Report No. K-TRAN: KU-09-6
FINAL REPORT

LATERAL CAPACITY OF ROCK SOCKETS IN LIMESTONE UNDER CYCLIC AND REPEATED LOADING

Robert L. Parsons, Ph.D., P.E.
Isaac Willems
Matthew C. Pierson
Jie Han, Ph.D., P.E.

The University of Kansas
Lawrence, Kansas

August 2010

A COOPERATIVE TRANSPORTATION RESEARCH PROGRAM BETWEEN:

KANSAS DEPARTMENT OF TRANSPORTATION
KANSAS STATE UNIVERSITY
UNIVERSITY OF KANSAS

KANSAS DEPARTMENT OF TRANSPORTATION

1 Report No. K-TRAN: KU-09-6	**2 Government Accession No.**	**3 Recipient Catalog No.**
4 Title and Subtitle Lateral Capacity of Rock Sockets in Limestone under Cyclic and Repeated Loading		**5 Report Date** August 2010 **6 Performing Organization Code**
7 Author(s) Robert L. Parsons, Ph.D., P.E., Isaac Willems, Matthew C. Pierson, Jie Han, Ph.D., P.E.		**8 Performing Organization Report No.**
9 Performing Organization Name and Address The University of Kansas Department of Civil, Environmental and Architectural Engineering 2150 Learned Hall, 1530 W 15th Street Lawrence, Kansas 66045-7609		**10 Work Unit No. (TRAIS)** **11 Contract or Grant No.** C1756
12 Sponsoring Agency Name and Address Kansas Department of Transportation Bureau of Materials and Research 700 SW Harrison Street Topeka, Kansas 66603-3745		**13 Type of Report and Period Covered** Final Report July 2008 to July 2010 **14 Sponsoring Agency Code** RE-0502-01
15 Supplementary Notes For more information write to address in block 9.		

16 Abstract

This report contains the results from full scale lateral load testing of two short rock socketed shafts in limestone, and the development of recommendations for p-y analysis using those results. Two short shafts 42 inches in diameter were constructed to depths of approximately seven feet in limestone in Wyandotte County, Kansas. The shafts were loaded laterally during three separate test events in 2009. The shafts were tested under cyclic loading (load reversal) at loads up to 400 kips; repeated loading up to 800 kips, and to failure near 1000 kips.

Test data showed that shaft behavior was essentially elastic during cyclic loading for loads of 400 kips and lower (40% of ultimate capacity). The shafts experienced permanent, accumulating deformations during repeated loads of 600 and 800 kips.

Modeling of the results showed the lateral load behavior could be effectively modeled in LPILE using the “weak rock” model included with LPILE software.

17 Key Words Lateral Load, socketed shafts, Kansas Limestone, p-y curves, LPILE		**18 Distribution Statement** No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161	
19 Security Classification (of this report) Unclassified	**20 Security Classification (of this page)** Unclassified	**21 No. of pages** 49	**22 Price**

Form DOT F 1700.7 (8-72)

LATERAL CAPACITY OF ROCK SOCKETS IN LIMESTONE UNDER CYCLIC AND REPEATED LOADING

Final Report

Prepared by

Robert L. Parsons, Ph.D., P.E.

Isaac Willems

Matthew C. Pierson

Jie Han, Ph.D., P.E.

The University of Kansas
Lawrence, Kansas

A Report on Research Sponsored By

THE KANSAS DEPARTMENT OF TRANSPORTATION
TOPEKA, KANSAS

August 2010

PREFACE

The Kansas Department of Transportation's (KDOT) Kansas Transportation Research and New-Developments (K-TRAN) Research Program funded this research project. It is an ongoing, cooperative and comprehensive research program addressing transportation needs of the state of Kansas utilizing academic and research resources from KDOT, Kansas State University and the University of Kansas. Transportation professionals in KDOT and the universities jointly develop the projects included in the research program.

NOTICE

The authors and the state of Kansas do not endorse products or manufacturers. Trade and manufacturers' names appear herein solely because they are considered essential to the object of this report.

This information is available in alternative accessible formats. To obtain an alternative format, contact the Office of Transportation Information, Kansas Department of Transportation, 700 SW Harrison, Topeka, Kansas 66603-3745 or phone (785) 296-3585 (Voice) (TDD).

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the views or the policies of the state of Kansas. This report does not constitute a standard, specification or regulation.

ABSTRACT

This report contains the results from full scale lateral load testing of two short rock socketed shafts in limestone, and the development of recommendations for p-y analysis using those results. Two short shafts 42 inches in diameter were constructed to depths of approximately six to seven feet in limestone in Wyandotte County, Kansas. The shafts were loaded laterally during three separate test events in 2009. The shafts were tested under cyclic loading (load reversal) at loads up to 400 kips; repeated loading up to 800 kips, and to failure near 1000 kips.

Test data showed that shaft behavior was essentially elastic during cyclic loading for loads of 400 kips and lower (40% of ultimate capacity). The shafts experienced permanent, accumulating deformations during repeated loads of 600 and 800 kips.

Modeling of the results showed the lateral load behavior could be effectively modeled in LPILE using the “weak rock” model included with LPILE software.

ACKNOWLEDGMENTS

The authors wish to thank the people of the Kansas Department of Transportation (KDOT) for their financial and logistical support that made this research possible. We particularly want to thank the people of the KDOT Geotechnical and Maintenance Units for their help in bringing this project to fruition. We also wish to thank Mr. Jim Weaver with the University of Kansas (KU) for his help in designing and fabricating the equipment and Mr. Justin Clay of KU for his help in fabrication of the equipment for Test 1 and with some of the theoretical background presented in Chapter 2. We also wish to thank Mr. Paul Axtell and Mr. Dan Brown, both of Dan Brown and Associates, who helped with the testing and interpretation of data. The help of all who participated is greatly appreciated.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1 - INTRODUCTION

This report contains the results from a full scale lateral load test of two short rock socketed shafts in limestone, and the development of recommendations for p-y analysis using those results. The shafts were tested under cyclic loading (load reversal) at loads up to 400 kips; repeated loading up to 800 kips, and to failure near 1000 kips. A detailed description of the testing, analysis, and p-y curve recommendations is provided.

Drilled shafts are a type of deep foundation that is capable of supporting very large vertical and lateral loads. Drilled shafts are constructed by drilling a hole from the ground surface to the target depth or formation and filling the hole with reinforcing steel and concrete to create a reinforced concrete column from the surface to the desired depth.

Lateral load capacity is of particular interest with regard to bridge and abutment foundations because of the significant loading conditions they experience. Lateral load capacity may be estimated during the design process by several methods, with one of the most common being a p-y analysis. This type of analysis requires the use of p-y curves, or load-deflection curves. These curves vary among soil types and rock formations, although general curves have been developed and are available for use in widely available software packages such as COM624 (public domain) and LPILE (proprietary software, Ensoft).

The purpose of this project was to test the lateral capacity and develop p-y curves for limestone in Kansas. Two short shafts 42 inches in diameter were constructed to depths of six to seven feet in limestone in Wyandotte County, Kansas. The shafts were loaded laterally during three separate test events in 2009. During the first event, the shafts were loaded in a cyclic manner (load reversal) at multiple increments up to 400 kips. The shafts were then loaded in one direction to 550 kips. The equipment was then reconfigured and the shafts loaded to 800 kips with repeated loading-unloading cycles at 600 and 800 kips. The loading frame was then reinforced and the shafts were loaded to failure, which occurred near 1000 kips.

Analysis of the data showed that commonly used p-y curves included within the LPILE software could be used to develop an accurate model of the static behavior of the shafts. Cyclic loading of the shafts had little effect on shaft capacity at lower loads; however permanent deformation began to accumulate at loading levels between 40 and 60 percent of ultimate capacity.

CHAPTER 2 - THEORETICAL BACKGROUND

This chapter contains an abridged discussion of the p-y curve method. For a more detailed discussion of the p-y curve method the reader is referred to the technical manual, *LPILE Plus 5.0 for Windows*, A program for the analysis of piles and shafts under lateral loads (Reese et al, 2004).

For the p-y method the pile-soil interaction is modeled as a series of nonlinear springs as shown in Figure 2.1, where “p” represents lateral load on a spring and “y” represents displacement of the spring. The non-linear relationship is captured by the modulus Es, which decreases according to some function as displacement increases. An example of a p-y curve based on a hyperbolic function is shown in Figure 2.2.

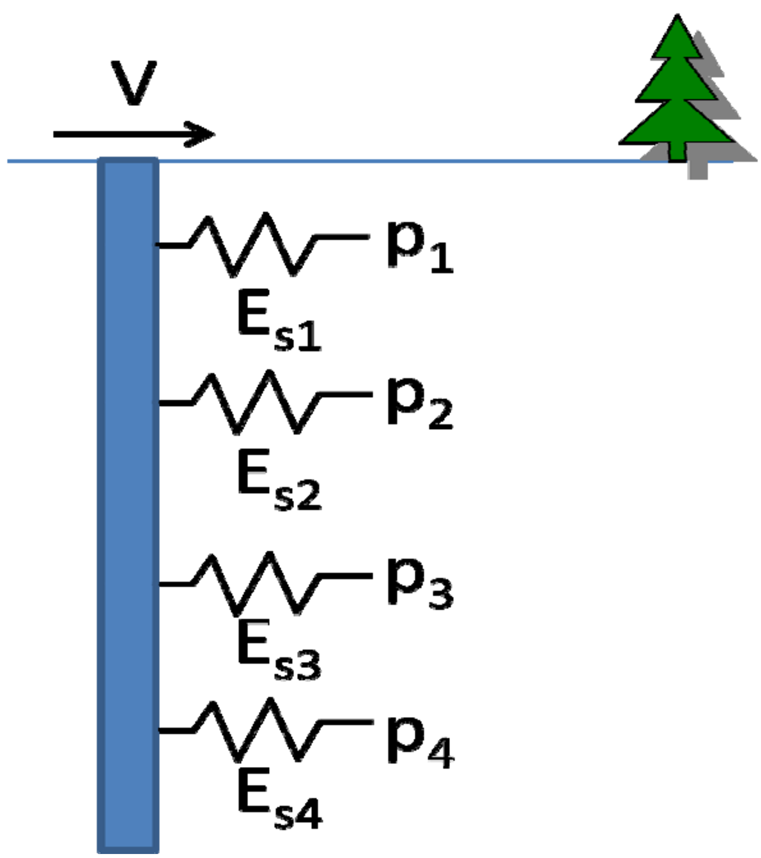

Figure 2.1: P-Y model of pile-soil interaction

The p-y method was extended to the analysis of single rock-socketed drilled shafts under lateral loading by Reese (1997). The method developed by Reese includes consideration of the secondary structures of rock masses using a rock strength reduction factor. This reduction factor can be determined from the Rock Quality Designation (RQD). Reese's (1997) method for estimating ultimate reaction per unit length, however, ignored the contribution of shear resistance between shaft and rock. Also RQD cannot be used to fully describe all secondary rock structures, such as spacing and condition of discontinuities.

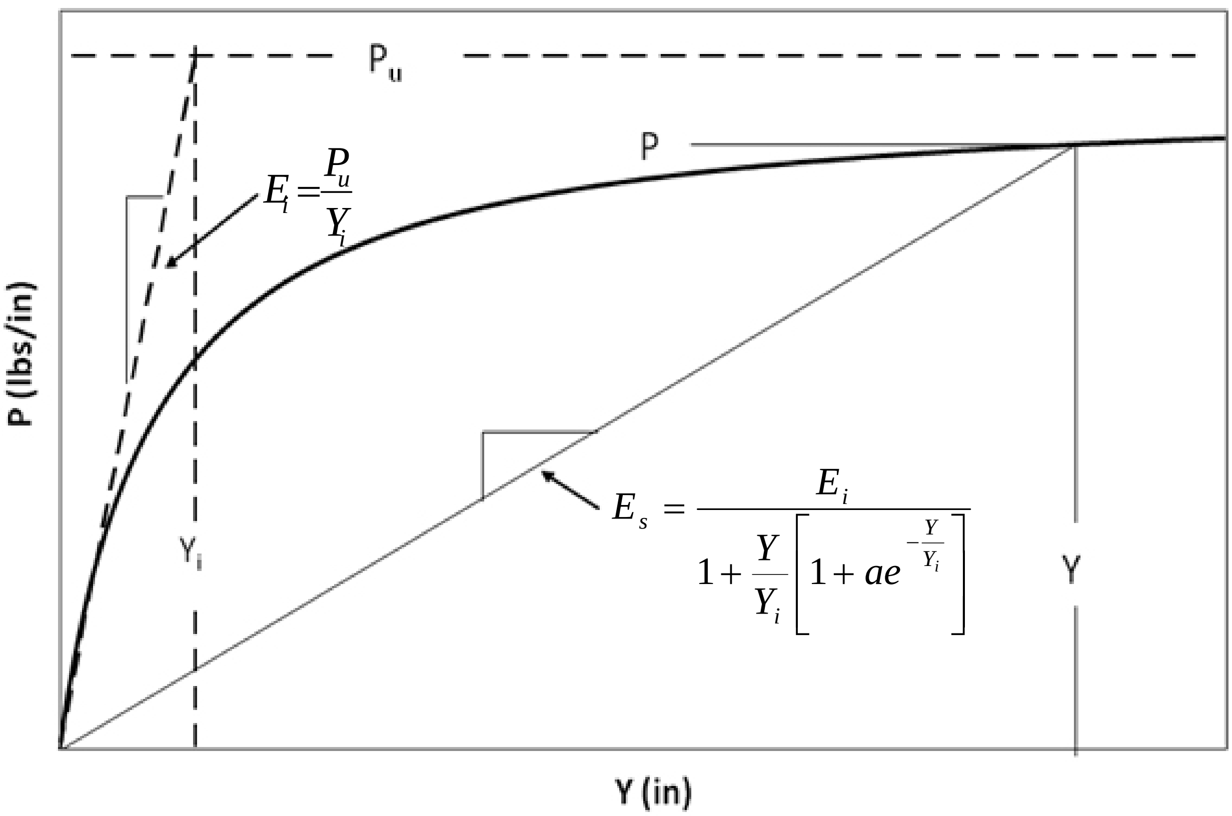

Figure 2.2: Example of hyperbolic p-y curve

In order to characterize the rock response under lateral loading, an interim p-y criterion for weak rock was suggested. Due to the lack of adequate test data, the term "interim" was applied to this criterion. With this interim criterion, Com624P or LPILE can be run to obtain the lateral response of rock-socketed drilled shafts. This model has been incorporated into LPILE v 5.0 Plus (Reese et al 2004).

For this approach, the ultimate reaction P_u (units of force per length) of rock is given by:

$$\mathrm{Pu} = \propto_r q_{ur} b \left(1 + 1.4\frac{x_r}{b}\right) \text{ for } 0 \leq z_r \leq 3b$$

$$\mathrm{Pu} = 5.2 \propto_r q_{ur} b \text{ for } z_r \geq 3b$$

Where:

q_{ur} = uniaxial compressive strength of intact rock;

α_r = strength reduction factor, used to account for fracturing of rock mass, it is assumed to be 1/3 for RQD of 100% and it increases linearly to 1 at a RQD of zero;

b = diameter of the drilled shaft, and;

x_r = depth below rock surface.

The slope of initial portion of p-y curves is given by:

$$K_{ir} \approx k_{ir} * E_{ir}$$

Where:

K_{ir} = initial tangent to p-y curve;

E_m = initial modulus of the rock

k_{ir} = dimensionless constant

The expressions for k_{ir}, derived by correlation with experimental data, are as follows:

$$k_{ir} = \left(100 + \frac{400x_r}{3b}\right) \text{ for } 0 \leq z_r \leq 3b$$

$$k_{ir} = 500 \text{ for } z_r \geq 3b$$

The p-y curves developed from these relationships follow the shape shown in Figure 2.3. This figure shows a p-y curve with three segments; from the origin to y_A, from y_A to y_m, and from y_{rm} to failure.

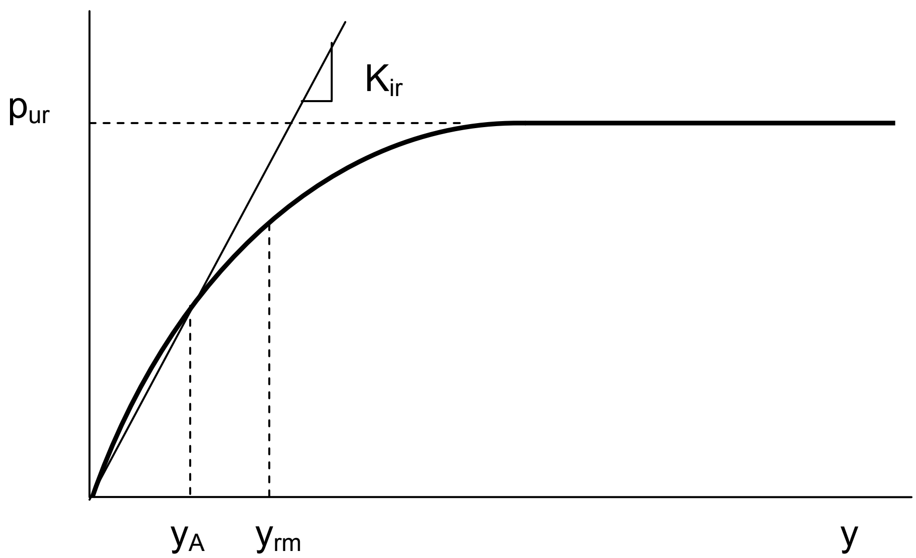

Figure 2.3: Sketch of p-y curve for weak rock (adapted from Reese, 1997)

The equations relating p and y for the curve in Figure 2.3 area as follows:

$$p = K_{ir}y \text{ for } y \leq y_A$$

$$p = \frac{p_{ur}}{2}\left(\frac{y}{y_{rm}}\right)^{0.25} \text{ for } y > y_A,\ P < P_{ur}$$

$$p = p_{ur} \text{ for } y > 16y_{rm}$$

and $y_{rm}=k_{rm}b$ where

k_{rm} = a constant between 0.0005 and 0.00005 that controls the overall stiffness of the p-y curves, and; $y_A = \left(\frac{p_{ur}}{2y_{rm}^{0.25}k_{ir}}\right)^{1.333}$

CHAPTER 3 - DESCRIPTION OF TESTING

This project entailed construction and lateral load testing of two rock-socketed drilled shafts. The shafts were constructed in the northeast quadrant of the intersection of I-70 and I-435 in Wyandotte County, Kansas (Figures 3.1 and 3.2). The shafts were constructed in the fall of 2007 and tested in the summer and fall of 2009. The shafts were set in the Plattsburg Limestone and spaced 144 inches apart center to center.

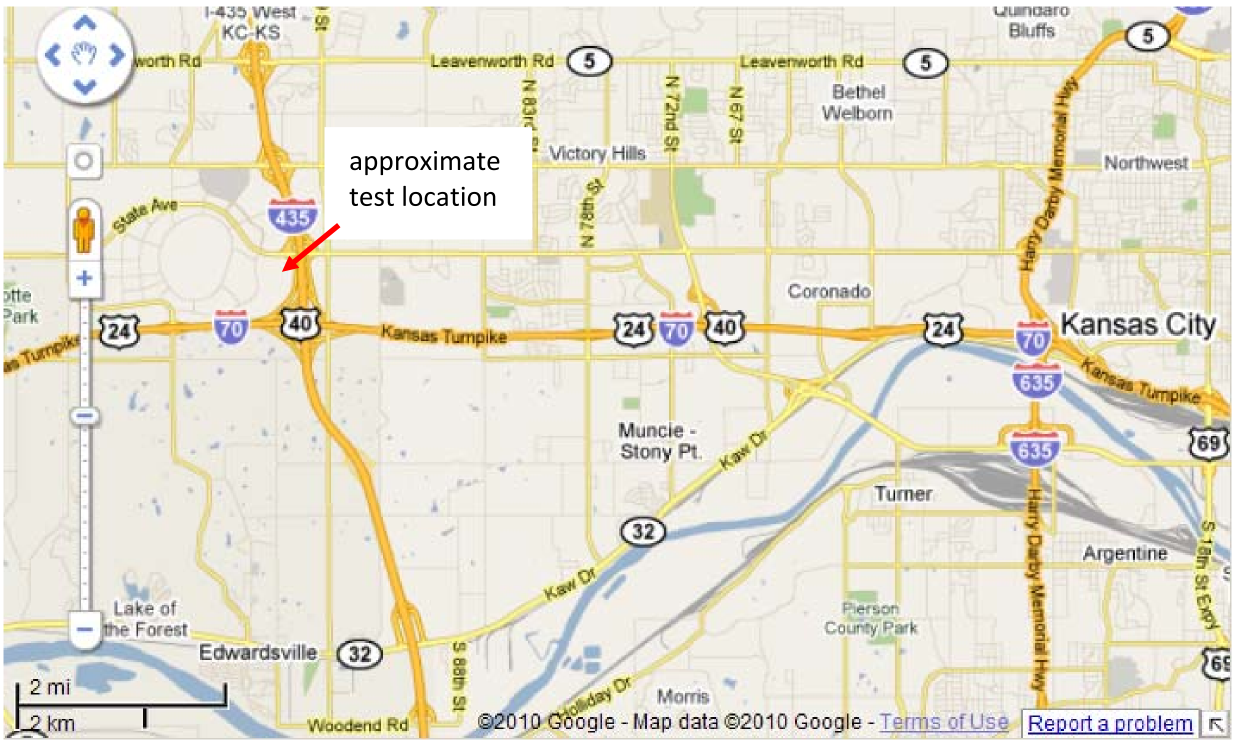

Figure 3.1: Regional map (Google Maps, 2010)

3.1 Site Investigation

Borings were taken near the shaft locations on July 11, 2007. Boring logs are shown in Appendix A, along with unconfined testing information. The site geology consisted of minimal to no soil overburden, 1.5-2.5 feet of weathered to hard sandstone over hard limestone. The overburden and sandstone were removed so the sockets were entirely in limestone.

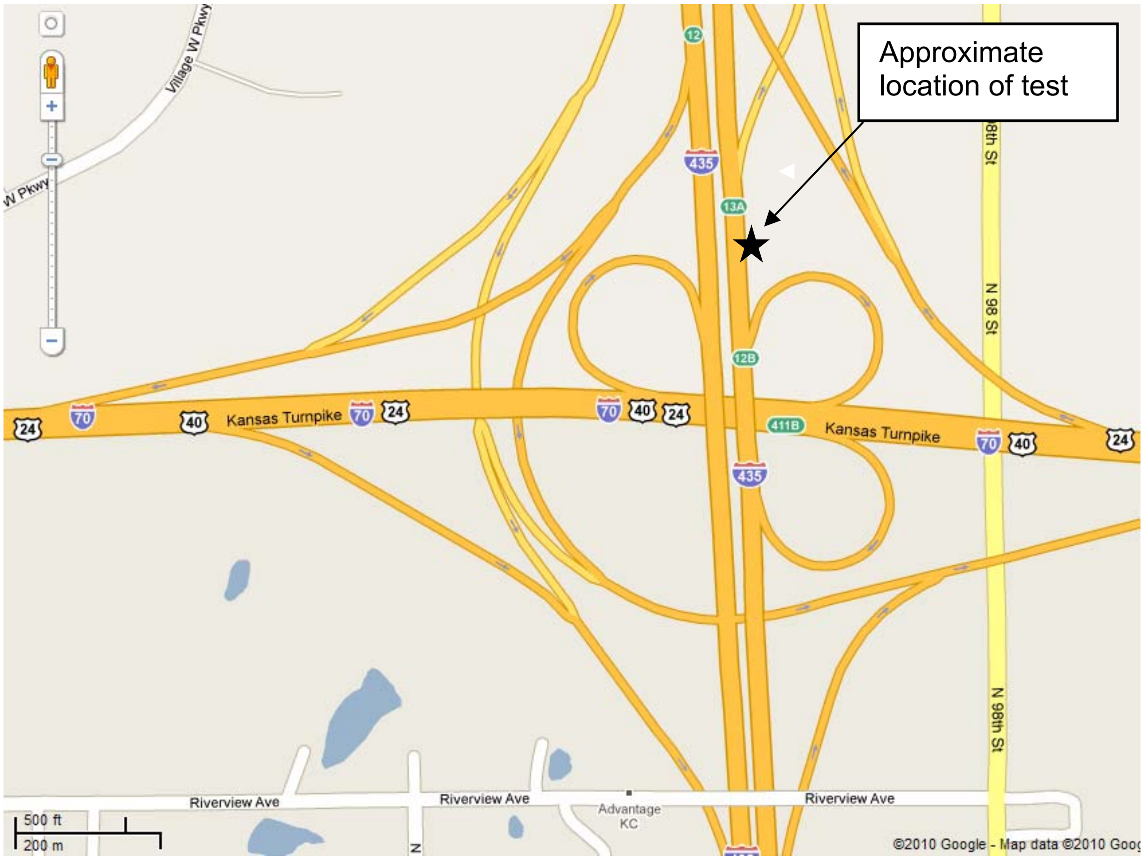

Figure 3.2: Site map (Google Maps, 2010)

3.2 Shaft Details

The shafts were 42 inches in diameter and cast in sockets approximately six feet deep for the north shaft and seven feet deep for the south shaft. Shaft reinforcement consisted of twelve #11 longitudinal bars and hoops made of #5 bars on with one foot spacing within the socket and a spacing of approximately 6 inches above ground at the point of load application (Figure 3.3). The load was applied approximately one foot above ground level. Concrete was KDOT standard drilled shaft mix.

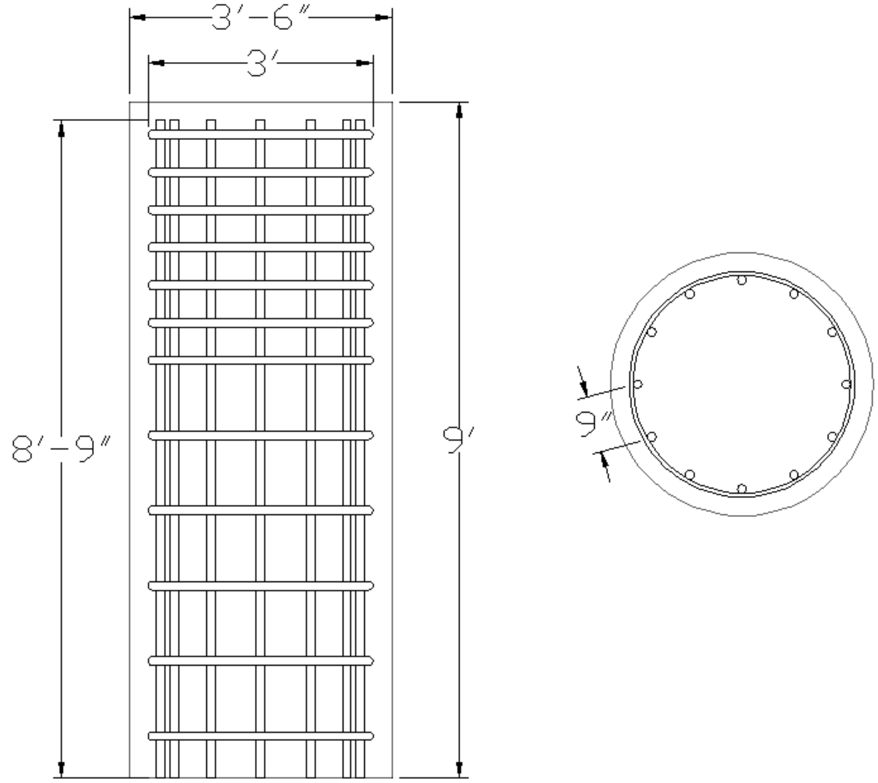

Figure 3.3: Reinforcing cage layout

3.3 Testing

Lateral load testing was conducted a part of three separate tests. The first test was conducted July 29, 2009 and consisted of cyclic (load reversal) testing up to 400 kips for a series of primary load increments, where 400 kips was the maximum load that could be achieved in both directions with the equipment configuration used. The equipment was configured such that essentially two separate load frames could load the shafts in opposite directions simultaneously. One set of equipment with three 200 kip hydraulic cylinders was used to jack the shafts apart, and a second set with two 200 kip cylinders was used to pull the shafts together (Figure 3.4). Cycles of loading were applied to the shafts by alternating loading between these sets of equipment. Five or ten cycles were applied at each primary load increment. Additional measurements were taken at intermediate increments.

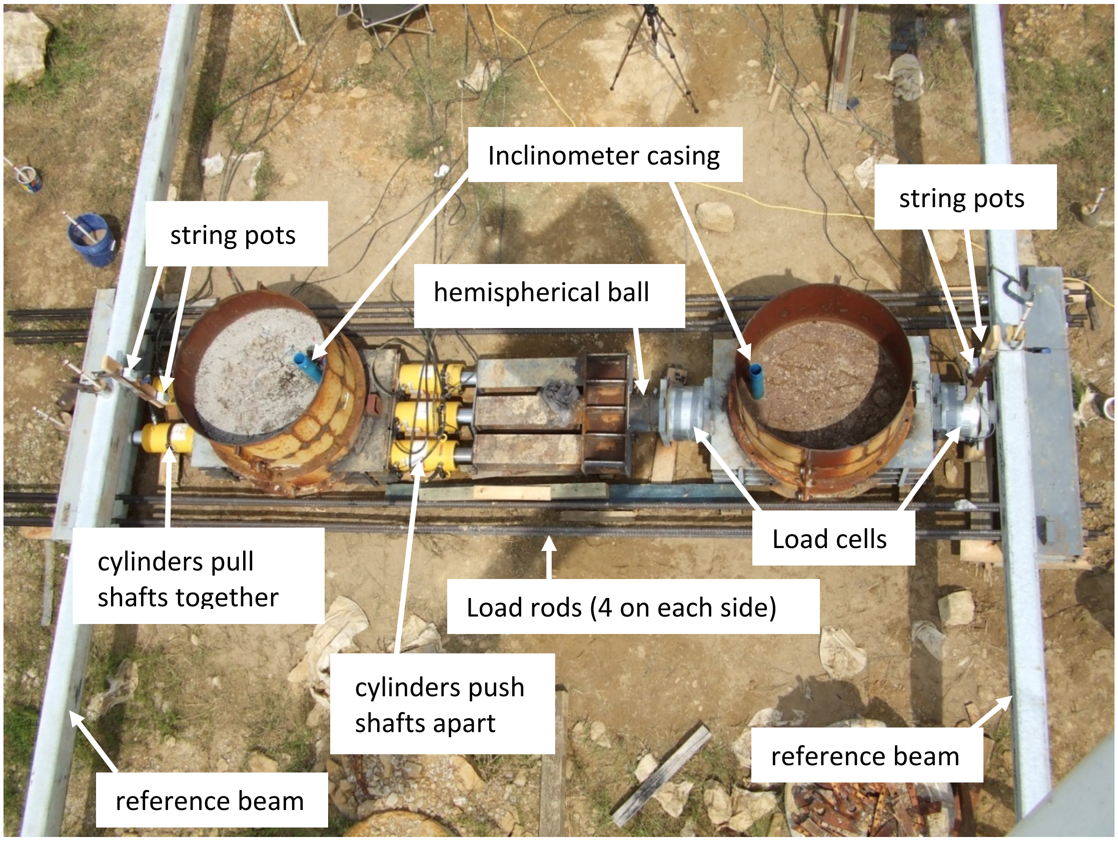

Figure 3.4: Test 1 setup

Load was measured using two separate systems, load cells and hydraulic pressure. The hydraulic pressure was monitored by gauge and by pressure transducer. The load cells were limited to a capacity of 400 kips and served as a backup to the pressure transducer and gauge. Deformation was measured at two locations on each shaft with UniMeasure P510 string pots fixed to reference beams and inclinometer measurements in each shaft. Pressure transducer, string pot, and load cell data was recorded automatically on a laptop computer. Photogrammetry was used as a backup system. Pressure transducer and string pot information was recorded by a laptop and data acquisition system. Inclinometer data was recorded by KDOT personnel with a data logger prior to each test and after each set of load cycles.

The second test was conducted on November 10, 2009. For this test the equipment was reconfigured so that all five cylinders could be used together to load the shafts to failure as shown in Figure 3.5. Repeated loads were applied at 600 and 800 kip load levels with 10 cycles at each load step. As loading continued above 800 kips, one of the loading beams began to yield, forcing the test to be stopped.

The yielding beam was reinforced and the test was restarted on December 21, 2009. Loading proceeded to failure at approximately 1,000 kips for both shafts.

Figure 3.5: Loading configuration for Tests 2 and 3

CHAPTER 4 - RESULTS OF TESTING

This chapter contains a discussion of the testing of the host rock, concrete for the shaft, and the deformations observed during the testing.

4.1 Rock and Materials Testing

Two borings were made and cores recovered at the site in the vicinity of the rock sockets. The boring logs and are presented in Appendix A. Little soil overburden was present on the site. Rock consisted of a 1.5 to 2 feet of sandstone over limestone, however the soil and sandstone were removed so all testing took place in the limestone. A number of rock samples were tested in unconfined compression and the results are reported in Appendix A. Seven of these tests were at elevations considered relevant to this study and the results of those tests are reported in Table 4.1.

This rock core data was considered to represent two layers; an upper, more weathered layer and a lower more competent layer. Representative values for unconfined compressive strength (q_u) and initial intact rock modulus (E) were estimated from plots so that vertical spatial variation could be considered. These plots are shown in Figures 4.1 and 4.2, and were developed by Dan Brown and Associates (DBA).

Table 4.1: Rock Core Test Data used for Analysis

	Sample No.	Depth (Ft)	Unconfined Compression q_u (psi)	Elastic Modulus E (ksi)	Dry Density γd (pcf)	Moisture Percent w%
Upper Layer	14-1-2	3.33	799	292	154.4	2.7
	15-1-3	3.33	2701	448	149.5	4.3
Lower Layer	14-2-1	4.05	4458	958	150.6	3.7
	14-2-2	7.03	7778	1333	157.5	2.1
	15-2-1	4.30	5979	1118	156.9	2.7
	15-2-2	5.30	5056	1042	156.0	3.1
	15-2-3	6.95	4778	660	152.2	4.6

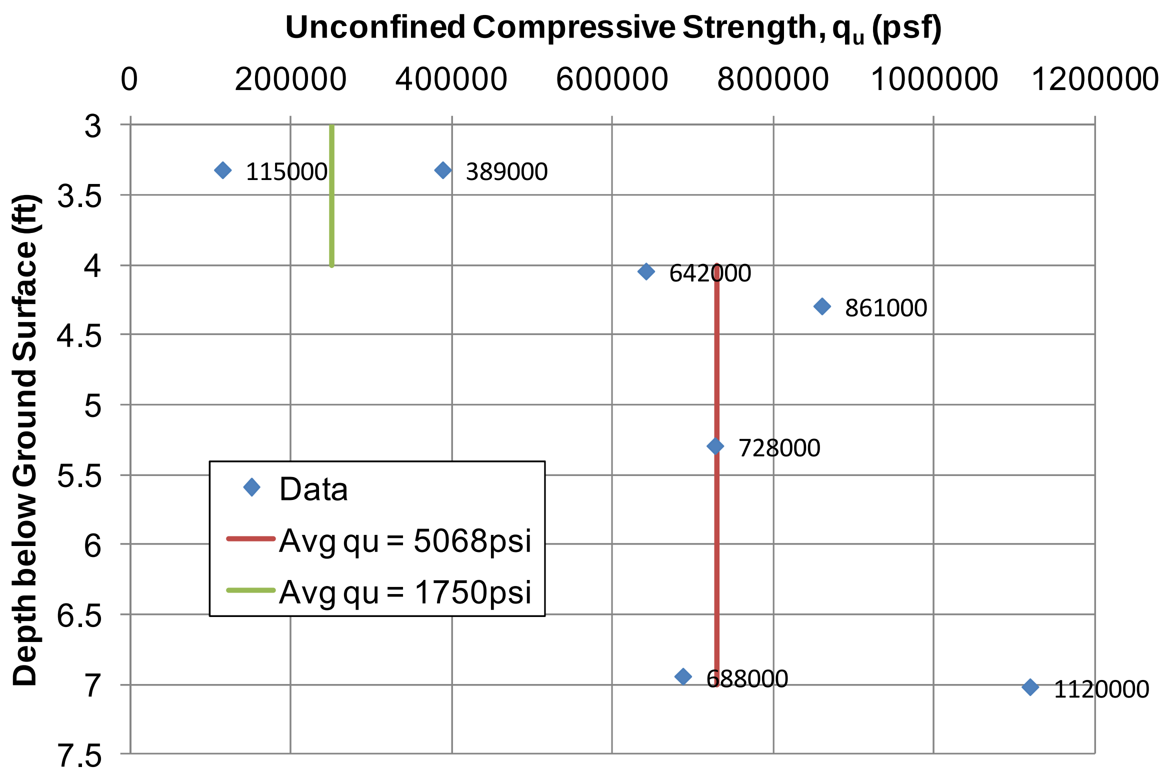

Figure 4.1: Representative unconfined compressive strength (DBA).

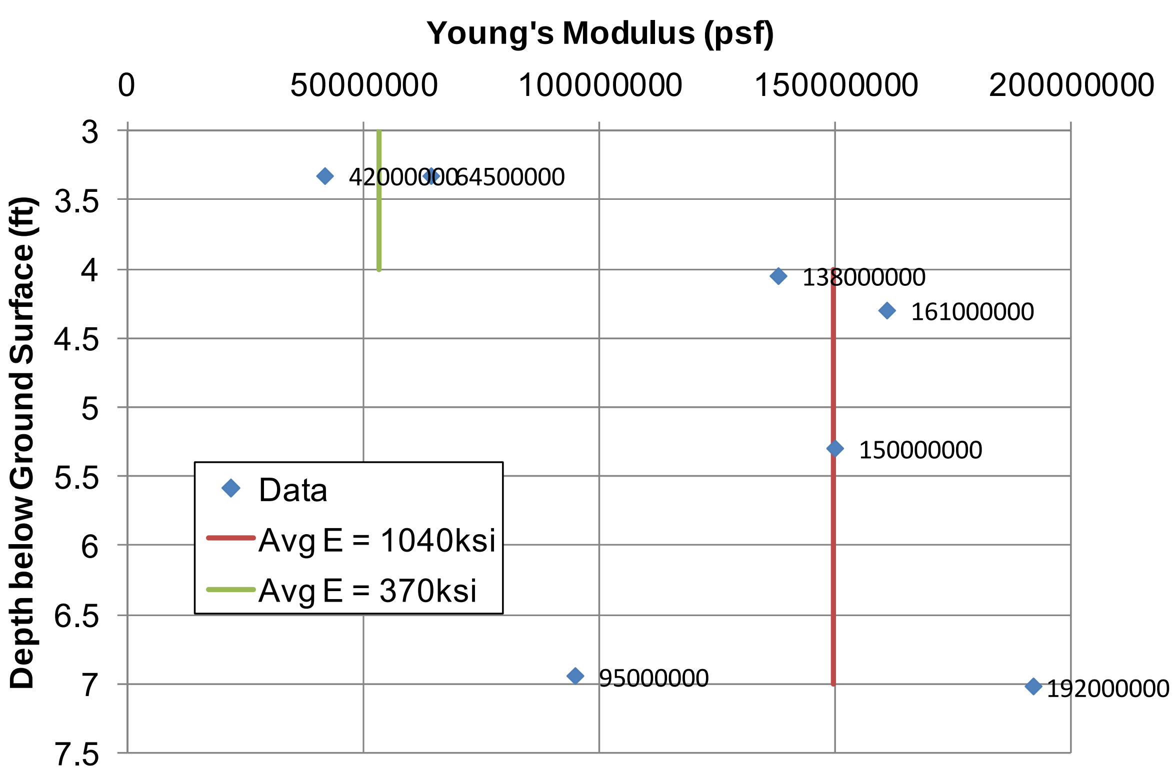

Figure 4.2: Representative intact rock modulus (DBA).

Concrete cylinders were taken when the shafts were constructed and the 28-day curing strength was determined. Values of 7,588 psi and 7,020 psi were measured for an average of 28-day strength of 7,304 psi. Given the additional strength gain that should have occurred prior to actual testing of the shafts and based on cylinders from a concurrent study, a model strength of 7,500 psi was used.

4.2 Field Data

Three separate test events were conducted on the shafts as described in Chapter 3. Figures 4.3 and 4.4 show the deformation for each test event as measured by the top string pots. Data for individual cycles are not shown in these graphs. These figures both show increasing rates of deflection with load to failure, which occurred at approximately 1,000 kips for both shafts. Data for the lower string pots on each shaft were similar.

These figures, after adjustments for the vertical position of the string pots, served as the primary physical test information used to calibrate the LPILE models. The string pot deformation data was checked against inclinometer data, and inclinometer data was used as an absolute reference when combining information from Test 1, 2, and 3.

Additional observations can be made in addition to the general trend of the data. Little to no permanent accumulation of deformation was observed for cyclic loading of the shafts at 400 kips or lower. Accumulation of deformation was significantly greater at the 800 kip loading increment than for the 600 kip loading increment. The south shaft deformed significantly more than the north shaft under the same loading, reaching a deformation of nearly 0.7 inches after cycling at 800 kips while the north shaft had a deformation of approximately 0.3 inches at the same point. This may have been due to natural material variability, or to a road cut that was present approximately 20 feet behind the south shaft in the direction of loading, which could have made it possible for sliding along a weak plane to have occurred in that direction.

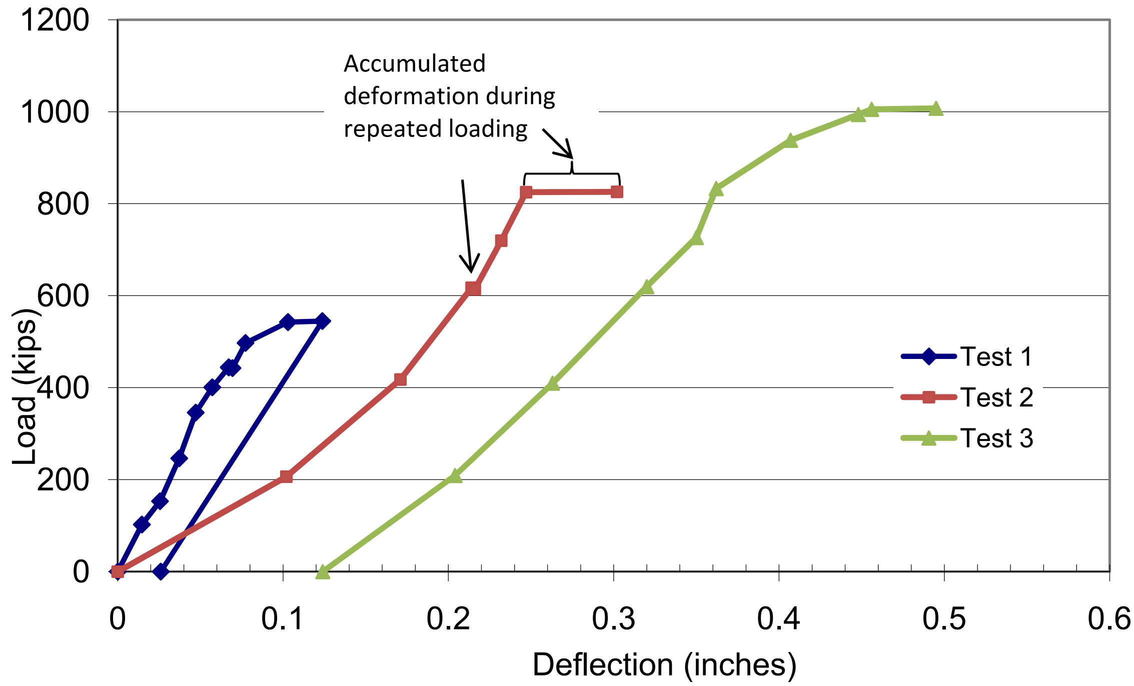

Figure 4.3: Deflection of the north shaft as measured by the top string pot

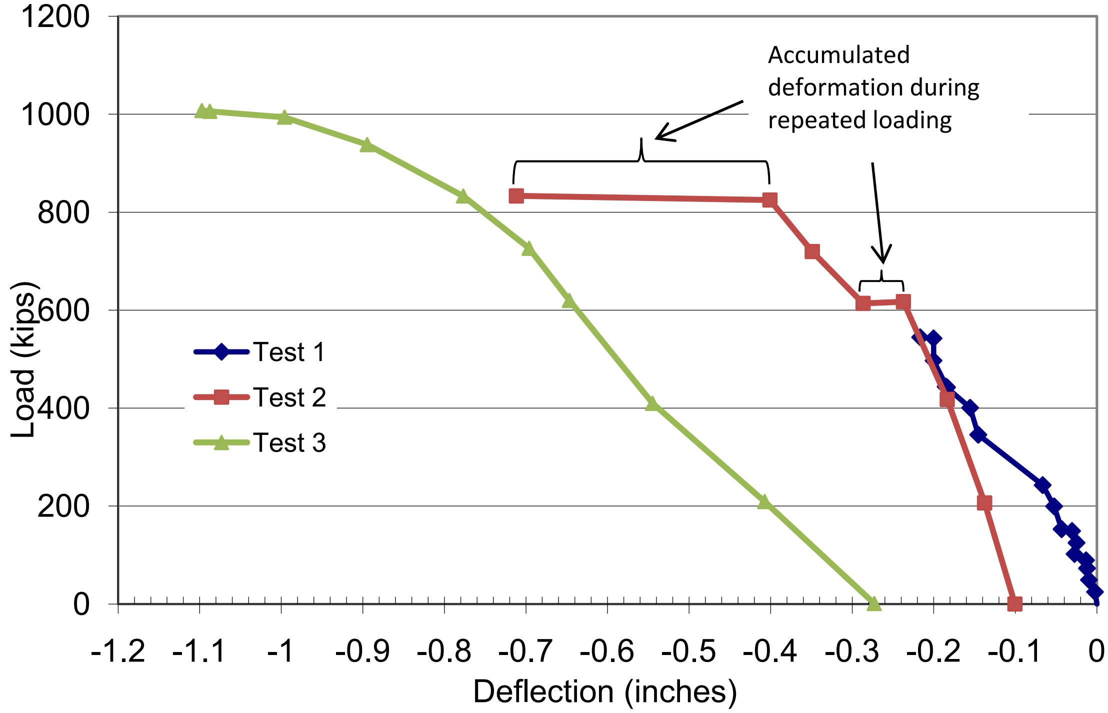

Figure 4.4: Deflection of the south shaft with load as measured by the top string pot

For the north shaft there was no permanent deformation between Test 1 and 2, and there may have even been a small additional rebound between testing events. However, during Test 2 the shaft behaved as if it had a lower modulus in the early stages than it had during Test 1, but then stiffened when loading exceeded 600 kips. For the south shaft this behavior was reversed. The shaft experienced a small permanent deformation as a result of Test 1 and had higher modulus during reloading up to 600 kips. The behavior of the south shaft is consistent with the loading of many geomaterials, where it would be expected that some permanent deformation would be made to the material during the initial loading, and during repeated loadings the geomaterial would have elastic behavior with a higher modulus in that loading range. The mechanics behind the behavior of the north shaft are not well understood, but may be behavior similar to a wobbly tooth where the shaft gradually rebounded to its original position under small lateral earth pressures; but was quickly moved past its maximum deformation level from Test 1 (550 kips) under loading of only 300 kips in Test 2.

4.3 Behavior During Cycling

Cyclic loading (load reversal) was applied at loads of 200, and 400 kips for five cycles each during Test 1. Ten cycles were applied for a load of 300 kips. During Test 2 the load frame was reconfigured for repeated loading where loads were applied and released in the same direction for ten cycles at loads of 600 and 800 kips. This data is presented for the string pots on the south shaft in Figures 4.5 and 4.6, except for two cycles at 200 kips which were not recorded.

Figures 4.7 and 4.8 show more detail for the deformations for the cyclic loading of the north shafts. For these shafts the deformation was reset to zero at the beginning of each set of cycles. These figures show that elastic behavior was observed for cycling below 400 kips.

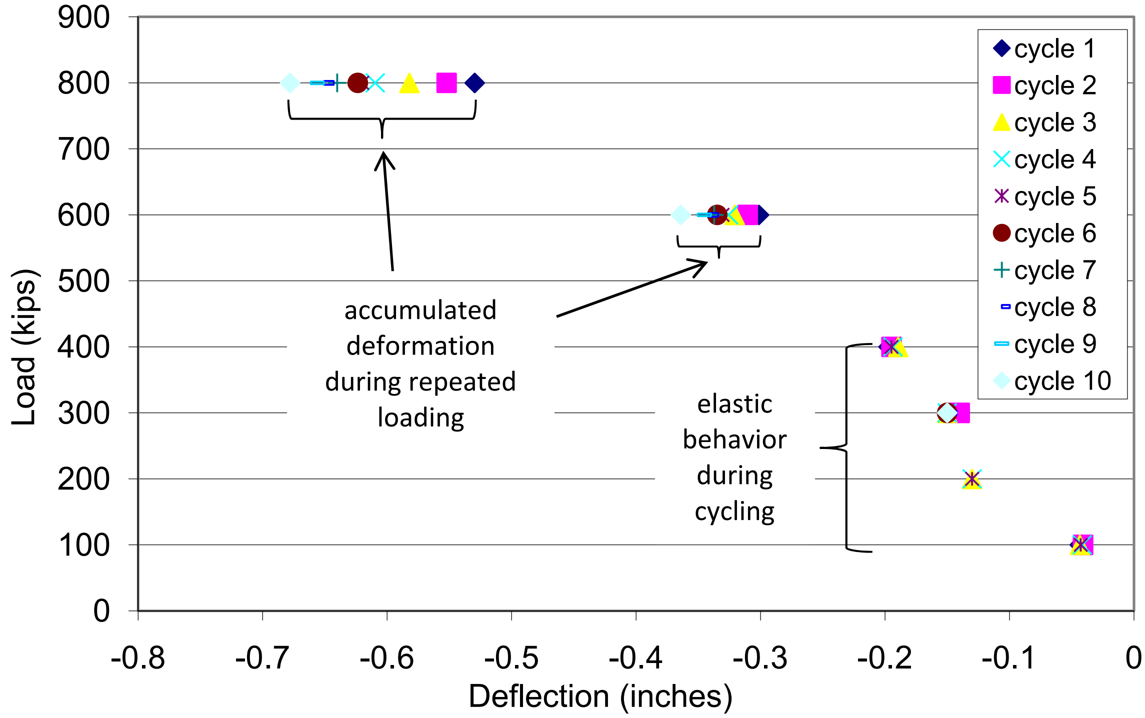

Figure 4.5: South shaft top string accumulated deformation with cyclic and repeated loading

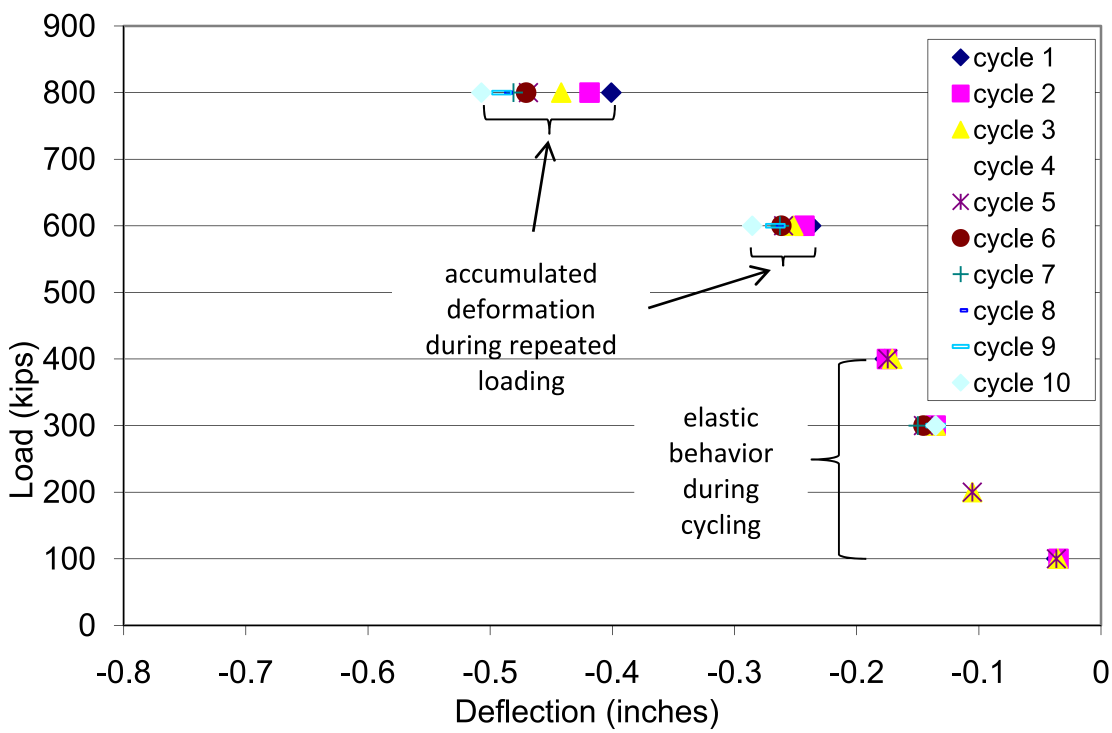

Figure 4.6: South shaft bottom string accumulated deformation with cyclic and repeated loading

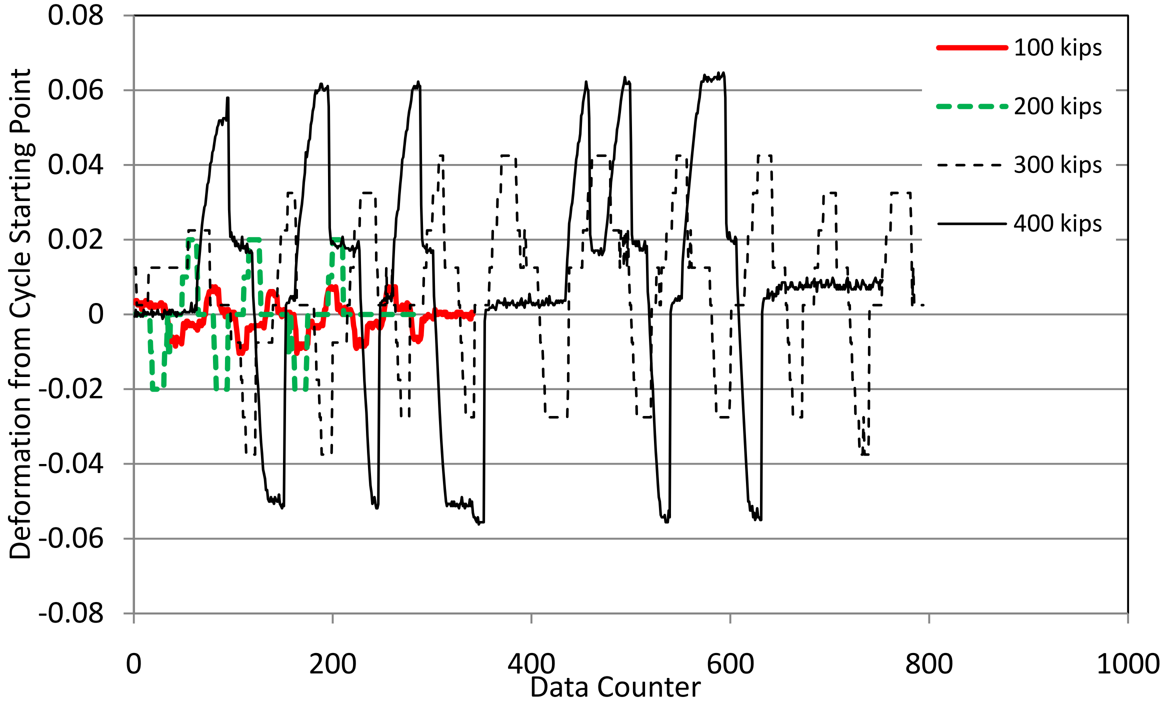

Figure 4.7: North shaft top string elastic behavior with cyclic loading at lower loads

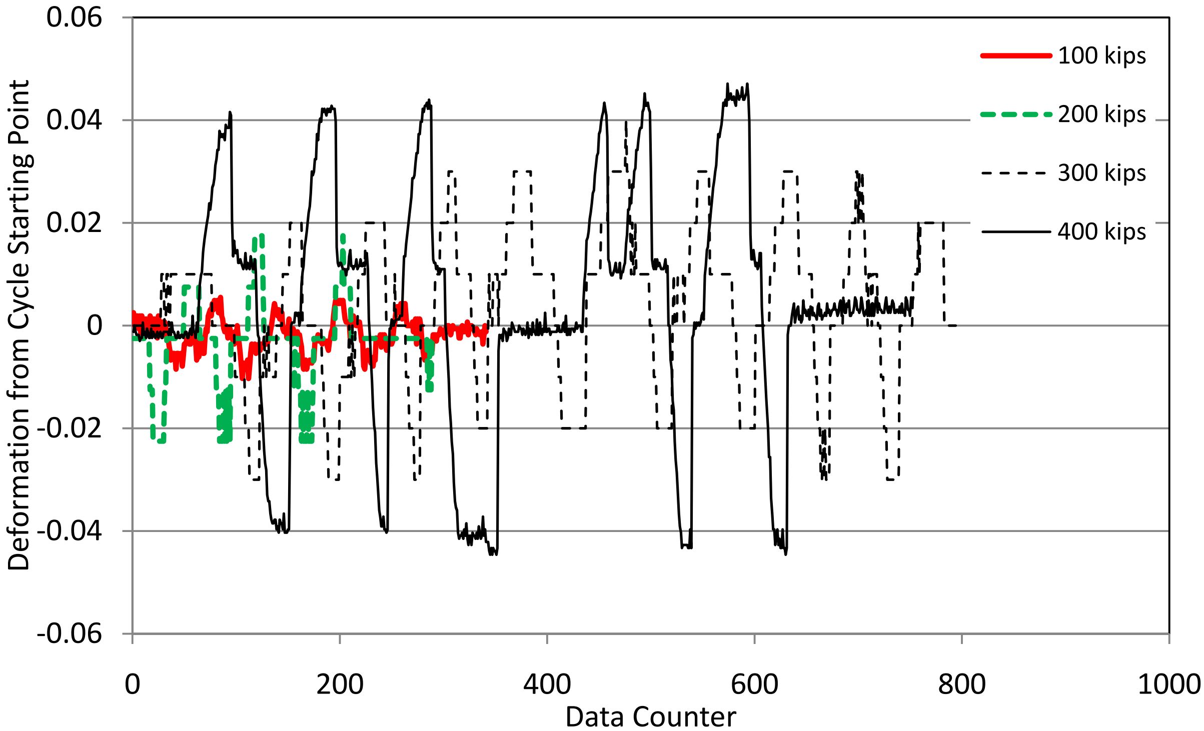

Figure 4.8: North shaft bottom string elastic behavior with cyclic loading at lower loads

Figure 4.9 shows deformations at the end of each loading step for repeated loadings of 600 kips and 800 kips for the top string pot on the north shaft. Similar behavior was observed for the bottom string pot. This behavior for the north shaft is similar to that observed for the south shaft.

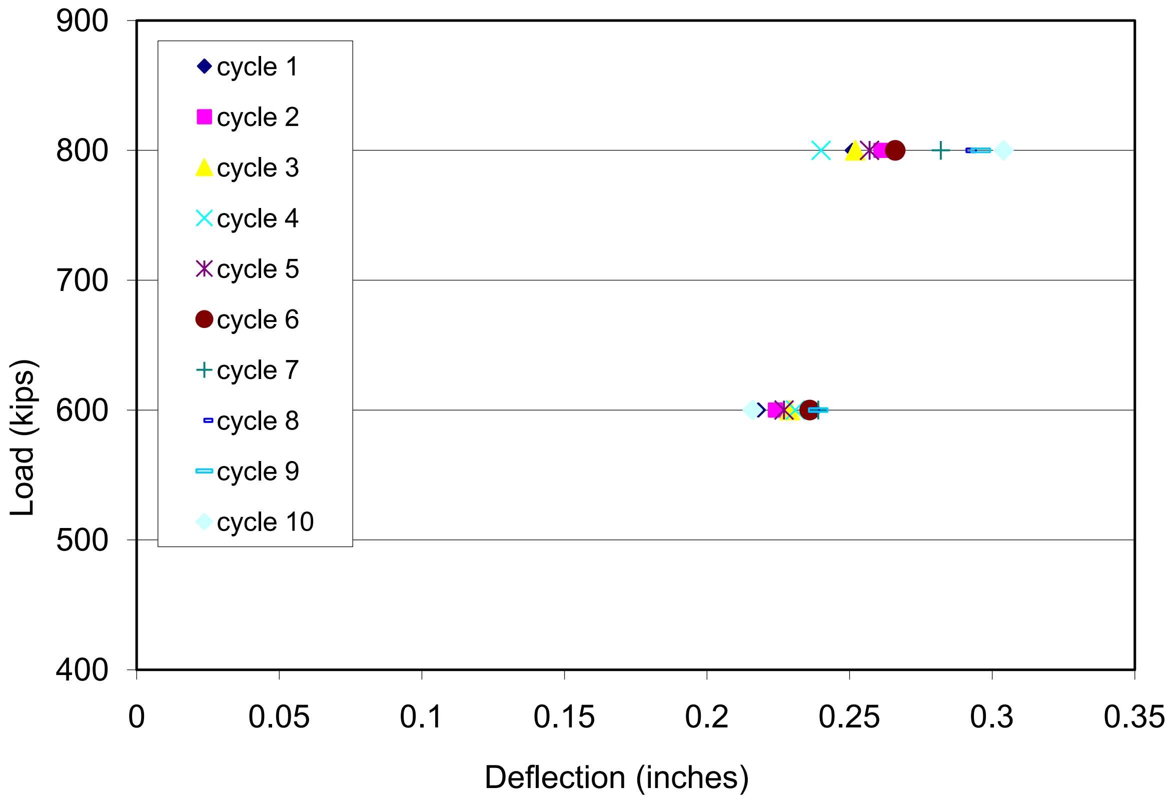

Figure 4.9: North shaft top string deflections during repeated loadings

CHAPTER 5 - LPILE MODELING

5.1 Modeling Parameters

The rock-socket test data was modeled using the commercial program LPILE for the purpose of identifying appropriate p-y modeling parameters for limestone in Kansas. The “weak rock” model contained within LPILE combined with a Type 3 analysis, which considers non-linear bending, was determined to be the most appropriate model based on recommendations from Dan Brown and Associates (Paul Axtell, personal communication). Properties used in the modeling are presented in Table 5.1.

Table 5.1: LPILE Modeling Parameters

Shaft Properties

Shaft Diameter	42 inches
Concrete Strengths	7500 psi
Longitudinal Reinforcement	12 - #11 bars
Distance from pile top (point of loading) to ground surface	12 inches
Yield stress of steel	60,000 psi
Steel modulus	29,000,000 psi

Rock Properties

	Upper Layer	**Lower Layer**
Intact Rock Strength	1750 psi	5068 psi
Intact Rock Modulus	370 ksi	1040 ksi
k	0.0005	0.0005

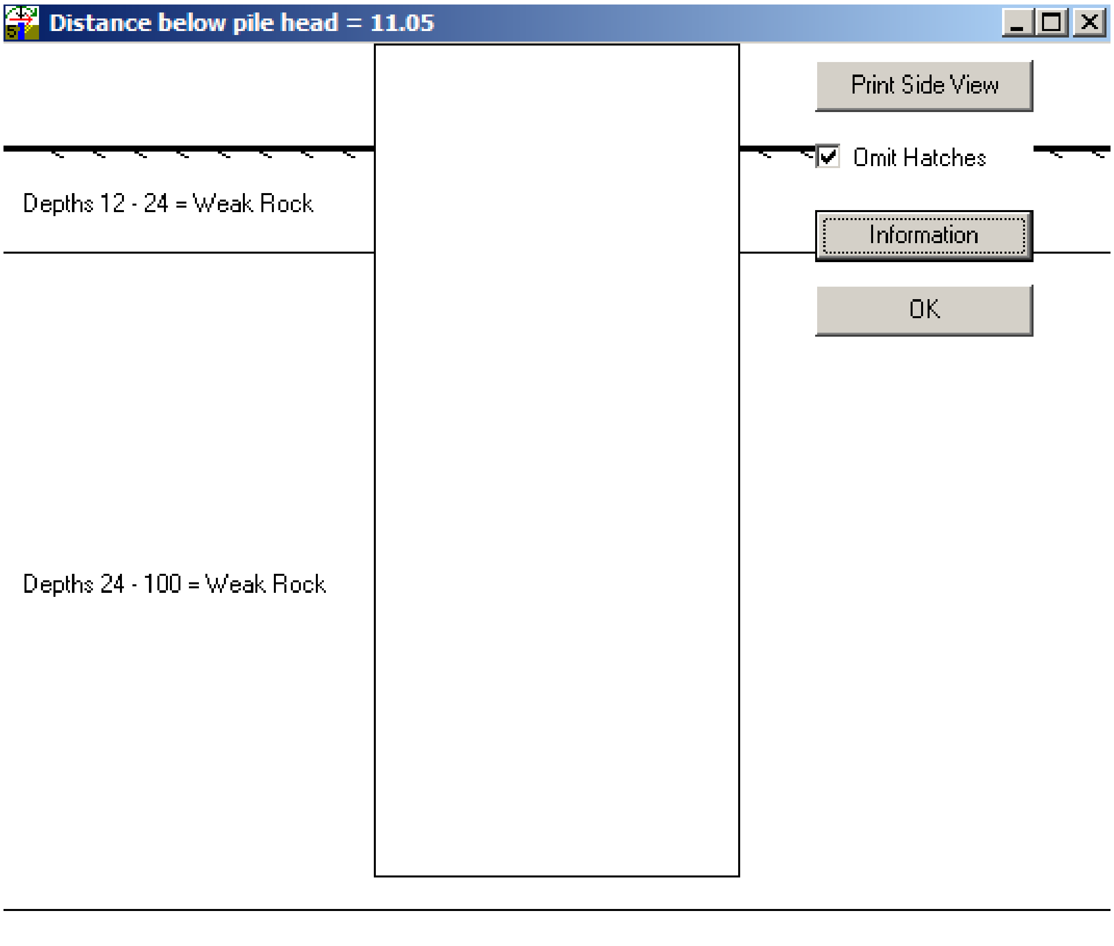

Figure 5.1: General layout of shaft in model

The layout of the model is shown in Figure 5.1. For modeling purposes the top of the shaft is the point of load application.

Once the geometry and reinforcement of the shaft are determined, there are only two remaining parameters that must be selected by the modeler. The value of k is adjustable, and the value of 0.0005 that was used is within the recommended range (Reese et al 2004). There is also some justification for reducing the rock modulus because the modulus of the rock mass should be less than the modulus of intact samples, however this should be accounted for to some degree by the inclusion of RQD within the model.

5.2 Discussion of Modeling

When fitting the load-deformation curves generated within LPILE to the load test data, the accumulated deformation that occurred during repeated loading needed to be accounted for. This was addressed by shifting the LPILE curves by the amount of the accumulated deformation. These LPILE curves are plotted with the field test data in Figures 5.2 and 5.3. These figures show that a good fit can be made between the weak rock LPILE model and the field test data. A selection of actual p-y curves generated within LPILE is presented in Figures 5.4. These p-y curves apply to both shafts.

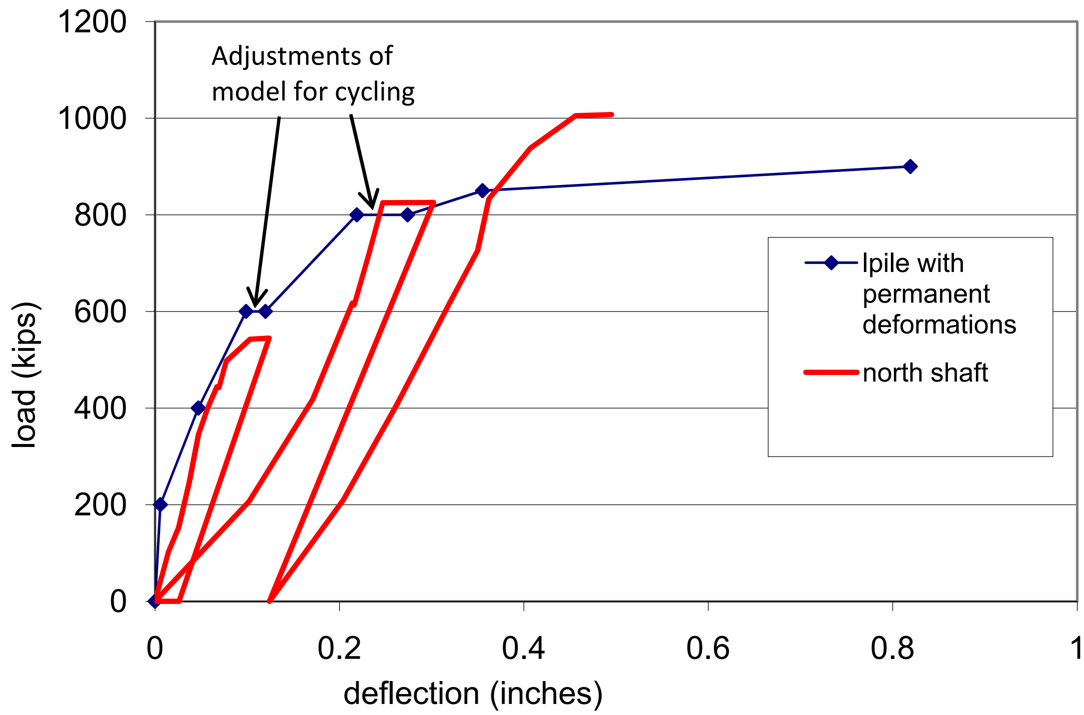

Figure 5.2: LPILE model and load test data for the north shaft

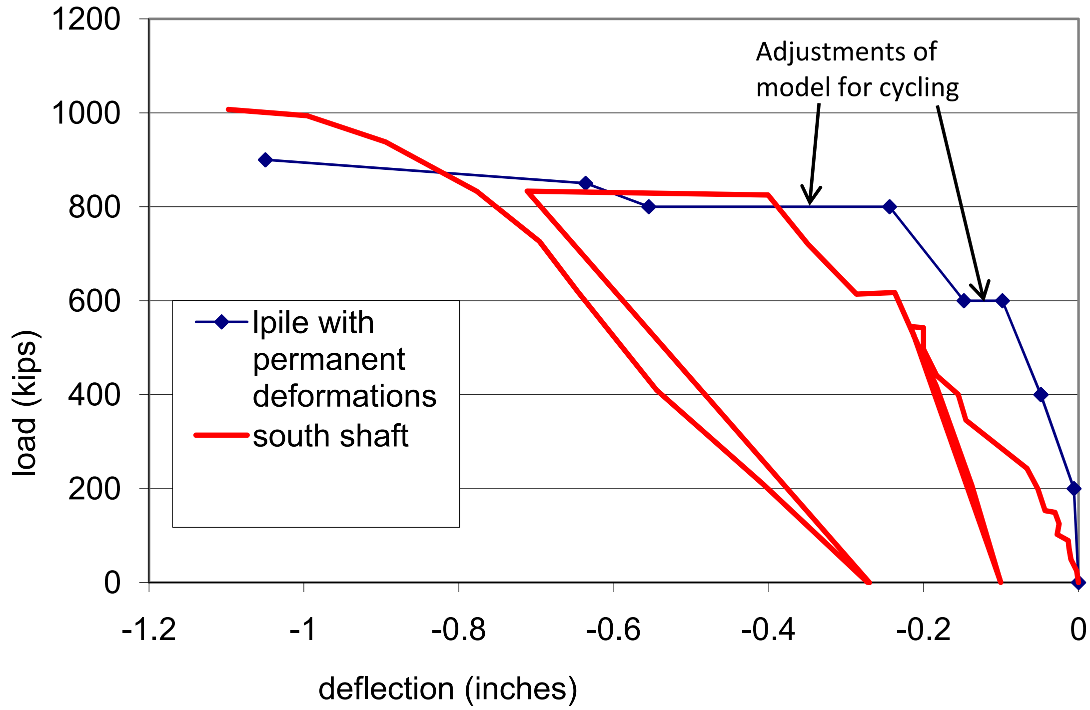

Figure 5.3: LPILE model and load test data for the south shaft

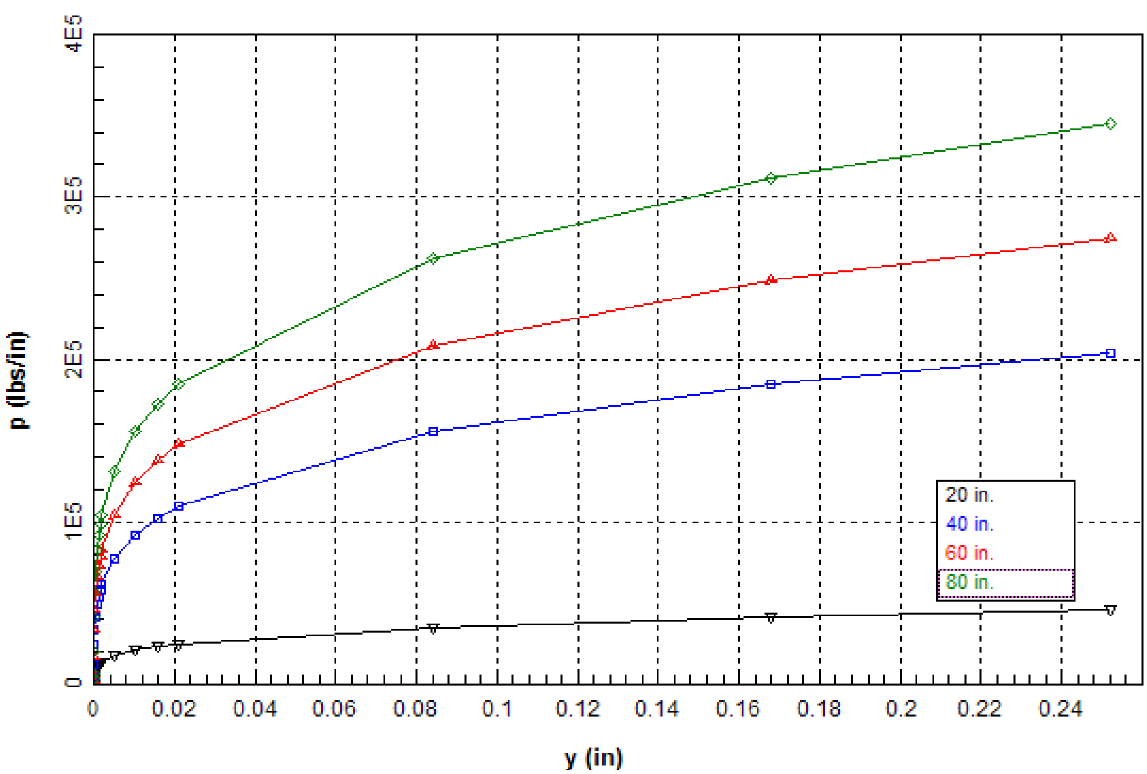

Figure 5.4: P-Y curves using the intact rock modulus

While the pile head deformations and ultimate load are approximated well by the models shown in Figures 5.2 and 5.3, deformation of the shaft does not match particularly well with the inclinometer data. Figure 5.5 shows the predicted deformation of the north shaft from the LPILE model. This figure shows essentially no bending of the shaft below a depth of 2.5 feet (3.5 feet below the point of load application). This is not consistent with the inclinometer data taken during the test (Figure 5.6), which shows movement of the shaft throughout the length of the shaft. Note, when considering the inclinometer data it is important to remember that the base of the shaft is assumed to have zero horizontal movement. This does not have to be the case as the shaft bottom will sometimes rotate back in the direction of loading. The lack of bending in the model suggests that the modulus used for the rock in the model is higher than the actual rock modulus. This is reasonable given that the modulus of a rock mass would be expected to be lower than the modulus of intact rock samples, and while the Reese method accounts for this to some degree, it may not be sufficient. Additionally, the modulus of the rock mass may have degraded further during repeated loading.

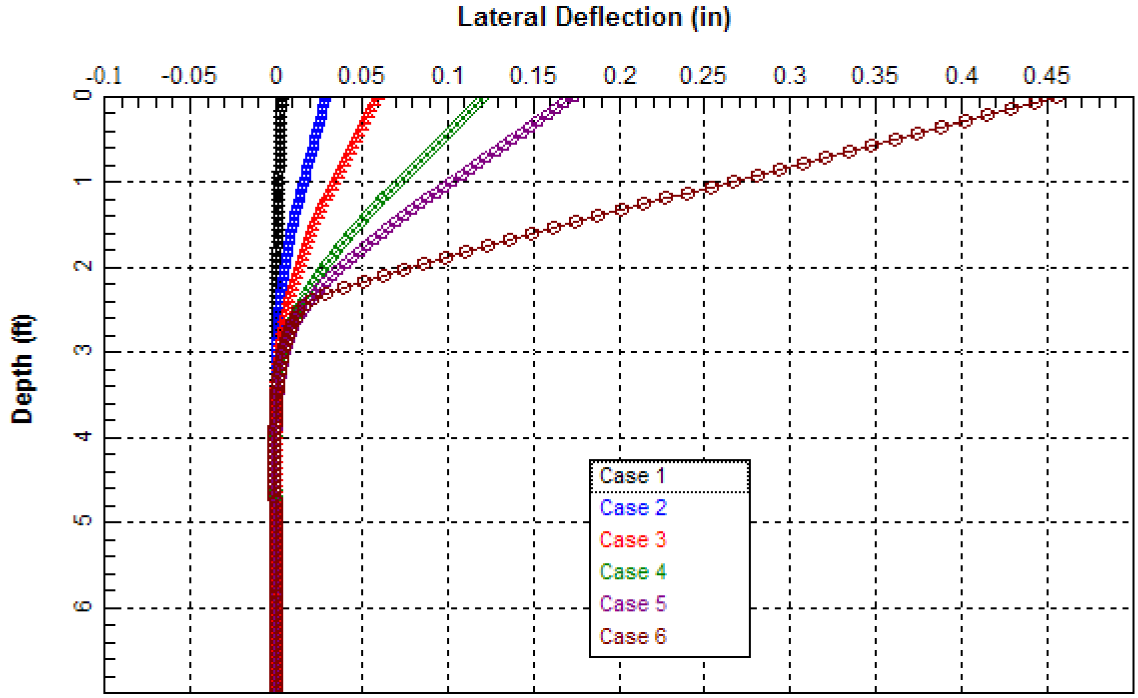

Figure 5.5: Predicted deformation of the north shaft in LPILE with intact rock modulus

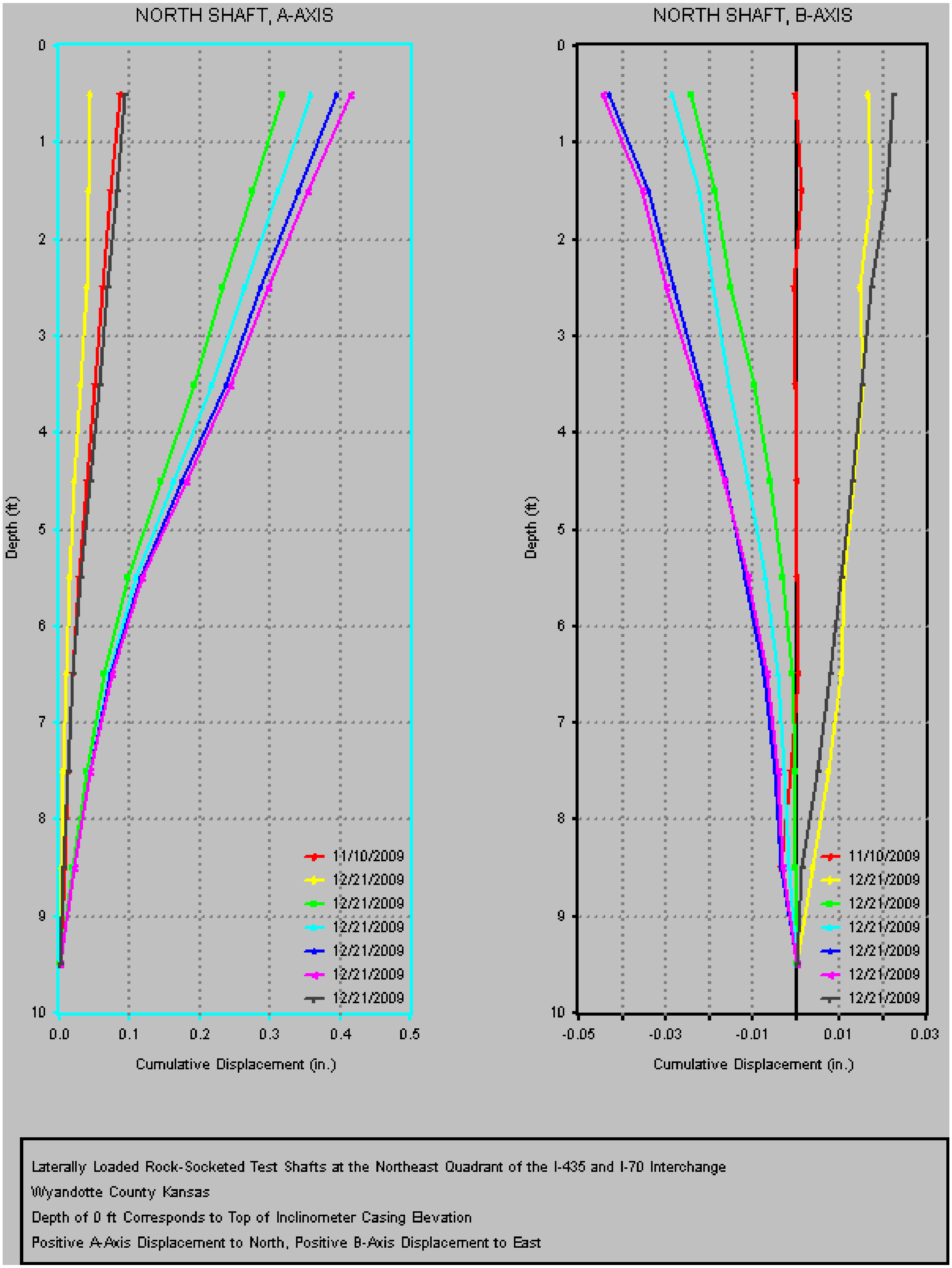

Figure 5.6: Inclinometer data for the north shaft during and after Test 3

Therefore the analysis was redone using a modulus that was 1/100 of the intact rock modulus for the north shaft and 1/150 of the intact rock modulus for the south shaft. The predicted load-deformation curves are shown in Figures 5.7 and 5.8. No adjustment is made in these figures for accumulated deformations due to cycling as this is assumed to be accounted for in the reduced modulus. These figures show the model predicts the general load-deflection trend well, although it underpredicts the ultimate capacity of the shafts by about 10 percent. Figure 5.9 shows the predicted bending of the shaft. This figure shows that predicted lateral movement at the top of the shaft is nearly identical to the field data and that some bending occurs all the way to the bottom of the shaft, and therefore represents a better match with the observed data.

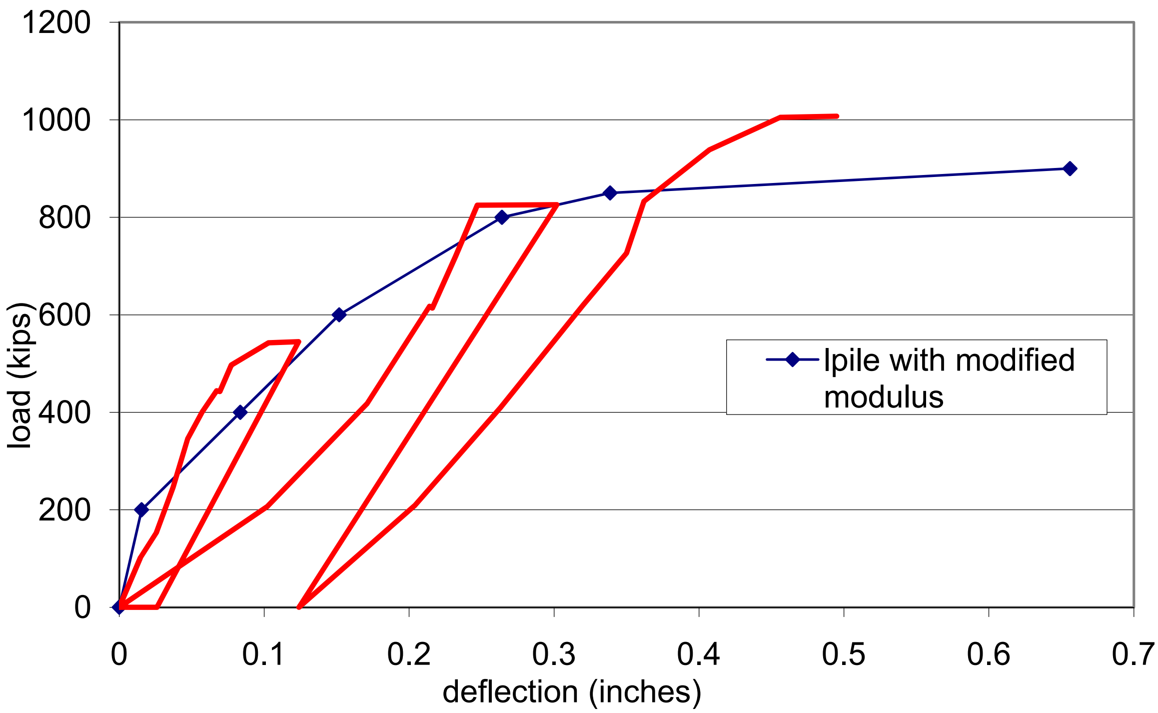

Figure 5.7: LPILE model of north shaft with a reduced rock modulus with field data

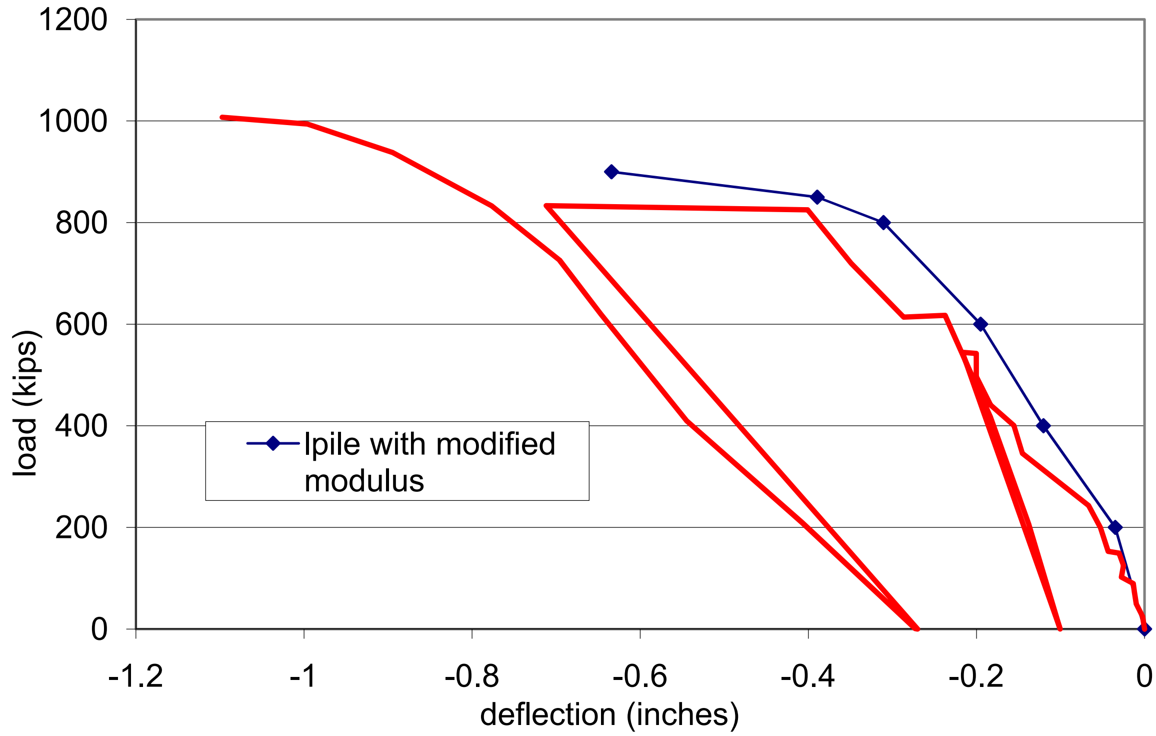

Figure 5.8: LPILE model of south shaft with a reduced rock modulus with field data

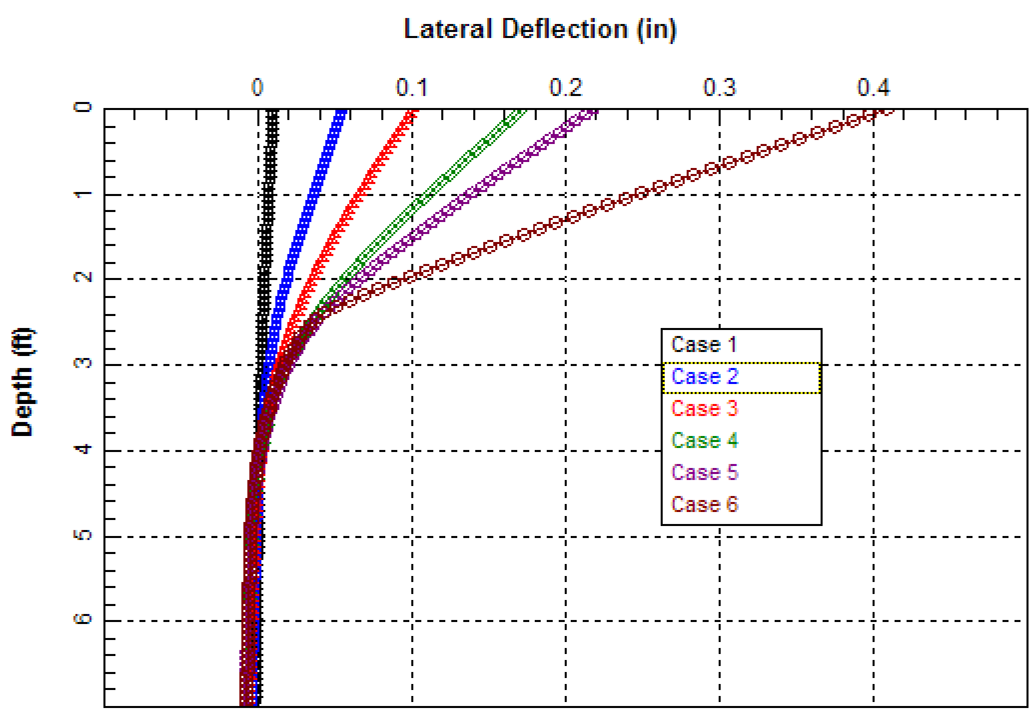

Figure 5.9: Predicted deformation of the north shaft in LPILE using 1/100 of the intact rock modulus

5.3 Effects of Changing Shaft Reinforcement

Figure 5.5 shows sharp bending in the middle of the north shaft as failure is approached at 900 kips in the model, and Figure 5.10 shows shaft stiffness approaching zero as the bending moment approached 20,000 in-kips, indicating that failure of the shaft materials was a major factor in shaft capacity. Therefore another model was created to explore the potential benefits of changing the reinforcement.

For this model the reinforcement was changed to #14 bars from #11. This change resulted in an increase in predicted capacity to 1,150 kips from 900 kips. Deflections were predicted to be less than 0.1 inch for a load of 900 kips (Case 5, Figure 5.11), and 0.37 inches at 1,150 kips. The increase in steel enabled the shaft to tolerate bending moments approaching 28,000 in-kips before stiffness went to zero.

Similarly, if the steel reinforcement is stronger than the design value of 60,000 psi, the model capacity of the shaft will increase. If a value of 70,000 ksi is used for the steel, the ultimate capacity increases to approximately 1,000 kips, which is the value observed in the field.

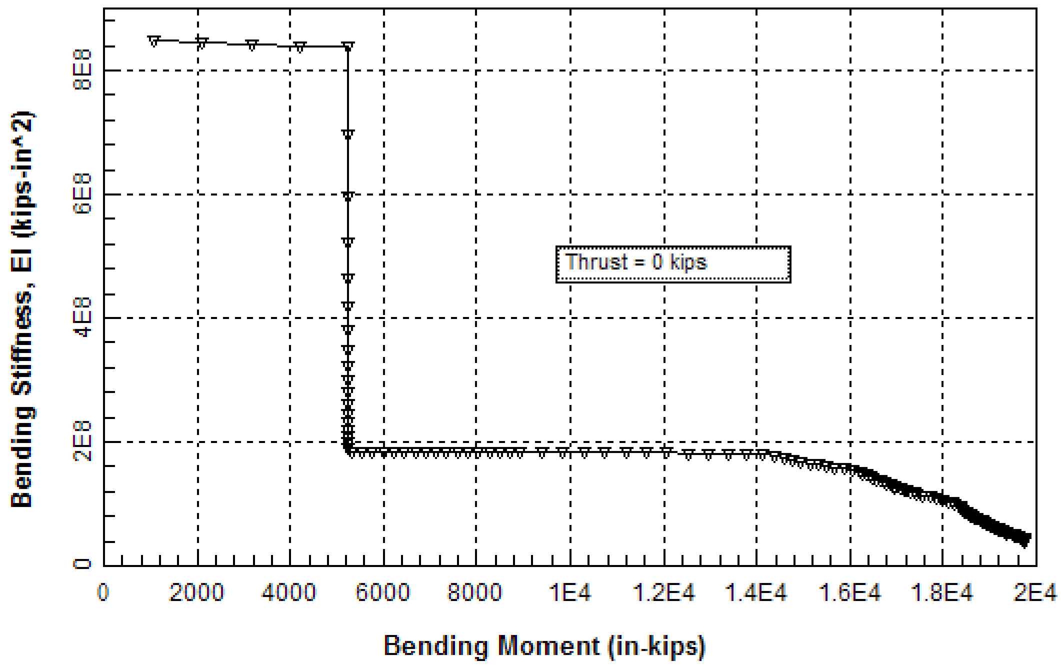

Figure 5.10: Bending stiffness of the north shaft with changes in moment

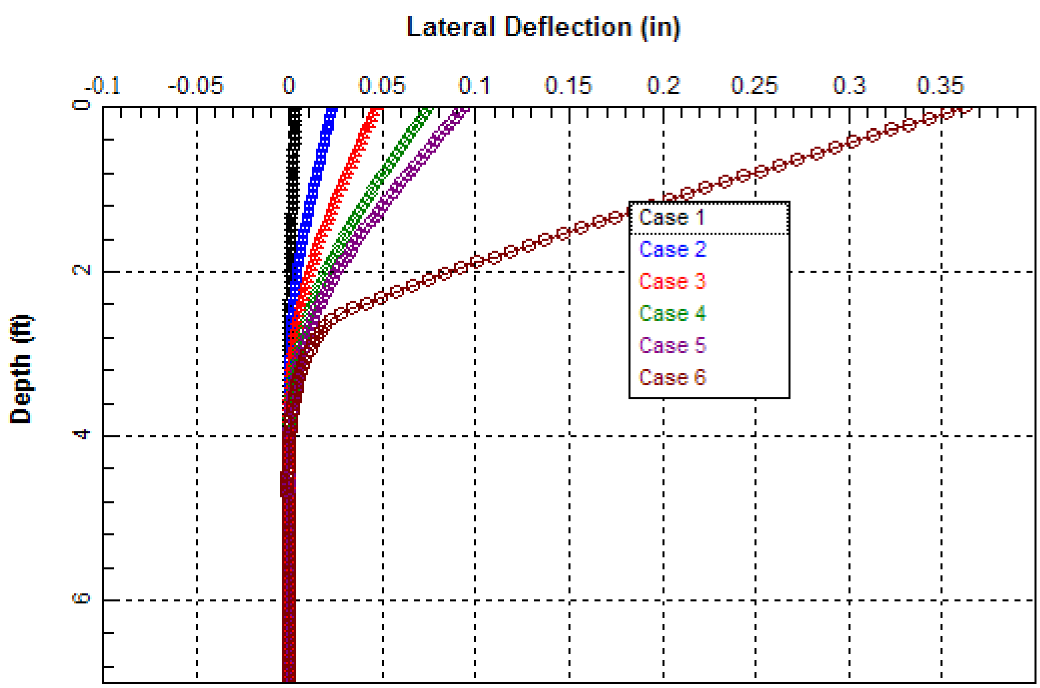

Figure 5.11: Predicted deflection for the north shaft with increased reinforcement

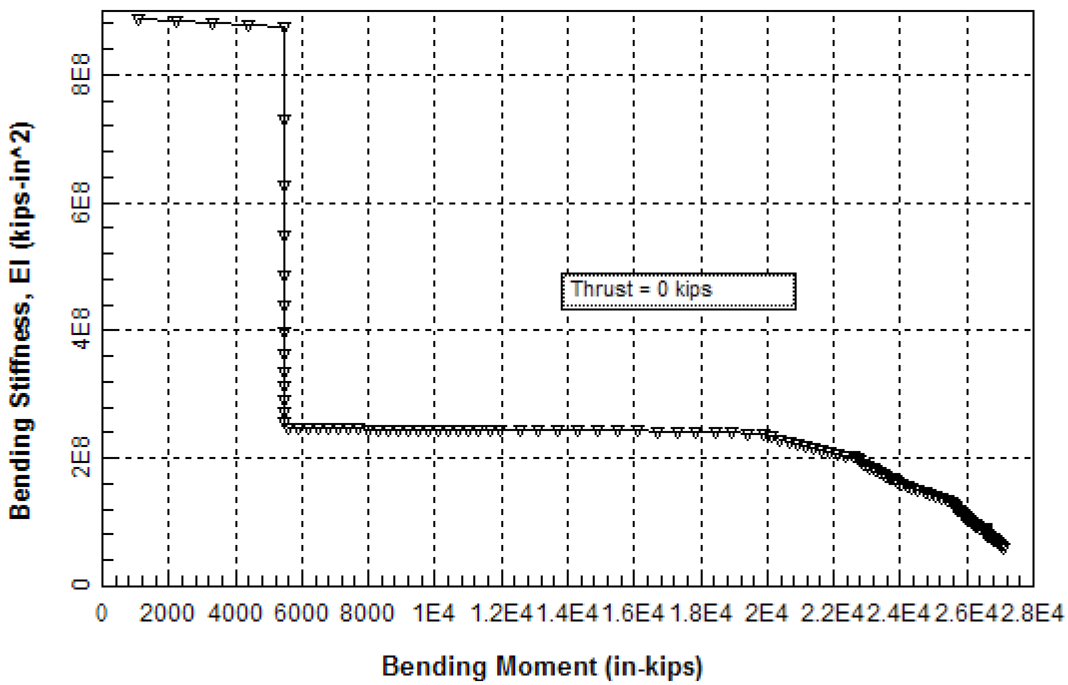

Figure 5.12: Bending stiffness of the north shaft with changes in moment with increased reinforcement

CHAPTER 6 - CONCLUSIONS AND RECOMMENDATIONS

Two 42-inch diameter drilled shafts in limestone were laterally loaded to failure. Cyclic and repeated loading steps were conducted at a series of load steps prior to failure. The following conclusions were drawn from the field data.

- The ultimate capacity of both shafts was approximately 1,000 kips.
- The ultimate capacity was reached at approximately 0.45 inches for the north shaft and 0.95 inches for the south shaft. Both of these deformation values include deformation that accumulated during periods of repeated loading. Maximum deformations for static load test conditions would likely have been less.
- Deformations for the south shaft may have been affected (increased) by the presence of a road cut approximately 20 feet behind the shaft.
- The shafts behaved in an elastic manner for five cycles of loading at 200 and 400 kips (40% of ultimate load) and 10 cycles at 300 kips.
- The shafts experienced permanent, accumulating deformations for repeated loading at 600 kips (60% of ultimate load), and even greater deformations at 800 kips.

The resulting field data was modeled using the commercial software LPILE. The model used was a Type 3 analysis of shafts in the weak rock model described in Chapter 2. The following conclusions were developed based on the modeling.

- The ultimate capacity and ground line deformations could be modeled reasonably well using the weak rock model contained within LPILE. Predicted ultimate capacity was within 10 percent of field measurements and the slope of the load-deformation curve (modulus) was consistent with field data when accumulated deformations were accounted for.

- For this model, most of the data to be entered is driven by the material properties and geometry, which makes construction of the model very straightforward.
- The user does have control over the value of k_{rm}. The authors used the value of 0.0005 for this parameter, which is the upper end of the recommended range.
- The model prediction of shaft bending showed minimal bending in the lower half of the shaft. Reducing the value of the rock modulus resulted in an increase in the predicted bending of the shaft, which better matched inclinometer measurements and did not change the ultimate capacity of the shaft significantly. A reduction of the modulus may be warranted given the rock mass likely accumulated damage during the repeated loading steps, which would have lowered the modulus of the rock mass.
- Increasing the strength of the reinforcing steel in the model reduced the predicted deformation and increased ultimate shaft capacity.

Based on these conclusions, the following preliminary recommendations are made for modeling of limestone in Kansas. They are considered preliminary because they are based on a single test program and should be updated as more data becomes available.

- Use of the weak rock model included within LPILE is recommended for Kansas limestone.

- Within this model it is recommended that a value of 0.0005 be used for k_{rm} if no other information is available.
- It is also recommended that for cyclic or repeated loading design where the number of cycles is expected to be relatively small (i.e. extreme events), the limestone can be considered elastic for loads of less than 40% of the ultimate load.
- If the intact rock modulus is the basis for selecting the rock modulus value used in LPILE, use of a reduced value may be warranted to more accurately model shaft bending.

REFERENCES

Google Maps (2010). http://maps.google.com/maps. Google, Inc.

Reese, L.C. (1997). *Analysis of Laterally Loaded Piles in Weak Rock*. Journal of Geotechnical and Geoenvironmental Engineering. ASCE. Reston, Virginia. v123 n11. 1010-1017.

Reese, L.C., S.T. Wang, W.M. Isenhower, and J.A. Arrellaga (2004). Technical Manual, LPILE Plus 5.0 for Windows, A Program for the Analysis of Piles and Shafts Under Lateral Loads. Ensoft, Inc. Austin TX.

APPENDIX A*

*Appendix A is available on CD only upon request.

Please send your request to library@ksdot.org.

K - TRAN

KANSAS TRANSPORTATION RESEARCH
AND
NEW - DEVELOPMENTS PROGRAM

A COOPERATIVE TRANSPORTATION RESEARCH PROGRAM BETWEEN:

KANSAS DEPARTMENT OF TRANSPORTATION

THE UNIVERSITY OF KANSAS

KANSAS STATE UNIVERSITY