To the Graduate Council:

I am submitting herewith a thesis written by James Charles Bernardi entitled "Behavior of Prestressed Concrete Piles Subjected to Slowly Applied Lateral Loads." I have examined the final copy of this thesis for form and content and recommend that it be accepted in partial fulfillment of the requirements for the degree of Master of Science, with a major in Civil Engineering.

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We have read this thesis and
recommend its acceptance:

Accepted for the Council:

Associate Vice Chancellor and
Dean of the Graduate School

BEHAVIOR OF PRESTRESSED CONCRETE PILES SUBJECTED TO SLOWLY APPLIED LATERAL LOADS

A Thesis

Presented for the

Master of Science Degree

The University of Tennessee, Knoxville

James Charles Bernardi

August, 1996

ACKNOWLEDGMENTS

I would like to take this opportunity to thank the many people responsible for making this thesis possible. Dr. Edwin G. Burdette, my major professor, has provided me with guidance and encouragement since the beginning of my undergraduate studies. I have tremendous respect for Dr. Burdette and I will always be grateful for his help. I would also like to thank my other committee members, Dr. J. Harold Deatherage and Dr. David W. Goodpasture. Both of these professors have contributed greatly to the research work for this thesis and to my academic studies as well.

There are several graduate students who have also provided assistance for this research project. I would like to thank Kevin Bowling, Richard Graves, Sam Teague and Laura Arico for their time and help with this project.

I would also like to express my appreciation to the Tennessee Department of Transportation. The funding provided by TDOT made this research work possible.

ABSTRACT

The Tennessee Department of Transportation uses prestressed, precast, concrete piles to support many of the jointless bridges across Tennessee. Due to the wide range of thermal changes experienced in Tennessee, these prestressed concrete piles may be subjected to deflections of 1.0 inch or more at the ground level. Concerns about the ability of prestressed concrete piles to withstand deflections of 1.0 inch or more and maintain their structural integrity led to field research conducted by The University of Tennessee Knoxville under contract with the Tennessee Department of Transportation and sub-contract with the University of Memphis.

Three prestressed concrete piles were subjected to a series of slowly applied lateral load tests. Testing conducted with two piles included long term load tests and cyclic load tests. The third pile was subjected to a long term load test, a cyclic test, and a test to ultimate.

The results of the tests conducted showed that prestressed concrete piles are capable of withstanding deflections of 1.0 inch or more and still maintain their structural integrity. The test results also led to the conclusion that piles subjected to a slower rate of loading led to behavior that was less stiff. The test to ultimate showed the extremely ductile behavior of a prestressed pile, in that it was capable of withstanding very large deflections before the structural integrity of the pile was compromised.

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CHAPTER 1

INTRODUCTION

The Tennessee Department of Transportation (TDOT) has been using jointless bridges as a part of their roadway system since the 1960's. These bridges consist of single and continuous span structures up to 200 ft. lengths with no expansion/contraction capabilities and continuous span structures with lengths from 200 ft. to lengths in excess of 2,000 ft. with expansion/contraction capabilities provided only at the abutments. Precast, prestressed concrete piles are used for many of these structures as pile groups supporting large bridge piers, end bearing and friction piles supporting the superstructure at various spans, and as piles supporting the bridge abutments (1).

In recent years, there has been a growing concern about the effects of seismic activity on jointless bridges, and in particular the behavior of the prestressed concrete piles in response to seismic loading. This concern is especially evident in the western regions of Tennessee which are located in seismic zones 2 or 3, representing a high risk for seismic activity. In an effort to better understand the behavior of prestressed concrete piles subjected to seismic loading, the Tennessee Department of Transportation, under contract with The University of Tennessee Knoxville and sub-contract with the University of Memphis, funded a research program to investigate this behavior. The first phase of this research began September 15, 1993, and ended September 30, 1994. The results of this first phase of research were reported by Cook (2), and by Burdette et al. (3). The second phase of research began August 15, 1994, and ended June 30, 1996. The

results of the dynamic testing during the second phase were reported by Graves (4) and Bowling (5).

Midway through the second phase, a decision was made to expand the scope of research to include testing of piles under slowly applied lateral loads. This decision stemmed from concern not only regarding the effects of seismic activity but also the effects of the wide range of thermal variations experienced in Tennessee. As a result of this wide range of thermal change, prestressed concrete piles may be subjected to deflections of 1 in. or more at the ground surface. A main concern of TDOT is the ability of prestressed concrete piles to withstand deflections of 1 in. or more and still maintain their integrity.

Four precast, prestressed concrete piles were located at a bridge construction site for proposed State Route 385 over the Loosahatchie River in Shelby County, Tennessee. One of the piles was subjected to the ongoing dynamic testing for the second phase of research. The remaining three piles were subjected to slowly applied lateral loads with various rates of loading, and a series of cyclic loading. Then, one of the piles was tested to ultimate.

The objectives of the research reported herein were to investigate the ability of prestressed concrete piles to withstand deflections of 1 in. or more under lateral loads and maintain their integrity, to compare the load-deflection behavior of piles subjected to various rates of loading as well as cyclic loading, and to examine the load-deflection behavior of a pile loaded to ultimate.

CHAPTER 2

REVIEW OF LITERATURE

Jointless bridges have been in use in the United States since the 1930's. With the ever increasing application of jointless bridges, research continues to evaluate the behavior of piles supporting the piers and abutments. The vast majority of research conducted has been with steel H-piles rather than with prestressed concrete piles. No research results were found that dealt with lateral loading of prestressed concrete piles founded in loess.

Tennessee adopted the use of continuous bridges with integral abutments (jointless bridges) in the mid-1960's. Over 1000 jointless bridges have been constructed in Tennessee, with calculated movements at the abutments ranging from 0.25 inch to 2 inches (1).

In 1994-1995, dynamic "pluck" tests were conducted, using prestressed concrete piles, in Shelby County, Tennessee, by The University of Tennessee, Knoxville, and the University of Memphis. The results of this research were reported in M.S. theses at The University of Tennessee, Knoxville, by Graves (4) and Bowling (5).

Bowling used a computer program (SEISAB) to model the response of jointless bridges subjected to lateral loading based on the results of data obtained from field tests. One of the bridges modeled was the Loosahatchie bridge located in Shelby County, Tennessee. The Loosahatchie bridge is 1130 ft. long with continuous spans and is supported by prestressed concrete pile bents. The bridge is jointless with

expansion/contraction capabilities provided at the abutments only. Seismic analysis of the bridge showed deflections of 1.6 ft. along the longitudinal direction of the bridge. Deflections of this magnitude translate to deflections of 1.0 in. or more at the ground level for piles supporting the superstructure close to the abutments.

The method of pile loading for the tests conducted by Graves and Bowling was essentially the same as it was for the testing reported herein. Piles tested by Graves and Bowling had a 10 kip pile cap which acted as an axial load. The piles tested for this research had no axial load applied. Results of the tests reported by Graves included static (secant) stiffness. For deflections ranging from 0.10 in. to 0.85 in., the static stiffness ranged from 55 kips/inch to 24 kips/inch.

Testing conducted to determine the feasibility of using prestressed concrete piles in jointless bridges was reported in a paper entitled "Prestressed Concrete Piles in Jointless Bridges", by Mounir et al. (6). The concrete piles tested were 12 in. square, one with 9 in. pitch spiral reinforcement and the other with 3 in. pitch spiral reinforcement. A concrete pile cap was cast for both piles to simulate the bridge abutment. The embedment length of the piles was 24 in. into the concrete cap. The piles were subjected to lateral loads. The load-deflection behavior for the pile with 9 in. pitch spiral reinforcement was almost linear from 0.10 in. up to 1.0 in. At approximately 1.3 in. of deflection and corresponding load of 10 kips, the load deflection curve flattened out. The pile was deflected to 2.1 in. while the load never exceeded 10 kips. The pile with the 3 in. pitch spiral reinforcement was deflected to 1.3 in. with a maximum load of approximately 9.5 kips obtained at 1.1 in. of deflection. Mounir concluded that

prestressed concrete piles were suitable for use in integral abutment bridges with a total allowable movement of 2.3 in. if the specific type of abutment joint used during his testing was incorporated.

Field lateral load tests were performed on embankment piles at three bridge sites in northern California (7). The piles tested were 10.75 in. O.D. steel pipe piles embeded to a depth of 28 ft. Several types of lateral load tests were performed including free-head tests. Lateral loads were applied at the ground surface between a pair of test piles with the use of a hydraulic jack. Lateral deflections were measured at the tops of the piles with deflection transducers. Testing was conducted by two different methods. One method consisted of induced deflections of 0.5 in. increments which were maintained for 2 hours each, and the second method consisted of induced loads of 5 kip increments maintained for a minimum of 2 hours per increment or until the pile head deflection was less than 0.010 in. per hour. Plots of load versus deflection demonstrated behavior that was almost linear. For the method of induced deflection, a load of 24 kips was recorded for a deflection of 1.0 in.; however, the plot indicates induced deflections of 0.25 in. increments. For the method of induced load, the same load produced a deflection of approximately 0.75 in. which demonstrates behavior that is stiffer for a given deflection. The time required to reach 1.0 in. of deflection was approximately 6 hours for the method of induced deflection, while the time required to reach 0.75 in. of deflection was approximately 8 hours for the method of induced load. After unloading the piles, the amount of residual deflection ranged between 0.31 in. to 0.58 in.

CHAPTER 3

DESCRIPTION OF TEST PROGRAM

3.1 INTRODUCTION

A total of six tests were conducted over a three day period. The first set of tests was conducted June 23, 1995, and the second set of tests was conducted on August 10 and August 11, 1995. Three piles were used during the course of testing. Testing conducted June 23 consisted of two slowly applied load tests with different rates of loading. Testing conducted August 10 consisted of one slowly applied load test and one cyclic test. Testing conducted August 11 consisted of two cyclic tests and one test to ultimate.

3.2 SITE LOCATION AND DESCRIPTION

Testing was conducted in Shelby County, Tennessee. Four precast, prestressed concrete piles were driven at a construction site beneath one of the bridge spans for proposed State Route 385 over the Loosahatchie River (Figure 1)* . The four piles are identified as A, B, C, and D for future reference to the particular tests associated with each pile. The static tests described herein were conducted using piles A, B, and C, and dynamic testing, mentioned earlier, was conducted using pile D.

The test site was located in a low lying area adjacent to the river. A large portion of this area is marshy or swampy with pockets of free standing water, especially after

* All figures and tables are located in the appendices.

prolonged periods of rain. As a result, Pile A was situated in approximately 4 in. of standing water during testing. Piles A and B were driven approximately 24 ft. apart. Piles C and D were driven approximately 38 ft. apart with a load reaction frame positioned midway between. Supporting bridge piles were located approximately 24 ft. from the piles tested.

3.2.1 DESCRIPTION OF PRESTRESSED PILES

All piles tested were precast, prestressed concrete driven to depths of approximately 30 - 35 ft. Each pile was 14 in. square and prestressed with six, 0.5 in. diameter, 270 k, 7 - wire strands. Pile A extended to a height of approximately 12 ft. above ground level, and pile B extended to a height of approximately 10.75 ft. above ground level. Pile C had been previously cut off to a height of approximately 3 ft. above ground level. Pile C was originally intended for dynamic testing and was cut off to accommodate an apparatus designed for the dynamic tests. Due to complications associated with the dynamic testing, it was decided to include pile C as part of the static testing.

3.2.2 DESCRIPTION OF SOIL

Test piles were located in a soil classified as loess, a soil type typical to the western regions of Tennessee. Loess is the result of aeolin deposits consisting mainly of silt and silt-size particles with a clay coating. This clay coating over the silt-size particles is a primary contributor to the cohesive property of loess; however, the precipitation of

chemicals in leached rain water may also add to the cohesion. Loess is normally a very stable soil in an unsaturated state; however, when it becomes saturated, the soil particles lose their binding strength and the soil collapses (12). Soil testing at the site was performed by Hall, Blake and Associates, Inc. of Memphis, Tennessee during November, 1995. Results of the boring samples taken indicated typically 8 - 11 ft. of silty clay, 17 - 19 ft. of sand, and 6 - 7 ft. of gravelly sand overlying a stiff clay. A typical soil profile is given as Figure 4.

3.3 TEST SETUP FOR PILES A AND B

Slowly applied load tests and cyclic tests were conducted using piles A and B. The setup for both methods of testing was the same; however, a computer was also used to monitor load and deflection during the cyclic tests. Both piles were tested simultaneously.

The piles were located approximately 24 ft. apart, and each pile was driven to a depth of approximately 35 ft. Pile A was skewed approximately 5 degrees and pile B was skewed approximately 14 degrees to the application of load. Wooden shims were used to compensate for the skew.

Load was applied to pile A 1.33 ft. above ground level and to pile B 1.08 ft. above ground level. This difference was due to the difference in ground elevation at the two piles. The loading apparatus consisted of 8 in. x 6 in. x 0.5 in. structural steel tubes 34 in. long placed against the backside of the piles. The wooden shims, used to compensate for skew, were placed between the steel tubes and the piles. The steel tubes were connected

with Dywidag post-tensioning steel threadbars, 1 in. nominal diameter, having an ultimate strength of 127.5 kips. The Dywidag bars were manufactured by Dywidag Systems International (10). Two steel tubes were used at pile A. This allowed for the placement of a hydraulic jack and a 100 kip load cell between the two steel tubes. Load was applied with the use of a hydraulic hand pump and was monitored by a Vishay strain indicator connected to the 100 kip load cell (Figure 3). During the cyclic tests the load was also monitored by computer.

Deflection was monitored with the use of dial indicators during the slowly applied load tests and with both dial indicators and linear variable displacement transducers (LVDT's) during the cyclic tests. Three dial indicators were positioned on each pile. Two indicators were positioned on the back side of each pile 2.25 in. from the outer edges. Indicators were positioned approximately 27 in. above ground level at pile A and approximately 23 in. above ground level at pile B. One indicator was centered on the side of each pile to aid in detecting any rotation of the pile during testing. One LVDT was centered on the back side of each pile at approximately the same height as the dial indicators.

During the cyclic tests, the load and deflection were recorded by connecting the leads from the load cell and LVDT's to an acquisition board which was connected to a Dell 486 DX computer. A trace of load versus time and a trace of deflection versus time were produced. From these traces, the load and corresponding deflection could be determined at various time intervals.

3.4 TEST SETUP FOR PILE C

A slowly applied load test, a cyclic test, and a test to ultimate were conducted using Pile C. The test setup for all three methods of testing was the same; however, as with the testing of piles A and B, a computer was also used to monitor load and deflection during the cyclic and ultimate test.

The loading apparatus was similar to that used for piles A and B. Since pile C was tested individually, the use of a steel load frame was necessary. Structural W shapes were bolted together and anchored to a 8 ft. x 8 ft. x 3 ft. concrete pad (Figure 4). The load frame was located approximately 21.6 ft. from the pile. Load was applied in the same manner as it was for piles A and B with a hydraulic jack, hand pump, Dwidag post-tensioning bars and structural tubing braced against the back side of the pile and load frame. The load was monitored with a Vishay strain indicator connected to a 25 kip load cell, and also with the computer.

Three dial indicators were positioned on the back side of the pile to monitor deflection during the slowly applied load test. Two indicators were positioned approximately 25 in. above ground level. One indicator was centered on the pile and the other located 2.25 in. from the edge to aid in detecting any rotation of the pile. A third indicator was positioned in the center of the pile approximately 7 in. above ground level. This indicator was used to monitor deflection as close to ground level as possible. For the cyclic and ultimate tests, two LVDT's , positioned at approximately the same height as the dial indicators, were also used to monitor deflection with the aid of the computer.

One LVDT was positioned at the height of the two upper dial indicators and one was positioned at the height of the lower dial indicator.

As with the testing for piles A and B, the load cell and LVDT's were connected to the computer, and traces of load versus time and deflection versus time produced.

3.5 DATA ACQUISITION

All of the tests performed were deflection controlled tests. For the slowly applied load tests, load was applied until a target deflection was reached. When the target deflection was reached, loading was discontinued. Load and deflection were recorded and the deflection maintained for a predetermined time interval. At the end of the time interval, load and deflection were again recorded, and then loading once again was applied until the next target deflection was reached. This procedure was repeated until the maximum target deflection was reached.

For the cyclic tests, as with the slowly applied load tests, load was applied until a target deflection was reached. When the target deflection was reached, loading was discontinued. Load and deflection were recorded and the load maintained for a few seconds. The load was then slowly released allowing the pile to return to its original position with minimal residual deflection. This residual deflection was recorded and load applied again until the next target deflection was reached. This procedure was repeated until the maximum target deflection was reached.

The test to ultimate, defined as the point where an increase in deflection resulted in a continuous decrease in load, was conducted in the same manner as the slowly

applied load tests. Load was applied until a target deflection was reached and the load and deflection recorded. The load was not maintained for a given time interval, but was immediately increased until the next target deflection was reached. This procedure was repeated until the pile had deflected to the point where safety and damage to test equipment became a concern.

CHAPTER 4

COM624P COMPUTER PROGRAM

4.1 INTRODUCTION

Computer programs are very useful tools for engineers, especially in the design and analysis of bridges. TDOT currently uses a computer program developed by Ensoft, Inc. of Austin, Texas to aid in the design and analysis of Tennessee's bridges. This program, called COM624P, is a laterally loaded pile analysis program for the microcomputer (11). An analysis of the piles tested for this research was performed using COM624P and the results of the computer analysis compared to the data obtained from the field research.

4.2 DESCRIPTION OF COM624P

COM624P was developed for use in the analysis of deflections and stresses of piles subjected to lateral loads. The program is based on the p-y method of analysis which allows the deflected shape of the pile to be computed through an iterative process even though the soil reaction along the pile is a nonlinear function of the pile deflection. Once the deflection along the length of the pile is known, the shear, bending moment, and soil resistance can be computed. The three most important factors in determining p-y curves are the pile geometry, the properties of the soil, and the type of loading.

Variables associated with the pile geometry include:

- the total length of the pile (in.)

- embedment length (in.)
- pile modulus (psi)
- moment of inertia (in^4)
- cross sectional area (in^2)
- pile diameter (in.)

Variables associated with the soil properties include:

- type of soil (clay, sand, etc.)
- depth of soil layers (in.)
- soil modulus, k (pci)
- average undrained shear strength, c (psf)
- unit weight, γ (pcf)
- internal friction angle for sand, ϕ (degrees)
- the strain corresponding to one-half the maximum principal stress difference, $\varepsilon 50$

The help menu of COM624P provides tables with typical values of soil parameters for various types of soil; however, in order to model a pile with the computer program in a way that accurately represents the actual field conditions, extensive soil tests need to be conducted in the field and in the lab to determine the soil properties. Tests recommended for determining soil properties include (11):

- in-situ vane-shear tests with parallel sampling for soil identification
- typical lab tests to determine the unit weight of the soil

- unconsolidated-undrained triaxial compression tests having a confining stress equal to the overburden pressure with c being defined as half the total maximum principal-stress difference
- miniature vane tests of samples in tubes
- unconfined compression tests

The variables which seem to have the greatest influence on the accuracy of the computer model are the values of the average undrained shear strength and the strain corresponding to one-half of the maximum principal stress difference. Great care should be taken to obtain these values as accurately as possible.

COM624P makes it possible to model a pile under various boundary conditions with respect to the type of loading. The pile can be modeled as free head, partially fixed, or fixed head with known values input for shear and moment, shear and rotational restraint, or shear and slope, respectively. The pile analysis performed for this research was conducted with the pile modeled as free head with zero moment and shear values corresponding to the lateral loads applied during field testing. The application of load was at the same height above ground level as that for the tests conducted using pile C. Values of the pile geometry and material properties were input into the program, and values of the soil properties furnished by Hall, Blake and Associates, Inc., were also input. The program was run and the deflections at the pile head compared to those recorded during the actual field testing.

An input/output file of one of the computer runs is included as Appendix C.

CHAPTER 5

PRESENTATION OF RESULTS

5.1 INTRODUCTION

A total of six tests were performed. Three tests were conducted using piles A and B, and three tests were performed on pile C. Plots of load versus deflection and stiffness versus deflection were developed for each set of tests. The test results are presented in the order in which the test was performed.

5.2 RESULTS OF TEST 1

Test 1 was conducted using piles A and B. The test was conducted at a loading rate of 0.10 in. every 15 minutes over a 5 hour time period. Recorded loads and deflections at the beginning and end of each time period are presented in Table 1. The data were further reduced to loads and deflections recorded at the beginning of each time period only. This reduction of data is given in Table 2.

As was noted earlier, three dial indicators (gages) were positioned on each pile. Gages 1, 2, and 3 were positioned on pile A, and gages 4, 5, and 6 were positioned on pile B. Gages 3 and 4 were positioned on the sides of the piles to aid in detecting rotation of the pile. After analysis of the recorded deflections, it was determined that any rotation of the piles was insignificant.

The data presented in Table 1 show that, during the 15 minute time interval, the load decreased somewhat and the deflection increased by a small amount. The maximum

load attained during the test was 25.5 kips with a maximum deflection of 1.7 in. at pile A and 1.3 in. at pile B. The difference in apparent stiffness of the two piles can most likely be attributed to the fact that the load was applied higher on pile A, and there was standing water around pile A, while the soil around pile B was much drier and firmer.

Plots of load versus deflection for recorded values at the beginning and end of each time period are shown in Figures 5 and 6. Load versus deflection plots for the reduced data of values recorded at the beginning of each time period only are shown in Figures 7 and 8. The plots for both piles produced a relatively smooth curve up to a load of 18.4 kips which was 1.5 hours into the test. At this point there is a drop in the load with no change in deflection. The reason for this spike was due to problems associated with the loading apparatus. The problem was corrected and loading continued. At 2 hours into the test and a load of 19.3 kips, there was a sudden drop in load with a significant increase in deflection. This drop was evident in the plots of both piles. It is suspected that both piles cracked at this load. The piles continued to support additional load up to the maximum load noted earlier at which time the test was concluded. Upon release of the load, the piles rebounded with 0.29 in. of residual deflection recorded for pile A and 0.26 in. of residual deflection at pile B.

A plot of secant stiffness versus deflection is given in Figure 23. The plot shows pile A as being less stiff; however, as was noted earlier this can be attributed to the height of the applied load at each pile and the soil conditions surrounding each pile.

5.3 RESULTS OF TEST 2

Test 2 was conducted using piles A and B. The test was conducted immediately following test 1 at a loading rate of 0.10 in. every 1 minute. The maximum target deflection for this test was 1 in. Load and deflection were recorded at the beginning of each time period only, and for pile A only (Table 3). The maximum load attained was 19.3 kips at a deflection of 1 in. A plot of load versus deflection is given in Figure 9, and a plot of secant stiffness versus deflection is given in Figure 22.

5.4 RESULTS OF TEST 3

Test 3 was conducted using pile C. The main objective of this test was to reach a target deflection of 1.0 in. over a 6 hour time period. This resulted in a loading rate of 0.05 in. every 20 minutes. An initial deflection of 0.1 in. was induced with target deflections of 0.05 in. every 20 minutes obtained thereafter. As with test 1, load and deflection were recorded at the beginning and end of each time period (Table 4), and then reduced to recordings at the beginning of each time period only (Table 5).

Dial gages 1 and 2 were positioned at a height approximately 25.5 in. above ground level and gage 3 was positioned approximately 7.25 in. above ground level. This accounts for the higher deflections recorded for gages 1 and 2.

The maximum load attained during the test was 20.6 kips with a maximum deflection of 0.97 in. recorded at gage 3. Plots of load versus deflection for recorded values at the beginning and end of each time period are shown in Figures 10 and 11. Load versus deflection plots for the reduced data of values recorded at the beginning of each

time period only are shown in Figures 12 and 13. After the initial 0.10 in. of deflection, the load-deflection curves in Figure 12 follow a relatively uniform slope of 25.3 kips/in. up to a load of 14.1 kips. At this point the curves flatten out with a slope of 10.3 kips/in. It was assumed that the pile cracked at the load of 14.1 kips. The pile continued to support additional load up to the maximum noted earlier at which time the test was concluded. Upon release of the load, the pile rebounded with 0.24 in. of residual deflection recorded. The rebound curve is not shown on either plot due to the fact that no intermediate points were recorded during the rebound.

A plot of secant stiffness versus deflection for gage 1 is given in Figure 25.

5.5 RESULTS OF TEST 4

Test 4 was a cyclic test conducted using piles A and B. As was noted earlier, LVDT's were used to monitor deflection during testing. LVDT 1 was positioned at pile A, and LVDT 0 was positioned at pile B. The test was broken down into a series of tests, identified as test 4b - test 4f, with data obtained for different levels of deflection. Test 4a was a trial run to check the proper calibration of instrumentation with the computer. The results of this test are not included in the presentation of data.

5.5.1 TEST 4b

The maximum target deflection for test 4b was 0.40 in. This target deflection was reached in 0.10 in. increments. Load was applied until a deflection of 0.10 in. was reached. The load was maintained for a few seconds and then slowly released until zero

load was obtained. After a few seconds, load was again applied until the next increment of 0.20 in. was reached, maintained for a few seconds, and the load slowly released. The same process was followed for cycles up to 0.30 in. and 0.40 in. The maximum load attained for this series was 13.8 kips at 0.40 in. of deflection at pile A and 0.32 in. of deflection at pile B. Recorded values of load and deflection are given in Table 6, and plots of load versus deflection are shown in Figure 14. The total elapsed time for this series was approximately 7.3 minutes.

5.5.2 TEST 4c

The maximum target deflection for test 4c was 0.70 in. The test was conducted in the same manner as test 4b except that the first induced deflection was 0.50 in., which was 0.10 in. greater than the target deflection of test 4b. Cycles were completed for deflections of 0.60 in. and 0.70 in. The maximum load attained for this series was 17.1 kips at 0.70 in. of deflection at pile A and 0.55 in. of deflection at pile B. Recorded values of load and deflection are given in Table 7, and plots of load versus deflection are shown in Figure 15. The total elapsed time for this series was approximately 11.3 minutes.

5.5.3 TEST 4d

The maximum target deflection for test 4d was 0.90 in. The first induced deflection was 0.80 in., which was 0.10 in. greater than the target deflection of test 4c. The cycle was completed for 0.80 in. of deflection and then for the target deflection. The

maximum load attained for this series was 18.8 kips at 0.85 in. of deflection at pile A and 0.64 in. of deflection at pile B. Recorded values of load and deflection are given in Table 8, and plots of load versus deflection are shown in Figure 16. The total elapsed time for this series was approximately 5.5 minutes.

5.5.4 TEST 4e

Unlike the other cyclic tests, there was only one increment of loading for this series. The target deflection was 1.0 in. Load was slowly applied until maximum deflections of 1.15 in. at pile A and 0.86 in. of deflection at pile B were reached. The maximum load attained was 20 kips. The total elapsed time for this series was approximately 10.5 minutes. Recorded values of load and deflection are given in Table 9. Plots of load versus deflection, excluding points after the load was released, are shown in Figure 17.

5.5.5 TEST 4f

This series was conducted in the same manner as test 4e. The target deflection was 2.0 in. The maximum load attained for this series was 28.5 kips at 1.89 in. of deflection at pile A and 1.06 in. of deflection at pile B. The total elapsed time for this series was approximately 18.8 minutes. Recorded values of load and deflection are given in Table 10, and plots of load versus deflection are shown in Figure 18.

5.5.6 COMBINED RESULTS OF TEST 4

An envelope of load versus deflection was produced from the incremental deflections obtained during each series of loading. A plot of this envelope is shown in Figure 19. Consistent with test 1, the deflections for pile A are higher than pile B for a given load. As was explained earlier, this is due to the height of the applied load at both piles and the soil conditions surrounding the two piles. A plot of stiffness versus deflection for the envelope developed is shown in Figure 26.

5.6 RESULTS OF TEST 5

Test 5 was a cyclic test conducted using pile C. Two LVDT's were used to monitor deflection. LVDT 1 was positioned approximately 25 in. above ground level, and LVDT 0 was positioned approximately 7 in. above ground level. The test was performed in the same manner as test 4, except that cycles were conducted continuously at 0.10 in. increments up to a target deflection of 1.0 in. The total elapsed time for the test was approximately 17 minutes. Recorded values of load and deflection are given in Table 11. The maximum load attained was 18.7 kips at a deflection of 1.02 in. recorded for the top LVDT, and 0.79 in. for the bottom LVDT. Plots of load versus deflection for both LVDT's are shown in Figure 20, and a plot of load versus deflection produced from the envelope of the loading cycles is shown in Figure 21. A plot of secant stiffness versus deflection developed from the envelopes of both curves is shown in Figure 27.

5.7 RESULTS OF TEST 6

Test 6 was conducted using pile C. Upon completion of the cyclic test, pile C was tested to ultimate. "Ultimate" in this test was defined as that point in the test where increased deflection led to a decreased load. The total elapsed time for the test was approximately one and one-half hours. Recorded values of load and deflection are given in Table 12. A plot of load versus deflection is shown in Figure 22. The maximum load attained for this test was 27 kips at a deflection of 5.8 in. As was noted earlier, pile C was assumed to have cracked at a load of 14.1 kips during the long term loading of test 3. At a deflection of 2.2 in. the load dropped from 25.5 kips to 24.8 kips. It was assumed that another crack developed at this point. The pile continued to support additional load up to the maximum of 26.9 kips at 3.2 in. of deflection. At this point, there was a sudden drop in load to 25 kips. It was assumed that one of the prestressing strands snapped at this point. Once again the pile continued to support additional load up to a deflection of 5.8 in. and the maximum load of 27 kips. At this point, the load dropped abruptly to 13.9 kips. This abrupt drop was due to the snapping of another prestressing strand. The pile continued to support additional load up to 18.5 kips and a deflection of 8.8 in., at which time yet another prestressing strand snapped and the load dropped to 12 kips. The pile was deflected to 14 in. with no significant increase in load beyond 12 kips. At this time, the test was concluded due to the severe angle at which the pile was deflected, posing a safety threat to personnel and potential damage to the loading apparatus.

A plot of stiffness versus deflection is shown in Figure 28.

5.8 RESULTS OF COM624P COMPUTER ANALYSIS

As was noted earlier, the prestressed piles were modeled using the computer program COM624P. Simulations were based on the height of applied load being 17 in. above ground level. This is the height at which load was applied at piles A and C. Several runs of the program were conducted using various magnitudes of lateral loading corresponding to those recorded for test 3. The results obtained from the computer analysis were compared to the results obtained from actual field testing. A plot comparing load versus deflection for COM624P and test 3 is shown in Figure 35. At all levels of applied load up to the point of cracking, and just beyond, the deflections obtained from the computer analysis were within 16 % of those obtained from actual field tests. The curve for the COM624P computer analysis indicates behavior that is a little stiffer than that for test 3. This can be attributed to the fact that deflections produced by COM624P correspond to the height of the applied load which was 17 in. above ground level. Although the height of applied load was at 17 in. above ground level for test 3, the dial indicators were positioned approximately 25.5 in. above ground level. This accounts for the difference in the values of deflection for test 3 being somewhat higher for a given load. As was noted earlier, the curve for test 3 flattens out somewhat at an applied load of 14.1 kips, the point at which it was assumed the pile cracked. The curve for COM624P does not flatten out as well because the program did not take into account the cracking moment of the pile.

Assumptions, based on the load versus deflections plots for tests 1 and 3, were that piles A and B had cracked at an applied load of 19.3 kips, and that pile C had

cracked at an applied load of 14.1 kips. Calculations were performed to determine the cracking moment of the piles based on a compressive strength for the concrete of 6000 psi., and a final stress of the prestressing strands of 175 ksi. The cracking moment was determined to be approximately 640 kip-in. The computer program was run using an applied load of 14.1 kips. Results of the computer analysis indicated a maximum moment of 664 kip-in. occurring at a depth of 61 in. below the ground surface. This supported the assumption that the piles were indeed cracked. The deflection, at the height of the applied load, obtained from the computer analysis was 0.451 in. The deflections obtained from actual field tests for the same applied load were 0.429 in. at pile A, and 0.439 in. at pile C. These results suggest that COM624P can be used to accurately model the behavior, up to the point of cracking, of prestressed concrete piles subjected to lateral loads.

CHAPTER 6

DISCUSSION OF RESULTS

6.1 SLOWLY APPLIED TESTS

A comparison of the results obtained in test 1 and test 2 are presented in a plot of load versus deflection (Figure 29), and a plot of stiffness versus deflection (Figure 30). The comparison is for pile A only. As was noted earlier, test 1 was conducted at a loading rate of 0.10 in. every 15 minutes, while test 2 was conducted at a loading rate of 0.10 in. every minute. The plots show that for a given deflection the behavior of the pile during test 2 was less stiff than that for test 1. A true comparison of the two tests is difficult to achieve due to the fact that test 2 was conducted immediately after test 1 and the soil was already deformed.

A better comparison of the behavior of a pile subjected to different rates of loading can be made between test 1, pile A, and test 3, pile C. As was noted earlier, test 1 was conducted at a loading rate of 0.10 in. every 15 minutes. Test 2 was conducted at a loading rate of 0.05 in. every 20 minutes. The height of the applied load was at approximately 17 in. above ground level for both piles. A comparison of the results for both tests is shown in plots of load versus deflection (Figure 31) and stiffness versus deflection (Figure 32). The plots show that for a given deflection the behavior of pile C was less stiff than that for pile A. In spite of the fact that pile A was situated in standing water and the soil surrounding pile C was much drier, the behavior of pile A was somewhat stiffer, a fact which can be attributed to the faster rate of loading.

Upon release of the load, both piles A and C (test 1, and test 3) rebounded with approximately the same residual deflection. Due to time constraints, it was not possible to determine how much of the residual deflection would have been recovered over an extended period of time.

It was assumed that all three of the piles cracked at deflections much less than 1.0 in. The computer program, COM624P, helped to confirm that assumption. Based on the calculated cracking moment for a prestressed concrete pile, the lateral load necessary to produce that moment was approximately 14 kips. While it was believed that Pile C cracked at a load of 14.1 kips, cracking was not evident for piles A and B until a load of 19.3 kips was reached. It is possible that both piles A and B cracked at a load closer to 14 kips and the change in stiffness due to the crack was not as evident in the plots developed for piles A and B as it was for the plots developed for pile C.

The fact that the piles cracked before a deflection of 1.0 in. was reached had little effect on their ability to support additional load and continue to deflect well beyond 1.0 in and return to their original position. After cracking, the prestressing strands were still well within the elastic range of behavior, thus allowing the pile to continue to deflect while supporting additional load, and rebound to its original position with minimal residual deflection after the load was released.

6.2 CYCLIC TESTS

As noted earlier all of the cyclic tests were performed after the piles were already cracked. The load-deflection curves developed for all tests are marked by the presence of

a pronounced hysteresis with the deflection under zero load returning almost, but not quite, to zero. The curves show that each pile was less stiff with each new cycle of loading. The load-deflection behavior for the cyclic tests deviates significantly more from linear than the behavior of the piles during the long term tests prior to cracking.

A comparison of the envelopes developed from test 4 and test 5 are shown in a plot of load versus deflection (Figure 33), and a plot of stiffness versus deflection (Figure 34). When comparing the results of pile A, test 4, and the results of pile C (top LVDT), test 5, pile C demonstrated a behavior that was less stiff for a given deflection. The elapsed time to reach a deflection of 1.0 in. at pile A for test 4 was approximately 8 minutes. The elapsed time to reach the same deflection at pile C (top LVDT) was approximately 16 minutes. Consistent with a comparison of test 1 and test 3, the slower rate of loading results in behavior that is less stiff.

6.3 TEST TO ULTIMATE

As noted earlier, ultimate was defined as that point where increased deflection led to decreased load. The ultimate point for this test was at a load of 27 kips and a deflection of 5.8 in. This is the point where it is believed that the second prestressing strand snapped. While the pile was deflected to such an angle that it would never return to its original position, failure, in the sense that the pile was broken into two pieces, never occurred. The test demonstrated the ability of a prestressed concrete pile to sustain large lateral deflections and still maintain structural integrity.

CHAPTER 7

CONCLUSIONS

The effects of the thermal variations experienced in Tennessee may very well cause deflections of 1.0 in. or more at the ground level for piles supporting jointless bridges. The results of the tests conducted for this research showed that the prestressed concrete piles cracked at deflections considerably less than 1.0 in. However, even though the piles were cracked, they were capable of supporting additional lateral load at deflections well beyond 1.0 in. and maintain their structural integrity. When the load was released, the piles returned to their original position with minor residual deflection. The ability of the piles to support the magnitude of the loads applied, deflect well beyond 1.0 in., and rebound to their original position is due to the prestressing strands. Thus, prestressed concrete piles are very capable of withstanding deflections of 1.0 in. or more at the ground level without compromise to the structural integrity of the pile.

When a pile was subjected to cyclic loading, the behavior of the pile demonstrated a load-deflection relationship that was less stiff with each cycle of loading. A comparison of different rates of loading showed that a slower loading rate resulted in behavior that was less stiff. These facts are important in the design of jointless bridges where relatively high displacements may occur, from day to day, due to a wide range of temperature change.

The test to ultimate demonstrated the extremely ductile behavior of a prestressed concrete pile. The piles are capable of withstanding large deflections in the inelastic range before the structural integrity of the pile is compromised.

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APPENDICES

APPENDIX A:

TABLES

Table 1. Recorded loads and deflections at beginning and end of each time period (Test 1, Piles A and B).

TIME	LOAD (lbs)	DIAL GAGE READING (inches)					
		1	2	3	4	5	6
1:00	0	0.000	0.000	0.000	0.000	0.000	0.000
1:00	5700	0.100	0.100	0.005	0.022	0.097	0.099
1:15	4900	0.106	0.105	0.007	0.017	0.100	0.102
1:15	9300	0.200	0.201	0.014	0.038	0.187	0.191
1:30	8500	0.205	0.203	0.020	0.033	0.188	0.192
1:30	12200	0.300	0.299	0.028	0.050	0.268	0.272
1:45	11200	0.303	0.301	0.038	0.042	0.268	0.273
1:45	13900	0.391	0.390	0.042	0.054	0.334	0.340
2:00	13100	0.391	0.390	0.042	0.050	0.331	0.336
2:00	15400	0.468	0.467	0.040	0.061	0.387	0.392
2:04	14200	0.429	0.428	0.041	0.058	0.364	0.369
2:10	16400	0.500	0.499	0.032	0.073	0.412	0.416
2:30	15400	0.487	0.486	0.028	0.075	0.405	0.408
2:30	18400	0.601	0.601	0.035	0.095	0.499	0.504
2:45	16800	0.595	0.595	0.035	0.093	0.495	0.501
2:45	17200	0.603	0.602	0.036	0.095	0.501	0.502
2:48	16700	0.581	0.581	0.038	0.092	0.488	0.490
2:53	18400	0.700	0.700	0.045	0.110	0.589	0.595
3:00	17400	0.701	0.701	0.045	0.108	0.585	0.592
3:00	19300	0.800	0.802	0.049	0.125	0.680	0.688
3:15	18300	0.805	0.805	0.055	0.122	0.681	0.689
3:20	18500	0.940	0.940	0.061	0.139	0.801	0.812
3:28	18300	0.940	0.940	0.062	0.139	0.801	0.812
3:30	20300	1.001	1.001	0.062	0.149	0.841	0.853
3:45	20100	1.002	0.998	0.064	0.146	0.838	0.848
3:45	22300	1.100	1.099	0.060	0.160	0.912	0.925
4:00	21200	1.108	1.104	0.066	0.158	0.910	0.922
4:00	23000	1.200	1.200	0.064	0.170	0.983	0.995
4:15	21700	1.205	1.203	0.067	0.193	0.980	0.965
4:15	23300	1.302	1.302	0.067	0.181	1.056	1.070
4:30	22000	1.311	1.310	0.075	0.188	1.057	1.072
4:30	23400	1.400	1.400	0.077	0.200	1.124	1.139
4:45	22700	1.409	1.407	0.078	0.199	1.127	1.142
4:45	24300	1.500	1.498	0.077	0.210	1.195	1.207
5:00	23500	1.503	1.501	0.084	0.210	1.194	1.206
5:00	25200	1.600	1.599	0.084	0.222	1.265	1.278
5:15	24100	1.606	1.605	0.088	0.219	1.264	1.277
5:15	25500	1.700	1.699	-------	0.231	1.331	1.344
5:22	24600	1.708	1.707	-------	0.233	1.332	1.345
5:27	17000	1.256	1.254	-------	-------	1.051	1.061
5:47	17100	1.256	1.254	-------	0.233	1.051	1.061
5:50	6000	0.488	0.488	0.040	0.066	0.442	0.451
5:55	6400	0.488	0.488	-------	-------	0.442	0.451
6:00	0	0.288	0.288	0.025	0.025	0.263	0.250

Table 2. Recorded loads and deflections at beginning of each time period (Test 1, Piles A and B).

TIME	LOAD	DIAL GAGE READING (inches)					
	(lbs)	1	2	3	4	5	6
1:00	0	0.000	0.000	0.000	0.000	0.000	0.000
1:00	5700	0.100	0.100	0.005	0.022	0.097	0.099
1:15	9300	0.200	0.201	0.014	0.038	0.187	0.191
1:30	12200	0.300	0.299	0.028	0.050	0.268	0.272
1:45	13900	0.391	0.390	0.042	0.054	0.334	0.340
2:00	15400	0.468	0.467	0.040	0.061	0.387	0.392
2:10	16400	0.500	0.499	0.032	0.073	0.412	0.416
2:30	18400	0.601	0.601	0.035	0.095	0.499	0.504
2:45	17200	0.603	0.602	0.036	0.095	0.501	0.502
2:53	18400	0.700	0.700	0.045	0.110	0.589	0.595
3:00	19300	0.800	0.802	0.049	0.125	0.680	0.688
3:20	18500	0.940	0.940	0.061	0.139	0.801	0.812
3:30	20300	1.001	1.001	0.062	0.149	0.841	0.853
3:45	22300	1.100	1.099	0.060	0.160	0.912	0.925
4:00	23000	1.200	1.200	0.064	0.170	0.983	0.995
4:15	23300	1.302	1.302	0.067	0.181	1.056	1.070
4:30	23400	1.400	1.400	0.077	0.200	1.124	1.139
4:45	24300	1.500	1.498	0.077	0.210	1.195	1.207
5:00	25200	1.600	1.599	0.084	0.222	1.265	1.278
5:15	25500	1.700	1.699	-------	0.231	1.331	1.344
5:27	17000	1.256	1.254	-------	-------	1.051	1.061
5:47	17100	1.256	1.254	-------	0.233	1.051	1.061
5:50	6000	0.488	0.488	0.040	0.066	0.442	0.451
5:55	6400	0.488	0.488	-------	-------	0.442	0.451
6:00	0	0.288	0.288	0.025	0.025	0.263	0.250

Table 3. Recorded loads and deflections (Test 2, Piles A and B).

TIME	LOAD (lbs)	DIAL GAGE READING (inches)					
		1	2	3	4	5	6
6:14	0	0.000	0.000	0.024	0.021	0.000	0.000
6:15	5300	0.100	0.100	-------	-------	-------	-------
6:16	8500	0.200	0.201	-------	-------	-------	-------
6:17	10800	0.300	0.302	-------	-------	-------	-------
6:18	12400	0.400	0.401	-------	-------	-------	-------
6:19	13900	0.500	0.500	-------	-------	-------	-------
	15200	0.600	0.600	-------	-------	-------	-------
6:20	16300	0.700	0.700	-------	-------	-------	-------
	17300	0.800	0.800	-------	-------	-------	-------
6:21	18400	0.900	0.900	-------	-------	-------	-------
6:23	19300	1.000	1.000	0.064	0.178	0.804	0.811
6:25	15600	0.800	0.800	-------	-------	-------	-------
	10000	0.431	0.432	-------	-------	-------	-------
	5600	0.232	0.232	-------	-------	-------	-------
6:27	0	0.066	0.066	0.008	0.026	0.014	0.014

Table 4. Recorded loads and deflections at beginning and end of each time period (Test 3, Pile C).

TIME (min)		LOAD	DEFLECTION (in)			
CLOCK	TEST	(lbs)	TARGET	GAGE 1	GAGE 2	GAGE 3
12:45	0	0	0.000	0.000	0.000	0.000
	0	5500	0.100	0.100	0.100	0.100
	40	4900	0.100	0.110	0.110	0.110
	40	6800	0.150	0.150	0.150	0.150
1:45	60	6000	0.150	0.153	0.153	0.153
	60	8000	0.200	0.200	0.200	0.200
	80	7400	0.200	0.211	0.211	0.211
	80	9400	0.250	0.260	0.261	0.250
	100	9000	0.250	0.272	0.270	0.262
	100	10900	0.300	0.320	0.319	0.300
2:45	120	10500	0.300	0.324	0.326	0.309
	120	12400	0.350	0.377	0.378	0.350
	140	11900	0.350	0.384	0.384	0.359
	140	14100	0.400	0.439	0.440	0.400
	160	13400	0.400	0.444	0.445	0.410
	160	14800	0.450	0.499	0.499	0.450
3:45	180	14100	0.450	0.505	0.504	0.458
	180	15700	0.500	0.561	0.560	0.500
	200	14800	0.500	0.565	0.563	0.508
	200	16300	0.550	0.624	0.622	0.550
	220	15700	0.550	0.630	0.628	0.558
	220	16800	0.600	0.686	0.685	0.600
4:45	240	16300	0.600	0.693	0.695	0.610
	240	17600	0.650	0.751	0.748	0.650
	260	17000	0.650	0.758	0.755	0.660
	260	18100	0.700	0.812	0.809	0.700
	280	17700	0.700	0.827	0.824	0.715
	280	18800	0.750	0.874	0.873	0.750
5:45	300	18100	0.750	0.883	0.881	0.760
	300	19200	0.800	0.935	0.917	0.800
	320	18400	0.800	0.941	0.916	0.810
	320	20100	0.850	------	------	0.895
	340	19100	0.850	------	------	0.899
	340	20600	0.900	------	------	0.955
6:45	360	19700	0.950	------	------	0.960
	360	20100	1.000	------	------	0.975

Table 5. Recorded loads and deflections at beginning of each time period (Test 3, Pile C).

TIME (min)		LOAD	DEFLECTION (in)			
CLOCK	TEST	(lbs)	TARGET	GAGE 1	GAGE 2	GAGE 3
12:45	0	0	0.000	0.000	0.000	0.000
12:45	0	5500	0.100	0.100	0.100	0.100
	40	6800	0.150	0.150	0.150	0.150
1:45	60	8000	0.200	0.200	0.200	0.200
	80	9400	0.250	0.260	0.261	0.250
	100	10900	0.300	0.320	0.319	0.300
2:45	120	12400	0.350	0.377	0.378	0.350
	140	14100	0.400	0.439	0.440	0.400
	160	14800	0.450	0.499	0.499	0.450
3:45	180	15700	0.500	0.561	0.560	0.500
	200	16300	0.550	0.624	0.622	0.550
	220	16800	0.600	0.686	0.685	0.600
4:45	240	17600	0.650	0.751	0.748	0.650
	260	18100	0.700	0.812	0.809	0.700
	280	18800	0.750	0.874	0.873	0.750
5:45	300	19200	0.800	0.935	0.917	0.800
	320	20100	0.850	-----	-----	0.895
	340	20600	0.900	-----	-----	0.955
6:45	360	20100	1.000	-----	-----	0.975

Table 6. Recorded loads and deflections for cyclic test (Test 4b, Piles A and B).

TIME (seconds)	LOAD (lbs)	DEFLECTION LVDT 1 (PILE A) (inches)	DEFLECTION LVDT 0 (PILE B) (inches)
0	0	0.00	0.00
66	57	0.00	0.00
75	1661	0.02	0.02
82	3032	0.04	0.04
88	4628	0.07	0.07
96	5809	0.10	0.09
106	5764	0.10	0.09
109	1754	0.04	0.03
112	37	0.01	0.01
131	29	0.00	0.00
143	2778	0.04	0.04
156	5595	0.09	0.09
166	8299	0.15	0.15
174	9505	0.19	0.18
185	9306	0.20	0.18
185	2941	0.09	0.08
189	66	0.01	0.01
219	33	0.01	0.01
239	5718	0.11	0.10
249	9437	0.20	0.18
259	11688	0.27	0.23
264	12081	0.29	0.25
281	11935	0.29	0.25
281	3983	0.13	0.11
285	20	0.02	0.02
339	228	0.01	0.01
359	6592	0.14	0.13
375	11173	0.26	0.23
387	13492	0.36	0.30
396	13833	0.40	0.32
429	13541	0.39	0.32
429	7067	0.23	0.19
435	100	0.03	0.02

Table 7. Recorded loads and deflections for cyclic test (Test 4c, Piles A and B).

TIME (seconds)	LOAD (lbs)	DEFLECTION LVDT 1 (PILE A) (inches)	DEFLECTION LVDT 0 (PILE B) (inches)
0	0	0.00	0.00
75	763	0.01	0.01
91	5480	0.10	0.10
108	11208	0.26	0.23
125	14951	0.44	0.36
134	15638	0.49	0.40
145	15399	0.49	0.40
145	7253	0.24	0.21
150	42	0.03	0.02
239	626	0.02	0.01
258	6135	0.13	0.12
272	11395	0.28	0.25
288	15249	0.47	0.39
304	16739	0.59	0.48
326	16301	0.60	0.48
331	97	0.03	0.03
570	1277	0.03	0.03
607	10075	0.25	0.23
628	14700	0.47	0.39
642	16878	0.62	0.50
654	17420	0.69	0.55
673	17151	0.70	0.55
680	410	0.05	0.05

Table 8. Recorded loads and deflections for cyclic test (Test 4d, Piles A and B).

TIME (seconds)	LOAD (lbs)	DEFLECTION LVDT 1 (PILE A) (inches)	DEFLECTION LVDT 0 (PILE B) (inches)
0	0	0.00	0.00
56	1314	0.01	0.01
76	10339	0.23	0.21
88	14669	0.46	0.38
95	16680	0.60	0.48
105	18059	0.75	0.58
133	17704	0.75	0.58
133	8990	0.34	0.28
138	342	0.03	0.03
236	996	0.01	0.02
251	7590	0.17	0.16
267	14019	0.45	0.36
276	16501	0.64	0.50
293	18832	0.84	0.63
327	18782	0.85	0.64
328	10673	0.45	0.35
332	233	0.05	0.04

Table 9. Recorded loads and deflections for cyclic test (Test 4e, Piles A and B).

TIME (seconds)	LOAD (lbs)	DEFLECTION LVDT 1 (PILE A) (inches)	DEFLECTION LVDT 0 (PILE B) (inches)
0	0	0.00	0.00
59	583	0.00	0.00
74	6899	0.14	0.13
86	11891	0.30	0.26
102	15507	0.56	0.43
122	18587	0.84	0.62
470	18587	0.84	0.65
483	18587	0.90	0.69
503	19049	1.04	0.78
512	19658	1.15	0.86
629	20000	1.15	0.86

Table 10. Recorded loads and deflections for cyclic test (Test 4f, Piles A and B).

TIME (seconds)	LOAD (lbs)	DEFLECTION LVDT 1 (PILE A) (inches)	DEFLECTION LVDT 0 (PILE B) (inches)
0	0	0.00	0.00
36	1392	0.01	0.00
47	7355	0.15	0.13
60	14237	0.45	0.35
70	16997	0.70	0.50
77	18261	0.91	0.53
175	18572	0.92	0.65
185	18578	0.92	0.65
519	18907	0.90	0.65
550	24132	1.12	0.71
557	24000	1.14	0.71
565	25084	1.30	0.81
569	25011	1.31	0.81
576	25979	1.44	0.88
631	25863	1.54	0.92
665	25863	1.58	0.94
754	25150	1.58	0.94
962	25498	1.65	0.94
1006	25765	1.68	0.94
1079	24033	1.70	0.94
1108	25857	1.73	0.96
1109	26139	1.74	0.97
1119	26207	1.76	0.98
1120	26997	1.79	0.99
1123	27739	1.83	1.02
1127	28513	1.89	1.06

Table 11. Recorded loads and deflections for cyclic test (Test 5, Pile C).

TIME (seconds)	LOAD (lbs)	DEFLECTION LVDT 1 (TOP) (inches)	DEFLECTION LVDT 0 (BOTTOM) (inches)
0	0	0.00	0.00
32	1225	0.02	0.01
44	2677	0.05	0.04
55	4441	0.10	0.08
89	1810	0.06	0.05
138	86	0.00	0.00
154	5649	0.14	0.11
166	7702	0.20	0.16
198	2432	0.10	0.09
200	139	0.02	0.02
234	2883	0.06	0.05
254	8112	0.22	0.18
267	9972	0.30	0.24
294	3112	0.15	0.13
295	72	0.03	0.02
318	3785	0.10	0.08
335	9355	0.28	0.22
353	12029	0.40	0.32
379	7647	0.30	0.24
380	73	0.03	0.03
413	5887	0.17	0.14
431	11806	0.41	0.32
443	13469	0.50	0.40
470	7391	0.33	0.27
496	90	0.01	0.01
508	4864	0.15	0.12
540	13287	0.51	0.40
555	14817	0.61	0.49
582	7951	0.38	0.31
583	192	0.07	0.06
613	6530	0.22	0.18
642	14667	0.61	0.49
658	15893	0.70	0.56
696	5978	0.31	0.27
698	69	0.06	0.05
726	5086	0.19	0.15
766	14694	0.64	0.51
782	16845	0.80	0.63
822	100	0.06	0.05
845	10925	0.43	0.35
866	15717	0.74	0.59
880	17884	0.91	0.72
905	8657	0.50	0.40
906	65	0.12	0.08
932	6855	0.30	0.24
962	15567	0.77	0.61
988	18739	1.02	0.79
1012	10781	0.60	0.49
1014	61	0.12	0.10

Table 12. Recorded loads and deflections, test to ultimate (Test 6, Pile C).

LOAD (lbs)	DEFLECTION (in)
0	0.000
5850	0.200
11042	0.400
13900	0.600
16200	0.800
18600	1.000
20900	1.200
21600	1.400
22700	1.600
23600	1.800
24400	2.000
25500	2.200
24800	2.300
25400	2.400
26000	2.600
26500	2.800
26900	3.000
26900	3.200
26300	3.300
25000	3.406
26600	3.594
26600	3.844
26000	4.094
26100	4.344
26100	4.594
26000	4.844
26100	5.094
26800	5.344
27000	5.594
27000	5.844
13900	6.094
15200	6.344
16000	6.844
17500	7.844
18500	8.844
12000	9.844
12500	10.844
12700	14.406

Table 13. Static (secant) stiffness (Test 1, Piles A and B).

LOAD (kips)	DEFLECTION PILE A (inches)	DEFLECTION PILE B (inches)	STATIC STIFFNESS (A) (kips/inch)	STATIC STIFFNESS (B) (kips/inch)
0.00	0.000	0.000	0.000	0.000
5.70	0.100	0.099	57.000	57.576
9.30	0.200	0.191	46.500	48.691
12.20	0.300	0.272	40.667	44.853
13.90	0.391	0.340	35.550	40.882
15.40	0.468	0.392	32.906	39.286
16.40	0.500	0.416	32.800	39.423
18.40	0.601	0.504	30.616	36.508
18.40	0.700	0.595	26.286	30.924
19.30	0.800	0.688	24.125	28.052
20.30	0.940	0.812	21.596	25.000
21.40	1.001	0.853	21.379	25.088
22.30	1.100	0.925	20.273	24.108
23.00	1.200	0.995	19.167	23.116
23.30	1.302	1.070	17.896	21.776
23.40	1.400	1.139	16.714	20.544
24.30	1.500	1.207	16.200	20.133
25.20	1.600	1.278	15.750	19.718
25.50	1.700	1.344	15.000	18.973
17.00	1.256	1.061	13.535	16.023
17.10	1.256	1.061	13.615	16.117
6.00	0.488	0.451	12.295	13.304
0.00	0.288	0.250	0.000	0.000

Table 14. Static (secant) stiffness (Test 2, Pile A).

LOAD (kips)	DEFLECTION PILE (A) (inches)	STATIC STIFFNESS (A) (kips/inch)
0.00	0.000	0.000
5.30	0.100	53.000
8.50	0.201	42.394
10.80	0.301	35.880
12.40	0.401	30.961
13.90	0.500	27.800
15.20	0.600	25.333
16.30	0.700	23.286
17.30	0.800	21.625
18.40	0.900	20.444
19.30	1.000	19.300
15.60	0.800	19.500
10.00	0.432	23.175
5.60	0.232	24.138
0.00	0.066	0.000

Table 15. Static (secant) stiffness (Test 3, Pile C).

LOAD (kips)	DEFLECTION PILE C (gage 1) (inches)	STATIC STIFFNESS (C) (kips/inch)
0.00	0.000	0.000
5.50	0.100	55.000
6.80	0.150	45.333
8.00	0.200	40.000
9.40	0.260	36.154
10.90	0.320	34.063
12.40	0.377	32.891
14.10	0.439	32.118
14.80	0.499	29.659
15.70	0.561	27.986
16.30	0.624	26.122
16.80	0.686	24.490
17.60	0.751	23.435
18.10	0.812	22.291
18.80	0.874	21.510
19.20	0.935	20.535

Table 16. Static (secant) stiffness (Test 4, Piles A and B).

LOAD (kips)	DEFLECTION LVDT 1 (PILE A) (inches)	DEFLECTION LVDT 0 (PILE B) (inches)	STATIC STIFFNESS (A) (kips/inch)	STATIC STIFFNESS (B) (kips/inch)
0.00	0.00	0.00	0.00	0.00
5.81	0.10	0.09	58.09	64.54
9.51	0.19	0.18	50.03	52.81
12.08	0.29	0.25	41.66	48.32
13.83	0.40	0.32	34.58	43.23
15.64	0.49	0.40	31.91	39.10
16.74	0.59	0.48	28.37	34.87
17.42	0.69	0.55	25.25	31.67
18.06	0.75	0.58	24.08	31.14
18.83	0.84	0.63	22.42	29.89
18.59	0.90	0.69	20.65	26.94
19.05	1.04	0.78	18.32	24.42
20.00	1.15	0.86	17.39	23.26

Table 17. Static (secant) stiffness (Test 5, Pile C).

LOAD (kips)	DEFLECTION LVDT 1 (TOP) (inches)	DEFLECTION LVDT 0 (BOTTOM) (inches)	STATIC STIFFNESS (1) (kips/inch)	STATIC STIFFNESS (0) (kips/inch)
0.00	0.00	0.00	0.00	0.00
4.44	0.10	0.08	44.41	55.51
7.70	0.20	0.16	38.51	48.14
9.97	0.30	0.24	33.24	41.55
12.03	0.40	0.32	30.07	37.59
13.47	0.50	0.40	26.94	33.67
14.82	0.61	0.49	24.29	30.24
15.89	0.70	0.56	22.70	28.38
16.85	0.80	0.63	21.06	26.74
17.88	0.91	0.72	19.65	24.84
18.74	1.02	0.79	18.37	23.72

Table 18. Static (secant) stiffness (Test 6, Pile C).

LOAD (kips)	DEFLECTION (in)	STIFFNESS (kips/inch)
0	0.000	0.000
5.85	0.200	29.250
11.04	0.400	27.600
13.90	0.600	23.167
16.20	0.800	20.250
18.60	1.000	18.600
20.90	1.200	17.417
21.60	1.400	15.429
22.70	1.600	14.188
23.60	1.800	13.111
24.40	2.000	12.200
25.50	2.200	11.591
24.80	2.300	10.783
25.40	2.400	10.583
26.00	2.600	10.000
26.50	2.800	9.464
26.90	3.000	8.967
26.90	3.200	8.406
26.30	3.300	7.970
25.00	3.406	7.340
26.60	3.594	7.401
26.60	3.844	6.920
26.00	4.094	6.351
26.10	4.344	6.008
26.10	4.594	5.681
26.00	4.844	5.367
26.10	5.094	5.124
26.80	5.344	5.015
27.00	5.594	4.827
27.00	5.844	4.620
13.90	6.094	2.281
15.20	6.344	2.396
16.00	6.844	2.338
17.50	7.844	2.231
18.50	8.844	2.092
12.00	9.844	1.219
12.50	10.844	1.153
12.70	14.406	0.882

APPENDIX B:

FIGURES

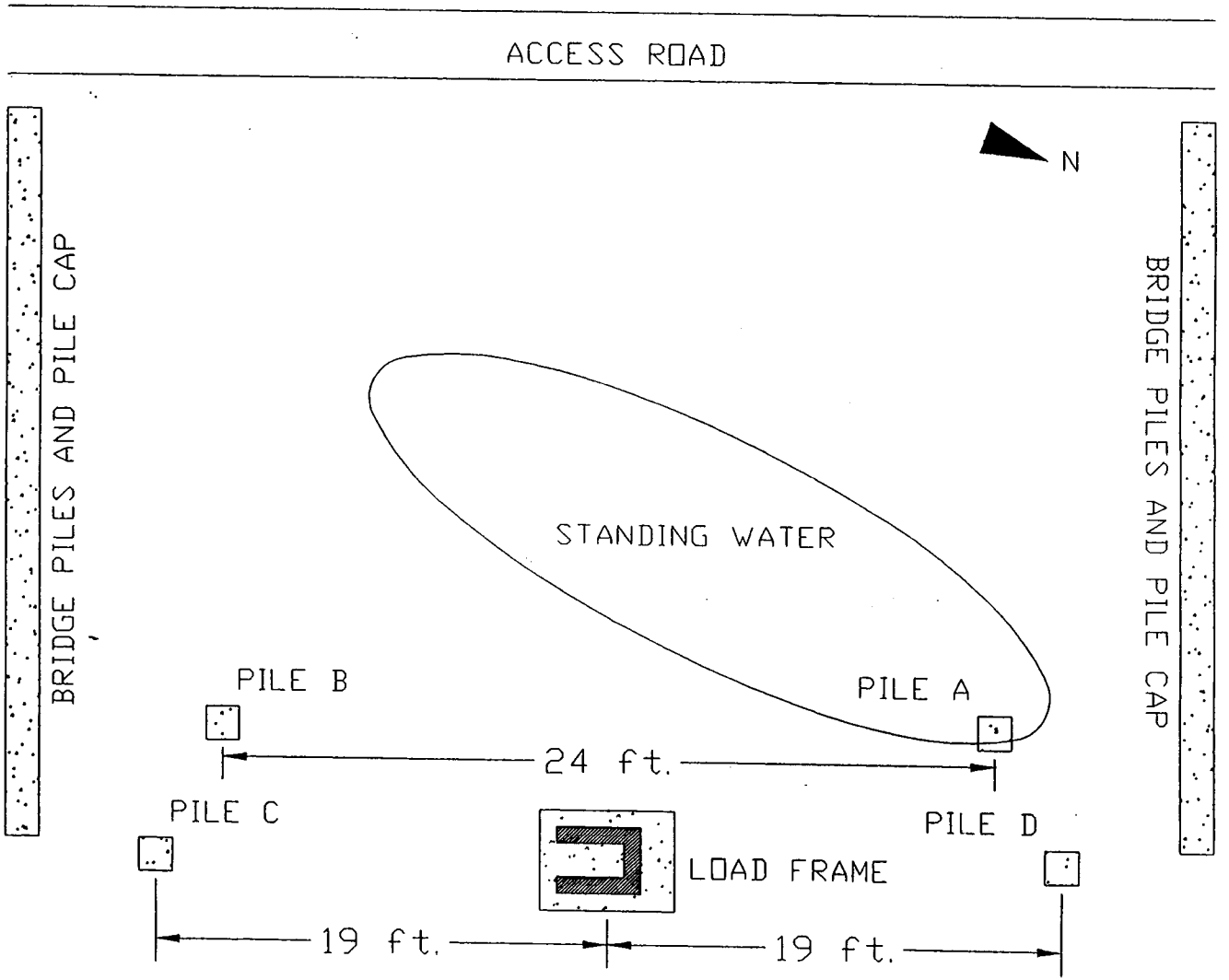


Figure 1. Test site (not to scale).

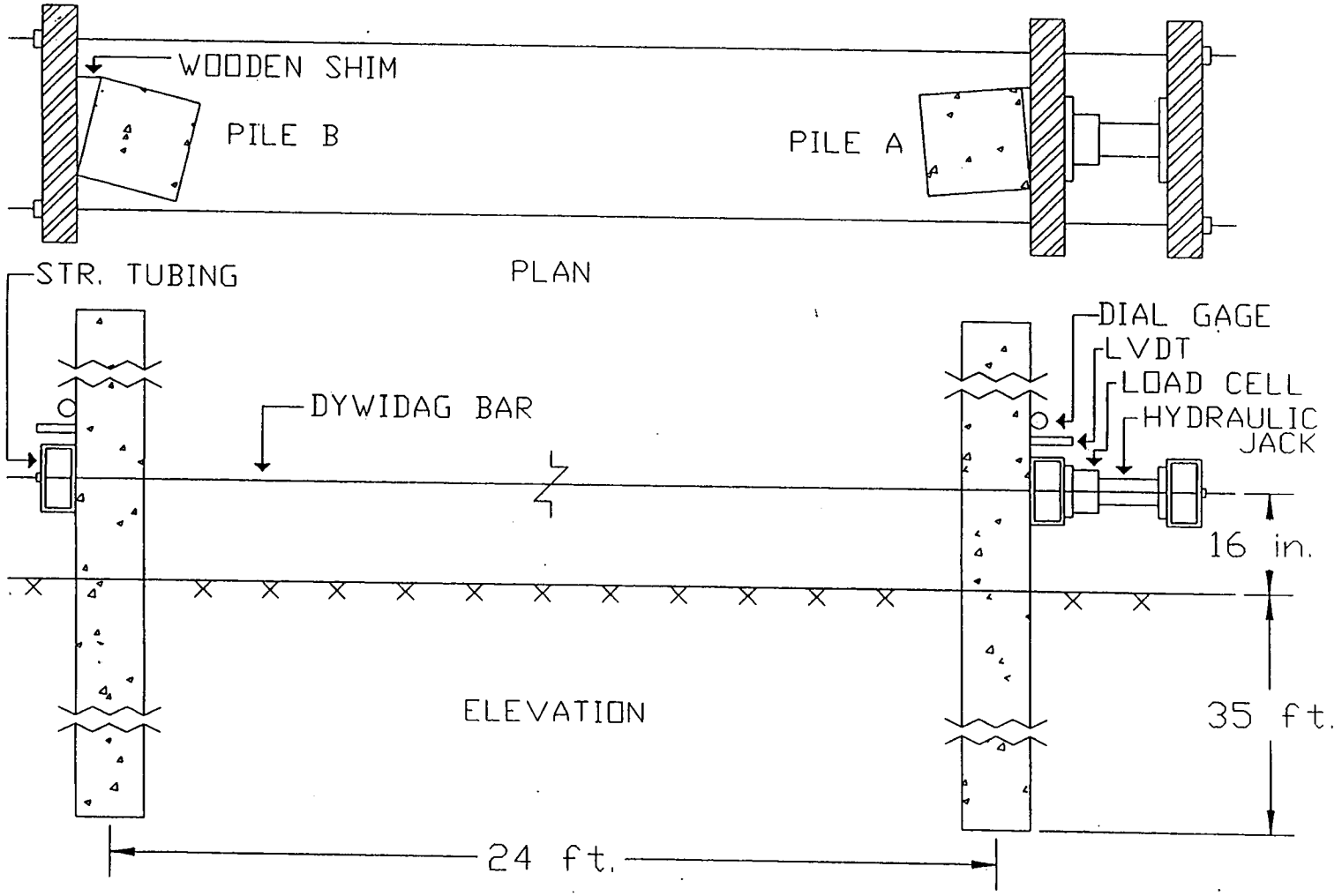


Figure 2. Test setup, piles A and B, (not to scale).

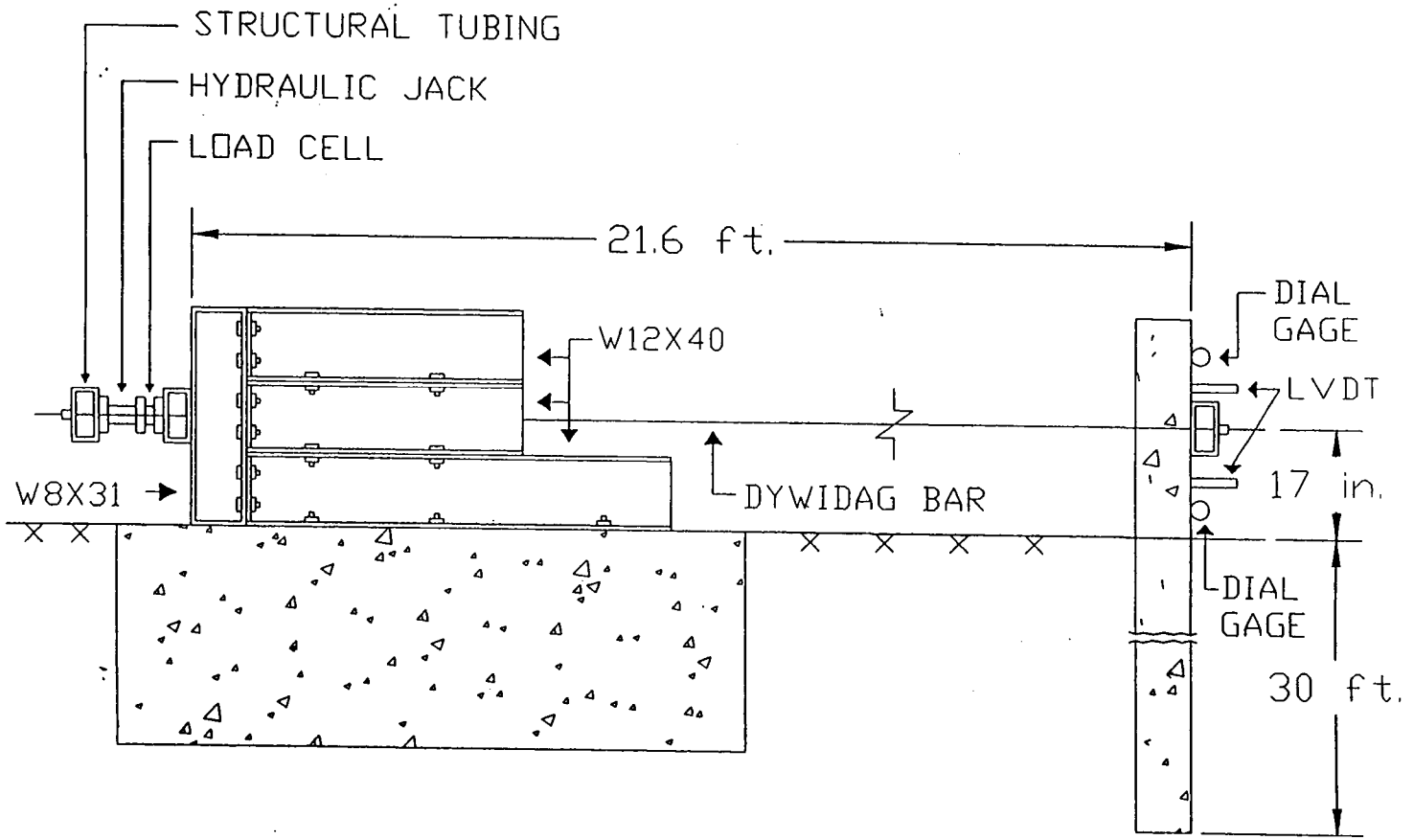


Figure 3. Test setup, pile C, (not to scale).

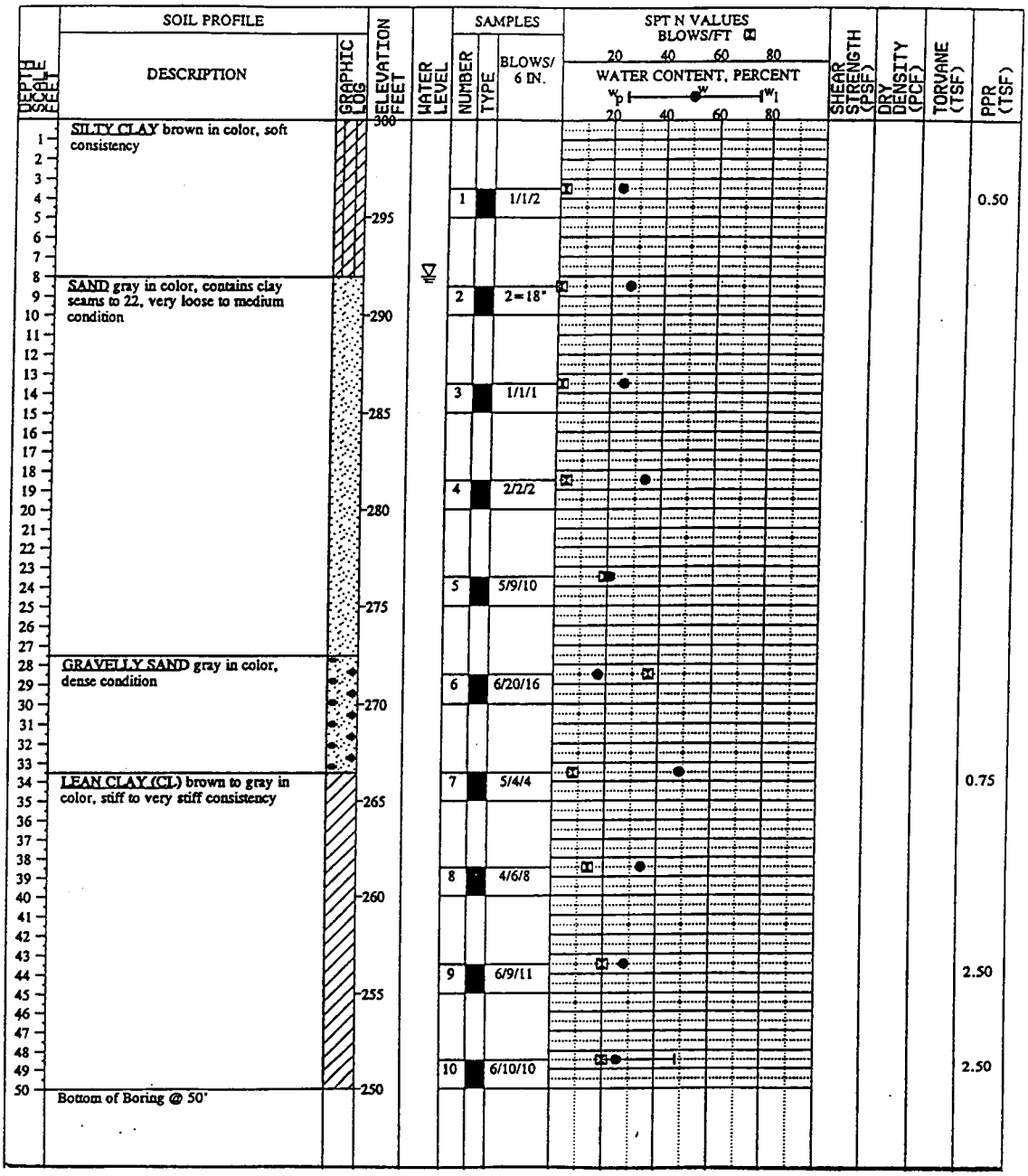


Figure 4. Soil profile for test site.

Source: Hall, Blake and Associates, Inc., Memphis, TN, 1995.

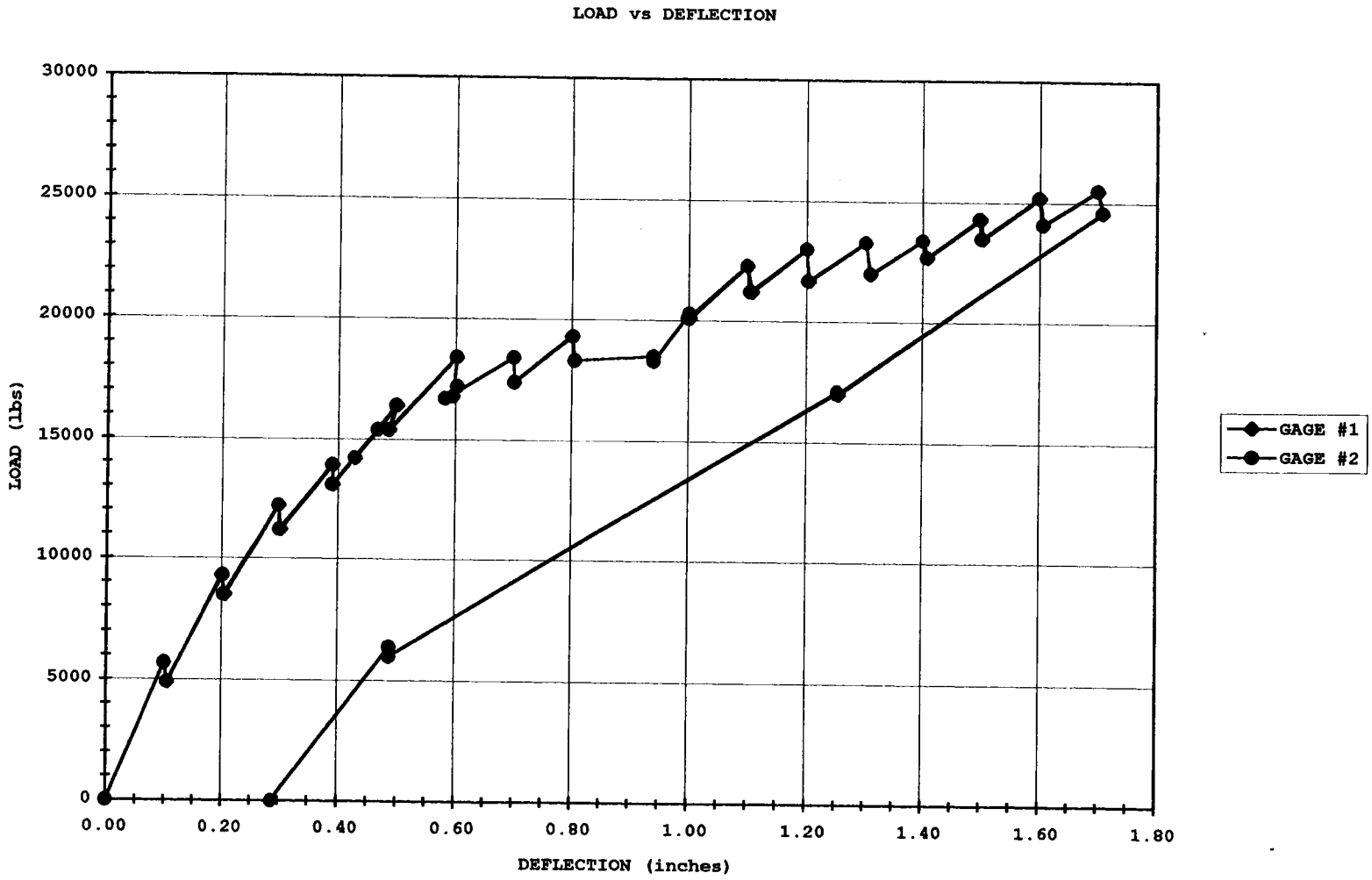


Figure 5. Load versus deflection at beginning and end of each time period, (Test 1, Pile A, Gages 1 and 2).

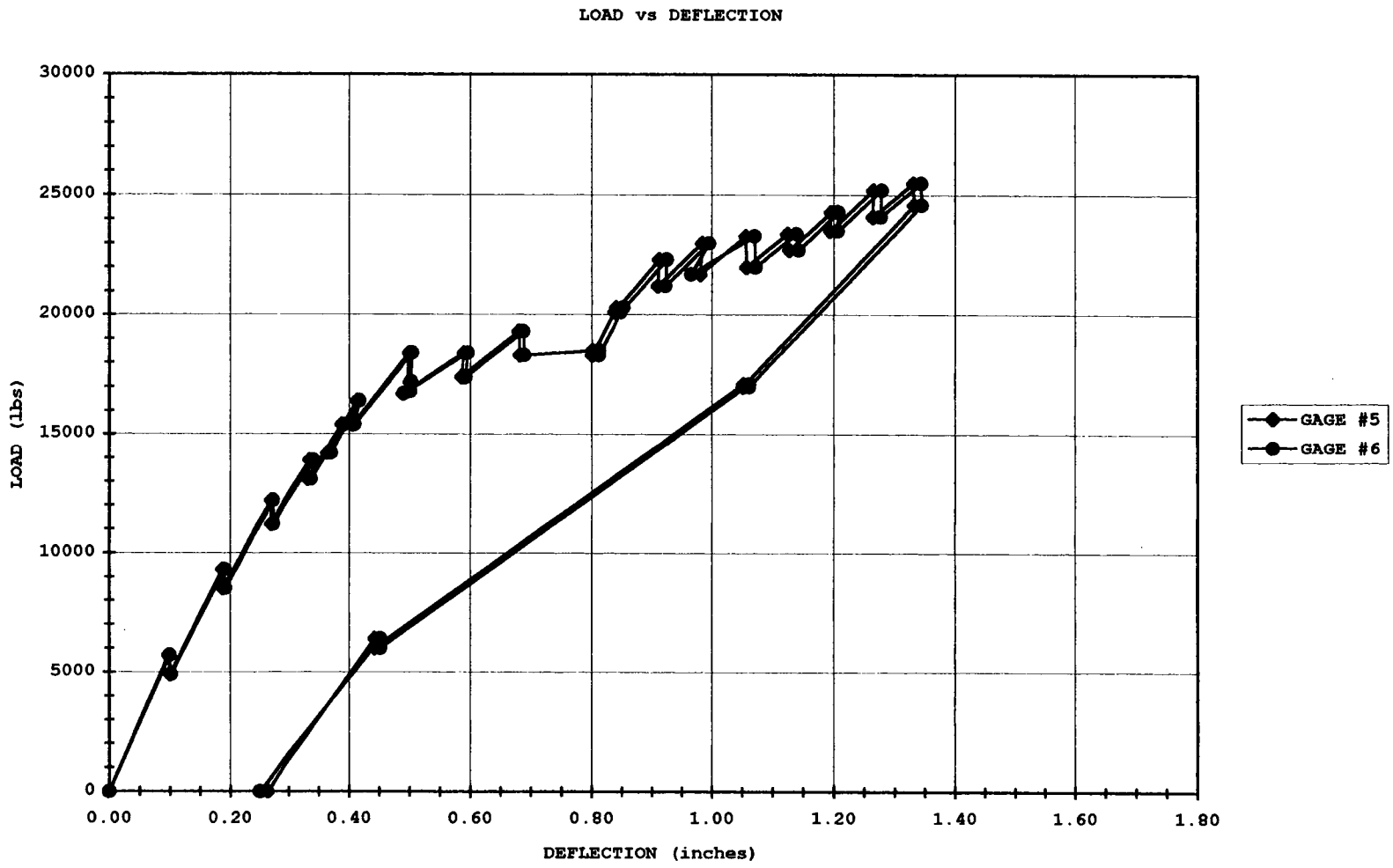


Figure 6. Load versus deflection at beginning and end of each time period, (Test 1, Pile B, Gages 5 and 6).

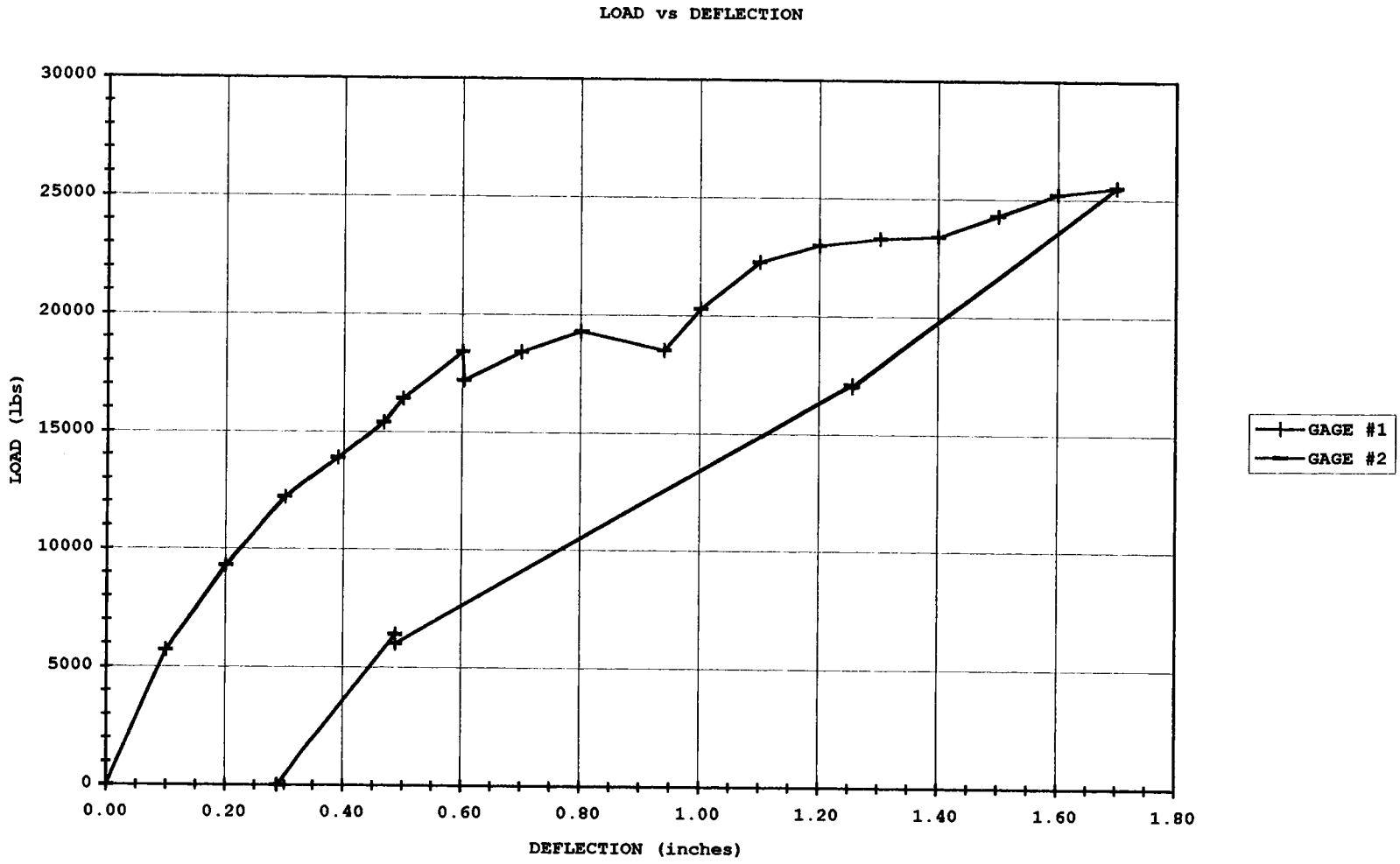


Figure 7. Load versus deflection at beginning of each time period, (Test 1, Pile A, Gages 1 and 2).

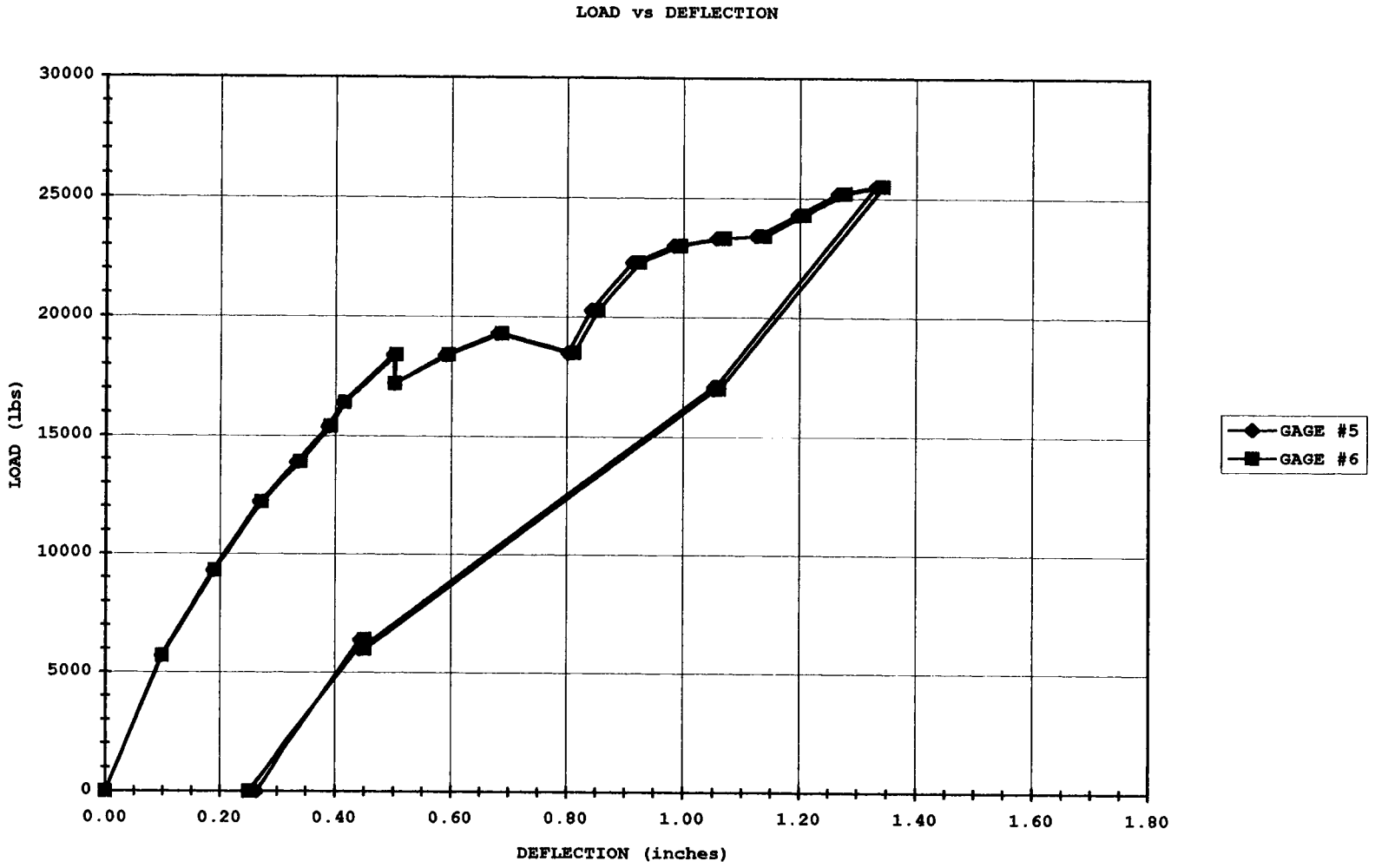


Figure 8. Load versus deflection at beginning of each time period, (Test 1, Pile B, Gages 5 and 6).

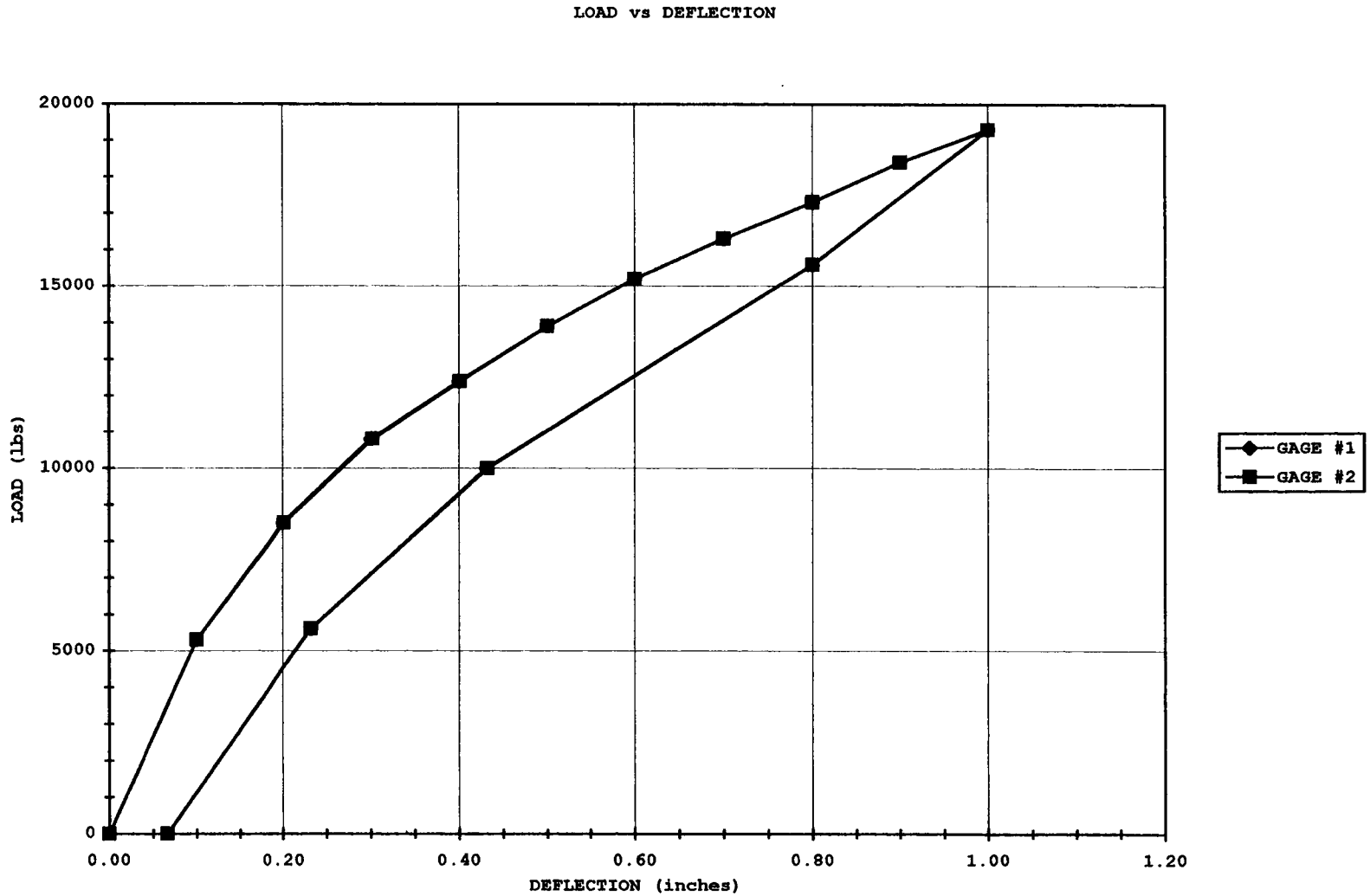


Figure 9. Load versus deflection (Test 2, Piles A and B, Gages 1 and 2).

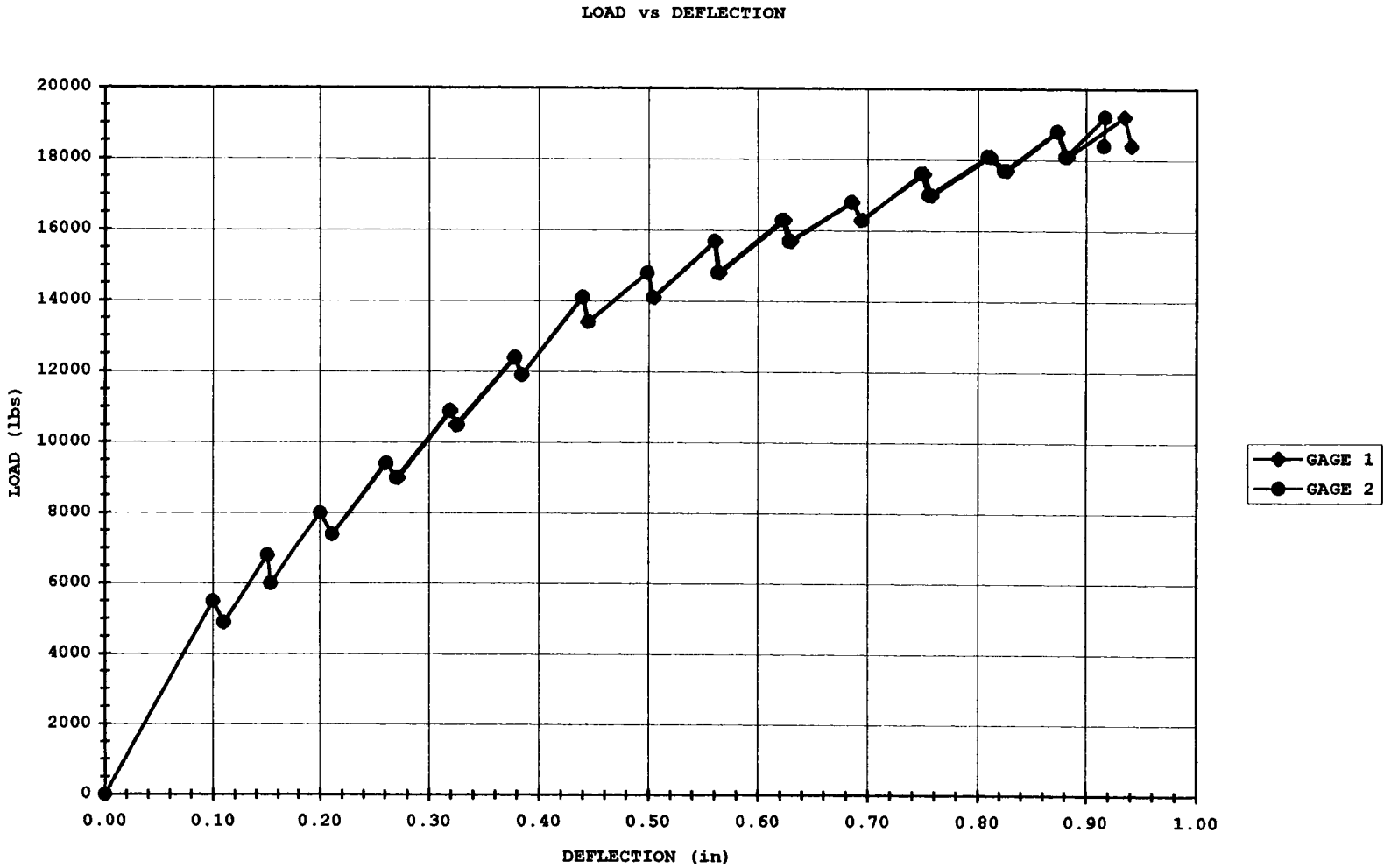


Figure 10. Load versus deflection at beginning and end of each time period, (Test 3, Pile C, Gages 1 and 2).

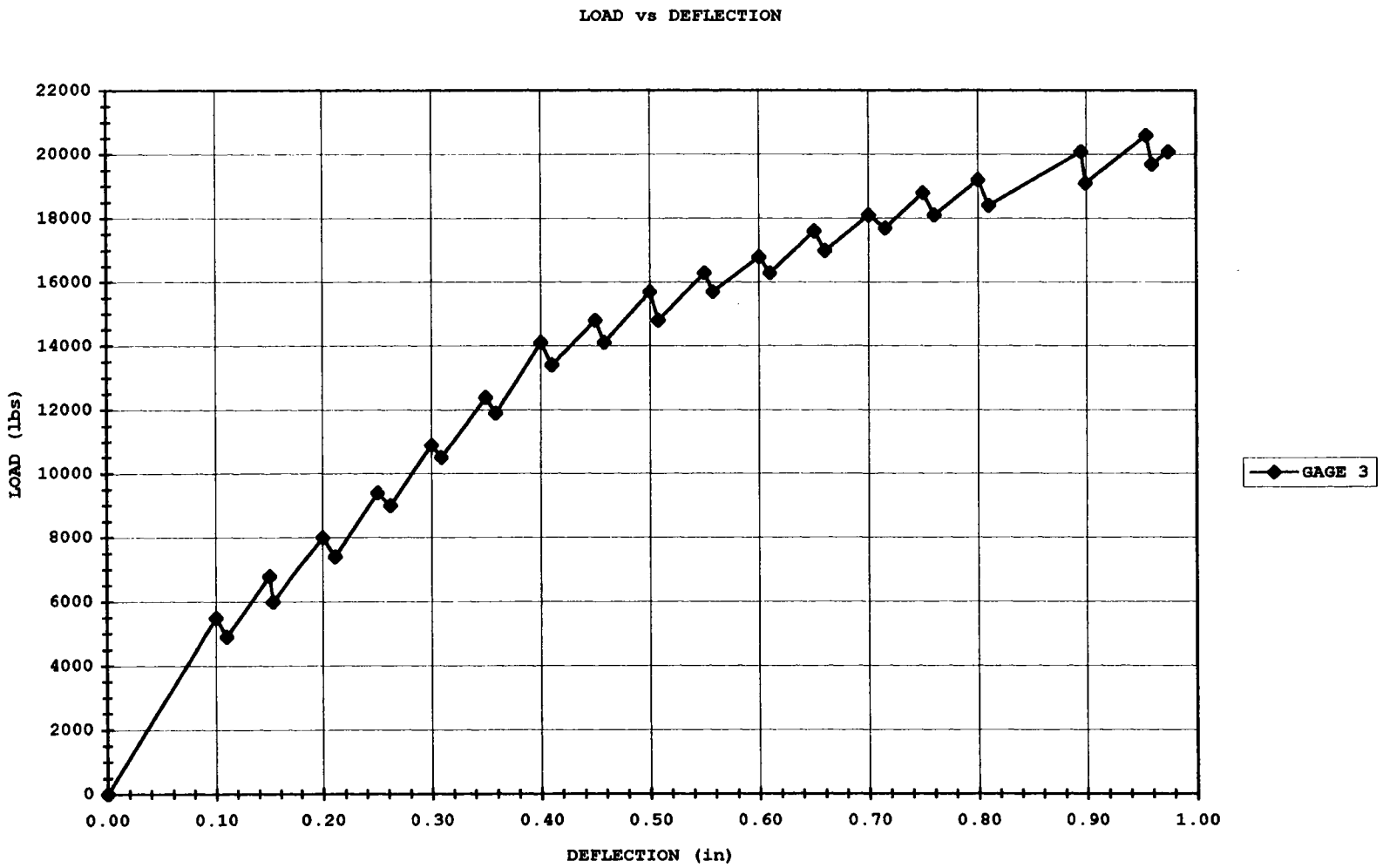


Figure 11. Load versus deflection at beginning and end of each time period, (Test 3, Pile C, Gage 3).

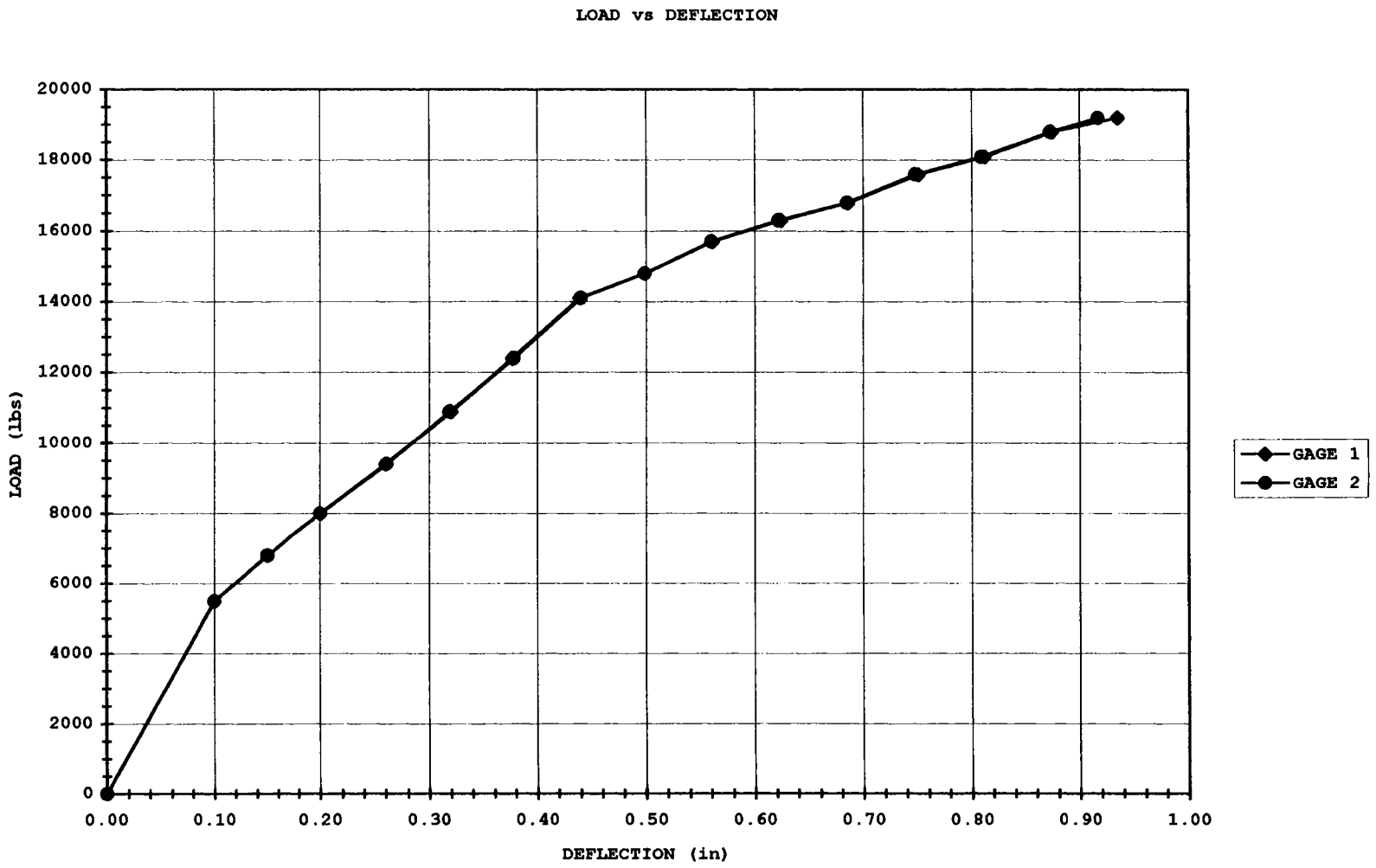


Figure 12. Load versus deflection at beginning of each time period, (Test 3, Pile C, Gages 1 and 2).

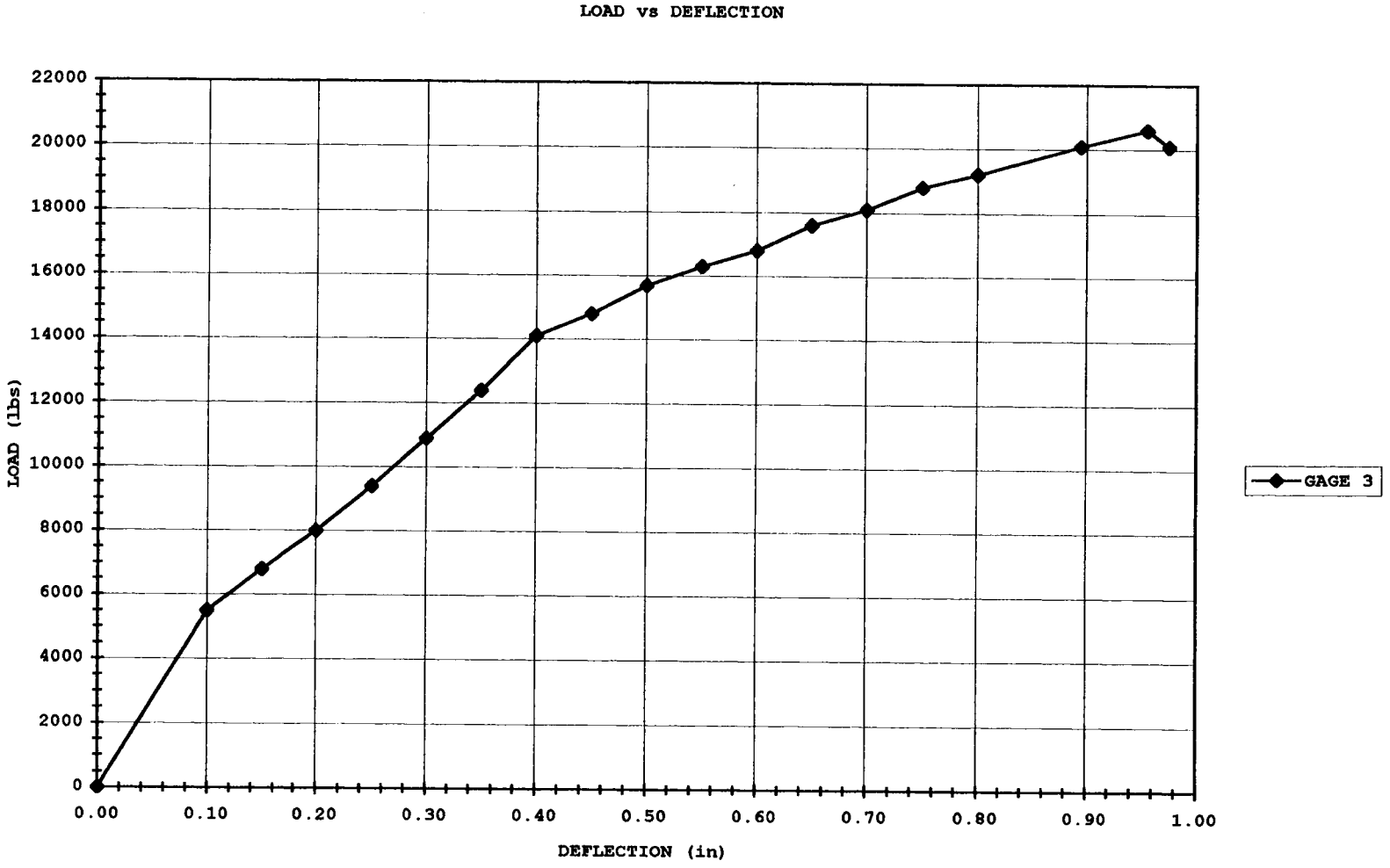
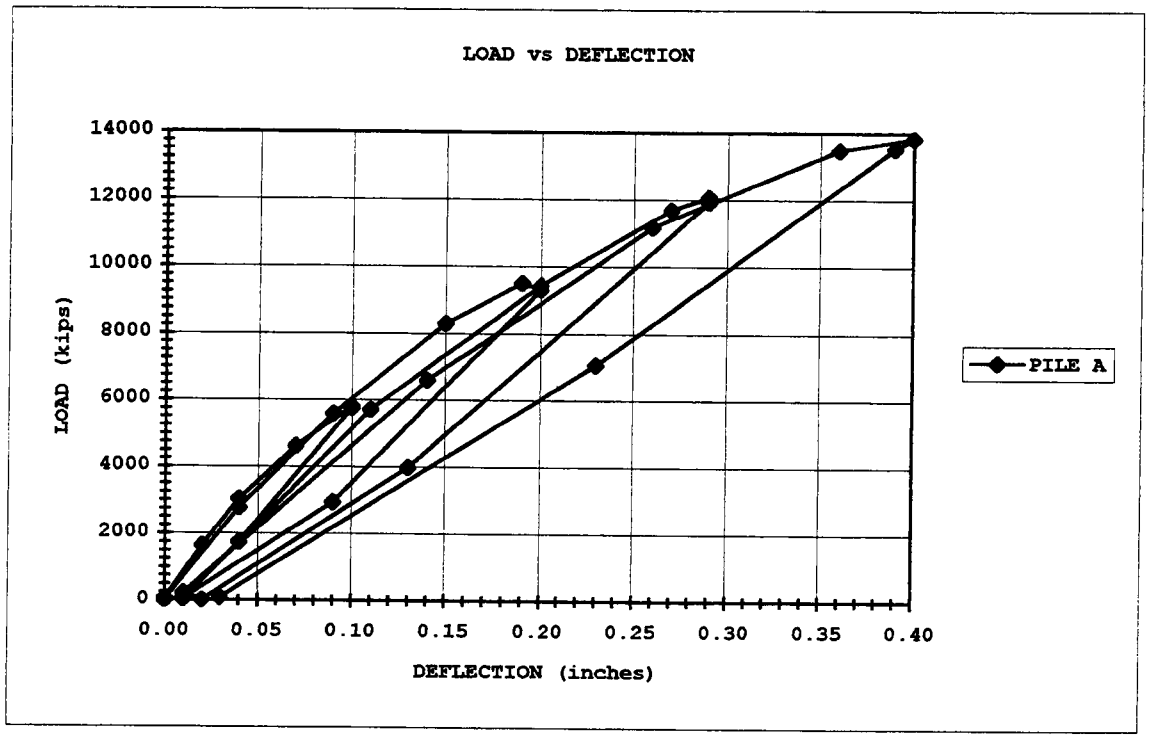


Figure 13. Load versus deflection at beginning of each time period, (Test 3, Pile C, Gage 3).



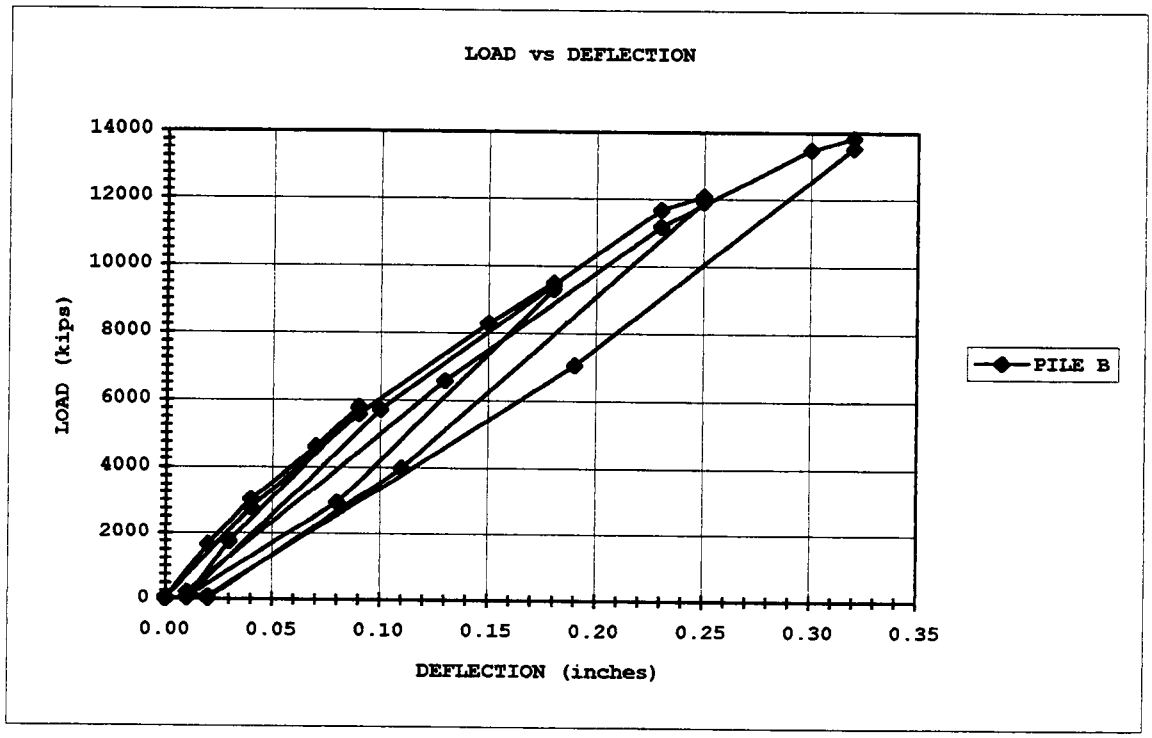
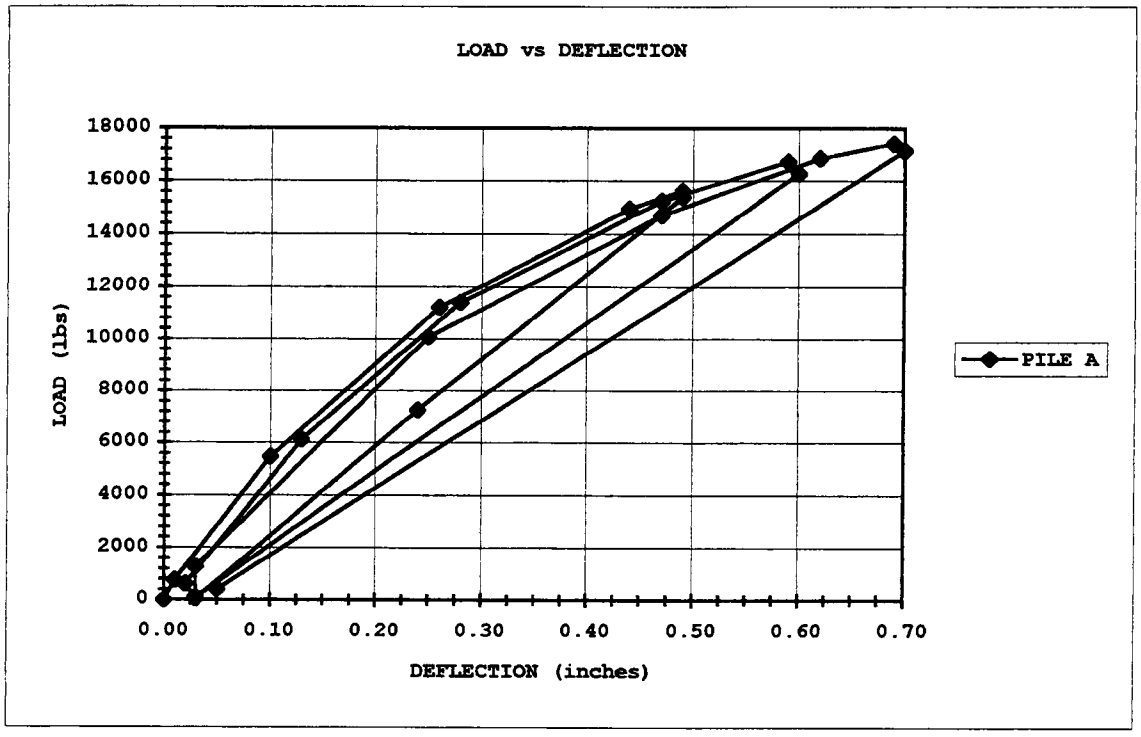


Figure 14. Load versus deflection, cyclic test, (Test 4b, Piles A and B).



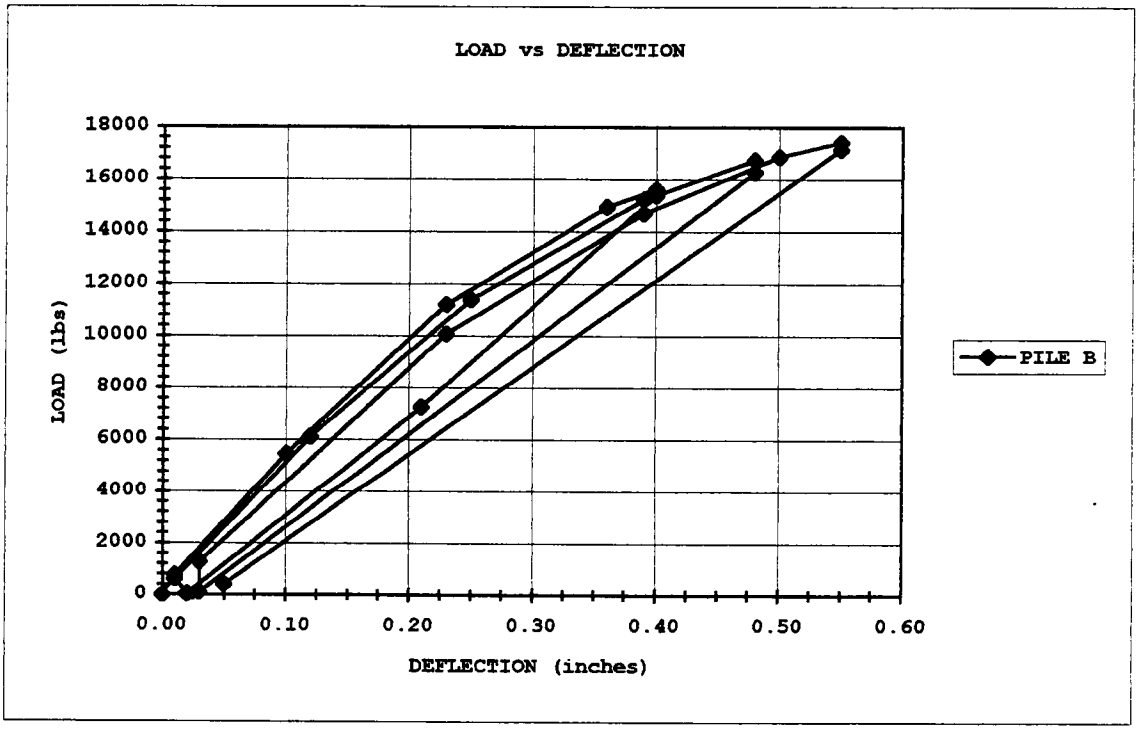
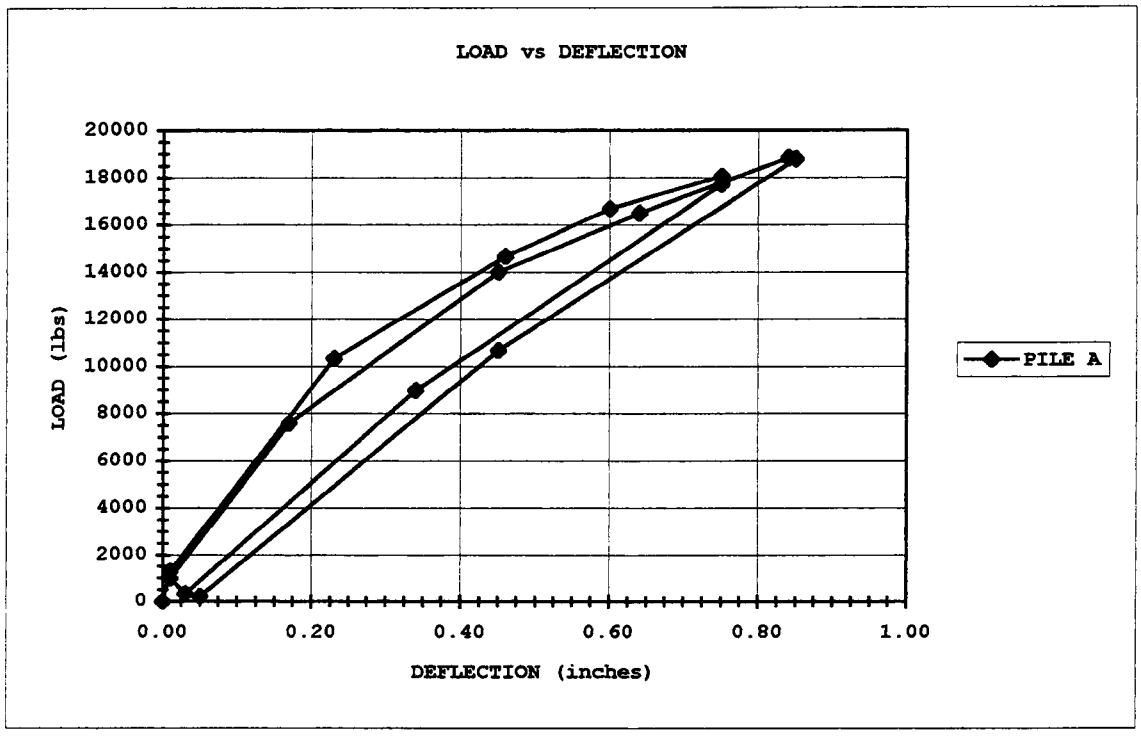


Figure 15. Load versus deflection, cyclic test, (Test 4c, Piles A and B).



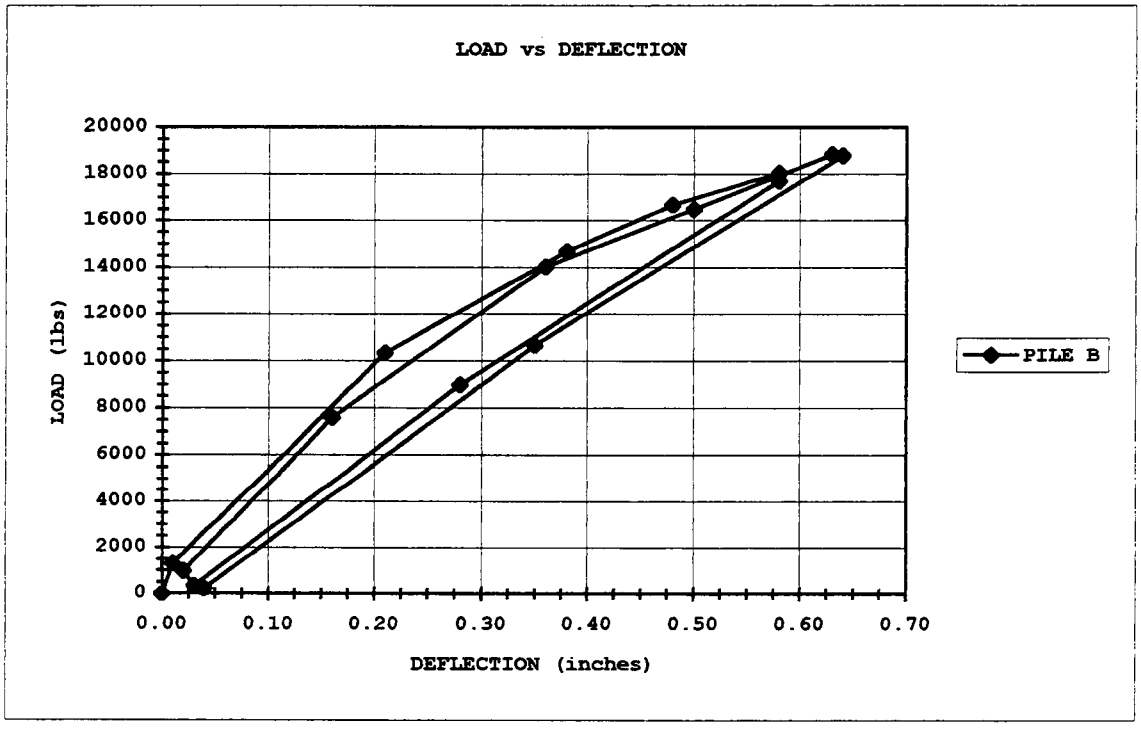
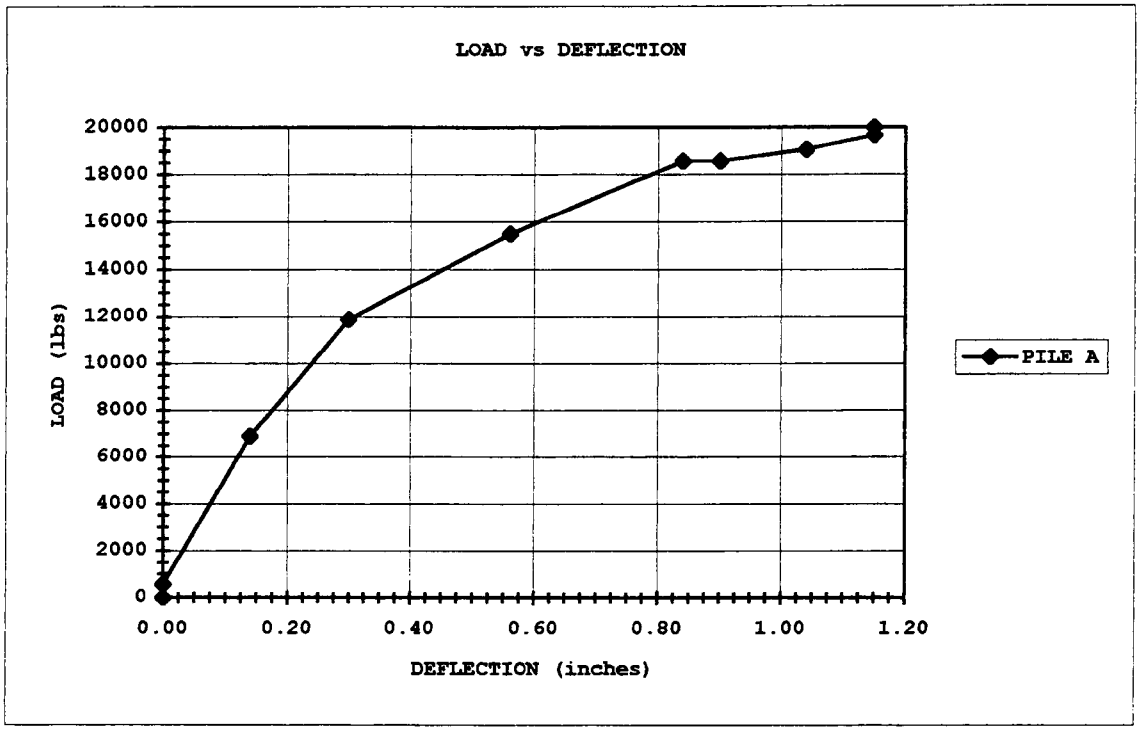


Figure 16. Load versus deflection, cyclic test, (Test 4d, Piles A and B).



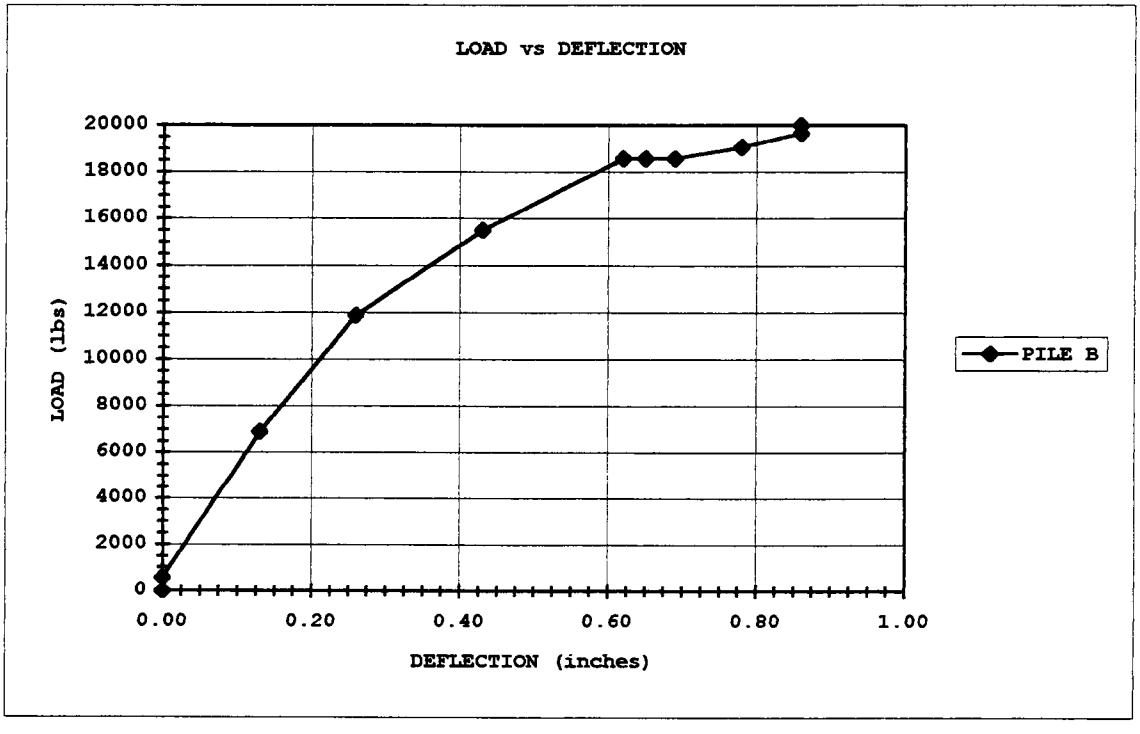
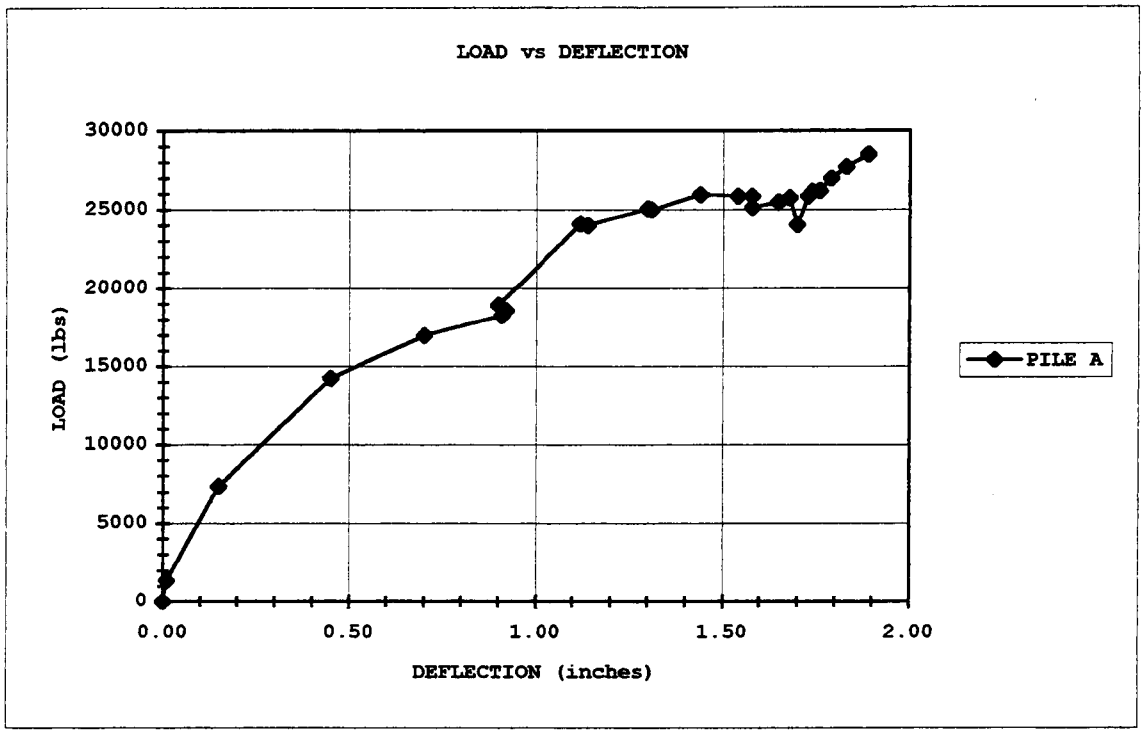


Figure 17. Load versus deflection, cyclic test, (Test 4e, Piles A and B).



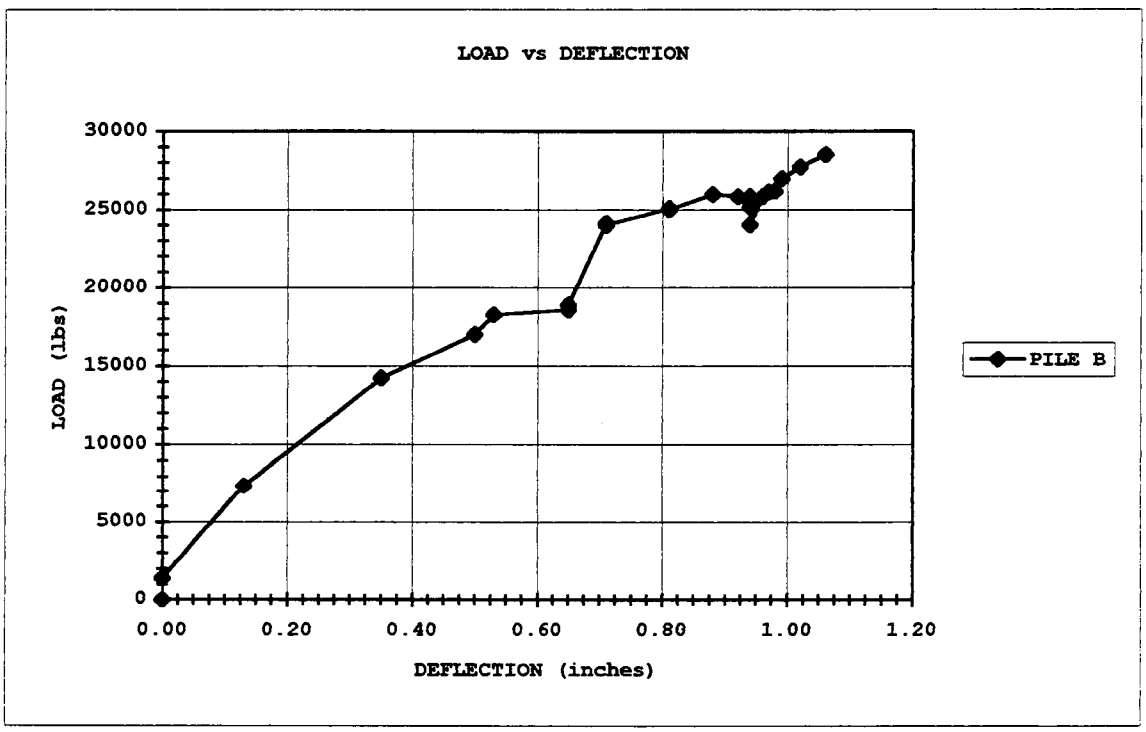


Figure 18. Load versus deflection, cyclic test, (Test 4f, Piles A and B).

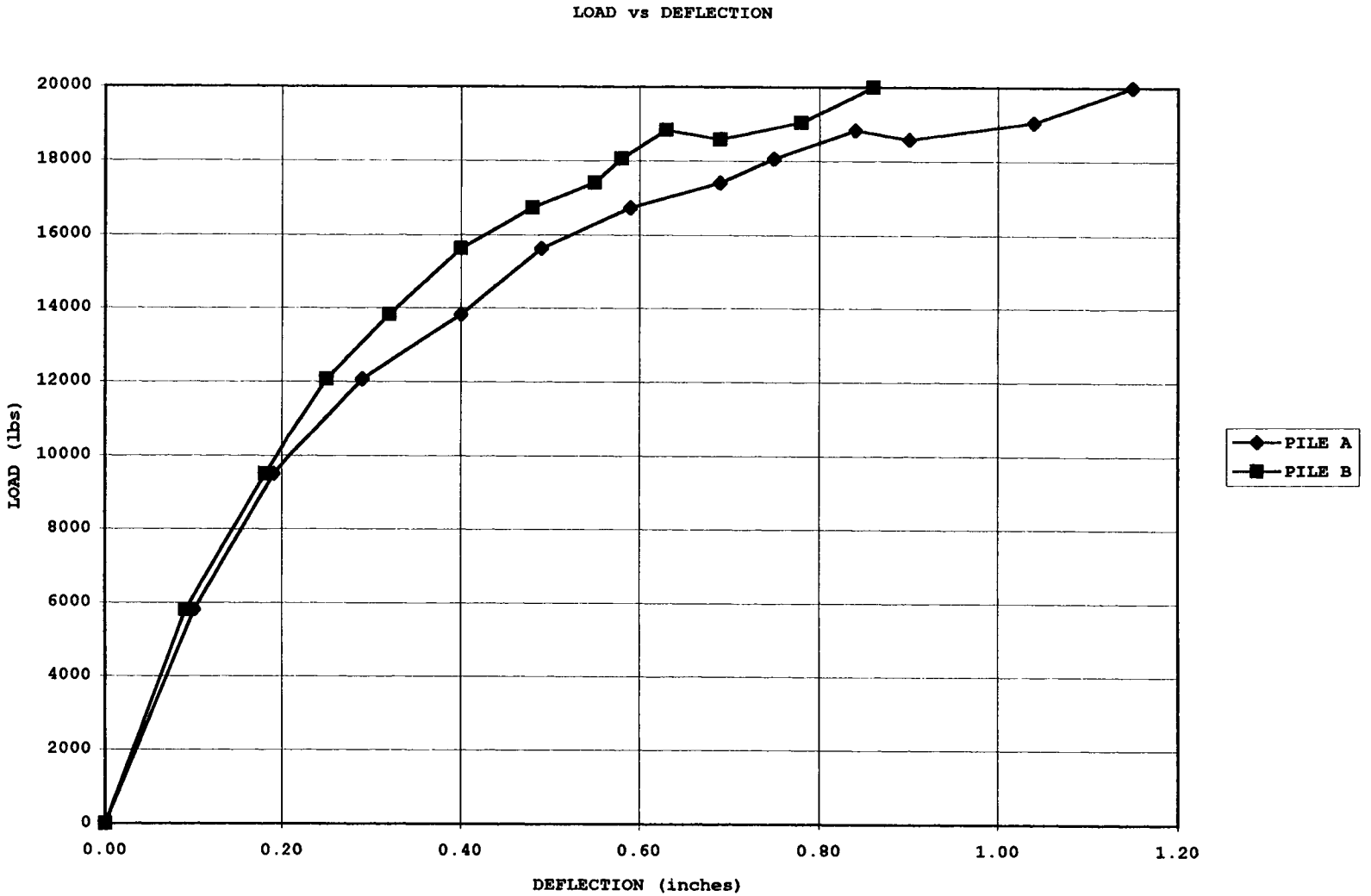
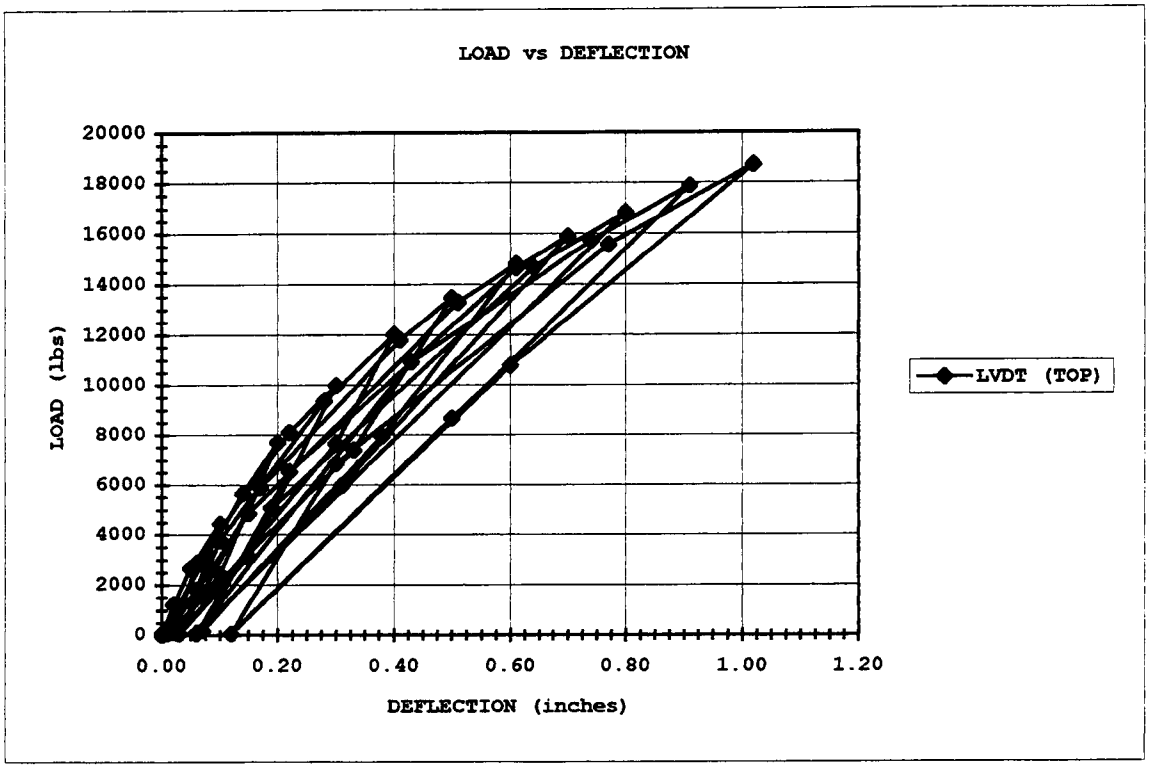


Figure 19. Envelope of load versus deflection, cyclic test, (Test 4, Piles A and B).



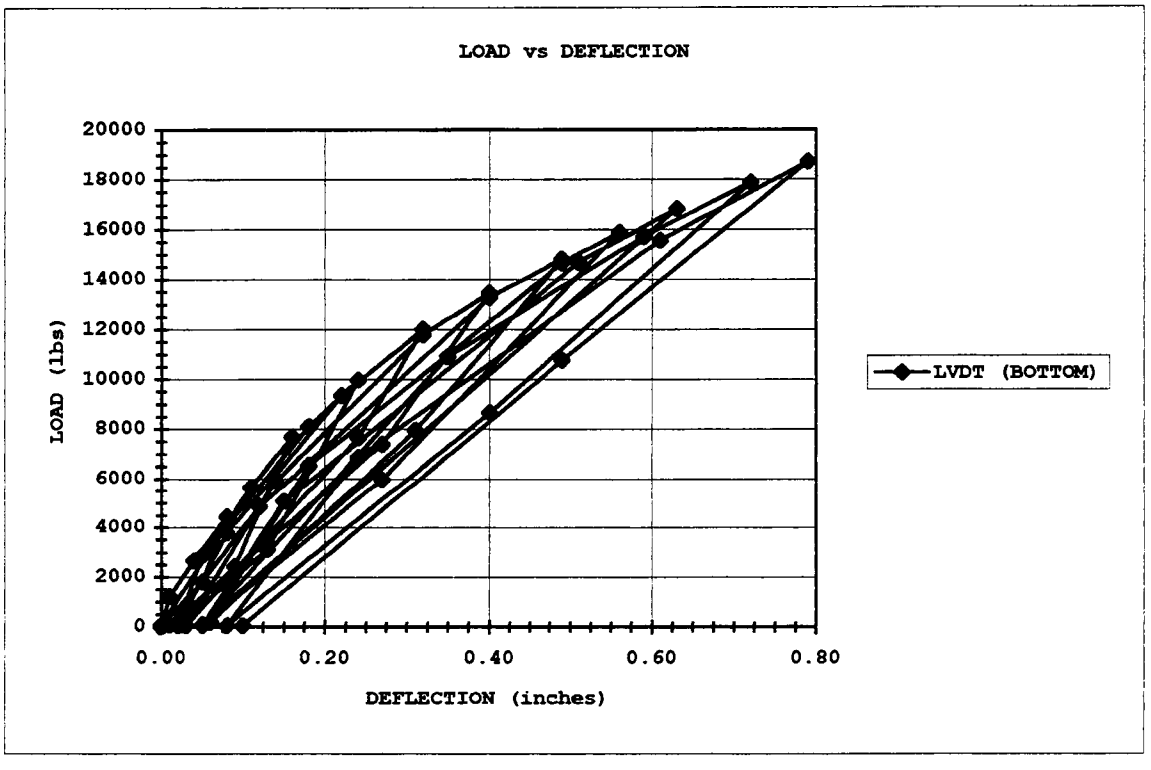


Figure 20. Load vs deflection, cyclic test, (Test 5, Pile C).

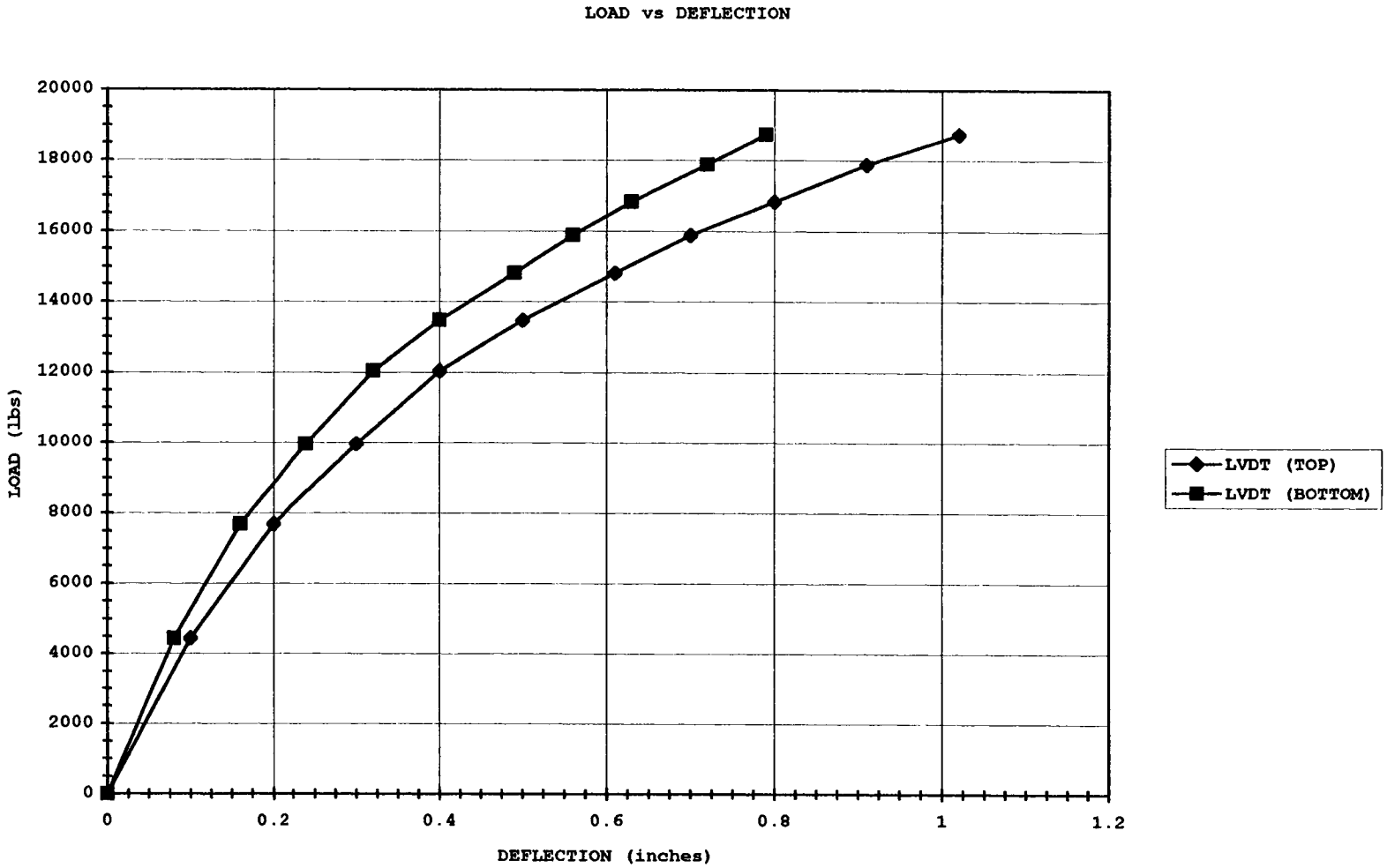


Figure 21. Envelope of load versus deflection, cyclic test, (Test 5, Pile C).

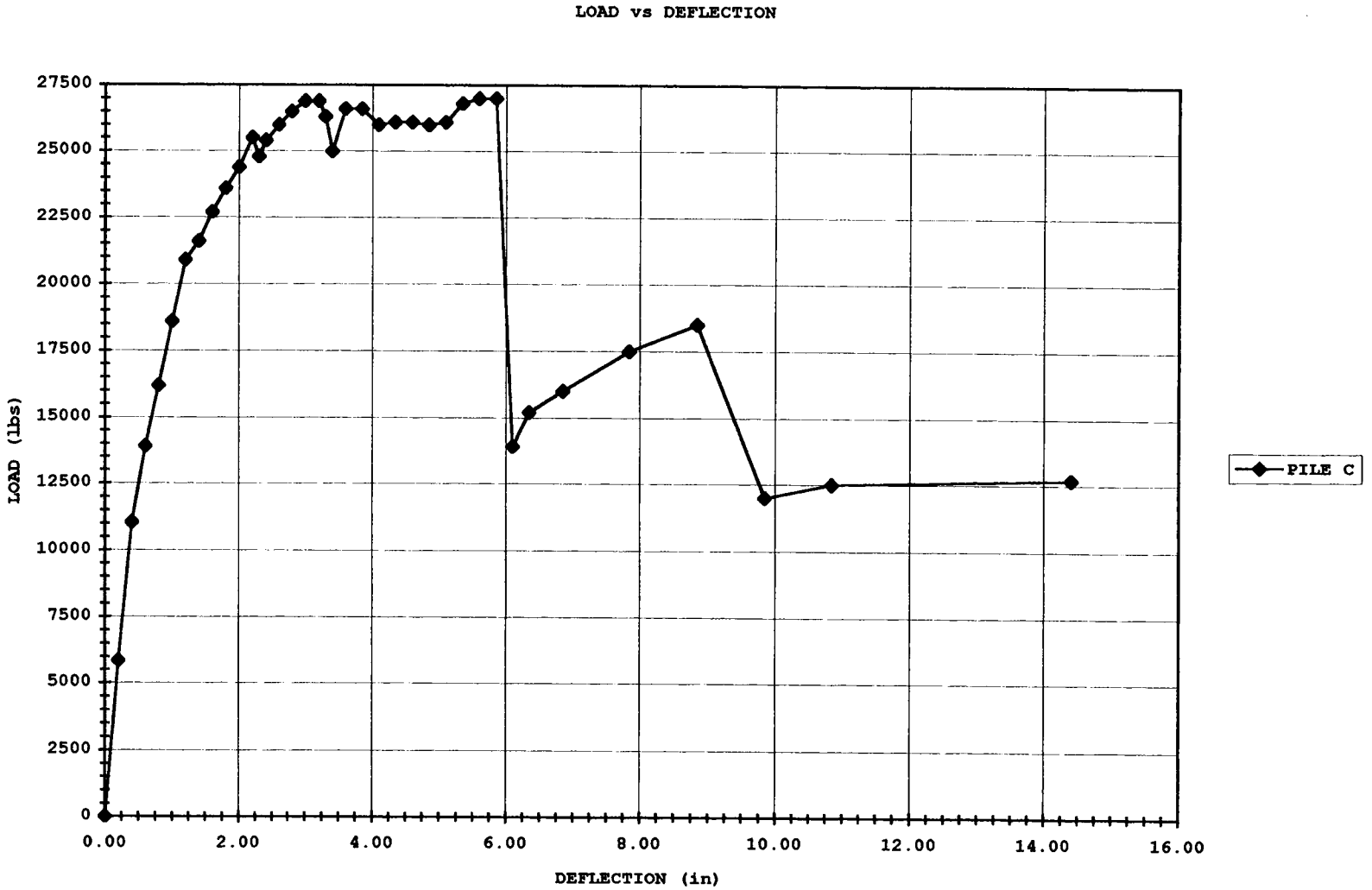


Figure 22. Load versus deflection, test to ultimate, (Test 6, Pile C).

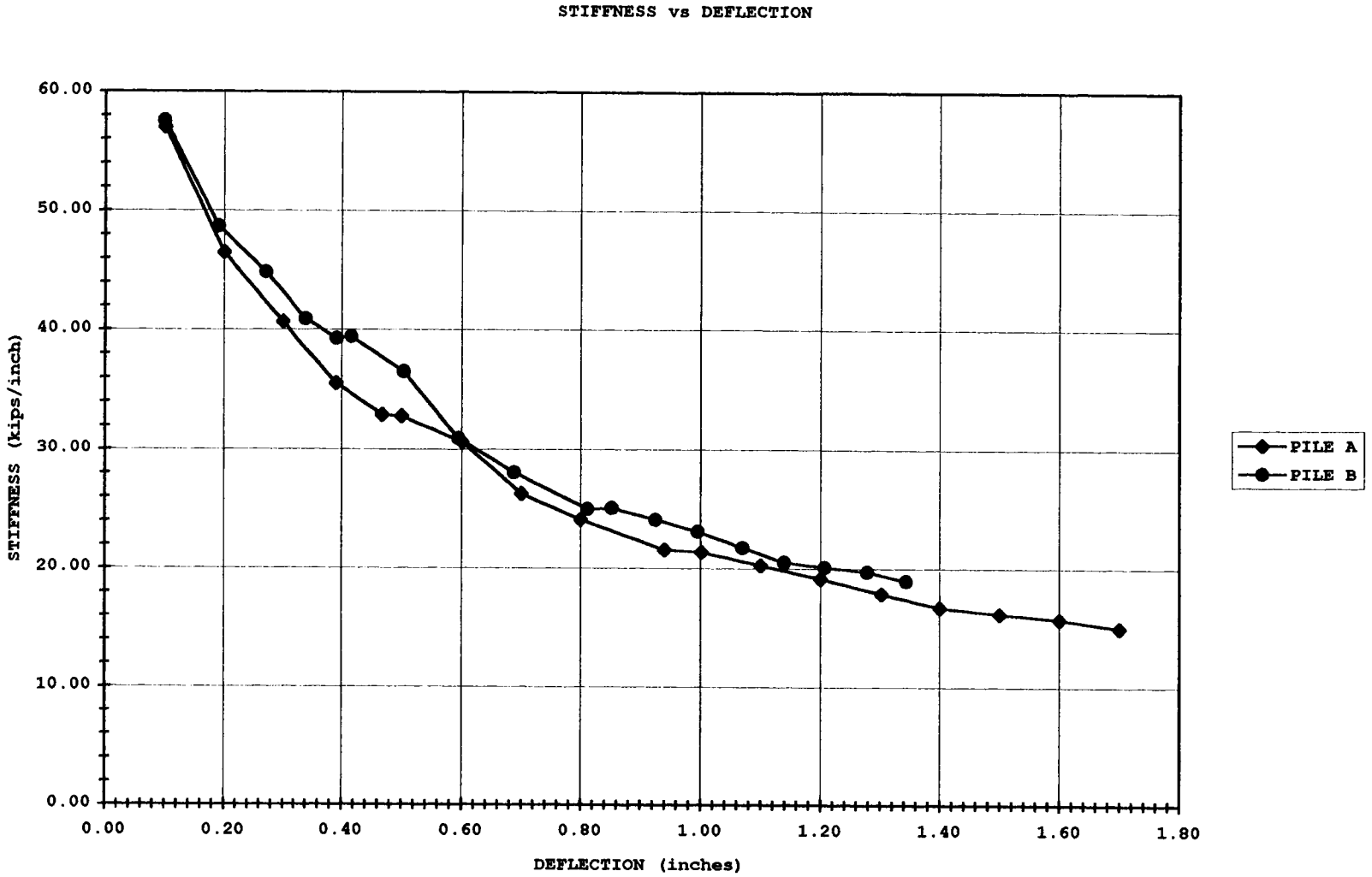


Figure 23. Secant stiffness versus deflection, (Test 1, Piles A and B).

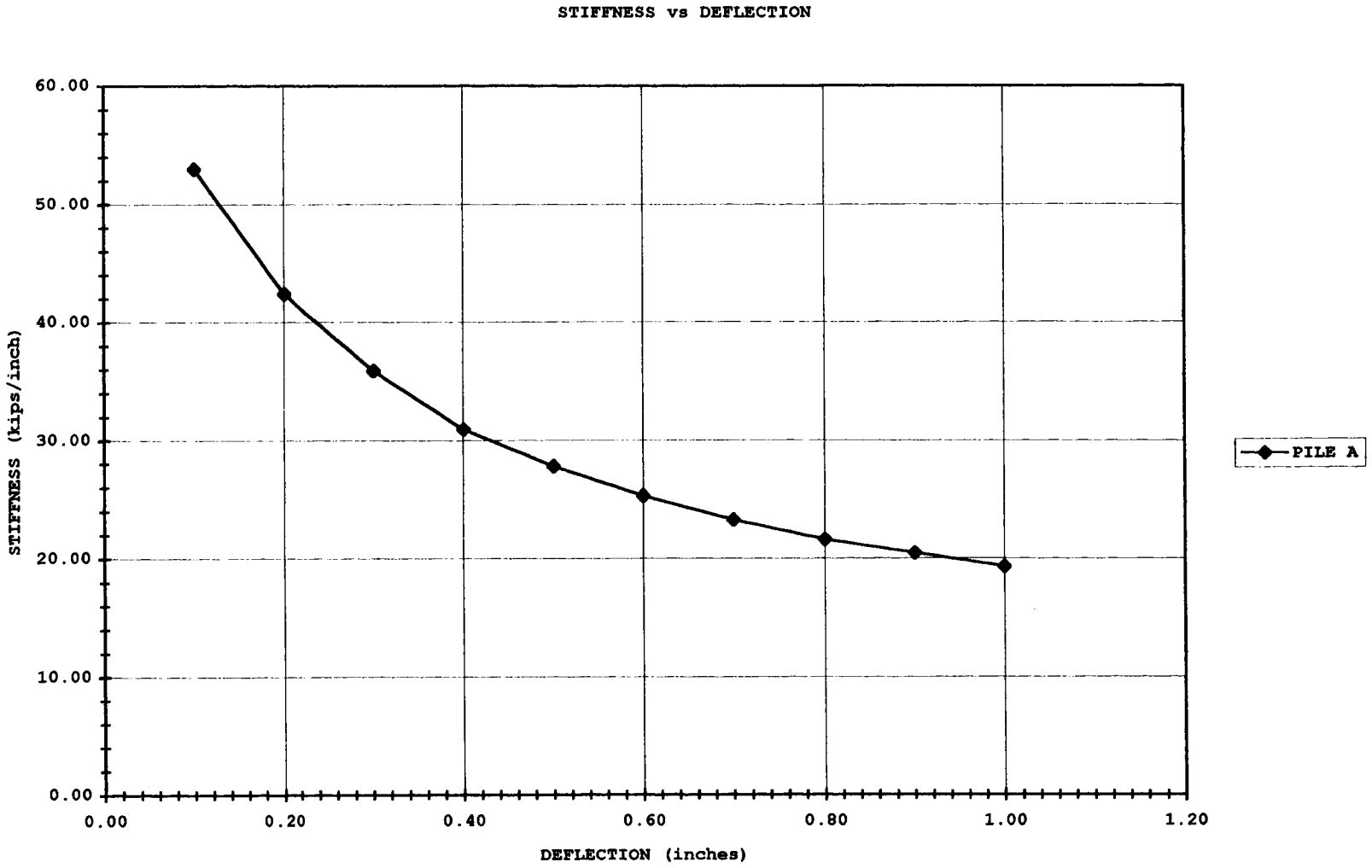


Figure 24. Secant stiffness versus deflection, (Test 2, Pile A).

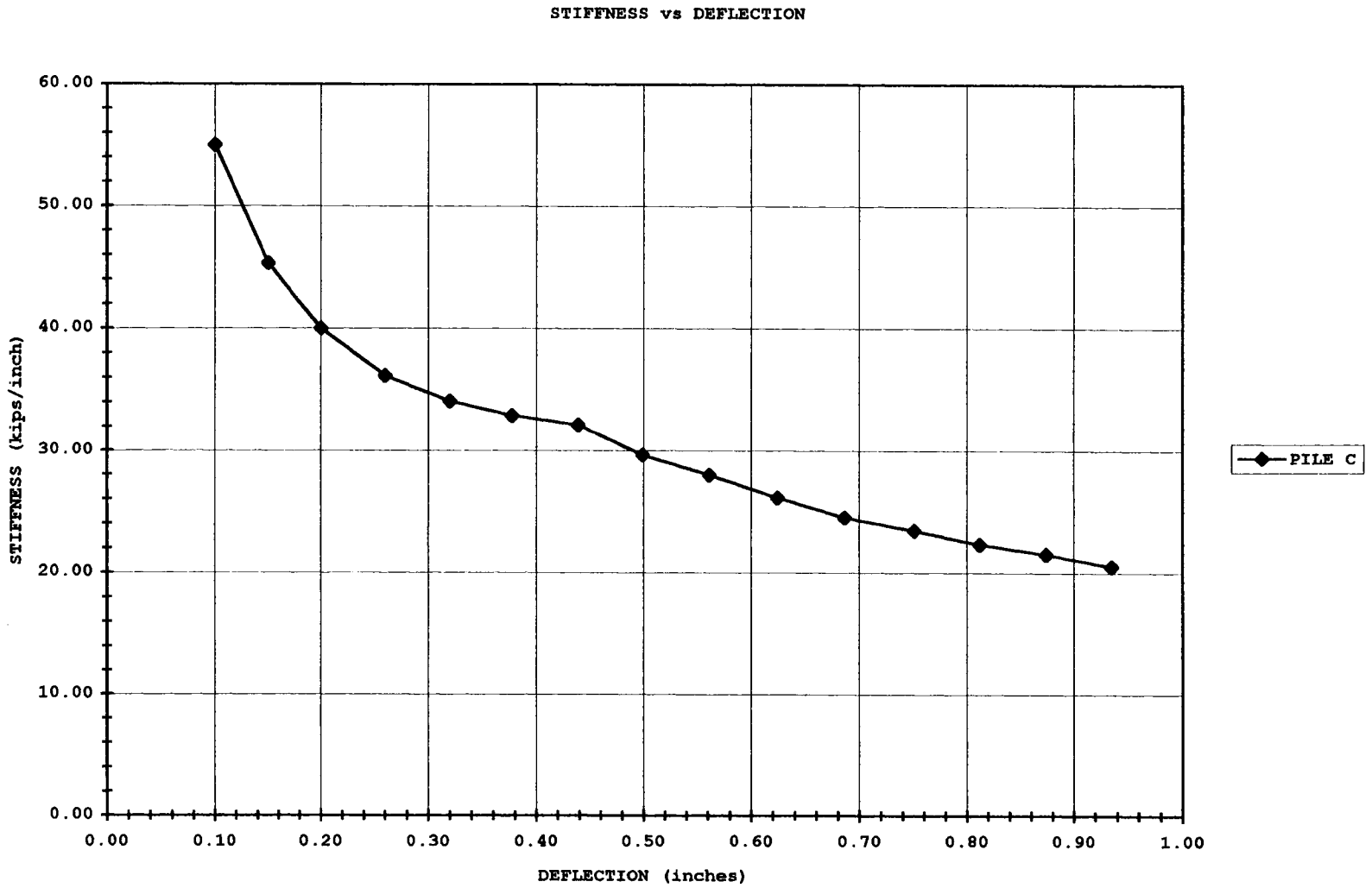


Figure 25. Secant stiffness versus deflection, (Test 3, Pile C).

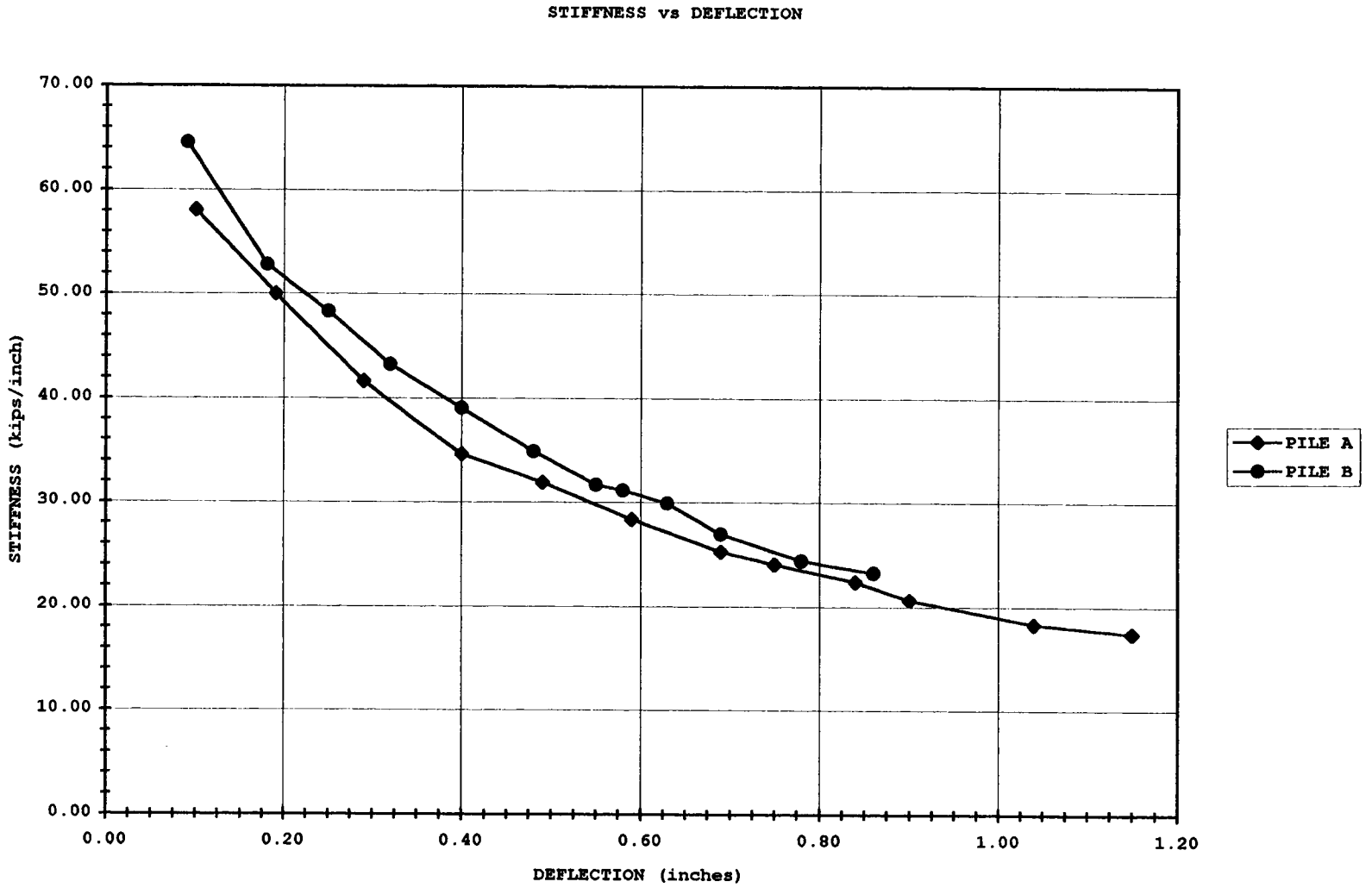


Figure 26. Secant stiffness versus deflection, (Test 4, Piles A and B).

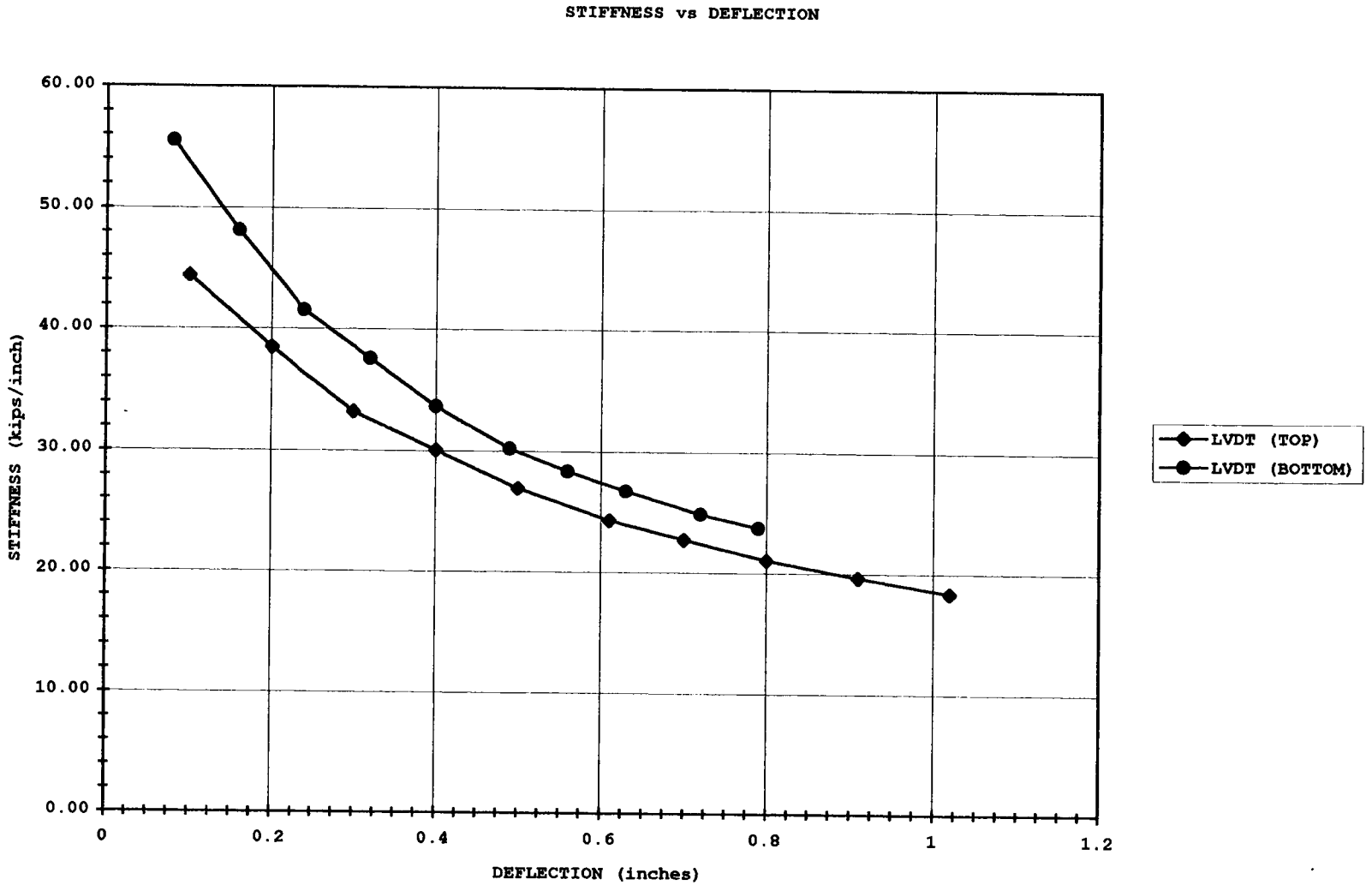


Figure 27. Secant stiffness versus deflection, (Test 5, Pile C).

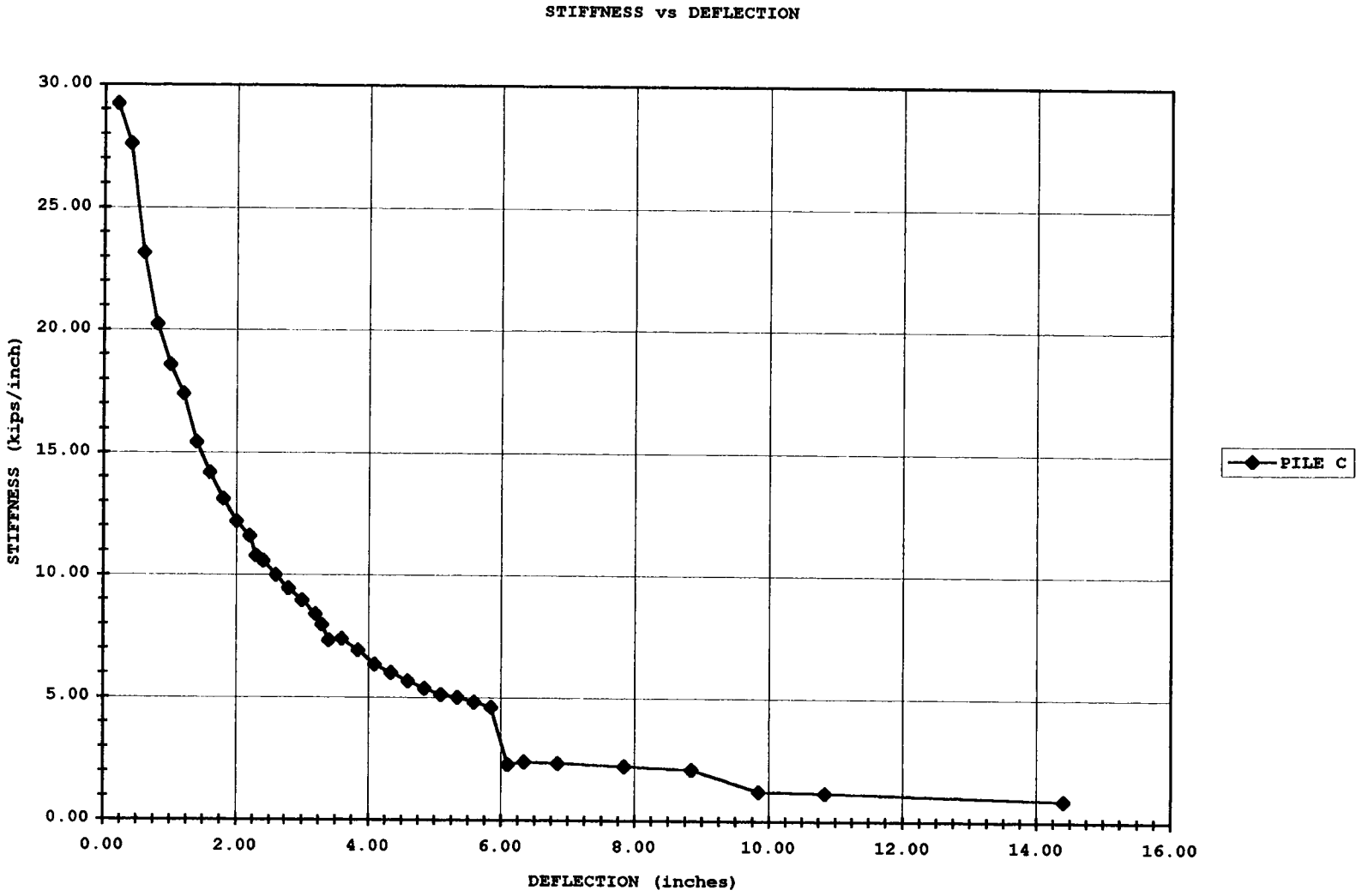


Figure 28. Secant stiffness versus deflection, (Test 6, Pile C).

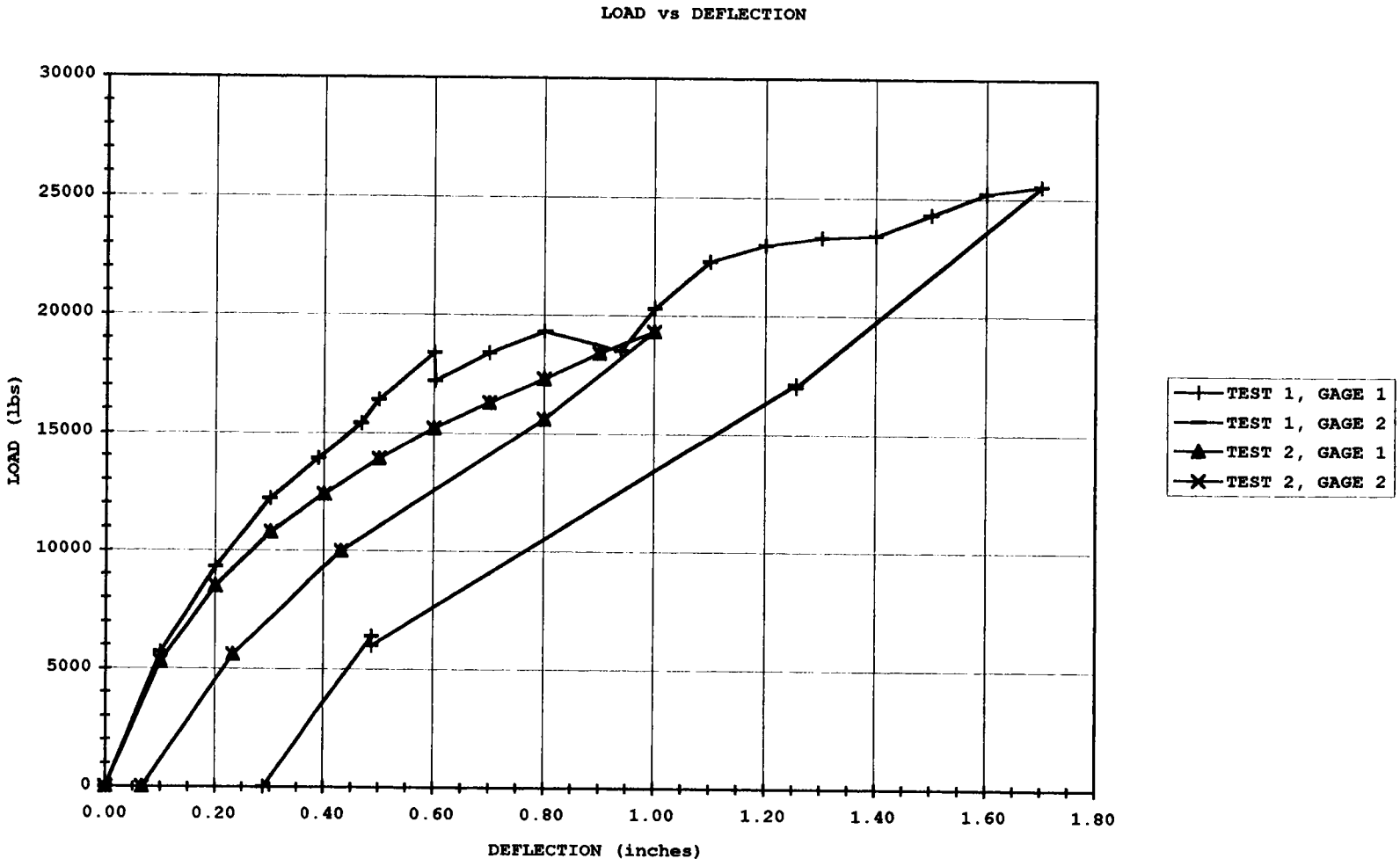


Figure 29. Comparison of load versus deflection, (Test 1, Pile A, Gages 1 and 2; Test 2, Pile A, Gages 1 and 2).

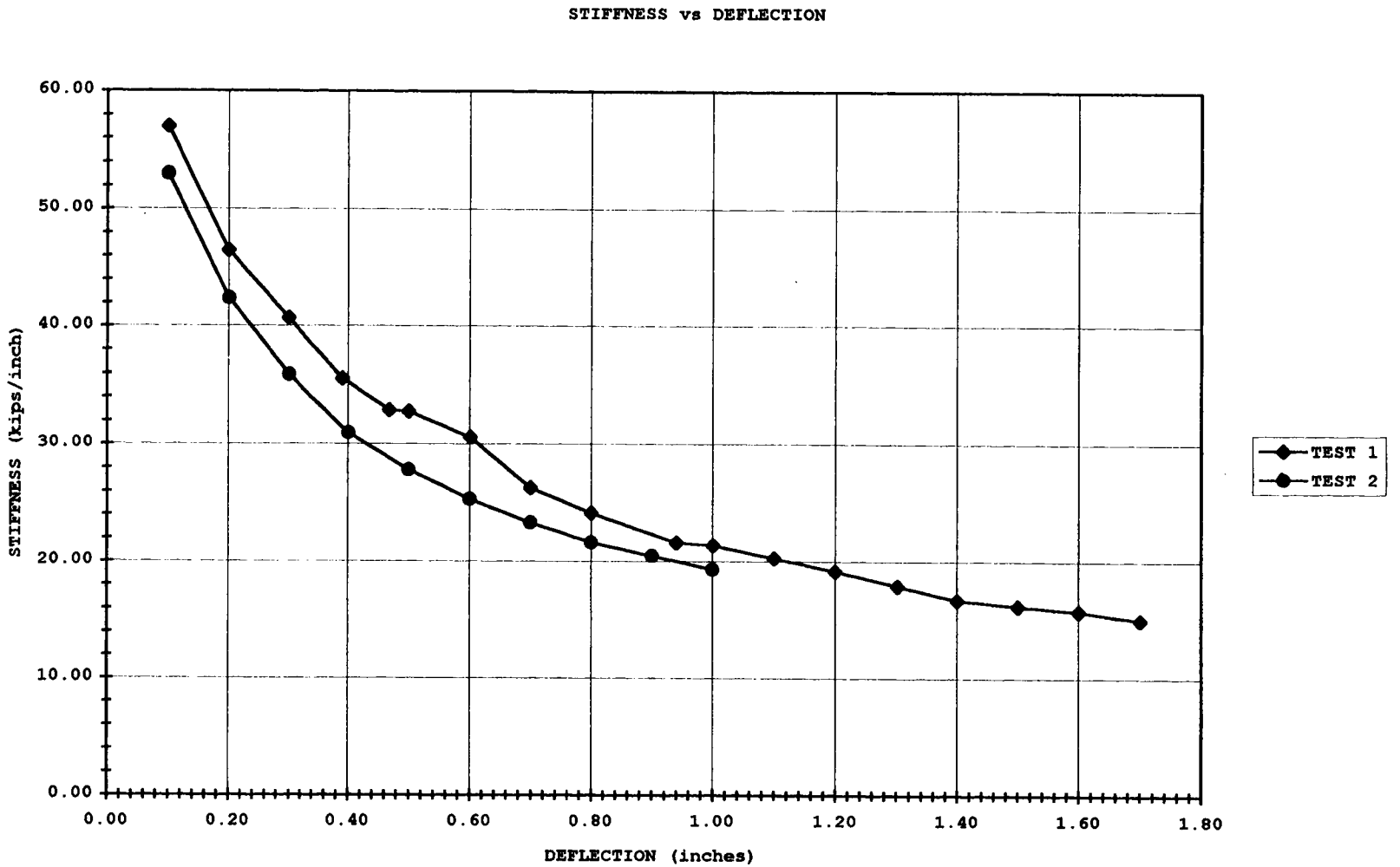


Figure 30. Comparison of secant stiffness versus deflection, (Test 1 and 2, Pile A).

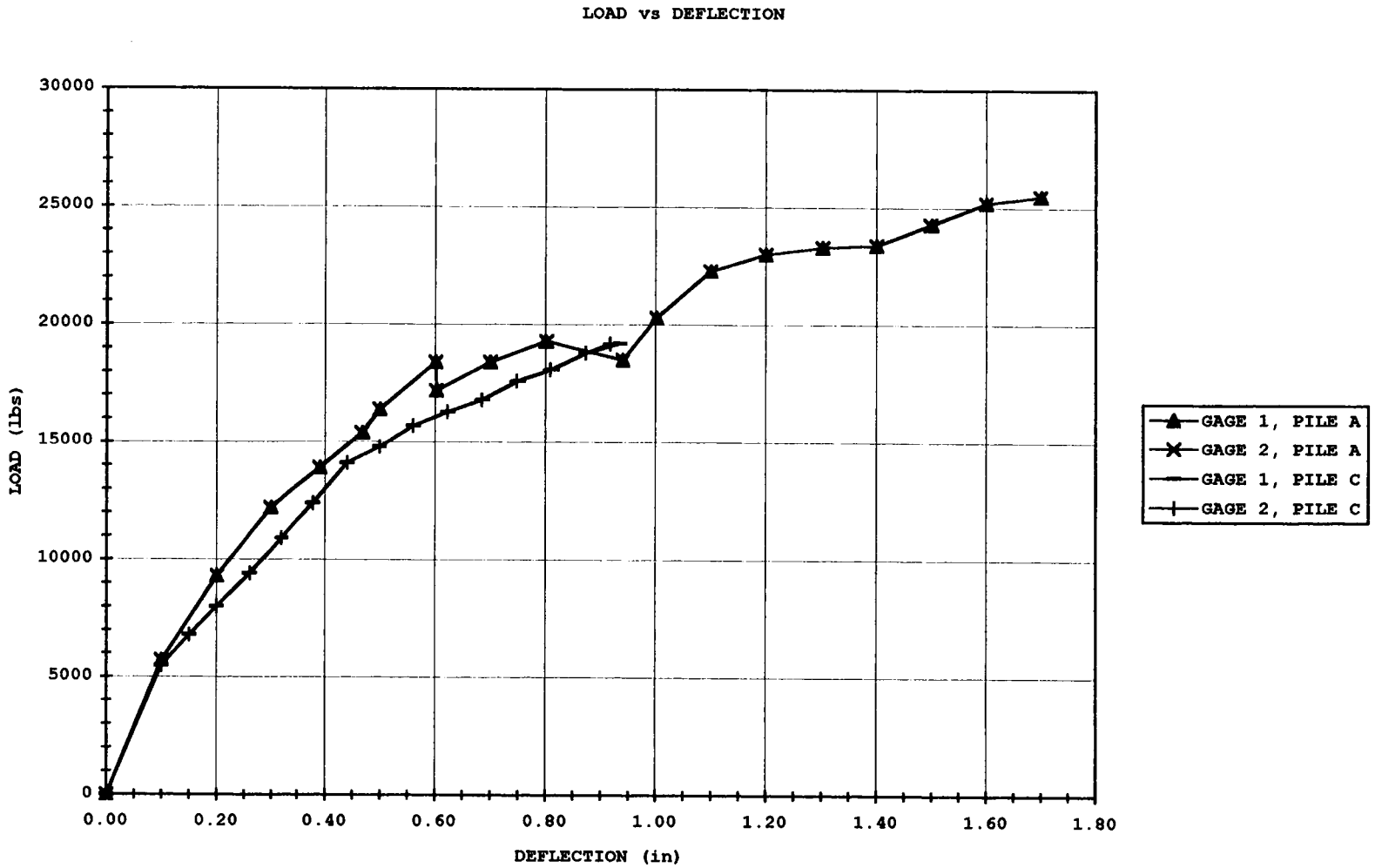


Figure 31. Comparison of load versus deflection, (Test 1, Pile A, Gages 1 and 2; Test 3, Pile C, Gages 1 and 2).

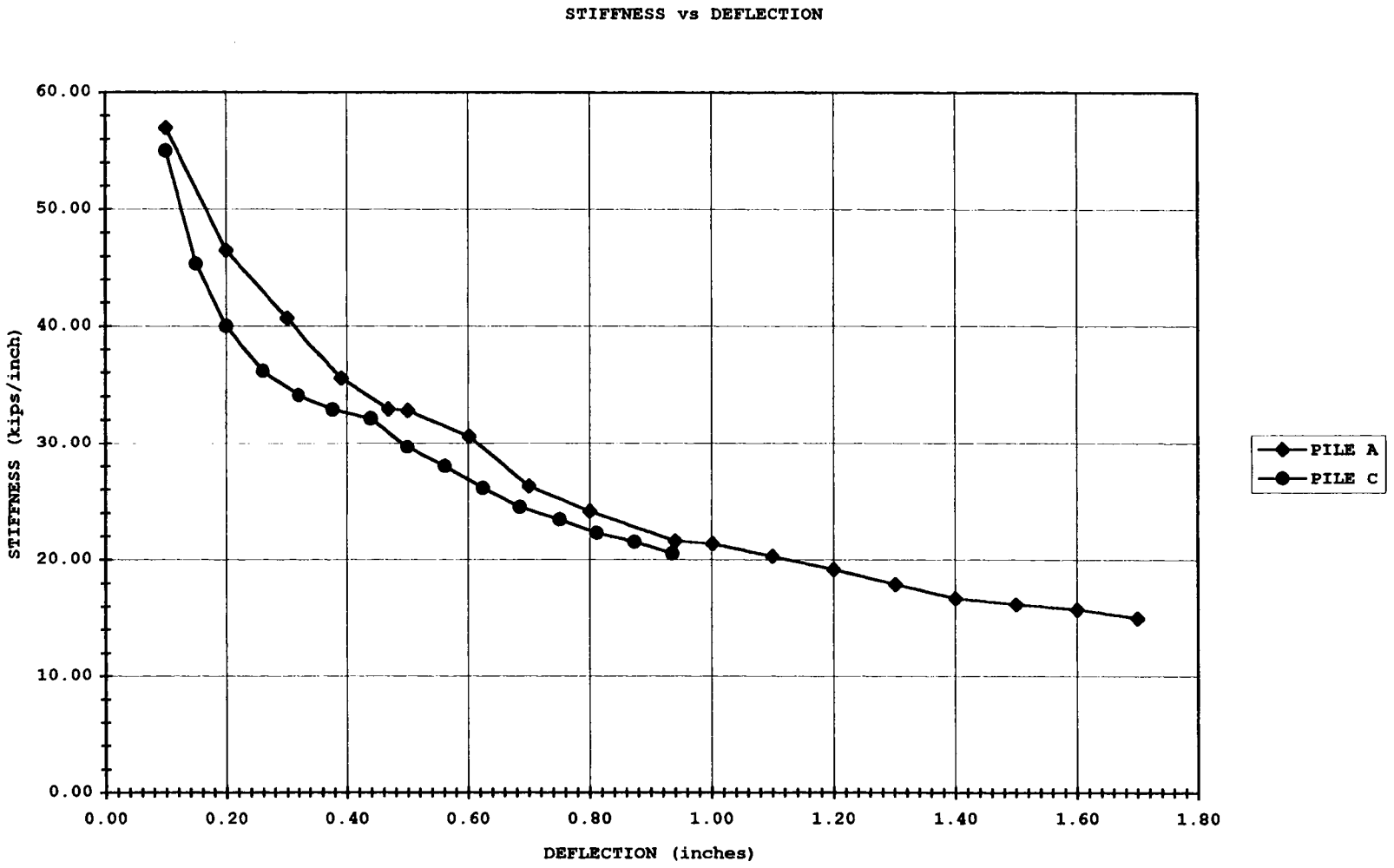


Figure 32. Comparison of secant stiffness versus deflection, (Test 1, Pile A; Test 3, Pile C).

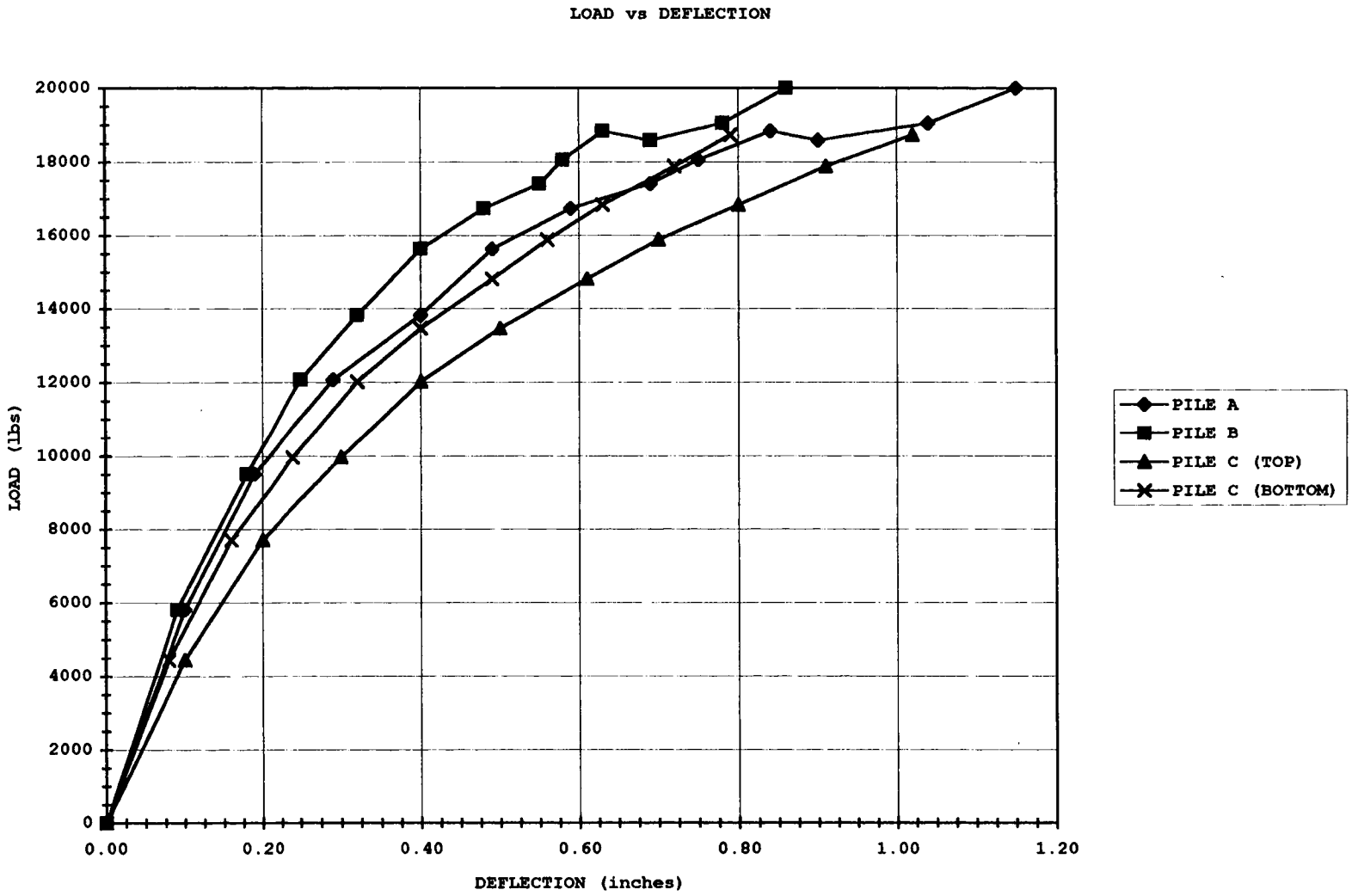


Figure 33. Comparison of load versus deflection, cyclic test, (Test 4, Piles A and B; Test 5, Pile C).

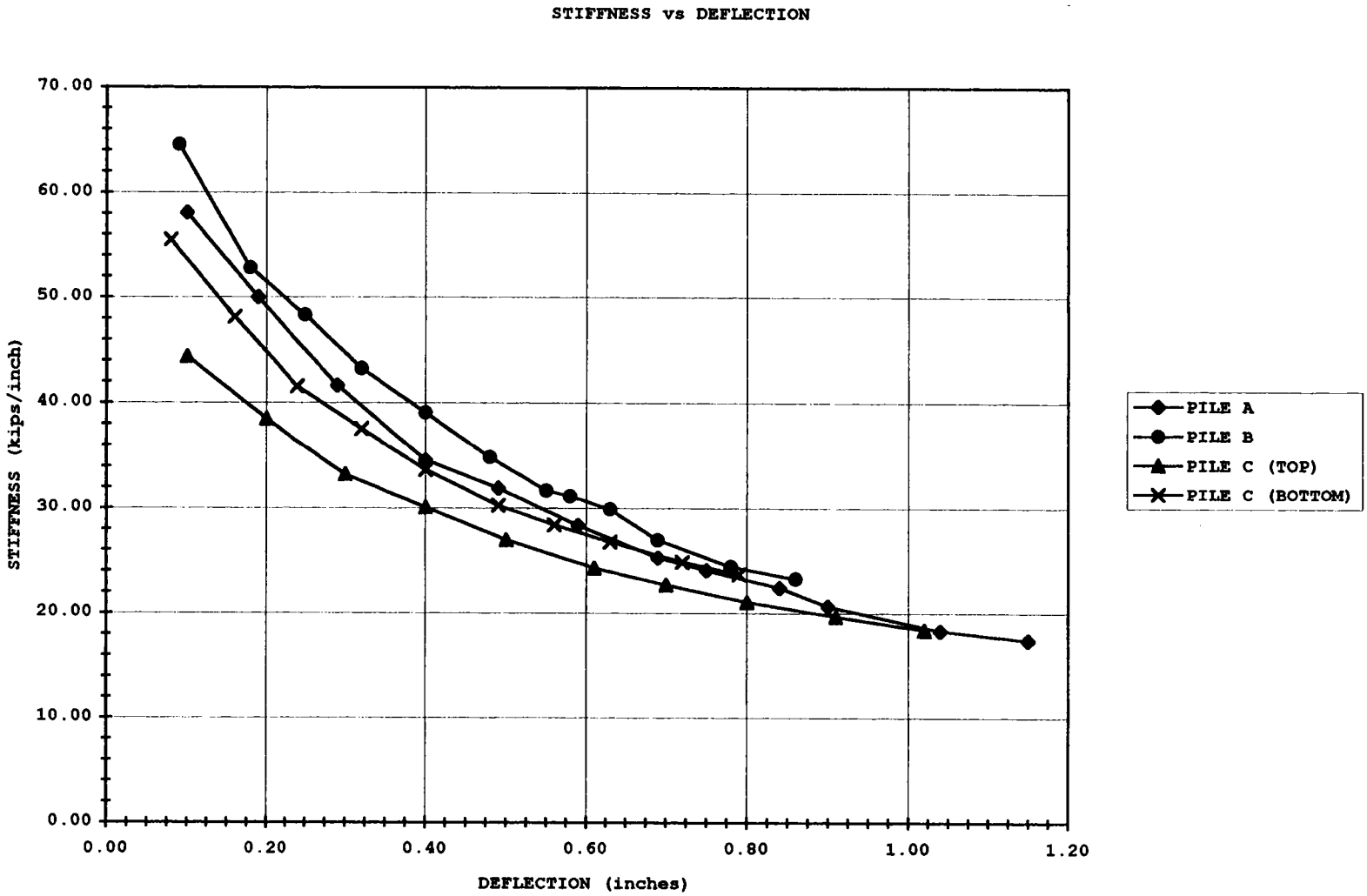


Figure 34. Comparison of secant stiffness versus deflection, (Test 4, Piles A and B; Test 5, Pile C).

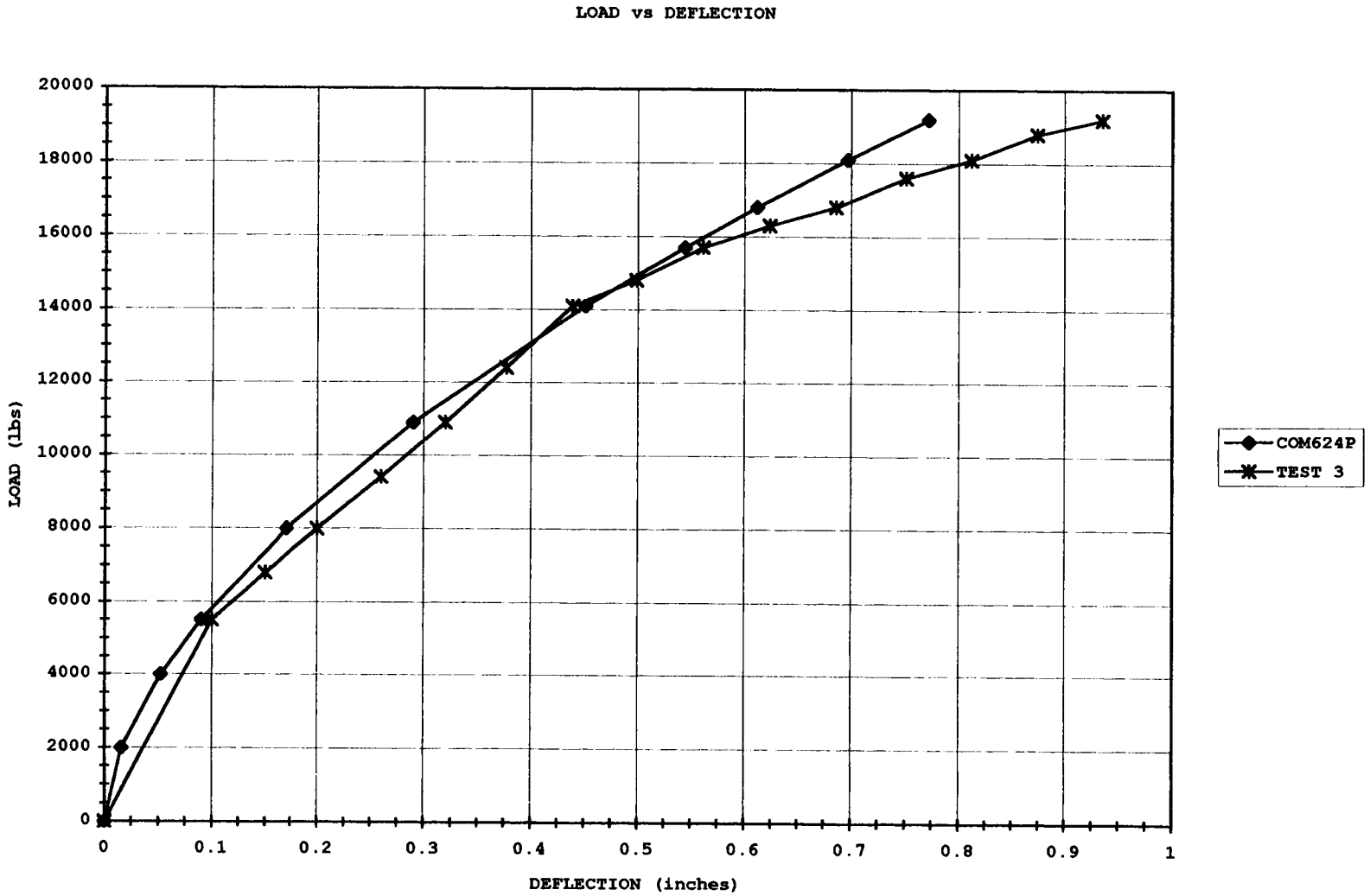


Figure 35. Comparison of load versus deflection, (COM624P, Test 3).

APPENDIX C:

COM624P INPUT/OUTPUT FILE

bridge pier 14.1 kip lateral load

UNITS--ENGLISH

****
PILE DEFLECTION, BENDING MOMENT, SHEAR & SOIL RESISTANCE
****

INPUT INFORMATION
****

THE LOADING IS STATIC

PILE GEOMETRY AND PROPERTIES

PILE LENGTH = 377.00 IN
MODULUS OF ELASTICITY OF PILE = .403E+04 KIP/IN**2
1 SECTION(S)

X	DIAMETER	MOMENT OF INERTIA	AREA
IN	IN	IN**4	IN**2
.00	15.800	.320E+04	.196E+03
377.00			

SOILS INFORMATION

X-COORDINATE AT THE GROUND SURFACE = 17.00 IN

SLOPE ANGLE AT THE GROUND SURFACE = .00 DEG.

2 LAYER(S) OF SOIL

LAYER 1
THE LAYER IS A SOFT CLAY
X AT THE TOP OF THE LAYER = 17.00 IN
X AT THE BOTTOM OF THE LAYER = 125.00 IN
VARIATION OF SOIL MODULUS, k = .100E+03 LBS/IN**3

LAYER 2
THE LAYER IS A SAND
X AT THE TOP OF THE LAYER = 125.00 IN
X AT THE BOTTOM OF THE LAYER = 400.00 IN
VARIATION OF SOIL MODULUS, k = .600E+02 LBS/IN**3

DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH

4 POINTS

X,IN	WEIGHT,LBS/IN**3
17.00	.53E-01
125.00	.53E-01
125.00	.73E-01
400.00	.73E-01

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH

4 POINTS

X,IN	C,LBS/IN**2	PHI,DEGREES	E50
17.00	.694E+01	.000	.500E-02
125.00	.694E+01	.000	.500E-02
125.00	.000E+00	31.000	-----
400.00	.000E+00	31.000	-----

FINITE DIFFERENCE PARAMETERS
NUMBER OF PILE INCREMENTS = 63
TOLERANCE ON DETERMINATION OF DEFLECTIONS = .100E-03 IN
MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS = 100
MAXIMUM ALLOWABLE DEFLECTION = .10E+03 IN

INPUT CODES
OUTPT = 1
KCYCL = 1
KBC = 1
KPYOP = 1
INC = 1

bridge pier 14.1 kip lateral load

UNITS--ENGLISH

OUTPUT INFORMATION
****

GENERATED P-Y CURVES

THE NUMBER OF CURVE IS = 5

THE NUMBER OF POINTS ON EACH CURVE = 17

DEPTH BELOW GS IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
.00	15.800	.7E+01	.5E-01	.500E-02

Y,IN	P,LBS/IN
.000	.000
.002	32.896
.049	103.615
.099	130.546
.148	149.438
.198	164.478
.247	177.179
.296	188.280
.346	198.208
.395	207.229
.444	215.527
.494	223.231
.543	230.437
.593	237.218
1.580	328.956
2.963	328.956
3.950	328.956

DEPTH BELOW GS IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
90.00	15.800	.7E+01	.5E-01	.500E-02

Y,IN	P,LBS/IN
.000	.000
.002	71.662
.049	225.722

.099	284.392
.148	325.547
.198	358.311
.247	385.979
.296	410.164
.346	431.790
.395	451.444
.444	469.520
.494	486.303
.543	502.001
.593	516.774
1.580	716.622
2.963	716.622
3.950	716.622

DEPTH BELOW GS IN	DIAM IN	PHI	GAMMA LBS/IN**3	A	B
180.00	15.80	31.0	.6E-01	.88	.50

Y IN	P LBS/IN
.000	.000
.022	232.630
.044	465.260
.066	697.890
.088	930.520
.110	1163.149
.132	1395.779
.154	1604.953
.176	1740.690
.198	1869.916
.219	1993.622
.241	2112.562
.263	2227.332
.593	3920.104
16.393	3920.104
32.193	3920.104
47.993	3920.104

DEPTH BELOW GS IN	DIAM IN	PHI	GAMMA LBS/IN**3	A	B
270.00	15.80	31.0	.7E-01	.88	.50

Y IN	P LBS/IN
.000	.000
.022	351.130
.044	702.260
.066	1053.390

.088	1404.520
.110	1755.649
.132	2106.779
.154	2457.909
.176	2809.039
.198	3160.169
.219	3511.299
.241	3862.429
.263	4213.559
.593	7836.592
16.393	7836.592
32.193	7836.592
47.993	7836.592

DEPTH BELOW GS	DIAM	PHI	GAMMA	A	B
IN	IN		LBS/IN**3		
360.00	15.80	31.0	.7E-01	.88	.50

Y	P
IN	LBS/IN
.000	.000
.022	469.630
.044	939.260
.066	1408.890
.088	1878.520
.110	2348.149
.132	2817.779
.154	3287.409
.176	3757.039
.198	4226.669
.219	4696.299
.241	5165.929
.263	5635.559
.593	10803.802
16.393	10803.802
32.193	10803.802
47.993	10803.802

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PILE LOADING CONDITION

LATERAL LOAD AT PILE HEAD = .141E+02 KIP
APPLIED MOMENT AT PILE HEAD = .000E+00 IN-KIP
AXIAL LOAD AT PILE HEAD = .000E+00 KIP

X	DEFLECTION	MOMENT	TOTAL STRESS	SHEAR	SOIL RESIST	FLEXURAL RIGIDITY
IN	IN	IN-KIP	LBS/IN**2	KIP	LBS/IN	KIP-IN**2
****	*******	*******	**********	*******	*********	**********
.00	.451E+00	.000E+00	.000E+00	.141E+02	.000E+00	.129E+08
5.98	.417E+00	.844E+02	.208E+03	.141E+02	.000E+00	.129E+08
11.97	.384E+00	.169E+03	.416E+03	.141E+02	.000E+00	.129E+08
17.95	.351E+00	.253E+03	.625E+03	.135E+02	.202E+03	.129E+08
23.94	.319E+00	.330E+03	.815E+03	.123E+02	.210E+03	.129E+08
29.92	.287E+00	.400E+03	.987E+03	.110E+02	.218E+03	.129E+08
35.90	.257E+00	.462E+03	.114E+04	.966E+01	.224E+03	.129E+08
41.89	.228E+00	.515E+03	.127E+04	.830E+01	.229E+03	.129E+08
47.87	.201E+00	.561E+03	.138E+04	.692E+01	.232E+03	.129E+08
53.86	.175E+00	.598E+03	.148E+04	.553E+01	.234E+03	.129E+08
59.84	.151E+00	.627E+03	.155E+04	.412E+01	.235E+03	.129E+08
65.83	.128E+00	.648E+03	.160E+04	.272E+01	.234E+03	.129E+08
71.81	.108E+00	.660E+03	.163E+04	.133E+01	.231E+03	.129E+08
77.79	.889E-01	.664E+03	.164E+04	-.381E-01	.226E+03	.129E+08
83.78	.719E-01	.659E+03	.163E+04	-.137E+01	.220E+03	.129E+08
89.76	.568E-01	.647E+03	.160E+04	-.267E+01	.212E+03	.129E+08
95.75	.434E-01	.627E+03	.155E+04	-.391E+01	.202E+03	.129E+08
101.73	.318E-01	.600E+03	.148E+04	-.507E+01	.189E+03	.129E+08
107.71	.218E-01	.567E+03	.140E+04	-.616E+01	.173E+03	.129E+08
113.70	.134E-01	.527E+03	.130E+04	-.713E+01	.152E+03	.129E+08
119.68	.648E-02	.481E+03	.119E+04	-.795E+01	.123E+03	.129E+08
125.67	.881E-03	.432E+03	.106E+04	-.834E+01	.547E+01	.129E+08
131.65	-.352E-02	.382E+03	.942E+03	-.829E+01	-.236E+02	.129E+08
137.63	-.685E-02	.332E+03	.820E+03	-.807E+01	-.483E+02	.129E+08
143.62	-.927E-02	.285E+03	.703E+03	-.772E+01	-.686E+02	.129E+08
149.60	-.109E-01	.240E+03	.592E+03	-.727E+01	-.845E+02	.129E+08
155.59	-.119E-01	.198E+03	.489E+03	-.672E+01	-.962E+02	.129E+08
161.57	-.123E-01	.159E+03	.393E+03	-.613E+01	-.104E+03	.129E+08
167.56	-.122E-01	.125E+03	.308E+03	-.549E+01	-.108E+03	.129E+08
173.54	-.118E-01	.937E+02	.231E+03	-.484E+01	-.109E+03	.129E+08
179.52	-.112E-01	.667E+02	.165E+03	-.420E+01	-.107E+03	.129E+08
185.51	-.104E-01	.435E+02	.107E+03	-.357E+01	-.103E+03	.129E+08
191.49	-.944E-02	.240E+02	.591E+02	-.297E+01	-.969E+02	.129E+08
197.48	-.843E-02	.791E+01	.195E+02	-.241E+01	-.896E+02	.129E+08
203.46	-.739E-02	-.492E+01	.121E+02	-.190E+01	-.812E+02	.129E+08
209.44	-.637E-02	-.148E+02	.366E+02	-.144E+01	-.723E+02	.129E+08
215.43	-.540E-02	-.222E+02	.547E+02	-.104E+01	-.632E+02	.129E+08
221.41	-.448E-02	-.272E+02	.672E+02	-.685E+00	-.540E+02	.129E+08
227.40	-.364E-02	-.304E+02	.750E+02	-.388E+00	-.452E+02	.129E+08
233.38	-.288E-02	-.319E+02	.787E+02	-.143E+00	-.368E+02	.129E+08
239.37	-.221E-02	-.321E+02	.792E+02	.542E-01	-.291E+02	.129E+08
245.35	-.163E-02	-.312E+02	.771E+02	.207E+00	-.220E+02	.129E+08
251.33	-.114E-02	-.296E+02	.731E+02	.320E+00	-.158E+02	.129E+08
257.32	-.729E-03	-.274E+02	.676E+02	.399E+00	-.104E+02	.129E+08

263.30	-.394E-03	-.248E+02	.613E+02	.447E+00	-.574E+01	.129E+08
269.29	-.129E-03	-.221E+02	.544E+02	.470E+00	-.191E+01	.129E+08
275.27	.759E-04	-.192E+02	.474E+02	.472E+00	.117E+01	.129E+08
281.25	.227E-03	-.164E+02	.405E+02	.458E+00	.356E+01	.129E+08
287.24	.333E-03	-.137E+02	.339E+02	.431E+00	.533E+01	.129E+08
293.22	.400E-03	-.112E+02	.277E+02	.396E+00	.656E+01	.129E+08
299.21	.437E-03	-.899E+01	.222E+02	.354E+00	.731E+01	.129E+08
305.19	.448E-03	-.700E+01	.173E+02	.310E+00	.766E+01	.129E+08
311.17	.440E-03	-.529E+01	.130E+02	.264E+00	.768E+01	.129E+08
317.16	.417E-03	-.385E+01	.949E+01	.218E+00	.743E+01	.129E+08
323.14	.384E-03	-.267E+01	.659E+01	.175E+00	.697E+01	.129E+08
329.13	.343E-03	-.175E+01	.431E+01	.136E+00	.635E+01	.129E+08
335.11	.297E-03	-.105E+01	.259E+01	.997E-01	.561E+01	.129E+08
341.10	.249E-03	-.554E+00	.137E+01	.686E-01	.478E+01	.129E+08
347.08	.198E-03	-.229E+00	.565E+00	.426E-01	.389E+01	.129E+08
353.06	.148E-03	-.435E-01	.107E+00	.222E-01	.295E+01	.129E+08
359.05	.968E-04	.364E-01	.898E-01	.746E-02	.197E+01	.129E+08
365.03	.460E-04	.458E-01	.113E+00	-.127E-02	.950E+00	.129E+08
371.02	-.463E-05	.000E+00	.000E+00	.000E+00	-.994E-01	.129E+08
377.00	-.552E-04	.000E+00	.000E+00	.000E+00	-.118E+01	.129E+08

COMPUTED LATERAL FORCE AT PILE HEAD = .14100E+02 KIP
COMPUTED MOMENT AT PILE HEAD = .00000E+00 IN-KIP
COMPUTED SLOPE AT PILE HEAD = -.56103E-02

THE OVERALL MOMENT IMBALANCE = .279E-11 IN-KIP
THE OVERALL LATERAL FORCE IMBALANCE = .760E-08 LBS

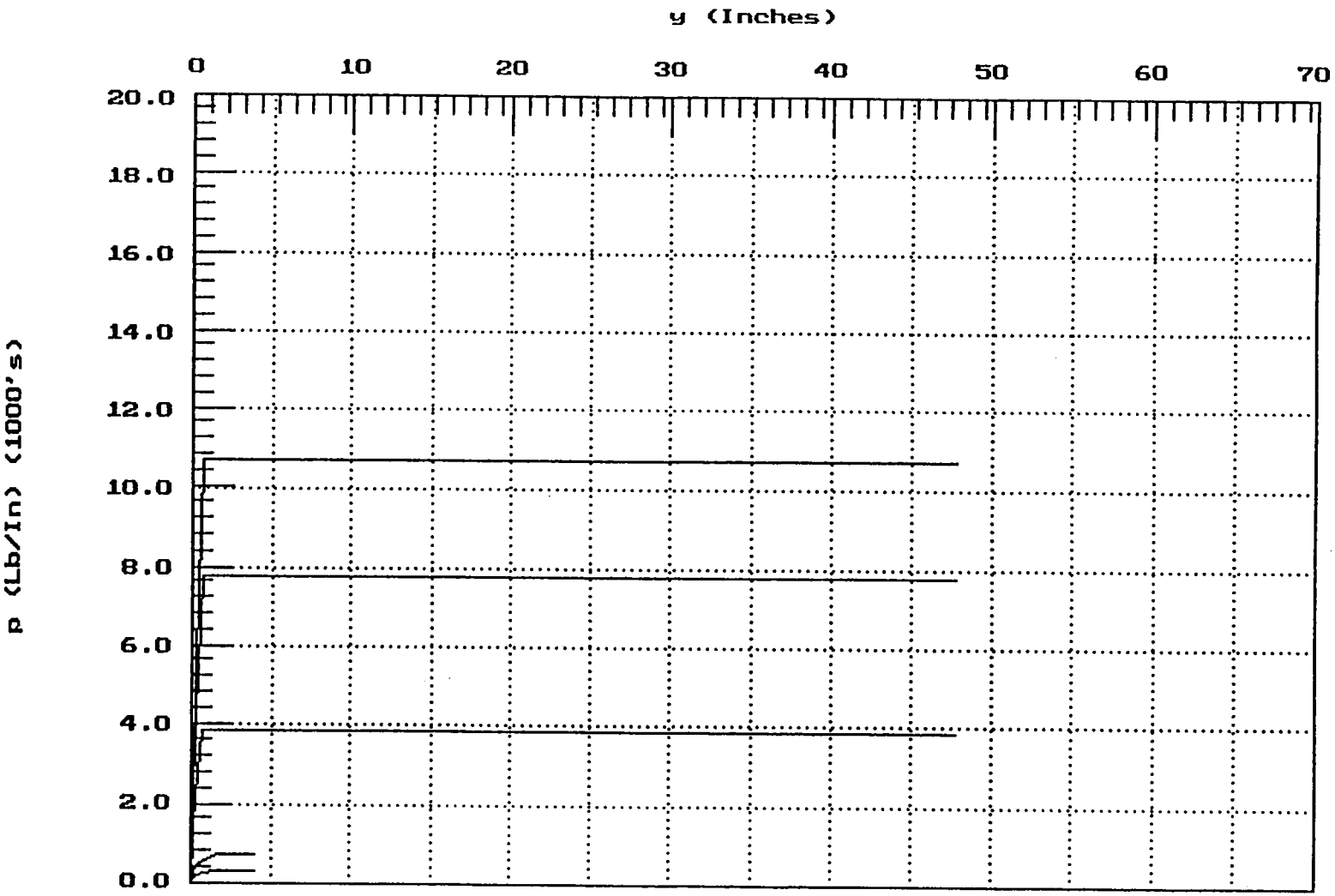
OUTPUT SUMMARY

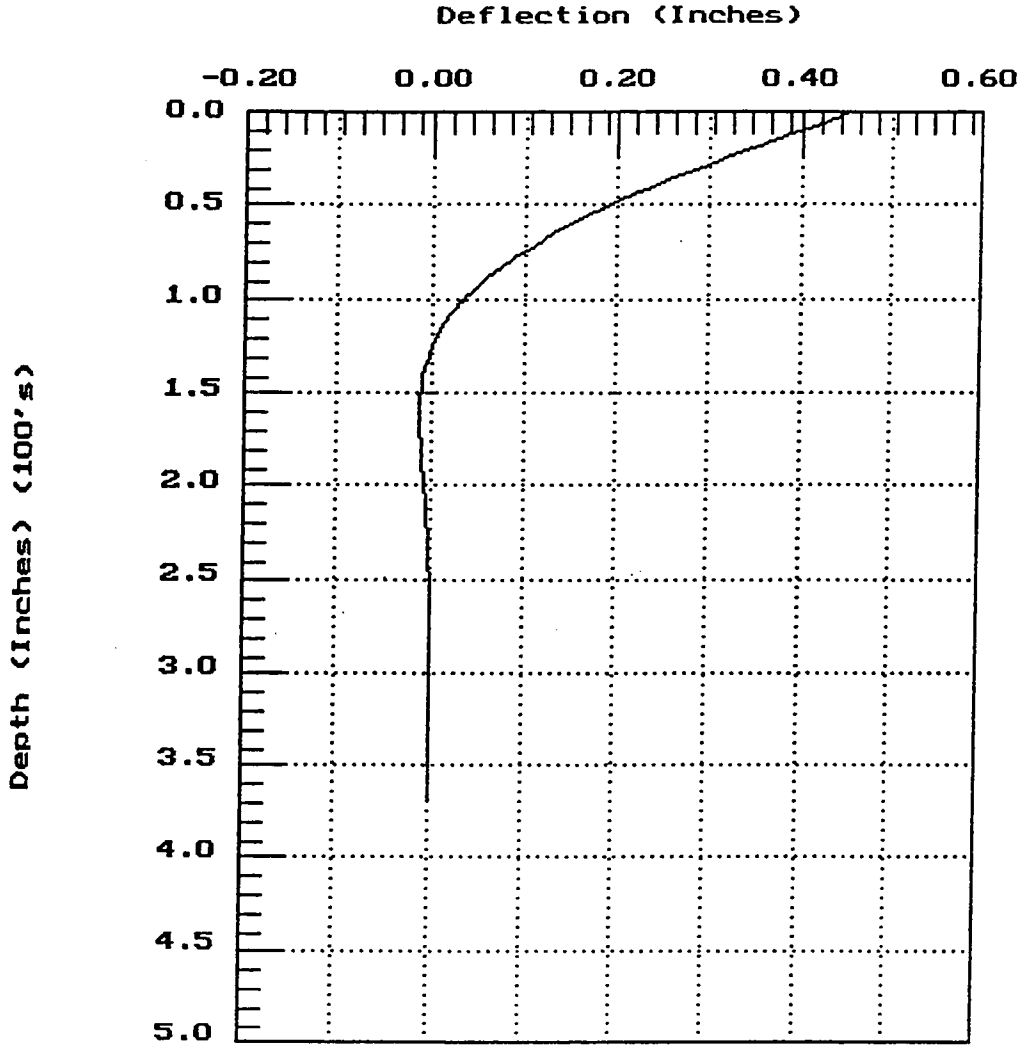
PILE HEAD DEFLECTION = .451E+00 IN
MAXIMUM BENDING MOMENT = .664E+03 IN-KIP
MAXIMUM TOTAL STRESS = .164E+04 LBS/IN**2

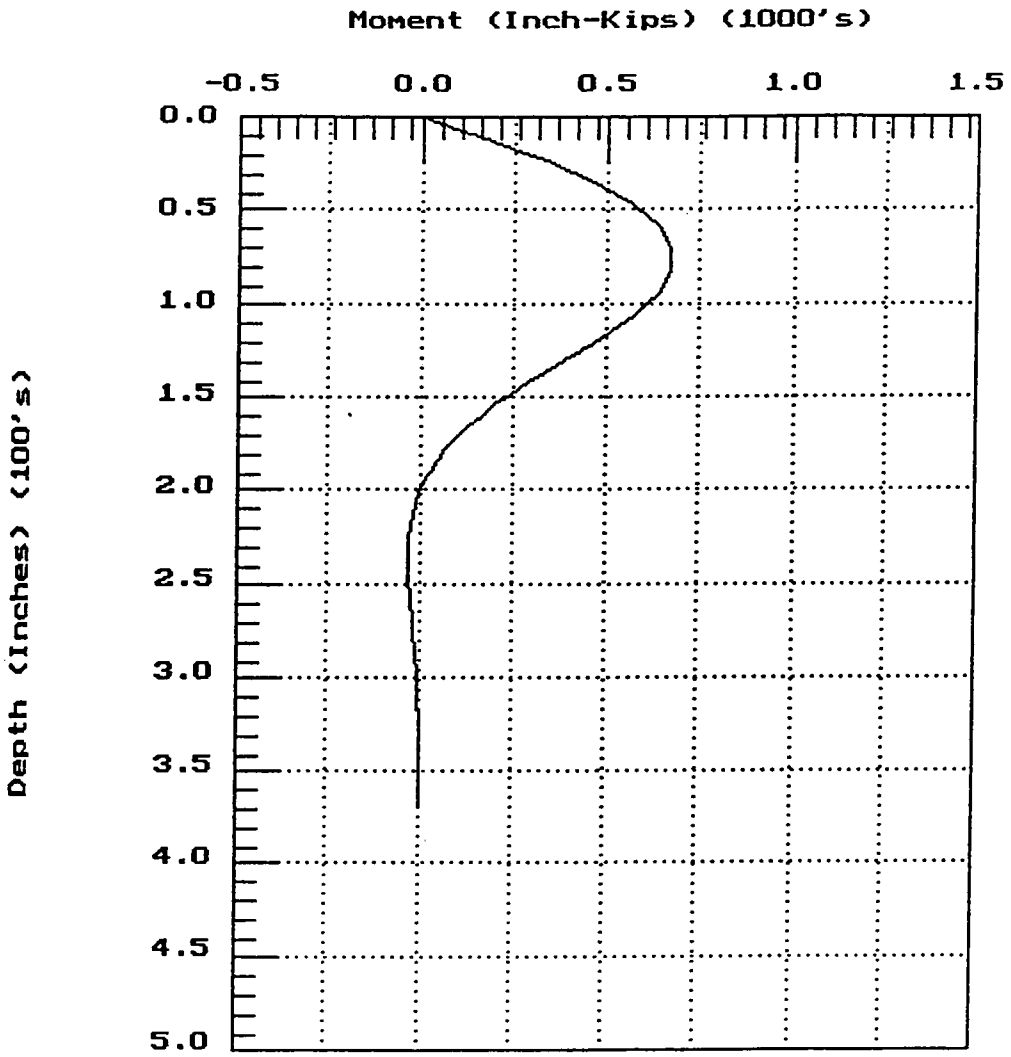
NO. OF ITERATIONS = 14
MAXIMUM DEFLECTION ERROR = .931E-04 IN

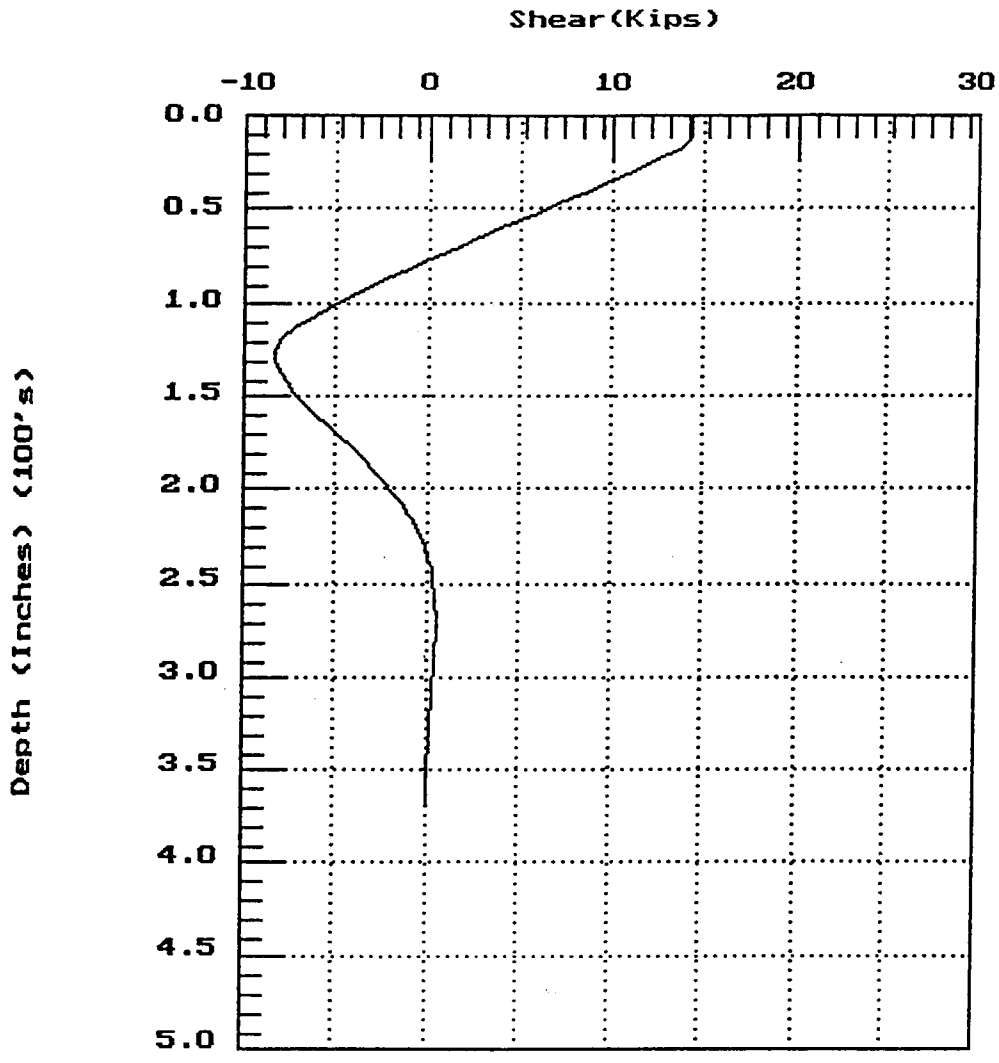
SUMMARY TABLE
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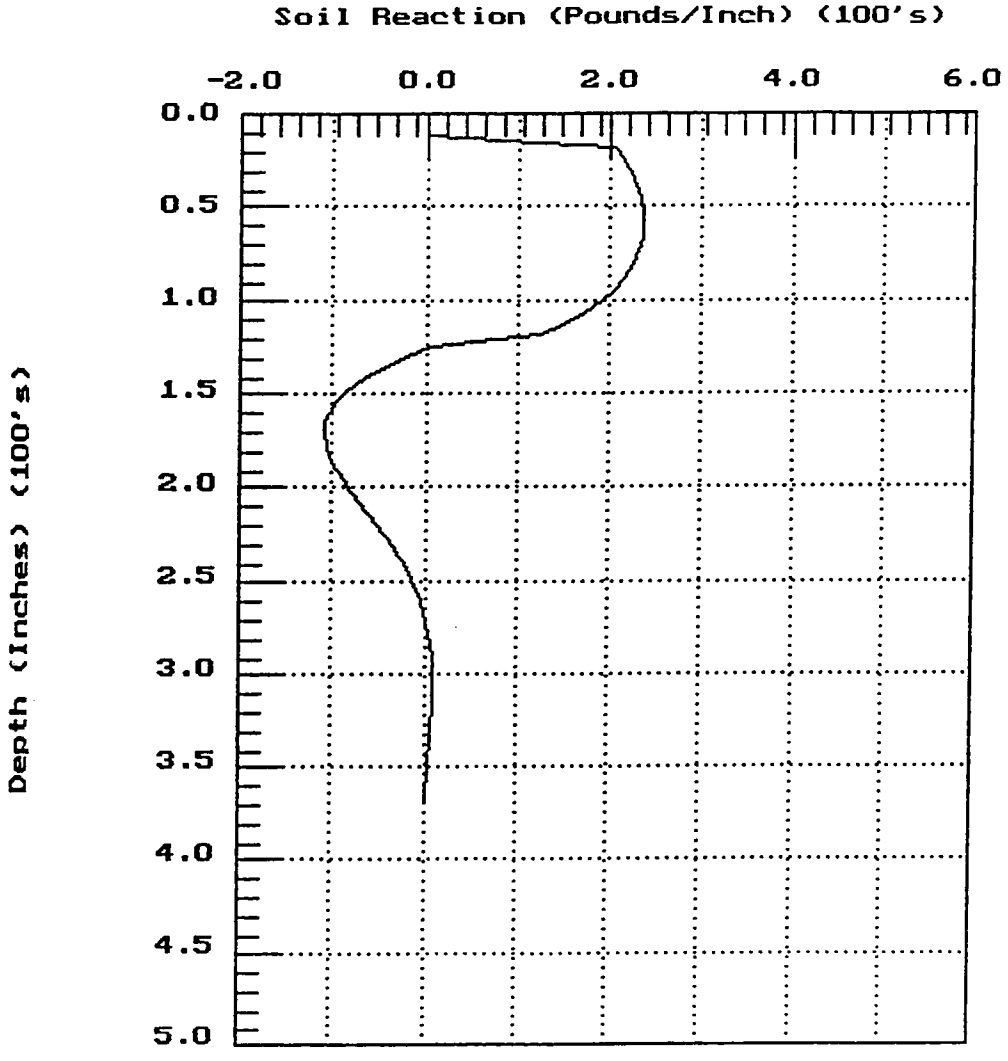
LATERAL LOAD (KIP)	BOUNDARY CONDITION BC2	AXIAL LOAD (KIP)	YT (IN)	ST (IN/IN)	MAX. MOMENT (IN-KIP)	MAX. STRESS (LBS/IN**2)
.141E+02	.000E+00	.000E+00	.451E+00	-.561E-02	.664E+03	.164E+04


y (Inches)
0
10
20
30
40
50
60
70
20.0
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
p (Lb/In) (1000's)


Deflection (Inches)
-0.20
0.00
0.20
0.40
0.60
Depth (Inches) (100's)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0


Moment (Inch-Kips) (1000's)
-0.5
0.0
0.5
1.0
1.5
Depth (Inches) (100's)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0


Shear(Kips)
-10
0
10
20
30
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
Depth (Inches) (100's)


Soil Reaction (Pounds/Inch) (100's)
-2.0
0.0
2.0
4.0
6.0
Depth (Inches) (100's)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0

VITA

Jim Bernardi was born in Johnson City, Tennessee on November 22, 1954. After graduating from Science Hill High School in June, 1972, he enrolled at East Tennessee State University. In June, 1977 he graduated with a Bachelor of Science degree in Environmental Health. After several years as a builder in the field of residential construction, he decided to return to school and pursue a career in engineering. In August, 1992 he enrolled at The University of Tennessee, Knoxville, in the department of Civil Engineering. He received a Bachelor of Science degree in Civil Engineering in May 1995. He started graduate school the following semester and received his Master of Science degree in Civil Engineering in August, 1996