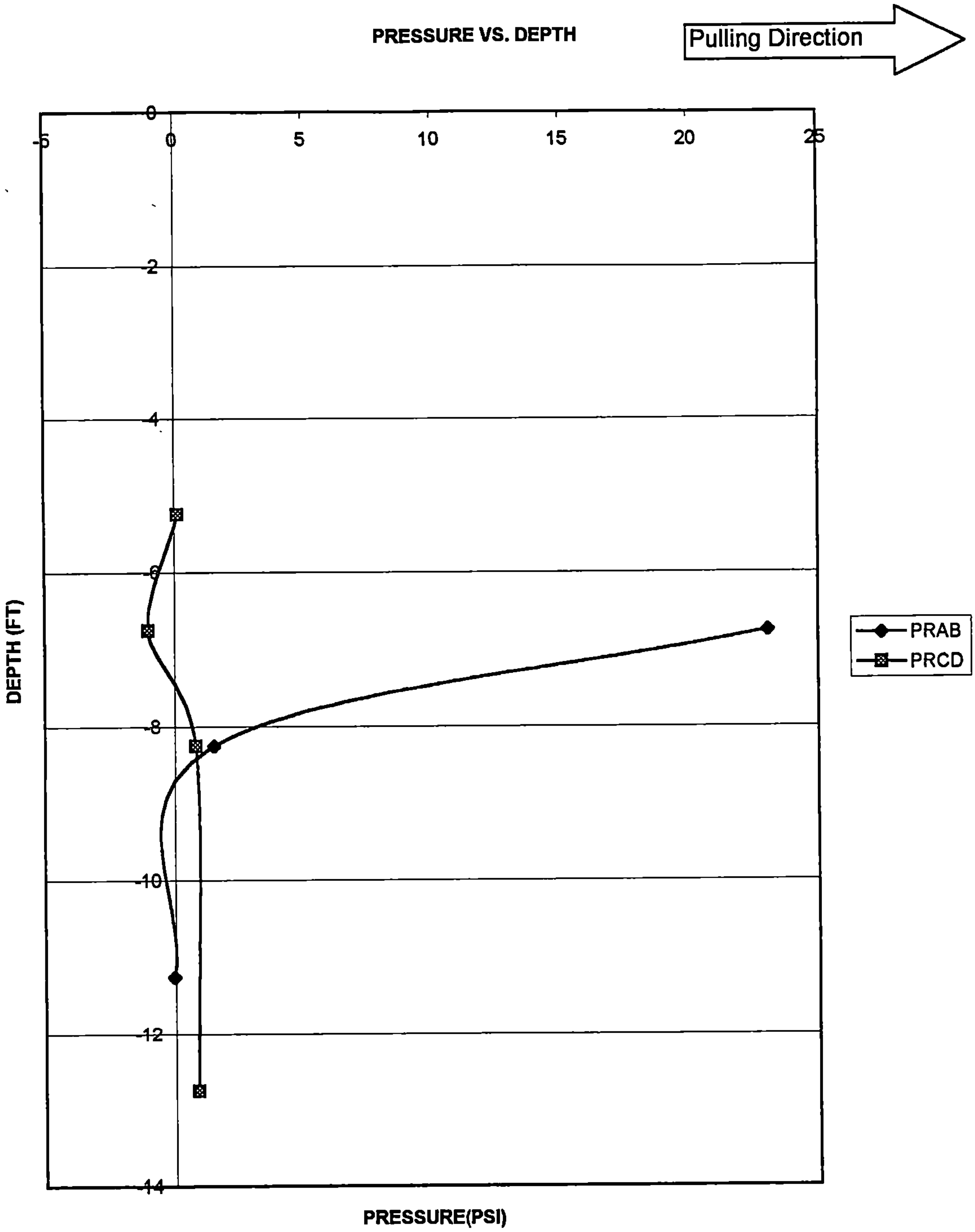

Figure A-28. Pressure vs Depth for pile # 3, pulling west. Second 1" test, Oct,30,1999.
Deflection of the pile=.504"

AB Pulling Side
Def.=.638"

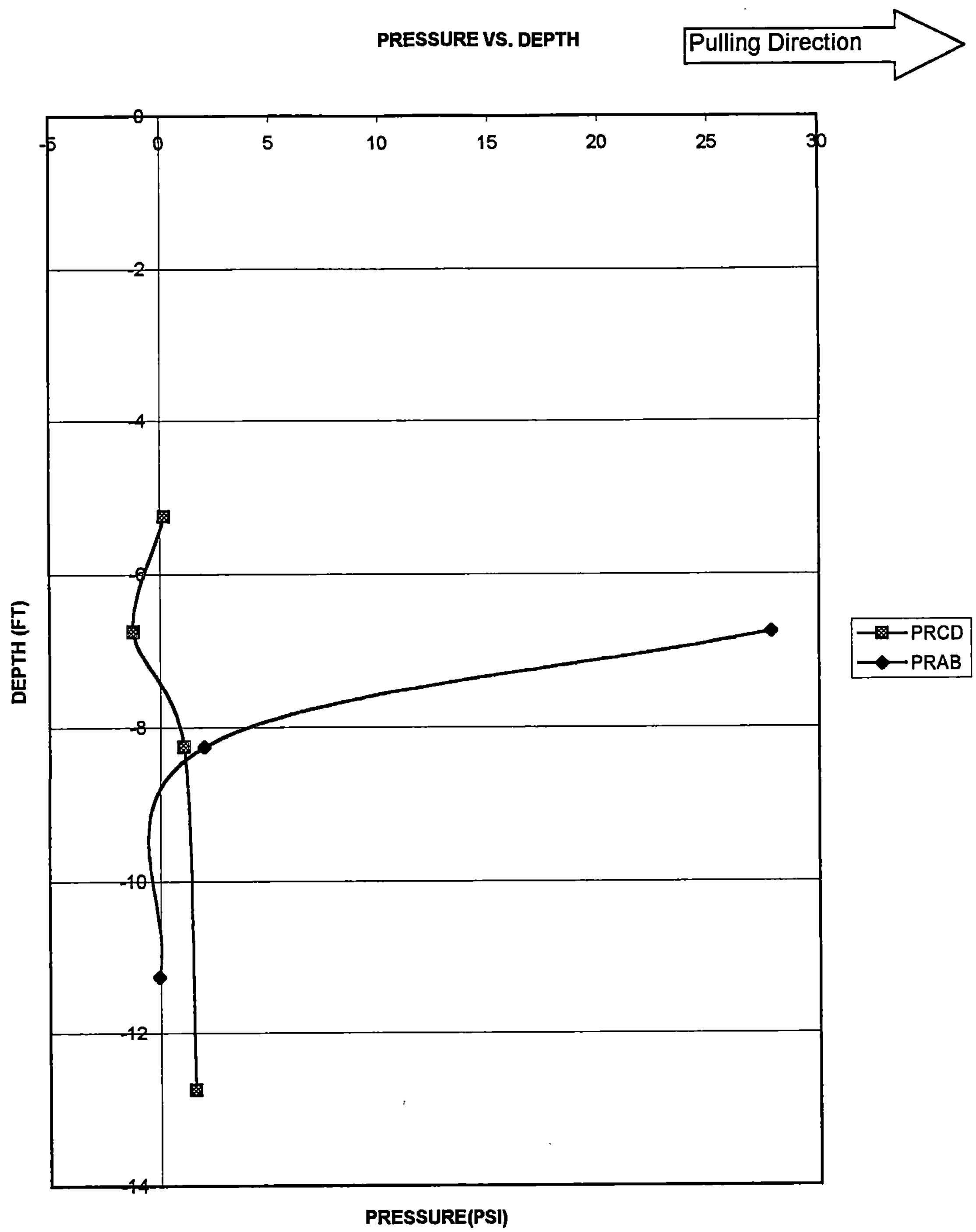

Figure A-29. Pressure vs Depth for pile # 3, pulling west. Second 1" test, Oct,30,1999.
Deflection of the pile=.638"

AB Pulling Side
Def.=.764"

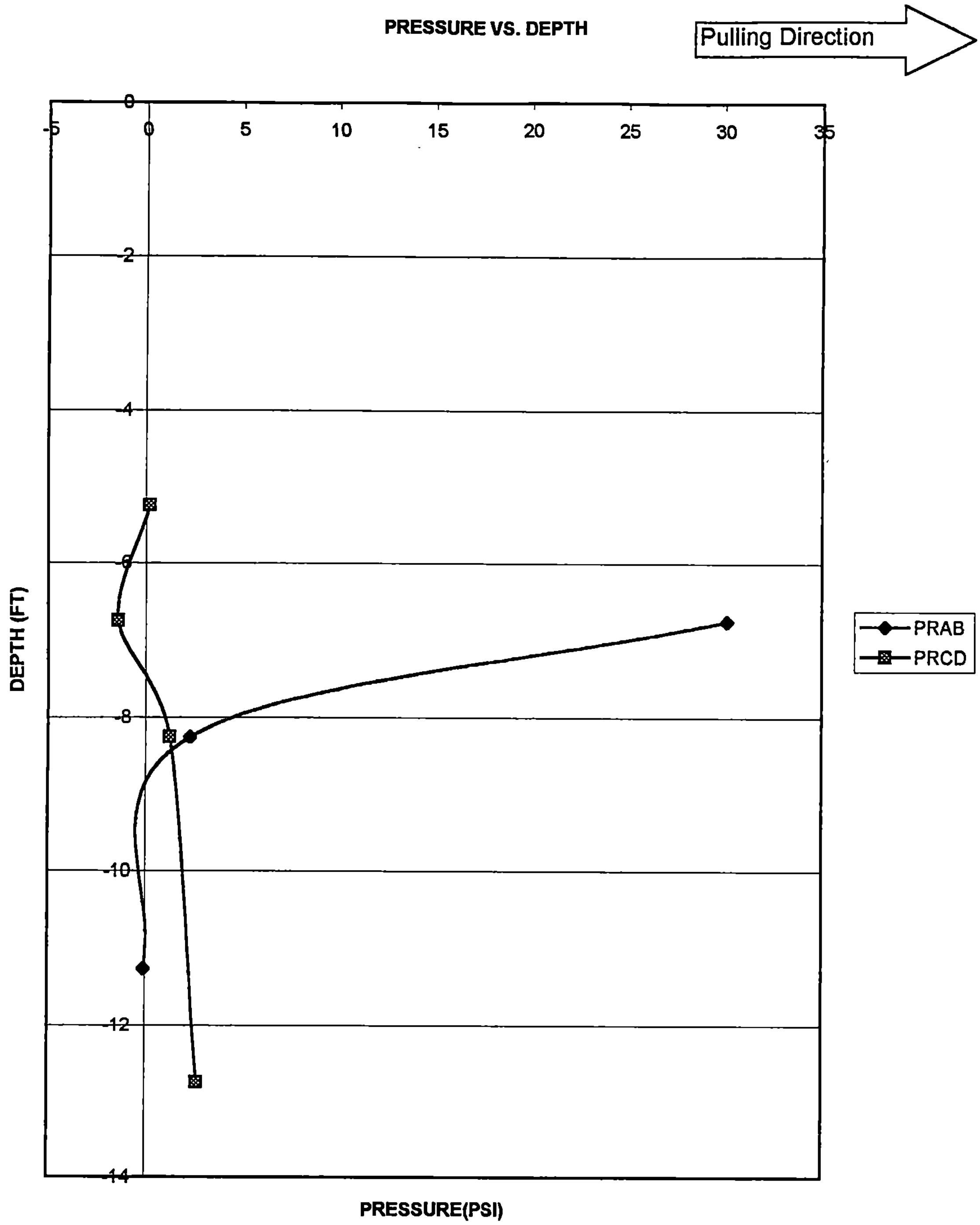

Figure A-30. Pressure vs Depth for pile # 3, pulling west. Second 1" test, Oct,30,1999.
Deflection of the pile=.764"

AB Pulling Side
Def.=.893"

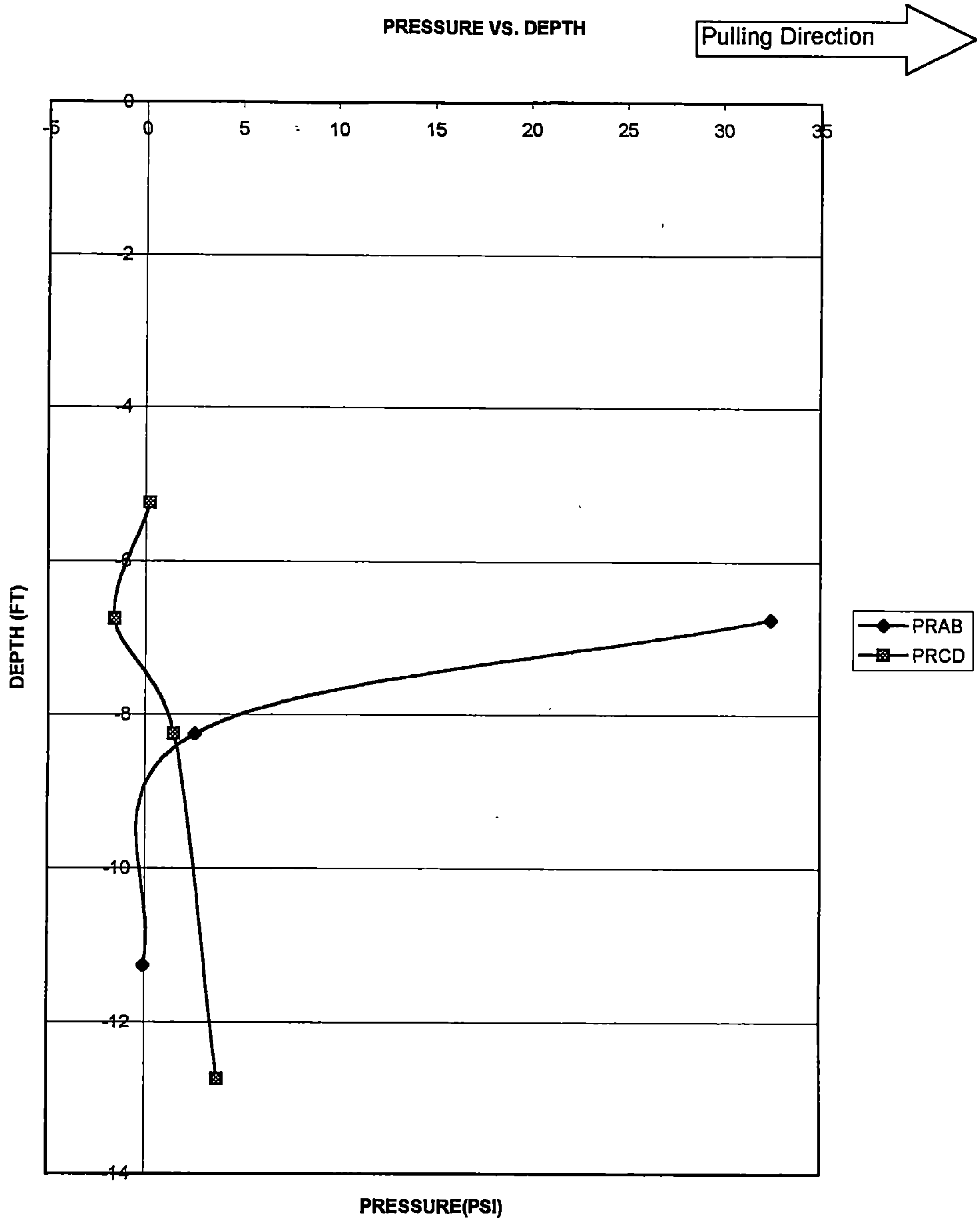

Figure A-31. Pressure vs Depth for pile # 3, pulling west. Second 1" test, Oct,30,1999.
Deflection of the pile=.893"

AB Pulling Side
Def.=1.009"

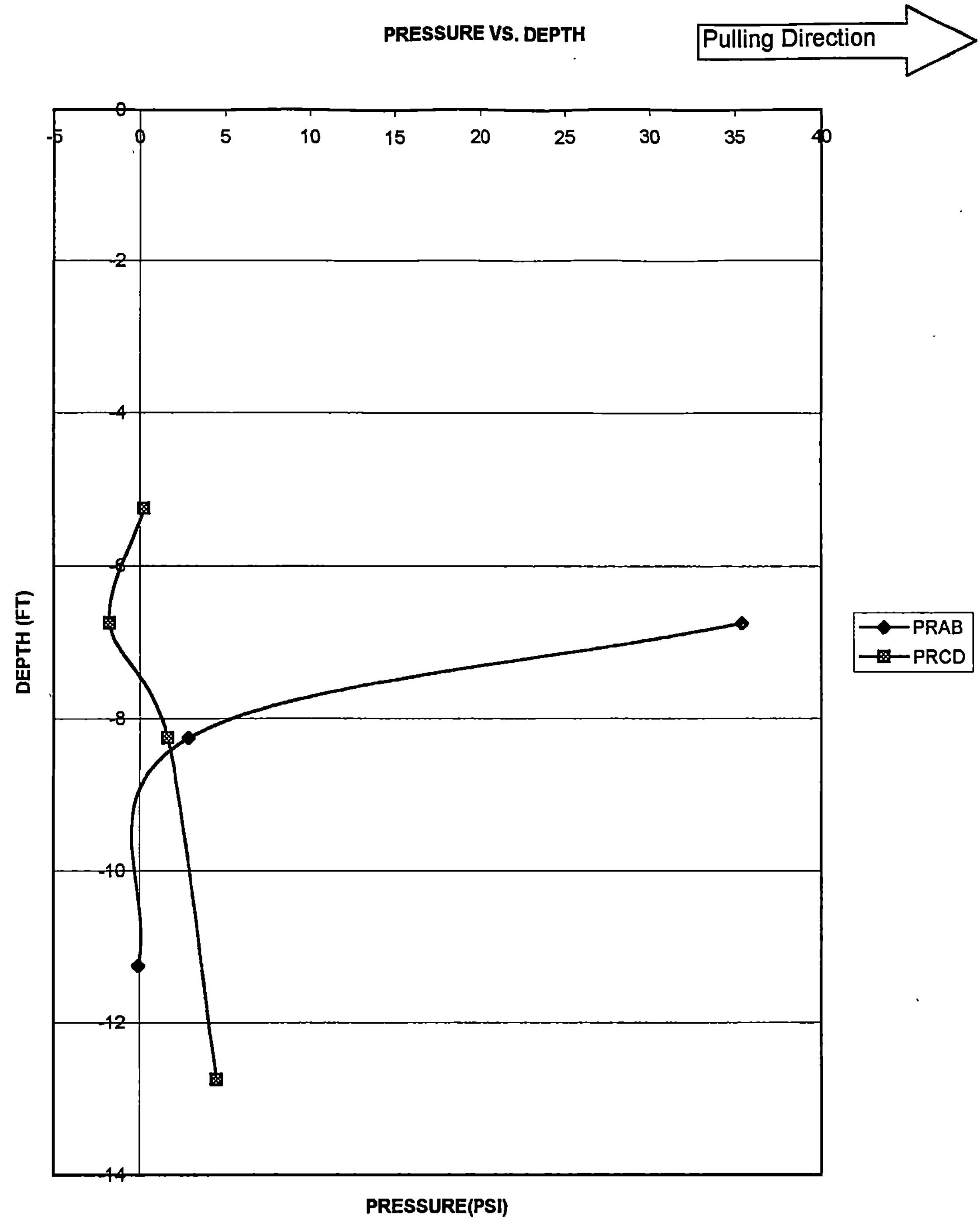

Figure A-32. Pressure vs Depth for pile # 3, pulling west. Second 1" test, Oct,30,1999.
Deflection of the pile=1.009"

AB Pulling Side
Def.=PRAB

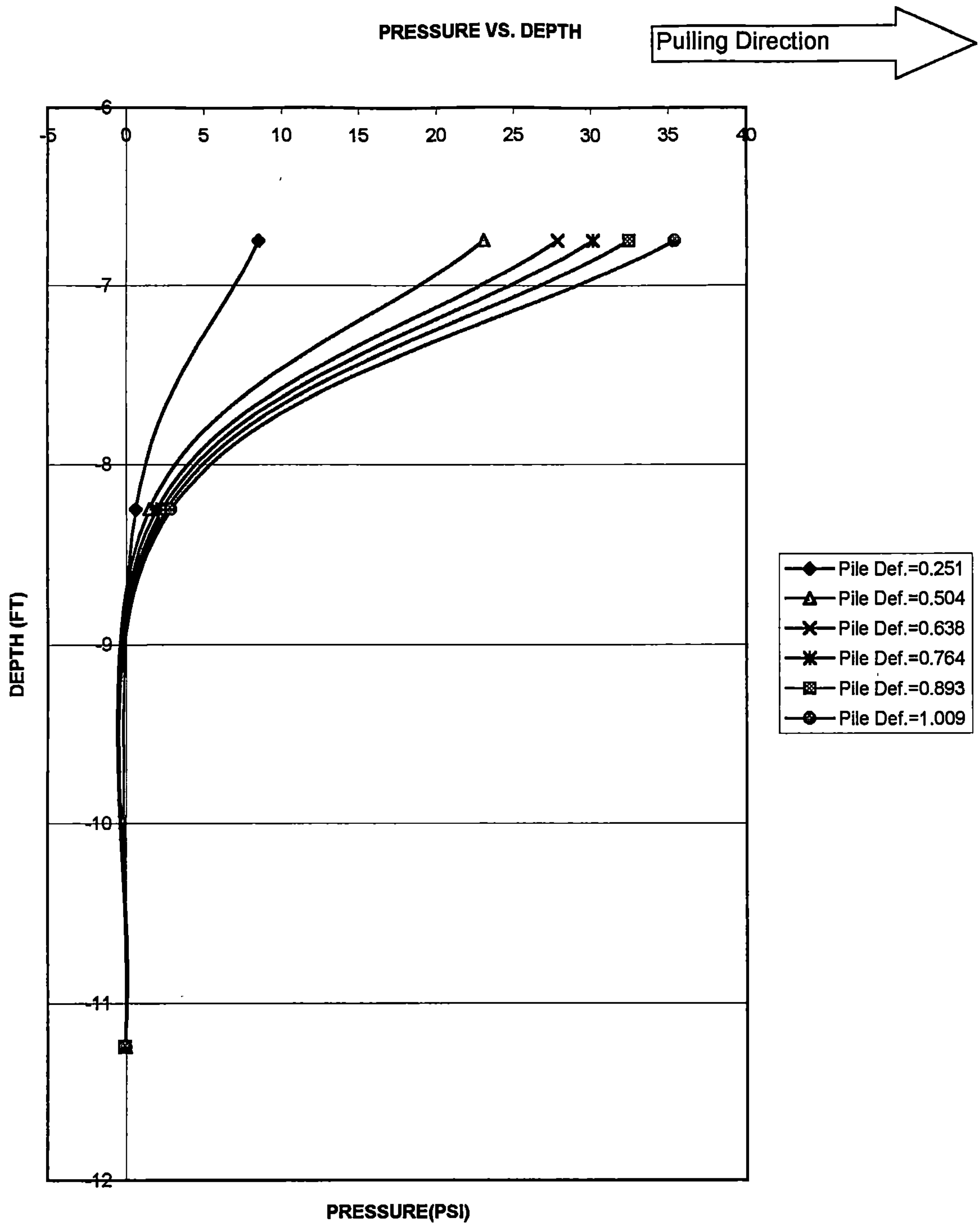

Figure A-33. Pressure vs Depth for pile # 3, pulling west. Second 1" test, Oct,30,1999.
AB Side

AB Pulling Side
Def.=PRCD

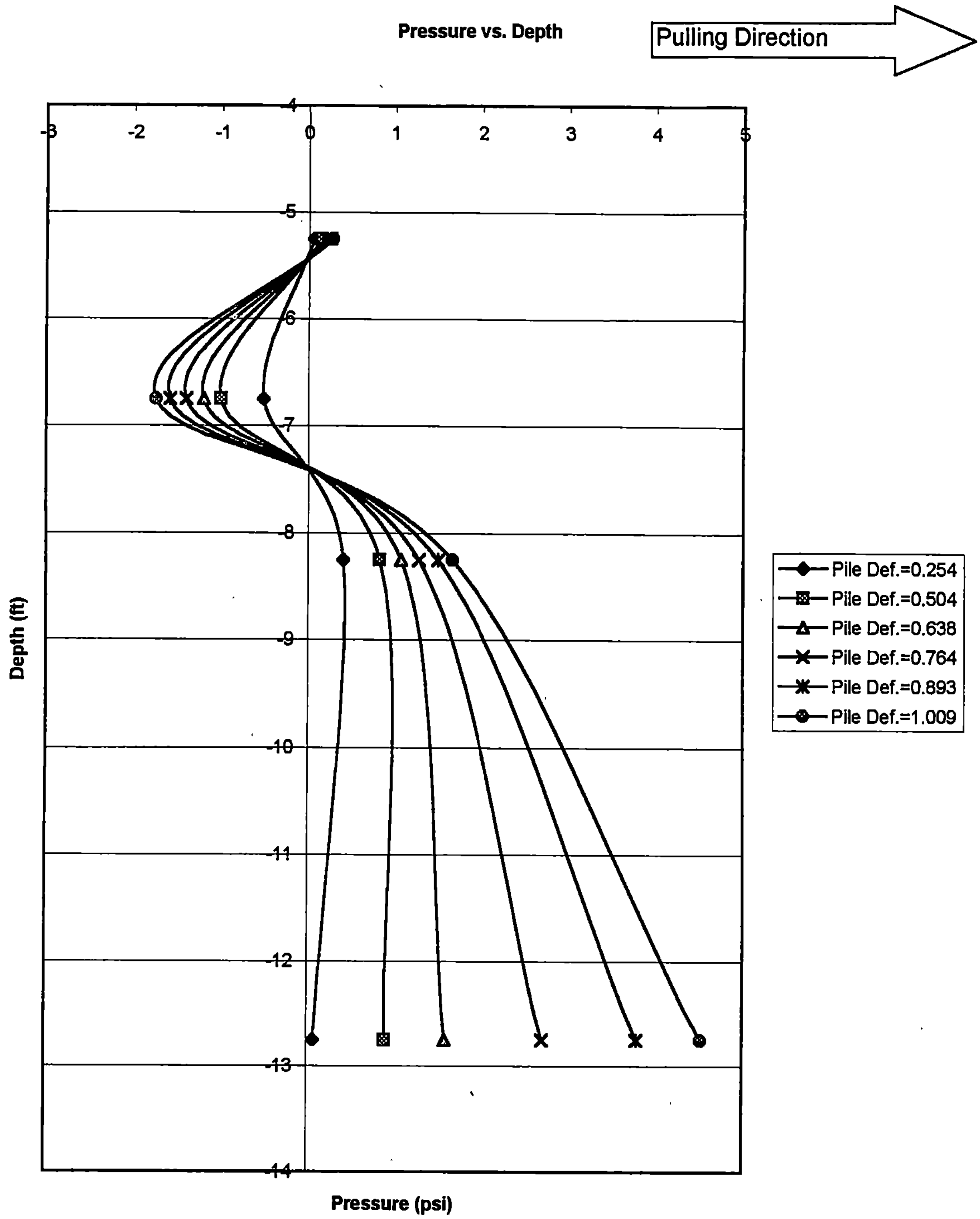

Figure A-34. Pressure vs Depth for pile # 3, pulling west. Second 1" test, Oct,30,1999.
CD Side

AB Pulling Side
Def.=.253"

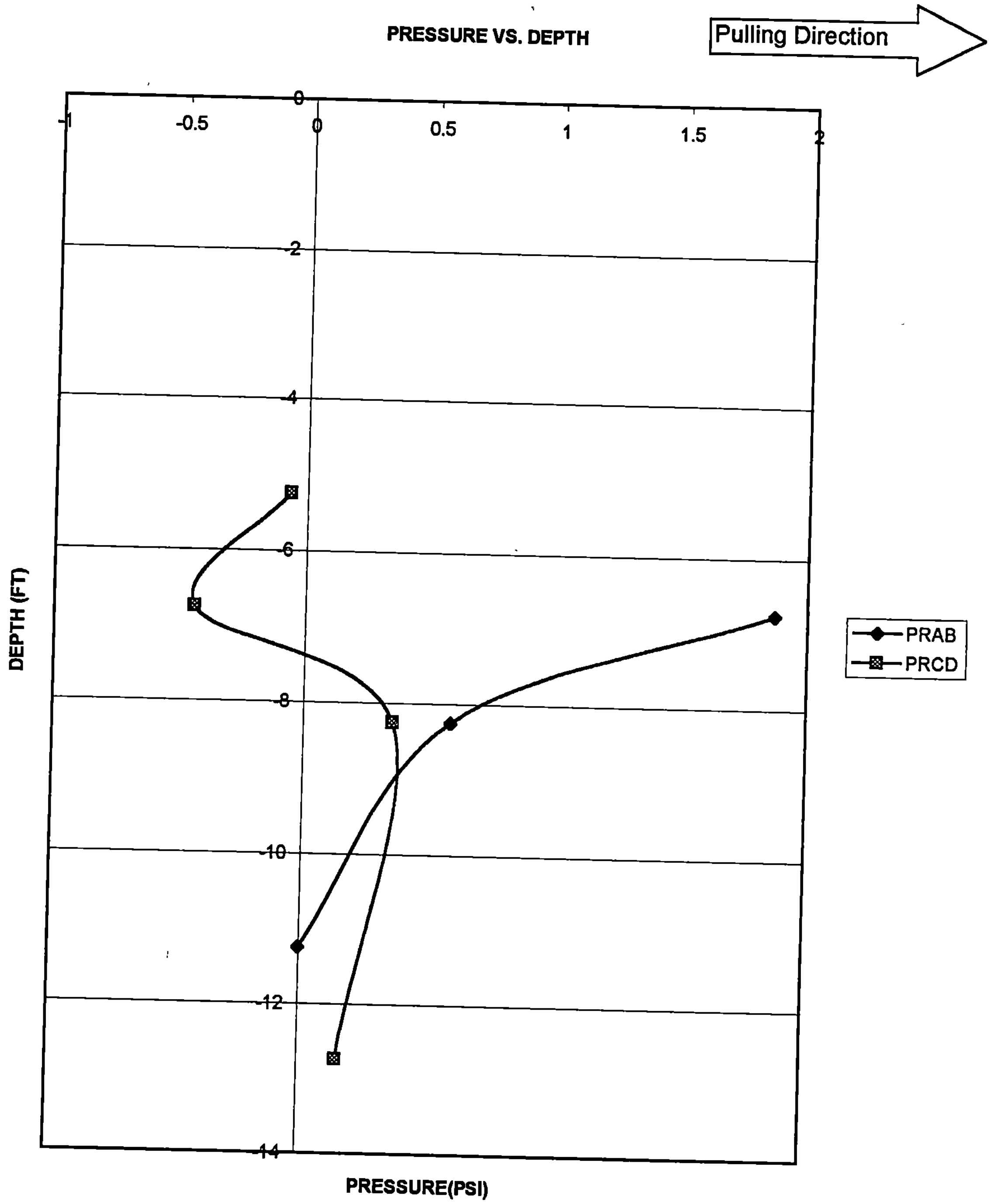

Figure A-35. Pressure vs Depth for pile # 3, pulling west. Third 1" test, Nov,6,1999.
Deflection of the pile=.253"

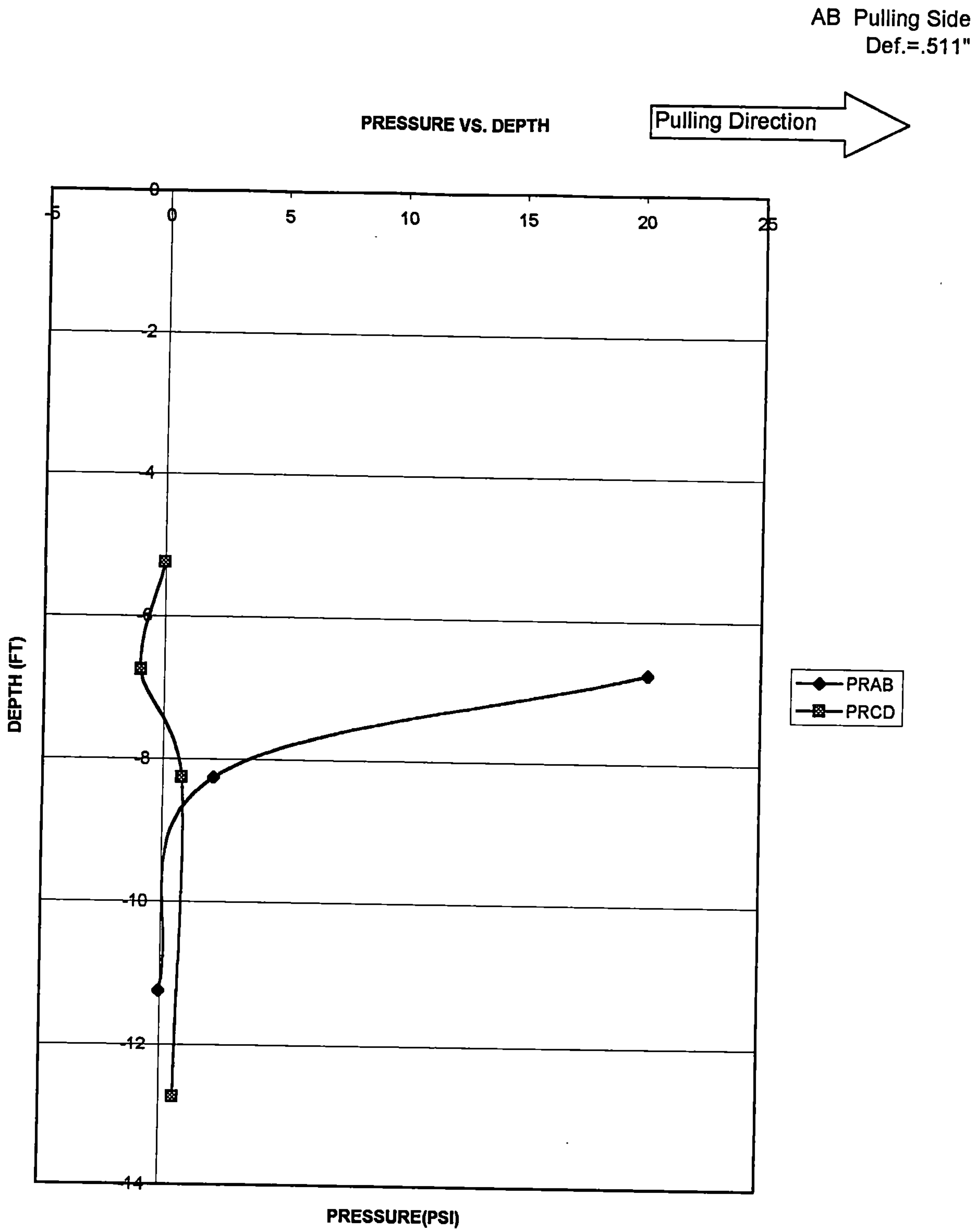

Figure A-36. Pressure vs Depth for pile # 3, pulling west. Third 1" test, Nov,6,1999.
Deflection of the pile=.511"

AB Pulling Side
Def.=.636"

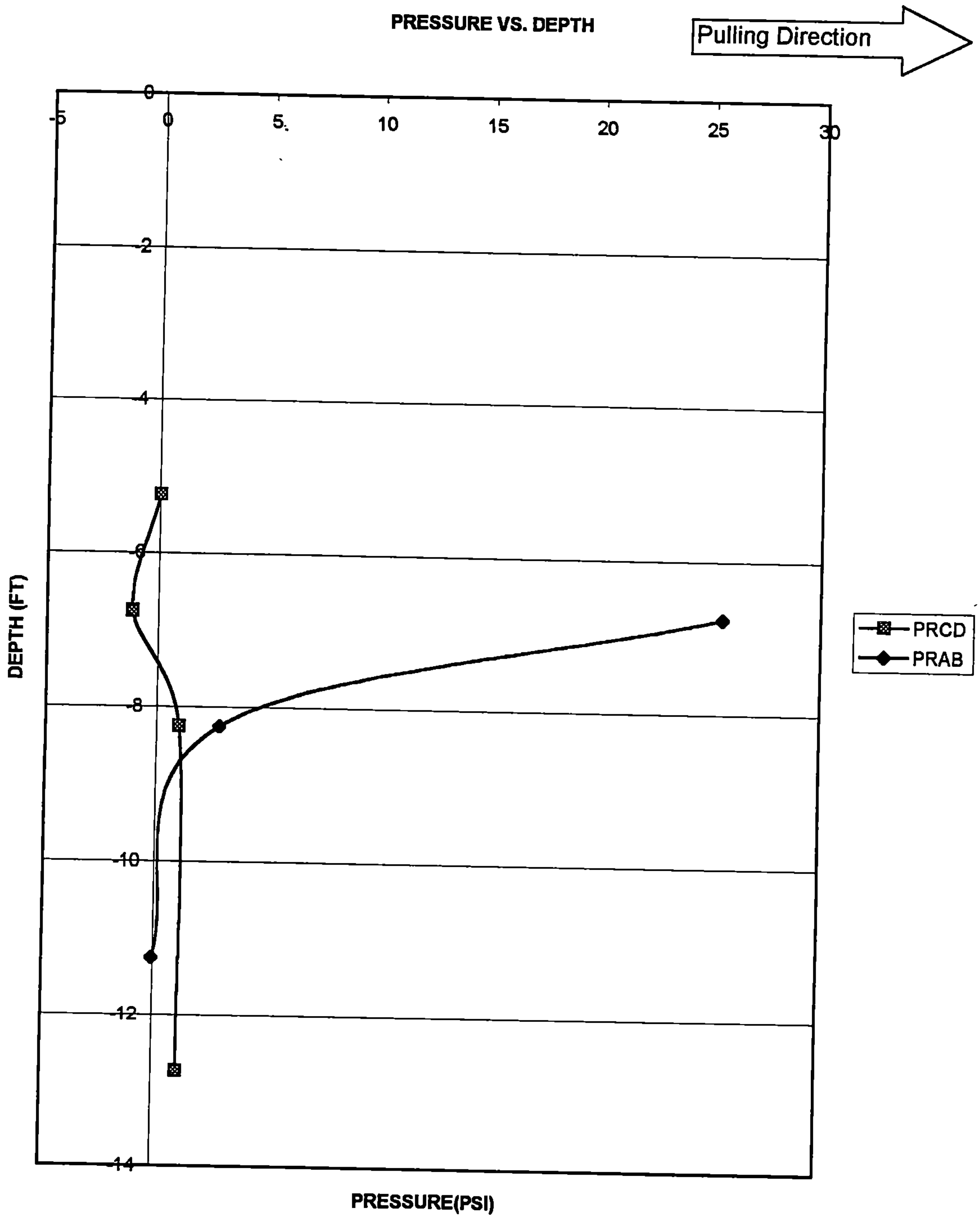

Figure A-37. Pressure vs Depth for pile # 3, pulling west. Third 1" test, Nov,6,1999.
Deflection of the pile=.636"

AB Pulling Side
Def.=.756"

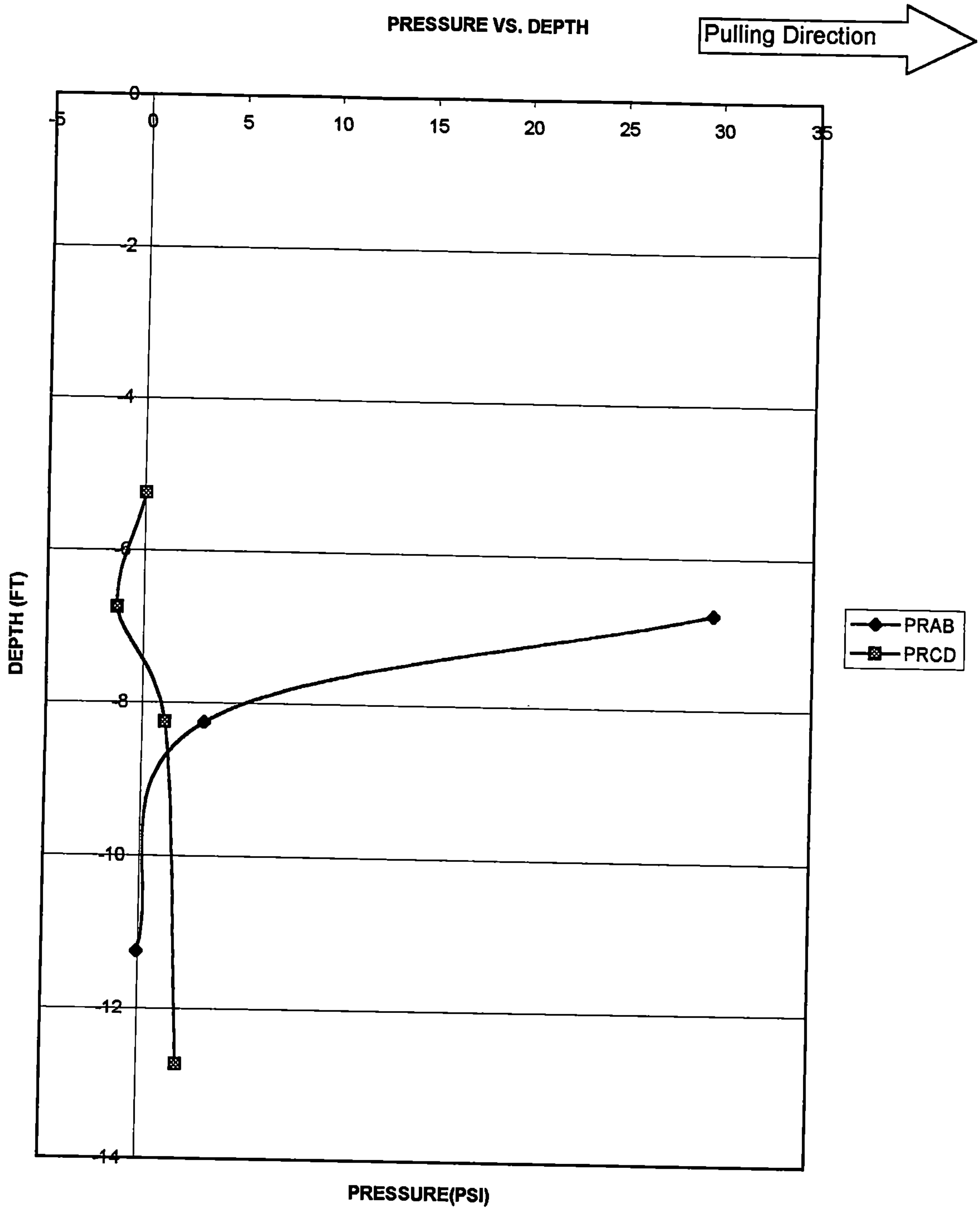

Figure A-38. Pressure vs Depth for pile # 3, pulling west. Third 1" test, Nov,6,1999.
Deflection of the pile=.756"

AB Pulling Side
Def.=.882"

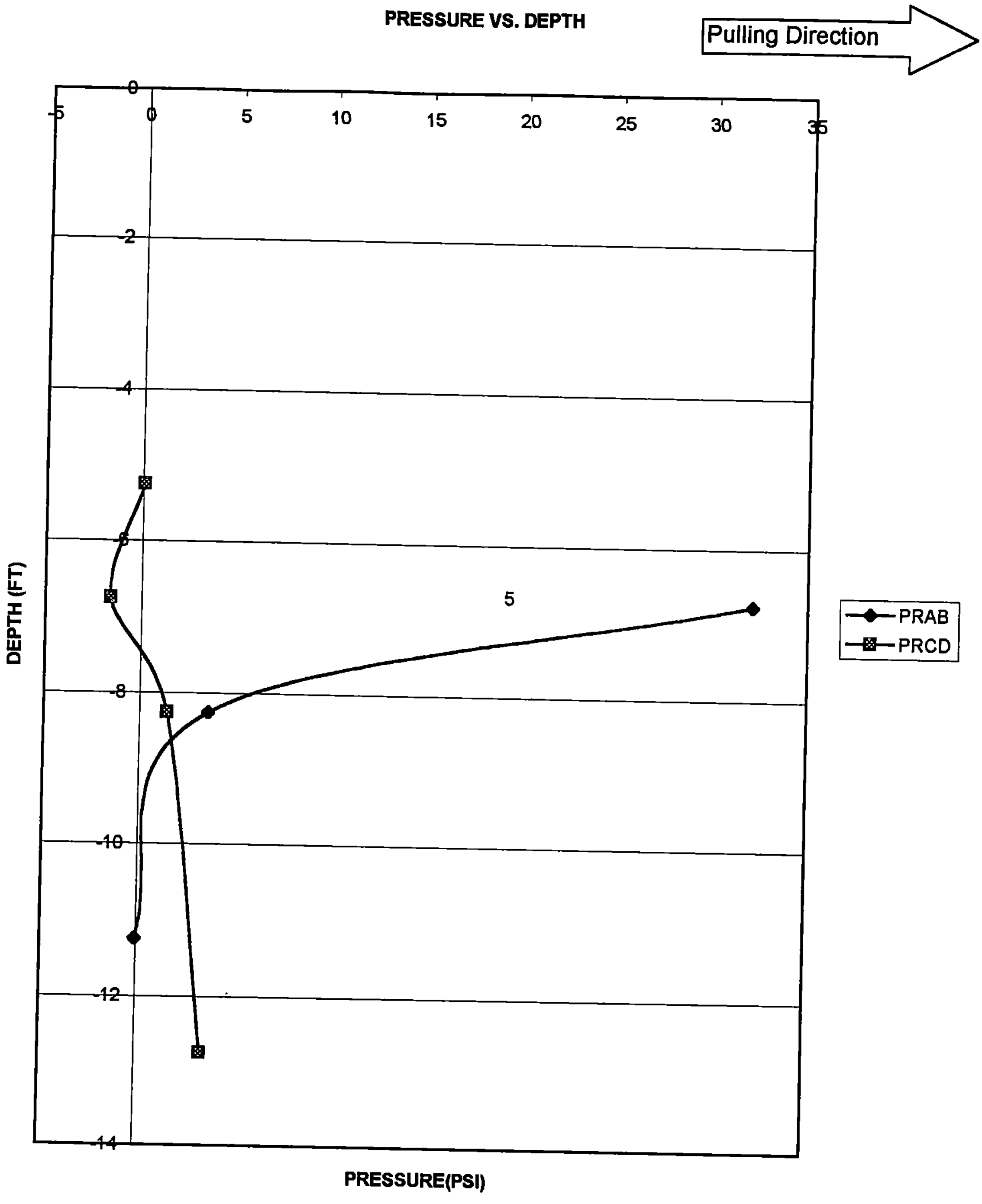

Figure A-39. Pressure vs Depth for pile # 3, pulling west. Third 1" test, Nov,6,1999. Deflection of the pile=.882"

AB Pulling Side
Def.=1.013"

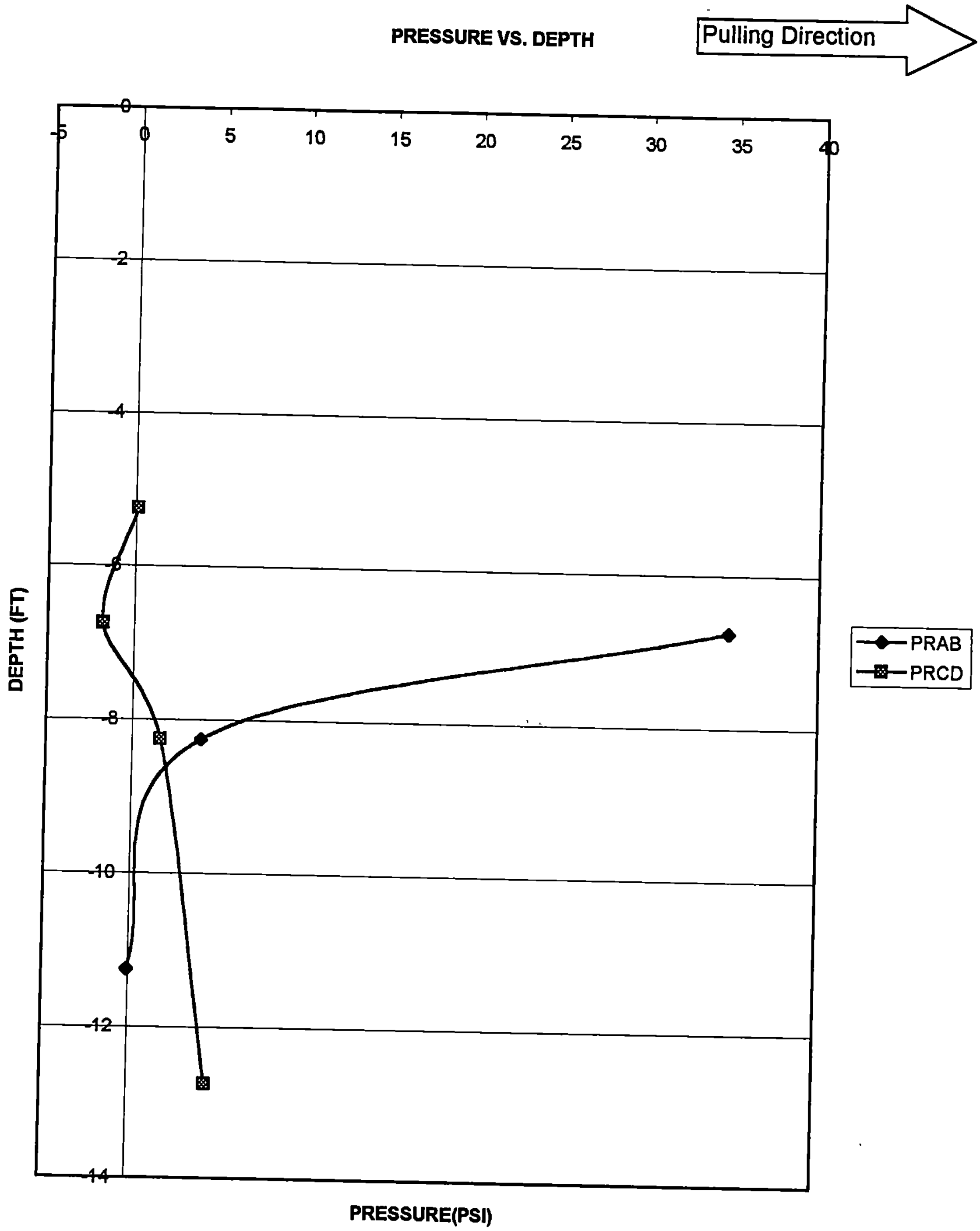

Figure A-40. Pressure vs Depth for pile # 3, pulling west. Third 1" test, Nov,6,1999.
Deflection of the pile=1.013"

AB Pulling Side
Def.=PRAB

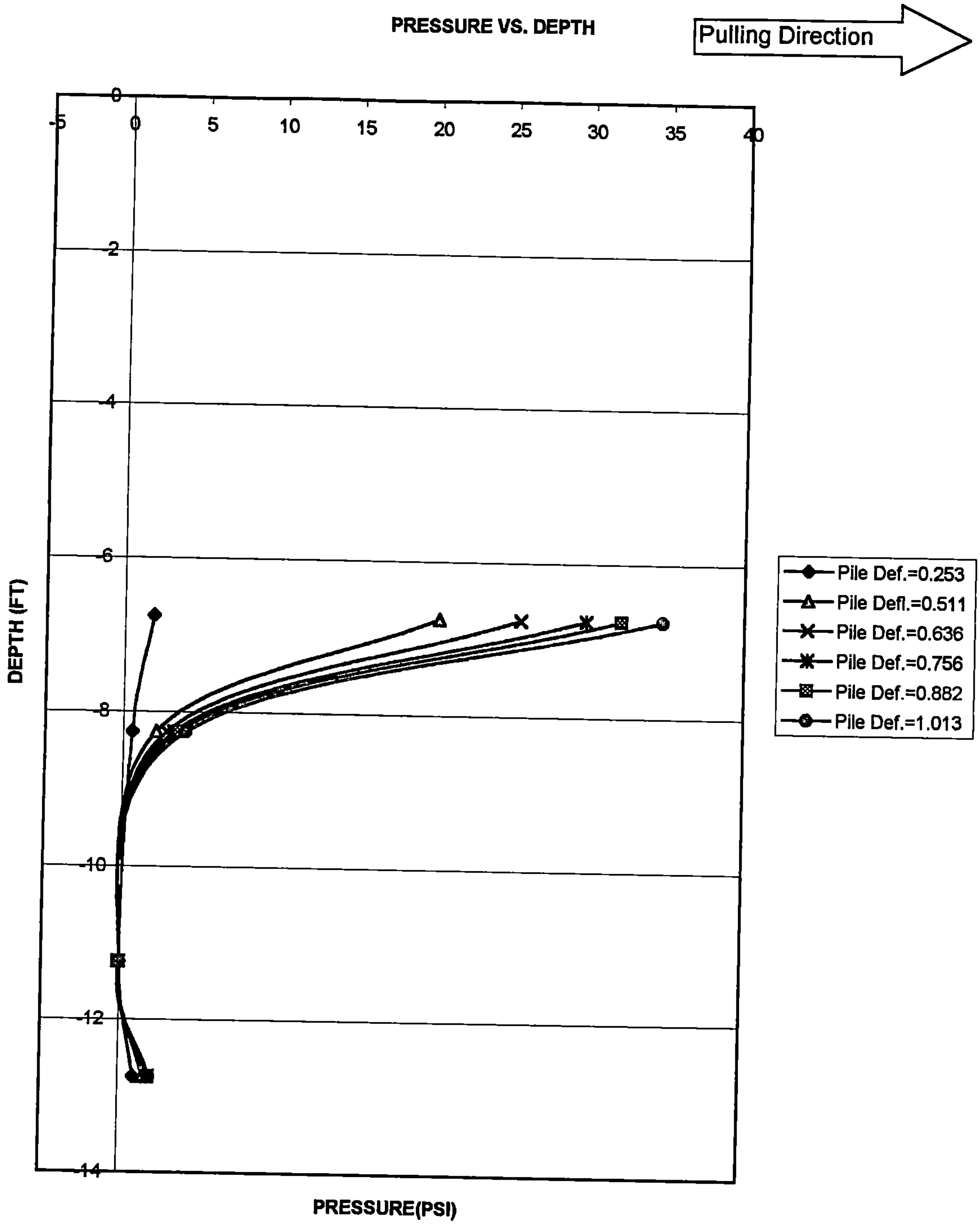

Figure A-41. Pressure vs Depth for pile # 3, pulling west. Third 1" test, Nov,6,1999.
AB Side

AB Pulling Side
Def.=PRCD

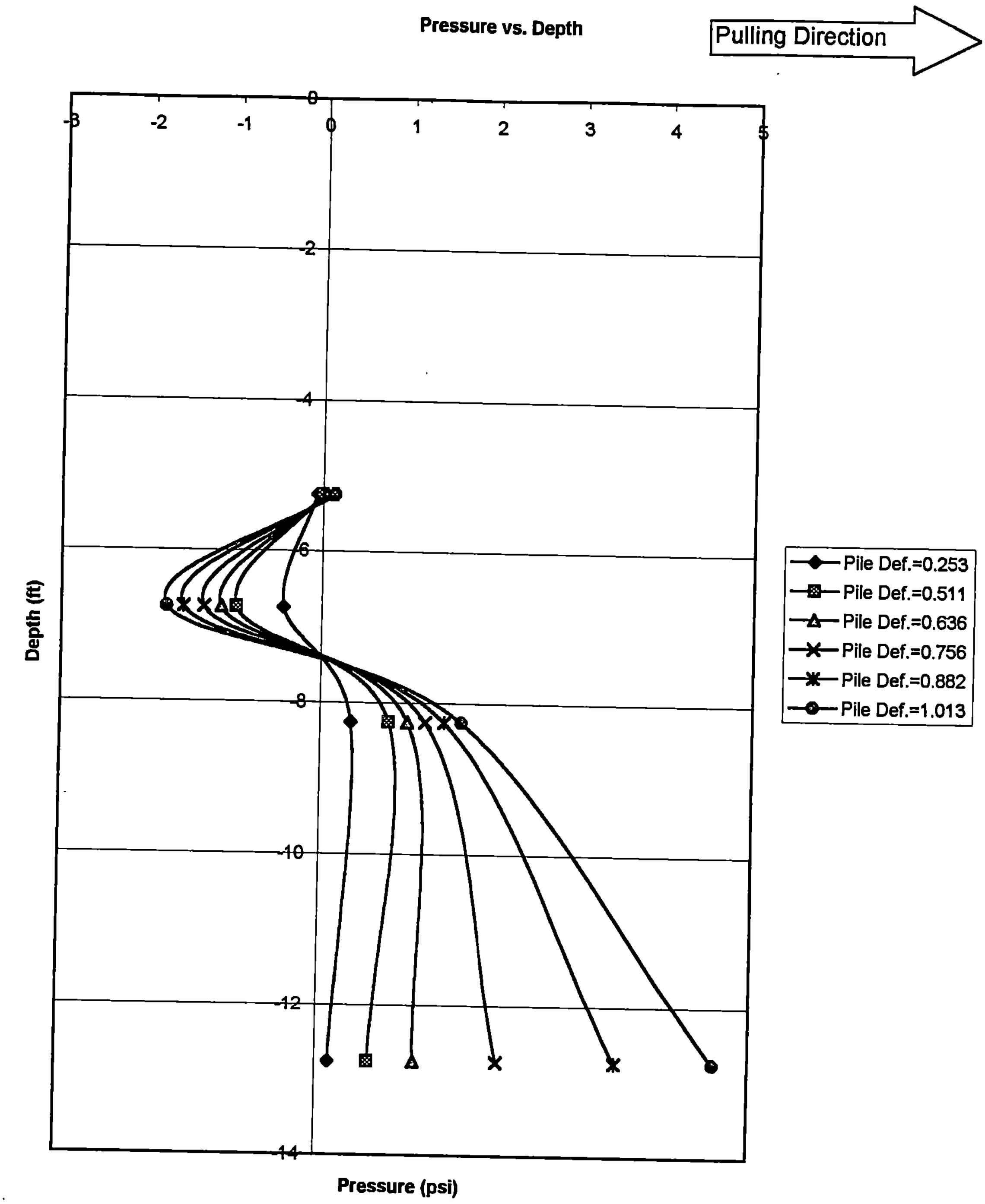

Figure A-42. Pressure vs Depth for pile # 3, pulling west. Third 1" test, Nov,6,1999.
CD Side

CD Pulling Side
Def.=.129"

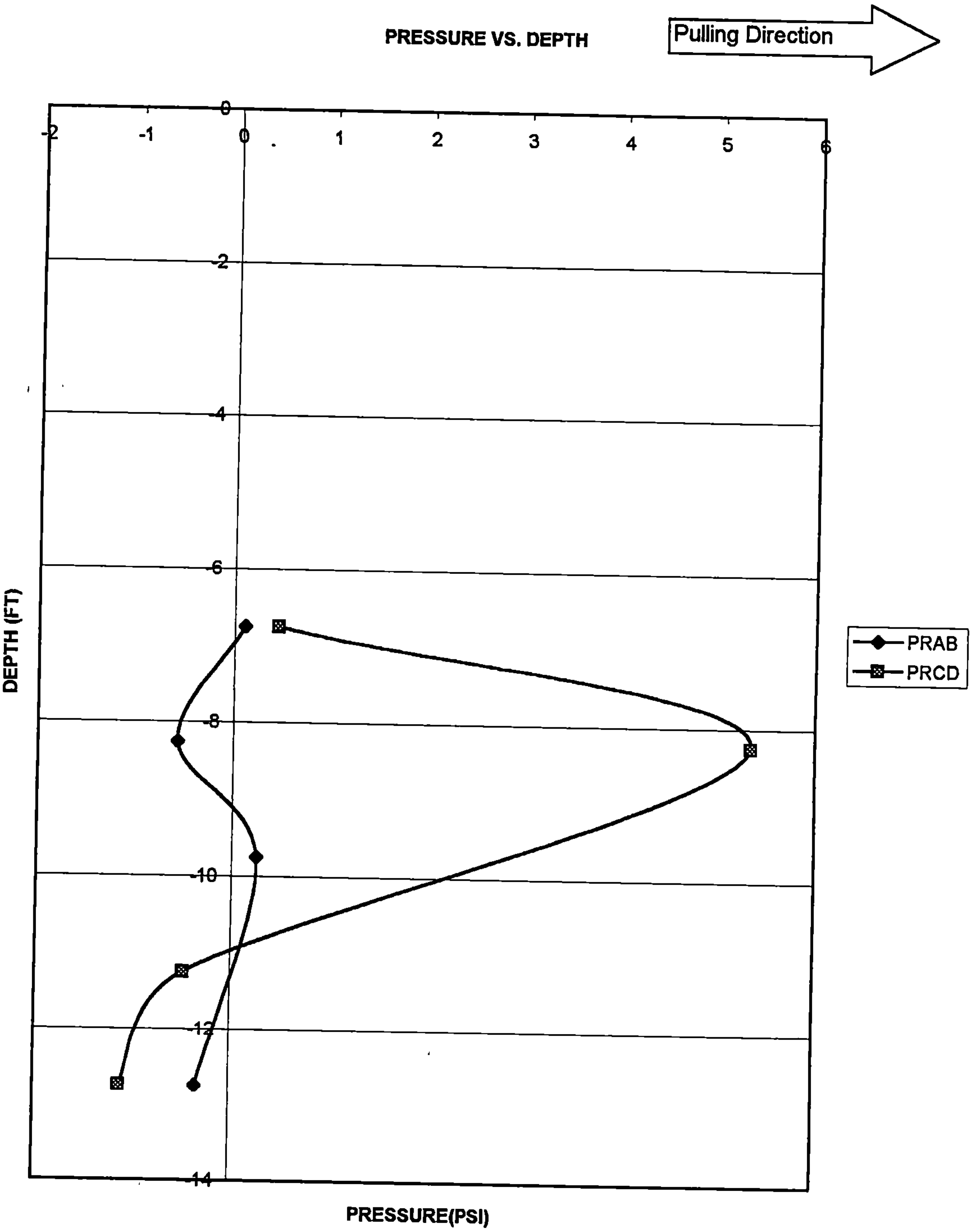

Figure A-43. Pressure vs Depth for pile # 3, pulling east. Second 1/2" test, Feb,5,1999
Deflection of the pile=.129"

CD Pulling Side
Def.=.247"

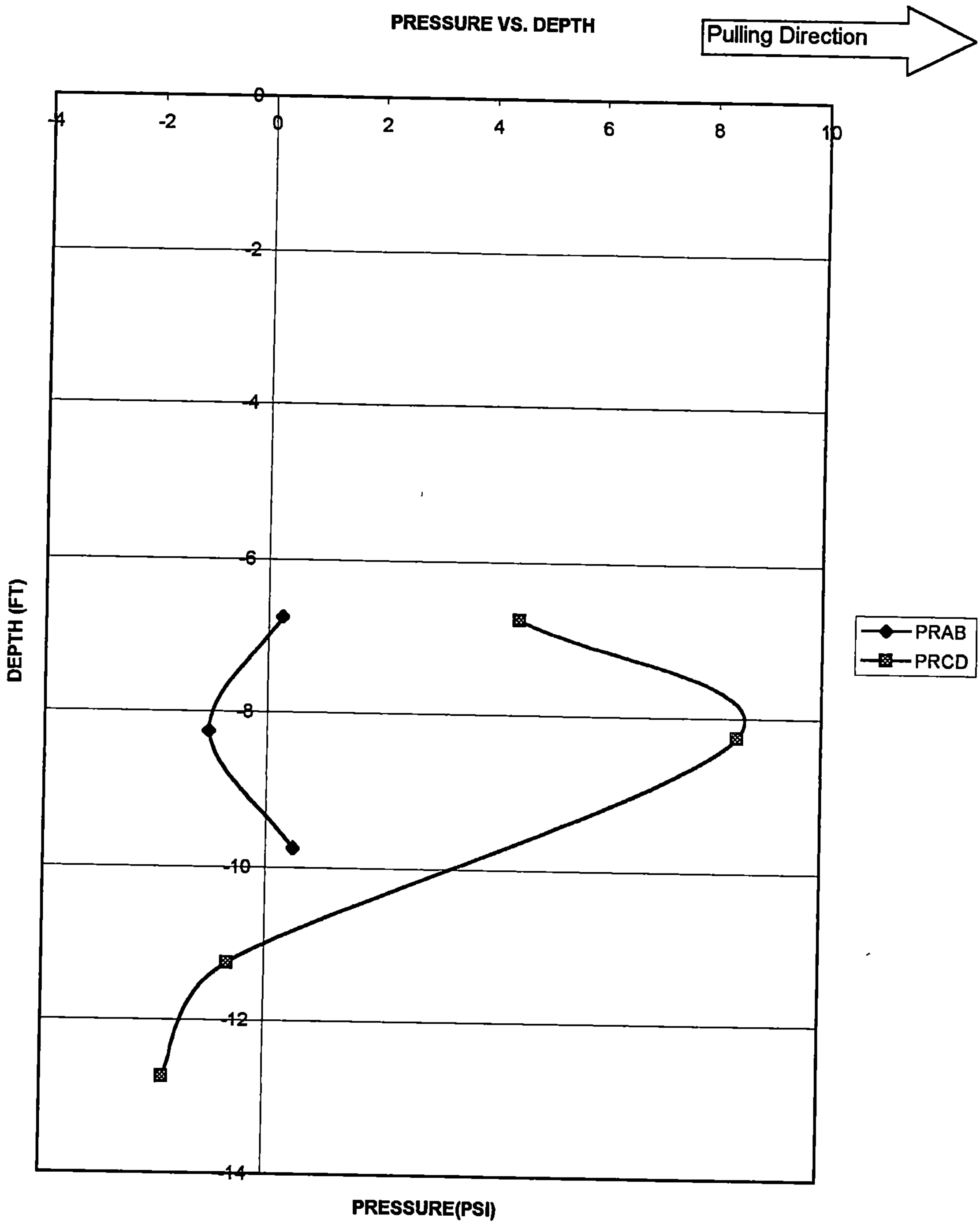

Figure A-44. Pressure vs Depth for pile # 3, pulling east. Second 1/2" test, Feb,5,1999
Deflection of the pile=.247"

CD Pulling Side
Def.=.380"

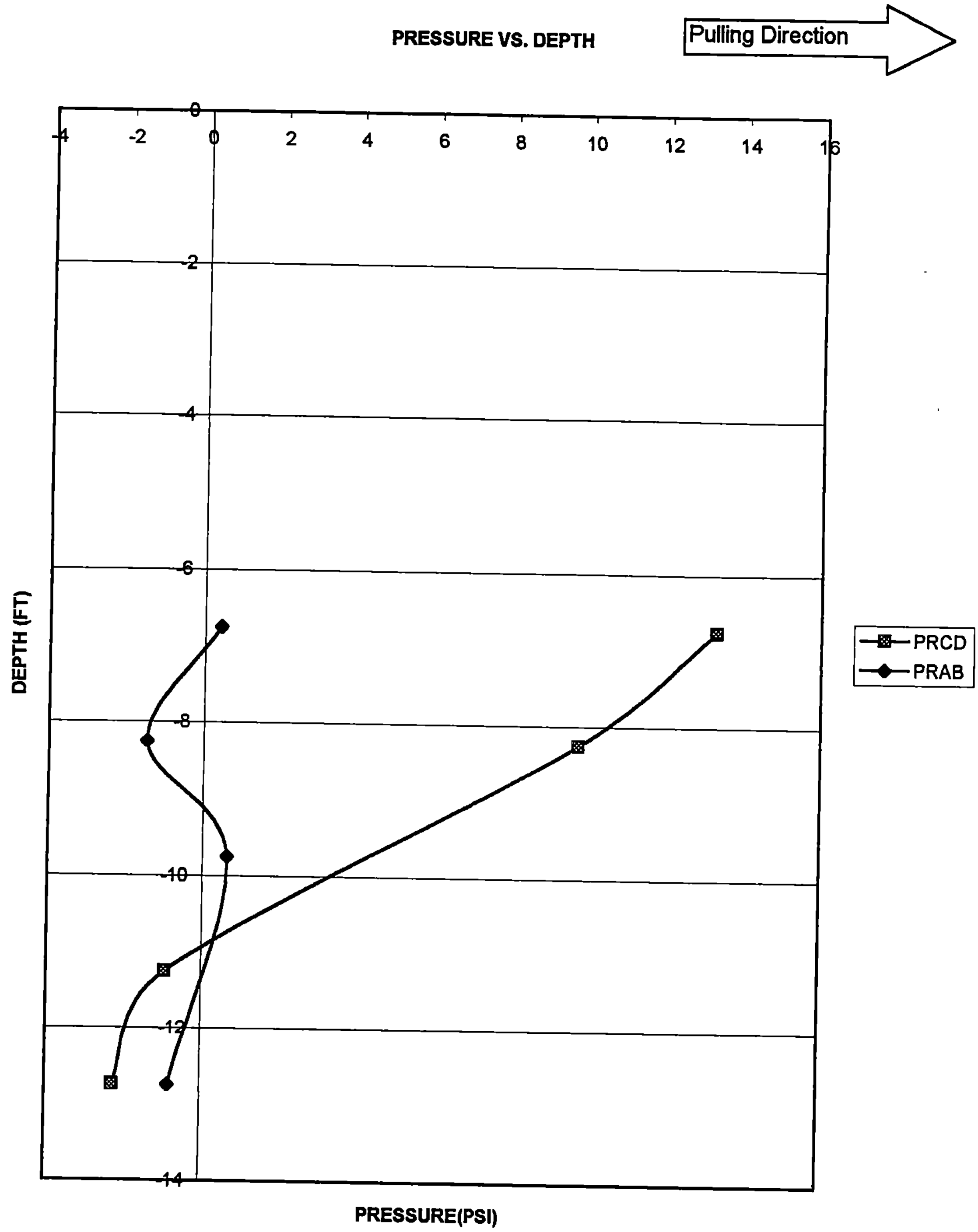

Figure A-45. Pressure vs Depth for pile # 3, pulling east. Second 1/2" test, Feb,5,1999
Deflection of the pile=.380"

CD Pulling Side
Def.=.501"

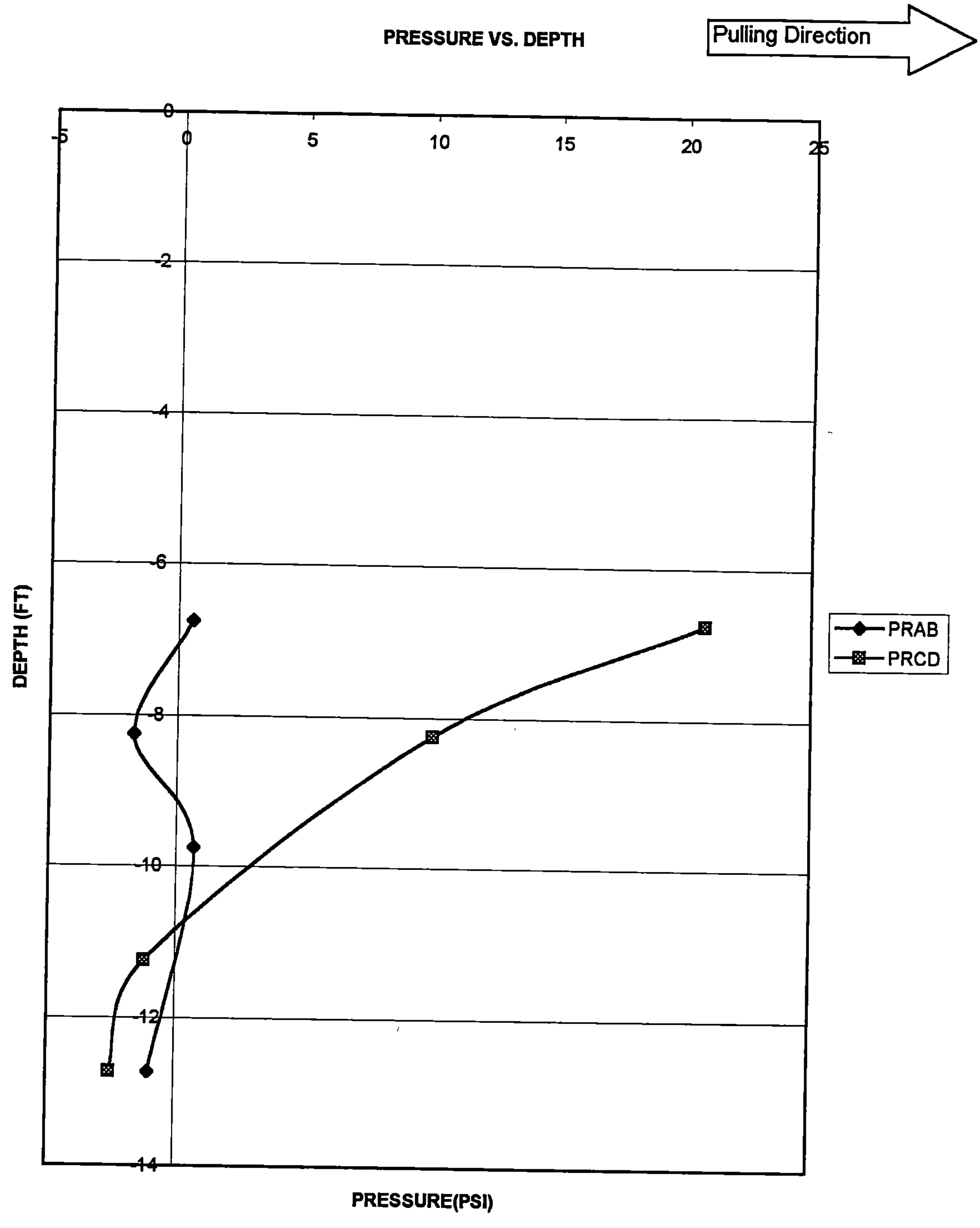

Figure A-46. Pressure vs Depth for pile # 3, pulling east. Second 1/2" test, Feb,5,1999
Deflection of the pile=.501"

CD Pulling Side
Def.=PRCD

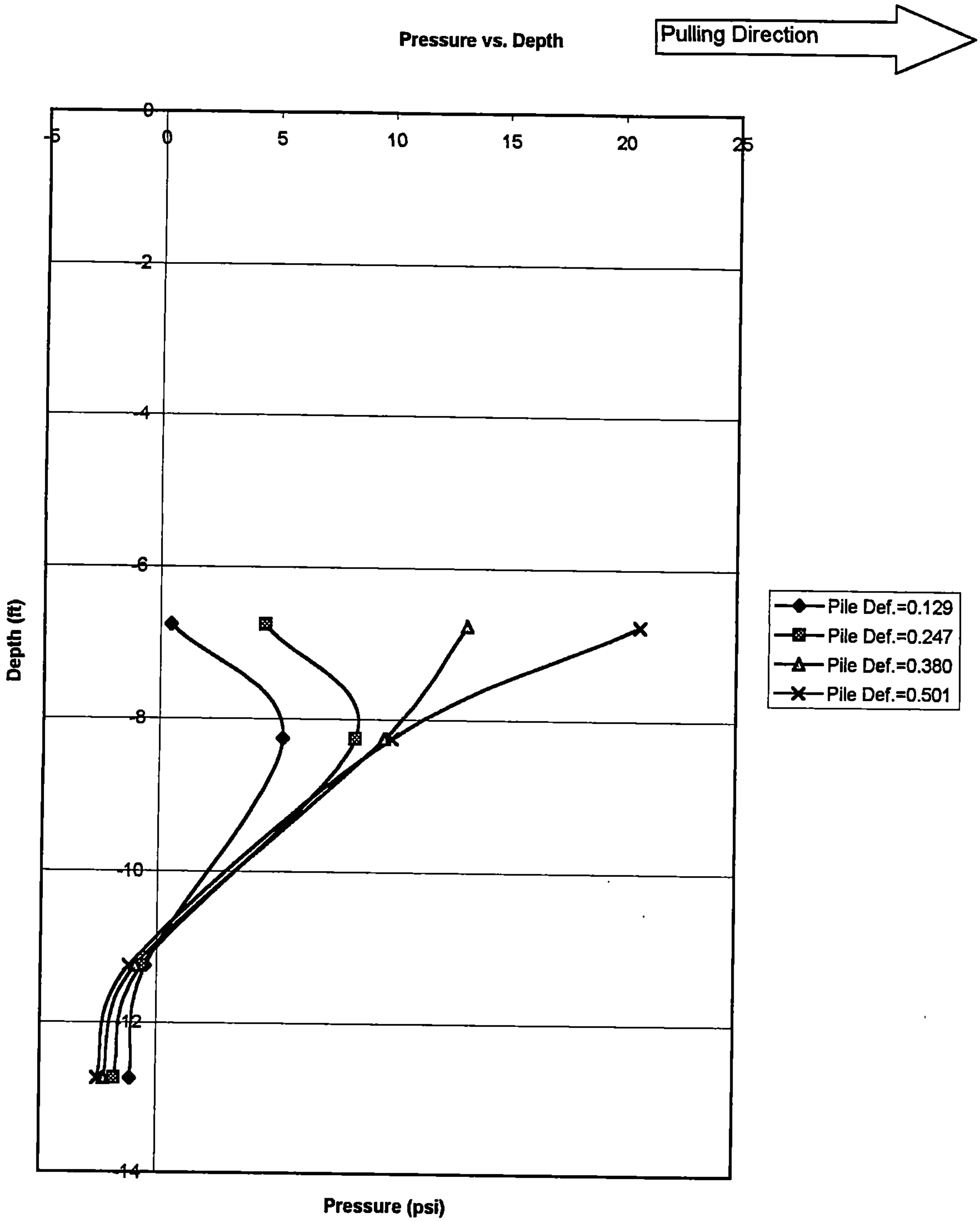

Figure A-47. Pressure vs Depth for pile # 3, pulling east. Second 1/2" test, Feb,5,1999
CD Side

CD Pulling Side
Def.=PRAB

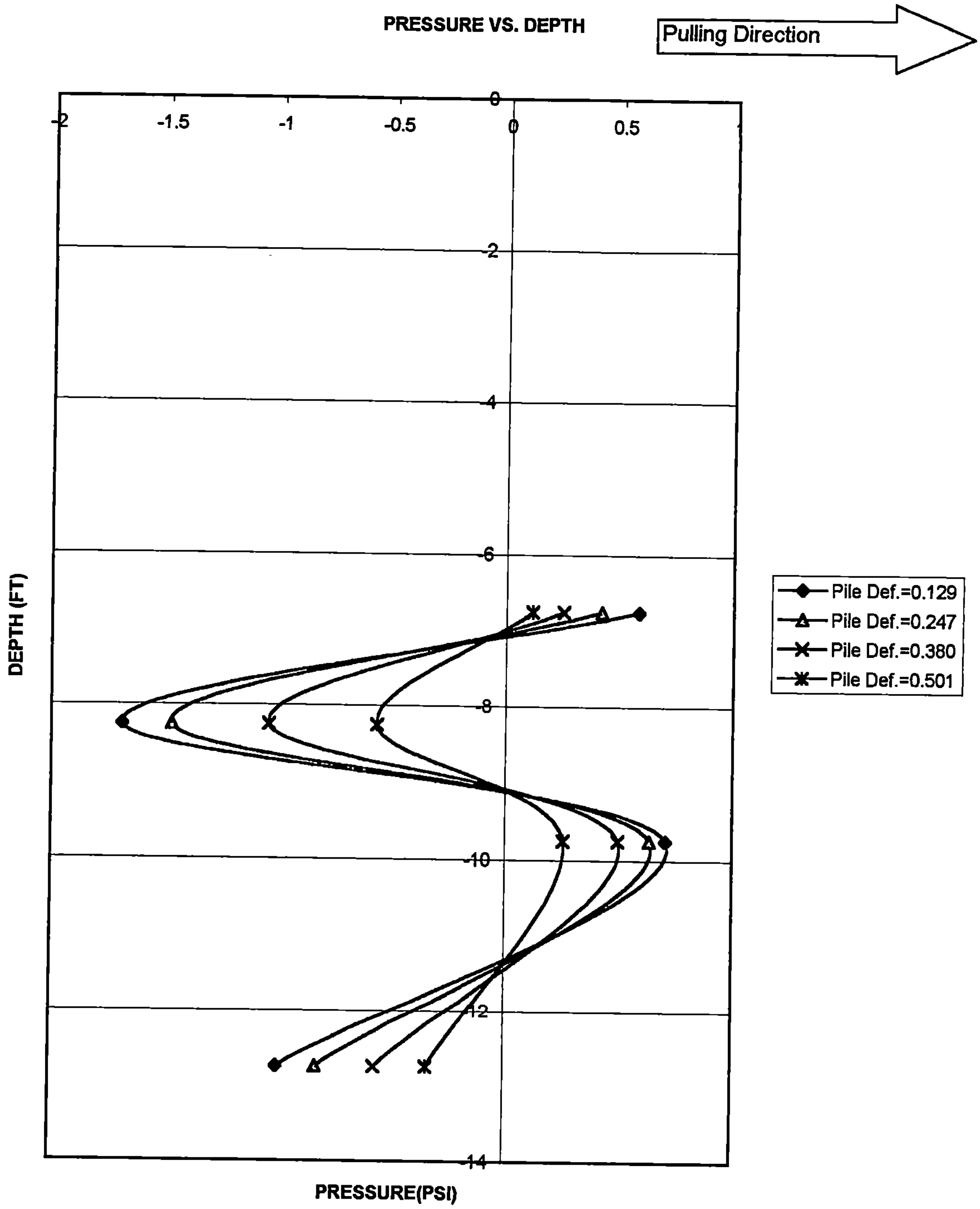

Figure A-48. Pressure vs Depth for pile # 3, pulling east. Second 1/2" test, Feb,5,1999
AB Side

CD Pulling Side
Def.=.255"

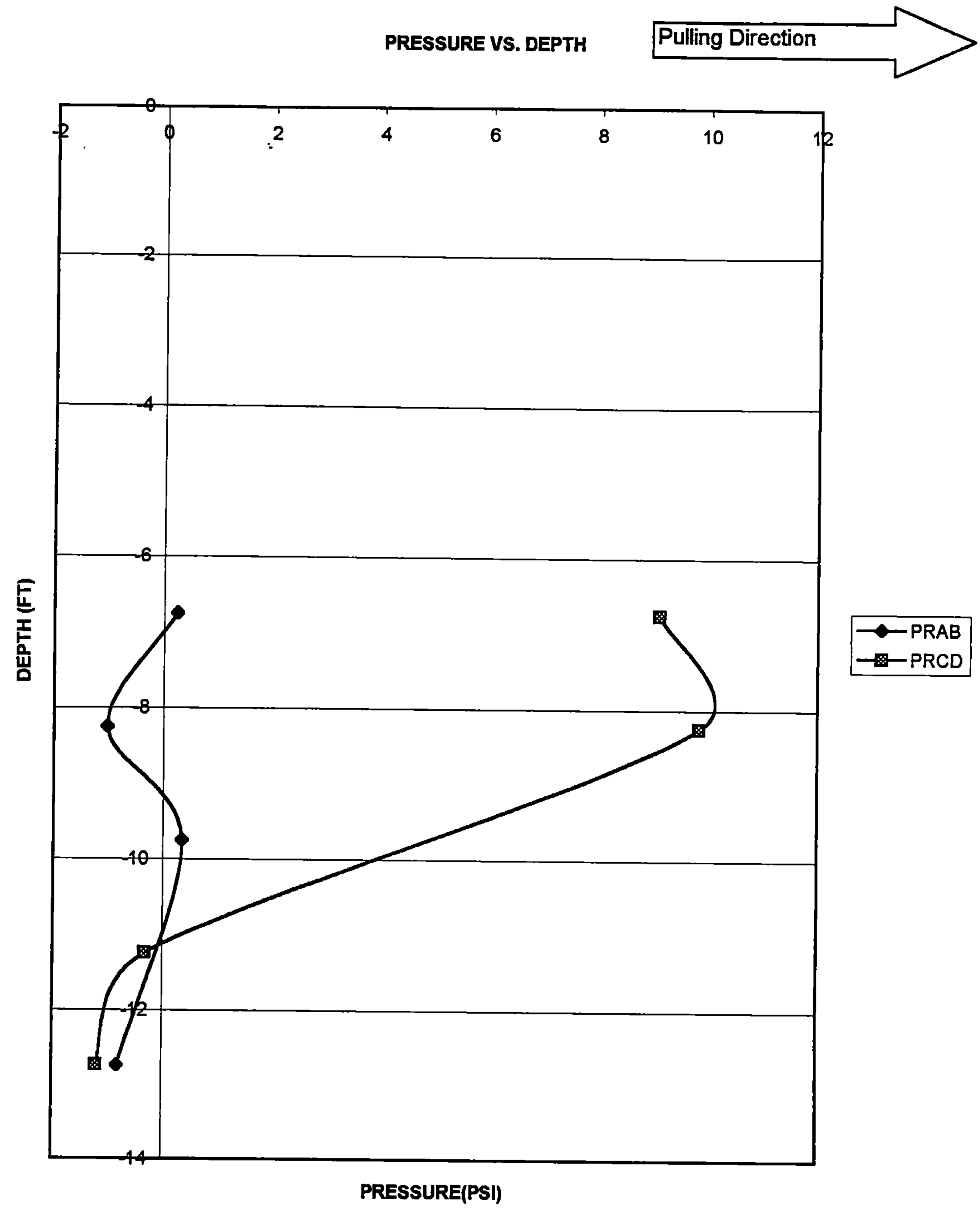

Figure A-49. Pressure vs Depth for pile # 3, pulling east. First 1" test, Feb,26,1999.
Deflection of the pile=.255"

CD Pulling Side
Def.=.512"

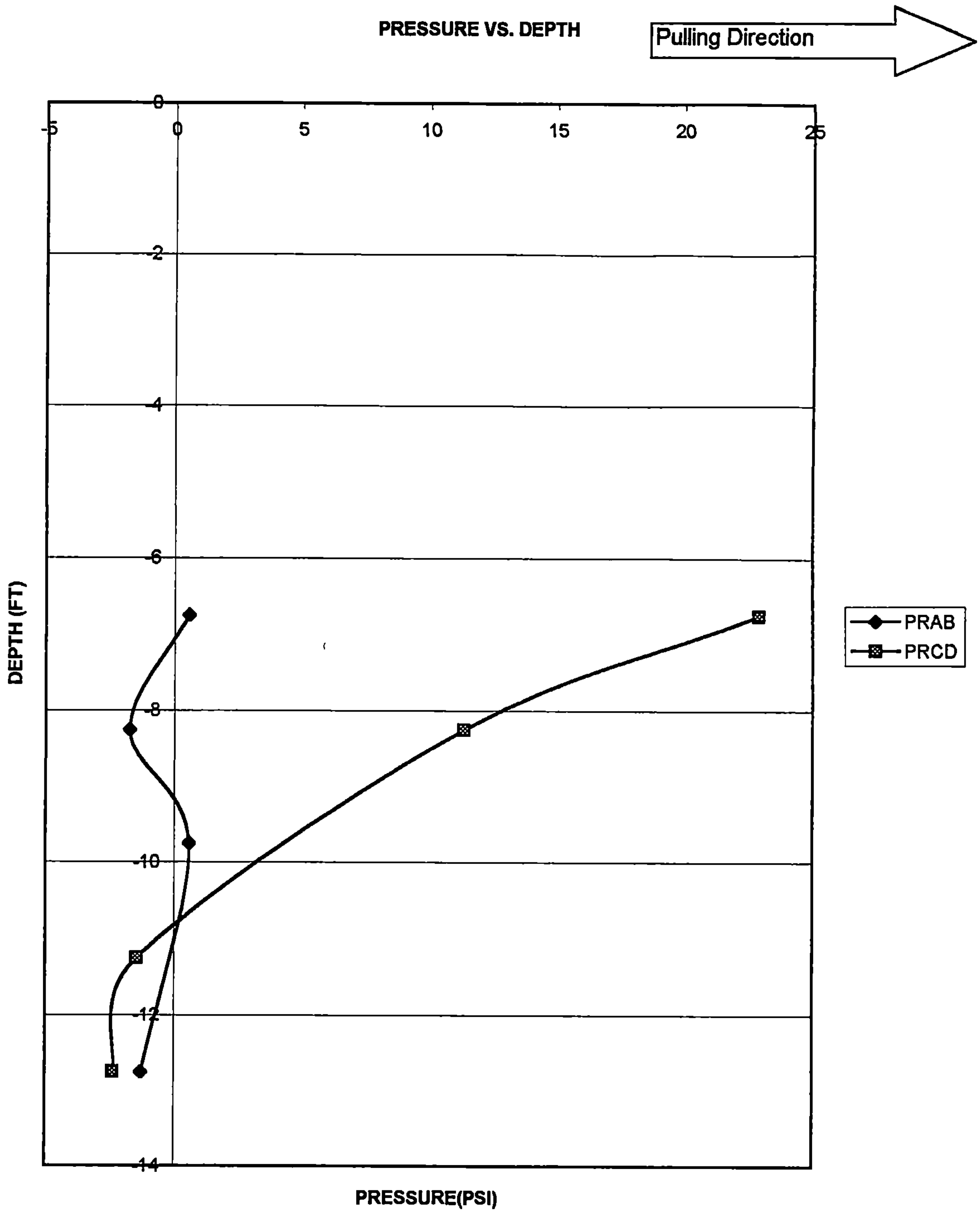

Figure A-50. Pressure vs Depth for pile # 3, pulling east. First 1" test, Feb,26,1999.
Deflection of the pile=.512"

CD Pulling Side
Def.=.652"

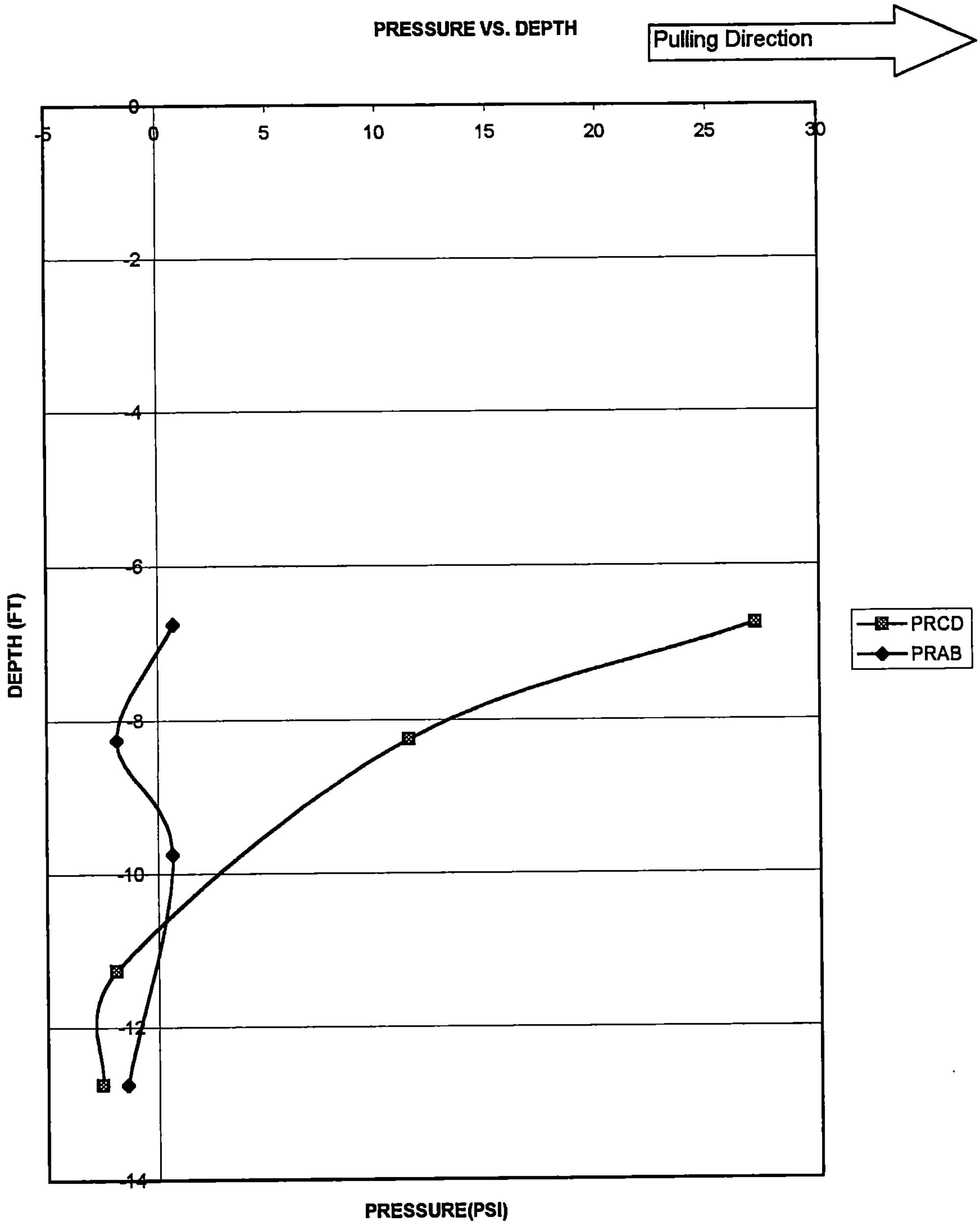

Figure A-51. Pressure vs Depth for pile # 3, pulling east. First 1" test, Feb,26,1999.
Deflection of the pile=.652"

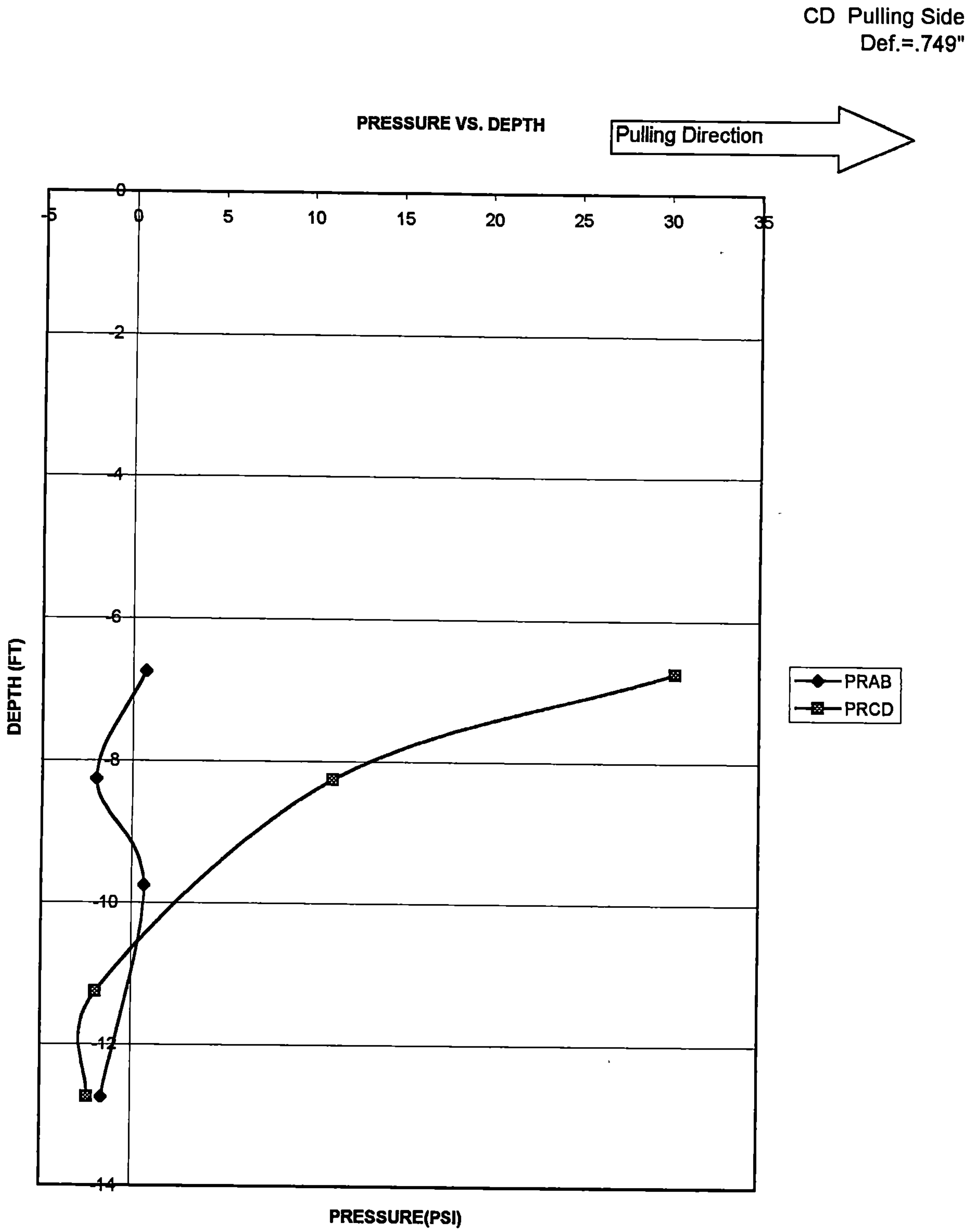

Figure A-52. Pressure vs Depth for pile # 3, pulling east. First 1" test, Feb,26,1999.
Deflection of the pile=.749"

CD Pulling Side
Def.=.912"

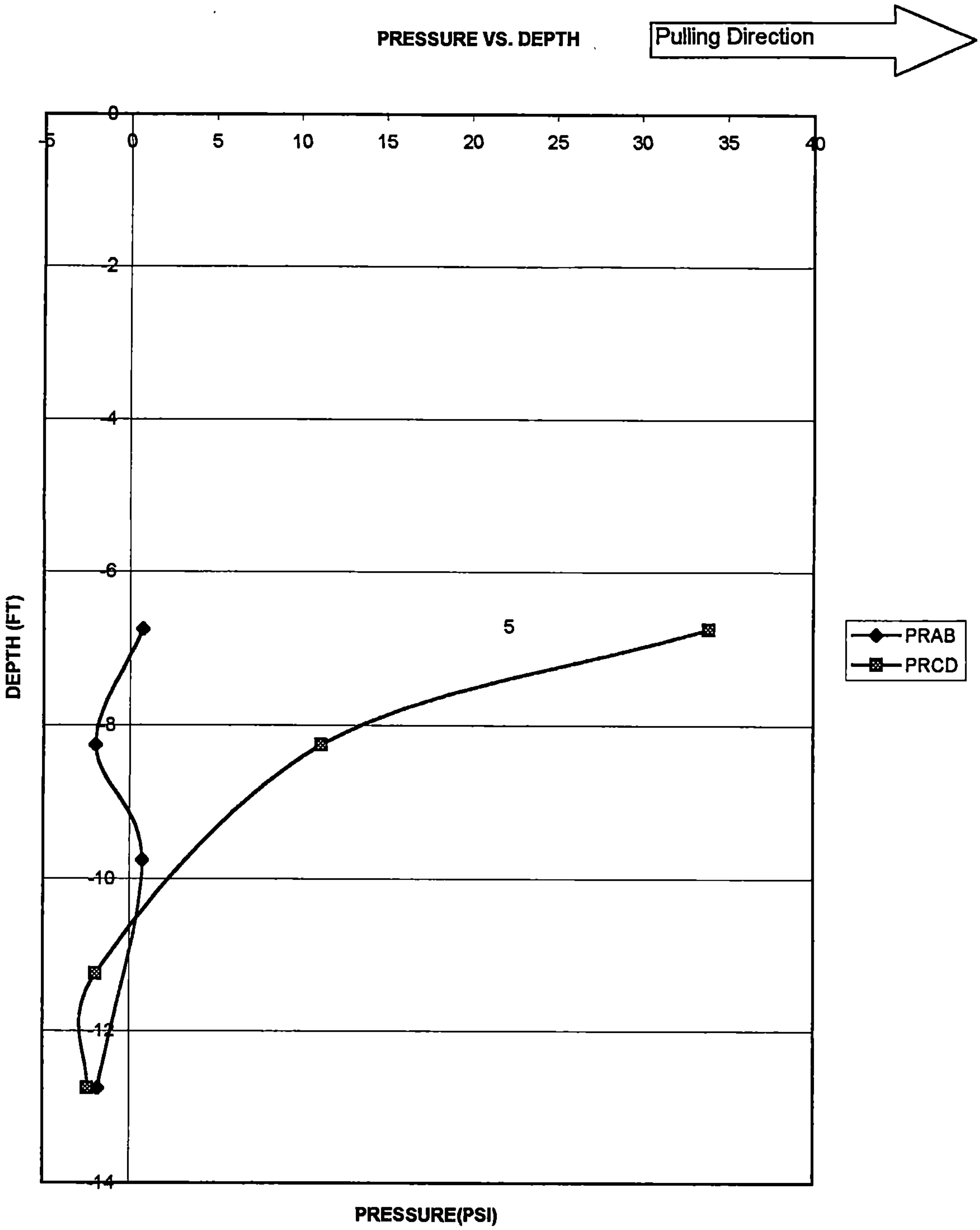

Figure A-53. Pressure vs Depth for pile # 3, pulling east. First 1" test, Feb,26,1999.
Deflection of the pile=.912"

CD Pulling Side
Def.=1.031"

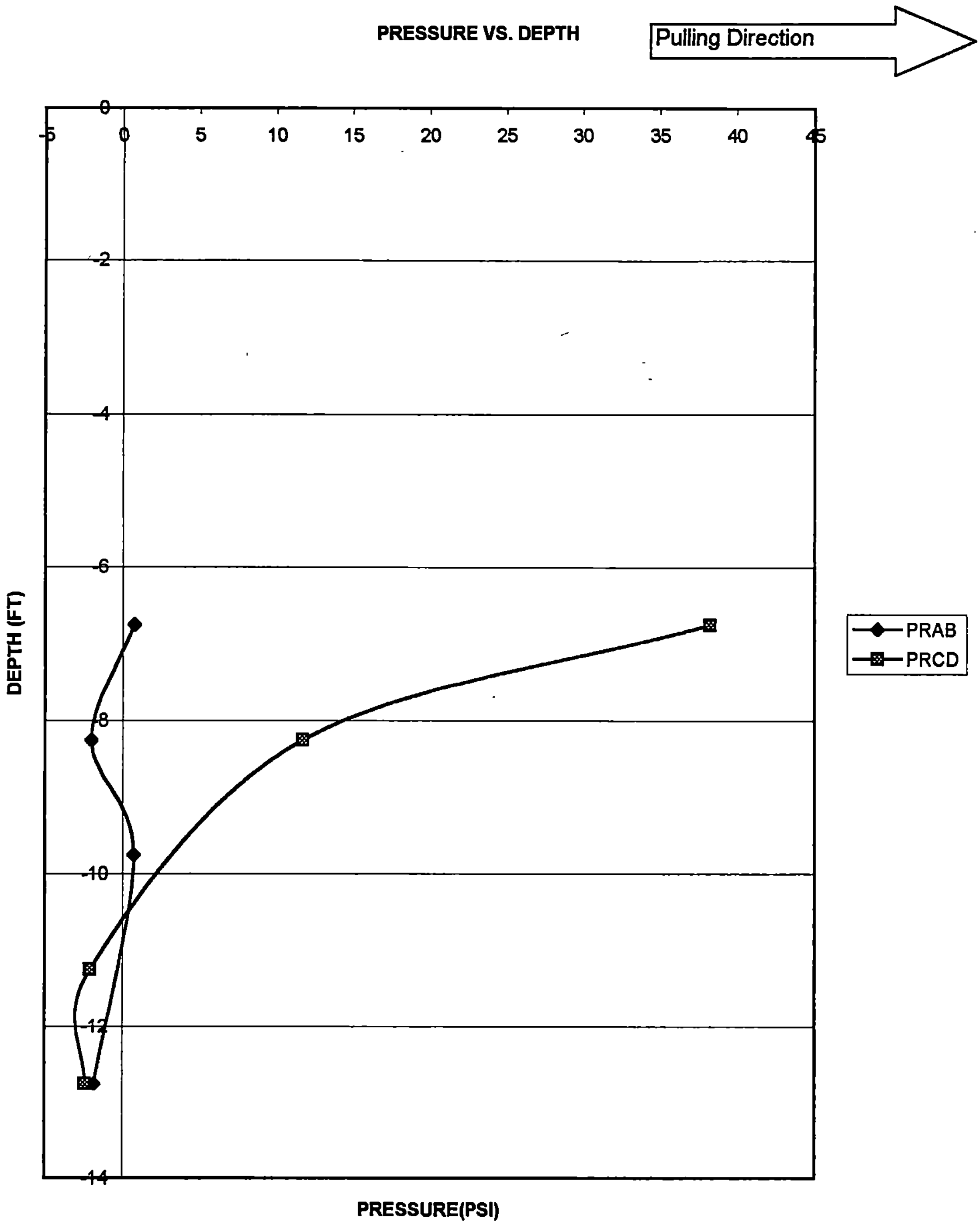

Figure A-54. Pressure vs Depth for pile # 3, pulling east. First 1" test, Feb,26,1999.
Deflection of the pile=1.031"

CD Pulling Side
Def.=PRAB

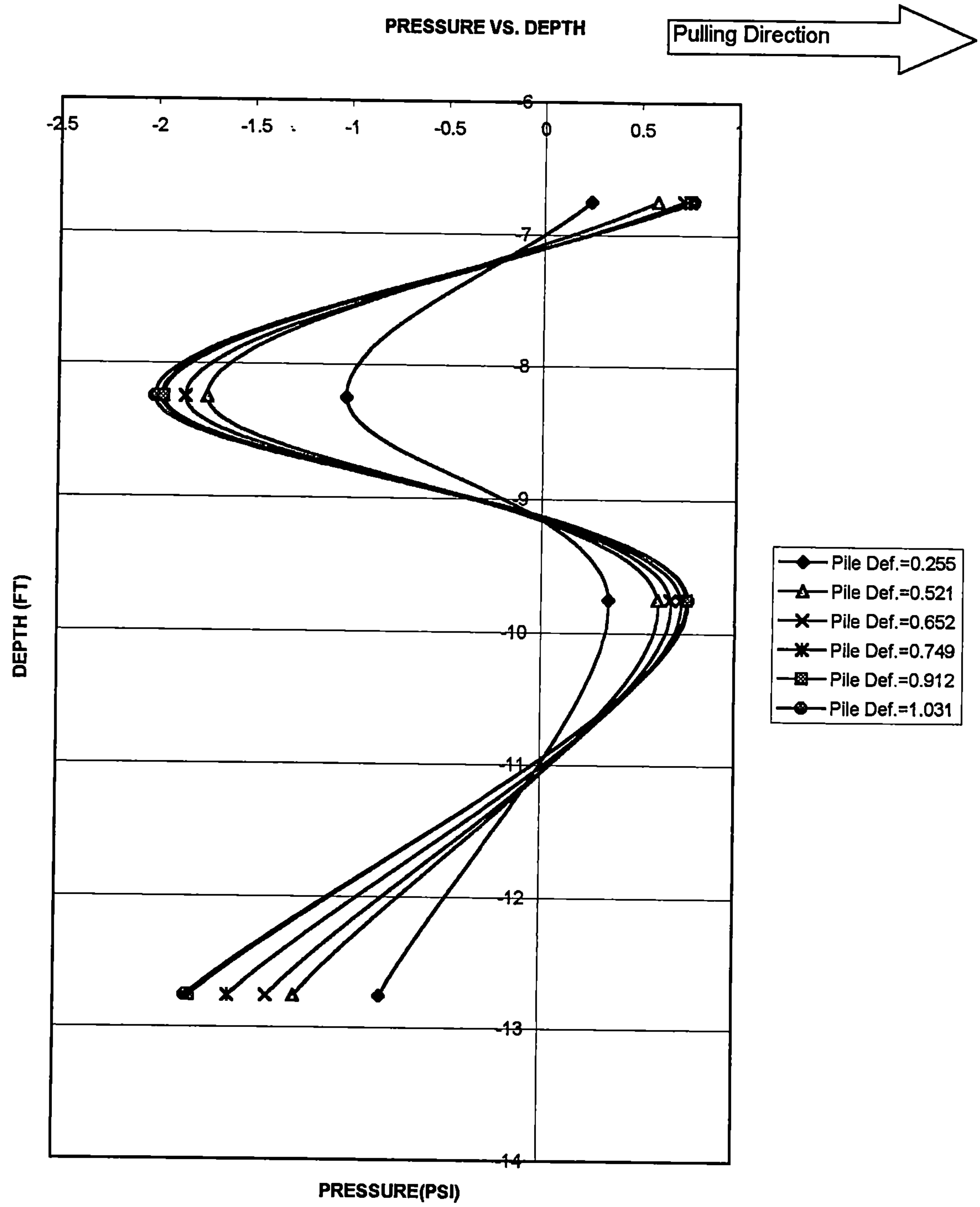

Figure A-55. Pressure vs Depth for pile # 3, pulling east. First 1" test, Feb,26,1999.
AB Side

CD Pulling Side
Def.=PRCD

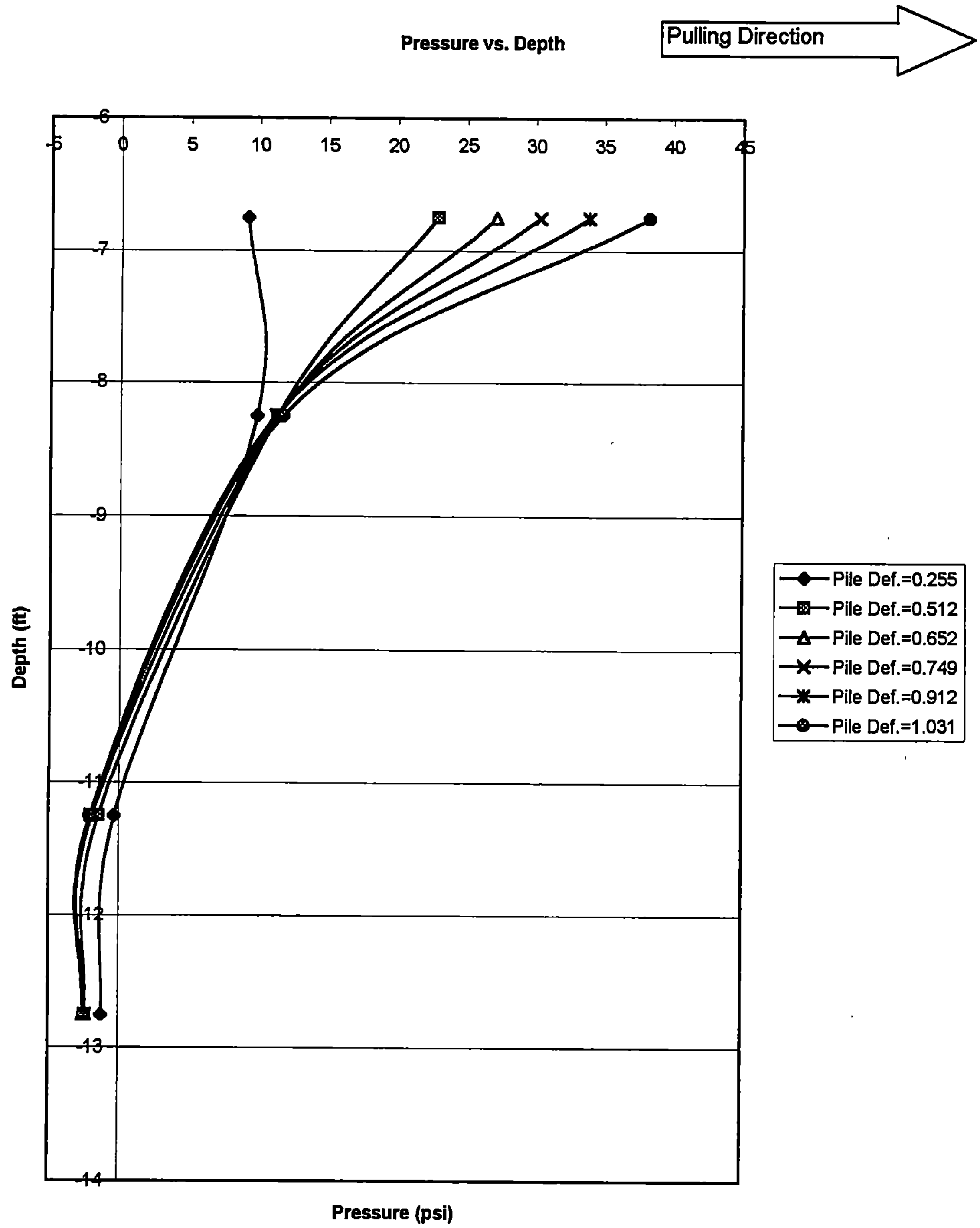

Figure A-56. Pressure vs Depth for pile # 3, pulling east. First 1" test, Feb,26,1999. CD Side

APPENDIX (B)

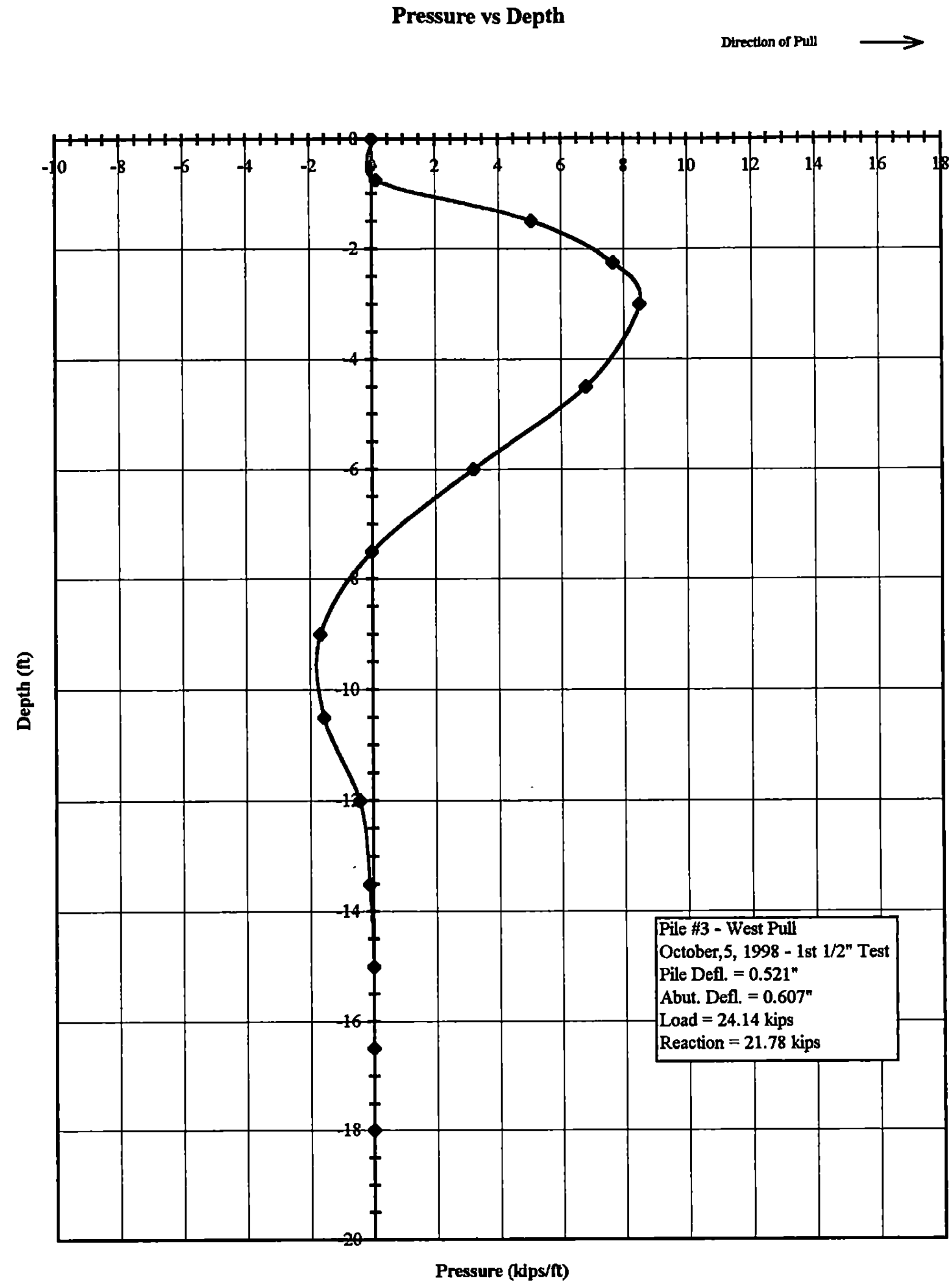

Figure (B-1). Pressure vs Depth for pile # 3, pulling west. First 1/2" test, Oct,05,1999. Strain gage data

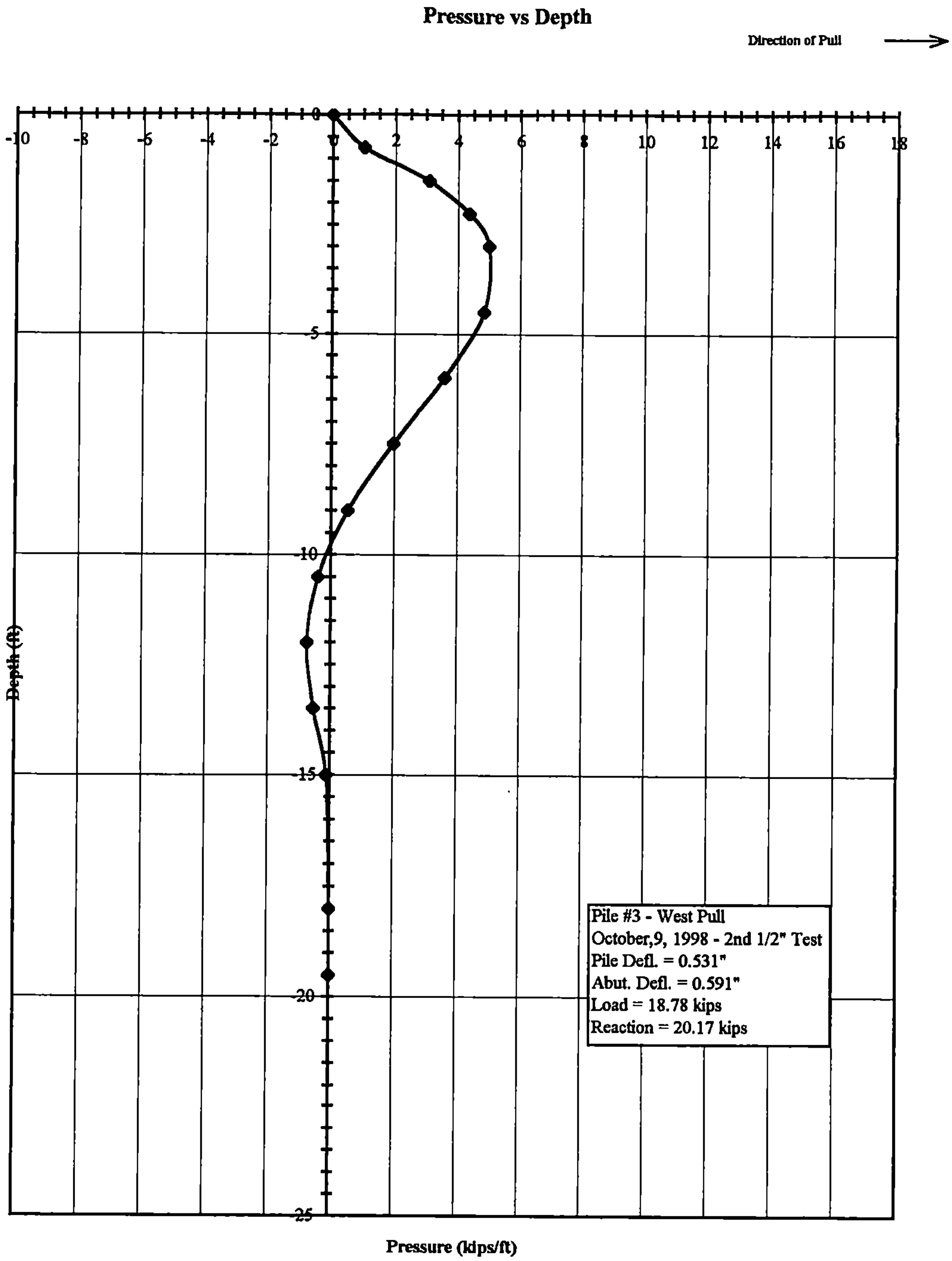

Figure (B-2). Pressure vs Depth for pile # 3, pulling west. Second 1/2" test, Oct,09,1999. Strain gage data.

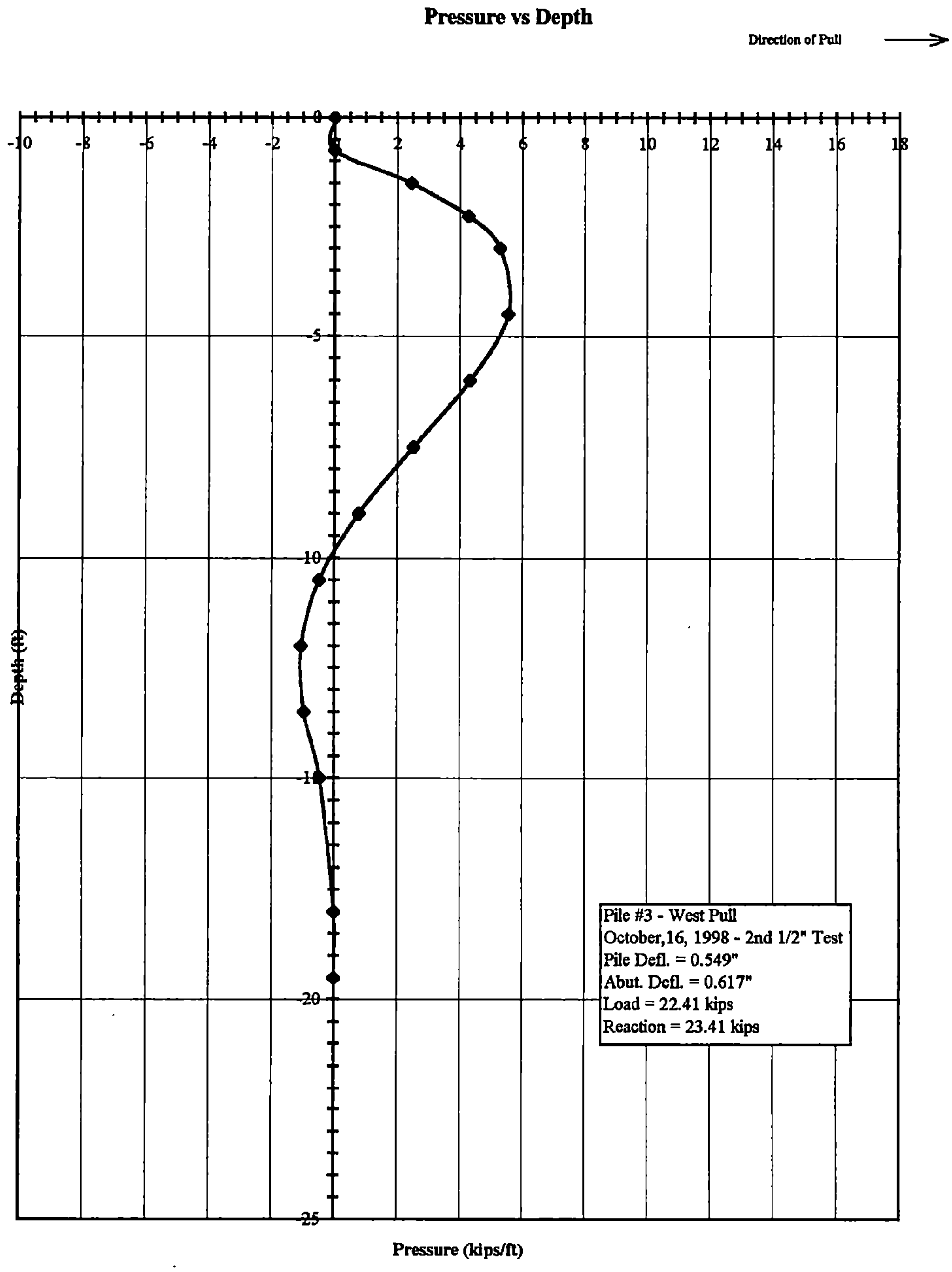

Figure (B-3). Pressure vs Depth for pile # 3, pulling west. Third 1/2" test, Oct,16,1999. Strain gage data.

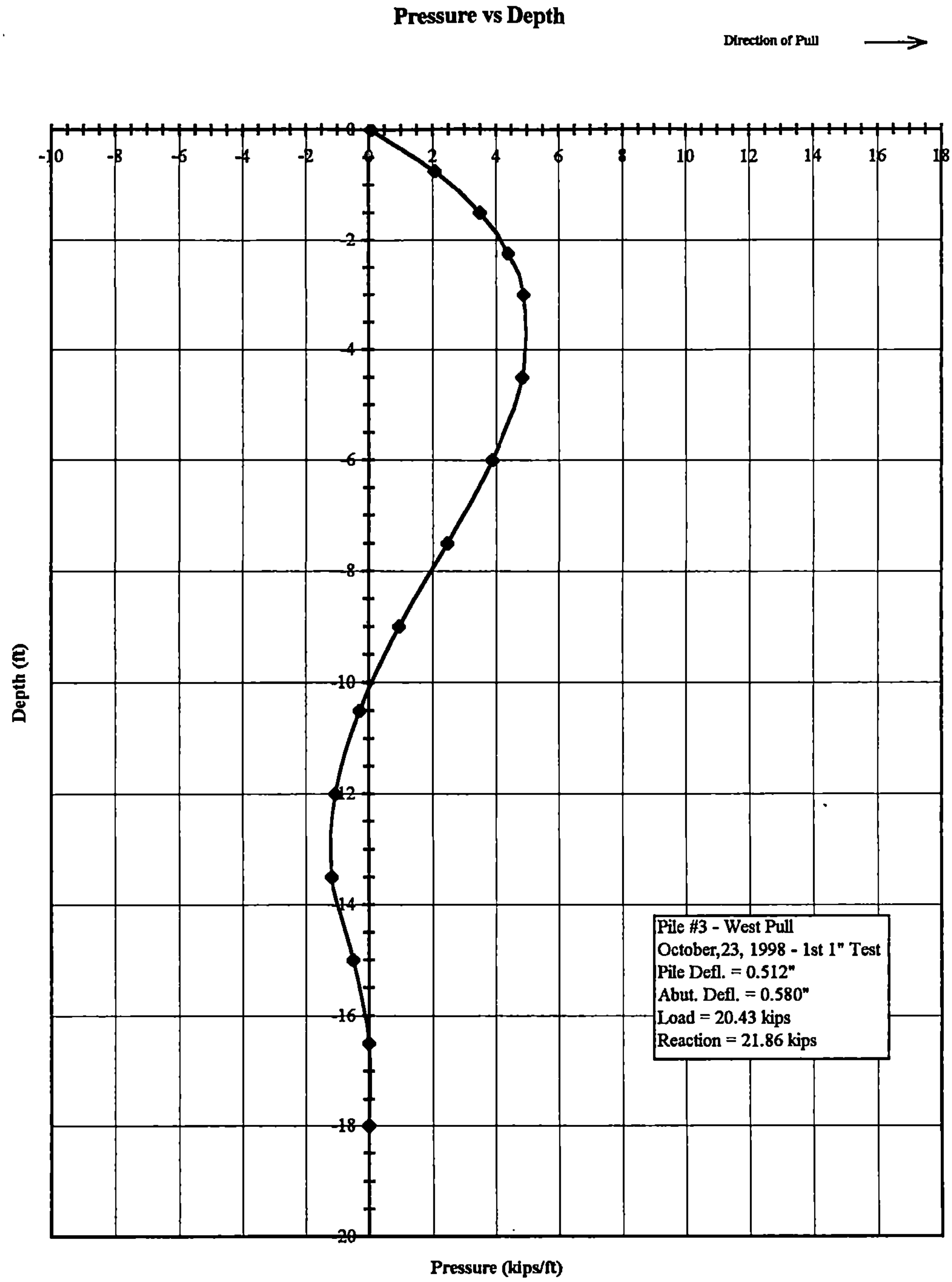

Figure (B-4). Pressure vs Depth for pile # 3, pulling west. First 1" test, Oct,23,1999.
Strain gage data, Deflection of the pile=0.5"

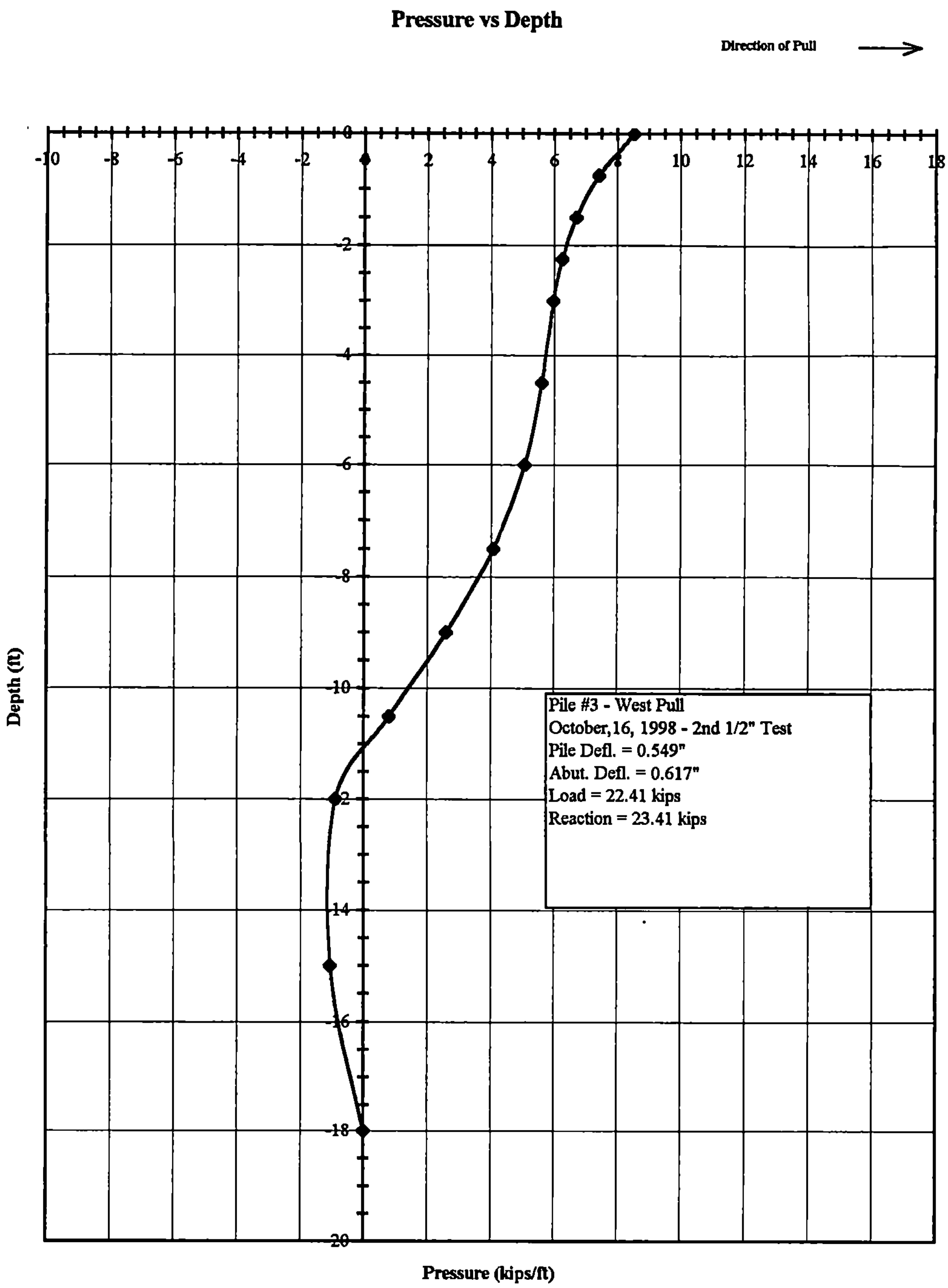

Figure (B-5). Pressure vs Depth for pile # 3, pulling west. First 1" test, Oct,23,1999.
Strain gage data, deflection of the pile=1"

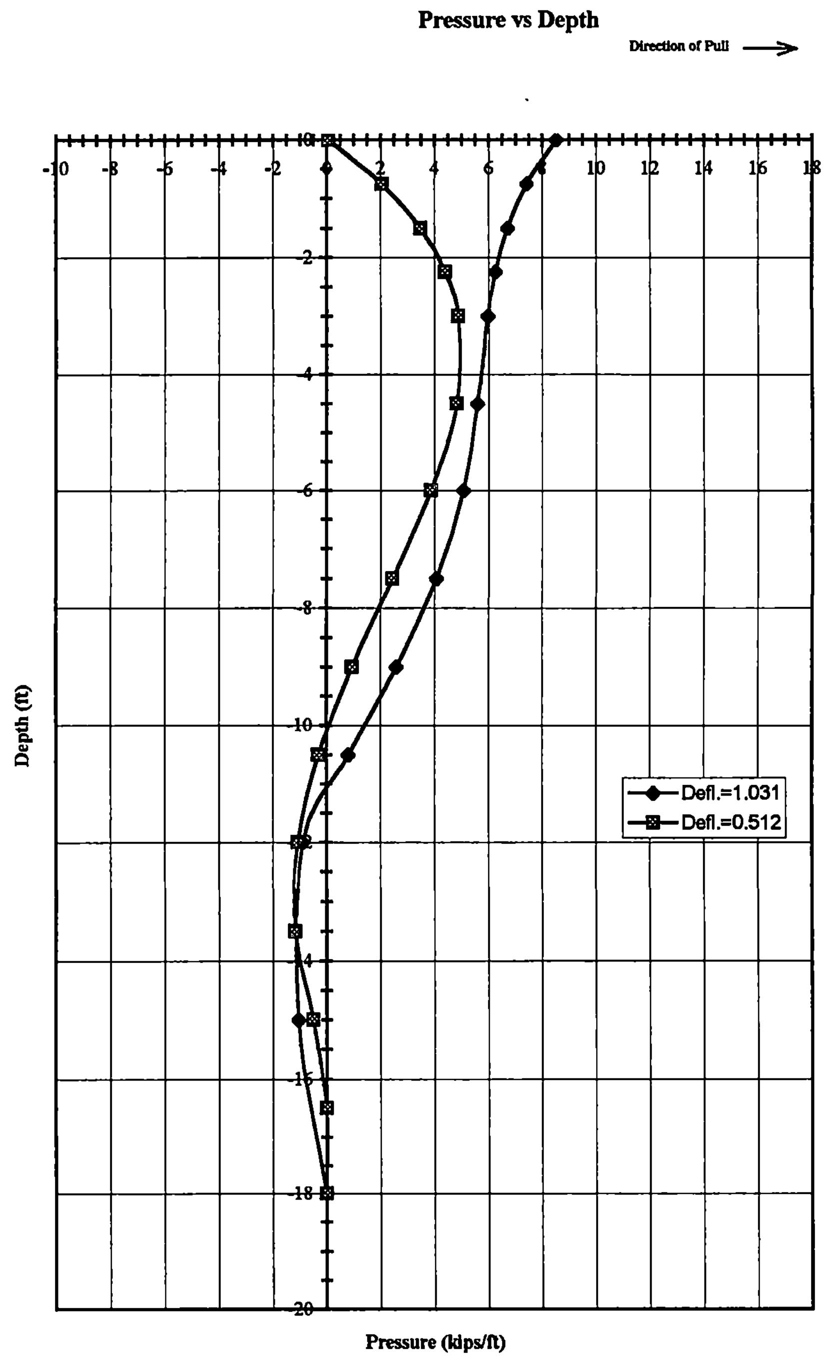

Figure (B-6). Pressure vs Depth for pile # 3, pulling west. First 1" test, Oct,23,1999.
Strain gage data

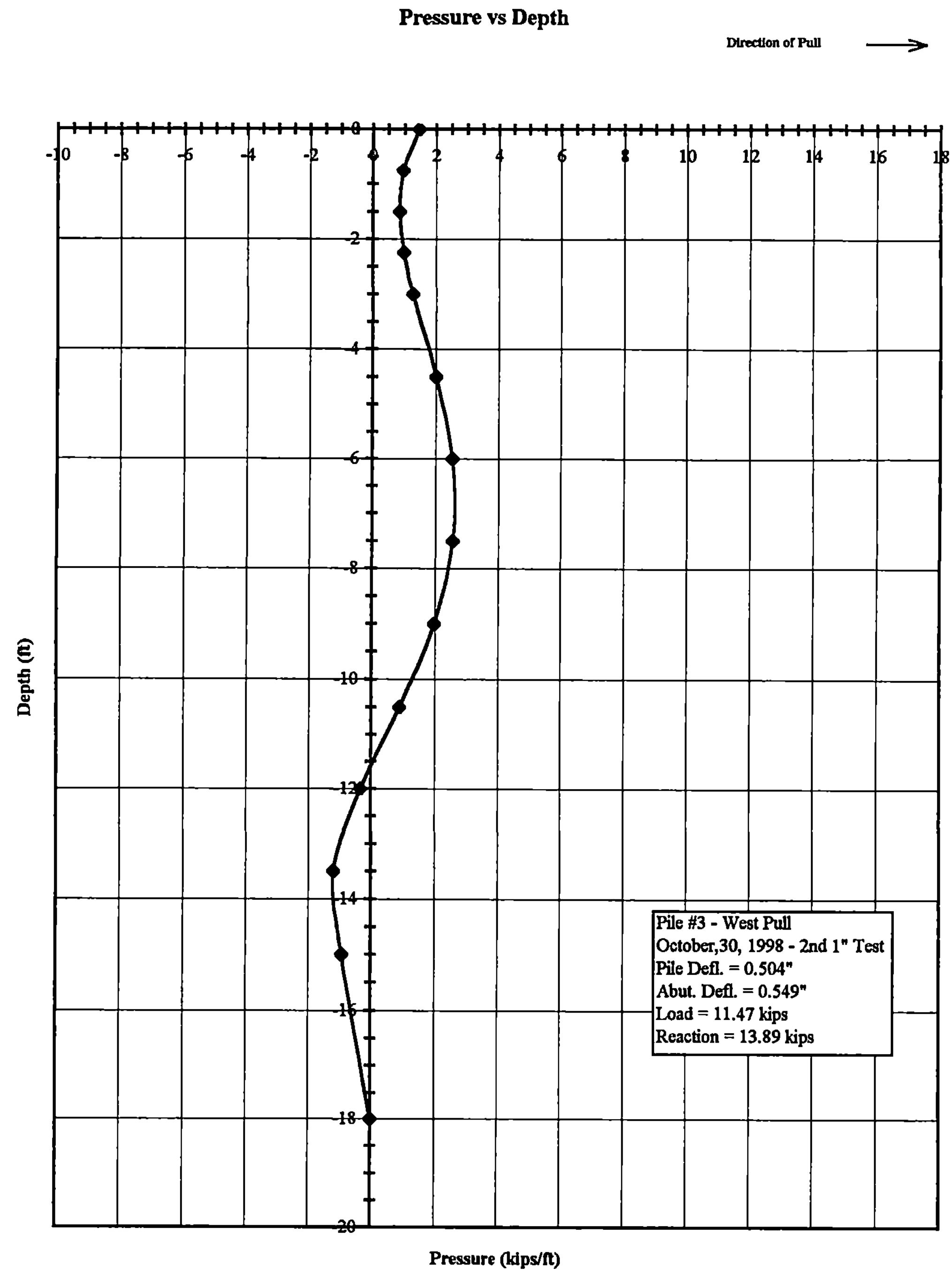

Figure (B-7). Pressure vs Depth for pile # 3, pulling west. Second 1" test, Oct,30,1999. Strain gage data, deflection of the pile=0.5"

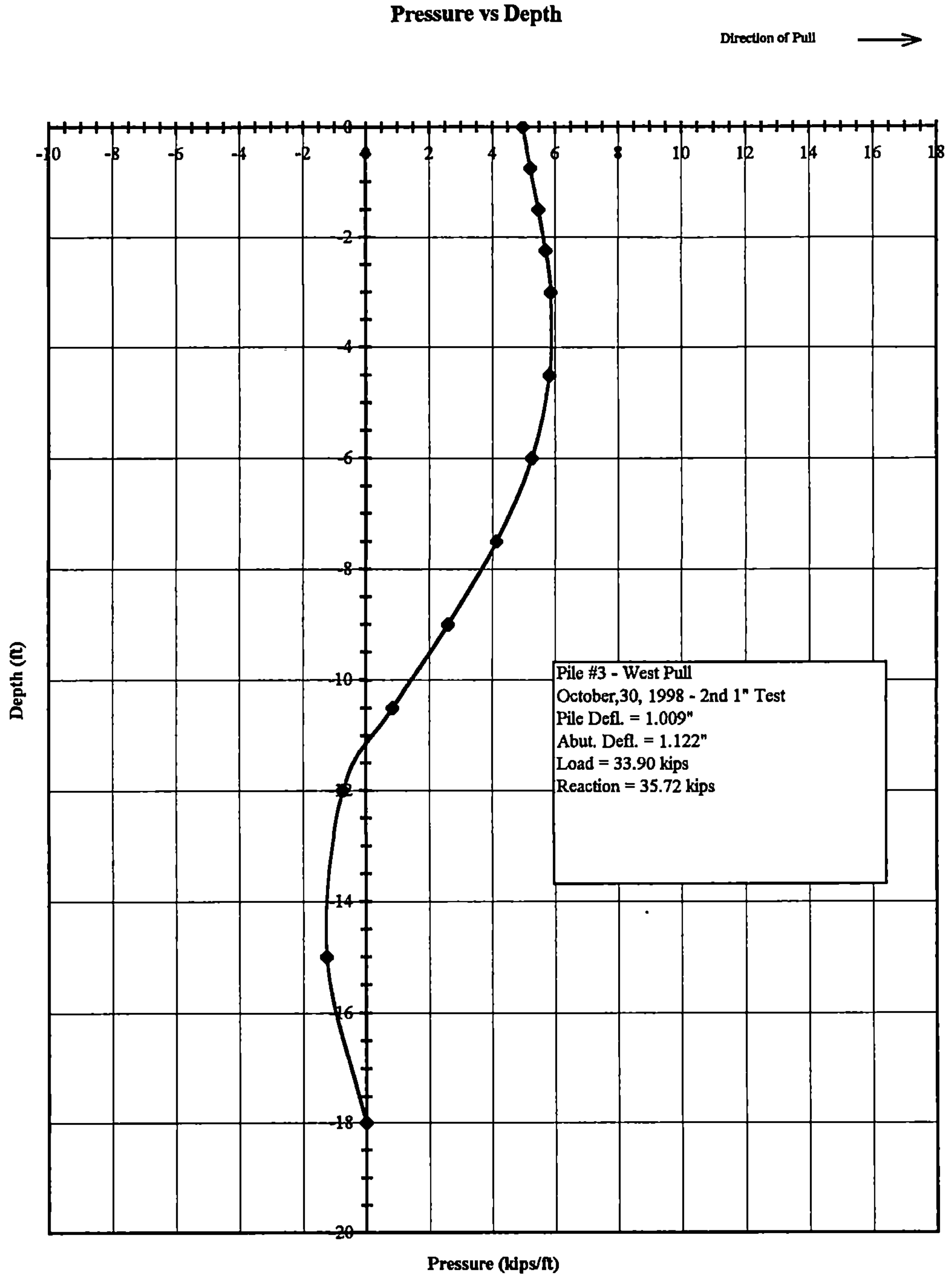

Figure (B-8). Pressure vs Depth for pile # 3, pulling west. Second 1" test, Oct,30,1999.
Strain gage data, deflection of the pile=1"

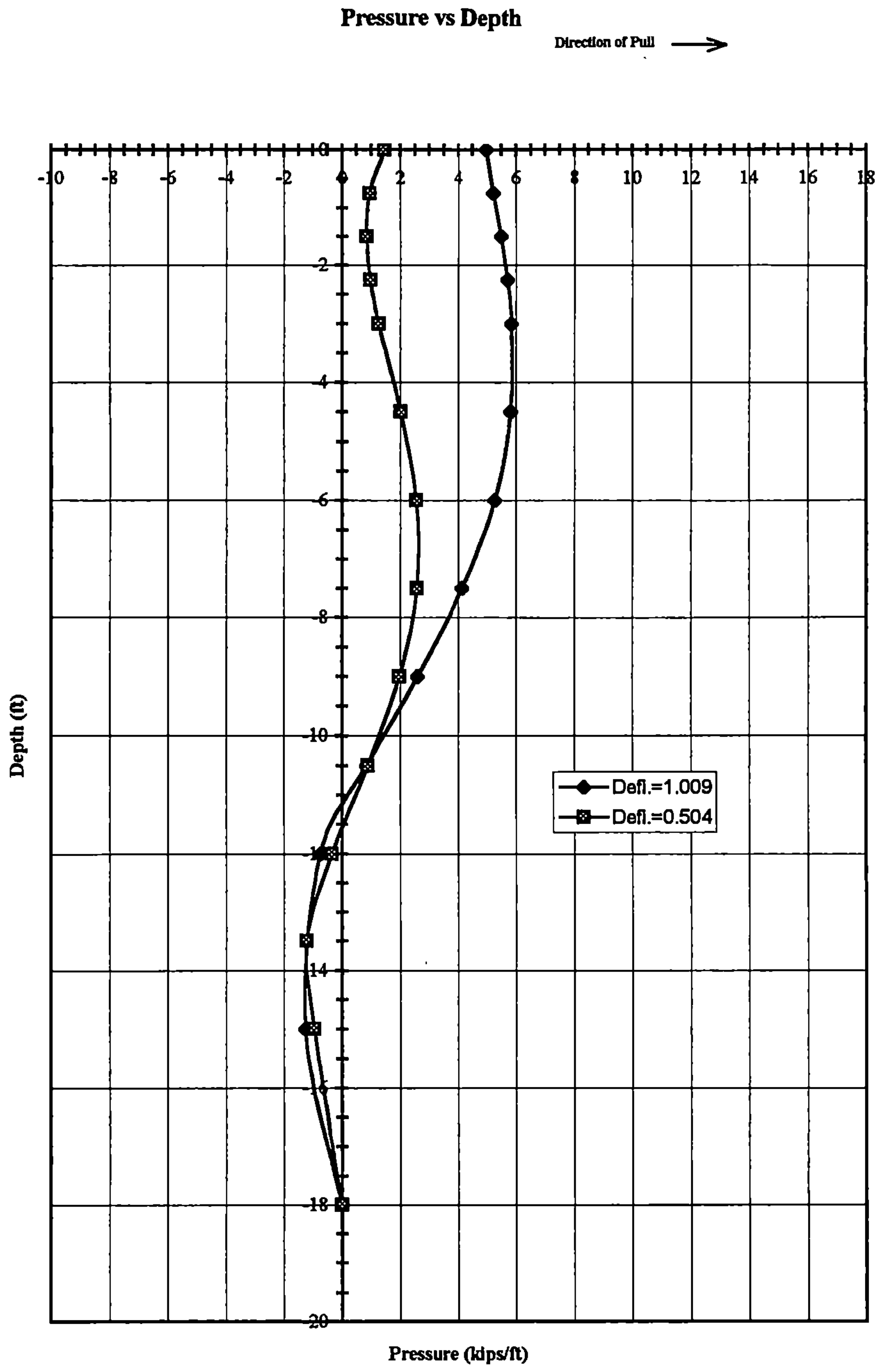

Figure (B-9). Pressure vs Depth for pile # 3, pulling west. Second 1" test, Oct,30,1999.
Strain gage data

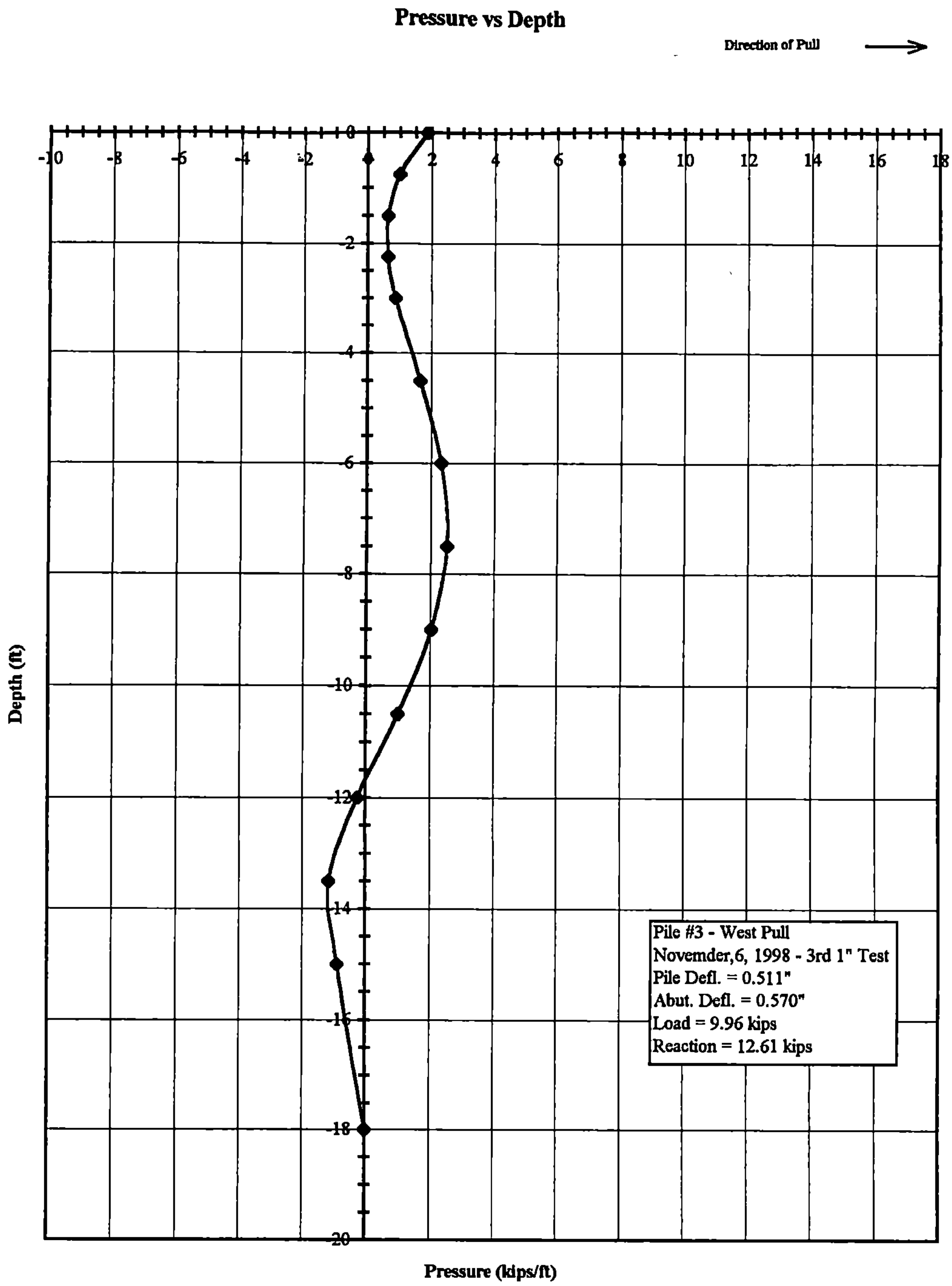

Figure (B-10). Pressure vs Depth for pile # 3, pulling west. Third 1" test, Nov,06,1999. Strain gage data, deflection of the pile=0.5"

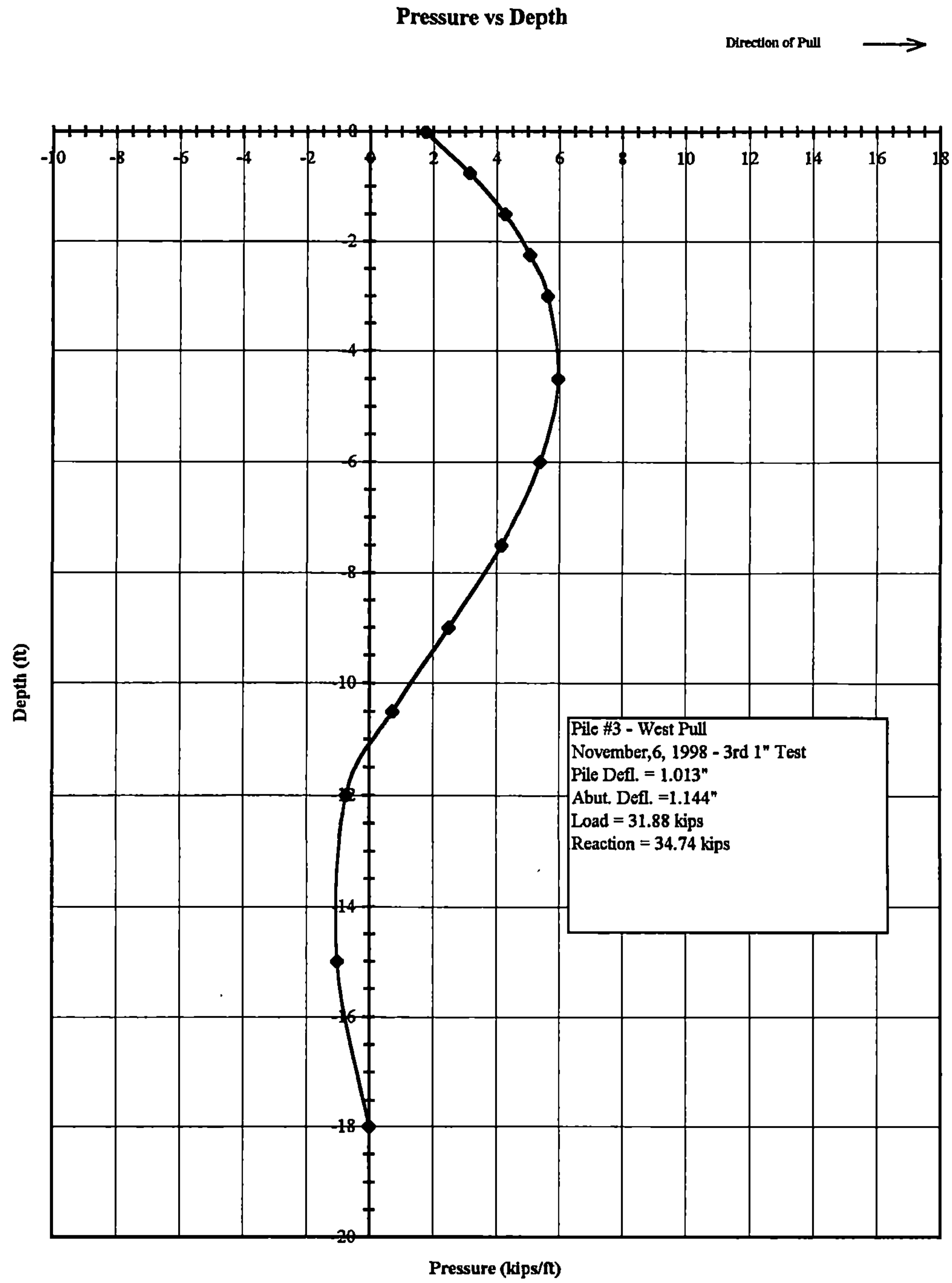

Figure (B-11). Pressure vs Depth for pile # 3, pulling west. Third 1" test, Nov,06,1999.
Strain gage data, deflection of the pile=1"

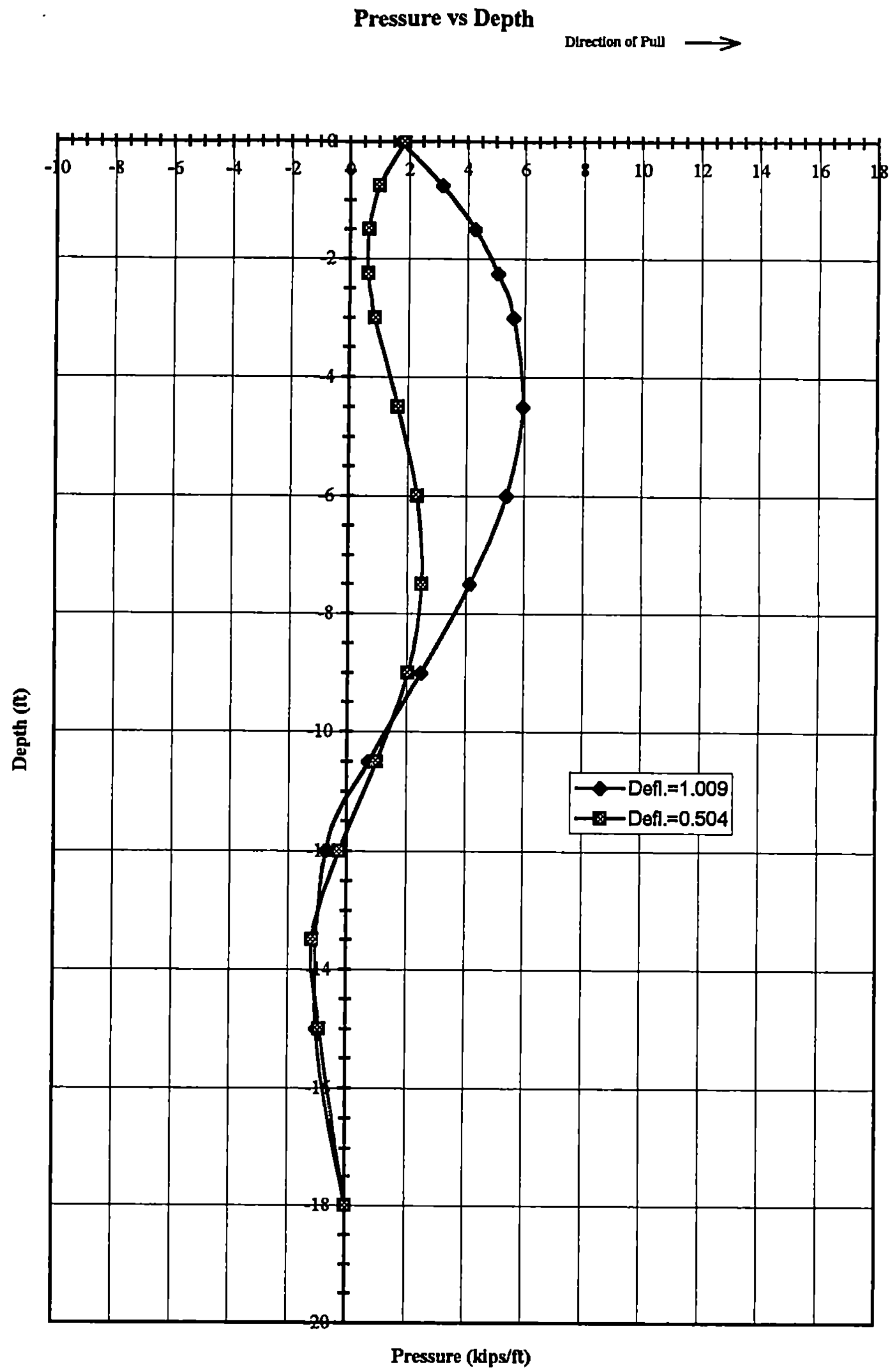

Figure (B-12). Pressure vs Depth for pile # 3, pulling west. Third 1" test, Nov,06,1999.
Strain gage data

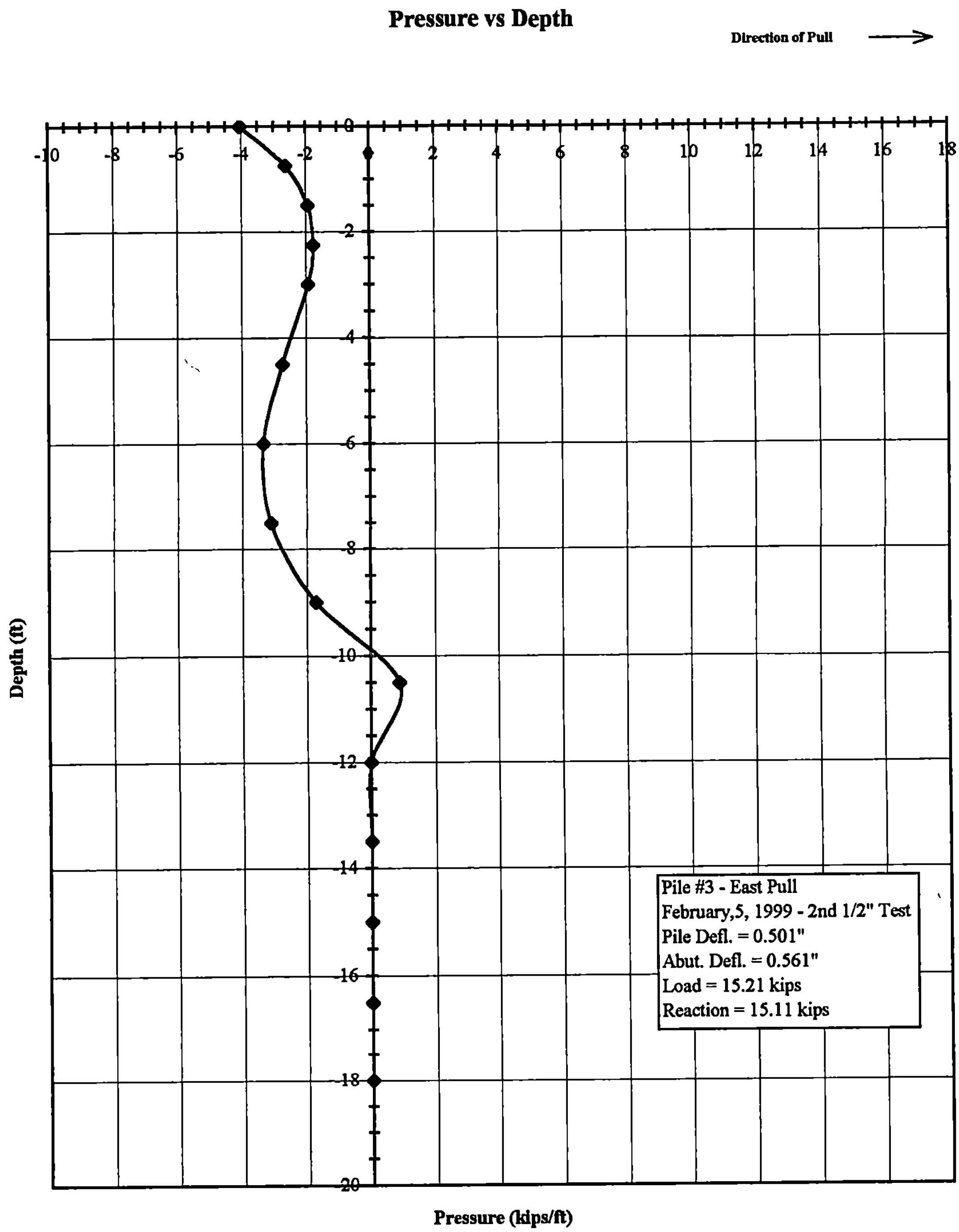

Figure (B-13). Pressure vs Depth for pile # 3,pulling east. Second 1/2" test,Feb,05,1999
Strain gage data

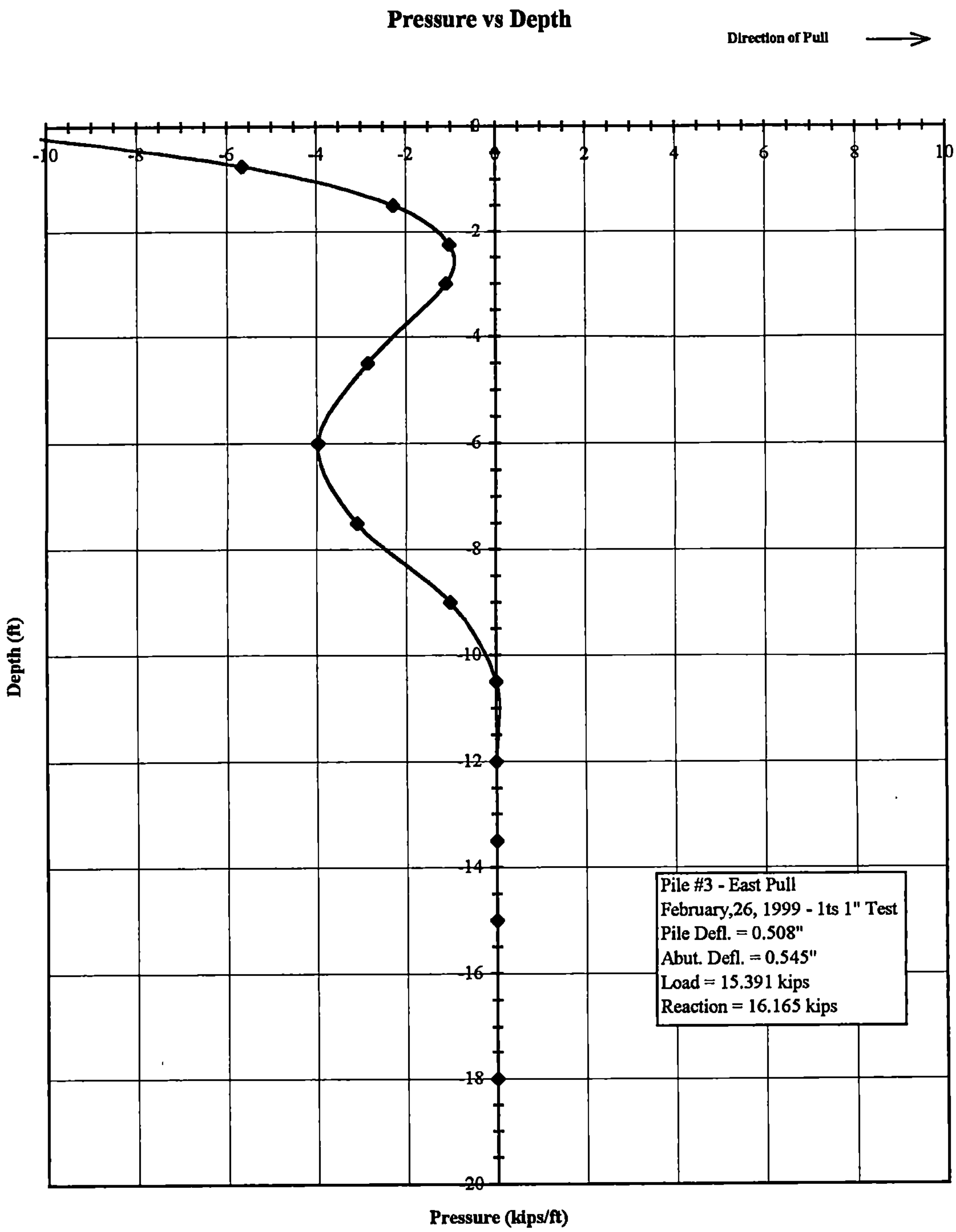

Figure (B-14), Pressure vs Depth from Strain Gage Data. First 1" test, Feb,26,1999
Strain gage data, deflection of the pile=0.5"

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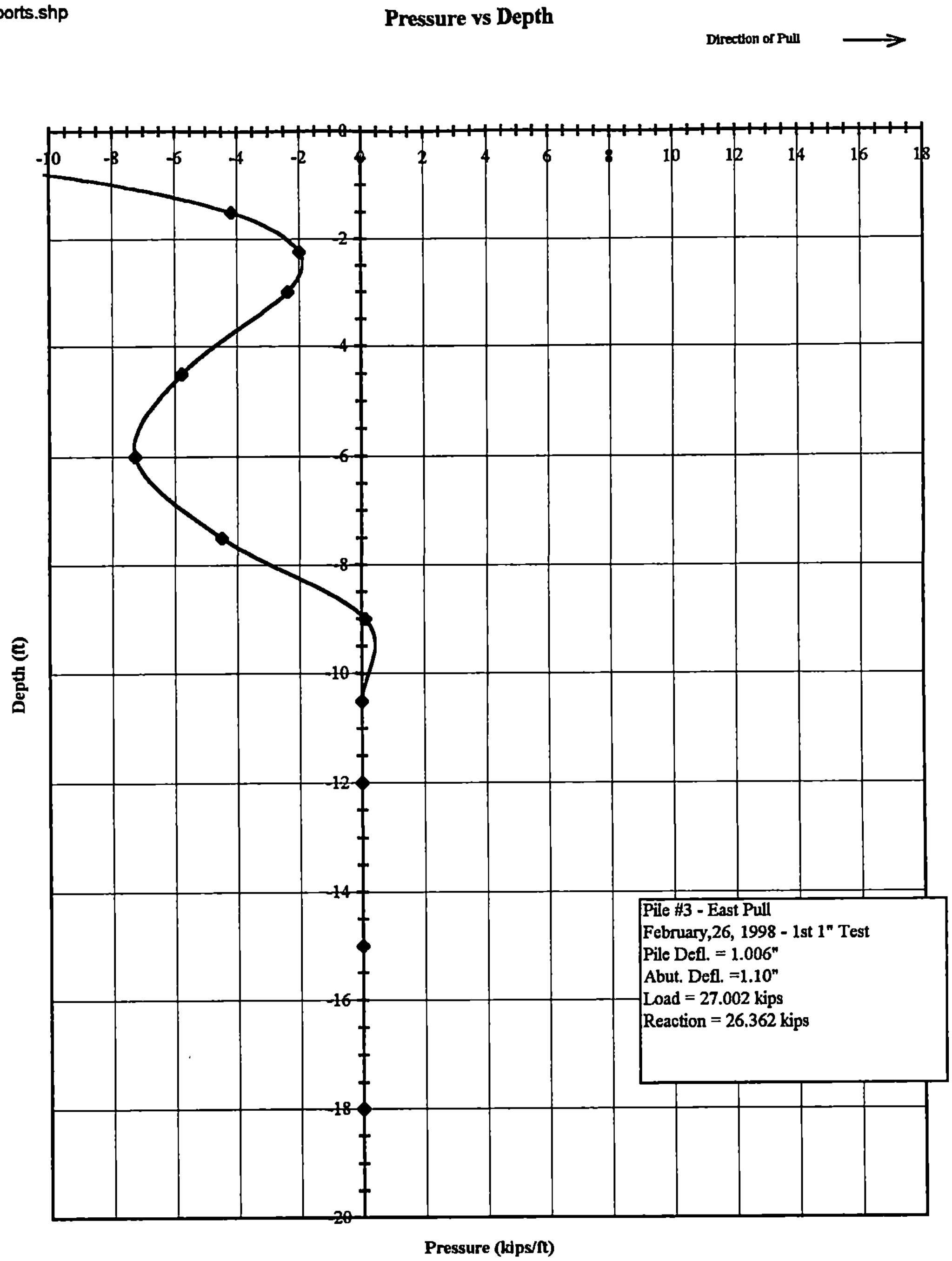

Figure (B-15). Pressure vs Depth from Strain Gage Data. First 1" test,Feb,26,1999
Strain gage data, deflection of the pile=1"

Pressure vs Depth

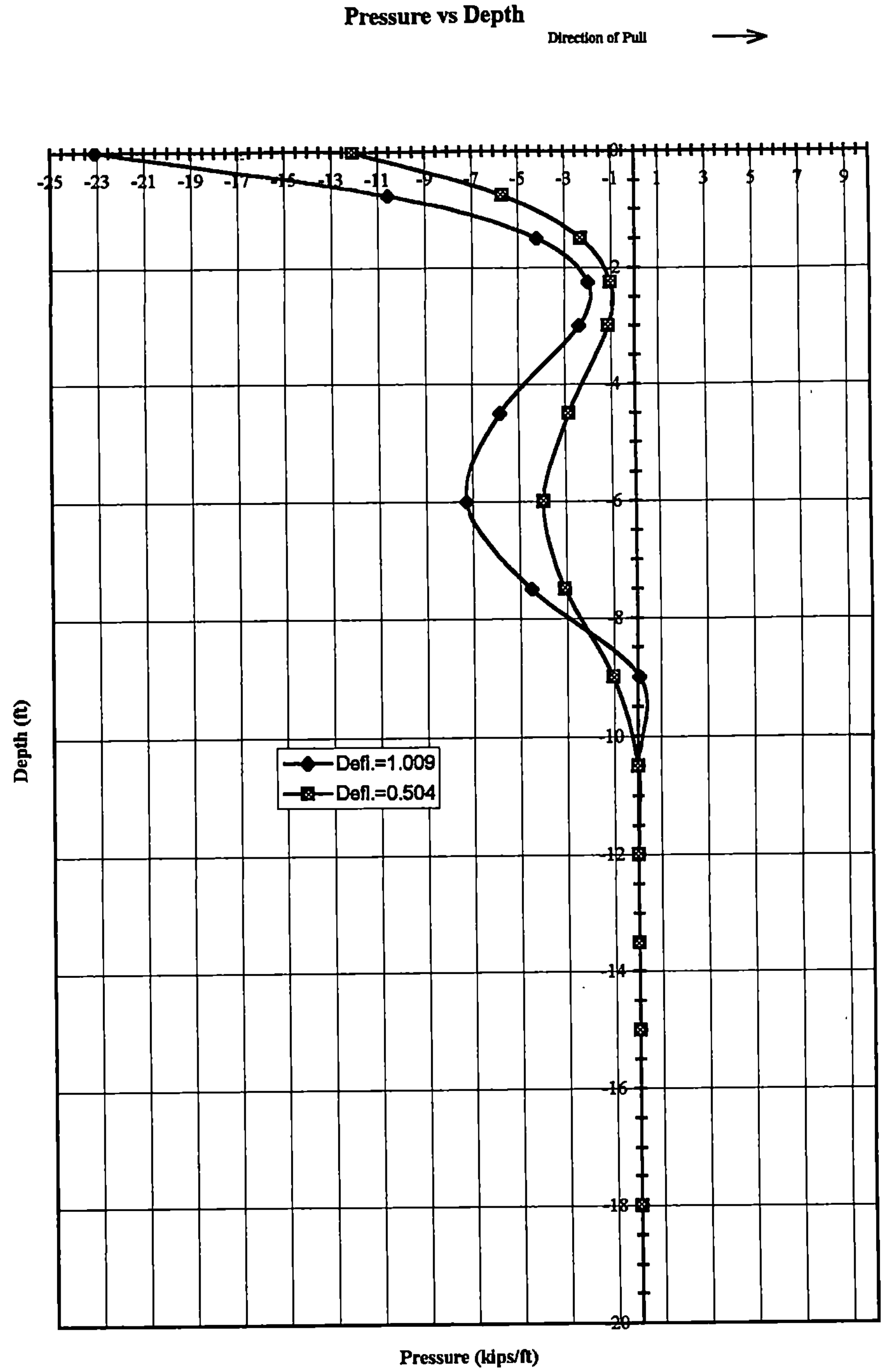

Figure (B-16). Pressure vs Depth from Strain Gage Data. First 1" test,Feb,26,1999
Strain gage data

VITA

Samer Al-Jamal was born in Riyadh, Saudi Arabia on January 6,1975. In January 1997 he received his Bachelor of Science degree in Civil Engineering from Jordan University of Science & Technology. He began graduate school in the fall of 1997 in University of Tennessee. He received his Master of Science degree in Civil Engineering in May 1999.