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AnEvaluation of LPILE in Comparison to the Measured Behavior of Prestressed Concrete Piles in Integral Abutments

Jarrell Brandon Tidwell
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To the Graduate Council:

I am submitting herewith a thesis written by Jarrell Brandon Tidwell entitled "AnEvaluation of LPILE in Comparison to the Measured Behavior of Prestressed Concrete Piles in Integral Abutments." 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.

Edwin G. Burdette, Major Professor

We have read this thesis and recommend its acceptance:

James H. Deatherage, Earl E. Ingram

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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Earl E. Ingram

Acceptance for the Council:

Anne Mayhew
Vice Chancellor and Dean of
Graduate Studies

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

**An Evaluation of LPILE in Comparison to the Measured
Behavior of Prestressed Concrete Piles
in Integral Abutments**

A Thesis

Presented for the

Master of Science

Degree

The University of Tennessee, Knoxville

Jarrell Brandon Tidwell

May 2004

Dedication

I dedicate this thesis to my parents and grandparents, Ricky and Angela Tidwell, James and Erma Tidwell, and Jimmy and Francis Nolen.

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I extend my thanks to the following graduate and undergraduate students who helped make this thesis possible: Don Lowe, Alan Eichelman, David Chapman, Adam Guidry, Patrick McMahon, and a special thanks to Sam Howard for his guidance, advice and unbelievable patience.

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I thank the Tennessee Department of Transportation. TDOT's funding made this project and my graduate career possible.

Once again, I want to thank Sam Howard. During my graduate and undergraduate careers, he helped me with many problems. One such problem, staying in college for 20 years is unacceptable.

Abstract

The Tennessee Department of Transportation (TDOT) has designed and built integral abutment bridges with prestressed concrete piles for many years. A product of constructing integral bridge abutments is that the piles supporting the structure are exposed to lateral deflections, which come mainly from thermal expansion and contraction of the bridge structure. TDOT funded a project in which the University of Tennessee, Knoxville investigated the effects of the lateral displacement of four 14 inch square prestressed concrete piles that were driven into virgin clay soil.

The tests involved full scale lateral loading each of the four prestressed piles; each pile supported a simulated integral bridge abutment. After each pile was driven into the ground, the model abutment and a “pulling slab” were cast on top of the pile. Leaving one foot of pile exposed between the abutment and the ground, University personnel instrumented each pile with strain gages on that exposed surface. The tests then included a basic testing regime of lateral displacements that depended on incremental displacement and time rate of loading. During the tests, moments, deflections, and shears were recorded at the pile-abutment interface.

LPILE is a program that uses a finite difference analysis to calculate the behavior of a pile. While the piles in the field were not instrumented below the surface of the ground, LPILE was used to develop moment versus depth diagrams. Also the analysis evaluated shear and displacement versus depth graphs. LPILE was evaluated on its ability to predict the values measured at the pile abutment interface in the field tests and the consistency with which LPILE located maximum values and inflection points. The LPILE analyses were crude approximations of measured values.

Table of Contents

Chapter	Page
1. Introduction.....	1
2. Literature Review.....	3
2.1 University of Tennessee Project History.....	3
2.2 Previous Research Work.....	4
3. Test Set-up.....	8
4. LPILE.....	11
5. Application and Evaluation of LPILE.....	13
5.1 Introduction.....	13
5.2 Soil Information.....	13
5.3 Method.....	17
5.4 Evaluation.....	20
5.5 MATLAB.....	25
5.6 Summary and Discussion.....	30
6. Conclusions.....	32
List of References.....	34

Appendices.....	37
Appendix A: Sample Graphs.....	38
Appendix B: LPILE.....	81
Appendix C: MATLAB.....	95
Vita.....	109

List of Tables

Table	Page
1. List of Rotational Stiffness of Set-up.....	14
2. Values Retrieved from the Subsurface Investigation.....	17
3. Field Results in Comparison to LPILE Shear Results.....	22
4. Field Results in Comparison to LPILE's Post-"tweaked" Moment and Displacement Results.....	24
5. Statistics of Variations in LPILE's Post-"tweaked" Predicted Values.....	26

List of Figures

Figure	Page
1. Test Setup for Phase II Tests.....	9
2. Bore Hole 1 Stress vs. Strain.....	15
3. Bore Hole 2 Stress vs. Strain.....	16
4. Plot of High and Low Shear Versus Displacement Boundaries with LPILE Results Graphed.....	28
5. Plot of High and Low Moment Versus Displacement Boundaries with LPILE Results Graphed.....	29
A1. Deflection vs. Depth Pile 2 Pulling North Assuming Soft Clay, ½ inch.....	39
A2. Deflection vs. Depth Pile 2 Pulling North Assuming Stiff Clay, ½ inch.....	40
A3. Shear vs. Depth Pile 2 Pulling North Assuming Soft Clay, ½ inch.....	41
A4. Shear vs. Depth Pile 2 Pulling North Assuming Stiff Clay, ½ inch.....	42
A5. Moment vs. Depth Pile 2 Pulling North Assuming Soft Clay, ½ inch.....	43
A6. Moment vs. Depth Pile 2 Pulling North Assuming Stiff Clay, ½ inch.....	44
A7. Deflection vs. Depth Pile 4 Pulling North Assuming Soft Clay, 1 inch.....	45
A8. Deflection vs. Depth Pile 4 Pulling North Assuming Stiff Clay, 1 inch.....	46
A9. Shear vs. Depth Pile 4 Pulling North Assuming Soft Clay, 1 inch.....	47
A10. Shear vs. Depth Pile 4 Pulling North Assuming Stiff Clay, 1 inch.....	48
A11. Moment vs. Depth Pile 4 Pulling North Assuming Soft Clay, 1 inch.....	49
A12. Moment vs. Depth Pile 4 Pulling North Assuming Stiff Clay, 1 inch.....	50
A13. Deflection vs. Depth Pile 2 Pulling South Assuming Stiff Clay, ½ inch.....	51
A14. Shear vs. Depth Pile 2 Pulling South Assuming Stiff Clay, ½ inch.....	52

A15.	Moment vs. Depth Pile 2 Pulling South Assuming Stiff Clay, ½ inch.....	53
A16.	Deflection vs. Depth Pile 2 Pulling South Assuming Stiff Clay, 1 inch.....	54
A17.	Shear vs. Depth Pile 2 Pulling South Assuming Stiff Clay, 1 inch.....	55
A18.	Moment vs. Depth Pile 2 Pulling South Assuming Stiff Clay, 1 inch.....	56
A19.	Deflection vs. Depth Pile 2 Pulling North Assuming Stiff Clay, 1 inch.....	57
A20.	Shear vs. Depth Pile 2 Pulling North Assuming Stiff Clay, 1 inch.....	58
A21.	Moment vs. Depth Pile 2 Pulling North Assuming Stiff Clay, 1 inch.....	59
A22.	Deflection vs. Depth Pile 3 Pulling South Assuming Stiff Clay, ½ inch.....	60
A23.	Shear vs. Depth Pile 3 Pulling South Assuming Stiff Clay, ½ inch.....	61
A24.	Moment vs. Depth Pile 3 Pulling South Assuming Stiff Clay, ½ inch.....	62
A25.	Deflection vs. Depth Pile 3 Pulling North Assuming Stiff Clay, ½ inch.....	63
A26.	Shear vs. Depth Pile 3 Pulling North Assuming Stiff Clay, ½ inch.....	64
A27.	Moment vs. Depth Pile 3 Pulling North Assuming Stiff Clay, ½ inch.....	65
A28.	Deflection vs. Depth Pile 3 Pulling South Assuming Stiff Clay, 1 inch.....	66
A29.	Shear vs. Depth Pile 3 Pulling South Assuming Stiff Clay, 1 inch.....	67
A30.	Moment vs. Depth Pile 3 Pulling South Assuming Stiff Clay, 1 inch.....	68
A31.	Deflection vs. Depth Pile 3 Pulling North Assuming Stiff Clay, 1 inch.....	69
A32.	Shear vs. Depth Pile 3 Pulling North Assuming Stiff Clay, 1 inch.....	70
A33.	Moment vs. Depth Pile 3 Pulling North Assuming Stiff Clay, 1 inch.....	71
A34.	Deflection vs. Depth Pile 4 Pulling South Assuming Stiff Clay, ½ inch.....	72
A35.	Shear vs. Depth Pile 4 Pulling South Assuming Stiff Clay, ½ inch.....	73
A36.	Moment vs. Depth Pile 4 Pulling South Assuming Stiff Clay, ½ inch.....	74
A37.	Deflection vs. Depth Pile 4 Pulling North Assuming Stiff Clay, ½ inch.....	75

A38.	Shear vs. Depth Pile 4 Pulling North Assuming Stiff Clay, ½ inch.....	76
A39.	Moment vs. Depth Pile 4 Pulling North Assuming Stiff Clay, ½ inch.....	77
A40.	Deflection vs. Depth Pile 4 Pulling South Assuming Stiff Clay, 1 inch.....	78
A41.	Shear vs. Depth Pile 4 Pulling South Assuming Stiff Clay, 1 inch.....	79
A42.	Moment vs. Depth Pile 4 Pulling South Assuming Stiff Clay, 1 inch.....	80
B1.	LPILE Menu to Enter Data for the Pile and Soil.....	83
B2.	Pile Properties- Pile 3 0.5-inch South Post-“tweaked”.....	83
B3.	Loading Type and Boundary Conditions- Pile 3 0.5-inch South Post-“tweaked”.....	84
B4.	Soil Type- Pile 3 0.5-inch South Post-“tweaked”.....	84
B5.	LPILE Menu to Execute Analysis.....	85
B6.	LPILE Menu to Review Graphical Results.....	85
C1.	Background and Purpose for the Program.....	106
C2.	Inquiry for LPILE Results to be Graphed.....	106
C3.	Dialog Box for Displacement Range and Increments.....	107
C4.	Conclusion Box Informing that Program is Finished.....	107
C5.	MATLAB Command Window Displaying Requested Results.....	108

CHAPTER 1

Introduction

In Tennessee's long history of road construction, engineers have struggled to find the most efficient bridge design. After weighing the costs and benefits of various aspects of design, TDOT began to widely use integral bridges. In an integral bridge abutment, the abutment is cast around the ends of the beams. Thus, the abutment piles must be able to withstand axial and lateral loads. The bridge length is limited by the capacity of either the supporting piles or the abutment itself to sustain these loads, which are a function of many different variables including horizontal displacement, soil characteristics, and pile stiffness. With this design, expansion joints are becoming obsolete in short span bridges. Integral bridges increase construction speeds and reduce initial and maintenance costs while maximizing redundancy and service life.

The University of Tennessee, Knoxville, in work sponsored by the Tennessee Department of Transportation (TDOT), completed two phases of research on integral bridge abutments. In Phase I, which began in April of 1996 and concluded in December 1999, steel HP 10x42 piles were tested. In July of 2000, Phase II began. Subsequently, four, 14 inch square prestressed concrete piles were tested; testing on concrete piles was completed in August of 2002.

Prestressed concrete piles have been used in Tennessee's bridge construction in the soil in the western part of the state for some time. Until recently, the prestressing

strands were cut along with the prestressed piles, leaving only the pile to integrate into the abutment. Currently, approximately three feet of these prestressing strands are left exposed to integrate into the abutment along with the pile.

In the tests on steel H-piles, strain gages were used to obtain strains and then moments along the length of the pile. No strain gages were placed below the ground on the prestressed concrete piles; thus, moment diagrams were not obtained. However, as noted later herein, a test to failure of one pile, followed by excavation to examine the broken pile, led to some understanding of where the maximum moments occurred. The primary objective of this thesis is to use a finite difference program, LPILE, to develop moment diagrams for the prestressed concrete piles in the Phase II tests and to discuss the variability in the predicted results.

CHAPTER 2

Literature Review

2.1 University of Tennessee Project History

Beginning in 1996 and continuing through 1999, *Phase I* of a two-phased project that was conducted at The University of Tennessee and sponsored by TDOT was performed, a project that eventually led to *Phase II*, which began in 2000 and ended in 2003. The purpose of the project was to analyze the behavior of laterally loaded piles supporting integral bridge abutments. *Phase I* involved testing HP10 x 42 steel piles. *Phase II* was similar to that of *Phase I* in that the objective was the same, to analyze the adequacy of TDOT's current design criteria for integral bridge abutments with a particular focus on the pile-abutment interface. A major difference and the reason for two separate phases is that 14-inch, square, prestressed, precast concrete piles were tested in *Phase II*.

Phase I, as mentioned before, involved tests on HP 10 x 42 steel piles driven approximately 38 ft. on site. Three of the five piles tested were driven into virgin soil, and the other two piles were driven into compacted fill material. The fifth pile was embedded two feet into the abutment as opposed to the one-foot embedment for piles one through four. Rotational restraint, consistent with that in an integral bridge, was provided by a reaction beam. All the piles were instrumented with strain gages along their length. In return, moment was calculated from this measured strain. These field results

suggested “a strong case can be made that TDOT’s current design criteria are conservative and that horizontal displacements as large as 1.5 inches and even beyond can be reasonably tolerated” (1). In addition, the test data demonstrate significant increases in the system’s capacity when one extra foot is embedded into the abutment (2).

Phase II, where the test-setup is described in Chapter 3, involved tests on 14-inch, square, prestressed, precast concrete piles. Again, tests confirmed “a value of displacement of as much as 1.5 in. would appear to be reasonable” (3). In addition, integrated exposed strand extending into the abutment produced a higher capacity of the system (4).

2.2 Previous Research Work

With the technological field growing at an exponential pace due to computers and electronics, computer solutions are often used to mimic or reproduce field results. Although increasing computational power is available daily, soil variability is very difficult if not impossible to simulate in a computer program. Extensive work must be done and results established to produce the approximate soil spring stiffness, a p - y curve with units of force per length.

In the LPILE PLUS 3.0 User’s Manual, Reese (5) validated LPILE through previous field work by developing “the criteria for the response of soils under lateral loading that are used in the analyses.” In this case, the output accurately depicts the behavior of the piles in field tests. Alternatively, he provides quantitative soil profiles as design guidelines; the accuracy of this global soil classification is the topic of ongoing

research. In an extensive literature review, Meymand (6) cites that Gill's (7) results "suggest that universal p - y curves may be inappropriate for some cases."

In a paper written by Yang and Jeremić (8), they performed a static pushover analysis on single piles in a Finite Element Program (FEM) and compared their results to those yielded by methods used in practice (LPILE). In the pile simulation, they used a modulus equal to that of aluminum and a .429m square pile. In the finite element program, a von Mises material was used to model the clay, and a Drucker-Prager material is used to simulate the sand. The piles were modeled in uniform and layered soil conditions. The FEM's resulting bending moments were used to compute the pressure, p , by differentiating twice. When this pressure is graphed versus displacement, p - y curves are born. They compared these results to those internally generated by LPILE. The p - y curves that LPILE generated for the sands consistently had lower resistances at shallow depths. The same scenario held true for the clay profile, but the values were much lower at shallow depths. With this observation, they concluded, "a comparison with results from programme LPILE, used extensively in practice, show some discrepancies ultimate pressures in shallow soil layers" (8).

In an analysis to improve a pre-existing spreadsheet (CLM 2.0) quantifying pile behavior, Clarke and Duncan (9) checked LPILE and CLM2.0 results with measured field results for both single piles and pile groups, using the actual soil profiles and p - y multipliers. They studied three different single piles. The first pile was a closed-end steel pipe pile with an outside diameter of 10.75 in. and an inside diameter of 10 in. The second pile was a 10BP42 (same as what is now called an HP 10 x 42) pile, which was

“capped” to restrain rotation. Similar to the first pile, the third pile was a closed-end steel pipe pile with an outside diameter of 12.75 in. and an inside diameter of 12 in. They concluded from tests on the first pile, which was a “flagpole” analysis, that the measured deflections are about 30% smaller than the computed deflections, while the third pile analyzed resulted in deflections that were “in close agreement with field measurements.” In the capped analysis, pile two, the computed deflections were smaller than those measured in the field (9).

In a project performed by The University of Illinois at Urbana-Champaign and funded by the Illinois Department of Transportation, Ball (10) studied the effects of retrofitting a bridge in Illinois with micropiles. He modeled pile groups (in GROUP 5) with caps restraining rotation and single piles (in LPILE) without rotational restraint to check measured deflections on micropiles tested in the field. Using the estimated soil profiles, GROUP 5 and LPILE generated expected deflections. Three cases, with p - y curves (1) automatically generated, (2) specified, and (3) no modifiers, were considered in the group analysis. Although the literature is not very clear as to what methods were used in the single pile analysis, the conclusions reached were that the predicted displacements “the author had developed using LPILE modeling lined up fairly well with the experimental deflections” (10).

In a thesis presented for a Master of Science degree, Myers (11) checked LPILE results with the test results yielded in the aforementioned *Phase I* of the abutment project at The University of Tennessee. The unaltered p - y curves for stiff clay above the water table generated within LPILE produced shears and deflection outputs that disagreed with

experimental results. With anemic deflections and excessive shears, he concluded that the internal curves were too stiff and manipulated the p - y curves with the p - y modifiers. The p - y modifiers required to match the LPILE results to the field results ranged from 1.15 and 3.89, which led him to conclude that “the modification factors used to unstiffen the p - y curves were by no means consistent.” After the proper modification factors were instituted, the comparisons “indicate that LPILE does provide a reasonable, albeit approximate, method to design piles supporting integral abutments.” Ingram (2), who worked on the same project and produced a dissertation for a Ph.D., looked carefully at the work by Myers and extracted the same conclusions from the data. Ingram included a limited sensitivity analysis and added a known boundary condition, rotational stiffness, from which he concluded: “Provided with a relatively accurate rotational stiffness for the pile connection and proper soil data, LPILE can be used to predict reasonably accurate response for a given lateral displacement” (2).

CHAPTER 3

Test Set-up

Phase II of the research project conducted at The University of Tennessee and funded by TDOT involved the behavior of prestressed concrete piles. The piles were 14 inches square, approximately 6000 psi concrete, containing six 0.5-inch diameter, Grade 270, low relaxation 7-wire prestressing strands. The strands were spaced approximately 4.5 inches apart, had a minimum cover of 2.5 inches, and were aligned three to a side, thus creating a strong and weak axis. The piles were oriented such that testing in the north and south directions caused bending about the strong axis. The piles were 40 feet long and were driven approximately 36 feet into the virgin clay atop “Morgan Hill,” across Alcoa Highway from the University’s Agriculture Campus. In three piles, 1, 2 and 4, three feet of exposed strand and one foot of pile were cast into the abutment. The abutment was 2 feet thick, 10 feet wide transverse to loading, and 3 feet long parallel with loading as illustrated in Figure 1. Above the abutment, a slab 2.5 feet thick, 10 feet wide transverse to loading, and 14 feet long parallel to loading was cast monolithically with the abutment. The concrete was TDOT class A, 3000 psi minimum 28 day strength. On Pile 1, the resistance to rotation varied among the several tests. This thesis focuses on Piles 2 through 4 in which two W24x117 beams were bolted to the side opposite the direction of pull to give resistance to pile head rotation similar to the rotational stiffness of a bridge (12). Perpendicularly crossing the two W24x117 beams,

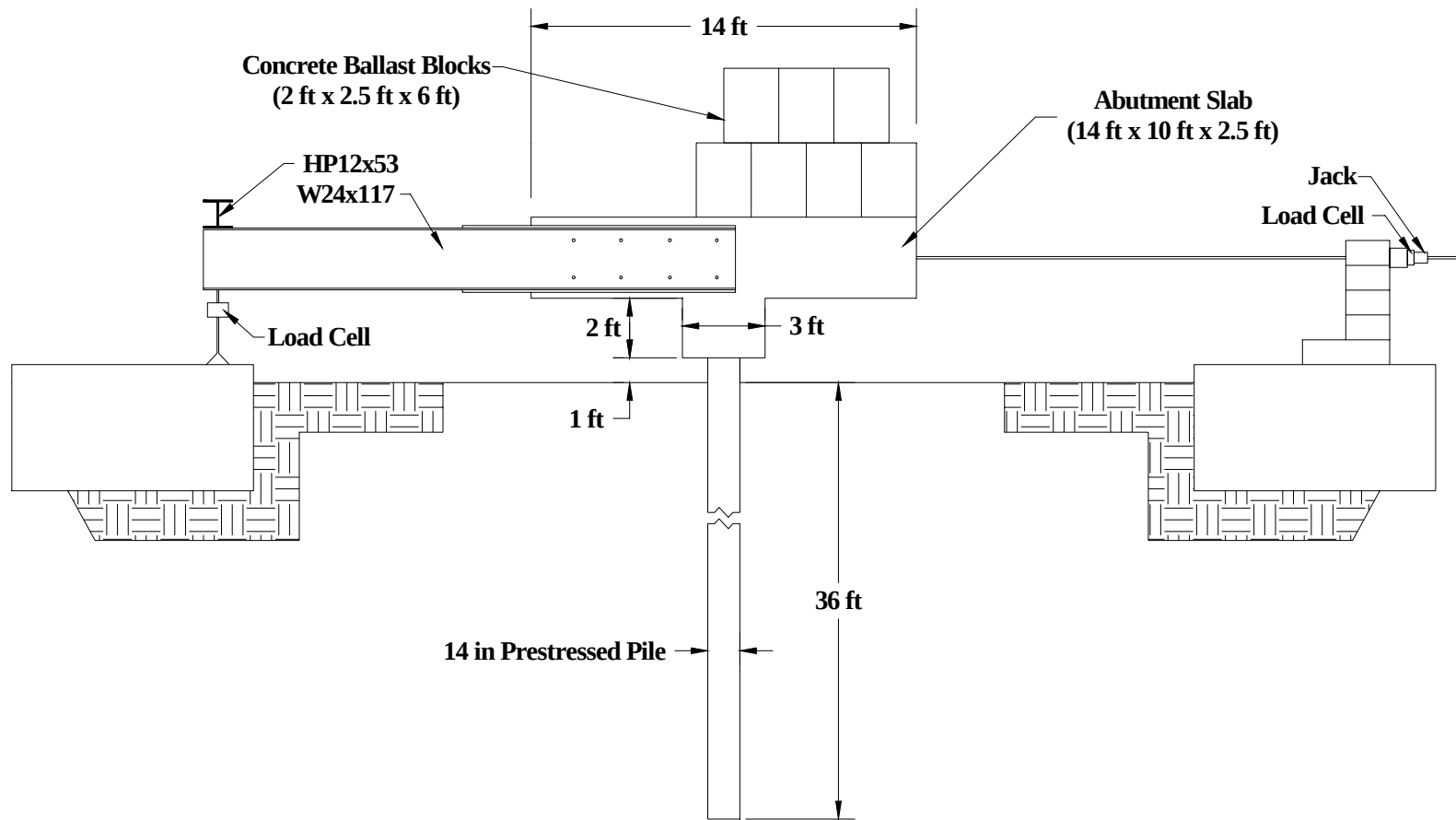


Figure 1: Test Setup for Phase II Tests

an HP12x53 lay clamped with a center marked for load cell placement to measure the reactive force. The addition of the beams caused an imbalance in moment. To correct this problem, seven 4,500-pound concrete ballast were placed on top of the slab in such a way that the static moment was balanced. The ballast blocks were also used to increase the axial load on the pile. To apply a horizontal force to the pile, a steel pipe was cast into the slab and a one-inch dywidag bar passed through the pipe and extended to the pull-frame, which was attached to a concrete “pulling pad” cast into the ground. Here, the lateral pull load was applied and measured. From the horizontal force with the vertical distance from the pile-abutment interface to the center of the dywidag bar and the vertical “hold-down force” with the horizontal distance from the center of the pile to the applied load, the moment was calculated at the pile-abutment interface. Foil strain gages measured strains in the pile from the ground elevation to the pile-abutment interface, and a mechanical gage (DMEC) was used on Piles 1 and 2. Several Linear Variable Differential Transformers (LVDTs) were placed to measure the vertical and horizontal displacements of the test set-up. Two LVDTs measured the horizontal displacement of the pile. One was placed immediately below the pile-abutment interface and was designated as the “Pile Top” LVDT. The other was placed approximately 1 foot below the “Pile Top” LVDT and designated as the “Ground” LVDT, which was the basis for all measured deflections.

CHAPTER 4

LPILE

Along with the introduction of computers, the demand for a method of computing the behavior of piles, a task impractical by hand which can only be accomplished by solving differential equations, increased. Thus, with intent to provide the engineering world with the capability to analyze and to design deep foundations by means of software, LPILE was created. LPILE was spawned by COM624P, which was a code written by the Federal Highway Administration (FHWA) and operated on a mainframe. LPILE was much more user friendly with graphical user inputs. As with most software's lineage, LPILE underwent improvements, and nine reproductions later, LPILE 4.0 was made available. This thesis investigates LPILE PLUS 3.0, the eighth replica of the first, as this version is available at The University of Tennessee.

Coupling a finite difference method on a beam-column and non-linear soil response, LPILE provides a means of analyzing laterally loaded piles subject to lateral load and moment. The program centers around constitutive p - y curves that dictate the behavior of a laterally loaded pile along the length of the pile. LPILE operates by internally generating approximate p - y curves from universal equations dependent upon basic soil information (soil classification, shear strength, internal friction angle, strain at one-half the maximum principal stress differential, and unit weight), or by user defined p - y curves. The nine different global equations within LPILE that are used to calculate

approximate p - y curves were derived from experimental results. In addition, p - y modifiers can be implemented to “tweak” the results. Next, the user defines the properties of the pile and known loading conditions at the pile top. As an associative link between the soil displacement and pile displacement, the p - y curves are used to generate the pile’s responses by means of integration and differentiation. For example, the p - y curves, which have units of force per length, are integrated along the length of the pile to generate a projected shear. After the analysis is finished, the user can view the responses such as deflection, moment, shear, and soil reaction versus the depth down the pile. For derivations, assumptions, and technical information, the author refers to the LPILE PLUS 3.0 Technical Manual (13).

CHAPTER 5

Application and Evaluation of LPILE

5.1 Introduction

Although many factors intricately contribute in varying degrees to a pile's behavior, known loading conditions, soil information, and pile materials and geometry are input into the LPILE program to approximate the behavior. In the test set-up shown in Figure 1, four loading conditions were known at the pile-abutment interface. The shear was equal to the pulling force and was measured by a load cell. Deflection was measured by LVDTs at the top of the pile. Moment was obtained two ways: (1) the external forces multiplied by the respective perpendicular distances from the center of the pile-abutment interface, and (2) verified by mechanical gages measuring strain, thus leading to moment. Rotational stiffness can be calculated by substituting the testing program's geometries along with the hold-down apparatus' material properties and geometries into an equation of a moment-rotation relationship. Don Lowe (12) calculated and reported these values in his thesis. Table 1 presents the slightly modified rotational stiffness values.

5.2 Soil Information

In order to investigate the sub-surface properties around the piles, University of Tennessee personnel drilled two boreholes in the general vicinity of the four piles. The

Table 1: List of Rotational Stiffness of Set-up

Pile, Direction	Rotational Stiffness
Pile 2 South	274000 k-ft/rad
Pile 2 North	274000 k-ft/rad
Pile 3 South	253000 k-ft/rad
Pile 3 North	269000 k-ft/rad
Pile 4 South	242000 k-ft/rad
Pile 4 North	259000 k-ft/rad

two holes were drilled to an approximate depth of 15 ft. while systematically retrieving 2.87 in. Shelby Tube samples along the depth of the hole. From each hole, five samples were extracted and taken to the laboratory for unconfined compression tests. Although samples were capped and taped to preserve the moisture content, the samples were promptly tested in the ensuing days to ensure accurate data. After each sample was removed from the Shelby Tube, it was sheared and shaved to an acceptable height vs. diameter ratio. Then the samples were properly prepared, and the unconfined compression tests were performed. The speed of an unconfined compression test is dictated by the machine, which has preset height vs. diameter dependent dials to control the rate of loading. Determination of the strain at 50% of ultimate strength, ε_{50} , and the ultimate strength, σ_u , were the objectives of the laboratory tests. The stress-strain curves are shown in Figures 2 and 3, and the ε_{50} values are shown in Table 2.

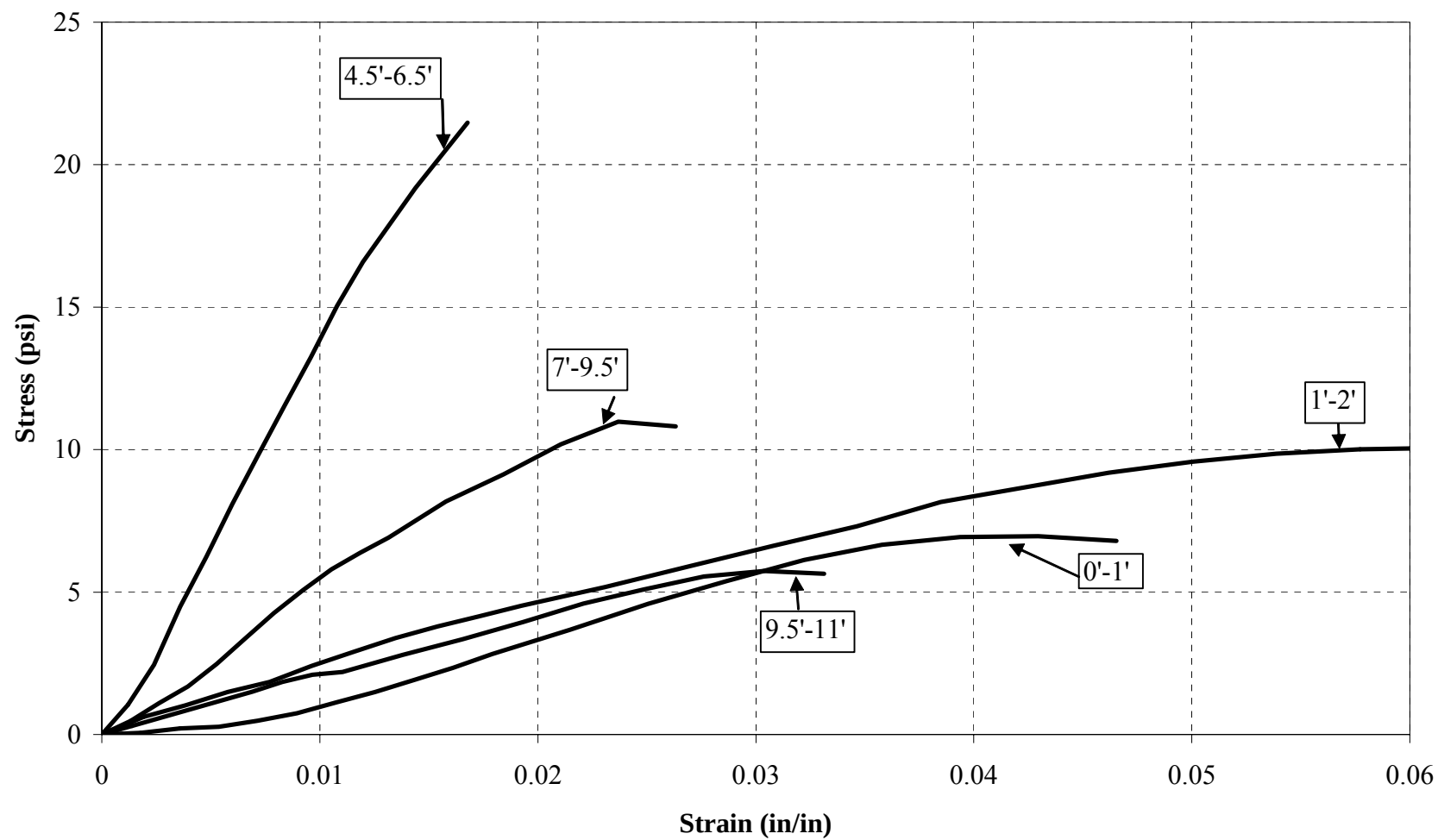


Figure 2: Bore Hole 1 Stress vs. Strain

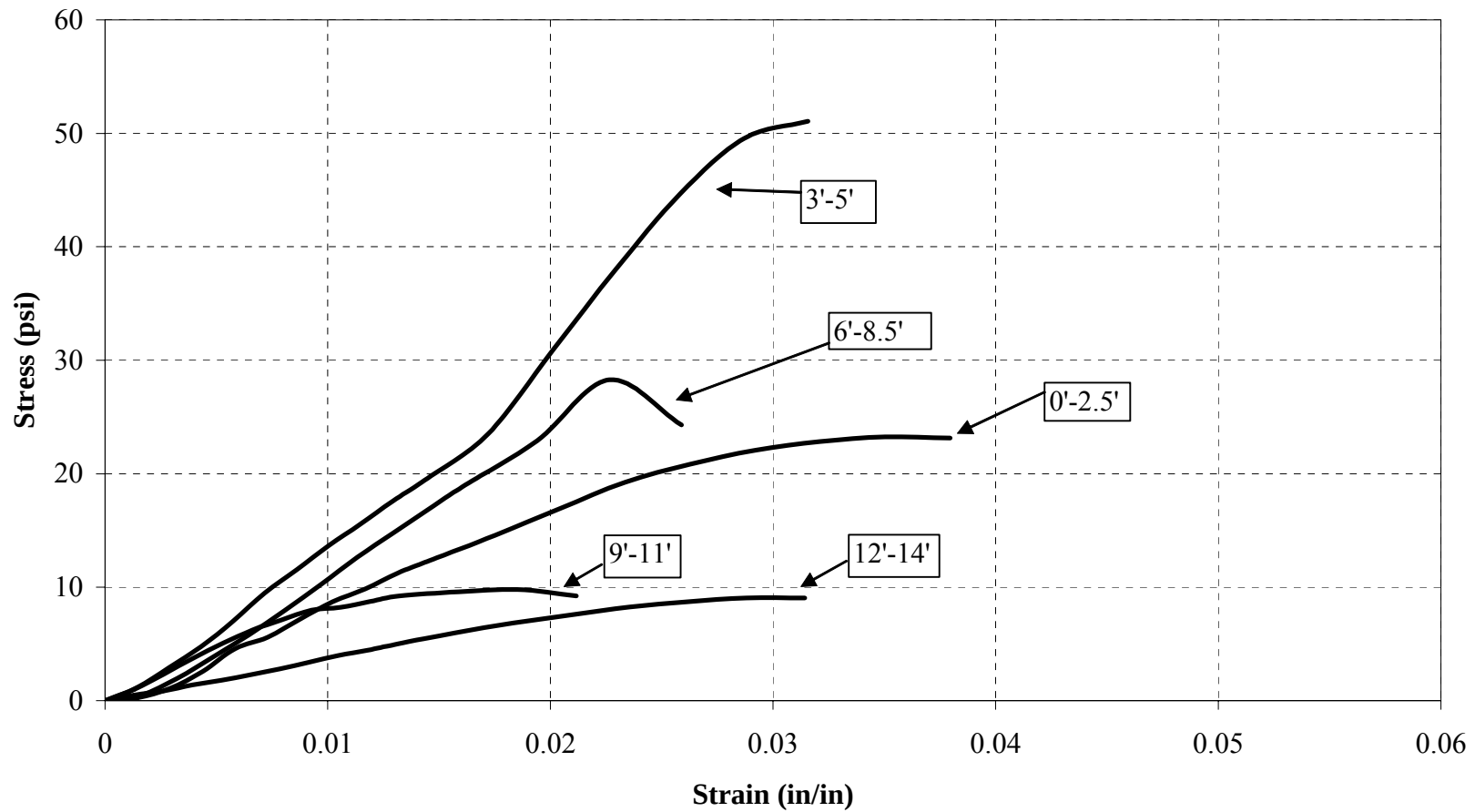


Figure 3: Bore Hole 2 Stress vs. Strain

Table 2: Values Retrieved From the Subsurface Investigation

	e_{50} (in./in.)	e_{50} (in./in.)
Depth (ft)	Borehole 1	Borehole 2
0-2	0.022	0.014
4.5-6.5	0.012	
3-5		0.018
7-9.5	0.01	
6-8.5		0.013
9.5-11	0.014	
9-11		0.005
12-14		0.012

5.3 Method

The loading conditions are entered into LPILE with four different possible combinations: (1) displacement and moment, (2) shear and moment, (3) shear and slope, and (4) shear and rotational stiffness. This thesis presents results from using three of the four loading conditions excluding the shear and slope combination. The most common test regimen used to test the piles included three ½-inch tests followed by three one-inch tests. Thus, a reasonable choice was to run an LPILE analysis on each pile at a ½-inch displacement and at a one-inch displacement in each direction. To avoid an attempt to model further damage to the pile directly related to subsequent testing repetitions, the author decided to analyze the test where the first objective displacement was reached. For example, in the ½-inch regime, the first ½-inch test would be evaluated, which in the field would be followed by two more ½-inch tests before starting the one-inch tests, again where the first one-inch test would be evaluated.

With the assumption that the soil samples were representative of the surrounding area, the ε_{50} and σ_u from the field data and the unit weight from the LPILE *User's Manual* (5) were held constant throughout all analyses in LPILE to demonstrate actual variability in the calculations of the pile's predicted behavior. Because Myers (11) reported that the program's output was insensitive to the soil's unit weight, the author decided to use the suggested values given in the manual. LPILE offers only two options to classify clay without freestanding water, stiff clay and soft clay. The assumption of stiff clay without free water was used to classify the soil throughout all the analyses, and p - y modifiers were not introduced so that any disparities among results could be attributed to calculations within LPILE. A sensitivity analysis was performed to pinpoint variability between the different assumptions of soil classification between stiff and soft clay because the ε_{50} matched closely with those Reese (14) suggests for medium to soft clays. The results of this sensitivity analysis are given in Figures A1- A12. First, a figure presents the results from assuming soft clay. In the subsequent figure, the results from assuming stiff clay are illustrated. The variations between the assumptions of soil classification were small enough to be neglected. All analyses were then made assuming stiff clay, and all data analyses were based on stiff clay. The internal friction angle, ϕ , for the clay was assumed zero. The soil data were entered into LPILE based on the data from the two boreholes and was assumed constant from 10 feet below the ground surface to the bottom tip of the pile.

Not only are many factors intricately interacting to affect the behavior of the

pile, but also introducing a cracked concrete dimension only further complicates the analytical process. In a pile removal investigation after a test to failure of Pile 2, Eichelman (4) reported crushed concrete at the pile-abutment interface and approximately five to six feet below the interface, which gave a large amount of insight as to where the maximum moments occurred. These values were used to define pile properties at various cross-sections. The pile, from the bottom of the abutment to the tip, was divided into 100 different sections using LPILE's graphical user interface for "Pile Properties." New values of moment of inertia (I) and area were calculated and entered for the cracked sections along with the prismatic moments of inertia for the uncracked lengths of pile. To begin with, the cracked sections below the ground surface were taken as approximately 12 inches beginning about 5.5 feet below the abutment and about four inches immediately below the abutment for the one-inch tests; in the half-inch tests, the piles were cracked approximately four inches in both areas with cracked sections. The width of the piles and the modulus of elasticity were assumed constant over the entire pile length. In addition, the axial load due to the prestressing force and dead load of the concrete slab and ballast was calculated to be approximately 100 kips for all tests.

Now that LPILE was ready to begin, a process needed to be established where LPILE's output could converge with field data by means of manipulating unknown quantities such as cracking lengths. This process was predicated upon a given boundary condition so that LPILE's calculated product could be manipulated to approach values from field work. This process is referred to as a "tweaking" stage.

For lack of a better term, this will refer to the process of modifying the cracking lengths needed to converge the LPILE values for each individual analysis with those recorded in the field. With pseudo-parity between field and predicted results, one would expect that the pile is accurately represented in LPILE; based on newly modified pile properties that led to converged shear results, the moment and displacement behaviors predicted from LPILE could be evaluated with respect to their tendencies to represent field results. The moment-displacement boundary condition was chosen to produce shear results and to define a platform in which shear results from LPILE would parallel field shear results. The shear values at the pile-abutment interface were then analyzed in comparison to the results obtained in the field tests. Then, with the other two boundary conditions, LPILE was evaluated with respect to its ability to produce accurate representations of moment and displacement values using the pile properties that led to convergence of the shear results, with focus and verification on the pile-abutment interface. Based on the input values that led to this convergence, LPILE was used to compare calculated results to field data.

5.4 Evaluation

Due to a multitude of contributing factors that affect pile behavior and to the variable effects of these factors, one would expect the results from LPILE to deviate somewhat from those measured in actual field work. The model predicted shear behaviors that ranged from 28% below a measured value to almost 49% above a

measured value and had approximately a 19.5% mean magnitude of deviation with a standard deviation of 15%. When LPILE overstated the shear magnitude, it did so at an average of 33%. Conversely, LPILE understated the shear magnitude at an average of 13%. Overall, the percentage difference was approximately 2.5% over the measured value with a standard deviation over 25%.

In an attempt to account for these large discrepancies in the initial results, a “tweaking” stage was implemented with the objective to match the model and field results. First, the cracked lengths were varied with independence from other models with a range of reasonable values approximately from four inches to 12 inches for each individual test. These cracked lengths were a bit smaller than the cracked lengths observed in the laboratory after the pile removal because of the fact that the deflection range of ½-inch to one-inch was small in comparison to the large deflections experienced in the test to failure, which exceeded 10 inches. Then, the moments of inertia (I) were changed to values calculated for the respective cracked concrete section. Since some models did not converge to reasonable tolerances, extreme cracking lengths with the associated inertias were implemented and examined. To match shear values as closely as possible for each test, the moments of inertia were varied widely, ranging from a fully cracked pile along its length to a fully prismatic member. Oddly, other than the fully cracked pile situation, the largest cracking length needed for the shear results to converge was 12 inches. Six LPILE models had fully prismatic inertias throughout the full length of the pile, and all but one under-predicted the shear. Three models had cracked section inertias

along the full length of the pile, and all over-predicted the shear. Three models used the reasonable cracking lengths that closely matched the cracking patterns in the field results; all were one-inch tests and matched shear very well. No correlation or pattern could be drawn to describe what factors affected the pile's shear behavior from these shear versus depth plots. No correlation could be seen as to how the field results were affected by the exposed strand length in relation to values predicted by LPILE. Table 3 shows the results of each field test and compares experimental shear results and shear results from LPILE's basic analysis based on a constant cracking lengths for all analytical models. The table also illustrates, as a percentage, the difference between the modeled value and the measured value with respect to the measured value. In other words, it shows the magnitude of deviation between the

Table 3: Field Results in Comparison to LPILE Shear Results

Pile, Direction	Field Pile-Top Deflection (inches)	Field Shear (kips)	Field Moment (kip-ft)	Basic Shear (LPILE) (kips)	Percent Error	Tweaked Shear (LPILE) (kips)	Percent Error
Pile 2, North	0.528	52.1	79	42.5	-18.4	46.8	-10.1
Pile 2, South	0.501	50.1	95.6	42.5	-15.2	46.2	-7.8
Pile 2, North	1.044	76.5	112.5	55	-28.1	64.2	-16.1
Pile 2, South	1.001	68.2	119.7	55.5	-18.6	64.2	-5.9
Pile 3, North	0.579	28	73.6	41	46.6	31.6	13
Pile 3, South	0.616	52.1	92.7	47	-9.7	51.3	-1.5
Pile 3, North	1.199	46.7	87	54	15.6	46.6	-0.2
Pile 3, South	1.205	58.1	94.1	55	-5.3	59.3	2.1
Pile 4, North	0.599	28.9	86.2	43	48.7	34.5	19.3
Pile 4, South	0.6	52.3	109.8	49	-6.3	53.3	1.9
Pile 4, North	1.186	47.7	120.2	58.5	22.7	47.8	0.3
Pile 4, South	1.206	61.3	132.2	60.5	-1.2	60.6	-1.1

model's value and the measured value; a negative sign means the modeled shear registered below the measured shear and vice versa for the positive sign. The "tweaked" shear values, with the independent cracking lengths for each analysis, at the interface in comparison to the field results are also shown in Table 3. Although the "tweaking" stage narrowed the percentages, the model predicted behaviors that ranged from 16% below a measured value to almost 19% above a measured value and had a 6.5% mean magnitude of deviation with a standard deviation over 6.5%. When LPILE over predicted the shear behavior, it did so at an average of nearly 11%. Conversely, LPILE under predicted the shear behavior at an average of 6%. Overall, the percentage was approximately 0.5% below the measured value with a standard deviation of 9.5%.

After the "tweaking" stage where LPILE predicted shears within an acceptable range, a boundary condition comparison was used to analyze how well LPILE predicted displacements and moments by using the new moments of inertia and cracking lengths that led to the merge of shear results. The procedure consisted of taking the three aforementioned loading combinations, moment and displacement, shear and rotational stiffness, and shear and moment, entering them into LPILE, running the analysis, producing graphs of the pile's behavior versus depth, and comparing the variability in the predicted results. Example plots are shown in Figures A13 through A42. In Table 4, the values LPILE predicted at the pile-abutment interface are shown along with and in comparison to the data obtained in the field. The moment results at the interface from the shear-moment boundary

Table 4: Field Results in Comparison to LPILE’s Post-“tweaked” Moment and Displacement Results

			Shear-Moment Boundary Condition		Shear-Rotational Stiffness Boundary Condition		Shear-Moment Boundary Condition		Shear-Rotational Stiffness Boundary Condition	
	Field Moment (k-ft)	Field Pile Top Deflection (in.)	Moment (k-ft)	Displacement (in.)	Moment (k-ft)	Displacement (in.)	Moment Percentage Error	Displacement Percentage Error	Moment Percentage Error	Displacement Percentage Error
Pile 2, North	79	0.528	-	0.72	167	0.28	-	36.4	111.0	-47.0
Pile 2, South	95.6	0.501	-	0.65	167	0.29	-	29.7	74.3	-42.1
Pile 2, North	112.5	1.044	-	1.65	279	0.6	-	58.0	148.1	-42.5
Pile 2, South	119.7	1.001	-	1.19	242	0.48	-	18.9	101.9	-52.0
Pile 3, North	73.6	0.579	-	0.64	167	0.28	-	10.5	126.4	-51.6
Pile 3, South	92.7	0.616	-	0.22	65	0.42	-	-64.3	-29.9	-31.8
Pile 3, North	87	1.199	-	1.14	208	0.36	-	-4.9	139.5	-70.0
Pile 3, South	94.1	1.205	-	1.21	170	0.26	-	0.4	80.7	-78.4
Pile 4, North	86.2	0.599	-	0.56	167	0.28	-	-6.5	93.3	-53.3
Pile 4, South	109.8	0.6	-	0.05	67	0.45	-	-91.7	-39.3	-25.0
Pile 4, North	120.2	1.186	-	1.24	229	0.4	-	4.6	90.7	-66.3
Pile 4, South	132.2	1.206	-	1.17	121	1.06	-	-3.0	-8.6	-12.1

condition were not recorded in the table because, in this case, they were known boundary conditions; thus, only the displacements were analyzed. Statistics illustrating variations between field results and predicted results are shown in Table 5. The statistics are calculated the same as before. The three graphs central to this analysis were moment vs. depth, shear vs. depth, and displacement vs. depth. From these three graphs, the two major focal points are the values at the pile-abutment interface, because these were the values measured in the field tests. Also the consistency in the shape of the three curves produced by the three combinations of boundary conditions, while difficult to quantify, may be evaluated by visual inspection of the graphs.

5.5 MATLAB

With the incongruence of LPILE's forecasted results for specific tests, the author wrote a MATLAB program to evaluate these results, preliminary and post-“tweaking,” on a macro scale. MATLAB is an elegant computer language for technical computing and modeling. It conglomerates pre-written programs that allow the user to focus on the problem at hand and not on computational programming details required by basic languages. Thus, using these preexisting programs, the user saves personal and computational time.

To accomplish this evaluation, high and low boundaries had to be selected for shear and moment. As noted in section 5.4, this thesis “tweaked” only shear values to align as closely to the field measurements as possible. Because of potential future

Table 5: Statistics of Variations in LPILE’s Post-“tweaked” Predicted Values

	Shear-Moment Boundary Condition		Shear-Rotational Stiffness Boundary Condition	
	Moment	Displacement	Moment	Displacement
Range of Percentages	-	-92.0% to +58.0%	-39.0% to +148%	-78.5% to -12%
Mean Magnitude of Deviation	-	27.5%	87.0%	47.5%
Standard Deviation of Magnitudes	-	29.5%	43.5%	19.0%
Mean of Under-stated Values	-	-34.0%	-26.0%	-47.5%
Mean of Over-stated Values	-	22.5%	96.0%	-
Overall mean	-	-1.0%	74.0%	-47.5%
Overall Standard Deviation	-	41.0%	64.5%	19.0%

expansion, this program is two-fold. It allows the user to analyze the maximum and minimum shears and moments over any specified array of displacements between zero and two inches, and it has the capabilities to plot LPILE’s predictions in comparison to a larger picture rather than one specific test. Although this thesis focuses on ½-inch and one-inch displacements in LPILE, this MATLAB program uses mathematical formulae to reproduce maximum and minimum behaviors for displacements from zero inches to two inches. After the generic aforementioned testing regime was completed on the pile, subsequent tests ensued to record pile behavior at larger displacements. These tests ranged from 1.5 inches to 4.0 inches excluding the test to failure that exceeded 10 inches. With the shear and moment versus displacement curves at the pile-abutment interface readily available from the field tests, the maximum and minimum recorded shears, moments, and slopes were selected from zero to one-inch and from one to two-inch displacements. For example, from the zero to one-inch displacement, the maximum

shear coinciding with the most precipitous slope of the shear diagram and the minimum shear corresponding with the smallest slope were selected to offer a range of shears at a given displacement. Each line was then expressed mathematically using linear regression. The process was repeated for the displacements of one to two inches; however, maximum values of shear plateaued between one and two inches. These mathematical formulae, which are input into MATLAB, describe the maximum recorded behavior of the pile over a spectrum of displacements.

The program begins by prompting the user to decide if the LPILE results are of interest. Depending on whether the user selects “Yes” or “No,” the program decides whether to plot the LPILE data. Next, the user inputs a beginning and ending displacement and the number of increments to divide this specified displacement array. The program proceeds to graph only the requested information and produce a summary table. The final graphs are presented in Figures 4 and 5. Figure 4 is a graph of shear ranges for displacements ranging from zero to two inches. LPILE shear results from preliminary and post-“tweaking” analyses are also graphed in Figure 4 to illustrate that the shear results may not have paralleled one single test, but they fell in between the ranges experienced in the field. Figure 5 graphs the same variables as in Figure 4 but using moment. Figure 5 paints quite a different picture than Figure 4. In Figure 4, LPILE forecasts shears within an acceptable range, but Figure 5 shows that moments predicted by LPILE exceeded any moment experienced in the field. A code and “walk-through” of the program are presented in Appendix B.

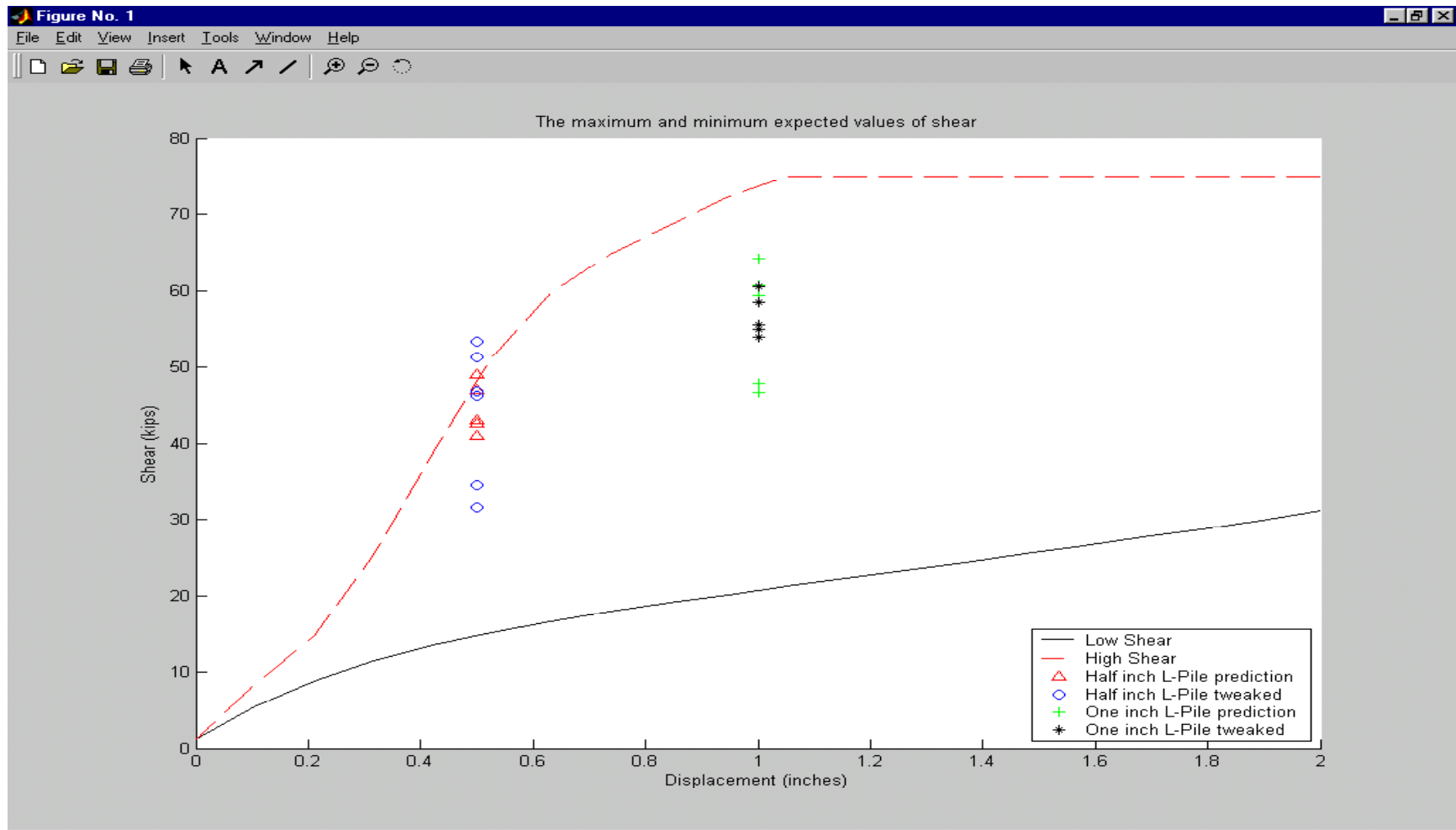


Figure 4: Plot of High and Low Shear versus Displacement Boundaries with LPILE Results Graphed

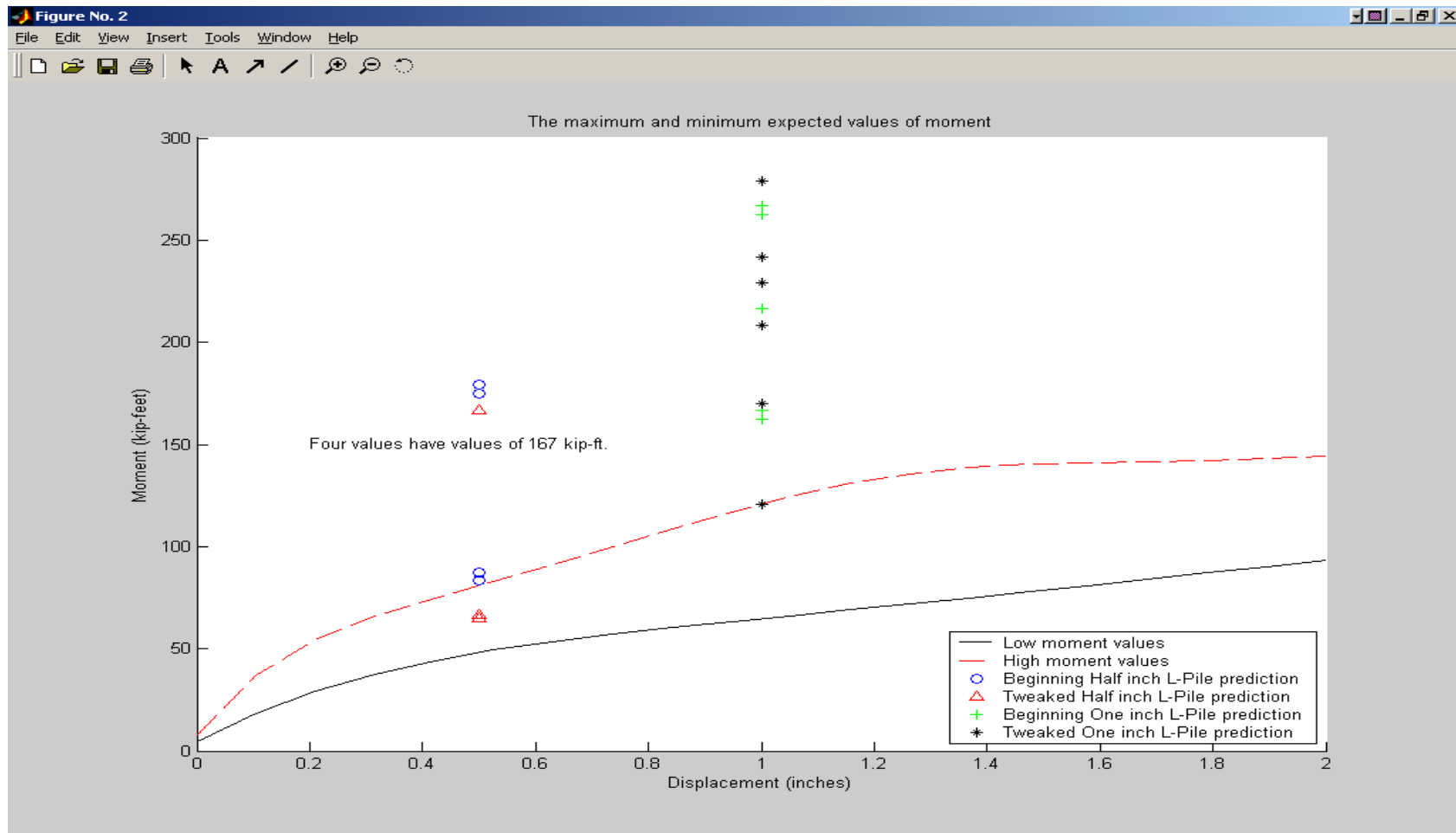


Figure 5: Plot of High and Low Moment versus Displacement Boundaries with LPILE Results Graphed

5.6 Summary and Discussion

After the required information was gathered and input into LPILE, the moment-displacement boundary condition, which forecasts shear behavior, was selected as a base to compare predicted and field results. These preliminary shear values were evaluated. It was decided that the values did not match their respective test values; thus, a “tweaking” stage was implemented to narrow the gap between field and computer results. Once the “tweaking” stage was completed through using the modified pile properties, a comparison was made of how results within LPILE differed given different boundary conditions. The results and statistics are summarized, and a visual inspection of the graphs illustrates the inconsistency of LPILE’s ability to produce simulated field data. A MATLAB program was written to evaluate LPILE on a macro scale. The program accounts for forecasted shears and moments, preliminary and post-“tweaking,” and plots these LPILE values on a graph where a range of maximum and minimum values for a given displacement is defined from field data.

In addition to the predicted shear and moment results, the displacements can be predicted from two input boundary conditions, thus, two opportunities to accurately forecast field displacement. A visual inspection of a deflection versus depth graph, as illustrated in the figures in Appendix A, reveals that deflections are predicted both high and low with no predictable pattern beside the fact that the shear-rotational stiffness boundary condition usually predicts low and the shear-moment boundary condition usually predicts high. A conclusion can be drawn that the margins are large and the results are polarized, specific to which boundary condition is specified, and LPILE is not accurate or precise at predicting displacements very well.

From the three combinations of the known boundary conditions, three curves are produced, and other than magnitude, the shapes of the curves are consistent among the boundary conditions. Consistency is evaluated based on placement of the maximum respective values below the ground surface and of the inflection points. Given a defined placement and length of cracked concrete in the pile, the placement of the maximum value is simple to predict, but for those piles that have constant moments of inertia, LPILE is consistent in placing the maximum values and the inflection points.

As for predicting values as a whole, LPILE cannot seem to converge to predicted results at the pile-abutment interface. In LPILE, boundary conditions dictate the response of the soil, and with four known boundary conditions and three combinations for a single test, LPILE is not very accurate or precise in predicting actual behavior at the pile-abutment interface. On top of that fact, the evaluation was directed toward the prediction of results that were consistent with field data rather than an evaluation where no data existed; i.e. a designer must start from “scratch”. A designer does not have the opportunity to modify the input variables and is required to make an educated guess as to how a particular pile will react. Then, those assumptions are implemented into the LPILE model. Without a proper approximation on cracking lengths and the placement of those cracked sections, LPILE could inaccurately and imprecisely predict the behavior.

Errors in the prediction of the soil’s behavior could be attributed to the fact that concrete piles do not need an appreciable deflection before leaving the elastic range, which most piles do in their lifetime leading to irrecoverable displacements. In the finite difference equations, errors are introduced with changes in bending stiffness, which reinforce fundamental discrepancies in the applicability of LPILE in designing concrete piles.

CHAPTER 6

Conclusions

With the advent of the computer, computer programs that simulate reality are in high demand and under scrutiny. This thesis presents an analysis of a computer program, LPILE, that attempts to simulate the response of a pile to lateral loading. In previous literature, authors conclude that LPILE generally represents field results “fairly well.” Soil reactions predicted by LPILE, excluding p - y modifiers, are shown to differ from finite element modeling or field results at shallow depths, which form the most important zone in a pile’s behavior. The fact that p - y multipliers vary widely in previous literature indicates differing soil reactions.

Data were entered into LPILE, results evaluated, and then attuned as closely as possible to the field results, and statistics show that the values converge within an acceptable range of plus/minus 20%. Prior to the “tweaking” stage, the preliminary shear results clarify the ability of LPILE to accurately predict shear within a maximum-minimum range of shear defined from field results, which is illustrated in Figure 4. Therefore, LPILE predicts shear results with moment-displacement boundary condition fairly well.

Although the shear results converge to tolerable values, the moment results led to different conclusions. An examination of the preliminary and post-“tweaking” moment results in the macro-evaluation graphically establishes that they extend far above the

ceiling of the range defined by field results, which signifies stiff p - y curve approximations. These values are reflected in the statistics on the final moment values. Therefore, a conclusion can be drawn to buttress previous research in that LPILE predicts stiff soil reactions, which in turn predict high, yet conservative, moments.

The “bottom line” is that LPILE gives, at best, results which are highly approximate. Based on the evaluation of predicted versus measured behavior presented herein, the results from LPILE should be considered a crude approximation of what may be the actual behavior of a pile.

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APPENDICES

APPENDIX A:
SAMPLE GRAPHS

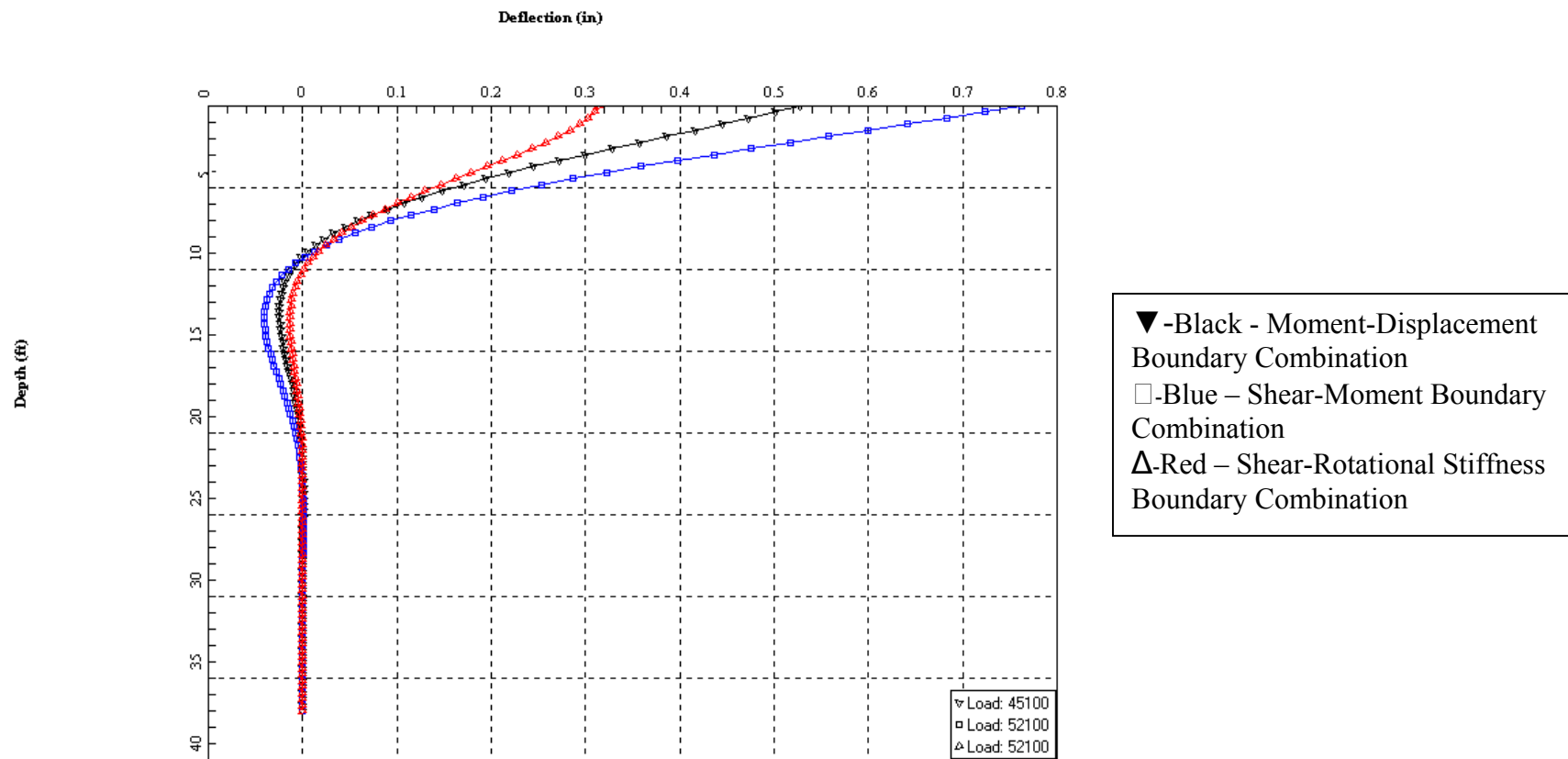


Figure A1: Deflection vs. Depth Pile 2 Pulling North Assuming Soft Clay, ½ inch

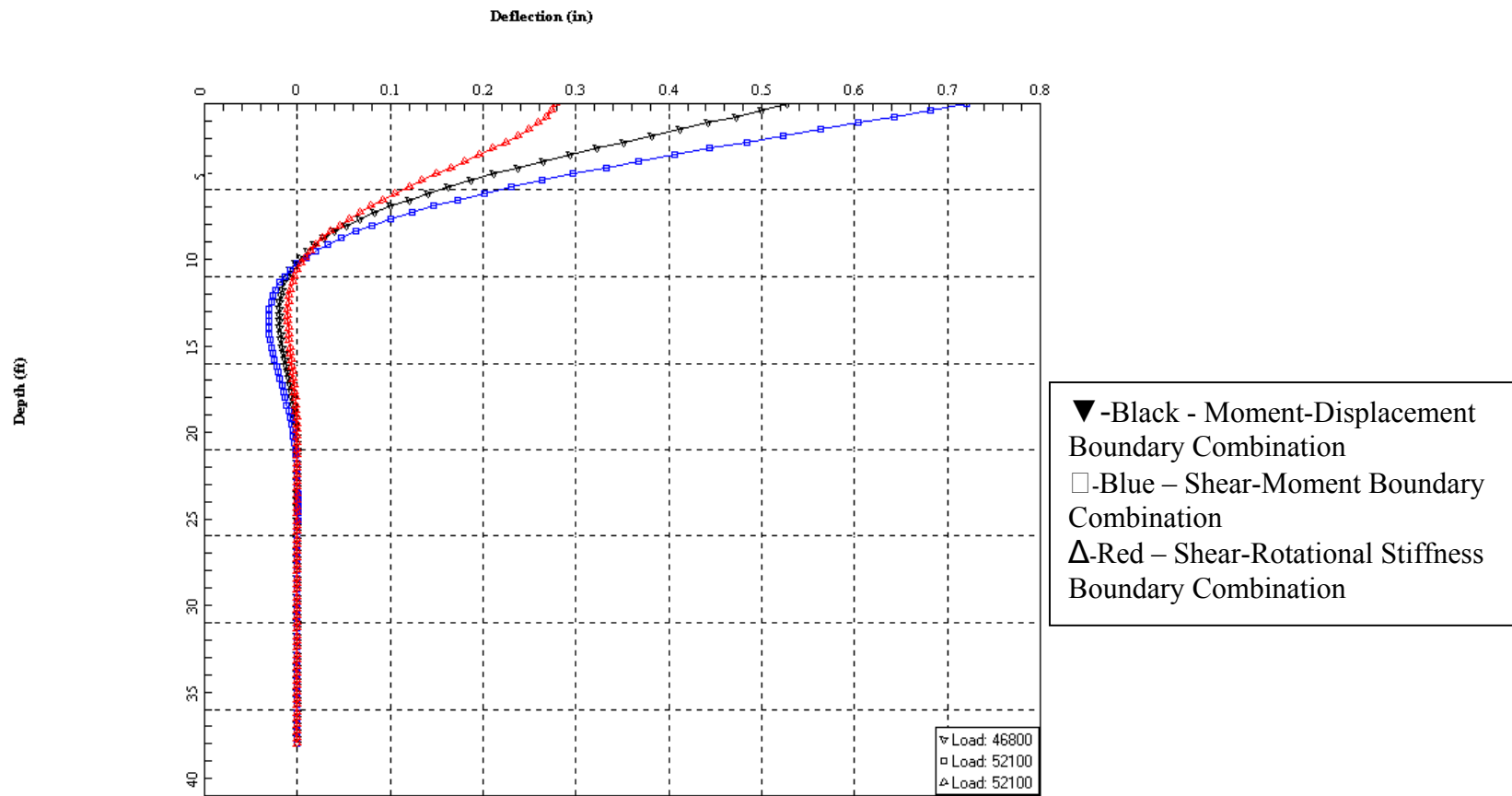


Figure A2: Deflection vs. Depth Pile 2 Pulling North Assuming Stiff Clay, ½ inch

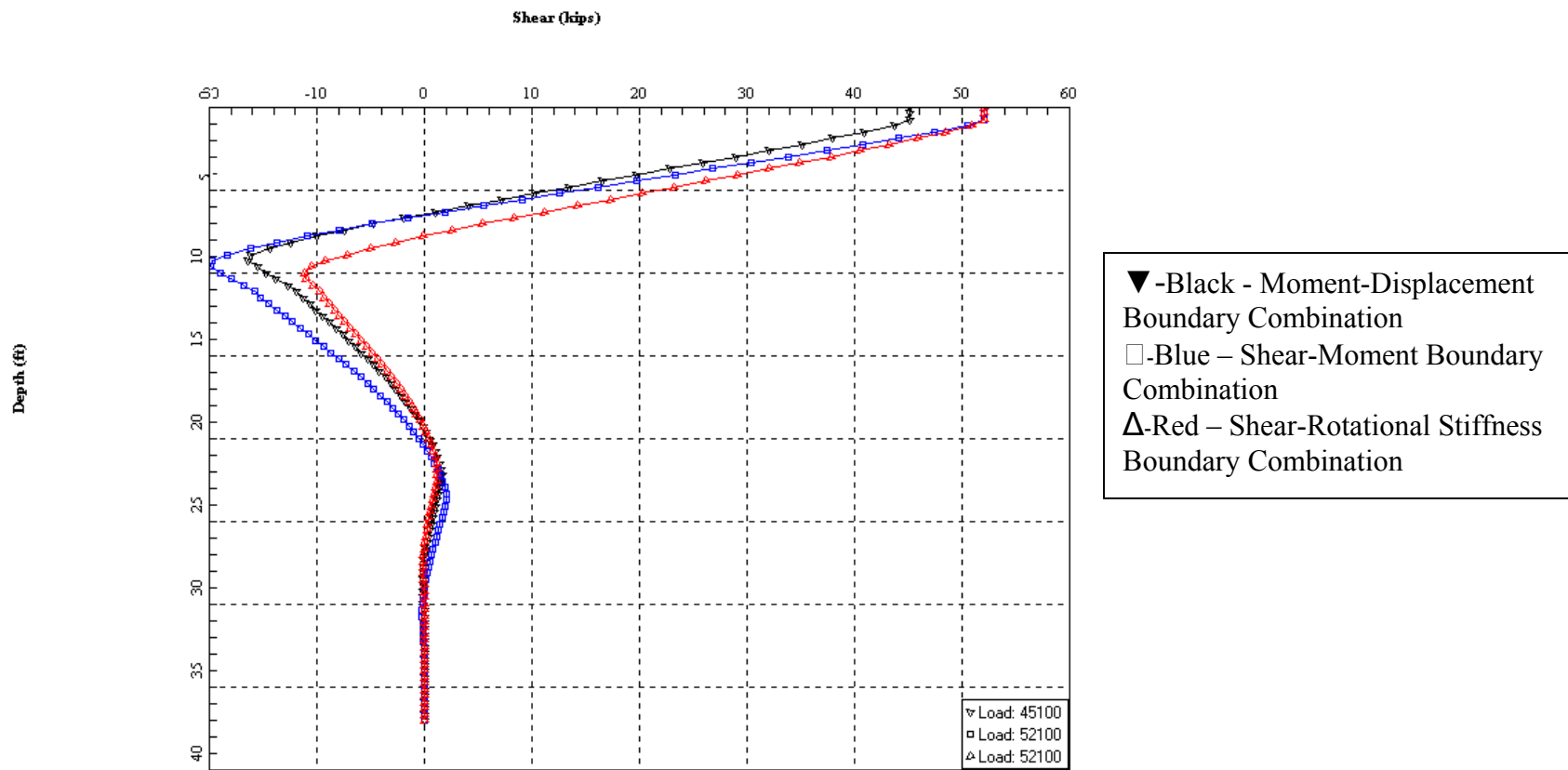


Figure A3: Shear vs. Depth Pile 2 Pulling North Assuming Soft Clay, ½ inch

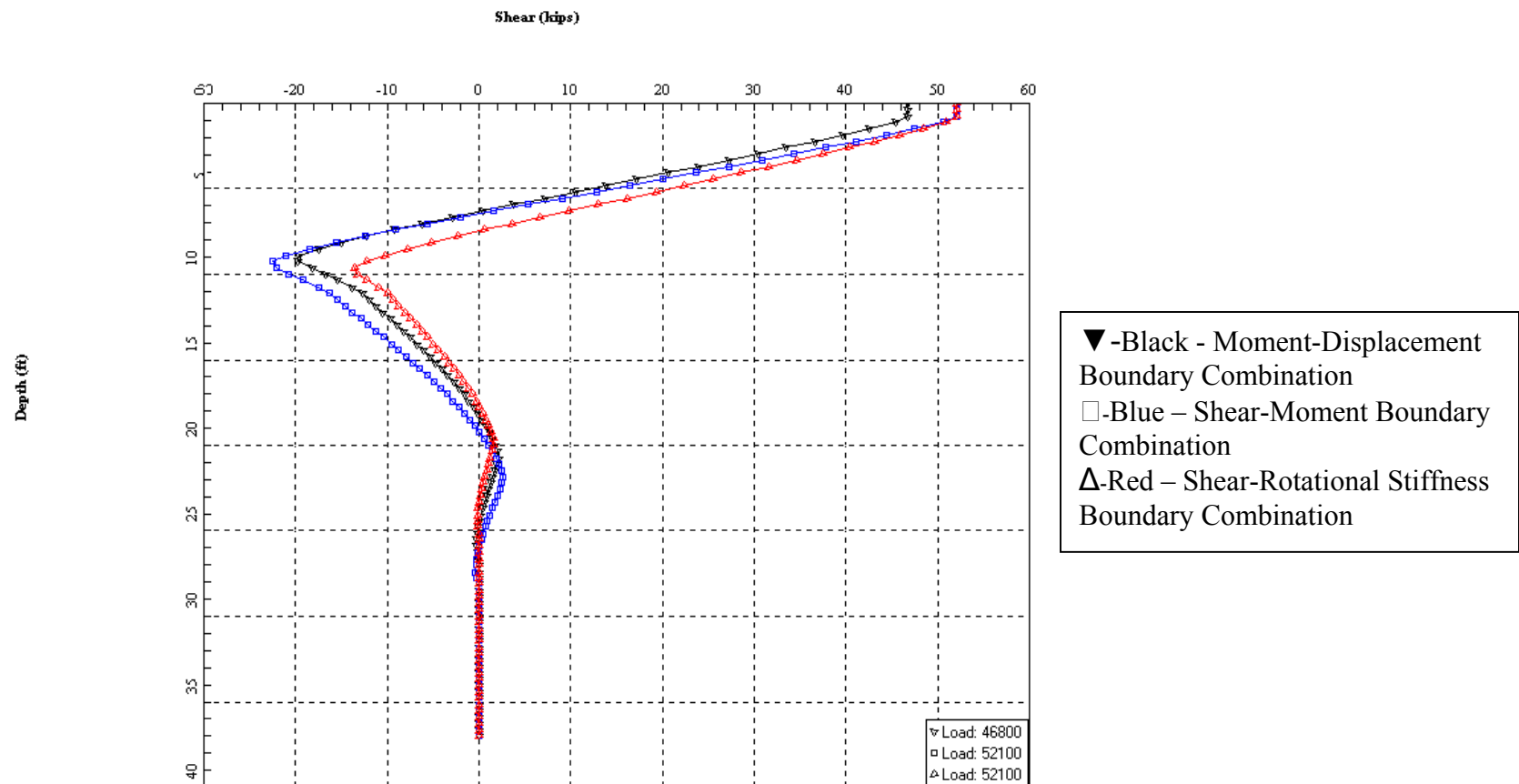


Figure A4: Shear vs. Depth Pile 2 Pulling North Assuming Stiff Clay, ½ inch

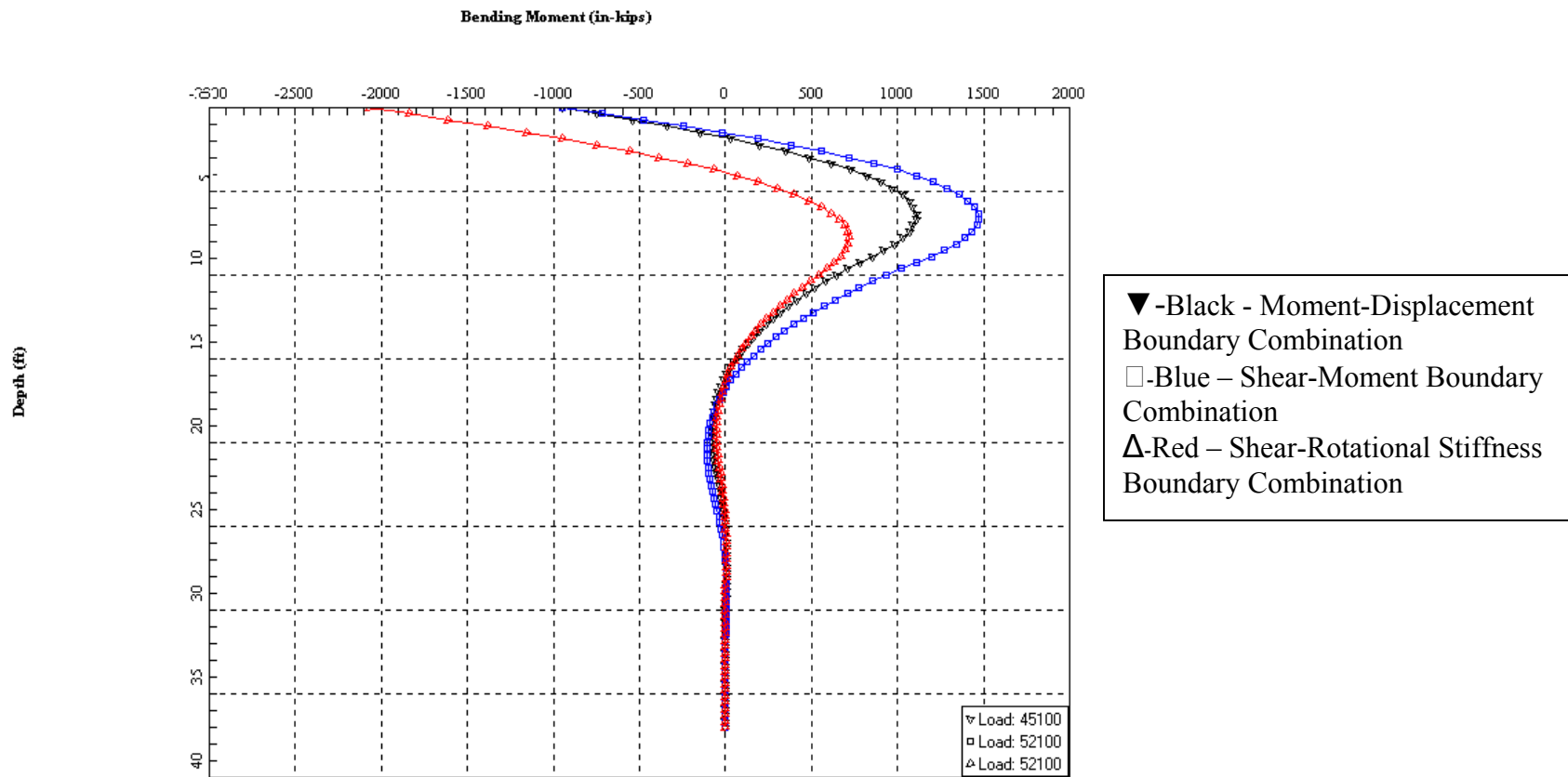


Figure A5: Moment vs. Depth Pile 2 Pulling North Assuming Soft Clay, ½ inch

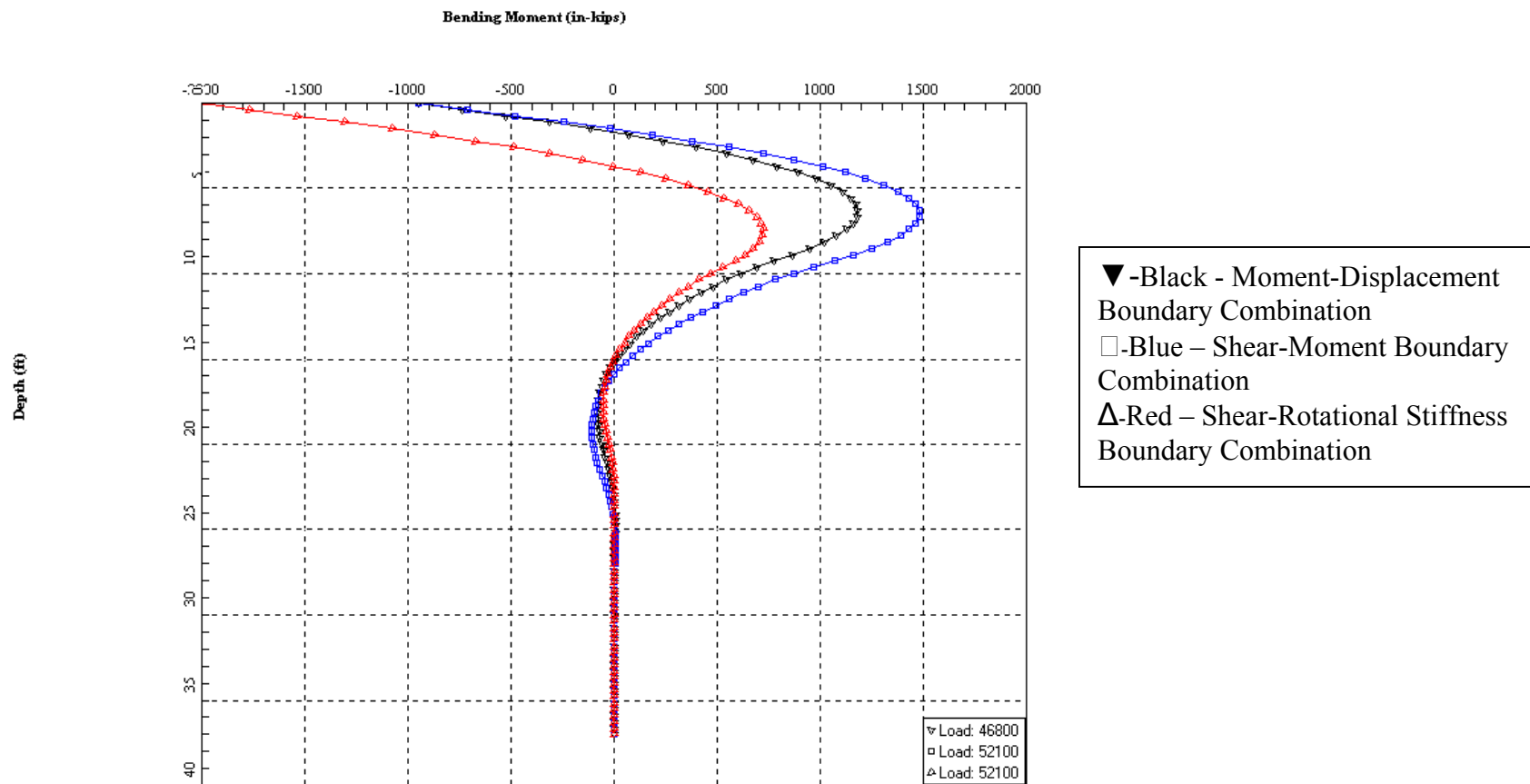


Figure A6: Moment vs. Depth Pile 2 Pulling North Assuming Stiff Clay, ½ inch

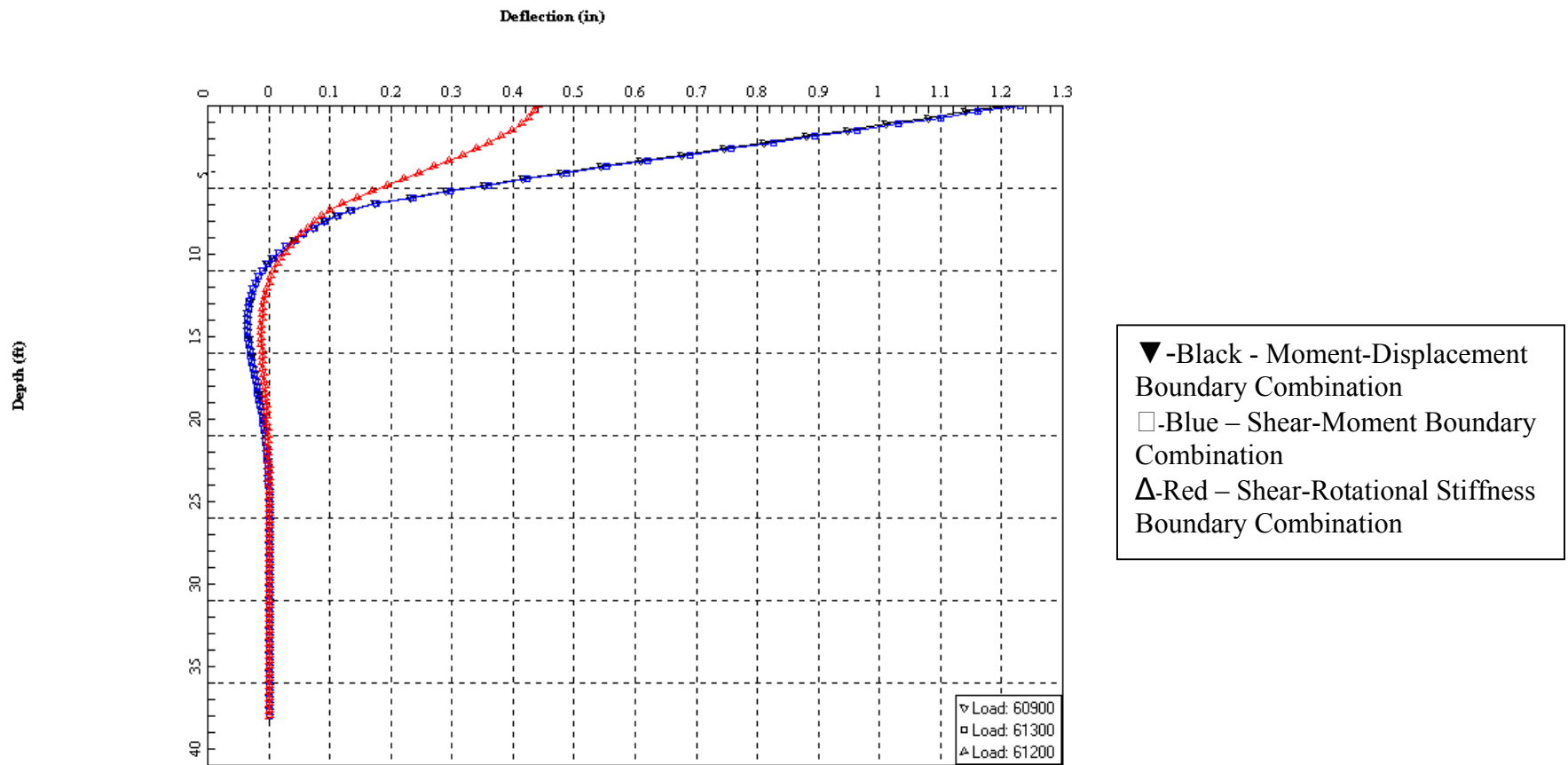


Figure A7: Deflection vs. Depth Pile 4 Pulling North Assuming Soft Clay, 1 inch

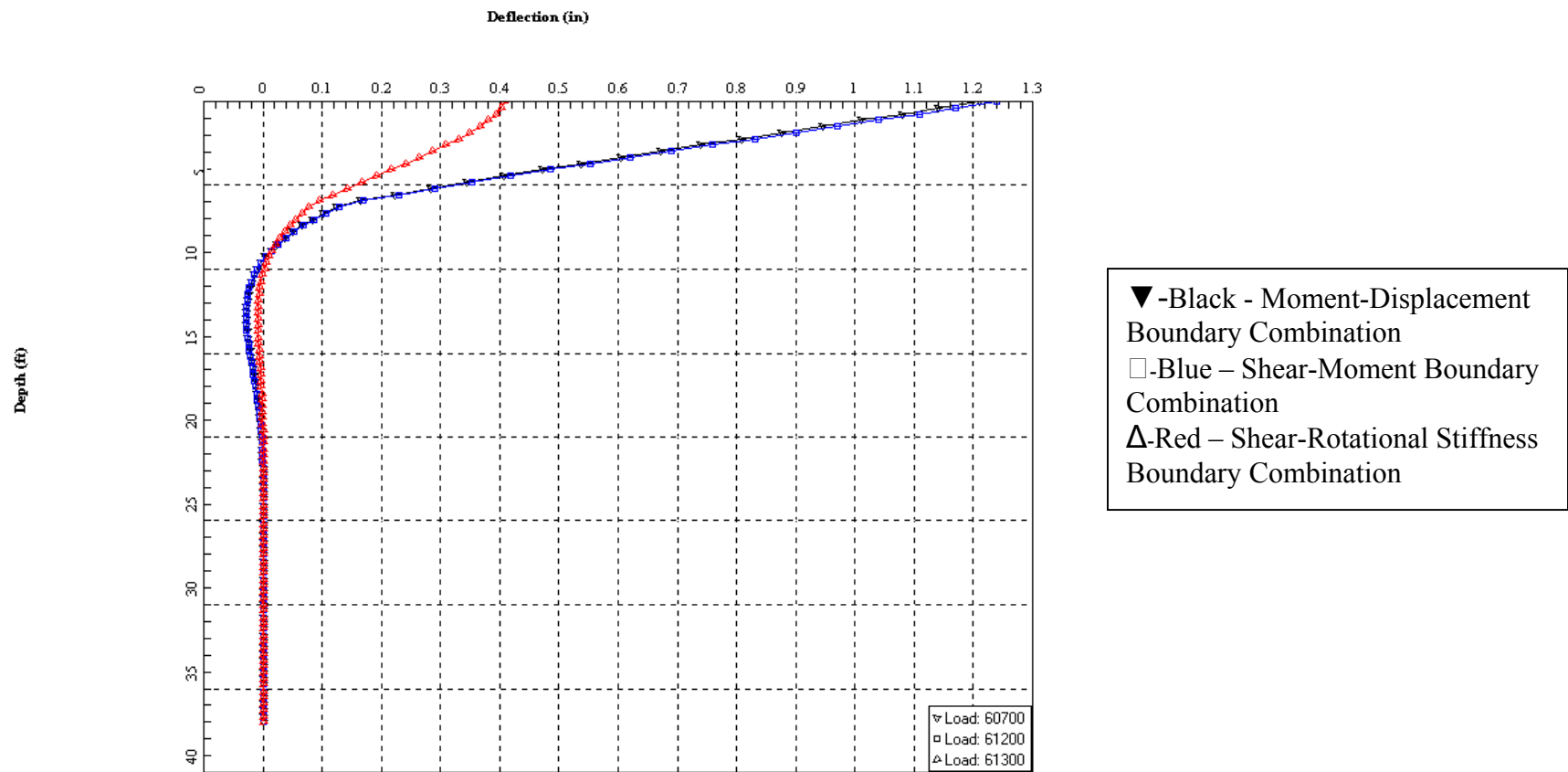


Figure A8: Deflection vs. Depth Pile 4 Pulling North Assuming Stiff Clay, 1 inch

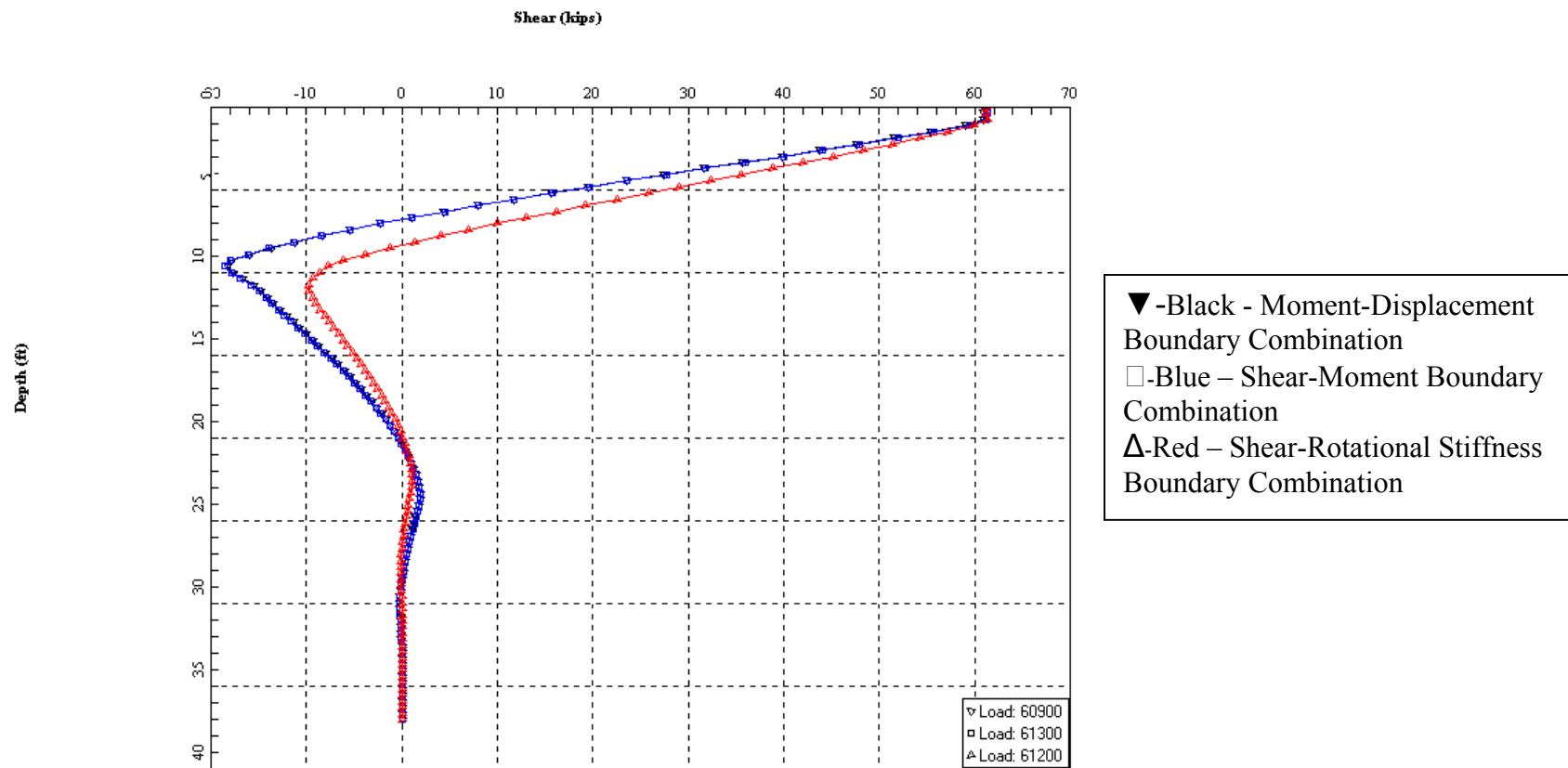


Figure A9: Shear vs. Depth Pile 4 Pulling North Assuming Soft Clay, 1 inch

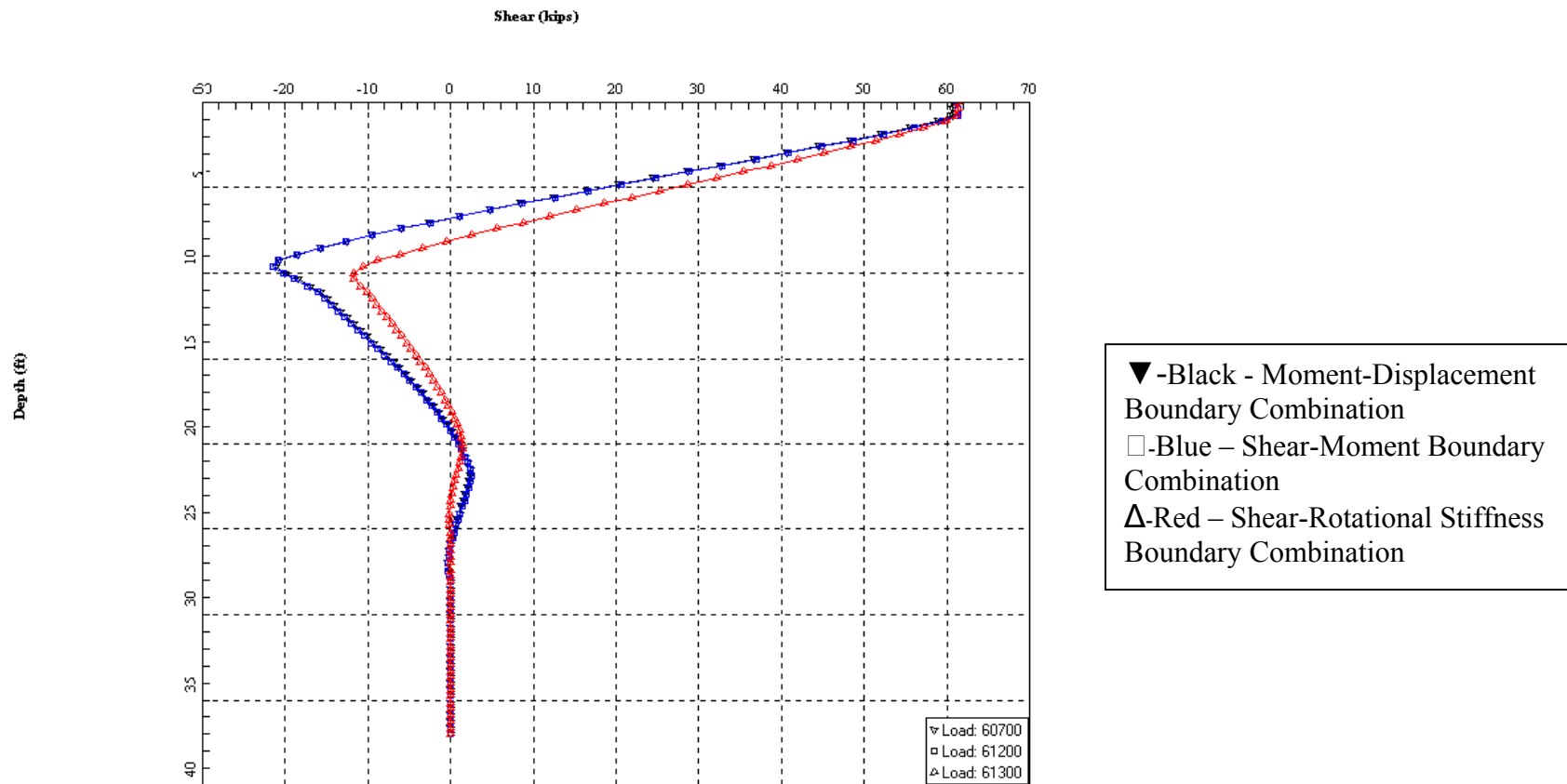


Figure A10: Shear vs. Depth Pile 4 Pulling North Assuming Stiff Clay, 1 inch

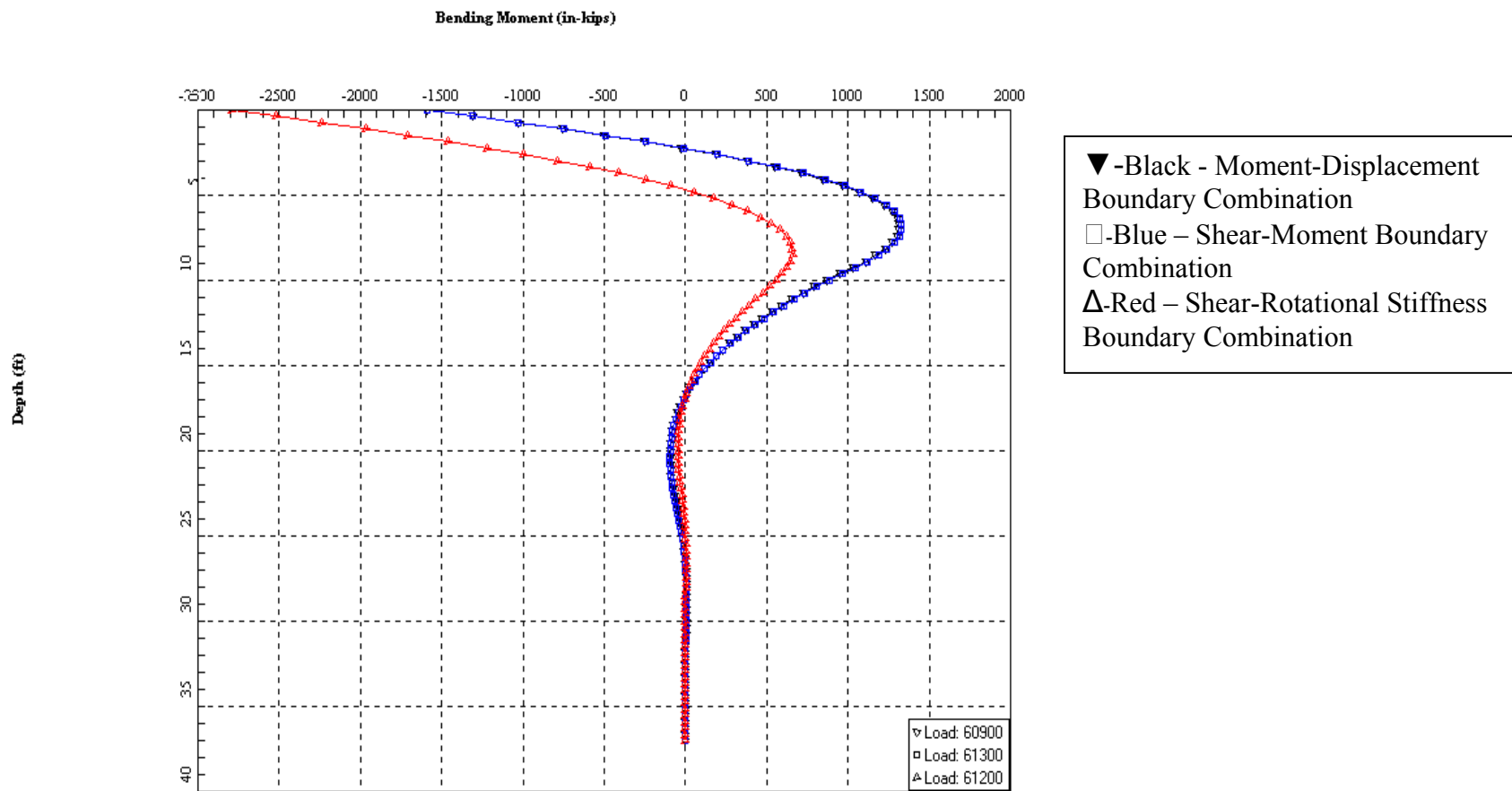


Figure A11: Moment vs. Depth Pile 4 Pulling North Assuming Soft Clay, 1 inch

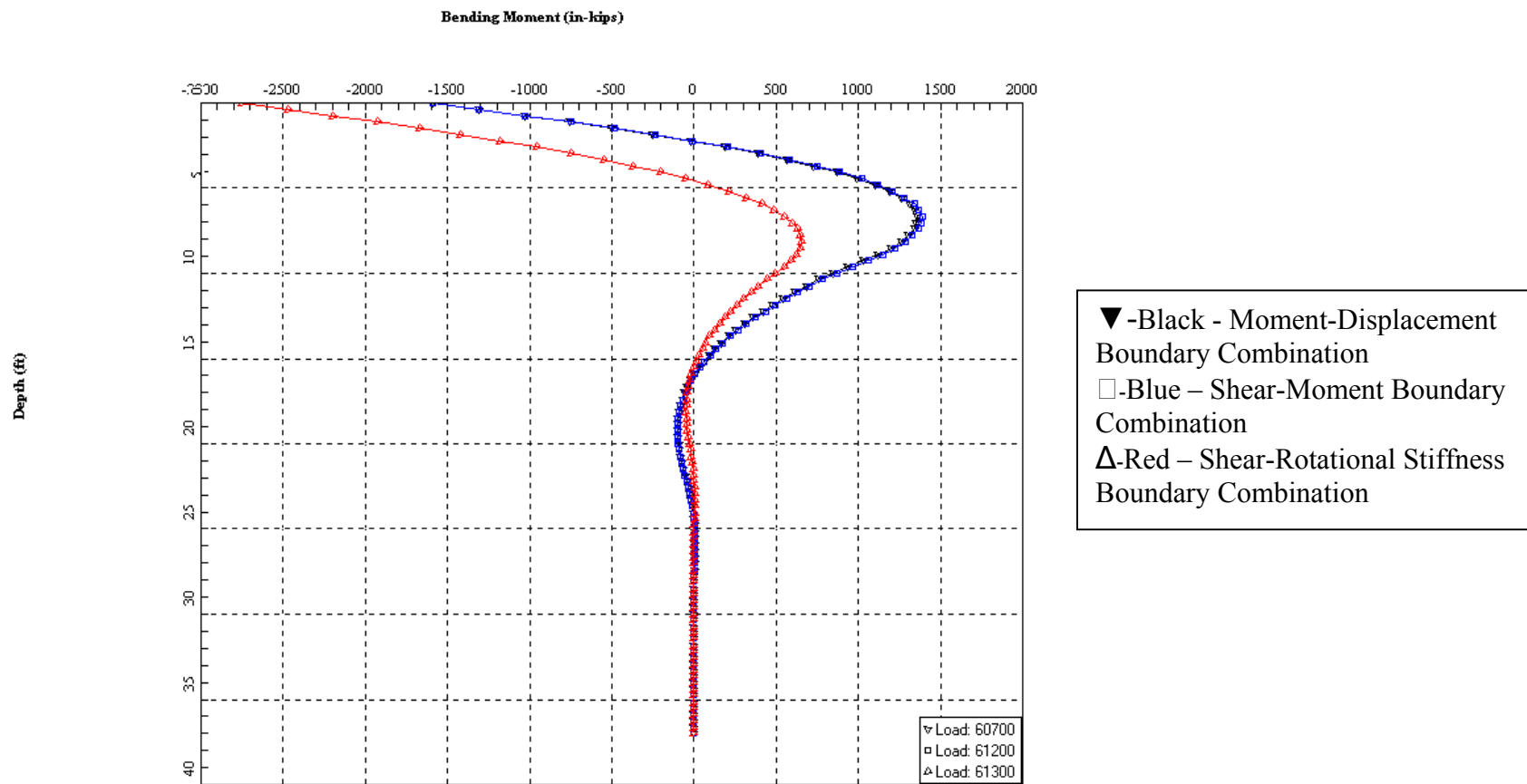


Figure A12: Moment vs. Depth Pile 4 Pulling North Assuming Stiff Clay, 1 inch

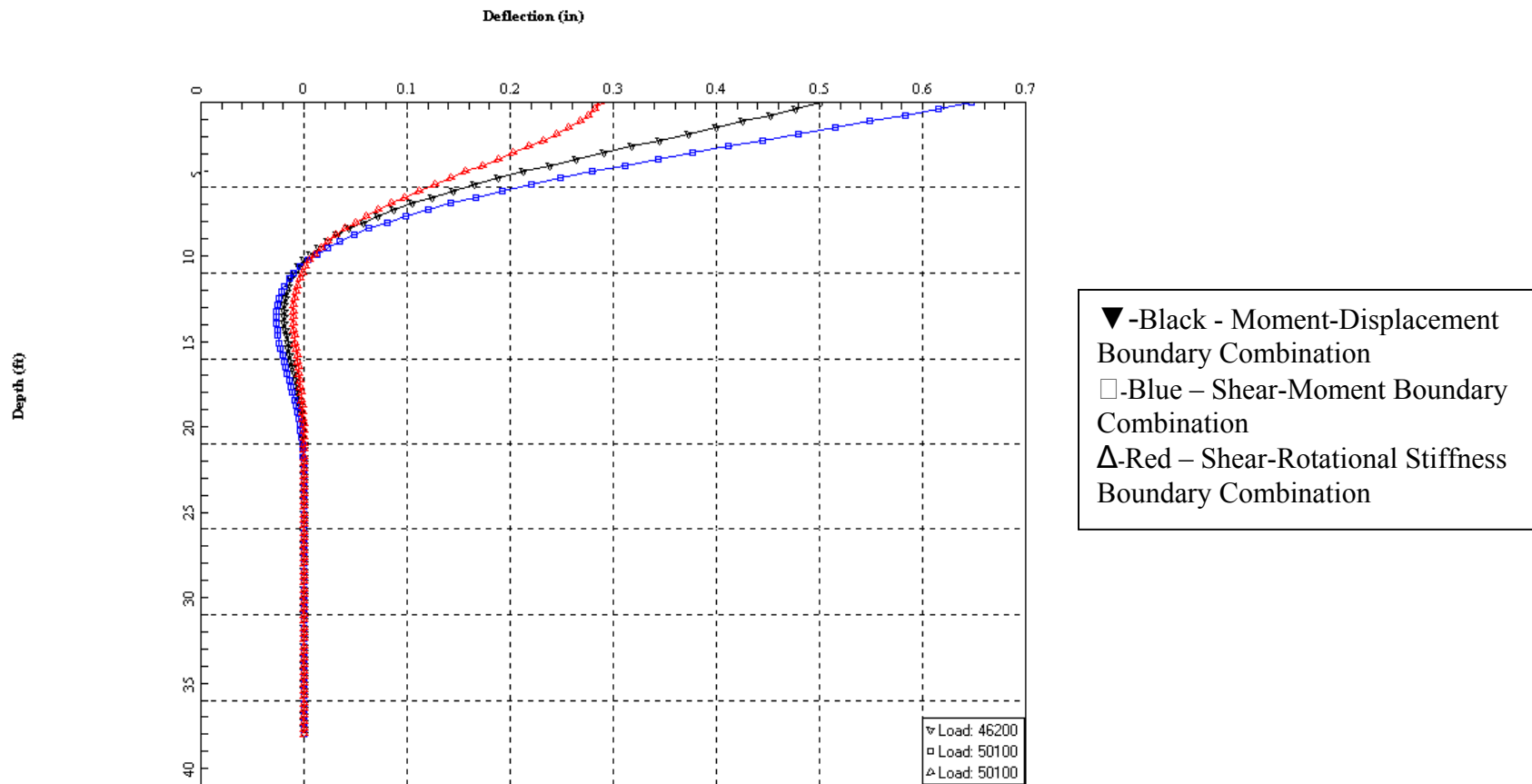


**Figure A13: Deflection vs. Depth Pile 2 Pulling South
Assuming Stiff Clay, ½ inch**

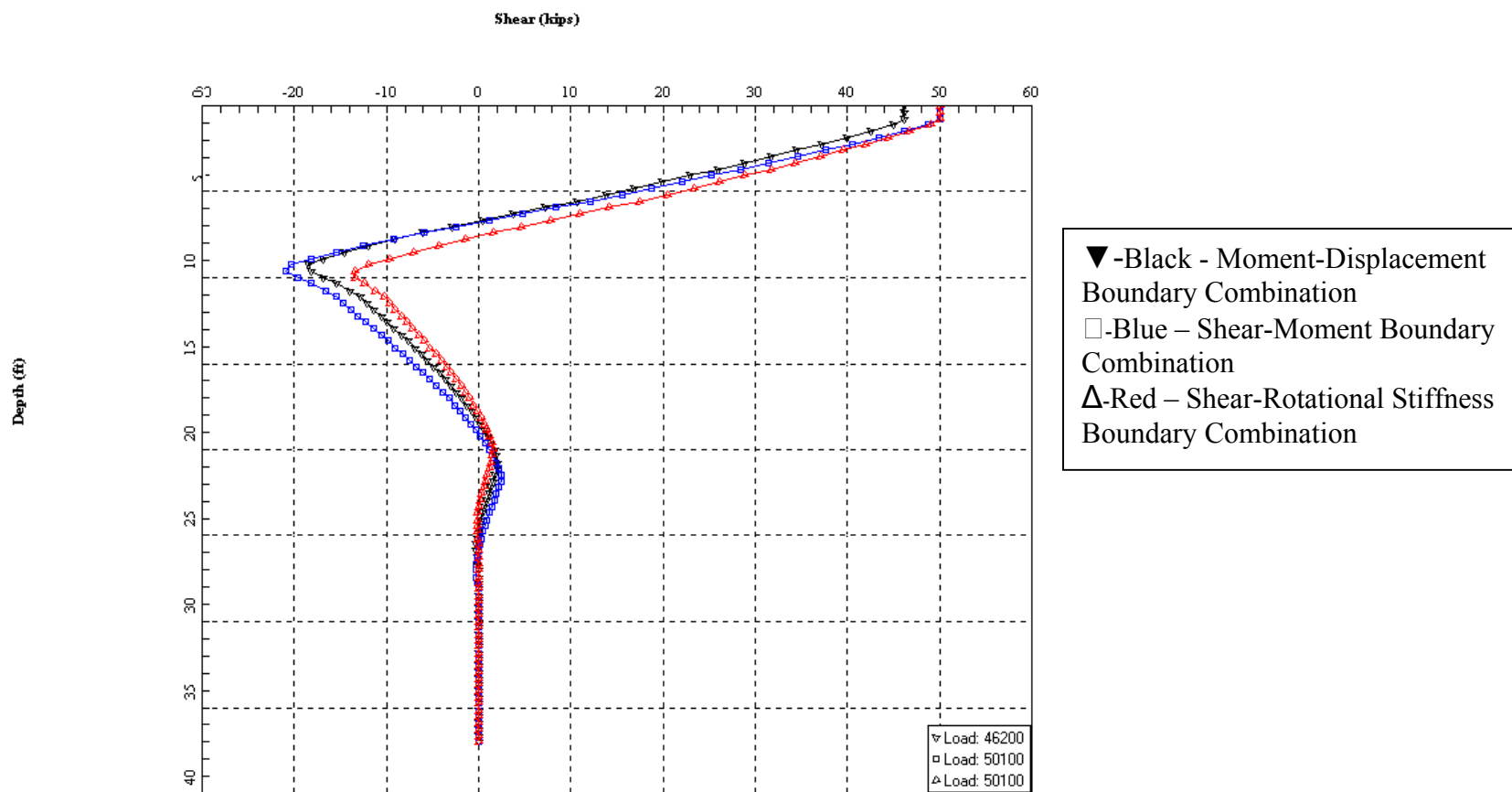


Figure A14: Shear vs. Depth Pile 2 Pulling South Assuming Stiff Clay, ½ inch

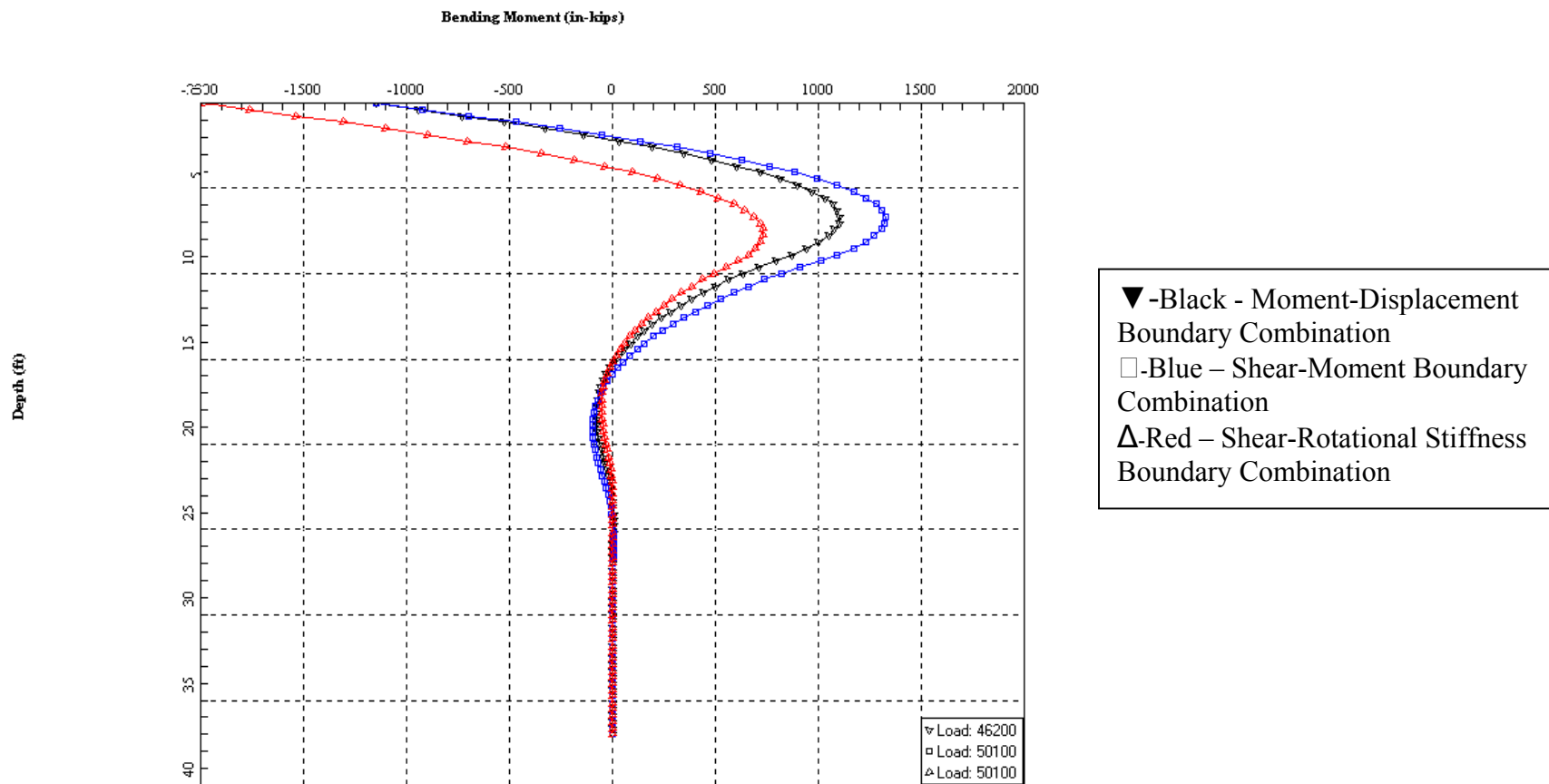


Figure A15: Moment vs. Depth Pile 2 Pulling South Assuming Stiff Clay, ½ inch

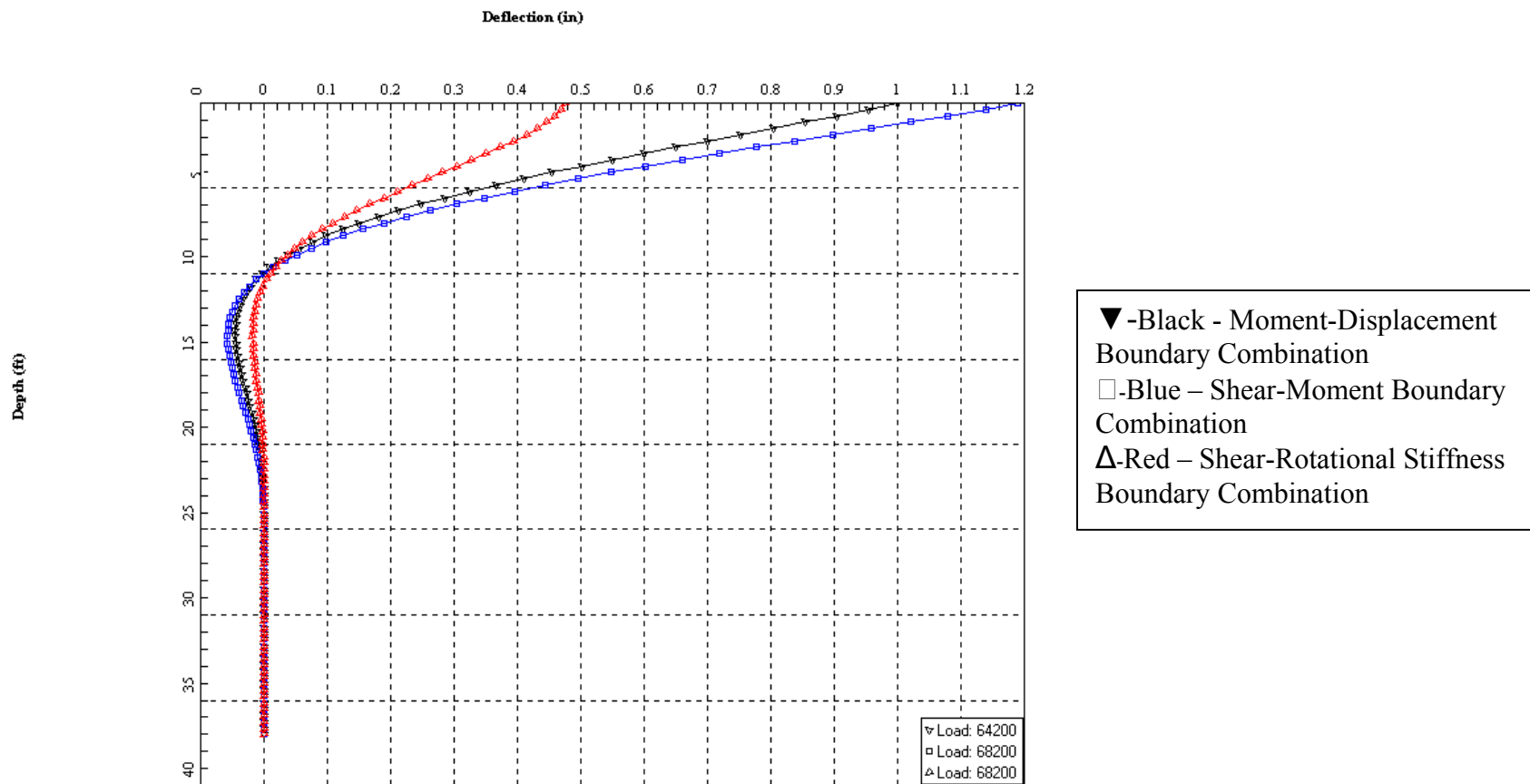


**Figure A16: Deflection vs. Depth Pile 2 Pulling South
Assuming Stiff Clay, 1 inch**

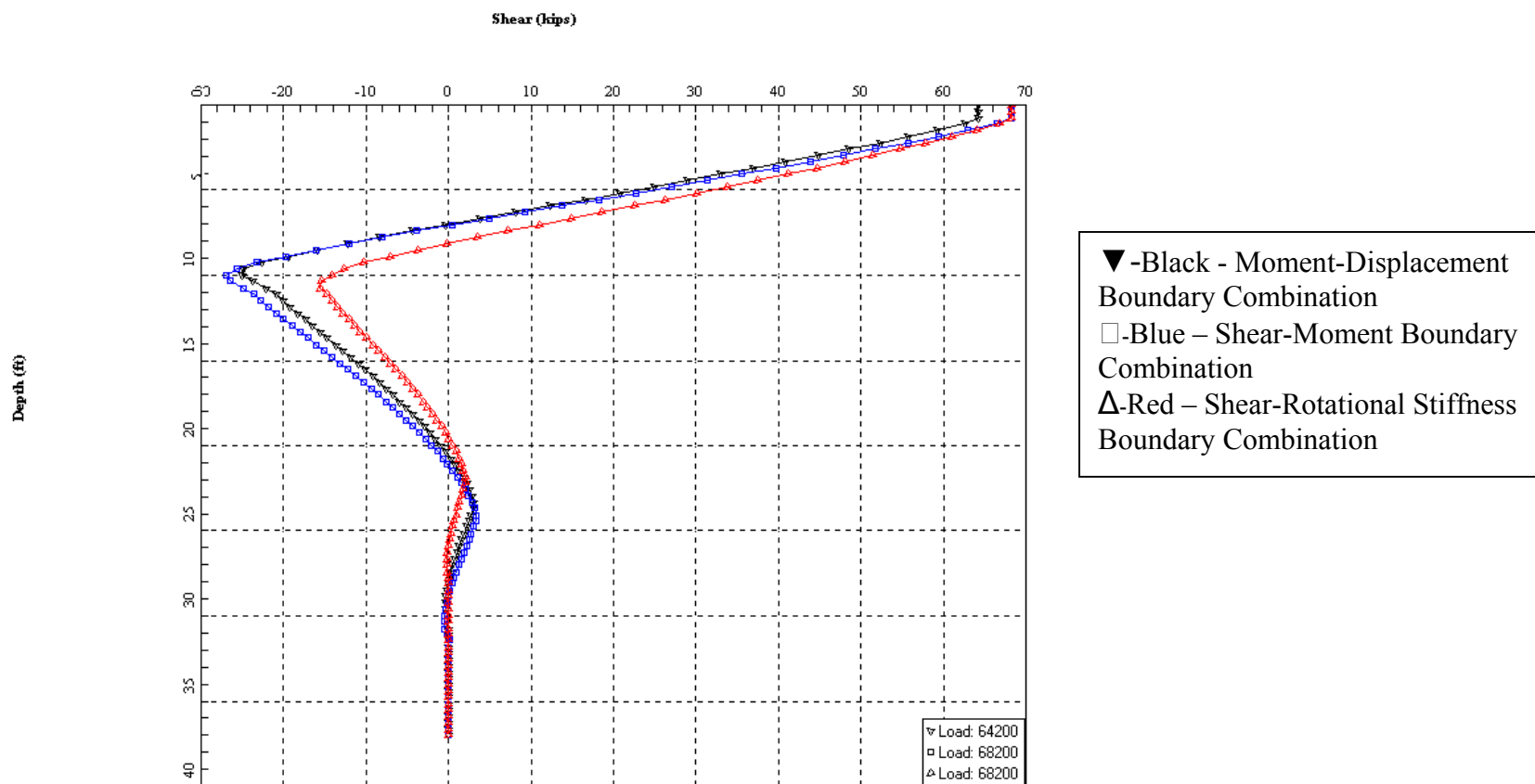


Figure A17: Shear vs. Depth Pile 2 Pulling South Assuming Stiff Clay, 1 inch

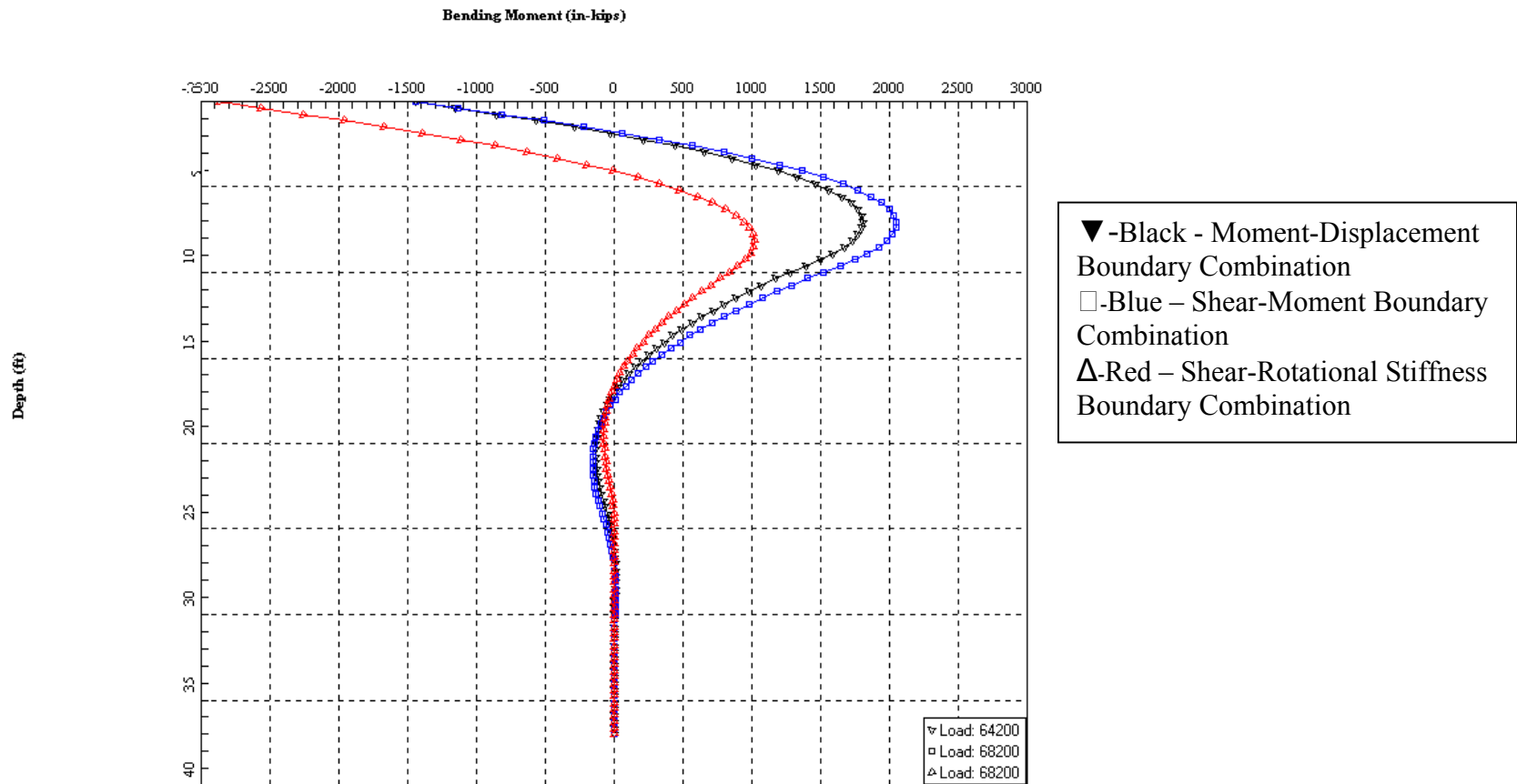


**Figure A18: Moment vs. Depth Pile 2 Pulling South
Assuming Stiff Clay, 1 inch**

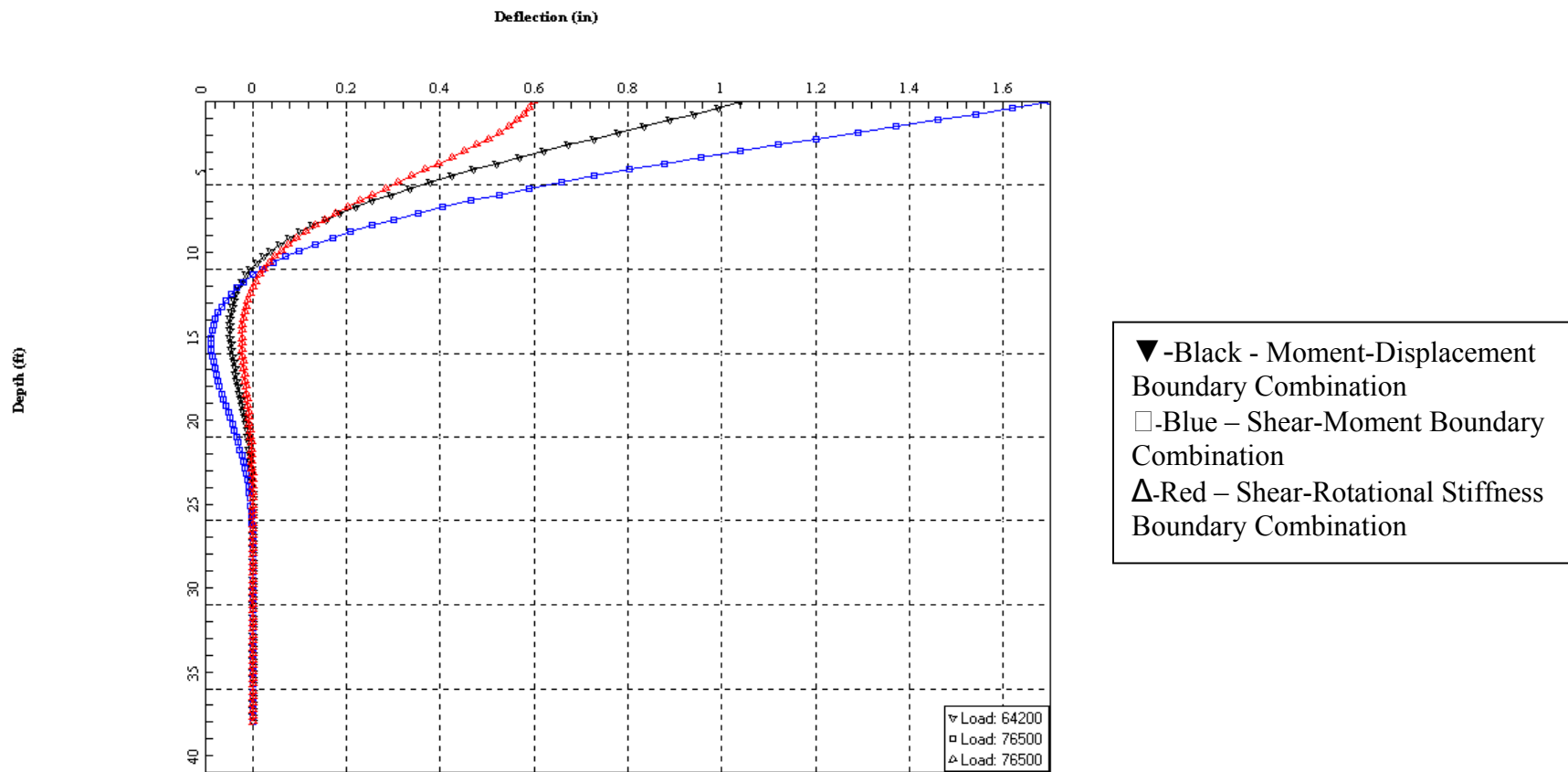


**Figure A19: Deflection vs. Depth Pile 2 Pulling North
Assuming Stiff Clay, 1 inch**

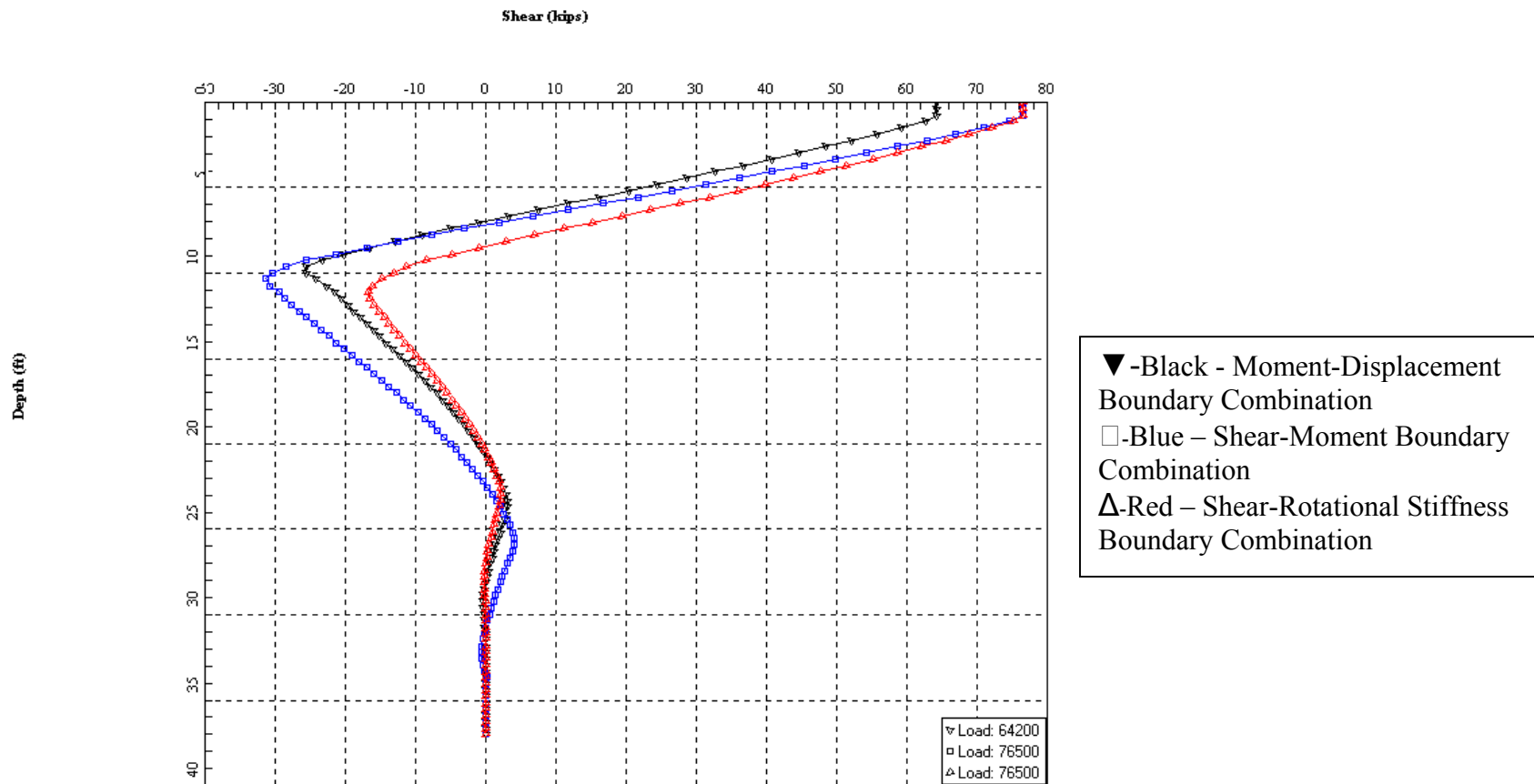


Figure A20: Shear vs. Depth Pile 2 Pulling North Assuming Stiff Clay, 1 inch

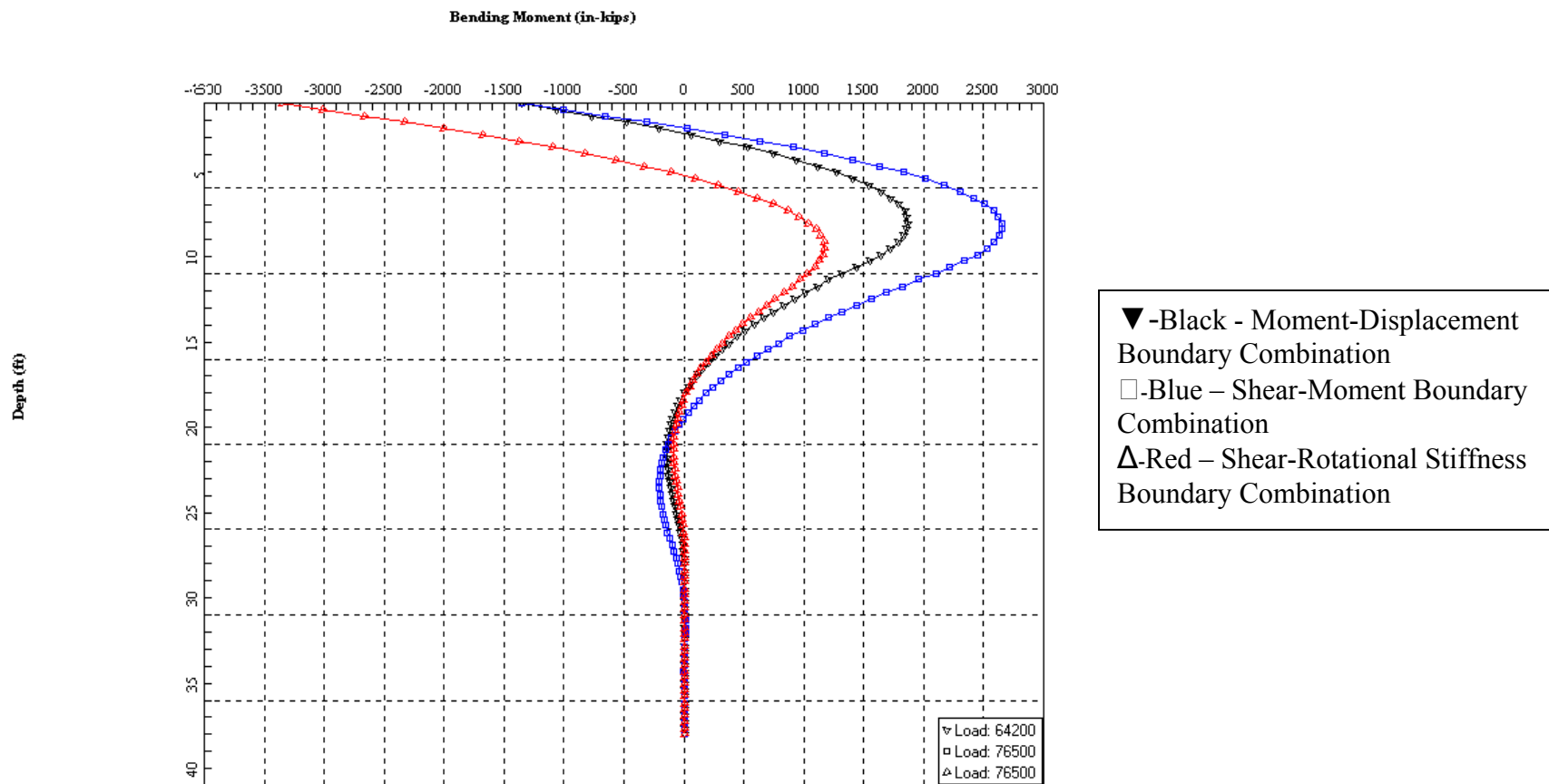


**Figure A21: Moment vs. Depth Pile 2 Pulling North Assuming Stiff
Clav, 1 inch**

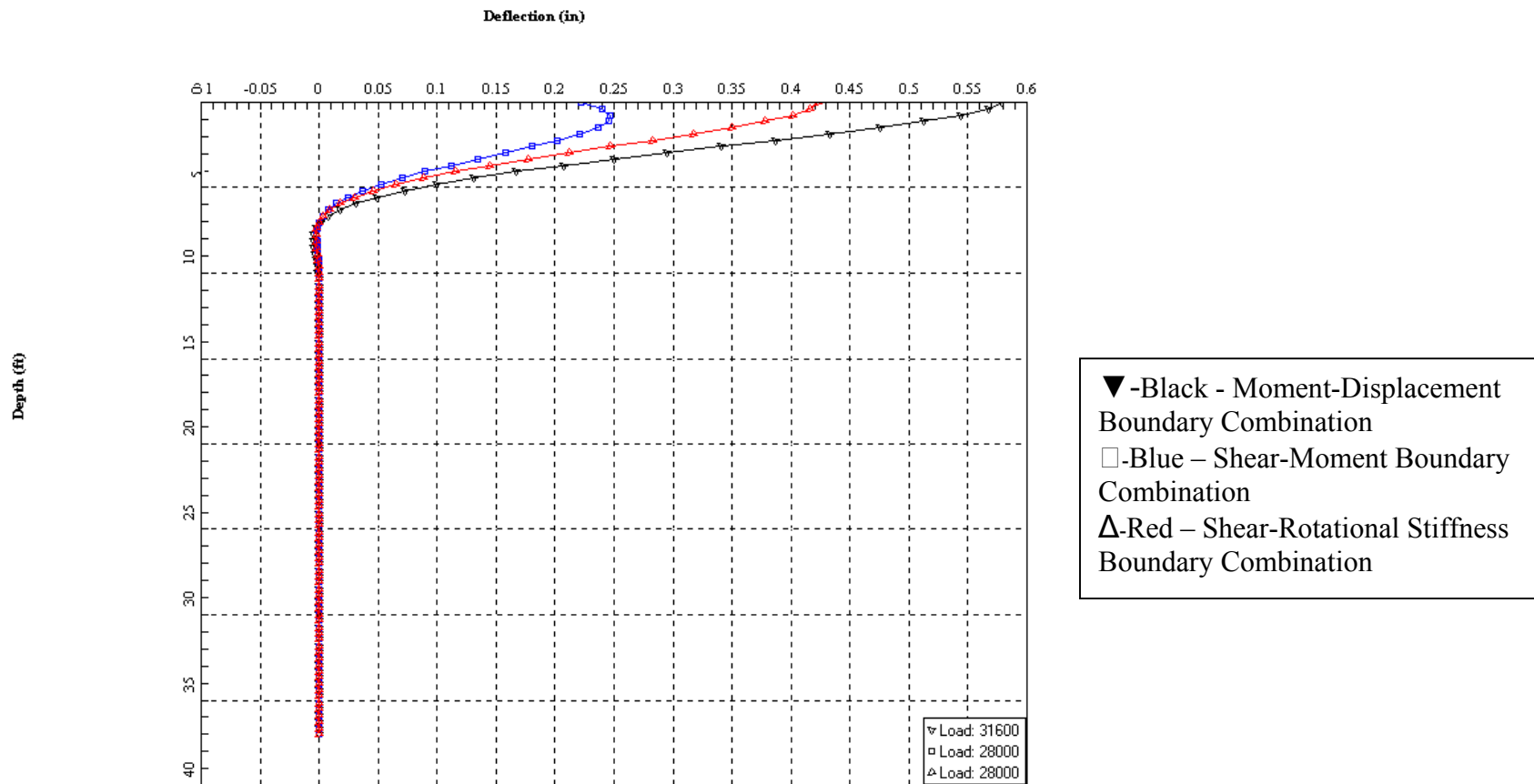


**Figure A22: Deflection vs. Depth Pile 3 Pulling South
Assuming Stiff Clay, ½ inch**

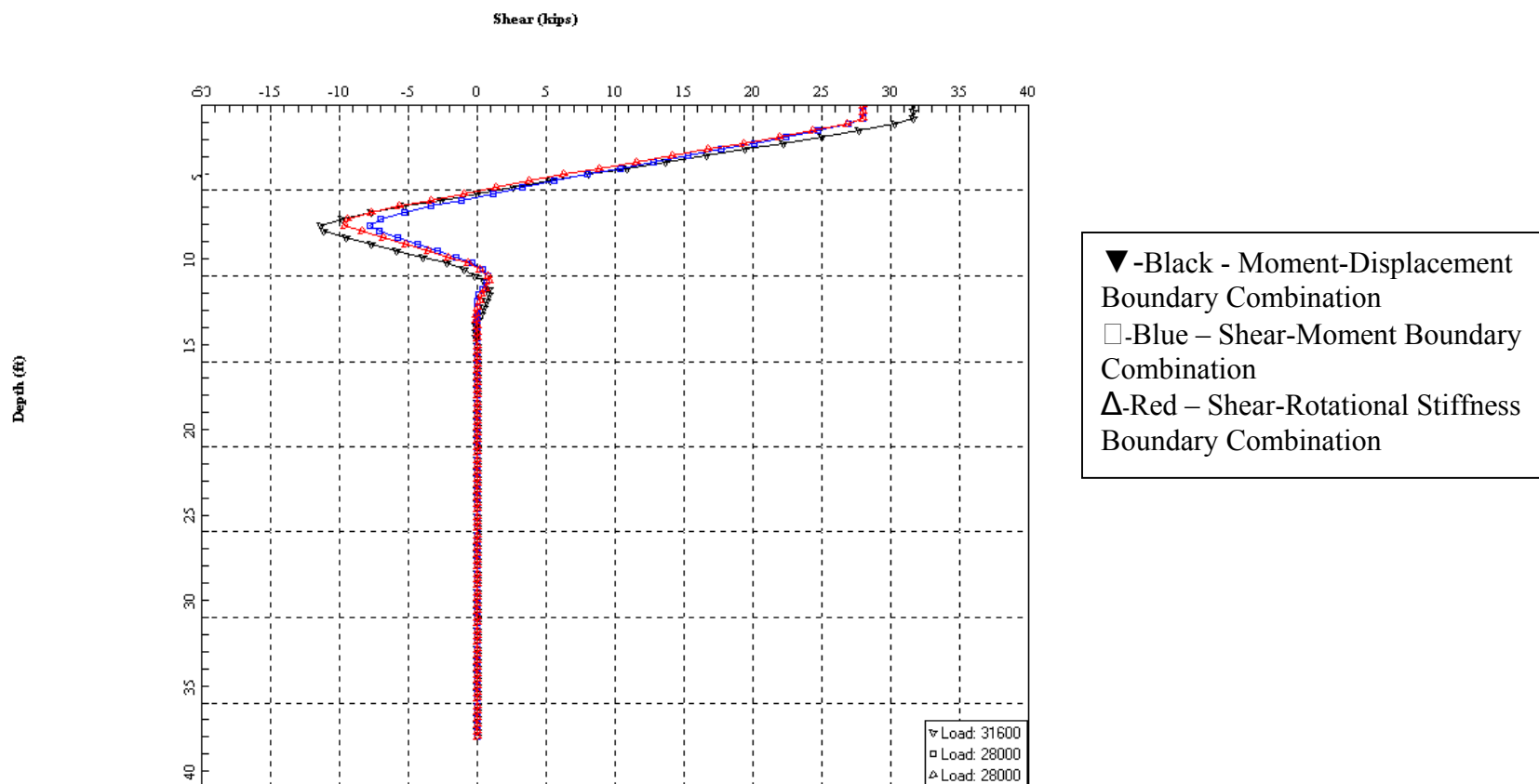


Figure A23: Shear vs. Depth Pile 3 Pulling South Assuming Stiff Clay, ½ inch

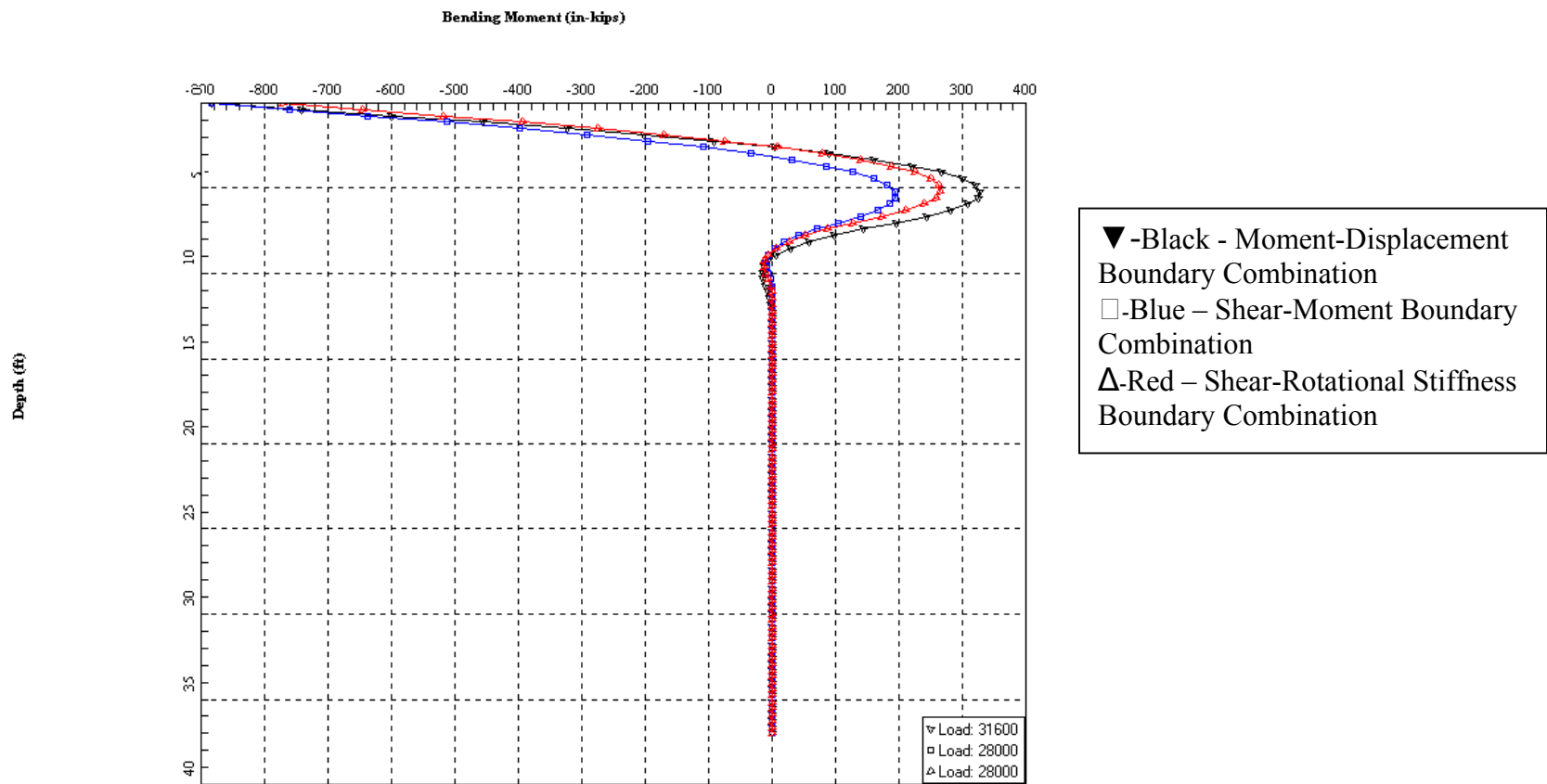


**Figure A24: Moment vs. Depth Pile 3 Pulling South
Assuming Stiff Clay, ½ inch**

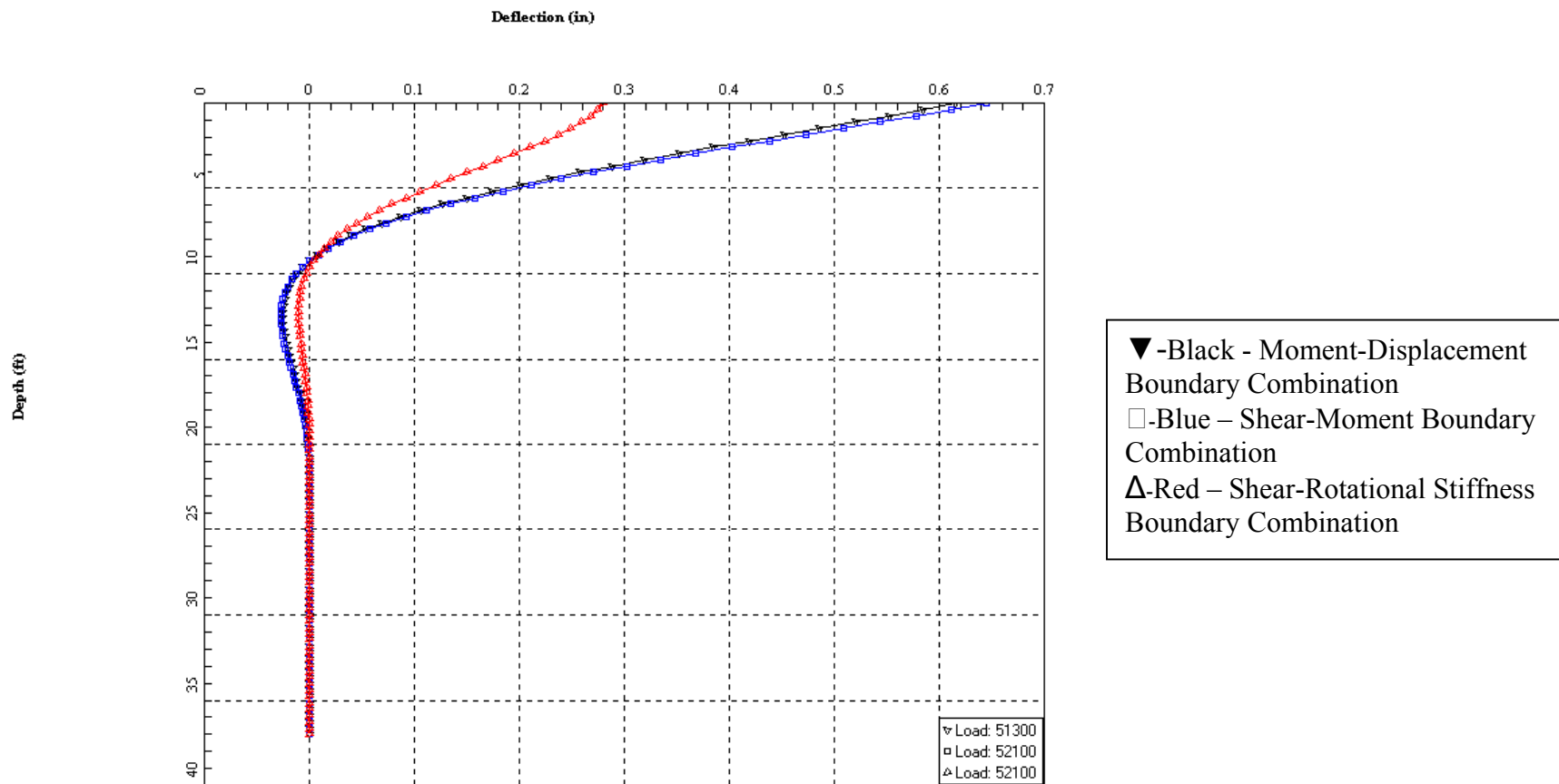


**Figure A25: Deflection vs. Depth Pile 3 Pulling North
 Assuming Stiff Clay, ½ inch**

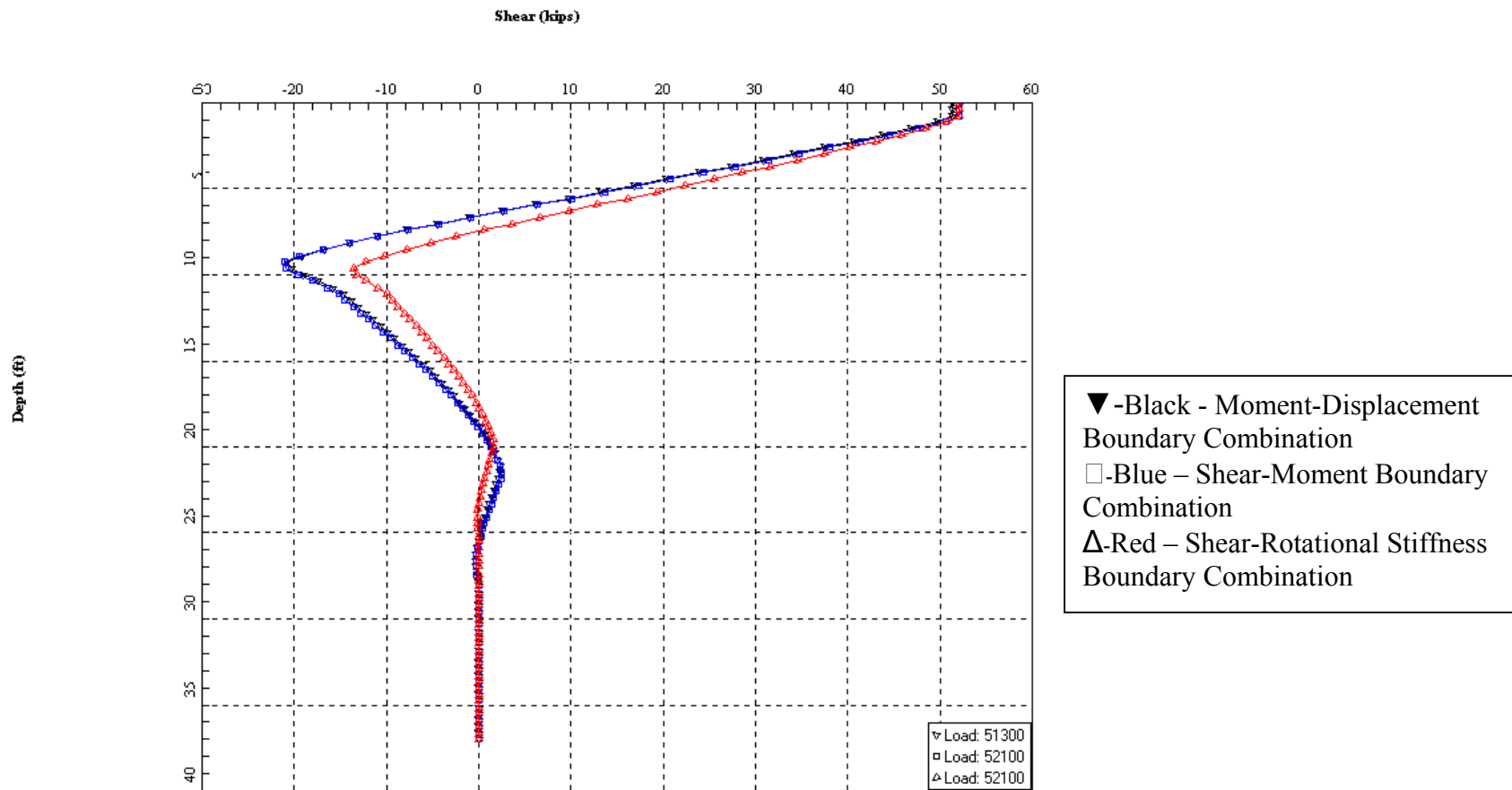


Figure A26: Shear vs. Depth Pile 3 Pulling North Assuming Stiff Clay, ½ inch

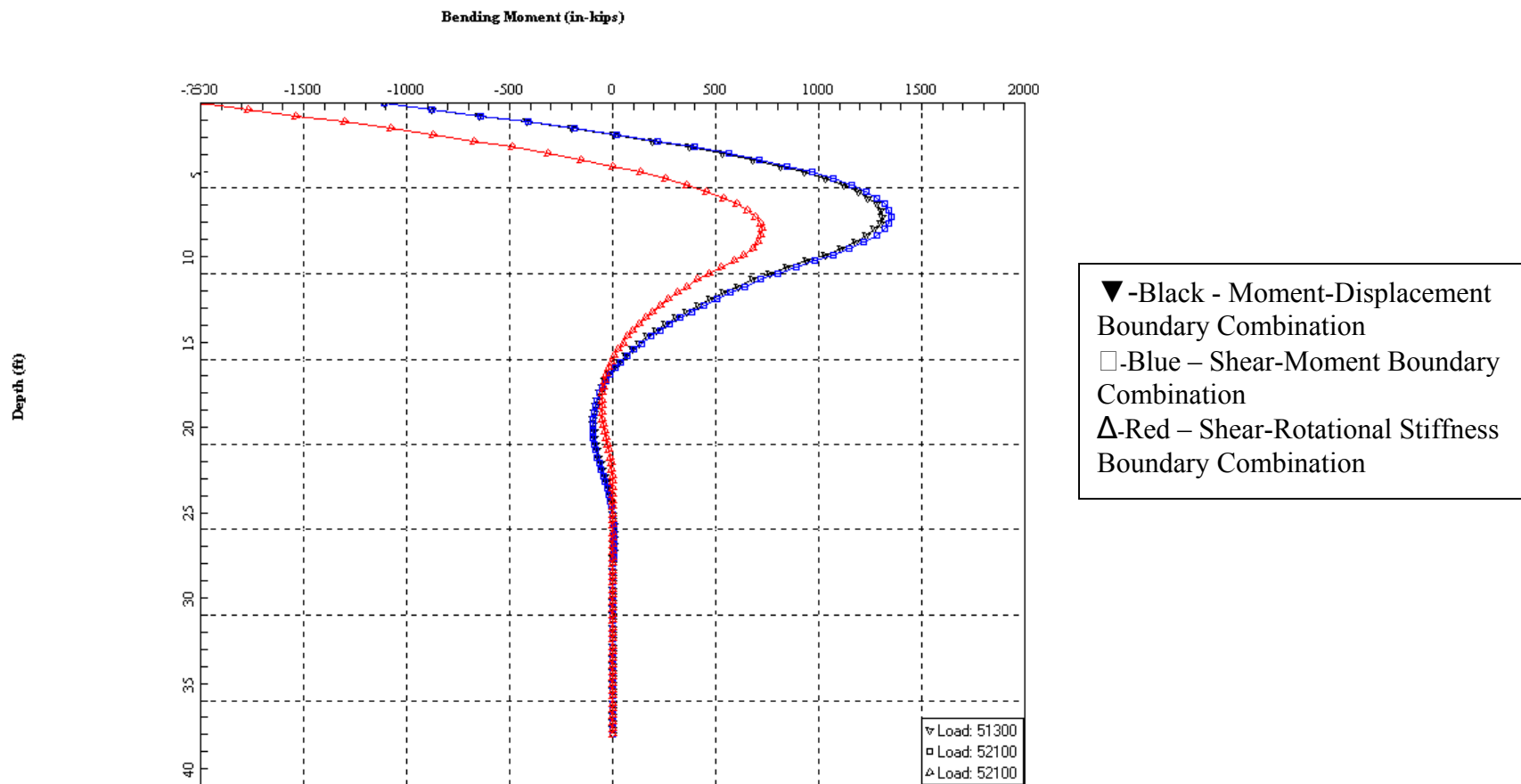


Figure A27: Moment vs. Depth Pile 3 Pulling North Assuming Stiff Clay, ½ inch

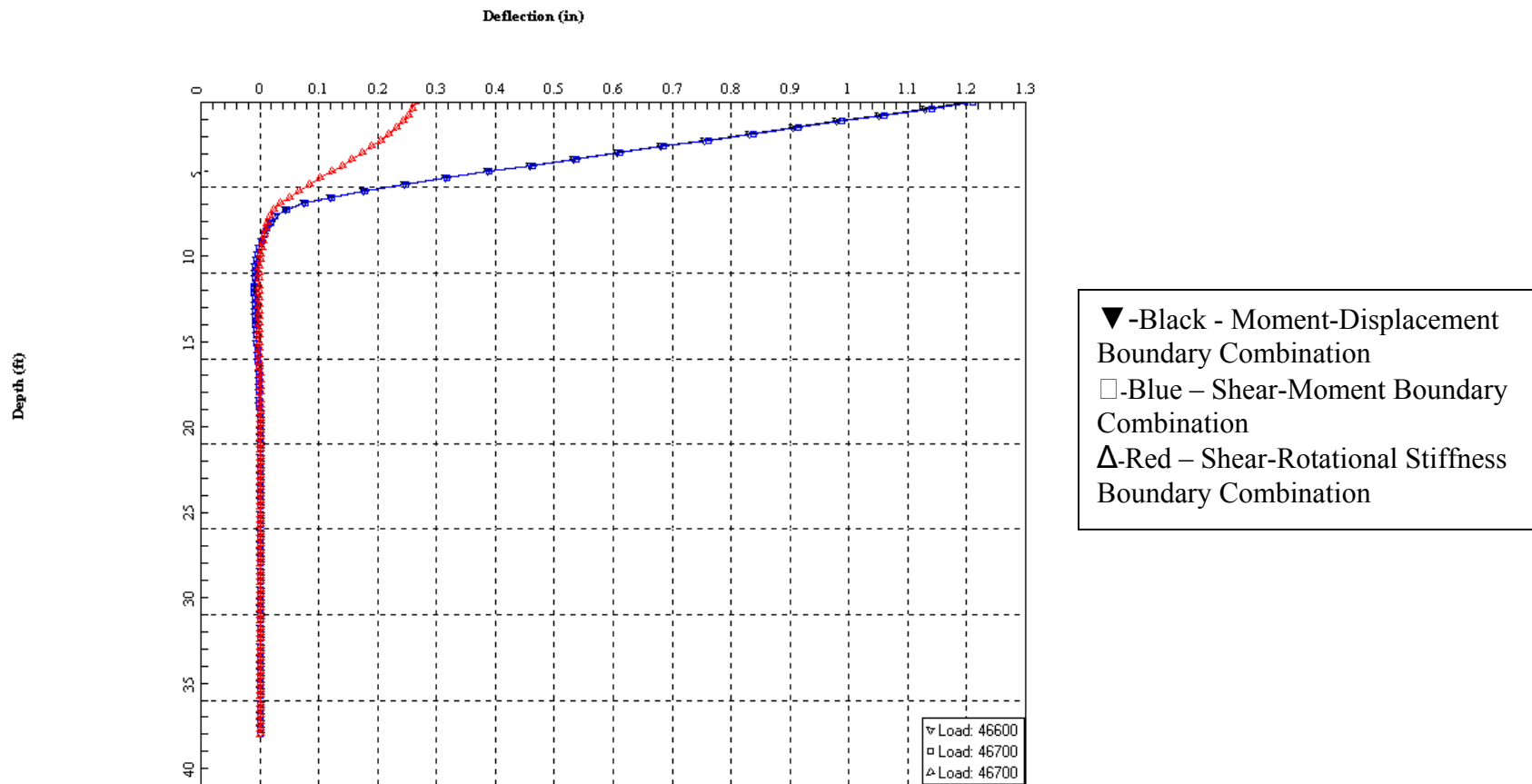


**Figure A28: Deflection vs. Depth Pile 3 Pulling South
Assuming Stiff Clay, 1 inch**

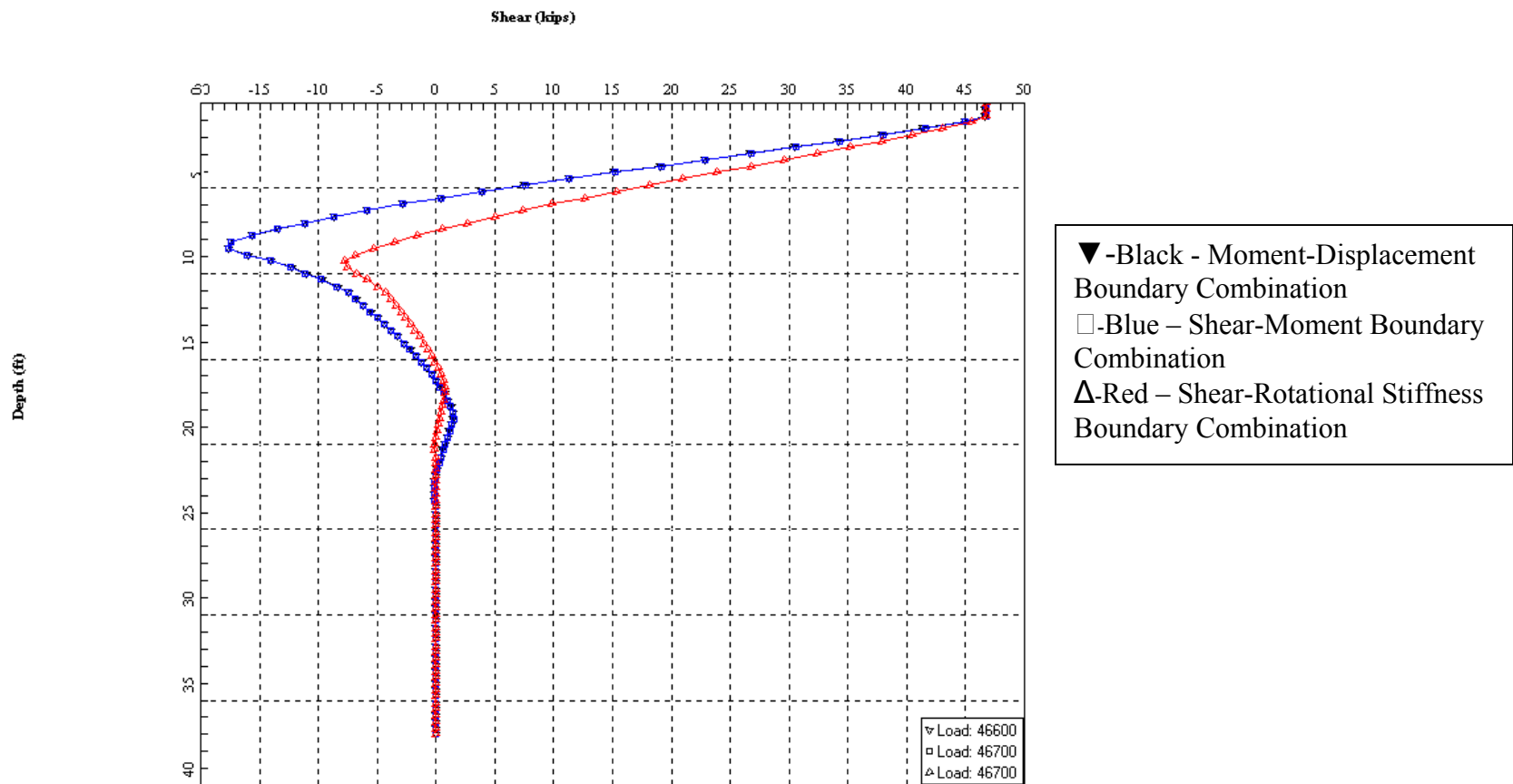


Figure A29: Shear vs. Depth Pile 3 Pulling South Assuming Stiff Clay, 1 inch

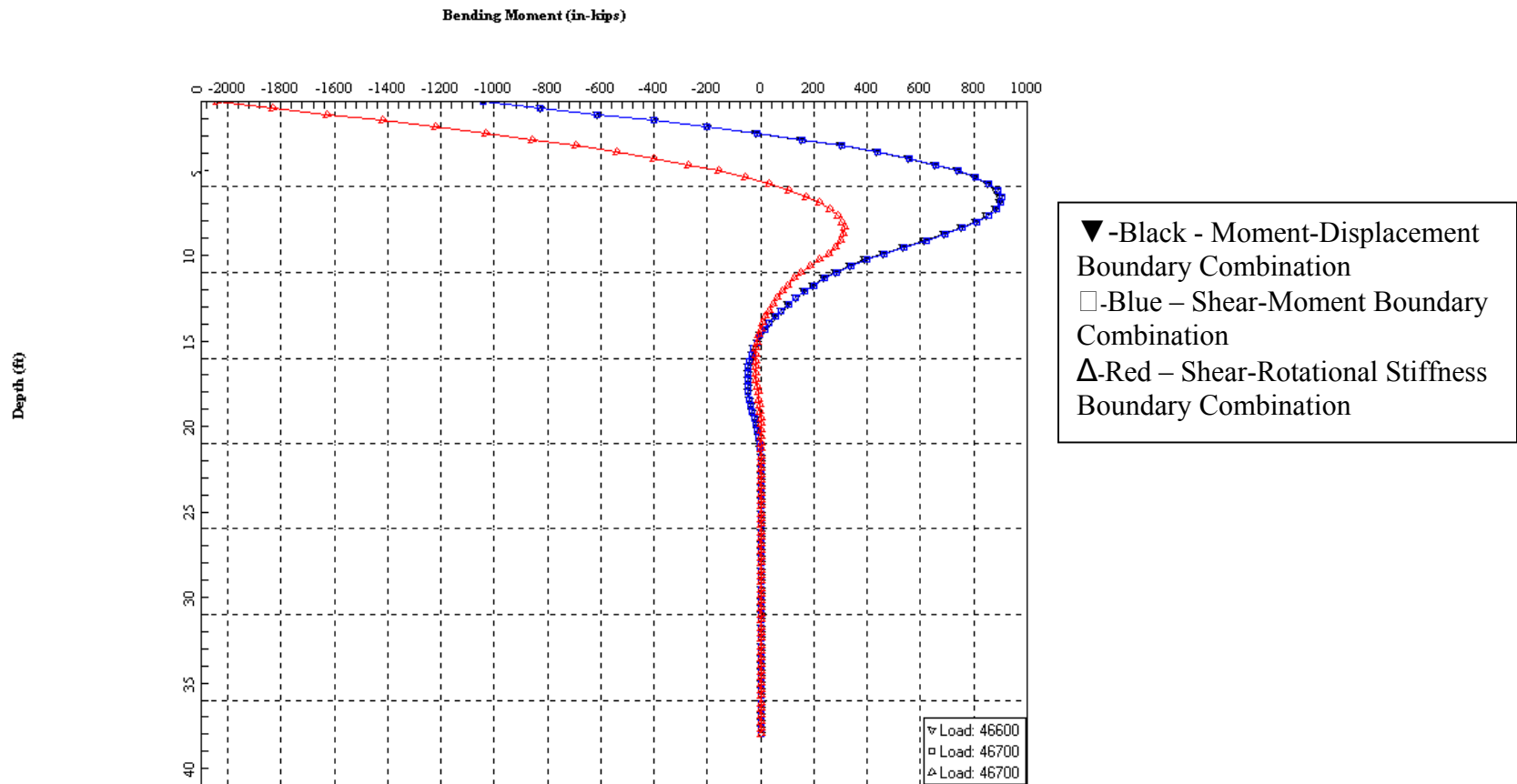


**Figure A30: Moment vs. Depth Pile 3 Pulling South
Assuming Stiff Clay, 1 inch**

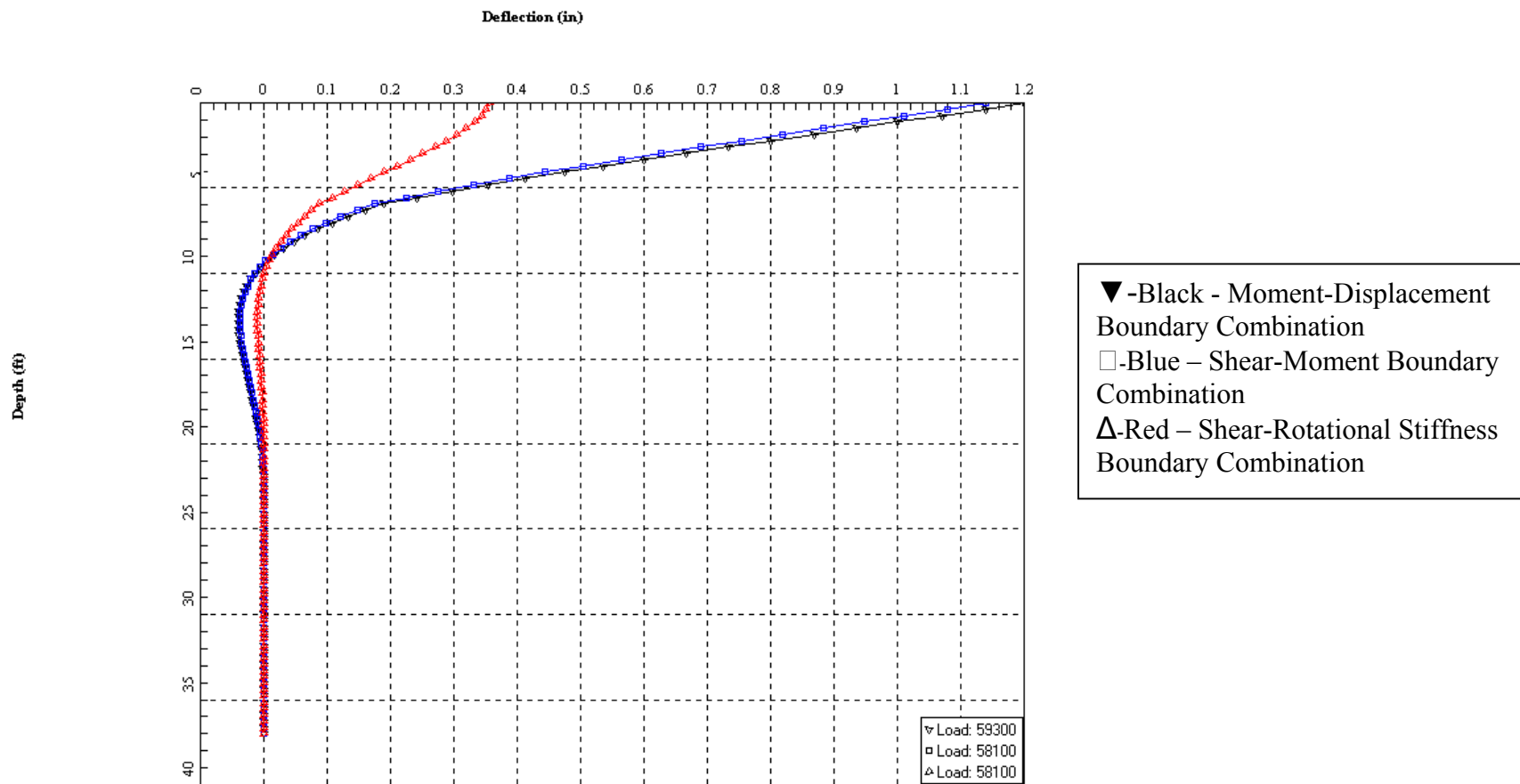


**Figure A31: Deflection vs. Depth Pile 3 Pulling North
 Assuming Stiff Clay, 1 inch**

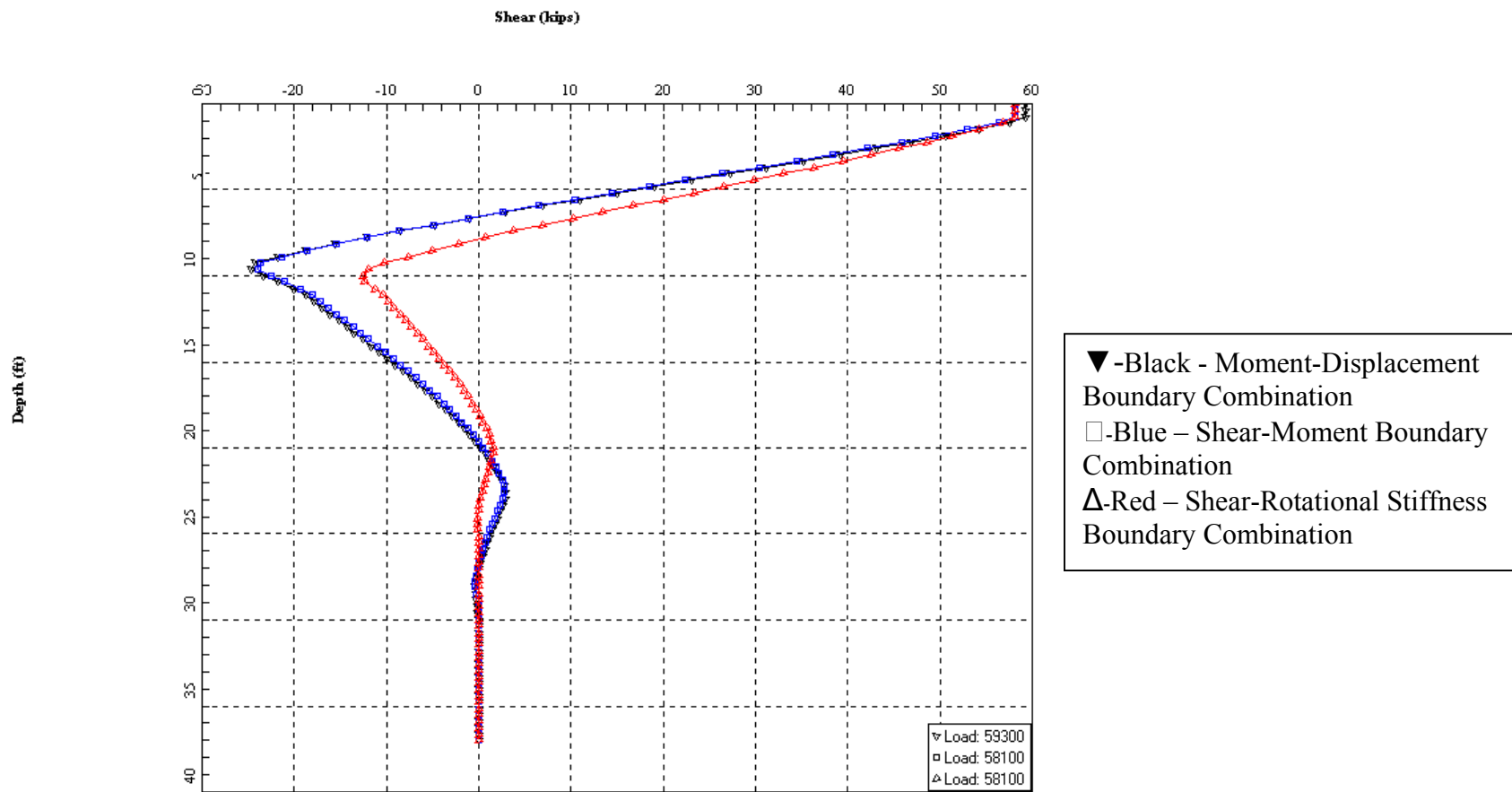


Figure A32: Shear vs. Depth Pile 3 Pulling North Assuming Stiff Clay, 1 inch

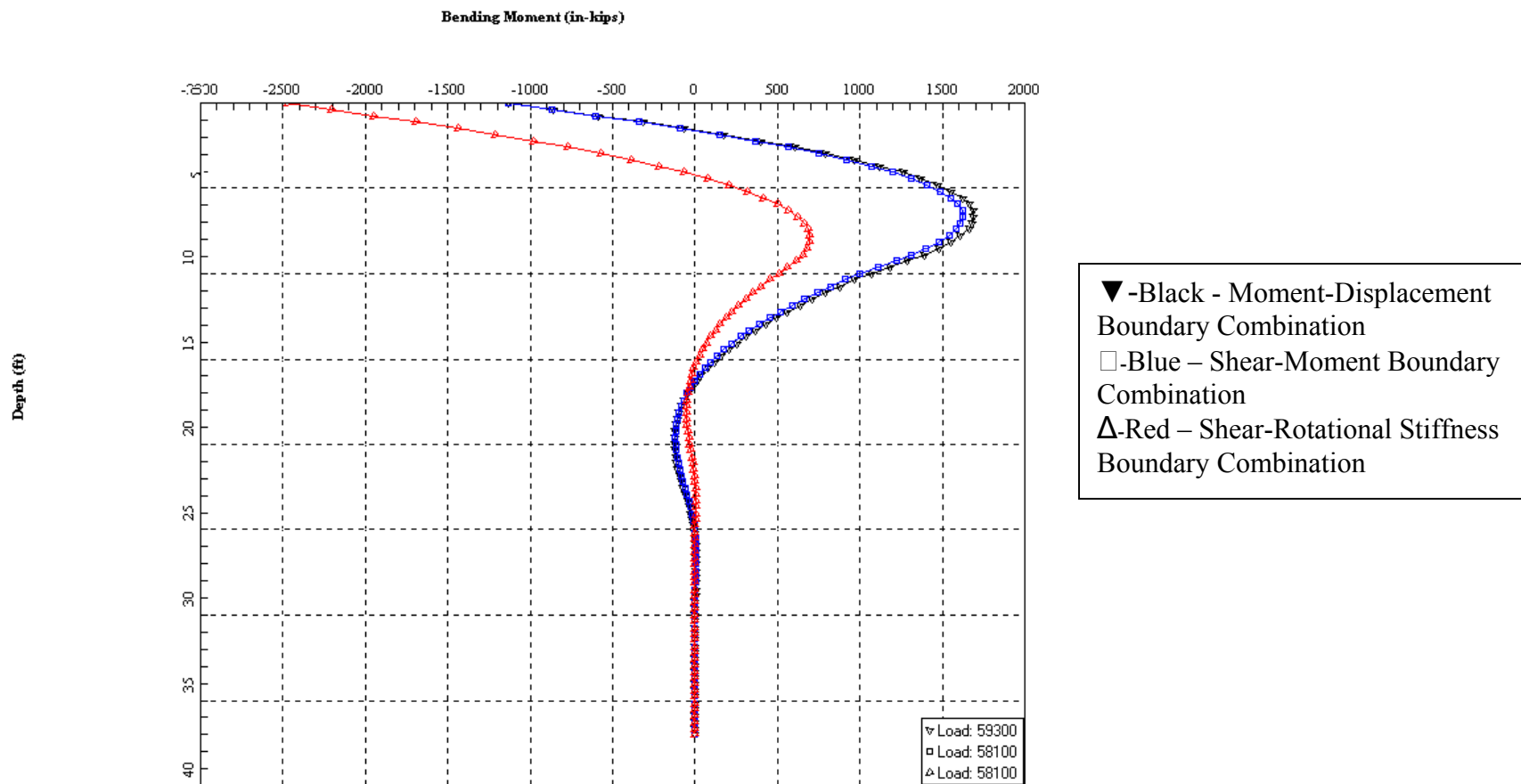


Figure A33: Moment vs. Depth Pile 3 Pulling North Assuming Stiff Clav. 1 inch

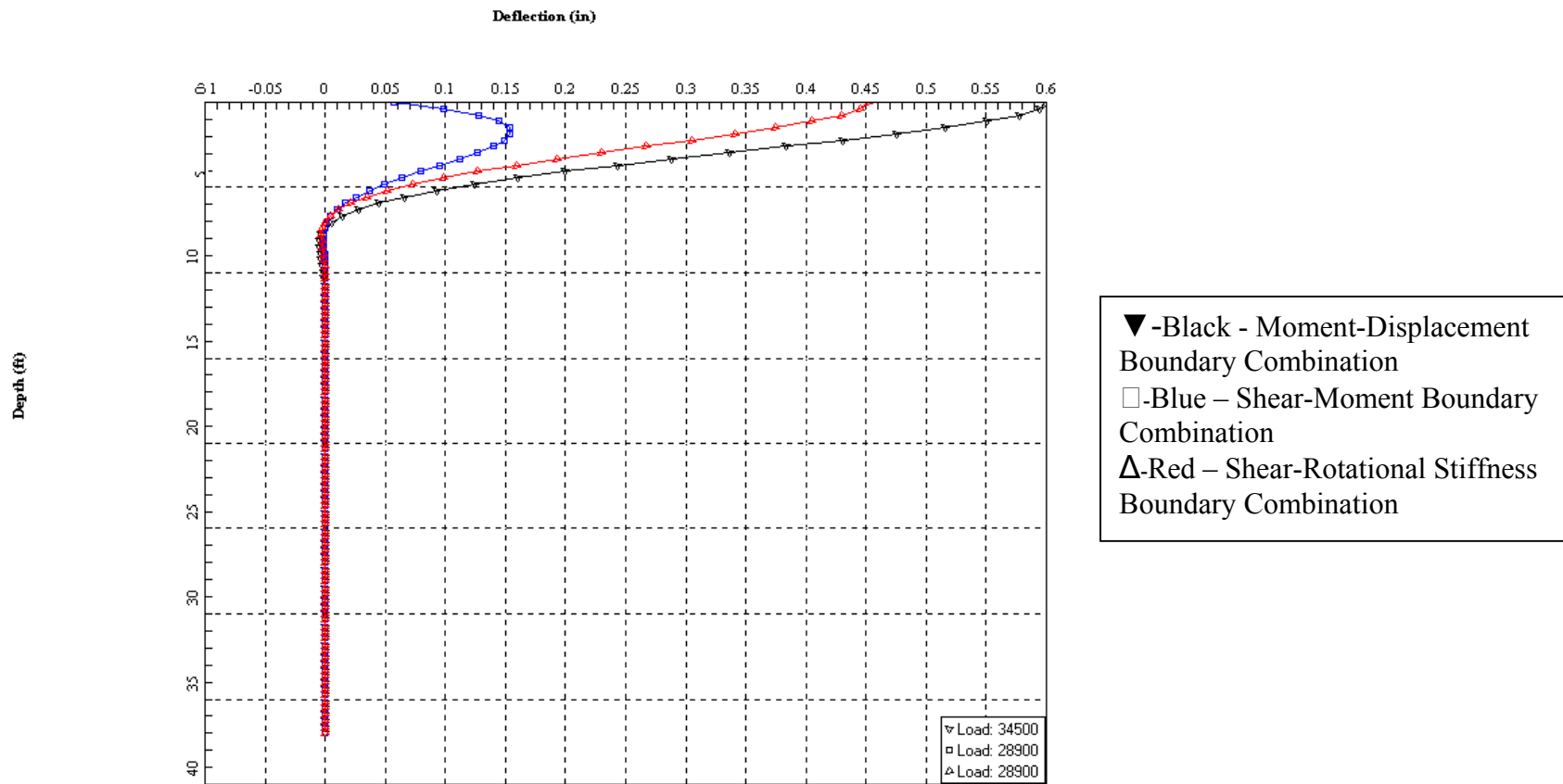


**Figure A34: Deflection vs. Depth Pile 4 Pulling South
Assuming Stiff Clay, ½ inch**

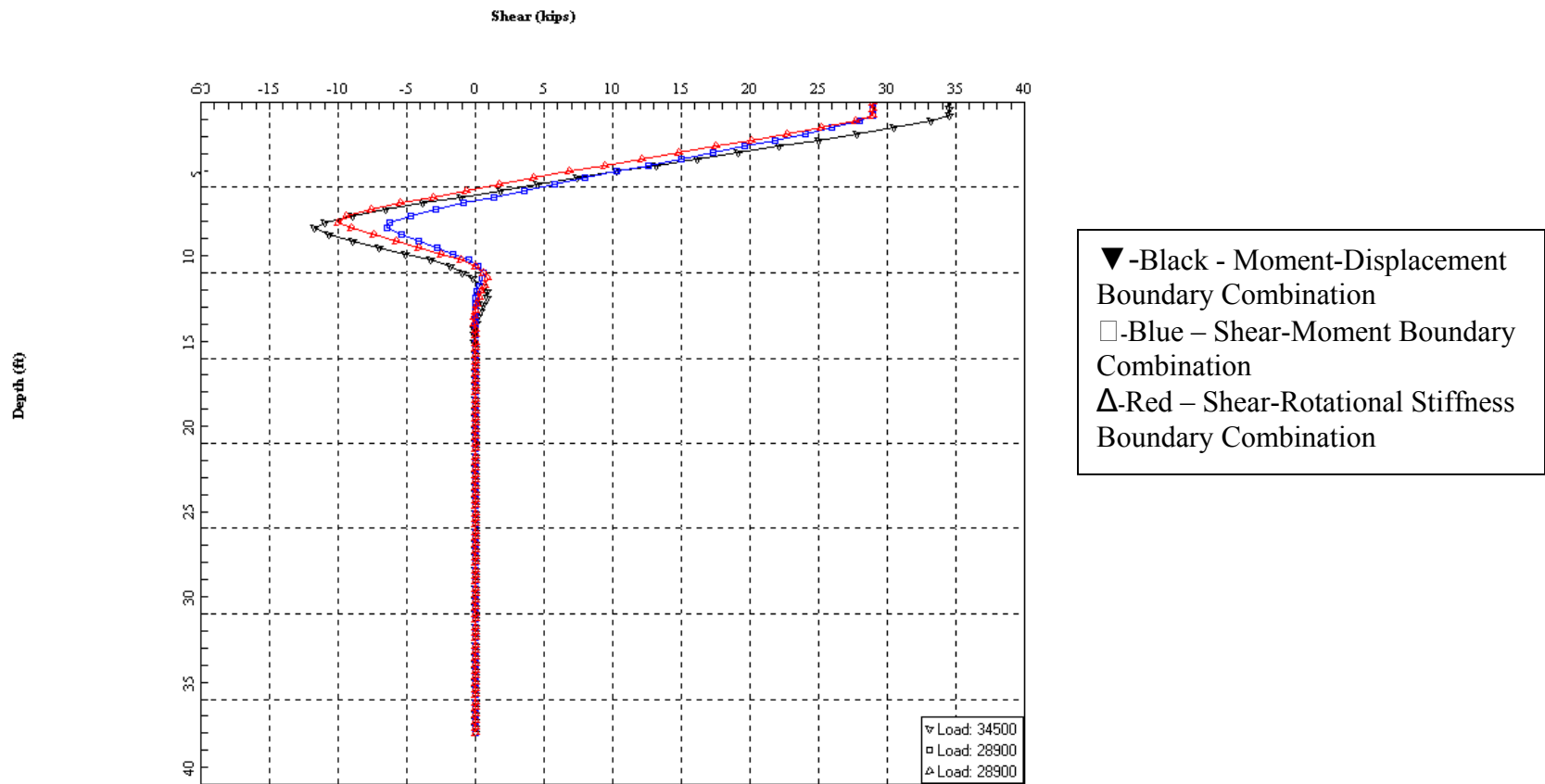


Figure A35: Shear vs. Depth Pile 4 Pulling South Assuming Stiff Clay, ½ inch

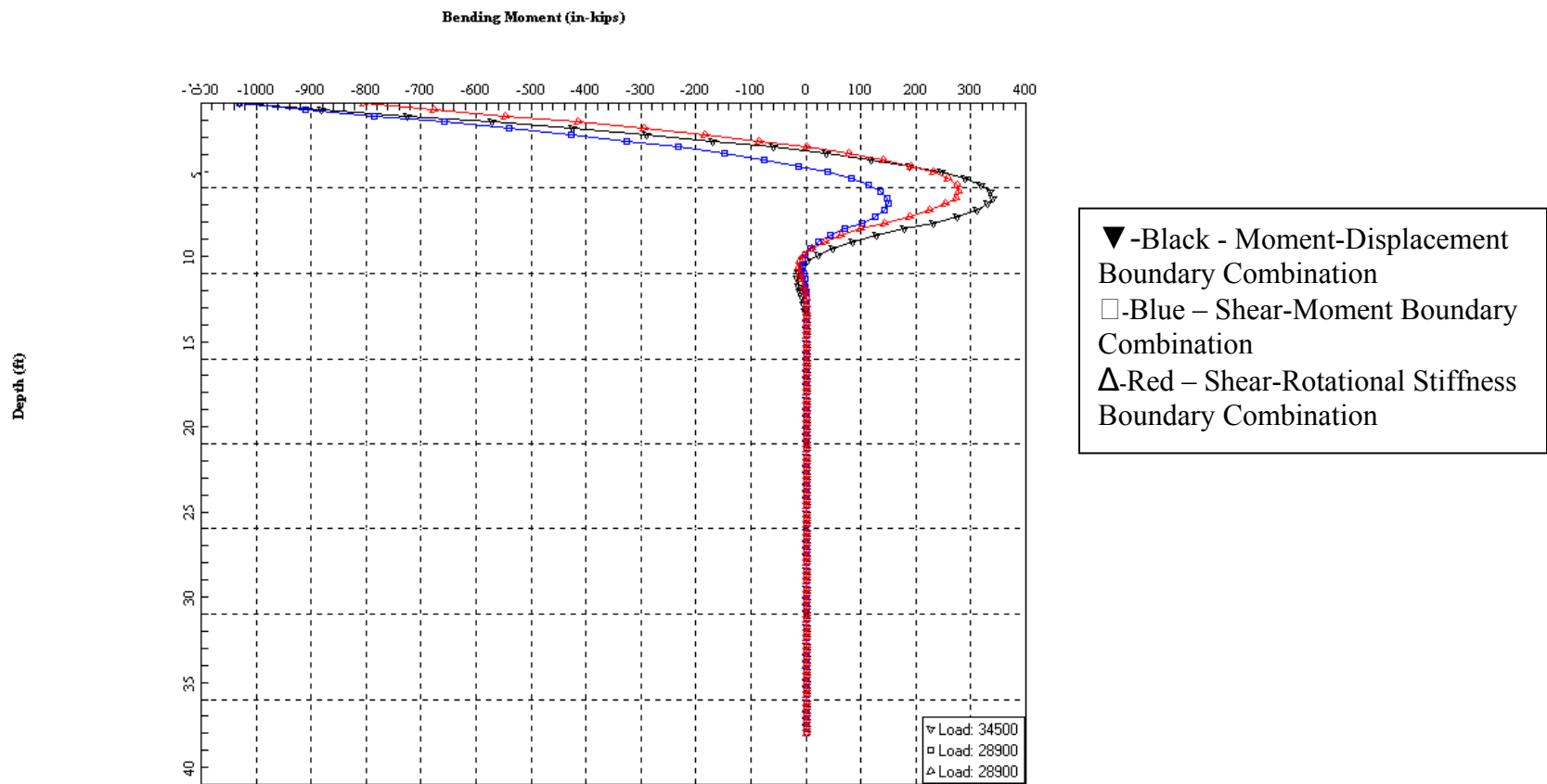


**Figure A36: Moment vs. Depth Pile 4 Pulling South
Assuming Stiff Clay, ½ inch**

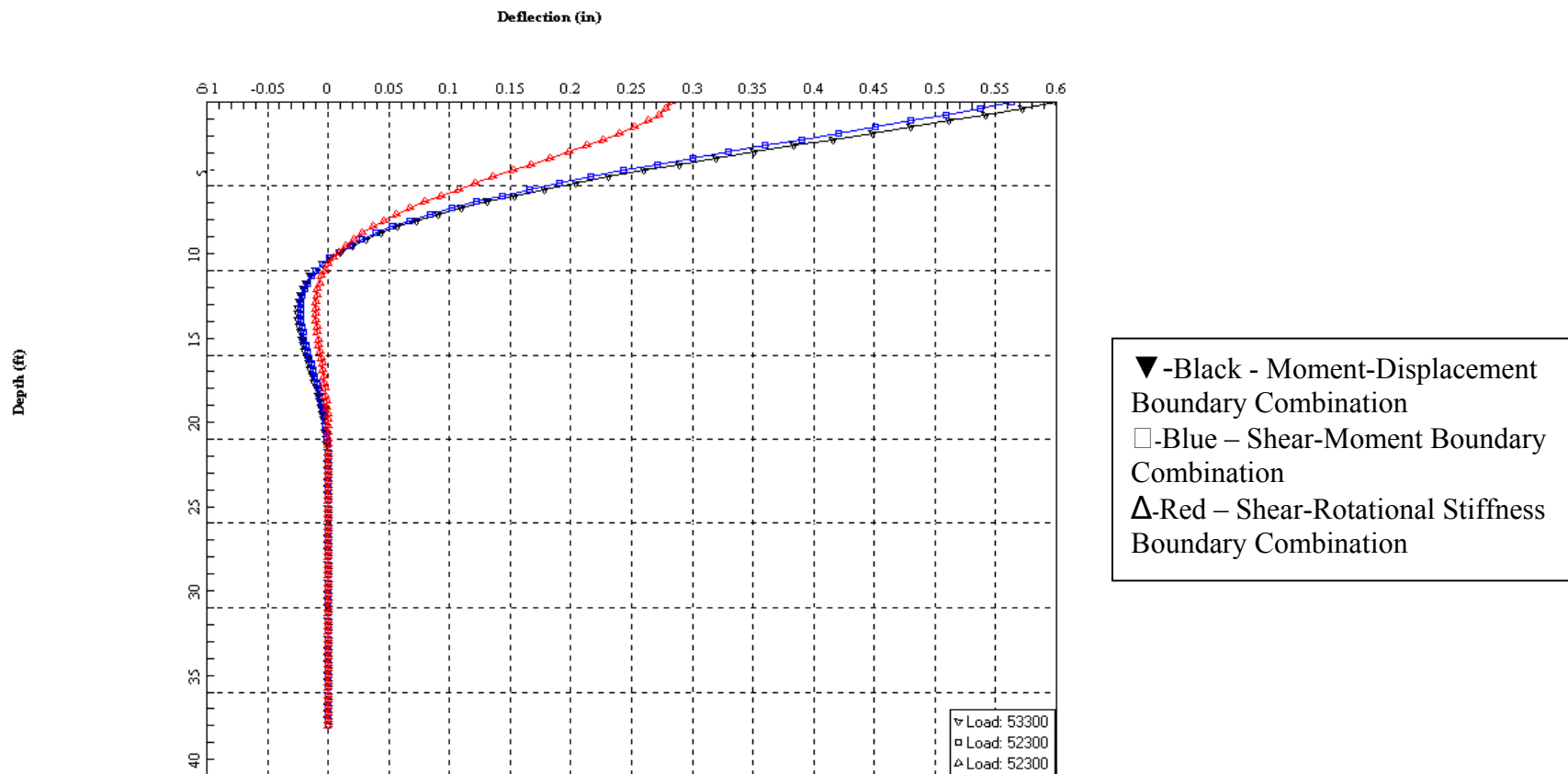


**Figure A37: Deflection vs. Depth Pile 4 Pulling North
Assuming Stiff Clay, ½ inch**

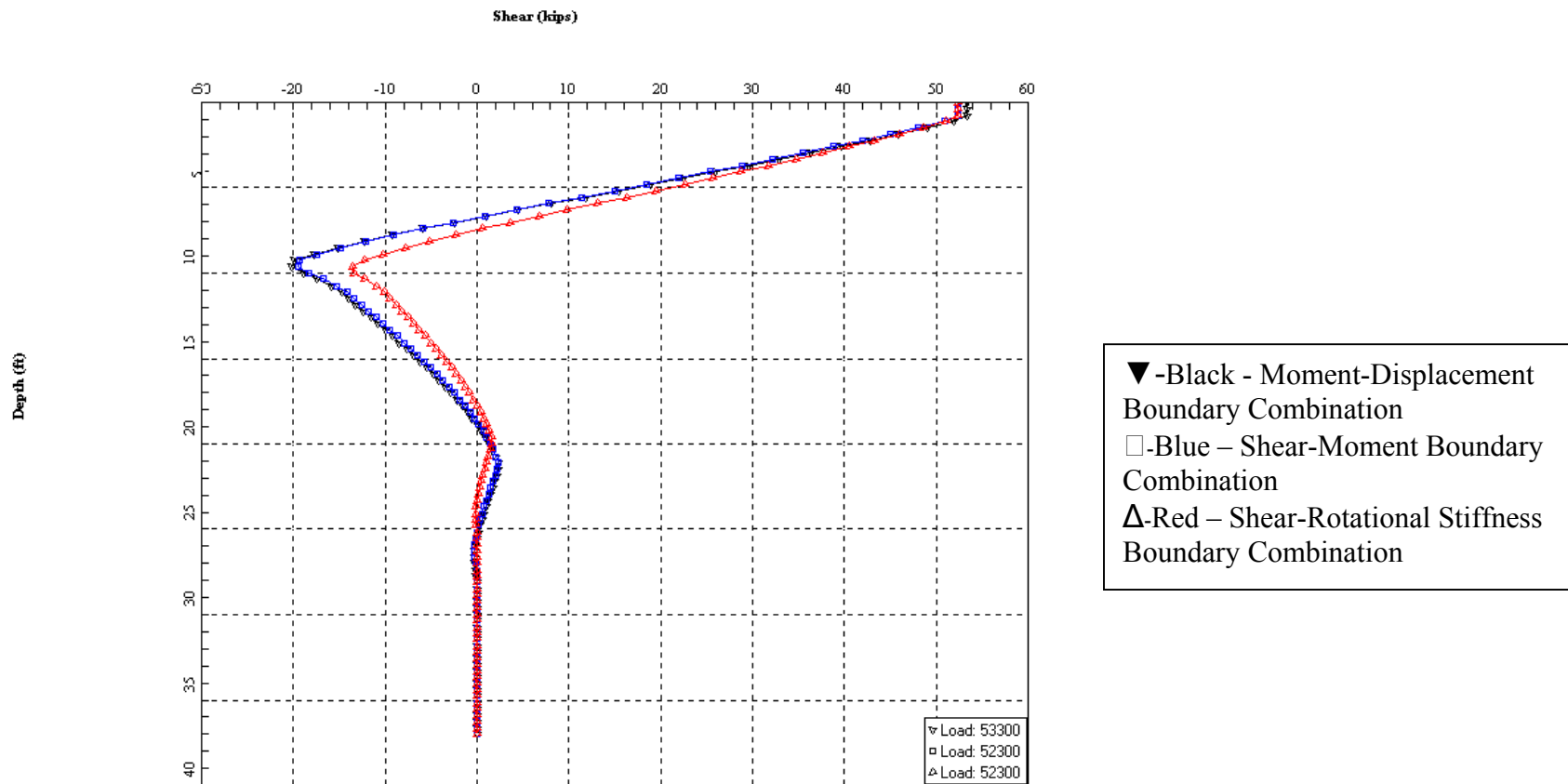


Figure A38: Shear vs. Depth Pile 4 Pulling North Assuming Stiff Clay, ½ inch

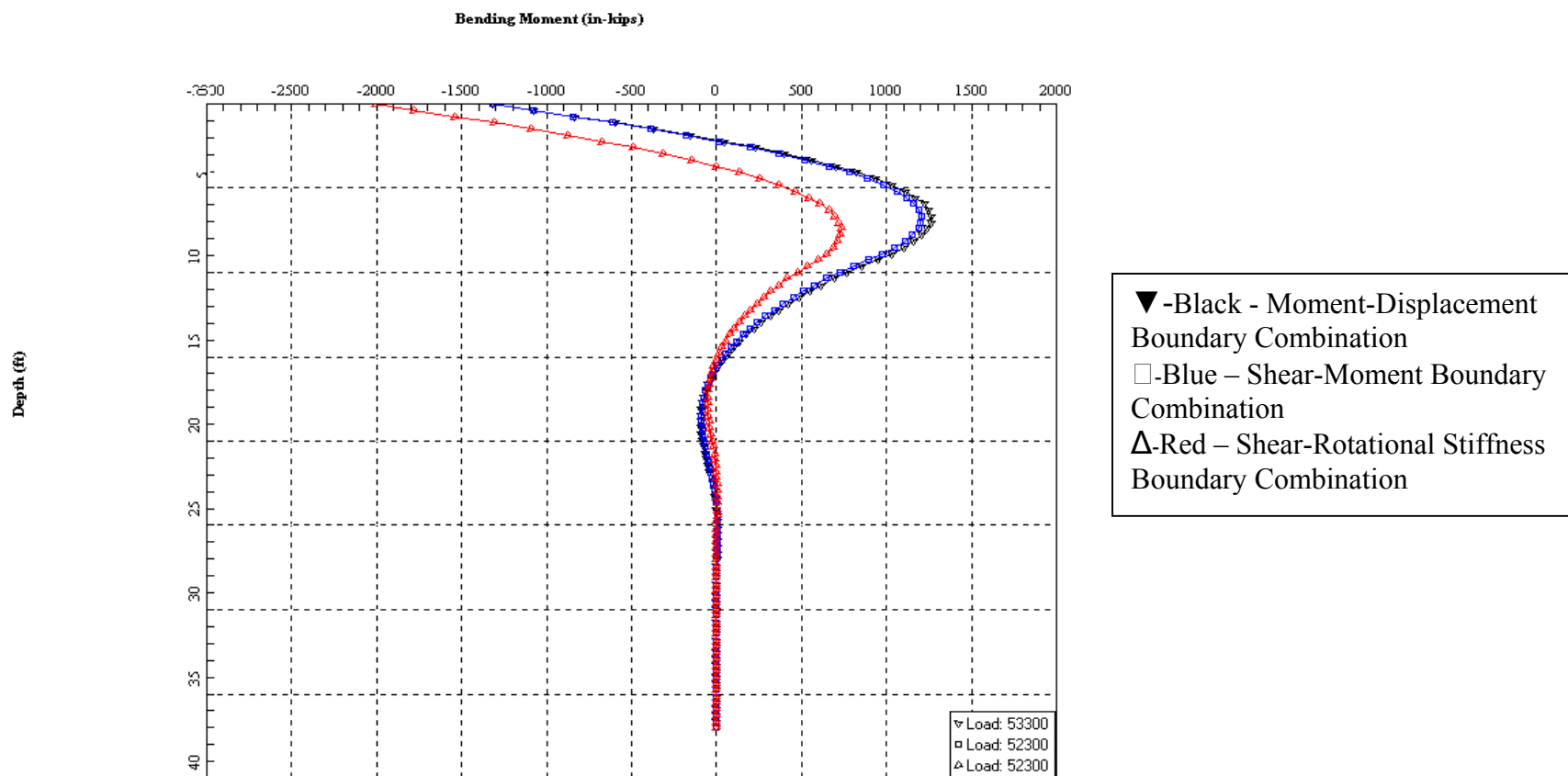


Figure A39: Moment vs. Depth Pile 4 Pulling North Assuming Stiff Clay, ½ inch

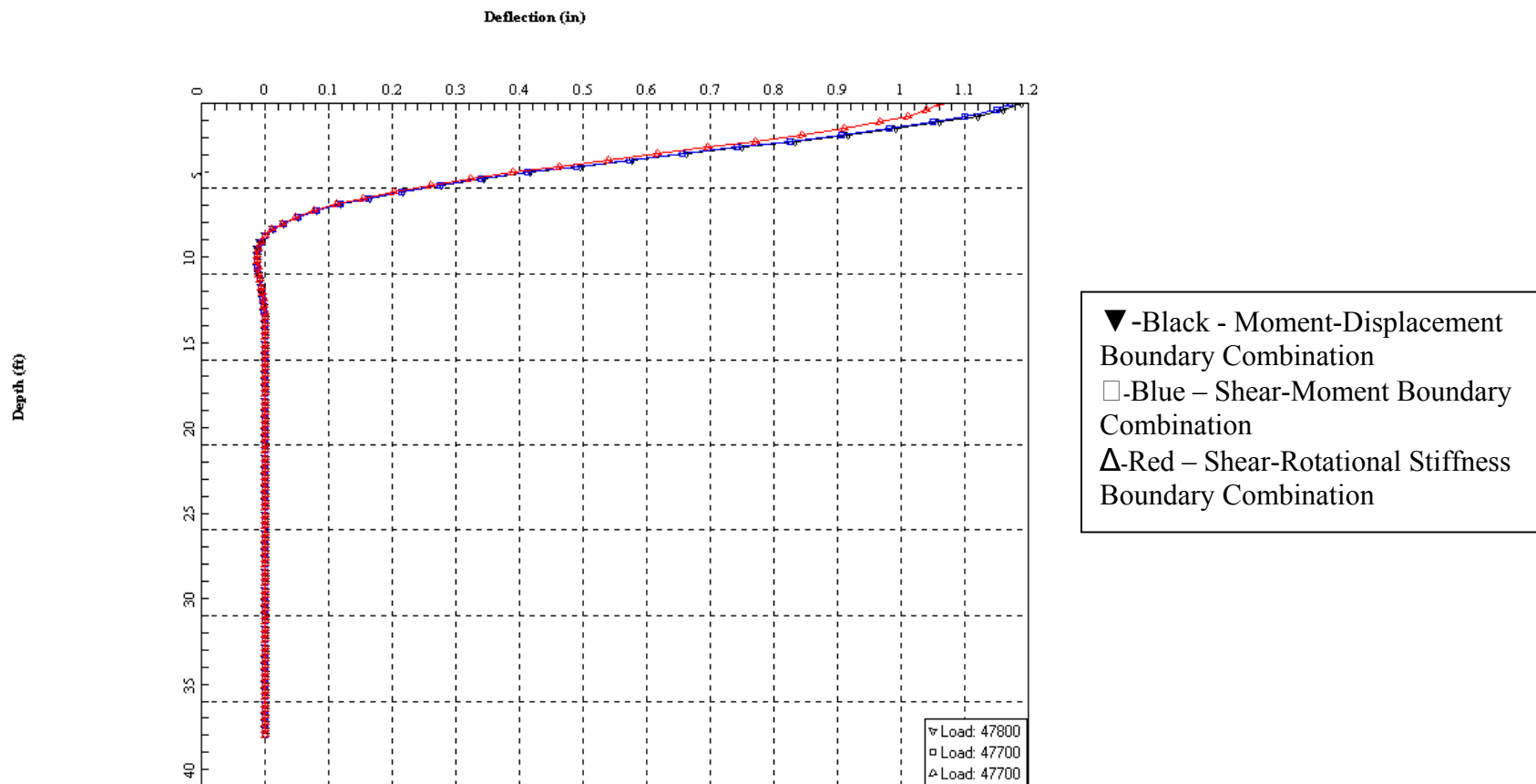


**Figure A40: Deflection vs. Depth Pile 4 Pulling South
Assuming Stiff Clay, 1 inch**

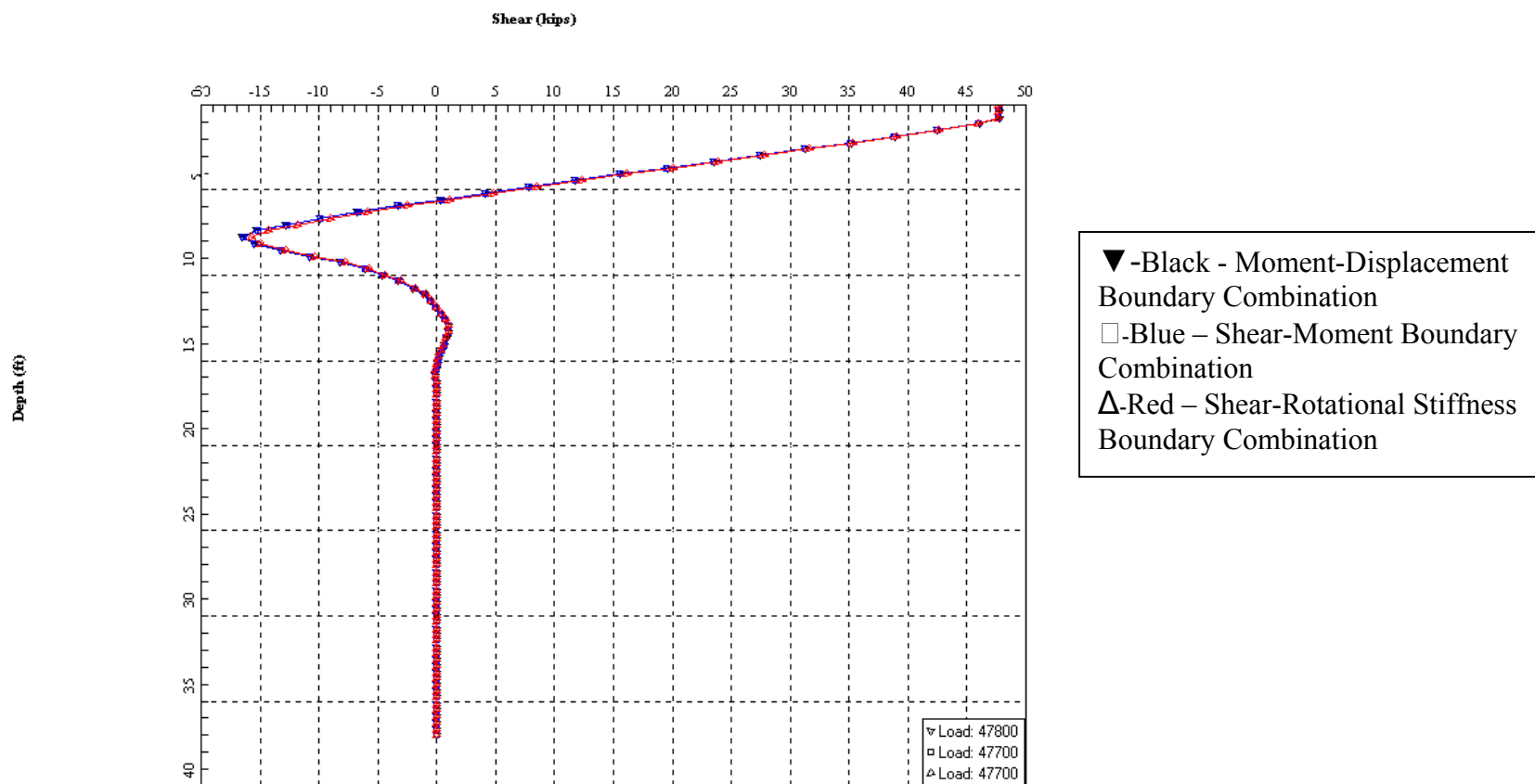


Figure A41: Shear vs. Depth Pile 4 Pulling South Assuming Stiff Clay, 1 inch

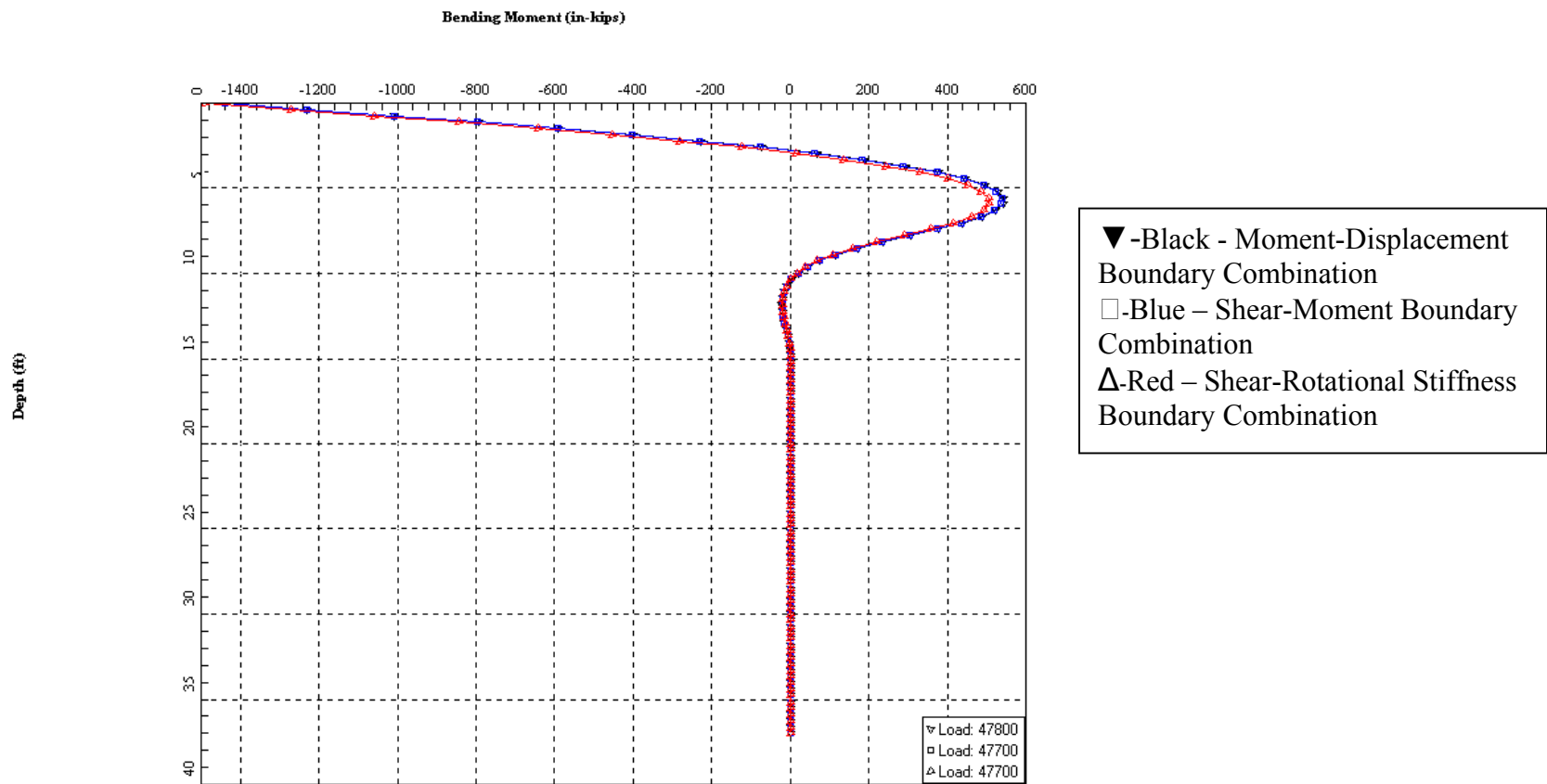


**Figure A42: Moment vs. Depth Pile 4 Pulling South
Assuming Stiff Clay, 1 inch**

APPENDIX B:

LPILE

B.1 Introduction

This appendix provides the reader with more details on LPILE. It gives an example test, in this case Pile 3, pulling south, 0.5 in., and additional information for that test. Figures B1 through B7 are the graphical user interfaces needed to enter the data, execute the analysis, and review the results. In section B.3, the input and output code is given. This code reviews the input information and presents the results in a tabular form. It is an alternate means of editing LPILE input and reviewing output; however, it is not the most user friendly. The graphs produced from LPILE analysis are presented in Appendix A.

B.2 Program User Graphics

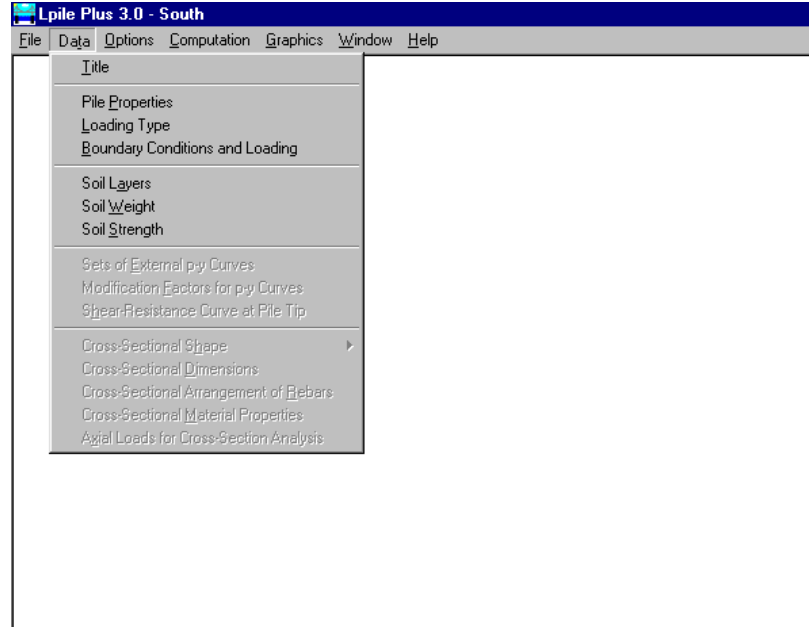


Figure B1: LPILE Menu to Enter Data for the Pile and Soil

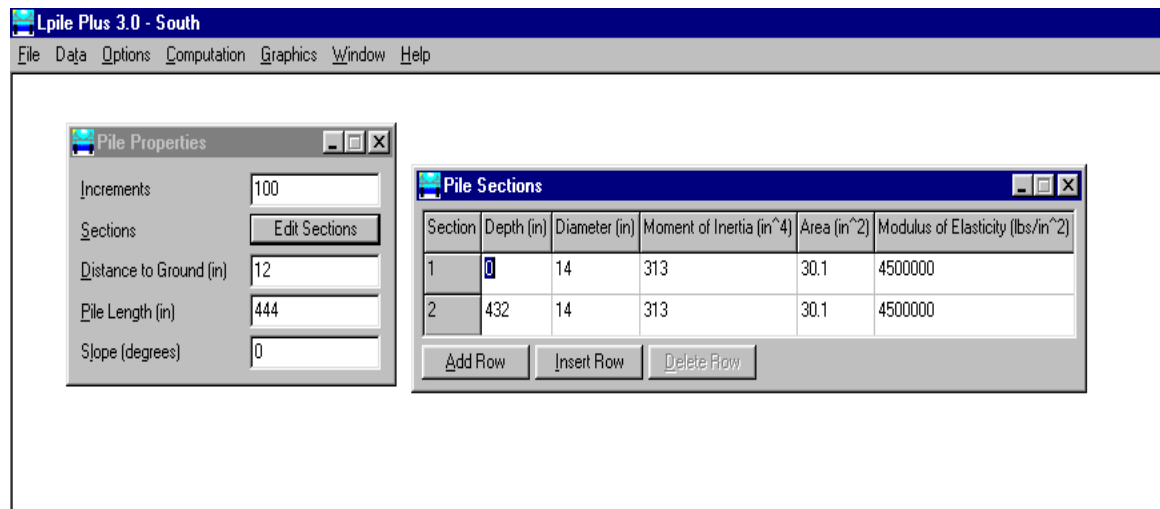


Figure B2: Pile Properties- Pile 3 0.5-inch South Post-“tweaked”

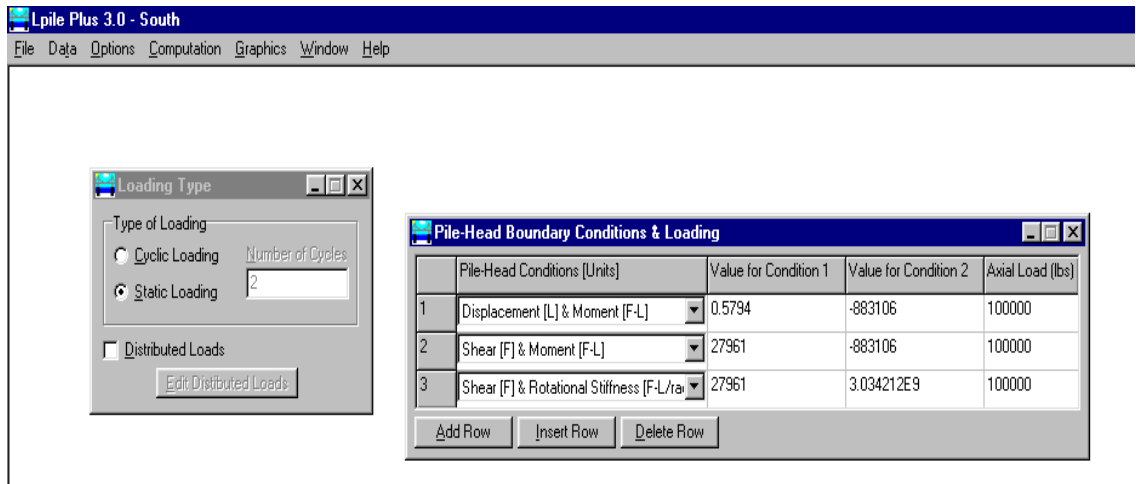


Figure B3: Loading Type and Boundary Conditions- Pile 3 0.5-inch South Post-“tweaked”

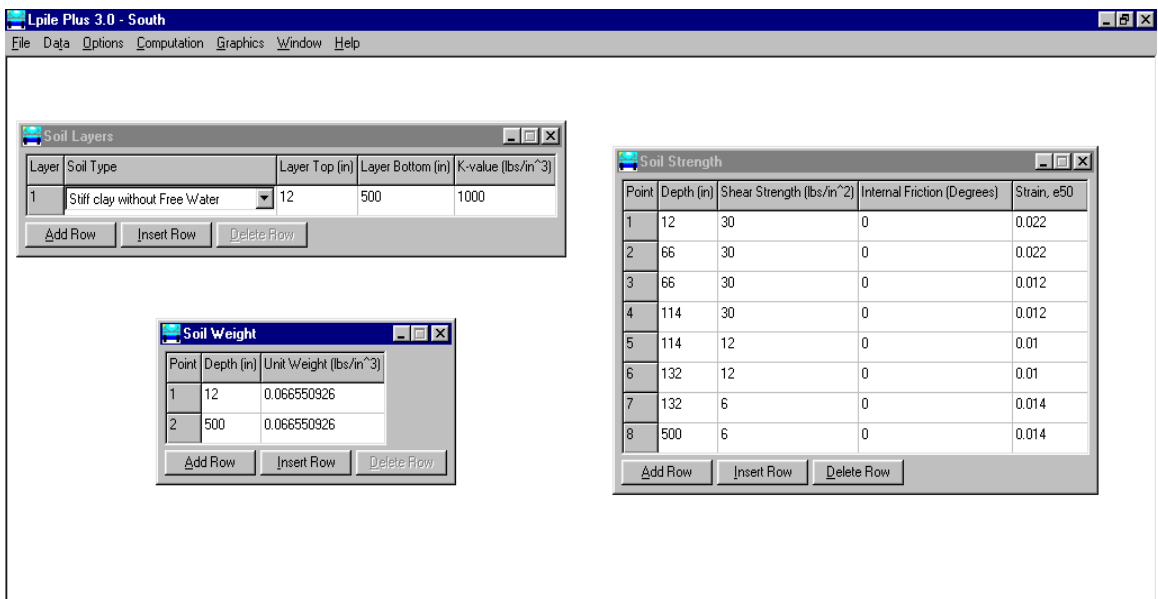


Figure B4: Soil Type- Pile 3 0.5-inch South Post-“tweaked”

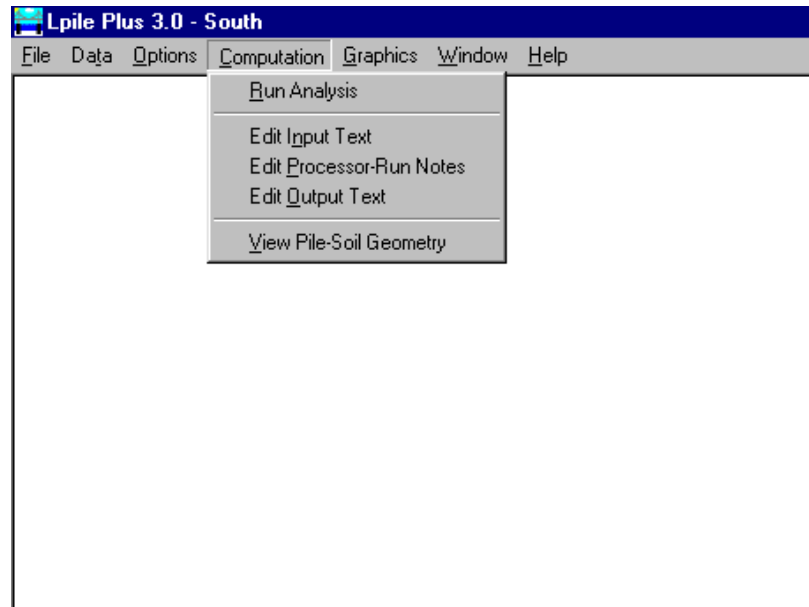


Figure B5: LPILE Menu to Execute Analysis

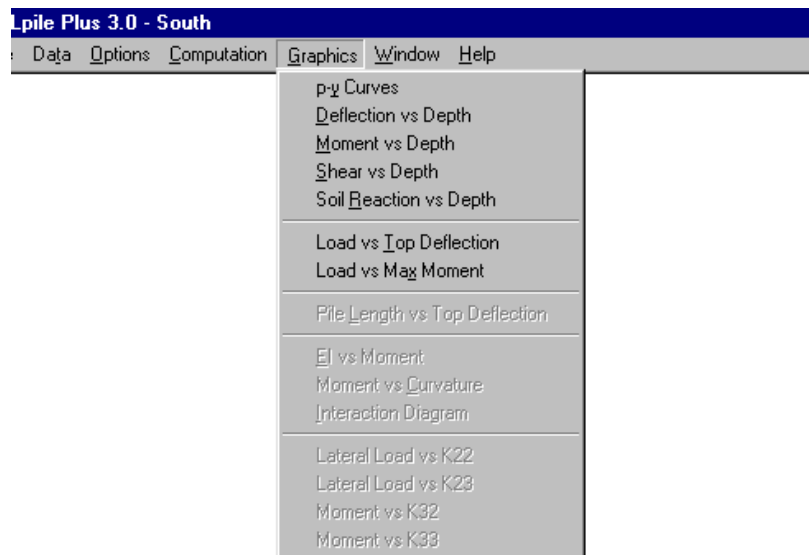


Figure B6: LPILE Menu to Review Graphical Results

B.3 Programming Code

PROGRAM LPILE plus Version 3.0
(C) COPYRIGHT 1997 ENSOFT, INC.
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New Pile

UNITS--ENGLISH UNITS

INPUT INFORMATION *****

THE LOADING IS STATIC

PILE GEOMETRY AND PROPERTIES -----

PILE LENGTH = 444.00 IN
2 POINTS

X	DIAMETER	MOMENT OF INERTIA	AREA	MODULUS OF ELASTICITY
IN	IN	IN**4	IN**2	LBS/IN**2
.00	14.000	.313E+03	.196E+03	.450E+07
432.00	14.000	.313E+03	.196E+03	.450E+07

SOILS INFORMATION -----

X AT THE GROUND SURFACE = 12.00 IN

SLOPE ANGLE AT THE GROUND SURFACE = .00 DEG.

4 LAYER(S) OF SOIL

LAYER 1

THE SOIL IS A STIFF CLAY WITH NO FREE WATER

X AT THE TOP OF THE LAYER = 12.00 IN

X AT THE BOTTOM OF THE LAYER = 66.00 IN

MODULUS OF SUBGRADE REACTION = .100E+04 LBS/IN**3

LAYER 2

THE SOIL IS A STIFF CLAY WITH NO FREE WATER

X AT THE TOP OF THE LAYER = 66.00 IN

X AT THE BOTTOM OF THE LAYER = 114.00 IN

MODULUS OF SUBGRADE REACTION = .100E+04 LBS/IN**3

LAYER 3

THE SOIL IS A STIFF CLAY WITH NO FREE WATER

X AT THE TOP OF THE LAYER = 114.00 IN

X AT THE BOTTOM OF THE LAYER = 132.00 IN

MODULUS OF SUBGRADE REACTION = .100E+04 LBS/IN**3

LAYER 4

THE SOIL IS A STIFF CLAY WITH NO FREE WATER

X AT THE TOP OF THE LAYER = 132.00 IN

X AT THE BOTTOM OF THE LAYER = 500.00 IN

MODULUS OF SUBGRADE REACTION = .100E+04 LBS/IN**3

DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH

8 POINTS

X,IN	WEIGHT,LBS/IN**3
12.00	.67E-01
66.00	.67E-01
66.00	.67E-01
114.00	.67E-01
114.00	.67E-01
132.00	.67E-01
132.00	.67E-01
500.00	.67E-01

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH

8 POINTS

X,IN	C,LBS/IN**2	PHI,DEGREES	E50
12.00	.300E+02	.000	.220E-01
66.00	.300E+02	.000	.220E-01
66.00	.300E+02	.000	.120E-01

114.00	.300E+02	.000	.120E-01
114.00	.120E+02	.000	.100E-01
132.00	.120E+02	.000	.100E-01
132.00	.600E+01	.000	.140E-01
500.00	.600E+01	.000	.140E-01

BOUNDARY AND LOADING CONDITIONS

LOADING NUMBER 1

BOUNDARY CONDITION CODE	=	4
DEFLECTION AT THE PILE HEAD	=	.579E+00 IN
MOMENT AT THE PILE HEAD	=	-.883E+06 IN-LBS
AXIAL LOAD AT THE PILE HEAD	=	.100E+06 LBS

LOADING NUMBER 2

BOUNDARY-CONDITION CODE	=	1
LATERAL LOAD AT THE PILE HEAD	=	.280E+05 LBS
MOMENT AT THE PILE HEAD	=	-.883E+06 IN-LBS
AXIAL LOAD AT THE PILE HEAD	=	.100E+06 LBS

LOADING NUMBER 3

BOUNDARY CONDITION CODE	=	3
LATERAL LOAD AT THE PILE HEAD	=	.280E+05 LBS
ROTATIONAL RESISTANCE AT THE PILE HEAD	=	.303E+10 IN-LBS
AXIAL LOAD AT THE PILE HEAD	=	.100E+06 LBS

FINITE-DIFFERENCE PARAMETERS

NUMBER OF PILE INCREMENTS	=	100
DEFLECTION TOLERANCE ON DETERMINATION OF CLOSURE	=	.100E-04 IN
MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS	=	100
MAXIMUM ALLOWABLE DEFLECTION	=	.10E+03 IN

OUTPUT CODES

KOUTPT = 1

KPYOP = 1

INC = 4

DEPTH	DIAM	C	CAVG	GAMMA	AVG	E50
IN	IN	LBS/IN**2	LBS/IN**3	LBS/IN**3		
.00	14.000	.30E+02	.30E+02	.67E-01	.220E-01	
	Y	P				
	IN	LBS/IN				
	.000E+00	.000E+00				
	.123E-03	.709E+02				
	.616E-03	.106E+03				
	.123E-02	.126E+03				
	.616E-02	.188E+03				
	.123E-01	.224E+03				
	.616E-01	.335E+03				
	.123E+00	.398E+03				
	.308E+00	.501E+03				
	.616E+00	.596E+03				
	.924E+00	.659E+03				
	.123E+01	.709E+03				
	.308E+01	.891E+03				
	.616E+01	.106E+04				
	.123E+02	.126E+04				
	.139E+02	.126E+04				
	.154E+02	.126E+04				

OUTPUT INFORMATION

*****

*****

* COMPUTE LOAD-DISTRIBUTION AND LOAD-DEFLECTION *

* CURVES FOR LATERAL LOADING *

*****

LOADING NUMBER 1

BOUNDARY CONDITION CODE	=	4
DEFLECTION AT THE PILE HEAD	=	.579E+00 IN
MOMENT AT THE PILE HEAD	=	-.883E+06 IN-LBS
AXIAL LOAD AT THE PILE HEAD	=	.100E+06 LBS

X DEFLECTION MOMENT SHEAR SLOPE TOTAL FLEXURAL
SOIL

	IN	IN	LBS-IN	LBS	RAD.	LBS/IN**2	LBS-IN**2	LBS/IN
	STRESS	RIGIDITY	REACTION					
	*****	*****	*****	*****	*****	*****	*****	*****
.0	.579E+00	-.883E+06	.316E+05	-.144E-02	.203E+05	.141E+10	.000E+00	
17.8	.475E+00	-.323E+06	.277E+05	-.901E-02	.773E+04	.141E+10	-.599E+03	
35.5	.295E+00	.896E+05	.166E+05	-.103E-01	.251E+04	.141E+10	-.643E+03	
53.3	.131E+00	.299E+06	.529E+04	-.763E-02	.721E+04	.141E+10	-.616E+03	
71.0	.314E-01	.309E+06	-.528E+04	-.360E-02	.742E+04	.141E+10	-.572E+03	
88.8	-.252E-02	.144E+06	-.112E+05	-.615E-03	.372E+04	.141E+10	.343E+03	
106.6	-.328E-02	.706E+04	-.399E+04	.206E-03	.668E+03	.141E+10	.409E+03	
124.3	-.279E-03	-.145E+05	.480E+03	.846E-04	.834E+03	.141E+10	.127E+03	
142.1	.111E-03	-.112E+04	.434E+03	-.771E-05	.535E+03	.141E+10	-.463E+02	
159.8	-.387E-09	.546E+03	-.102E+03	-.106E-05	.522E+03	.141E+10	.241E+01	
177.6	.149E-12	.807E-02	.109E+01	-.128E-10	.510E+03	.141E+10	-.491E+00	
195.4	.961E-26	.514E-15	.305E-12	-.812E-24	.510E+03	.141E+10	-.137E-12	
213.1	.619E-39	.327E-28	.196E-25	-.517E-37	.510E+03	.141E+10	-.885E-26	
230.9	.399E-52	.208E-41	.127E-38	-.329E-50	.510E+03	.141E+10	-.570E-39	
248.6	.257E-65	.133E-54	.816E-52	-.210E-63	.510E+03	.141E+10	-.368E-52	
266.4	.166E-78	.844E-68	.526E-65	-.133E-76	.510E+03	.141E+10	-.237E-65	
284.2	.107E-91	.537E-81	.339E-78	-.848E-90	.510E+03	.141E+10	-.153E-78	
301.9	.689-105	.341E-94	.218E-91	-.540-103	.510E+03	.141E+10	-.984E-92	
319.7	.444-118	.217-107	.141-104	-.343-116	.510E+03	.141E+10	-.634-105	
337.4	.286-131	.138-120	.907-118	-.218-129	.510E+03	.141E+10	-.409-118	
355.2	.184-144	.878-134	.584-131	-.139-142	.510E+03	.141E+10	-.263-131	
373.0	.119-157	.559-147	.376-144	-.883-156	.510E+03	.141E+10	-.170-144	
390.7	.765-171	.355-160	.243-157	-.561-169	.510E+03	.141E+10	-.109-157	
408.5	.493-184	.226-173	.156-170	-.357-182	.510E+03	.141E+10	-.704-171	
426.2	.318-197	.143-186	.101-183	-.227-195	.510E+03	.141E+10	-.454-184	
444.0	.409-210	.000E+00	.000E+00	-.287-208	.510E+03	.141E+10	-.585-197	

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = .644E-08
IN-LBS

THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = .223E-
08 LBS

OUTPUT SUMMARY

PILE-HEAD DEFLECTION = .579E+00 IN
COMPUTED SLOPE AT PILE HEAD = -.144E-02
MAXIMUM BENDING MOMENT = -.883E+06 LBS-IN
MAXIMUM SHEAR FORCE = .316E+05 LBS
NO. OF ITERATIONS = 23
NO. OF ZERO DEFLECTION POINTS = 35

LOADING NUMBER 2

BOUNDARY CONDITION CODE = 1
LATERAL LOAD AT THE PILE HEAD = .280E+05 LBS
MOMENT AT THE PILE HEAD = -.883E+06 IN-LBS
AXIAL LOAD AT THE PILE HEAD = .100E+06 LBS

X DEFLECTION MOMENT SHEAR SLOPE TOTAL FLEXURAL
SOIL

	IN	IN	LBS-IN	LBS	RAD.	LBS/IN**2	LBS-IN**2	LBS/IN
	.0	.222E+00	-.883E+06	.280E+05	.538E-02	.203E+05	.141E+10	.000E+00
	17.8	.236E+00	-.397E+06	.247E+05	-.266E-02	.940E+04	.141E+10	-.503E+03
	35.5	.158E+00	-.332E+05	.153E+05	-.522E-02	.125E+04	.141E+10	-.550E+03
	53.3	.700E-01	.160E+06	.560E+04	-.424E-02	.409E+04	.141E+10	-.527E+03
	71.0	.149E-01	.185E+06	-.333E+04	-.191E-02	.465E+04	.141E+10	-.475E+03
	88.8	-.142E-02	.707E+05	-.714E+04	-.208E-03	.209E+04	.141E+10	.298E+03
	106.6	-.780E-03	-.605E+04	-.154E+04	.989E-04	.645E+03	.141E+10	.286E+03
	124.3	.322E-04	-.280E+04	.660E+03	.338E-05	.573E+03	.141E+10	-.740E+02
	142.1	.105E-07	.282E+03	-.460E+02	-.497E-06	.517E+03	.141E+10	-.461E+01
	159.8	.541E-16	.106E-03	.169E-02	-.167E-12	.510E+03	.141E+10	-.773E-03
	177.6	.368E-29	.680E-17	.115E-15	-.107E-25	.510E+03	.141E+10	-.525E-16
	195.4	.249E-42	.437E-30	.780E-29	-.689E-39	.510E+03	.141E+10	-.356E-29

213.1	.168E-55	.281E-43	.527E-42	-.443E-52	.510E+03	.141E+10	-.240E-42
230.9	.113E-68	.181E-56	.356E-55	-.285E-65	.510E+03	.141E+10	-.162E-55
248.6	.763E-82	.116E-69	.239E-68	-.184E-78	.510E+03	.141E+10	-.109E-68
266.4	.512E-95	.749E-83	.161E-81	-.118E-91	.510E+03	.141E+10	-.732E-82
284.2	.343-108	.482E-96	.108E-94	-.759-105	.510E+03	.141E+10	-.490E-95
301.9	.230-121	.310-109	.722-108	-.488-118	.510E+03	.141E+10	-.328-108
319.7	.153-134	.199-122	.482-121	-.314-131	.510E+03	.141E+10	-.219-121
337.4	.102-147	.128-135	.322-134	-.202-144	.510E+03	.141E+10	-.146-134
355.2	.682-161	.825-149	.215-147	-.130-157	.510E+03	.141E+10	-.975-148
373.0	.454-174	.530-162	.143-160	-.836-171	.510E+03	.141E+10	-.649-161
390.7	.302-187	.341-175	.950-174	-.538-184	.510E+03	.141E+10	-.431-174
408.5	.201-200	.219-188	.631-187	-.346-197	.510E+03	.141E+10	-.287-187
426.2	.133-213	.141-201	.419-200	-.223-210	.510E+03	.141E+10	-.190-200
444.0	.175-226	.000E+00	.000E+00	-.286-223	.510E+03	.141E+10	-.250-213

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = .691E-08
IN-LBS

THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = -.162E-
08 LBS

OUTPUT SUMMARY

PILE-HEAD DEFLECTION = .222E+00 IN
 COMPUTED SLOPE AT PILE HEAD = .538E-02
 MAXIMUM BENDING MOMENT = -.883E+06 LBS-IN
 MAXIMUM SHEAR FORCE = .280E+05 LBS
 NO. OF ITERATIONS = 31
 NO. OF ZERO DEFLECTION POINTS = 37

LOADING NUMBER 3

BOUNDARY CONDITION CODE = 3
 LATERAL LOAD AT THE PILE HEAD = .280E+05 LBS
 ROTATIONAL RESISTANCE AT THE PILE HEAD = .303E+10 IN-LBS
 AXIAL LOAD AT THE PILE HEAD = .100E+06 LBS

X DEFLECTION MOMENT SHEAR SLOPE TOTAL FLEXURAL
SOIL

STRESS RIGIDITY REACTION

IN IN LBS-IN LBS RAD. LBS/IN**2 LBS-IN**2 LBS/IN

*****

.0	.423E+00	-.770E+06	.280E+05	-.254E-03	.177E+05	.141E+10	.000E+00
17.8	.350E+00	-.276E+06	.243E+05	-.681E-02	.669E+04	.141E+10	-.555E+03
35.5	.212E+00	.796E+05	.141E+05	-.787E-02	.229E+04	.141E+10	-.592E+03
53.3	.884E-01	.249E+06	.376E+04	-.561E-02	.609E+04	.141E+10	-.558E+03
71.0	.177E-01	.239E+06	-.565E+04	-.237E-02	.585E+04	.141E+10	-.495E+03
88.8	-.232E-02	.866E+05	-.838E+04	-.254E-03	.245E+04	.141E+10	.336E+03
106.6	-.136E-02	-.599E+04	-.208E+04	.143E-03	.644E+03	.141E+10	.328E+03
124.3	.110E-04	-.569E+04	.848E+03	.158E-04	.637E+03	.141E+10	-.564E+02
142.1	.713E-05	.512E+03	.300E+02	-.231E-05	.522E+03	.141E+10	-.233E+02
159.8	.701E-11	-.243E+00	.280E+01	.381E-09	.510E+03	.141E+10	-.124E+01
177.6	.450E-24	-.159E-13	.143E-10	.249E-22	.510E+03	.141E+10	-.643E-11
195.4	.290E-37	-.104E-26	.919E-24	.163E-35	.510E+03	.141E+10	-.414E-24
213.1	.186E-50	-.679E-40	.590E-37	.107E-48	.510E+03	.141E+10	-.266E-37
230.9	.120E-63	-.444E-53	.380E-50	.697E-62	.510E+03	.141E+10	-.171E-50
248.6	.769E-77	-.290E-66	.244E-63	.456E-75	.510E+03	.141E+10	-.110E-63
266.4	.494E-90	-.190E-79	.157E-76	.298E-88	.510E+03	.141E+10	-.706E-77
284.2	.318-103	-.124E-92	.101E-89	.194-101	.510E+03	.141E+10	-.454E-90
301.9	.204-116	-.808-106	.648-103	.127-114	.510E+03	.141E+10	-.292-103
319.7	.131-129	-.528-119	.417-116	.829-128	.510E+03	.141E+10	-.188-116
337.4	.844-143	-.344-132	.268-129	.541-141	.510E+03	.141E+10	-.121-129
355.2	.543-156	-.225-145	.172-142	.353-154	.510E+03	.141E+10	-.775-143
373.0	.349-169	-.147-158	.111-155	.230-167	.510E+03	.141E+10	-.498-156
390.7	.224-182	-.956-172	.711-169	.150-180	.510E+03	.141E+10	-.320-169
408.5	.144-195	-.624-185	.457-182	.980-194	.510E+03	.141E+10	-.206-182
426.2	.926-209	-.407-198	.294-195	.639-207	.510E+03	.141E+10	-.132-195
444.0	.119-221	.000E+00	.000E+00	.836-220	.510E+03	.141E+10	-.170-208

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = -.465E-07
IN-LBS

THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = .462E-
08 LBS

OUTPUT SUMMARY

PILE-HEAD DEFLECTION = .423E+00 IN
 COMPUTED SLOPE AT PILE HEAD = -.254E-03
 MAXIMUM BENDING MOMENT = -.770E+06 LBS-IN
 MAXIMUM SHEAR FORCE = .280E+05 LBS
 NO. OF ITERATIONS = 29
 NO. OF ZERO DEFLECTION POINTS = 36

SUMMARY TABLE
 *****

BOUNDARY CONDITION	BOUNDARY CONDITION	AXIAL LOAD	PILE HEAD DEFLECTION	MAX. MOMENT	MAX.
BC1	BC2	LBS	IN	IN-LBS	LBS
.5794E+00	-.8831E+06	.1000E+06	.5794E+00	-.8831E+06	.3160E+05
.2796E+05	-.8831E+06	.1000E+06	.2218E+00	-.8831E+06	.2796E+05
.2796E+05	.3034E+10	.1000E+06	.4228E+00	-.7698E+06	.2796E+05

APPENDIX C:

MATLAB

C.1 Introduction

This appendix provides the reader with more information on the MATLAB program that was written to compare LPILE results to a macro scale of pile results. In section C.2, the entire code is presented. In-line comments are included behind the percent sign (%). These in-line comments offer a brief explanation for each section of the code. In section C.3, a MATLAB walk-through is presented, with all of the external user-prompts in Figures C1 through C4. These external prompts are what a user is exposed to during program execution. Figure C5 is not a user-prompt; it reports results from the program in tabular form instead of graphical form. Also, the Figures 4 and 5 from Chapter 5 report results but are not included in this appendix.

C.2 Programming Code

%Be sure to have the L-Pile summary chart in the directory

```
clear all
clc
close all
```

```
ii=1;
```

```
%Disclaimer
```

```
tell=msgbox({'This program was developed from and is exclusive to the imperical data ',
'obtained at The University of Tennessee, Knoxville. The prestressed ',
'piles, which were driven 36 feet into virgin clay, were 14 inch square, ',
'6000 psi concrete, containing .5 inch diameter, Grade 270, low relaxation ',
'7-wire prestressing strands. It uses a regression method to fit ',
'Load-Deflection and Moment-Deflection curves. This program, which is ',
'valid for deflections between 0 inches and 2 inches was designed to give ',
'upper and lower bounds for "expected" shear and moments.'}, 'Notice', 'warn');
```

```
waitfor(tell);
```

```
button = questdlg('Do you want to compare the L-Pile shear results?', 'Yes', 'No');
```

```
%graphical user data input
```

```
dq = {'Enter the beginning Displacement in inches', 'Enter the ending Displacement in inches', ...
```

```
    'Enter the number of increments'};
```

```
dtile = 'Input for Pile Displacements';
```

```
nlines = 1;
```

```
def = {'0', '2', '20'};
```

```
answer = inputdlg(dq, dtile, nlines, def);
```

```
d1 = str2num(char(answer(1)));
```

```
d2 = str2num(char(answer(2)));
```

```
spac = str2num(char(answer(3)));
```

```
%Error checking
```

```
if d1>=0 & d2<=2
```

```
    vi=1;
```

```
    elseif d1<=0 | d2>=2
```

```

    vi=0;
end
    if d2<d1
        vi=0;

    end

while vi==0;
    hello1=msgbox({'Please enter values in the 0 to 2 inch range, and make',
        'sure they are in sequential order. (i.e. the ending displacement '
        'is greater than the beginning displacement.)'})

waitfor(hello1);

dq = {'Enter the beginning Displacement in inches','Enter the ending Displacement in
inches',...
    'Enter the number of increments'};
dtitle = 'Input for Pile Displacements';
nlines = 1;
def = {'0','2','20'};
answer = inputdlg(dq,dtitle,nlines,def);
d1 = str2num(char(answer(1)));
d2 = str2num(char(answer(2)));
spac = str2num(char(answer(3)));
    if d1>=0 & d2<=2
        vi=1;

    end
end

%+++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
+++++++
%calculating values
if vi==1

    x= linspace(d1,d2,spac);

    % Lower Bound zero to two inches.

    ypulllow= -0.4344*x.^6 + 4.7716*x.^5 - 20.55*x.^4 + 45.792*x.^3 - 56.086*x.^2 +
45.894*x + 1.2593;

```

```

    zmomentlow= -0.6619*x.^6 + 8.8337*x.^5 - 46.942*x.^4 + 125.11*x.^3 - 174.6*x.^2
+ 148.61*x + 4.4569;

```

```

%+++++
+++++

```

```

% Upper bound zero to one inch

```

```

%Moment for zero to two inches

```

```

    zmomenthigh= -43.9.*x.^6 + 323.69.*x.^5 - 910.9.*x.^4 + 1220.9.*x.^3 - 825.76.*x.^2

```

```

...
    + 349.11.*x + 7.848

```

```

%high pull load is a piece-wise curve

```

```

%control b1

```

```

if d1<1 & d2>1

```

```

    b1=(round(((1-d1)/(d2-d1))*spac));

```

```

end

```

```

if d1<1 & d2<1

```

```

    b1=spac;

```

```

end

```

```

if d1>1 & d2>1

```

```

    b1=0;

```

```

end

```

```

%Deflections between 0 and 1 inch

```

```

    newx1=x;

```

```

    ii=1;

```

```

%for ranges between 0 and 1

```

```

%where d1<1 & d2>1

```



```

if d1<1 & d2>1

for newx1 = linspace(d1,1,b1)

    ypullhigh1(ii)=-1583.7*newx1.^6 + 5429.2*newx1.^5 - 6900.1*newx1.^4 +...
        3856.2*newx1.^3 - 855.61*newx1.^2 + 125.56*newx1 + 0.7476;

    ii=ii+1;

    if ii>b1
        break
    end

end

ypullhigh=[ypullhigh1];
end

%for ranges where d2<1
if d1<1 & d2<1
for newx1 = linspace(d1,d2,b1)

    ypullhigh1(ii)=-1583.7*newx1.^6 + 5429.2*newx1.^5 - 6900.1*newx1.^4 +
        3856.2*newx1.^3 - 855.61*newx1.^2 + 125.56*newx1 + 0.7476;

    ii=ii+1;

    if ii>b1
        break
    end

end

ypullhigh=[ypullhigh1];
end

%+++++
+++++
%Deflections between 1 and 2 inches

%control b2
newx2=x;
ii=1;

```

```

if d1<1 & d2>1

    b2=(round(((1-d1)/(d2-d1))*spac));

end

if d1>1 & d2>1

    b2=spac;

end

if d1<1 & d2<1

    b2=0;

end

%when d1<1, a continuation for values above 1 inch

if d1<1
for newx2 = linspace(1.00001,d2,b2)

    if d2<=1
        ypullhigh2=[];
        break
    end

    ypullhigh2(ii) = 75;
    ii=ii+1;

    if ii>b2
        break
    end

end

end

end

%when d1>1 & d2>1

```

```

if d1>1 & d2>1
for newx2 = linspace(d1,d2,b2)

    if d1>=1
        ypullhigh1=[];
    end

    if d2<=1
        ypullhigh2=[];
        break
    end

    ypullhigh2(ii) = 75;
    ii=ii+1;

    if ii>b2
        break
    end

end

end

ypullhigh=[ypullhigh1 ypullhigh2];

%Ensure the High shears are higher than the low shears

[m,n]=size(ypullhigh);
[M,N]=size(ypulllow);
kk=1;
if ypulllow(kk) >= ypullhigh(kk)

    ypullhigh(kk) = ypulllow(kk) ;

    kk=kk+1;

end

```

```
%+++++
+++++
```

```
%Reading Excel file for L-pile shear results
[A] = xlsread('C:\Documents and Settings\All
Users\Documents\bt\Thesis\Matlab\Conc_Pile_summary', 'correct p-y curves')
```

```
halfinreg=[A(5,6) A(20,6) A(34,6) A(50,6) A(64,6) A(78,6) ];
oneinreg=[ A(9,6) A(24,6) A(38,6) A(54,6) A(68,6) A(82,6)];
```

```
halfintweak=[A(5,8) A(20,8) A(34,8) A(50,8) A(64,8) A(78,8) ];
oneintweak=[ A(9,8) A(24,8) A(38,8) A(54,8) A(68,8) A(82,8)];
```

```
xhalf=[.5 .5 .5 .5 .5 .5 ];
xone=[1 1 1 1 1 1];
```

```
%+++++
+++++
```

```
%Reading Excel file for L-pile moment results before tweaking
[B] = xlsread('C:\Documents and Settings\All
Users\Documents\bt\Thesis\Matlab\original_moment_disp_comparison', 'moment disp
comparison')
```

```
begin_halfinmom=[B(8,5) B(9,5) B(12,5) B(13,5) B(16,5) B(17,5) ];
begin_oneinmom=[ B(10,5) B(11,5) B(14,5) B(15,5) B(18,5) B(19,5)];
```

```
%+++++
+++++
```

```
%Reading Excel file for L-pile moment results after tweaking
[c] = xlsread('C:\Documents and Settings\All
Users\Documents\bt\Thesis\Matlab\tweaked_moment_disp_comparison', 'moment disp
comparison')
```

```
halfinmom=[c(8,5) c(9,5) c(12,5) c(13,5) c(16,5) c(17,5) ];
oneinmom=[ c(10,5) c(11,5) c(14,5) c(15,5) c(18,5) c(19,5)];
```

```
xhalfmom=[.5 .5 .5 .5 .5 .5 ];
```

```

xonemom=[1 1 1 1 1 1];

%+++++
+++++

clc

%tabulating data
table=[x' ypulllow' ypullhigh' zmomentlow' zmomenthigh'];
fprintf('\n\n\nThe expected moments and shears at the given displacements \n\n');
fprintf('          Low          High          Low          High\n');
fprintf(' Displacement  Shear      Shear      Moment      Moment\n');
fprintf('   (in.)      (kips)      (kips)    (k-ft)    (k-ft)\n');
fprintf('-----      -----      -----      -----      -----\n');
fprintf('    %5.2f      %5.2f      %5.2f    %5.2f    %5.2f\n',table');

pause(1)

%plotting data
scrsz = get(0,'ScreenSize');
figure('Position',[1 scrsz(4) scrsz(3) scrsz(4)])
figure(1);
hold on

if strcmp(button,'Yes')

    plot(x, ypulllow, 'k-', x, ypullhigh, 'r--' );
    plot(xhalf,halfinreg, 'r^', xhalf,halfintweak, 'bo', xone, oneintweak, 'g+', xone,
oneinreg, 'k*');
    xlabel('Displacement (inches)');
    ylabel('Shear (kips)');
    title('The maximum and minimum expected values of shear');
    legend('Low Shear','High Shear', 'Half inch L-Pile prediction', 'Half inch L-Pile
tweaked',...
'One inch L-Pile prediction', 'One inch L-Pile tweaked',4)

elseif strcmp(button,'No')

    plot(x, ypulllow, 'k-', x, ypullhigh, 'r--' );

```

```

xlabel('Displacement (inches)');
ylabel('Shear (kips)');
title('The maximum and minimum expected values of shear');
legend('Low Shear','High Shear',4)
end

scrsz = get(0,'ScreenSize');
figure('Position',[1 scrsz(4) scrsz(3) scrsz(4)])
figure(2);
hold on

if strcmp(button,'Yes')

    plot(x, zmomentlow, 'k-', x, zmomenthigh, 'r-- ');
    plot(xhalfmom,begin_halfinmom, 'bo', xhalfmom, halfinmom, 'r^',
xonemom,begin_oneinmom, 'g+',xonemom, oneinmom, 'k*');
    xlabel('Displacement (inches)');
    ylabel('Moment (kip-feet)');
    title('The maximum and minimum expected values of moment');
    legend('Low moment values','High moment values', 'Beginning Half inch L-Pile
prediction',...
'Tweaked Half inch L-Pile prediction', 'Beginning One inch L-Pile prediction',...
'Tweaked One inch L-Pile prediction',4)
    text(.2,150,'Four values have values of 167 kip-ft.')

elseif strcmp(button,'No')

    plot(x,zmomentlow,x,zmomenthigh);
    xlabel('Displacement (inches)');
    ylabel('Moment (kip-feet)');
    title('The maximum and minimum expected values of moment');
    legend('Low moment values','High moment values',2);
    fprintf('\n\n\n');
end

msgbox({'You have two graphs, Figures 1 and 2, for a graphical',
'representation and a table in the Matlab Command Window.'});

end

```

C.3 User Interfaces

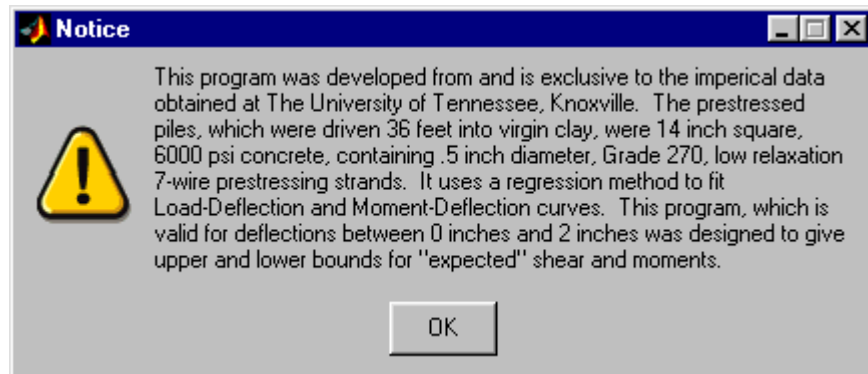


Figure C1: Background and Purpose for the Program

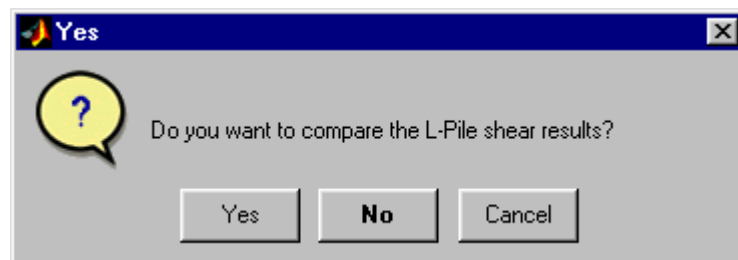


Figure C2: Inquiry for LPILE Results to be Graphed

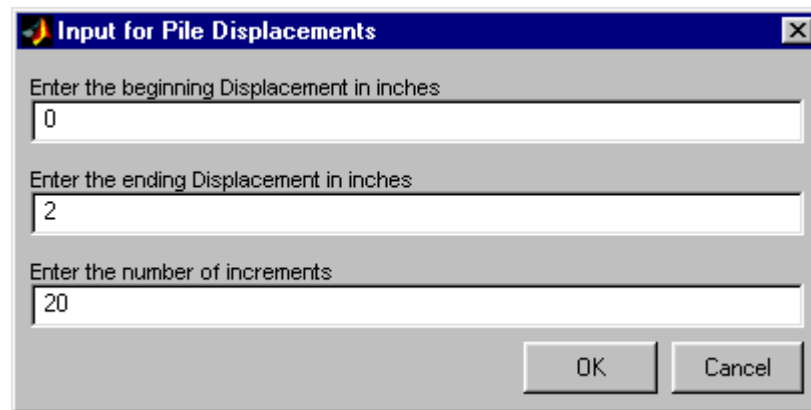


Figure C3: Dialog Box for Displacement Range and Increments

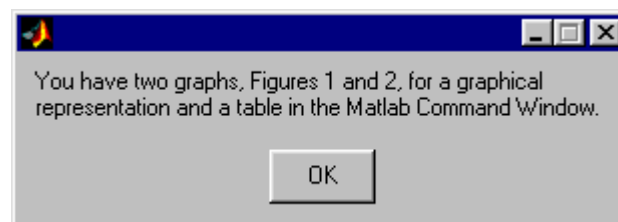


Figure C4: Conclusion Box Informing that Program is Finished

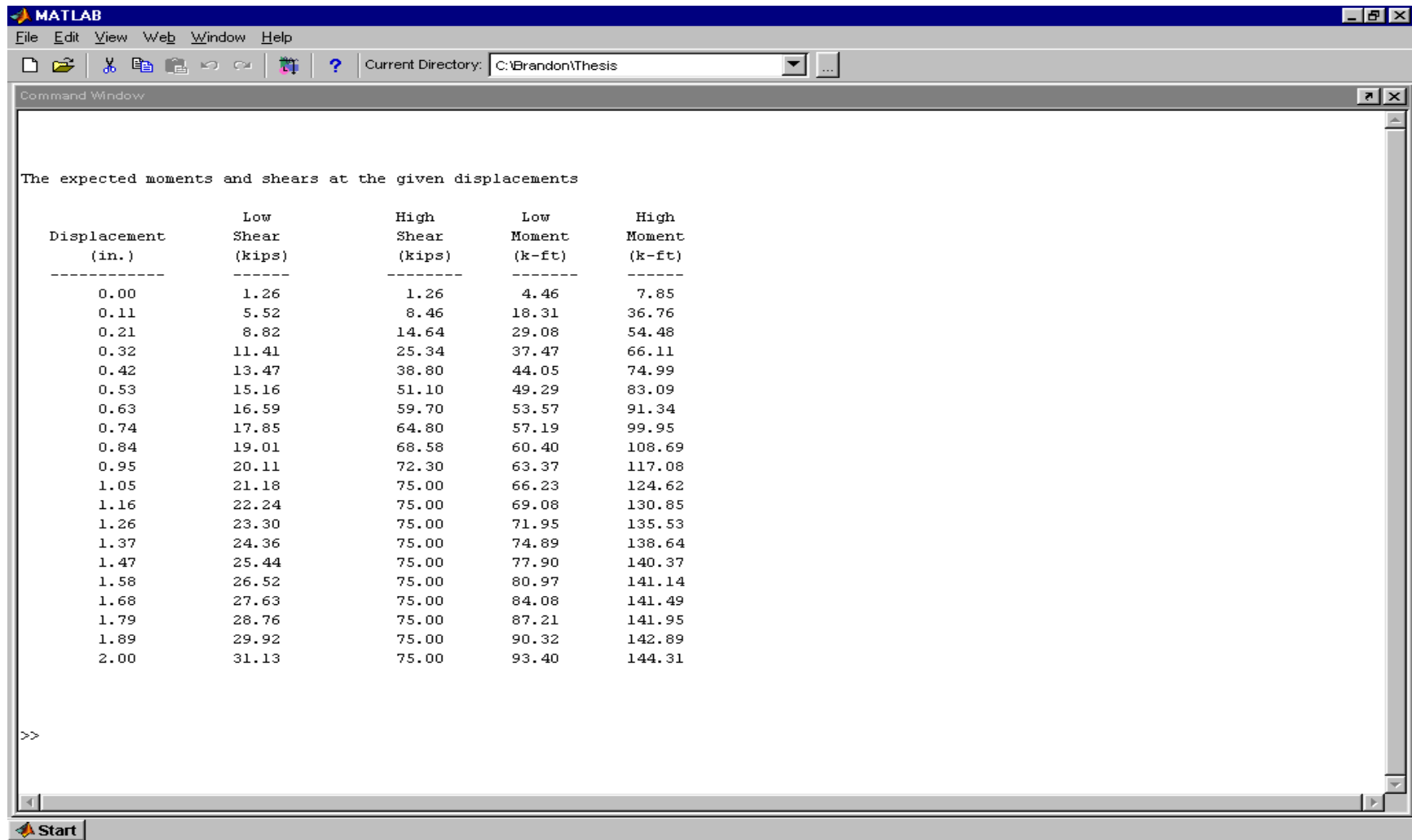


Figure C5: MATLAB Command Window Displaying Requested Results

VITA

Jarrell Brandon Tidwell was born in Nashville, Tennessee on April 14, 1980. He was reared in Bon Aqua, Tennessee. He graduated from Hickman County High School in May of 1998. In the fall of that same year, he began his undergraduate studies at The University of Tennessee. In December of 2002, he graduated with a Bachelor of Science degree in Civil Engineering, and in the following semester, Spring of 2003, he enrolled into Graduate School at The University of Tennessee. He received his Master of Science degree in Civil Engineering in May of 2004.