ABSTRACT

Joseph Blair Nixon, Verification of the Weathered Rock Model for P-y Curves. (Under the direction of Dr. Mohammed A. Gabr)

Four full-scale field lateral load tests were conducted to independently verify the Weathered Rock Model for P-y Curves published by Cho (2002). Class A performance predictions were developed and presented to North Carolina Department of Transportation officials prior to testing. Performance predictions were calculated using the Weathered Rock Model, Reese’s Method for P-y Curves in Weak Rock (Reese, 1997), and the Stiff Clay Model (Reese, Cox, and Koop, 1975). Field testing was conducted on two sites in Durham County, North Carolina; both sites were located within the Durham Triassic Basin. Two fully instrumented drilled shafts were constructed at each site; the instrumentation plan allowed for the measurement of top deflections, head rotation, shaft deflection and strain with depth. Results obtained from each test shaft are compared with respective Class A performance predictions. Results of verification testing show that the Weathered Rock Model can be used to accurately model the lateral deflection behavior of drilled shafts embedded in weathered rock profiles. Distribution of the subgrade reaction (k_h) evaluated from testing results is compared with that published by Cho (2002) for other types of weathered rock. The magnitude of the increase in k_h below the point of rotation for Triassic Weathered Rock is not as significant as that realized by Cho (2002). Recommended design procedures for using the Weathered Rock Model with either rock dilatometer data or geologic data are presented.
VERIFICATION OF THE WEATHERED ROCK MODEL FOR P-y CURVES

By

JOSEPH BLAIR NIXON

A thesis submitted to the Graduate Faculty of
North Carolina State University
In partial fulfillment of the
Requirements for the Degree of Master of Science

CIVIL ENGINEERING

RALEIGH, NORTH CAROLINA
2002

APPROVED BY:

Chair of Advisory Committee
M. A. Gabr, Ph.D., P.E.

Co-Chair of Advisory Committee
R. H. Borden, Ph. D., P.E.

M. S. Rahman, Ph.D.
DEDICATION

The work presented hereafter is dedicated to my grandfather, Mr. David Samuel Gray, Jr. of Bristol, Tennessee. Memories of him have taught me to cherish those still here. I wish he could have witnessed this accomplishment, I’m sure he would have been proud.
BIOGRAPHY

Joseph Blair Nixon was born on September 23, 1976 to William Lewis and Laura Jane Nixon of Waxhaw, North Carolina. He was the youngest of three children, David and Pamela. Joseph attended Waxhaw Elementary, Parkwood Middle, and Parkwood High Schools for his primary education. Upon high school graduation, he attended The University of South Carolina at Lancaster for one year. At the end of that year, David and his sister-in-law, Laura Lynn, encouraged him to attend a nearby technical college. Taking a “leap of faith” he enrolled at York Technical College in Rock Hill, South Carolina. There he studied Civil Engineering for the first time under the direction of Mr. Jim Fitzpatrick Jr., P.E. Mr. Fitzpatrick’s passion for the discipline instilled a great desire in Joseph to excel; at graduation ceremonies he received the President’s Award for Excellence in Engineering, a surprising recognition for someone that struggled to graduate high school. Armed with encouragement from Mr. Fitzpatrick and his family, Joseph enrolled at North Carolina State University to further his education in Civil Engineering. There, in the fall of 1998, he met Carrie Elizabeth Trimble. The two struck an unbreakable bond from the beginning. After graduation, they married on July 28, 2001 in Cornelius, North Carolina. Carrie went to work and Joseph enrolled, again, to pursue his Master’s of Science in Civil Engineering at North Carolina State.
ACKNOWLEDGEMENTS

I would like to express my appreciation to advisor and committee chair, Dr. Mohammed A. Gabr, his direction and faith in me were essential to the completion of this work. Also I would like to thank my committee co-chair, Dr. Roy H. Borden Jr., his time, teaching, and endless words of encouragement will never be forgotten. The teaching and efforts of these two men have fashioned the engineer I am today. Thanks are also extended to Dr. M. Shamimur Rahman for his role on my review committee and interest in my work.

I owe a large debt of gratitude to several friends for their help with this research. Kook Hwan Cho and Shane Clark laid the groundwork for this thesis; without their endless hours of work and support, this research program would not of been available for my participation. Eric Williams of the North Carolina Department of Transportation was extremely helpful with the logistics of performing four field load tests in a two-week time frame. A new friendship with Alex Smith began as a result of this project; selflessly he worked long hours preparing for field testing and assisting with prediction and test data analysis. All of this, and it wasn’t even the subject of his own thesis. I will never forget the sacrifices he made.

To my family: Mom, Dad, Pamela, David, Laura Lynn, Stephen, Sarah, Will, Davis, Mr. and Mrs. Trimble, Donna, Tommy, and Thomas. I am not allotted enough words to express my deep gratitude for the support and prayers all of you have extended for me over the course of this long road. A man is nothing without his family; I am living proof of that fact.

Individually, I want to recognize my mother. Your never-ending hours of prayer have protected me through the darkest periods of my life. The depth of your faith in God and love of your family truly amazes me. With everything I am, thank you. I hope you are proud.

Finally, to you Carrie, you have taught me what love is truly about. Your sacrifice, patience, and support have allowed me to finish this work. While it has seemed as if our lives have been on hold for 18 months now, we have actually laid the
groundwork for what is to come. I couldn’t imagine spending the rest of my life with anyone else.
# TABLE OF CONTENTS

LIST OF TABLES ................................................................................................................VIII

LIST OF FIGURES .............................................................................................................. IX

CHAPTER 1. INTRODUCTION .......................................................................................... 1
  1.1 Problem Statement ................................................................................................. 2
  1.2 Objectives .................................................................................................................. 2

CHAPTER 2. LITERATURE REVIEW ............................................................................... 3
  2.1 P-y Curves based on the Stiff Clay Model ............................................................. 3
  2.2 P-y Curves for Weak Rock .................................................................................... 4
  2.3 The Weathered Rock Model .................................................................................... 7
    2.3.1 Laboratory Testing (Clark, 2001) ................................................................. 8
    2.3.2 Field Testing ..................................................................................................... 12
      2.3.2.1 P-y Curves from Measured Strain Data ................................................... 15
      2.3.2.2 Rock Dilatometer Testing ....................................................................... 18
    2.3.3 Concepts of the Weathered Rock Model ........................................................ 21
      2.3.3.1 Subgrade Reaction ($k_h$) for Weathered Rock ....................................... 22
      2.3.3.2 Determination of the Subgrade Reaction ($k_h$) for Weathered Rock from
            Empirical Equations and Geologic Parameters ........................................... 29
      2.3.3.3 Determination of the Subgrade Reaction ($k_h$) for Weathered Rock based on
            Rock Dilatometer Testing ........................................................................... 30
      2.3.3.4 Determination of the Ultimate Lateral Resistance ($P_{ult}$) for Weathered Rock
            .................................................................................................................... 31
      2.3.3.5 Field Test Predictions ............................................................................. 35
    2.4 Summary of Literature Review .............................................................................. 37

CHAPTER 3. LATERAL LOAD TESTING FOR THE VERIFICATION OF THE
WEATHERED ROCK MODEL FOR P-Y CURVES ............................................................ 38

  3.1 Verification Load Testing ......................................................................................... 38
    3.1.1 Instrumentation Plan ......................................................................................... 40
  3.2 Interstate 40 Load Tests ......................................................................................... 42
    3.2.1 Geology ............................................................................................................. 43
    3.2.2 Geotechnical Properties of the Test Site ......................................................... 44
LIST OF TABLES

Table 1. List of Test Sites and Rock Types (Cho, 2002) .............................. 13
Table 2. Rock Mass Rating (RMR) Method (Bieniawski, 1976) .................... 27
Table 3. Parameters m_b, S, and a (Hoek et al., 1995) .............................. 33
Table 4. m_i Value (Hoek and Brown, 1988) ........................................... 34
Table 5. Verification Test Sites and Rock Types ....................................... 39
Table 6. I-40 Test Site Core Log .......................................................... 46
Table 7. I-40 Laboratory Test Results ................................................... 47
Table 8. I-40 Rock Dilatometer Results – k_{ho} Values .............................. 48
Table 9. Parameters for I-40 Predictions – Dilatometer ............................ 52
Table 10. k_h and P_{ult} Values for I-40 Predictions – Dilatometer ............. 52
Table 11. P_{ult} Values for I-40 Long Shaft Predictions – Dilatometer-Reduced GSI .............................. 53
Table 12. k_h Values for I-40 Short Shaft Predictions – Geologic Based-Reduced GSI .............................. 54
Table 13. I-85 Test Site Core Log .......................................................... 70
Table 14. I-85 Laboratory Test Results (Parish, 2001) ............................... 71
Table 15. I-85 Rock Dilatometer Results – k_{ho} Values .............................. 72
Table 16. Parameters for I-85 Performance Predictions – Dilatometer and Geologic Based .............................. 74
Table 17. k_h and P_{ult} Values for I-85 Load Test Predictions – Dilatometer ............ 74
Table 18. k_h Values for I-85 Load Test Predictions – Geologic Based ......... 75
Table 19. GSI Values for the Verification Load Tests ............................... 96
Table 20. LTBASE Input File Format .................................................... 101
LIST OF FIGURES

Figure 1. Comparison between Predicted and Measured Responses (Gabr, 1993) ............ 5
Figure 2. Sketch of P-y Curve for Rock (Reese, 1997) ...................................................... 6
Figure 3. Test Pile Set-up and Surcharging System (Cho, 2002) ....................................... 9
Figure 4. Typical Moment Curvature (Cho, 2002) ............................................................. 9
Figure 5. P-y Curves for Simulated Weathered Rock without Surcharge (Cho, 2002) .... 10
Figure 6. P-y Curves for Simulated Weathered Rock with Surcharge (Cho, 2002) ....... 10
Figure 7. Transformed Hyperbolic Curve (Cho, 2002) .................................................. 11
Figure 8. Curve Fitting Results for Laboratory Tests, No Surcharge (Cho, 2002) ........... 12
Figure 9. Test Shaft Layout (Cho, 2002) ......................................................................... 14
Figure 10. Strain Gage and Inclinometer Casing (Cho, 2002) ....................................... 15
Figure 11. Top Displacements of the Short and Long Shaft Measured from Dial Gages – Nash County (Cho, 2002) ............................................................................. 16
Figure 12. Deflection Profile from Slope Inclinometer Readings – Wilson County Long Shaft (Cho, 2002) ........................................................................................................... 16
Figure 13. Components of the Rock Dilatometer (Cho, 2002) ........................................ 19
Figure 14. Typical Family of Pressure vs. Volume Curves from Rock Dilatometer Testing (Cho, 2002) ............................................................................................................ 20
Figure 15. k_ho Evaluated from Laboratory Tests (Cho, 2002) ........................................ 22
Figure 16. k_ho Evaluated from Field and Laboratory Tests (Cho, 2002) ....................... 23
Figure 17. Elevation View of Laterally Loaded Drilled Shaft in Weathered Rock, Cho (2002) ......................................................................................................................... 24
Figure 18. Geotechnical Strength Index (Hoek and Brown, 1997) ................................. 26
Figure 19. Point of Rotation vs. K_R (Cho, 2002) ............................................................. 28
Figure 20. K_h Number for Depths below the Point of Rotation, I_T (After Cho, 2002) .... 29
Figure 21. (a) Components of Rock Mass Resistance, (b) Calculation of Normal Limit Stress, p_L (Zhang et al., 2000) .................................................................................. 32
Figure 22. Comparison between Measured and Estimated P_ult (Cho, 2002) .............. 35
Figure 23. Verification of P-y Curve Model – Caldwell County Short Shaft (Cho, 2002) ................................................................. 36
Figure 24. Verification of P-y Curve Model – Wilson County Long Shaft (Cho, 2002) ................................................................. 36
Figure 25. Drilling a Test Shaft – I-85 Site ................................................................................................................................. 39
Figure 26. Looking from the Hydraulic Jack, East to the Long Shaft – I-40 Load Test ....................................................... 40
Figure 27. Instrumented Reinforcement Cage .......................................................................................................................... 41
Figure 28. Local Area Map of the I-40 Test Site .......................................................................................................................... 42
Figure 29. Exposed Rock at the Elevation of the Test Pad ........................................................................................................... 43
Figure 30. I-40 Test Site Subsurface Profile .............................................................................................................................. 44
Figure 31. Rock Dilatometer Test Results – I-40 Test Site SB-1 ................................................................................................. 47
Figure 32. Rock Dilatometer Test Results – I-40 Test Site SB-2 ................................................................................................. 48
Figure 33. Example of P-y Curve Distribution Used – I-40 Short Shaft Shown ........................................................................... 51
Figure 34. I-40 Short Shaft Performance Predictions .................................................................................................................. 55
Figure 35. I-40 Long Shaft Performance Predictions .................................................................................................................. 56
Figure 36. Top Deflections of the I-40 Short and Long Shaft Measured from Dial Gages ................................................................. 57
Figure 37. Deflection Profiles after Dial Gage Adjustment – I-40 Short Shaft ........................................................................... 58
Figure 38. Deflection Profiles after Dial Gage Adjustment – I-40 Long Shaft ............................................................................ 58
Figure 39. I-40 Short Shaft Pile Head Deflection Performance ................................................................................................. 60
Figure 40. I-40 Long Shaft Pile Head Deflection Performance ................................................................................................. 60
Figure 41. Back Calculated P-y Curves for the Weathered Rock – I-40 Short Shaft ...................................................................... 61
Figure 42. Back Calculated P-y Curves for the Weathered Rock – I-40 Long Shaft ...................................................................... 62
Figure 43. Curve Fitting Results – I-40 Short Shaft ........................................................................................................................ 62
Figure 44. Curve Fitting Results – I-40 Long Shaft ........................................................................................................................ 63
Figure 45. Predicted and Back Calculated P-y Curves – I-40 Short Shaft Layer 1 ........................................................................ 64
Figure 46. Predicted and Back Calculated P-y Curves – I-40 Short Shaft Layer 3 ........................................................................ 64
Figure 47. Predicted and Back Calculated P-y Curves – I-40 Long Shaft Layer 1 ........................................................................ 65
Figure 48. Predicted and Back Calculated P-y Curves – I-40 Long Shaft Layer 2 ........................................................................ 65
Figure 49. Predicted and Back Calculated P-y Curves – I-40 Long Shaft Layer 3 ........................................................................ 66
Figure 50. Predicted and Back Calculated P-y Curves – I-40 Long Shaft Layer 4 ........................................................................ 66
Figure 51. Local Area Map of the I-85 Test Site ........................................................................ 67
Figure 52. Exposed Rock Profile at the Elevation of the Test Pad ........................................... 68
Figure 53. I-85 Test Site Subsurface Profile .............................................................................. 68
Figure 54. Rock Dilatometer Test Results – I-85 Test Site B1-Dur .................................................... 71
Figure 55. Rock Dilatometer Test Results – I-85 Test Site B2-Dur .................................................... 72
Figure 56. I-85 Short Shaft Performance Predictions ..................................................................... 76
Figure 57. I-85 Long Shaft Performance Predictions ..................................................................... 76
Figure 58. Top Displacements of the Short and Long Shaft Measured from Dial Gages ........................ 77
Figure 59. Deflection Profiles after Dial Gage Adjustment – I-85 Short Shaft .................................. 78
Figure 60. Deflection Profiles after Dial Gage Adjustment – I-85 Long Shaft .................................. 79
Figure 61. I-85 Short Shaft Pile Head Deflection Performance ..................................................... 80
Figure 62. I-85 Long Shaft Pile Head Deflection Performance ..................................................... 81
Figure 63. Back Calculated P-y Curves for the Weathered Rock – I-85 Short Shaft ........................... 82
Figure 64. Back Calculated P-y Curves for the Weathered Rock – I-85 Long Shaft .......................... 83
Figure 65. Curve Fitting Results – I-85 Short Shaft ..................................................................... 83
Figure 66. Curve Fitting Results – I-85 Long Shaft ..................................................................... 84
Figure 67. Predicted and Back Calculated P-y Curves – I-85 Short Shaft Layer 1 ......................... 85
Figure 68. Predicted and Back Calculated P-y Curves – I-85 Short Shaft Layer 2 ......................... 85
Figure 69. Predicted and Back Calculated P-y Curves – I-85 Short Shaft Layer 3 ......................... 86
Figure 70. Predicted and Back Calculated P-y Curves – I-85 Long Shaft Layer 1 ............................ 86
Figure 71. Predicted and Back Calculated P-y Curves – I-85 Long Shaft Layer 2 ............................ 87
Figure 72. Predicted and Back Calculated P-y Curves – I-85 Long Shaft Layer 3 ............................ 87
Figure 73. Predicted and Back Calculated P-y Curves – I-85 Long Shaft Layer 4 ............................ 88
Figure 74. Measured k_{ho} Values from Field Tests (Cho, 2002) ...................................................... 89
Figure 75. Measured k_{ho} from Verification Tests ........................................................................... 89
Figure 76. K_{h} Number for Depths below the Point of Rotation (I_T) – for Triassic Weathered Rock ................................................................................................................................. 90
Figure 77. GSI Reduction Factor, \( \alpha_{\text{GSI}} \), for Triassic Weathered Rock ........................................ 97
Figure 78. I-40 Short Shaft Pile Head Deflections with Recommendations ...................................... 98
Figure 79. I-40 Long Shaft Pile Head Deflections with Recommendations ...................................... 98
Figure 80. I-85 Short Shaft Pile Head Deflections with Recommendations .................. 99
Figure 81. I-85 Long Shaft Pile Head Deflections with Recommendations .................. 99
CHAPTER 1. INTRODUCTION

As today’s structures become larger, the demand placed on foundation materials grows accordingly until our depth of understanding the nature and mechanics of geotechnical materials become limiting factors. In areas of weathered and decomposed rock profiles, such as that of the Piedmont Physiographic Province of the Southeastern United States, definition of the soil-rock boundary is a recurring challenge for engineers and contractors. These subsurface profiles generally consist of surface residual soils derived from extensive weathering of the parent rock; with depth, soil grades into less-weathered material to a point where more evidence of the parent rock is realized. Eventually, no signs of weathering are encountered and fresh, competent rock is significant. One major difficulty that faces Geotechnical Engineers is not associated with either soil or competent rock, but lies in the transition between the two. It is within this transitional zone that joints systems become erratic to the point that quantifying their effect on behavior is extremely difficult, where water and release of stress significantly effect engineering properties, and over conservative designs are born from a deficiency understanding of engineering behavior. This zone of transition has been the subject of attempts to quantitatively define its existence in nature. Coates (1970) recommended that the Rock Quality Designation (RQD) be used to evaluate depth to sound rock. Peck (1976) stated that the distinction between rock-like and soil-like material in transitional zones is usually unpredictable.

As deep foundations extend deeper into subsurface profiles the need to quantify the engineering properties of transitional materials, especially as related to lateral loading, is necessary to control the conservative degree of designs. Also, consistent and conservative analysis models are necessary to evaluate the behavior of transitional materials when subjected to lateral loads. Lateral loads are imposed on deep foundations in a variety of manners, earth pressures, earthquakes, wind, and vehicular forces. Several researchers have published literature related to lateral loading of transitional materials, most notably Reese (1997). Models originally created for other materials have been used for designs involving transitional materials (Reese, Cox, and Koop, 1975). However,
again due to uncertainty, many government agencies specify overly conservative engineering properties for use with these design models.

A new addition to literature, published by Cho (2002), was the result of extensive research into the development of a P-y curve model for laterally loaded drilled shafts embedded in weathered rock (Weathered Rock Model). The Weathered Rock Model is the focus the verification testing presented in this thesis.

1.1 Problem Statement

In order to investigate the validity of the Weathered Rock Model, it must be the focus of tests independent of those used for development. Class A performance predictions should be used to estimate the behavior of full-scale drilled shafts in the field. Predictions should be determined using currently excepted design methods with engineering properties standardized by the North Carolina Department of Transportation (agency sponsoring research). Results should be presented to the conclusion of the most accurate method available for representing weathered materials.

1.2 Objectives

The main objective of this research is to independently investigate the validity of the Weathered Rock Model presented by Cho (2002). Class A performance predictions are compared with the results of four full-scale load tests performed within weathered profiles. These results are used to recommended design procedures, along with logical design parameters. The following objectives are pursued in this research:

1. Independently compare the results of field load tests with predictions made using several design methodologies.
2. Recommend conservative, yet accurate design procedures associated with the Weathered Rock Model.
3. Investigate the behavior of Triassic Weathered Rock under lateral loading.
4. Include the Weathered Rock Model in the computer program LTBASE (Borden and Gabr, 1987) for the analysis of laterally loaded drilled shafts in weathered rock profiles.
CHAPTER 2. LITERATURE REVIEW

The issue of predicting the behavior of laterally loaded drilled shafts embedded in weathered rock profiles has been a problem that has faced geotechnical engineers for many years. The difficulty lies in being able to capture the complexity of weathered rock in relatively simple, economical, and reasonably conservative design models. The P-y curve approach is considered by many engineers to be the most useful method for the analysis of laterally loaded piles and drilled shafts. One popular approach was the result of research performed by Reese, Cox, and Koop (1975); it is based on the assumption that P-y curves for stiff clay above the groundwater table can represent weathered rock. Gabr (1993) showed that the behavior of piers embedded in rock could be predicted using the stiff clay criterion. This design procedure is widely used by many agencies today with standardized engineering properties assumed to represent weathered rock. Reese (1997) published a method for the construction of P-y curves for weak rock. This design procedure was based on two load tests of piles embedded in a weathered rock profile. Reese termed the procedure “interim” due to the amount of test data on which it was based. Cho (2002) proposed a new method, the Weathered Rock Model, for the development of P-y curves for laterally loaded drilled shafts embedded in weathered rock. This method is based on research sponsored by the North Carolina Department of Transportation and included two laboratory load tests, finite element method modeling, and six full-scale field load tests. Verification of the Weathered Rock Model is the subject of this report.

2.1 P-y Curves based on the Stiff Clay Model

One popular method for developing P-y curves in weathered rock was taken from research published by Reese, Cox, and Koop (1975). With this method, the user idealizes weathered rock as stiff clay above the groundwater table. Reese et al. (1975) proposed the following equation for generating P-y curves for stiff clay material:
Using Equation 1, Gabr (1993) compared the measured and predicted behavior of a set of test piers embedded in rock. In order to parametrically study the effect of P-y magnitude on predicted behavior, $y_{50}$ was assumed to be $\varepsilon_{50}B$ as opposed to $2.5\varepsilon_{50}B$. With $y_{50} = \varepsilon_{50}B$, a stiffer P-y response and shorter initial slope was realized. This also allowed for a better representation of the non-linear behavior during early stages of loading, as shown in Figure 1. Results showed that the lateral response of the test piers was better represented using the Stiff Clay Model ($y_{50} = \varepsilon_{50}B$) as compared with the elastic theory.

The stiff clay criteria is commonly used for the design of laterally loaded drilled shafts in weathered rock profiles and is included in many computer design programs. The North Carolina Department of Transportation specifies the following engineering properties when analyzing laterally loaded drilled shafts embedded in weathered rock with the Stiff Clay Model: Cohesion = 200 kPa, $k = 543,000 \text{kN/m}^3$, $\varepsilon_{50} = 0.004$.

### 2.2 P-y Curves for Weak Rock

Reese (1997) proposed a method for the estimation of P-y curves in weathered rock profiles. This method was developed from two load tests of piles embedded in weathered rock and was termed “interim” due to the amount of data available for development.

The ultimate resistance, $p_{ur}$, was developed based on limit equilibrium and takes into account depth of rock:

\[
\frac{P}{P_{ur}} = \left( \frac{y}{16y_{50}} \right)^{0.25} \tag{1}
\]

\[
\frac{b_{qp}}{b_{ur}} = 2.5 \quad \text{if} \quad 0 \leq x_r \leq 3b \tag{2}
\]

\[
\frac{b_{qp}}{b_{ur}} = 4.11 \quad \text{if} \quad x_r \geq 3b \tag{3}
\]
Where, \( q_{ur} \) = compressive strength of the rock (usually the lower-bound as a function of depth),
\( \alpha_r \) = strength reduction factor,
\( b \) = width, or diameter of the pile,
\( x_r \) = depth below the rock surface.

Assuming that a pile were a beam resting against an elastic, homogeneous, and isotropic material, the initial modulus \( K_{ir} \) may be calculated from the following equation:

\[
K_{ir} = k_{ir}E_{ir}
\]  \hspace{1cm} (4)
Where, $k_{ir} = \text{dimensionless constant at a given depth below the rock surface},$

$E_{ir} = \text{initial modulus of the rock}.$

Reese (1997) suggested that the dimensionless constant, $k_{ir}$, varied with depth and reflected the assumption that the presence of the rock surface would have a similar effect on $k_{ir}$, as for $p_{ur}$:

$$k_{ir} = 100 + \frac{400x_r}{3b} \quad 0 \leq x_r \leq 3b \quad (5)$$

$$k_{ir} = 500 \quad x_r \geq 3b \quad (6)$$

Equations 5 and 6 were developed to represent the very stiff initial portions of P-y curves in order to model low deflections observed during initial loadings.

Reese divided the P-y curve into three sections as shown in Figure 2. Equation 4 represents the initial section from $y = 0$ to $y_A$; remaining sections are calculated using Equations 7 through 9.

![Figure 2. Sketch of P-y Curve for Rock (Reese, 1997)](image-url)
\[ p = K_{ir} y; \quad y \leq y_A \]  
\[ p = \frac{p_{ur}}{2} \left( \frac{y}{y_{rm}} \right)^{0.25} \quad y \geq y_A; \quad p \leq p_{ur} \]  
\[ y_{rm} = k_{rm} b \]

Where, \( k_{rm} = \) constant, ranging from 0.0005 to 0.00005, and establishes the overall stiffness of the curves.

The value of \( y_A \) can be determined by solving for the intersection of Equations 7 and 8, and is defined by the following equation:

\[ y_A = \left[ \frac{p_{ur}}{2 (y_{rm})^{0.25} K_{ir}} \right]^{1.333} \]  

Reese (1997) commented about these equations. “First, the equations have no influence on solutions beyond the value of \( y_A \) (Figure 2) and probably will have no influence on the designs based on the ultimate bending moment of a pile. Second, available theory, while incomplete, shows much lower values of \( K_{ir} \) in relation to the modulus of rock or soil. Third, the increase in \( K_{ir} \) with depth is consistent with results obtained from the lateral loading of piles in overconsolidated clays.”

### 2.3 The Weathered Rock Model

Cho (2002) published the results of an extensive research program, funded by the North Carolina Department of Transportation and carried out by North Carolina State University, that established a new P-y curve model for laterally loaded drilled shafts embedded in weathered rock. The research program consisted of three complementary approaches, Finite Element Method modeling using the computer program ABAQUS for three-dimensional analytical modeling of a wider range of scenarios than can be accounted for in laboratory and field testing. The second involved laboratory testing to model the characteristics of P-y curves in simulated weathered rock material; third, was field testing of full-scale drilled shafts to develop and verify P-y curves in weathered rock.
rock. Results from all three components were used to develop the concepts of the Weathered Rock Model.

2.3.1 Laboratory Testing (Clark, 2001)

Laboratory tests were used to develop the appropriate shape of P-y curves in weathered rock. Since, large bulk samples of weathered rock could not be brought to the laboratory for use in testing, an alternate medium was developed to simulate weathered rock. Aggregate base course (ABC) was chosen to represent the material in question. ABC is a mixture of large gravel sized particles, sand size particles and smaller fines. One question that had to be addressed was what percentage of fines and gravel are needed to produce the appropriate testing material. Based on a large quantity of subsurface data gathered for the research program, it was noticed that on average 70% of core runs in weathered rock were recoverable. It was assumed that the other 30% of material were fines that washed away during the drilling process. Therefore a laboratory testing medium consisting of 70% large gravel sized particles and 30% fines was constructed.

Two 0.09 m diameter test piles were constructed and placed in the simulated weathered rock material. The test piles were instrumented with strain gages and surface deflections were monitored with dial gages. The area surrounding one test pile was subjected to 24 kPa of surcharge to simulate depth, as seen in Figure 3.

As described later in this chapter, measured strain data can be used to calculate the distribution of moment with depth, Figure 4. Through integration and differentiation of the moment curves obtained from data gathered at each load increment, values of P and y for any given depth were determined, Figures 5 and 6.
Figure 3. Test Pile Set-up and Surcharging System (Cho, 2002)

Figure 4. Typical Moment Curvature (Cho, 2002)
Figure 5. P-y Curves for Simulated Weathered Rock without Surcharge (Cho, 2002)

Figure 6. P-y Curves for Simulated Weathered Rock with Surcharge (Cho, 2002)
One hypothesis proposed by Cho (2002) was that the hyperbolic function could adequately model P-y curves for weathered rock. The hyperbolic function allows for the input of two material properties, for P-y curves the lateral subgrade reaction and the lateral ultimate resistance are used. The hyperbolic function as a P-y curve is given by the following equation:

\[ P(y) = \frac{y}{a + by} \]  

(11)

Where, \(a = \frac{1}{k_h}\),

\(k_h = \) initial tangent modulus (subgrade reaction),

\(b = \frac{1}{P_{ult}}\),

\(P_{ult} = \) ultimate resistance.

Transformed axes plots are used to determine if measured data can be represented by the hyperbolic function, as presented by Kondner and Zelasko (1963). By plotting \(y\) as the abscissa and \(y/P\) as the ordinate, the hyperbolic function is transformed to a linear function with y-intercept \(a, (1/k_h)\), and slope \(b, (1/P_{ult})\) as shown in Figure 7.

Figure 7. Transformed Hyperbolic Curve (Cho, 2002)
Figure 8 is a transformed axes plot of P-y data from laboratory testing. This figure supports the hypothesis proposed by Cho (2002), that the hyperbolic function can adequately represent P-y curves for weathered rock.

![Figure 8. Curve Fitting Results for Laboratory Tests, No Surcharge (Cho, 2002)](image)

### 2.3.2 Field Testing

Three test sites in North Carolina were chosen for field testing. These sites contained significant depths of weathered rock close to the ground surface. Table 1 lists the rock types encountered at each test site. The field testing was comprised of two components, first the measurement of load and deflection responses of full-scale laterally loaded drilled shafts embedded in weathered rock. From measured responses, P-y curves were back calculated. The second component was investigating the validity of using the rock dilatometer to obtain in-situ rock mass modulus values. The rock dilatometer is an adaptation of the pressuremeter to higher ranges of pressure for testing very stiff materials.

Two full-scale lateral load tests were performed at each of the three test sites in Nash, Caldwell, and Wilson Counties. Testing set-up was similar for each site as shown in Figure 9. Test shafts were constructed approximately 7.62 m apart, using conventional
earth augers with permanent steel casing “screwed in” to design tip elevations. Permanent casing was used to increase the stiffness of the test shafts so that significant deflections could be realized without failure of the shaft. Vertical steel reinforcement, in addition to the permanent casing, was included in each test shaft to increase flexural strength and allow for the attachment of strain gages. A loading frame constructed by the North Carolina Department of Transportation was used to transfer load from a centrally located hydraulic jack to each test shaft. The load frame was attached to a constructed test shaft approximately 0.3 m above the test pad elevation.

Vibrating wire strain gages were used to measure vertical strain with depth. The strain gages were mounted to 1m long sister bars that attached to the tension side of the reinforcement cage as shown in Figure 10. Plastic housing for the insertion of slope inclinometers was attached to the reinforcement cage 180° from the strain gages. The slope inclinometers allowed for measurement of shaft deflection with depth. At the surface, dial gages were used to monitor deflection in and out of the plane of loading. Dial gages were also placed on the top of each shaft to measure head rotation.

Table 1. List of Test Sites and Rock Types (Cho, 2002)

<table>
<thead>
<tr>
<th>Test Site</th>
<th>Rock Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nash-Halifax County</td>
<td>Sandstone</td>
</tr>
<tr>
<td>Caldwell County</td>
<td>Mica Schist</td>
</tr>
<tr>
<td>Wilson County</td>
<td>Crystalline Rock</td>
</tr>
</tbody>
</table>
Figure 9. Test Shaft Layout (Cho, 2002)
2.3.2.1 P-y Curves from Measured Strain Data

Cho (2002) used the following equation relating measured normal strains to shaft curvature:

\[ \varepsilon_y = -\frac{y}{\rho} = -\kappa y \]  

(12)

Where, \( y \) = distance to neutral axis,
\( \rho \) = radius of curvature, and,
\( \kappa \) = curvature of the beam.
Figure 11. Top Displacements of the Short and Long Shaft Measured from Dial Gages – Nash County (Cho, 2002)

Figure 12. Deflection Profile from Slope Inclinometer Readings – Wilson County Long Shaft (Cho, 2002)
Then assuming the shaft material was linear elastic over the loading range, Hooke’s Law for uniaxial stress was substituted into Equation 12 to obtain the following:

\[ \sigma_x = E \varepsilon_x = -\frac{E_y}{\rho} = -E \kappa y \]  \hspace{1cm} (13)

Where, \( \sigma \) = stress along the x-axis, and,
\( E \) = Young’s Modulus of the material.

Cho (2002) noted that this equation indicates the normal stresses acting along the shaft cross section vary linearly as the distance (y) from the neutral axis changes. For a circular cross section the neutral axis is located along the centerline. Assuming the moment caused by the normal stresses acts over the entire cross section, it can be estimated from the following equation:

\[ M_o = -\int \sigma_x y dA \]  \hspace{1cm} (14)

The following was used to determine the bending moment of the test shafts:

\[ M = EI \phi = EI \frac{d^2 y}{dz^2} \]  \hspace{1cm} (15)

Where, \( M \) = bending moment at depth, \( z \),
\( E \) = modulus of elasticity of the pile,
\( I \) = moment of inertia of the pile around the centroidal axis of the pile,
\( \phi \) = pile curvature,
\( y \) = pile lateral displacement, and,
\( z \) = depth.

Noting that \( -M_o \) from equation 14 is the bending moment, \( M \), and substituting for \( \sigma_x \) from equation 14, the bending moment can be expressed as follows:

\[ M = -\kappa E I \]  \hspace{1cm} (16)

Cho (2002) rearranges equation 16 to the form given:
\[ \kappa = \frac{1}{\rho} = \frac{M}{EI} \]  \hspace{1cm} (17)

Equation 17 is known as the moment-curvature equation and shows that the curvature of a shaft is directly proportional to the bending moment and inversely proportional to the flexural rigidity, EI (Cho, 2002).

Measured strain data from field load tests were used to develop moment curves with depth for each loading increment. A typical moment curve was presented in Figure 4. A fourth order regression curve was used to evaluate the function of the moment distribution with depth,

\[ y(x) = a + bx + cx^2 + dx^3 + ex^4 \]  \hspace{1cm} (18)

Where, a, b, c, d, and e = the coefficients of the regression line, and,

\( y = \) pile segment length (m).

Soil resistance, P (kN/m), with depth was evaluated by differentiating Equation 18 with respect to depth. When integrated, the moment function yields shaft deflection, \( y \) (m), with depth.

2.3.2.2 Rock Dilatometer Testing

One of the most difficult properties to estimate for the design of laterally loaded drilled shafts in weathered rock is the subgrade reaction, \( k_h \). Due to the decomposed nature of weathered rock intact, undisturbed samples are nearly impossible to retrieve. Even if samples were available, laboratory measurement of the lateral modulus without substantial disturbance would be extremely difficult and costly. For these reasons, a means of insitu measurement is necessary.

The rock dilatometer (model Probex 1, manufactured by ROCTEST, Plattsburgh, NY) is a specialized probe that utilizes an expandable bladder to apply pressure to the sidewall of a N-size borehole. The maximum working pressure that can be applied, according to the manufactures literature, is 30,020 kPa. Volume change in the probe is
measured at the probe level under incremental changes in pressure. The lateral rock modulus can be calculated from pressure and volume measurements.

The rock dilatometer exerts pressure against a borehole by means of an expandable bladder. A digital readout box is used to monitor volume change of the bladder as pressure is incrementally increased. Figure 13 is a schematic of the rock dilatometer. Figure 14 is a typical family of pressure-volume curves obtained from rock dilatometer testing.

Figure 13. Components of the Rock Dilatometer (Cho, 2002)
Figure 14. Typical Family of Pressure vs. Volume Curves from Rock Dilatometer Testing (Cho, 2002)

Lama (1852) expressed the radial expansion of an internally pressurized cylindrical cavity made of an infinitely elastic medium with the following equation:

\[
G = V \left( \frac{\Delta p}{\Delta v} \right)
\]  

(19)

Where, \( G \) = elastic shear modulus,

\( V \) = volume of the cavity, and

\( p \) = pressure in the cavity.

The ratio \( \Delta p/\Delta v \) represents the slope of the pressure vs. volume curve. The modulus is determined over the linear region of the test data. The shear modulus is converted into the elastic modulus using the following well-known relationship:

\[
G = \frac{E_r}{2(1 + \nu_r)}
\]  

(20)

Where, \( E_r \) = elastic modulus of the rock, and
\( \nu_r = \text{Poisson’s ratio of the rock.} \)

Equations 19 and 20 are combined and solved for the insitu rock modulus.

\[
E_r = 2(1 + \nu_r)\nu_m \left( \frac{\Delta p}{\Delta v} \right) \tag{21}
\]

\( V_m \) is the volume of the cavity at the midpoint of the range over which the modulus is determined.

\[
V_m = v_o + v_m \tag{22}
\]

Where, \( v_o \) = volume of the deflated probe, for the rock dilatometer used in this experimental program \( v_o = 1950 \text{ cc} \),

\( v_m \) = mean additional volume injected into the probe from the initial state, up to the midpoint of the selected pressure range.

The values of \( \Delta p \) and \( \Delta v \) are corrected for the pressure required to overcome the inertia of the bladder and volume changes from the intrinsic dilation of the system. When corrections and Equation 22 are applied to Equation 21, the resulting is given:

\[
E_r = 2(1 + \nu_r)(v_o + v_m) \left( \frac{1}{\frac{\Delta v}{\Delta p - \Delta p_i} - c} \right) \tag{23}
\]

Where, \( \Delta p_i \) = the change in pressure of the dilatable membrane corresponding to the applied pressure increment \( \Delta p \), and

\( c \) = volume correction factor determined from a calibration procedure, the value of \( c \) for the rock dilatometer used in this experimental program was \( 7.878 \times 10^{-4} \).

### 2.3.3 Concepts of the Weathered Rock Model

Based on the results of extensive research, Cho (2002) proposed the following concepts for development of P-y curves for weathered rock. For the model, P-y curves are constructed using the hyperbolic function. As previously discussed, the hyperbolic
function allows the input of two material properties, \( k_h \) (subgrade reaction) and \( P_{ult} \) (the lateral ultimate resistance). Cho (2002) proposed two methods for evaluating \( k_h \), one based on empirical equations using geologic parameters and another based on results of rock dilatometer testing. An independent relationship for the ultimate lateral resistance was adopted from research published by Zhang et al. (2002).

### 2.3.3.1 Subgrade Reaction (\( k_h \)) for Weathered Rock

The subgrade reaction can be evaluated from measured P-y curves by two methods. The first involves evaluating the y-intercept (“a” parameter) of a transformed hyperbolic plot; the other is calculating the initial tangent modulus of a back-calculated P-y curve. The coefficient of subgrade reaction, \( k_{ho} \), is determined by dividing \( k_h \) by the diameter of the shaft. When \( k_{ho} \) values for both laboratory tests were evaluated and plotted with depth, it became evident that the increase with depth was not linear, but followed more of an exponential growth function, Figure 15. Similar results were also realized when \( k_{ho} \) from field tests were plotted along with the laboratory test results, as shown in Figure 16.

![Figure 15. \( k_{ho} \) Evaluated from Laboratory Tests (Cho, 2002)](image-url)
Figure 16. $k_{h_0}$ Evaluated from Field and Laboratory Tests (Cho, 2002)

The point of rotation for each shaft marked the distinct point where the value of $k_h$ increased dramatically. Cho (2002) postulated that in order to obtain a correct representation of field behavior, $k_h$ should be evaluated for two separate regions, above and below the point of rotation. Figure 17 is an elevation view of a laterally loaded drilled shaft in weathered rock. The volume of soil supplying lateral resistance above the point of rotation is relatively small as compared to that below the point of rotation. Also, the soil behind the pile and below the point of rotation is subjected too much higher overburden pressures; therefore, all else being equal, the value of $k_h$ below the point of rotation must be greater than that above the point of rotation.
The deflection of a shaft can be generalized from the following equation (Prakash, 1990):

\[ y = f(x, T, L, k_h, EI, Q, M) \]  

(24)

Where, \( x \) = depth of embedment,

\[ T = \text{relative stiffness factor} = \left( \frac{EI}{n_h} \right)^{1/5} \]

\( L \) = pile length,

\( k_h \) = lateral subgrade modulus,

\( n_h \) = constant of subgrade modulus,

\( EI \) = pile stiffness,

\( Q \) = lateral load applied to the pile head, and

\( M \) = moment applied at the pile head.
Poulos and Davis (1980) presented a flexibility factor \(K_R\) that describes the relative stiffness of a shaft with respect to the soil:

\[
K_R = \frac{E_p I_p}{E_s L^4}
\]

Where, \(E_p\) = elastic modulus of the pile, 
\(I_p\) = moment of inertia of the pile, 
\(E_s\) = modulus of elasticity of the soil or weathered rock, and 
\(L\) = embedded length of the shaft.

Cho (2002) stated that the factors that have the largest effect on the position of the point of rotation are the relative stiffness between the shaft and weathered rock, and the depth of embedment. Hoek and Brown (1997) presented the following equation for calculating the modulus of elasticity for weathered rock:

\[
E_s(GPa) = \left( \frac{\sigma_{ci}}{100} \right)^{GSI-10} 10^{\frac{GSI-10}{40}}
\]

Where, \(\sigma_{ci}\) = unconfined compressive strength of weathered rock (GPa), and 
\(GSI = \) Geotechnical Strength Index.

Hoek (1994) developed The Geotechnical Strength Index as a means of quantifying parameters that effect rock strength. There are two methods for evaluating the GSI value of a material. Hoek and Brown (1997) presented Figure 18 for the calculation of GSI based on the insitu rock structure and the quality of joint surfaces.
For better quality rock (GSI > 25), Figure 18 lacks a desirable degree of precision. For this quality of material, it was recommended that the Rock Mass Rating (RMR), (Bieniawski, 1976) be used to evaluate GSI with the ground water rating set to 10 (dry) and the adjustment for joint orientation set to 0. Table 2 is presented for the determination of GSI for better quality rock. For poorer quality rock (GSI < 25), the
value of RMR is difficult to evaluate and balance between the different rating systems no longer gives a reliable basis for estimating rock mass strength. Therefore for poorer quality rock, it would be better to estimate GSI using Figure 18 (Hoek and Brown, 1997).

Table 2. Rock Mass Rating (RMR) Method (Bieniawski, 1976)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Range of Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Point Load Strength Index</td>
<td>Strength of Intact Rock Material</td>
</tr>
<tr>
<td>Rated Strength Index</td>
<td>&lt; 8 MPa</td>
</tr>
<tr>
<td>&lt; 200 MPa</td>
<td>&gt; 200 MPa</td>
</tr>
<tr>
<td>Rating</td>
<td>15</td>
</tr>
<tr>
<td>R.Q.D.</td>
<td>90-100 %</td>
</tr>
<tr>
<td>Rating</td>
<td>20</td>
</tr>
<tr>
<td>Spacing of Joints</td>
<td>&gt;3 m</td>
</tr>
<tr>
<td>Rating</td>
<td>30</td>
</tr>
<tr>
<td>Condition of Joints</td>
<td>Very rough surfaces, Not continuous, No Separation, Hard joint wall rock</td>
</tr>
<tr>
<td>Rating</td>
<td>25</td>
</tr>
<tr>
<td>Inflow per 10 m tunnel length</td>
<td>None or</td>
</tr>
<tr>
<td>General Conditions</td>
<td>Completely dry</td>
</tr>
<tr>
<td>Rating</td>
<td>10</td>
</tr>
</tbody>
</table>

Cho (2002) combined data from the laboratory, field testing and F.E.M. modeling to establish a relationship between the point of rotation and $K_R$. 

27
Based on Figure 19, Cho (2002) established Equation 27 for the calculation of depth to the point of rotation:

\[
T_o = L \left( 1 + 0.18 \log K_R \right) \quad (K_R \leq 1) \quad (27)
\]

As was shown previously in Figures 15 and 16, a substantial increase in \( k_h \) below the point of rotation is realized in weathered rock profiles. Again, based on data from field testing and F.E.M. modeling, Figure 20 was developed in order to establish the relationship between the increase in \( k_h \) below the point of rotation (\( k_h \) Number, \( I_T \)) and the ratio \( T_o/L \).
Figure 20. $K_h$ Number for Depths below the Point of Rotation, $I_T$ (After Cho, 2002)

From the data shown in Figure 20, the following equation for the $K_h$ number for depths below the point of rotation ($I_T$) is presented:

$$I_T = -28 - 383 \log \left( \frac{T_0}{L} \right)$$  \hspace{1cm} (28)

### 2.3.3.2 Determination of the Subgrade Reaction ($k_h$) for Weathered Rock from Empirical Equations and Geologic Parameters

One of the methods proposed by Cho (2002) for the estimation of $k_h$ is based on a set of empirical equations that utilizes geologic parameters. The following equation was presented for the calculation of the coefficient of subgrade reaction, $k_{ho}$:

$$k_{ho} (kN/m^3) = \sqrt{\sigma_{ci}} \times 10^3 \left( \frac{GSI-10}{100} \right)$$  \hspace{1cm} (29)
Where, $\sigma_{ci}$ = unconfined compressive strength of intact rock sample (kPa).

The distribution of $k_{ho}$ with depth is a function of the initial value of $k_{ho}$ and the characteristic of shaft deflection. Cho (2002) presented the following equation for the distribution of $k_h$ with depth, $n_h$:

$$
n_h (kN / m^4) = \left( \frac{2E_p I_p}{k_{ho} L^4} \right) \times 10^5
$$

(30)

Where, $k_{ho} =$ coefficient of subgrade modulus for weathered rock at the surface (kN/m$^3$), and

$L = \text{length of shaft.}$

Once the values of Equations 28 through 30 have been established, $k_h$ above and below the point of rotation can be calculated using the following equations, Cho (2002):

$$
k_h = (k_{ho} + n_h z)B \quad (0 \leq z \leq T_o)
$$

(31)

$$
k_h = \left( (k_{ho} + n_h T_o) + n_h (z - T_o) \right) I_T B \quad (T_o \leq z \leq L)
$$

(32)

Where, $z = \text{depth (m)}$, $B = \text{pile diameter (m)}$.

2.3.3.3 Determination of the Subgrade Reaction ($k_h$) for Weathered Rock based on Rock Dilatometer Testing

As explained previously, due to the nature of weathered rock it is virtually impossible to obtain quality laboratory modulus data. Cho (2002) proposed using the insitu-testing device, rock dilatometer, for the measurement of the rock mass modulus of elasticity. Adopting Equation 23, presented earlier, Cho (2002) proposed the following for the determination of the coefficient of subgrade reaction:

$$
k_{ho} (kN / m^3) = 2(1 + \nu_r) \times (v_o + v_m) \times \frac{1}{\frac{\Delta v}{\Delta p - \Delta p_i} - c}
$$

(33)
The following equations are given for calculating $k_h$ above and below the point of rotation:

\[
k_h = \left( k_{ho} \right) B \quad (0 \leq z \leq T_o)
\]

\[
k_h = k_{ho} I_T B \quad (T_o \leq z \leq L)
\]

Where, $B =$ pile diameter (m).

2.3.3.4 Determination of the Ultimate Lateral Resistance ($P_{ult}$) for Weathered Rock

The other parameter needed for P-y curves, using the hyperbolic function, is the ultimate lateral resistance ($P_{ult}$). Due to the capacity of the loading frame used in field testing and the strength of the material, there was very little data gathered in the yielding range of weathered rock. $P_{ult}$ values can be calculated from the inverse of the “$b$” parameter discussed previously. However, large errors are introduced when evaluating these parameters from tests where small values of deflection are encountered, as in lateral load testing. Cho (2002) adopted a model for the calculation of $P_{ult}$ from research published by Zhang et al. (2000).

Zhang et al. (2000) presented Figure 21 as a means of exhibiting the mechanisms that comprise the lateral resistance of rock. The total reaction of the rock mass consists of two components: the side shear and front normal resistance. From these two the ultimate resistance ($P_{ult}$) can be calculated using the following equation (Briaud and Smith, 1983; Carter and Kulhawy, 1992):

\[
P_{ult} = \left( p_L + \tau_{max} \right) B
\]

Where, $p_L =$ normal limit resistance,

$\tau_{max} =$ maximum shearing resistance along the sides of the shaft, and

$B =$ diameter of the shaft.
For simplicity, Zhang (1999) assumed $\tau_{\text{max}}$ to be the same as the maximum side resistance of an axially loaded shaft:

For a Smooth Socket:

$$\tau_{\text{max}} = 0.2\sqrt{\sigma_{ci}} \quad \text{(MPa)}$$  \hspace{1cm} (37)

For a Rough Socket:

$$\tau_{\text{max}} = 0.8\sqrt{\sigma_{ci}} \quad \text{(MPa)}$$  \hspace{1cm} (38)

Where, $\sigma_{ci} =$ unconfined compressive strength of intact rock sample (MPa).
In order to determine the normal limit resistance, $p_L$, the strength criterion for rock masses developed by Hoek and Brown (1980, 1988) was used. For jointed rock masses, the Hoek-Brown criterion is expressed by the following formula:

$$\sigma_1' = \sigma_3' + \sigma_{ci} \left( m_b \frac{\sigma_3'}{\sigma_{ci}} + S \right)^a$$  \hspace{1cm} (39)

Where, $\sigma_1'$ and $\sigma_3'$ = major and minor principal stresses, respectively,

$\sigma_{ci}$ = compressive strength of the intact rock material,

$m_b$ = value of the constant $m$ for the rock mass, and

$S$ and $a$ = constants that depend on the characteristics of the rock mass.

The constants $m_b$, $S$, and $a$ are determined from Table 3. The value for $m_i$ required for determination of $m_b$ is a function of rock type and can be evaluated from Table 4.

<table>
<thead>
<tr>
<th>Table 3. Parameters $m_b$, $S$, and $a$ (Hoek et al., 1995)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parameter</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>$m_b$</td>
</tr>
<tr>
<td>$S$</td>
</tr>
<tr>
<td>$A$</td>
</tr>
</tbody>
</table>
Table 4. \( m_i \) Value (Hoek and Brown, 1988)

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Class</th>
<th>Group</th>
<th>Texture</th>
<th>Course (22)</th>
<th>Medium (18)</th>
<th>Fine (9)</th>
<th>Very fine (4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SEDIMENTARY</td>
<td>Clastic</td>
<td>Conglomerate</td>
<td></td>
<td>19</td>
<td>9</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sandstone</td>
<td></td>
<td>(18)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(18) Greywacke</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Non-Clastic</td>
<td>Organic</td>
<td>(7)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Carbonate</td>
<td>(8–21)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Breccia (20)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sparitic Limestone (10)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Micritic Limestone (8)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Chemical</td>
<td>(13) Gypsum (16)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Anhydrite (13)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>METAMORPHIC</td>
<td>Non-foliated</td>
<td>Marble (9)</td>
<td>(19) Hornfels</td>
<td>24</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Slightly foliated</td>
<td>Migmatite (30)</td>
<td>Amphibolite (25–31)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Gneiss (33)</td>
<td>Schists (4–8)</td>
<td>(6) Phyllices (10)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Foliated*</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>IGNEOUS</td>
<td>Granite (33)</td>
<td>Rhyolite (16)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Light</td>
<td>Granodiorite (30)</td>
<td>Dacite (17)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Diorite (28)</td>
<td>Andesite (19)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dark</td>
<td>Gabбро (27)</td>
<td>Dolerite (19)</td>
<td>Basalt (17)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Norite (22)</td>
<td>(17)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Extrusive pyroclastic type</td>
<td>Agglomerate (20)</td>
<td>Broccia (18)</td>
<td>Tuff (15)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*These values are for intact rock specimens tested normal to bedding or foliation. The value of \( m_i \) will be significantly different if failure occurs along a weakness plane.

Assuming that the minor principal effective stress, \( \sigma'_3 \), is the effective overburden pressure, \( \gamma'z \), and that the normal limit resistance, \( p_L \), is the major principal effective stress, \( \sigma'_1 \), Hoek and Brown (1988) presented the following:

\[
p_L = \sigma'_1 = \gamma'z + \sigma_{ci} \left( m_h \frac{\gamma'z}{\sigma_{ci}} + S \right)^a
\]

(40)

Where, \( \gamma' \) = effective unit weight of the rock mass, and

\( z = \) depth from the rock mass surface.

As shown in Figure 22, Cho (2002) compiled \( P_{ult} \) data available from the field tests and compared it with values calculated using research presented by Zhang et al. (2000).
2.3.3.5 Field Test Predictions

Cho (2002) used the new P-y curve model to predict the results of the three previously described field tests. Figures 23 and 24 are some of the results of these predictions.
Figure 23. Verification of P-y Curve Model – Caldwell County Short Shaft (Cho, 2002)

Figure 24. Verification of P-y Curve Model – Wilson County Long Shaft (Cho, 2002)
2.4 Summary of Literature Review

The literature review presented two popular methods for analyzing laterally loaded drilled shafts in weathered rock profiles. Reese’s Method for the development of P-y curves for weak rock (Reese, 1997) was termed “interim” due to the limited amount of data available for development. The Stiff Clay Model (Reese, Cox, and Koop, 1975) is a popular method used when weathered rock is encountered; based on conservative material properties assumed by many agencies, this model is known to produce conservative results. A new model, published by Cho (2002), was outlined; it is the result of extensive laboratory and field testing as well as F.E.M. modeling. This model is the focus of the verification testing presented in the following chapters of this report.
CHAPTER 3. LATERAL LOAD TESTING FOR THE VERIFICATION OF THE WEATHERED ROCK MODEL FOR P-y CURVES

Four full-scale field load tests were performed for verification of the Weathered Rock Model published by Cho (2002). Class A performance predictions of the four test shafts were performed prior to field testing. During the progress of the load tests, measured pile head deflections were plotted against performance predictions. Performance predictions were developed using the Weathered Rock Model (Cho, 2002), Reese’s Method for P-y curves in Weak Rock (Reese, 1997), and the Stiff Clay Model (Reese, Cox, and Koop, 1975).

3.1 Verification Load Testing

The verification load tests were located on two sites in Durham County, North Carolina. Two tests were performed on a site situated inside the cloverleaf interchange of Interstate 40 (I-40) West and North Carolina Highway 55, in southern Durham County. Two more tests were carried out inside the exit ramp area of the interchange between Interstate 85 (I-85) North and Gregson Street, in central Durham County. The rock types encountered at each test site are listed in Table 5; the subsurface profiles consisted of residual soils, claystone, siltstone, and sandstone of the Durham Triassic Basin. At each test site, two 0.762 m diameter drilled shafts were constructed approximately 7.93 m apart. The shafts were drilled using a truck mounted rig and conventional rock augers; 12.7 mm thick permanent steel casing was “screwed in” to shaft tip elevation of each test shaft. Figure 25 is a picture of drilling a test shaft at the I-85 site. Permanent casing was utilized so that significant deflections of the weathered rock could be realized without failure of the shaft. Testing setup was similar to that used by Cho (2002); a schematic diagram was presented in the Literature Review, Figure 9. A loading frame supplied by the North Carolina Department of Transportation was used to transfer lateral load from a hydraulic jack to each test shaft. A 4,448 kN hydraulic jack and an electronic load cell were used to apply and monitor lateral loads during testing. Figure 26 is a picture looking from the hydraulic jack to the long shaft at the I-40 test site. The test shafts were
loaded in increments of approximately 89 kN up to 1512 kN. Unloading cycles down to 89 kN were performed as the loading progressed.

Table 5. Verification Test Sites and Rock Types

<table>
<thead>
<tr>
<th>Test Site</th>
<th>Rock Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-40</td>
<td>Triassic Claystone, Siltstone, and Sandstone</td>
</tr>
<tr>
<td>I-85</td>
<td>Triassic Claystone, Siltstone, and Sandstone</td>
</tr>
</tbody>
</table>

Figure 25. Drilling a Test Shaft – I-85 Site
3.1.1 Instrumentation Plan

Dial gages, strain gages, and slope inclinometers were used to measure deformations and strain with depth of the test shafts. Four surface dial gages were used to measure deflections and rotation. A fixed reference beam, in accordance with section 5.11 of ASTM D3966-90, was used to secure dial gages. Two dial gages were used to measure shaft rotation, so that deflection angles could be determined. One dial gage was used to measure deflection in the direction of loading, while another was used to measure movement perpendicular to the plane of loading.

Vibrating wire strain gages mounted to 1 m long sister bars were attached to the tension side of the vertical reinforcement cages and cast into the test shafts. A CR-10x data logger, manufactured by Campbell Scientific, Inc., recorded strain and temperature measured from the vibrating wire gages.
A continuous chain of slope inclinometers was used to measure lateral deflection of the test shafts with depth. Electrolytic (EL) vertical in-place inclinometers were inserted into a plastic housing that was secured to the vertical reinforcement cage. A continuous chain of inclinometer probes consisted of sensors with wheels that are attached to each other at pivot points 0.5 m apart. Signal cables from each inclinometer extended up through the plastic housing to a data acquisition system, for monitoring and collection by a computer program. Figure 27 is a picture of an instrumented reinforcement cage before insertion into the permanent casing, note that strain gages are on top as shown in the figure and inclinometer casing is opposite the gages.

Figure 27. Instrumented Reinforcement Cage
3.2 Interstate 40 Load Tests

The I-40 test site was situated at the northwest corner of the intersection of I-40 West and North Carolina Highway 55 in Durham County, North Carolina. The site was positioned within the confines of the cloverleaf exit ramp. Figure 28 is a local map of the area where the site was located. The test area footprint was 21 m by 12 m at the ground surface, and then sloped 3.1 m down to the test pad, El. 80.525 m. Figure 29 is a picture of the exposed rock at the elevation of the test pad.

![Figure 28. Local Area Map of the I-40 Test Site](image-url)
3.2.1 Geology

There are two major Triassic Basins in North Carolina, The Dan River basin and the Deep River basin. The Deep River basin is divided into three separate basins, the Durham, Sanford, and Wadesboro sub-basins (Parish, 2001). The I-40 test site was located within the Durham Triassic Basin (DTB). The DTB is primarily comprised of sedimentary rocks including red conglomerate, arkosic sandstone, siltstone, claystone and mudstone (Parish, 2001). The residual soils at the test site were predominately dark brown to dark red-brown silty clays with mica. The transition to weathered rock was encountered approximately 3 m below the ground surface, EL. 83.698 m. RQD values of the material ranged from 72% to 100% at SB-1 and 89% to 100% at SB-2. The subsurface profile of the test site is shown in Figure 30.
3.2.2 Geotechnical Properties of the Test Site

Subsurface borings were performed at the location of each test shaft. Samples from standard penetration testing (SPT) in the residual soils were dark brown to dark red-brown silty clay with mica. Blow counts (N-values) ranged from 9 (blows/300mm) to 16 (blows/300mm) at the surface and increased to 30 (blows/300mm) to 59 (blows/300mm) just above the weathered rock line, approximately 3.0 m below the ground surface. The weathered rock was cored using size H casing and NXWL core bits. The upper 3 m of weathered rock was claystone, after which there was a transition to siltstone then to sandstone. Core logs for each boring are given in Table 6. The NCDOT Materials and Test Unit tested core samples in unconfined compression. The unconfined compression ($\sigma_{ci}$) results are presented in Table 7 along with RQD values at corresponding depths. Upon completion of the rock coring, a rock dilatometer (model Probex 1 rock dilatometer

Figure 30. I-40 Test Site Subsurface Profile
manufactured by ROCTEST, Plattsburgh, NY) was used to measure pressure-volume data for the evaluation of the in-situ rock-mass modulus of elasticity for the weathered rock. Figures 31 and 32 are the pressure vs. volume curves obtained from the rock dilatometer testing for SB-1 and SB-2, respectively. The coefficient of subgrade reaction ($k_{ho}$) was determined with depth using measured rock dilatometer data and Equation 33; these data are presented in Table 8. The profile at the location of SB-1 has relatively higher modulus as presented in Table 8. At depths of 3.02 m and 4.02 m (in the case of SB-1) and 3.26 m (in the case of SB-2) there was lack of contact between the rock dilatometer probe and the sides of the core hole.

3.2.3 Description of Drilled Shafts

Two drilled shafts, 0.762 m in diameter, were constructed 7.93 m apart at the test site. The long shaft was constructed at the location of subsurface boring SB-1 and the short shaft at SB-2. Due to the depth of the weathered rock at the test site, a 0.914 m temporary casing was first installed to the rock line at the location of each test shaft; the test shafts were drilled and constructed inside the temporary casing (this can be viewed in Figure 27). Each shaft was constructed using 27.6 MPa concrete with a vertical reinforcement cage made of 12 – #32 mm diameter rebar on a 245 mm radius. Shear spirals consisted of #16 mm diameter rebar at a 127 mm pitch. Each test shaft had 12.7 mm thick permanent casing that extended to the tip elevations. The short shaft was embedded 3.356 m and the long shaft was embedded 4.057 m, each completely in weathered rock. Approximately 1 m of each shaft was left exposed to allow for the attachment of the load frame and surface instrumentation.

The short shaft was instrumented with 7 vibrating wire strain gages attached to 1-m long sister bars. The long shaft had 9 strain gages similarly attached to the reinforcement cage using sister bars. Both shafts were instrumented with continuous slope inclinometer probes inserted into a precast plastic housing.
### Table 6: I-40 Test Site Core Log

#### SB-1 – Long Shaft

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Rate (min/0.5m)</th>
<th>Run (m)</th>
<th>Rec (m)</th>
<th>RQD (m)</th>
<th>Field Classification and Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.55</td>
<td>0:40</td>
<td>1.52</td>
<td>1.52</td>
<td>1.52</td>
<td>Dk. Red Brown, med. hard siltstone, only horizontal fractures 1.70, 2.05, 2.40, 2.60, 2.70, 2.90 meters</td>
</tr>
<tr>
<td>3.07</td>
<td>1:30</td>
<td>1.52</td>
<td>1.52</td>
<td>1.10</td>
<td>Dk. Red Brown, friable to indurated, soft to mod. hard siltstone, 1 joint from 3.80 to 4.05 meters</td>
</tr>
<tr>
<td>4.59</td>
<td>2:40</td>
<td>1.52</td>
<td>1.52</td>
<td>1.52</td>
<td>Dk. Red Brown, friable to indurated, med. hard to mod. hard siltstone and sandstone, 1 joint at 4.89 meters at 70 degrees</td>
</tr>
<tr>
<td>6.11</td>
<td>2:00</td>
<td>1.52</td>
<td>1.52</td>
<td>1.19</td>
<td>Dk. Red Brown, friable to indurated, soft to mod. hard siltstone and sandstone, 6.89 to 7.25 Red Brown hard clay, 7.25 meter Lt. Red, friable to indurated, mod. hard sandstone</td>
</tr>
</tbody>
</table>

#### SB-2 – Short Shaft

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Rate (min/0.5m)</th>
<th>Run (m)</th>
<th>Rec (m)</th>
<th>RQD (m)</th>
<th>Field Classification and Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.54</td>
<td>1:39</td>
<td>1.52</td>
<td>1.30</td>
<td>86%</td>
<td>Dk. Red Brown, silty clay with rock fragments</td>
</tr>
<tr>
<td>3.06</td>
<td>2:21</td>
<td>1.52</td>
<td>1.52</td>
<td>1.52</td>
<td>Dk. Red Brown, friable to indurated, med. hard to mod. hard, siltstone-claystone, no fractures</td>
</tr>
<tr>
<td>4.58</td>
<td>1:21</td>
<td>1.52</td>
<td>1.52</td>
<td>1.35</td>
<td>Dk. Red Brown, friable to mod. indurated, soft to mod. hard claystone-siltstone, no fractures</td>
</tr>
<tr>
<td>6.10</td>
<td>1:35</td>
<td>1.52</td>
<td>1.52</td>
<td>1.36</td>
<td>Dk. Red Brown, friable to mod. indurated, soft to mod. hard sandstone, 1 joint at 7.45 meters at 70 degrees</td>
</tr>
</tbody>
</table>

46
Table 7. I-40 Laboratory Test Results

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Unconfined Compressive Strength (MPa)</th>
<th>RQD (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.20 – 6.39</td>
<td>25.9</td>
<td>78</td>
</tr>
<tr>
<td>3.50 – 3.63</td>
<td>12.2</td>
<td>72</td>
</tr>
<tr>
<td>5.24 – 5.41</td>
<td>12.2</td>
<td>89</td>
</tr>
<tr>
<td>6.10 – 6.25</td>
<td>34.9</td>
<td>100</td>
</tr>
</tbody>
</table>

MPa to psi: multiply by 145.04

Figure 31. Rock Dilatometer Test Results – I-40 Test Site SB-1
Figure 32. Rock Dilatometer Test Results – I-40 Test Site SB-2

Table 8. I-40 Rock Dilatometer Results – k_ho Values

<table>
<thead>
<tr>
<th>Boring Location</th>
<th>Depth (m)</th>
<th>k_ho (MPa/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SB-1</td>
<td>5.02</td>
<td>394.5</td>
</tr>
<tr>
<td></td>
<td>6.02</td>
<td>373.8</td>
</tr>
<tr>
<td></td>
<td>7.02</td>
<td>349.1</td>
</tr>
<tr>
<td>SB-2</td>
<td>4.26</td>
<td>161.0</td>
</tr>
<tr>
<td></td>
<td>5.26</td>
<td>195.6</td>
</tr>
<tr>
<td></td>
<td>6.26</td>
<td>436.9</td>
</tr>
<tr>
<td></td>
<td>7.26</td>
<td>396.4</td>
</tr>
</tbody>
</table>

MPa/m to pci: multiply by 3.684
3.2.4 I-40 Load Test Performance Predictions

Four class A performance predictions were made for each shaft:

1. “Predicted-Dilatometer” – P-y Curves are computed based on $k_h$ from rock dilatometer test data and the Weathered Rock Model (Cho, 2002).
2. “Predicted-Geologic Based” – P-y Curves are computed using the Weathered Rock Model with $k_h$ determined from empirical equations for the coefficient of subgrade reaction (Cho, 2002).
3. Reese’s Method for P-y Curves in Weak Rock and engineering properties measured in the laboratory and field (Reese, 1997).

3.2.4.1 I-40 Load Test – Predicted-Dilatometer

Based on unconfined compressive strength test results and rock dilatometer data, the subsurface profiles at each shaft was analyzed in a number of layers. P-y curves as a function of depth were developed for each layer based on corresponding strength and modulus data. The parameters used for calculating the P-y curves for this set of predictions are listed in Table 9. The values of GSI and $m_i$ were taken from Tables 2 and 4, respectively, as presented in section 2.3.3.1 of the Literature Review. Because there were a limited number of samples tested in unconfined compression, a reference modulus ratio ($k_{ho}/\sigma_{ci}$) was used to establish the compressive strength for layers where data were not available. The GSI value is determined by summing the ratings for each parameter listed in Table 2. The methodology for determining ratings for spacing of joints and condition of joints was to use those ratings that corresponded with measured RQD. Based on a recommendation put forth by Hoek and Brown (1997), the value of 10 was used for the ground water rating. Equation 33 was used to determine $k_{ho}$ from rock dilatometer test results; the value of $k_{ho}$ was assumed equal to the elastic modulus, $E_{st}$, of the weathered rock. The average value of the elastic modulus was determined by calculating a weighted average with depth. Depth to the point of rotation, $T_o$, was determined using Equation 27. Equation 34 was used to calculate $k_h$ above the point of
rotation and Equation 35 was used below the point of rotation. $P_{ult}$ was determined using Equations 36, 37, and 40. A spreadsheet was utilized to calculate values of $k_h$ and $P_{ult}$ for a number of P-y curves in each layer. These P-y curves were then entered into the computer program COM624P (Version 2.0, Reese and Wang, 1993) to evaluate the behavior of each shaft under incremental lateral loads. Figure 33 is presented to describe how the density of P-y curves was increased near layer interfaces; this allowed for a reduction in error imposed when the analysis software interpolated between curves. In an iterative process, the point of rotation determined from the COM624P analysis was reentered into the spreadsheet and a new set of P-y curves were generated. The new P-y curves were used for a second iteration and this process was repeated until the point of rotation converged to within 10% of the previous location. The point of rotation for the short shaft converged at 3.1 m below the point of load application, 0.64 m lower than the value originally calculated using the model. The point of rotation for the long shaft converged at a depth of 3.7 m below the point of load application, 1.04 m below the value originally calculated using the model. Table 10 lists the values of $k_h$ and $P_{ult}$ used to construct the P-y curves for both the short and long shaft predictions using dilatometer data. These values reflect the variability of rock strength and stiffness as obtained from laboratory and field testing.

An additional prediction for the long shaft was made that utilized a reduced GSI value. For this set of predictions, the $k_h$ values were derived from dilatometer testing, as described above. The reduced GSI value imposed a softening effect on calculated $P_{ult}$ values. The rational is based on the fact that Triassic Weathered Rock will slake when exposed to water and releases in stress. It was believed that the weathered rock in the vicinity of the test shaft could be subjected to releases in stress from the drilling process. A reduction factor of 0.53 was applied to the GSI values for each layer and the $P_{ult}$ values were determined. Table 11 lists the $P_{ult}$ values used in the dilatometer-reduced GSI predictions.
Figure 33. Example of P-y Curve Distribution Used – I-40 Short Shaft Shown
Table 9. Parameters for I-40 Predictions – Dilatometer

<table>
<thead>
<tr>
<th>Layer Number</th>
<th>Short Shaft</th>
<th>Long Shaft</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>Layer Thickness (m)</td>
<td>1.8</td>
<td>1.0</td>
</tr>
<tr>
<td>$\gamma$ (kN/m$^3$)</td>
<td>25</td>
<td>25</td>
</tr>
<tr>
<td>$\sigma_{ci}$ (MPa)</td>
<td>11.3</td>
<td>12.2</td>
</tr>
<tr>
<td>RQD (%)</td>
<td>100</td>
<td>89</td>
</tr>
<tr>
<td>GSI</td>
<td>87</td>
<td>74</td>
</tr>
<tr>
<td>$m_i$</td>
<td>9</td>
<td>9</td>
</tr>
<tr>
<td>$E_s$ (GPa)</td>
<td>0.161</td>
<td>0.1456</td>
</tr>
<tr>
<td>$k_h$ (MPa/m)</td>
<td>161.0</td>
<td>145.6</td>
</tr>
</tbody>
</table>

| Avg. $E_s$ (GPa) | 0.1991 | 0.32 |
| $K_R$ | $3.895 \times 10^{-2}$ | $9.189 \times 10^{-3}$ |
| Calculated $T_o$ (m) | 2.46 | 2.66 |
| $K_h$ Number, $I_T$ | 5.38 | 7.15 |
| # P-y Curves Used | 13 | 19 |

Table 10. $k_h$ and $P_{ult}$ Values for I-40 Predictions – Dilatometer

<table>
<thead>
<tr>
<th>Layer #</th>
<th>Curve #</th>
<th>Depth (m)</th>
<th>$k_h$ (MPa)</th>
<th>$P_{ult}$ (kN/m)</th>
<th>Layer #</th>
<th>Curve #</th>
<th>Depth (m)</th>
<th>$k_h$ (MPa)</th>
<th>$P_{ult}$ (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>0.7</td>
<td>122.7</td>
<td>4746.0</td>
<td>1</td>
<td>1</td>
<td>0.7</td>
<td>132.6</td>
<td>1469.7</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1.5</td>
<td>122.7</td>
<td>4848.2</td>
<td></td>
<td>2</td>
<td>1.0</td>
<td>132.6</td>
<td>1529.2</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>1.9</td>
<td>122.7</td>
<td>4898.6</td>
<td></td>
<td>3</td>
<td>1.3</td>
<td>132.6</td>
<td>1585.6</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>2.0</td>
<td>122.7</td>
<td>4911.2</td>
<td></td>
<td>4</td>
<td>1.4</td>
<td>132.6</td>
<td>1604.3</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>2.2</td>
<td>110.9</td>
<td>3019.0</td>
<td></td>
<td>5</td>
<td>1.6</td>
<td>300.6</td>
<td>4330.0</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>2.3</td>
<td>110.9</td>
<td>3033.7</td>
<td></td>
<td>6</td>
<td>1.7</td>
<td>300.6</td>
<td>4355.6</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>2.5</td>
<td>110.9</td>
<td>3063.0</td>
<td></td>
<td>7</td>
<td>2.2</td>
<td>300.6</td>
<td>4481.2</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>2.7</td>
<td>110.9</td>
<td>3092.0</td>
<td></td>
<td>8</td>
<td>2.6</td>
<td>300.6</td>
<td>4579.1</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>2.9</td>
<td>110.9</td>
<td>3120.7</td>
<td></td>
<td>9</td>
<td>2.7</td>
<td>300.6</td>
<td>4603.3</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>3.0</td>
<td>110.9</td>
<td>3135.0</td>
<td></td>
<td>10</td>
<td>2.8</td>
<td>300.6</td>
<td>4627.3</td>
</tr>
<tr>
<td></td>
<td>11</td>
<td>3.2</td>
<td>1790.6</td>
<td>8765.0</td>
<td></td>
<td>11</td>
<td>3.0</td>
<td>284.8</td>
<td>6579.8</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>3.3</td>
<td>1790.6</td>
<td>8793.0</td>
<td></td>
<td>12</td>
<td>3.1</td>
<td>284.8</td>
<td>6601.1</td>
</tr>
<tr>
<td></td>
<td>13</td>
<td>3.5</td>
<td>1790.6</td>
<td>8848.8</td>
<td></td>
<td>13</td>
<td>3.3</td>
<td>284.8</td>
<td>6643.5</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>14</td>
<td>3.5</td>
<td>284.8</td>
<td>6685.6</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>15</td>
<td>3.7</td>
<td>284.8</td>
<td>6727.5</td>
</tr>
<tr>
<td></td>
<td>16</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>16</td>
<td>3.8</td>
<td>2036.0</td>
<td>6748.4</td>
</tr>
<tr>
<td></td>
<td>17</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>17</td>
<td>4.0</td>
<td>1901.5</td>
<td>5969.4</td>
</tr>
<tr>
<td></td>
<td>18</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>18</td>
<td>4.1</td>
<td>1901.5</td>
<td>5990.3</td>
</tr>
<tr>
<td></td>
<td>19</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>19</td>
<td>4.3</td>
<td>1901.5</td>
<td>6032.0</td>
</tr>
</tbody>
</table>

CONVERSIONS
MPa to psi: multiply by 145.04
kN/m to kips/inch: divide by 175.13
### Table 11. $P_{ult}$ Values for I-40 Long Shaft Predictions – Dilatometer-Reduced GSI

<table>
<thead>
<tr>
<th>Layer #</th>
<th>Curve #</th>
<th>Depth (m)</th>
<th>$P_{ult}$ (kN/m)</th>
<th>Layer #</th>
<th>Curve #</th>
<th>Depth (m)</th>
<th>$P_{ult}$ (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>0.7</td>
<td>837.5</td>
<td>11</td>
<td>3.0</td>
<td>2621.4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1.0</td>
<td>903.1</td>
<td>12</td>
<td>3.1</td>
<td>2645.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>1.3</td>
<td>960.1</td>
<td>13</td>
<td>3.3</td>
<td>2692.2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>1.4</td>
<td>977.7</td>
<td>14</td>
<td>3.5</td>
<td>2738.2</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>5</td>
<td>1.6</td>
<td>1978.3</td>
<td>15</td>
<td>3.7</td>
<td>2783.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>1.7</td>
<td>2007.4</td>
<td>16</td>
<td>3.8</td>
<td>2805.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>2.2</td>
<td>2144.6</td>
<td>17</td>
<td>4.0</td>
<td>2650.9</td>
<td></td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>2.6</td>
<td>2245.9</td>
<td>18</td>
<td>4.1</td>
<td>2671.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>2.7</td>
<td>2270.3</td>
<td>19</td>
<td>4.3</td>
<td>2712.8</td>
<td></td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>2.8</td>
<td>2294.3</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**CONVERSIONS**

MPa to psi: mult by 145.04  
kN/m to kips/inch: divide by 175.13

Note: Depth is referenced from the Point of Load, El. 81.0 meters.

#### 3.2.4.2 I-40 Load Test – Predicted-Geologic Based

Predictions based on geologic parameters that are correlated with rock strength were developed using equations 29, 30, 31, and 32 for calculating $k_{ho}$, $n_h$, and $k_h$ (these equations utilize unconfined compressive strength data and GSI). The subsurface profile at each shaft was analyzed with the same number of layers and P-y curves used for dilatometer predictions. As the initial analysis of short shaft performance began, it was noticed that the calculated values of $k_{ho}$ were large as compared to those determined from rock dilatometer testing. It was also noticed that the measured values of RQD for the I-40 rock cores were much larger than those from the field testing sites documented in Cho (2002). The higher values of $k_{ho}$ calculated from the empirical equations were a direct result of applying the previously described procedure for selecting GSI values. For dilatometer predictions, GSI is used only for the calculation of ultimate strength, and modulus values are taken directly from rock dilatometer test results. With geologic based predictions, GSI is used to establish both strength and modulus parameters; therefore, high RQD values lead to high GSI values which equate to high estimated modulus parameters (Note: GSI is located in the exponent of the equation for $k_{ho}$, therefore the effect on results of the equation are very significant). A rational of “reduced GSI” was
adopted to soften $k_{ho}$ values from empirical equations so that predicted shaft head deflections would compare reasonably with dilatometer predictions.

GSI values were reduced by a multiplication factor determined by trial and error. The value of the reduction factor was varied until shaft head deflections determined were reasonable when compared with dilatometer predictions. Without reducing GSI, the predicted shaft head deflection for an applied lateral load of 1334 kN was 0.00499 m. When a weighted average reduction factor of 0.78 was applied to GSI values, shaft head deflection increased to 0.0135 m, compared to 0.0193 m predicted using dilatometer data. Table 12 lists the $k_h$ values used to construct each P-y curve. Class B performance predictions for the long shaft utilizing the method presented in this section are given at the end of this chapter.

Table 12. $k_h$ Values for I-40 Short Shaft Predictions – Geologic Based-Reduced GSI

<table>
<thead>
<tr>
<th>Layer #</th>
<th>Curve #</th>
<th>Depth¹ (m)</th>
<th>$k_h$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>0.7</td>
<td>183.2</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1.5</td>
<td>186.9</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>1.9</td>
<td>188.8</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>2.0</td>
<td>189.3</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>2.2</td>
<td>167.4</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>2.3</td>
<td>167.8</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>2.5</td>
<td>168.8</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>2.7</td>
<td>169.7</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>2.9</td>
<td>170.6</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>3.0</td>
<td>1452.3</td>
</tr>
<tr>
<td>2</td>
<td>11</td>
<td>3.2</td>
<td>2668.2</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>3.3</td>
<td>2672.1</td>
</tr>
<tr>
<td></td>
<td>13</td>
<td>3.5</td>
<td>2680.0</td>
</tr>
</tbody>
</table>

Note: Depth is referenced from the Point of Load, El. 81.0 meters. MPa to psi: multiply by 145.04

3.2.4.3 I-40 Load Test– Reese’s Method and Stiff Clay Model

P-y curves based on Reese’s method were developed using the concepts presented in Section 2.2 of the Literature Review. The same number of layers and P-y curves were used for Reese’s Method as used for predictions with the Weathered Rock Model. Unconfined compressive strength from laboratory testing and elastic modulus from
dilatometer testing was used to construct P-y curves by Reese’s Method, along with an average of the range of $k_m$ (0.000275) values reported by Reese (1997). These P-y curves were input to COM624P and pile head deflections, for incremental lateral loads, were determined.

COM624P contains a subroutine for the analysis of laterally loaded piles using the Stiff Clay Model. This selection was used with the following material properties: $k_{ho} = 543,000 \text{ kN/m}^3$, Cohesion = 200 kPa, and $\varepsilon_{50} = 0.004$. These material properties are standard for the North Carolina Department of Transportation when analyzing laterally loaded drilled shafts in weathered rock. Performance predictions for I-40 Short Shaft and Long Shaft are presented in Figures 34 and 35, respectively.

![Figure 34. I-40 Short Shaft Performance Predictions](image)
3.2.5 I-40 Load Test Results

Using a hydraulic jack, lateral load was applied to both shafts in increments of 89 kN up to a maximum load of 1512 kN. For the maximum applied lateral load of 1512 kN, the short shaft experienced 0.0113 m of deflection at the point of load application, while the long shaft deflected 0.0161 m.

3.2.5.1 Top Deflections and Inclinometer Readings

Dial gages were used to monitor shaft deflections above the ground surface for each increment of lateral load applied to the test shafts. Measured deflections for both the short and long shafts are presented in Figure 36. Both shafts exhibited nearly linear load-deflection behavior up to the maximum applied load. Larger deflections measured at the long shaft can be attributed to poorer quality of joints as well as close spacing of joints and the effects of standing water. The long shaft was located on the lower end of
the testing pad where perched water and rainwater accumulated. Triassic Weathered Rock is known to slake (degrade in strength) in the presence of water (Parish, 2001). It is postulated that slaking of the rock near the surface at the long shaft is the cause of the larger measured deflections. An axial statnemic test conducted by the NCDOT showed a reduction in side shear capacity of Triassic Weathered Rock, up to 54%, due to soaking an augured shaft hole for a period of 24 hours (AFT, 2002).

A system of continuous slope inclinometers was used to measure the deflection profiles with depth for both the short and long shafts. Inclinometer data are recorded as the cumulative sum of successive gage deflections beginning with the bottom-most gage. Since neither string of inclinometers extended below the shaft tip, the data must be adjusted to a known value of deflection. Shaft head deflections measured from dial gages were used to adjust this data. The deflection profiles before dial gage adjustment for both the short and long shafts are presented in the Appendix. The adjusted deflection profiles for the short and long shafts are given in Figures 37 and 38, respectively.

Figure 36. Top Deflections of the I-40 Short and Long Shaft Measured from Dial Gages
Figure 37. Deflection Profiles after Dial Gage Adjustment – I-40 Short Shaft

Figure 38. Deflection Profiles after Dial Gage Adjustment – I-40 Long Shaft
3.2.5.2 Predicted and Measured Test Shaft Performance

Based predicted and measured shaft behavior, Figures 39 and 40 demonstrate the applicability of the Weathered Rock Model. The geologic based-reduced GSI prediction compares favorably with the measured short shaft deflections and the dilatometer prediction would be considered a good conservative estimate. As for the long shaft, the dilatometer predictions seem to model the behavior of the shaft relatively well. The predictions based upon reduced GSI also performed well up to 600 kN, after which the effect of softening $P_{ult}$ became evident. Therefore, there is no need to soften the ultimate resistance when designing for Triassic Weathered Rock.

3.2.5.3 Back Calculated P-y Curves

Using the strain measurements with depth, moment curves were back calculated for each load increment using Equations 12, 16, and 17. A fourth order equation was used to regress moment data; the soil reaction ($P$, kN/m) with depth was calculated from the second derivative of moment curves. Deflection ($y$, meters) was evaluated directly from inclinometer data. The back calculated P-y curves from strain gage and inclinometer data are shown in Figures 41 and 42.

The back calculated P-y curves show that $k_h$ increased with depth; however a decrease in $k_h$ from 2.3 m to 3.8 m for the long shaft was measured. This could be attributed to changes in rock properties that were not discovered in the subsurface investigation or to error introduced in the back calculation process, partly due to small deflections around the point of rotation (Cho, 2002).
Figure 39. I-40 Short Shaft Pile Head Deflection Performance

Figure 40. I-40 Long Shaft Pile Head Deflection Performance
Transformed axes plots were used to curve fit the back calculated P-y data, as discussed in Chapter 2. These plots were used to establish the values of $k_h$ and $P_{ult}$ for the back calculated P-y curves. Two examples are shown in Figures 43 and 44, the remainder are presented in Appendix.

Figure 41. Back Calculated P-y Curves for the Weathered Rock – I-40 Short Shaft
Figure 42. Back Calculated P-y Curves for the Weathered Rock – I-40 Long Shaft

Figure 43. Curve Fitting Results – I-40 Short Shaft
3.2.5.4 Predicted and Back Calculated P-y Curves

Figures 45 through 50 are presented for comparison of P-y curves generated using the Weathered Rock Model and those back calculated from measured strain and deflection data. Because there was little variation in $k_h$ and $P_{ult}$ in the divided subsurface profiles, one predicted P-y curve is shown for each layer. For individual comparison purposes these graphs are plotted on differing scales.

In general, the Weathered Rock Model seems to under predict the available resistance near the ground surface and somewhat over predict resistance at deeper depths. However, the overall balance is such that there appears to be a compensating effect, in that shaft head deflections at any given lateral load are reasonably well represented.
Figure 45. Predicted and Back Calculated P-y Curves – I-40 Short Shaft Layer 1

Figure 46. Predicted and Back Calculated P-y Curves – I-40 Short Shaft Layer 3
Figure 47. Predicted and Back Calculated P-y Curves – I-40 Long Shaft Layer 1

Figure 48. Predicted and Back Calculated P-y Curves – I-40 Long Shaft Layer 2
Figure 49. Predicted and Back Calculated P-y Curves – I-40 Long Shaft Layer 3

Figure 50. Predicted and Back Calculated P-y Curves – I-40 Long Shaft Layer 4
3.3 Interstate 85 Load Tests

The I-85 test site was located within the exit ramp area of the Interstate 85 (I-85) North and Gregson Street interchange, in central Durham County. Figure 51 is a local area map showing the site location. The test area was 12 m by 8 m and was excavated 1.5 m down to the test pad elevation, El. 97.6 m. Figure 52 is a picture of the exposed rock at the elevation of the test pad.

3.3.1 Geology

The I-85 test site was located on the northwestern portion of the DTB. Approximately 1.2 m of residual soil was overlying the weathered rock at the test site. Coring was terminated approximately 5.1 m below the rock line in Triassic Weathered Rock. RQD values ranged from 49% to 96% at B1-Dur and 44% to 72% at B2-Dur (Parish, 2001). A subsurface profile of the test site can be found in Figure 53.

Figure 51. Local Area Map of the I-85 Test Site
Figure 52. Exposed Rock Profile at the Elevation of the Test Pad

Figure 53. I-85 Test Site Subsurface Profile
3.3.2 Geotechnical Properties of the Test Site

Two subsurface borings were performed, one each at the location of the test shafts. Information pertaining to the type of residual soils at the test site was not documented; the transition to weathered rock occurred approximately 1.2 m below the ground surface. The weathered rock was cored using size H casing and NXWL core bits. The upper 4.5 m of weathered rock was Triassic siltstone and claystone, after which there was a transition to Triassic sandstone. Table 13 presents a description of the core runs taken at each boring. Core samples were selected for unconfined testing by Dr. David Parish, North Carolina State University; results are presented in Table 14. This site was also part of a comprehensive research program into the Slake Durability and Engineering Properties of Durham Triassic Basin Rock, (Parish, 2001). After the rock coring was completed, a rock dilatometer (model Probex 1 rock dilatometer manufactured by ROCTEST, Plattsburgh, NY) was used to measure pressure-volume data for the evaluation of the in-situ rock-mass modulus of elasticity of the weathered rock. Figures 54 and 55 are pressure vs. volume curves for B1-Dur and B2-Dur, respectively. The coefficient of subgrade reaction ($k_{ho}$) was determined with depth using the measured rock dilatometer data and Equation 33, these results are presented in Table 15.

3.3.3 Description of Drilled Shafts

Two drilled shafts, 0.762 m in diameter, were constructed 7.93 m apart at the test site. The short shaft was constructed at the location B1-Dur and the long shaft at B2-Dur. To aid in shaft construction a 0.914 m temporary casing was installed through the overburden down to the rock line at each test shaft. Construction of the test shafts took place inside of the temporary casings. The test shafts were constructed using 27.6 MPa concrete with a vertical reinforcement cage made up of 10 - #32 mm diameter rebar on a 245 mm radius. Shear spirals consisted of #16 mm diameter rebar at a 127 mm pitch. Each test shaft had a 12.7 mm thick permanent steel casing down to the design tip elevations. The short shaft had an embedment depth of 2.789 m; the long shaft was embedded 4.21 m. Both shafts were completely embedded in weathered rock.
Approximately 1 m of each shaft was left exposed to allow for the attachment of the load frame and surface instrumentation.

### Table 13. I-85 Test Site Core Log

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Rate (min/0.5m)</th>
<th>Run (m)</th>
<th>Rec (m)</th>
<th>RQD (m)</th>
<th>Field Classification and Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.46</td>
<td>Time not taken</td>
<td>0.21</td>
<td>0.21</td>
<td>0.12</td>
<td>Red Brown to Gray, Severely Weathered, Moderate to Extremely Fractured Weathered Rock (Triassic Siltstone)</td>
</tr>
<tr>
<td>1.67</td>
<td>1:14 0:59 1:10</td>
<td>1.43</td>
<td>1.43</td>
<td>0.7</td>
<td>Red Brown to Gray, Moderately to Severely Weathered, Slightly to Extremely Fractured Weathered Rock (Triassic Siltstone-Claystone) Sandstone Layer: 1.77 – 1.86 meters</td>
</tr>
<tr>
<td>3.19</td>
<td>1:05 1:26 1:16</td>
<td>1.52</td>
<td>1.52</td>
<td>0.98</td>
<td>Red Brown to Gray, Moderately to Severely Weathered, Slightly to Extremely Fractured Weathered Rock (Triassic Siltstone-Claystone) Sandstone Layer: 4.08 – 4.17 meters</td>
</tr>
<tr>
<td>4.71</td>
<td>1:26 1:14 1:31</td>
<td>1.52</td>
<td>1.52</td>
<td>1.46</td>
<td>Red Brown to Gray, Moderately Weathered, Moderately to Slightly Fractured, (Triassic Siltstone-Sandstone) Siltstone: 4.72 – 5.09m &amp; 5.94 – 6.16m Sandstone: 5.09 – 5.94m &amp; 6.16 – 6.25m</td>
</tr>
</tbody>
</table>

### B2-Dur – Long Shaft

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Rate (min/0.5m)</th>
<th>Run (m)</th>
<th>Rec (m)</th>
<th>RQD (m)</th>
<th>Field Classification and Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.19</td>
<td>Time not taken</td>
<td>0.46</td>
<td>0.46</td>
<td>0.27</td>
<td>Moderately Fractured, Severely Weathered, Gray Weathered Rock (Triassic Siltstone-Sandstone)</td>
</tr>
<tr>
<td>1.65</td>
<td>1:10 1:05 0:47</td>
<td>1.52</td>
<td>1.52</td>
<td>0.67</td>
<td>Moderately to Extremely Fractured, Severely Weathered, Red Brown to Gray Weathered Rock (Triassic-Siltstone)</td>
</tr>
<tr>
<td>3.17</td>
<td>0:52 1:18 1:02</td>
<td>1.52</td>
<td>1.52</td>
<td>0.67</td>
<td>Slightly to Extremely Fractured, Moderately to Severely Weathered, Red Brown Weathered Rock (Triassic Siltstone)</td>
</tr>
<tr>
<td>4.69</td>
<td>1:26 1:22 0:59</td>
<td>1.52</td>
<td>1.46</td>
<td>1.10</td>
<td>Slightly to Extremely Fractured, Moderately to Severely Weathered, Red Brown Weathered Rock(Triassic Siltstone)</td>
</tr>
<tr>
<td>6.21</td>
<td>96% 72%</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 14. I-85 Laboratory Test Results (Parish, 2001)

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Unconfined Compressive Strength (MPa)</th>
<th>RQD (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.0 – 3.9</td>
<td>28.7</td>
<td>44%</td>
</tr>
<tr>
<td>3.6 – 4.7</td>
<td>45.5</td>
<td>64%</td>
</tr>
<tr>
<td>4.3 – 5.4</td>
<td>33.0</td>
<td>100%</td>
</tr>
<tr>
<td>4.7 – 5.5</td>
<td>28.5</td>
<td>44%</td>
</tr>
<tr>
<td>5.4 – 6.2</td>
<td>35.8</td>
<td>72%</td>
</tr>
<tr>
<td>5.5 – 6.1</td>
<td>30.8</td>
<td>96%</td>
</tr>
</tbody>
</table>

MPa to psi: multiply by 145.04

Figure 54. Rock Dilatometer Test Results – I-85 Test Site B1-Dur
Figure 55. Rock Dilatometer Test Results – I-85 Test Site B2-Dur

Table 15. I-85 Rock Dilatometer Results – $k_{ho}$ Values

<table>
<thead>
<tr>
<th>Boring Location</th>
<th>Depth (m)</th>
<th>$k_{ho}$ (MPa/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1-Dur</td>
<td>2.4</td>
<td>107.9</td>
</tr>
<tr>
<td></td>
<td>3.4</td>
<td>92.1</td>
</tr>
<tr>
<td></td>
<td>4.3</td>
<td>336.2</td>
</tr>
<tr>
<td></td>
<td>5.2</td>
<td>876.9</td>
</tr>
<tr>
<td></td>
<td>6.1</td>
<td>707.4</td>
</tr>
<tr>
<td>B2-Dur</td>
<td>2.5</td>
<td>224.2</td>
</tr>
<tr>
<td></td>
<td>3.4</td>
<td>106.3</td>
</tr>
<tr>
<td></td>
<td>4.3</td>
<td>1151.9</td>
</tr>
<tr>
<td></td>
<td>5.2</td>
<td>604.6</td>
</tr>
<tr>
<td></td>
<td>6.2</td>
<td>1132.0</td>
</tr>
</tbody>
</table>

MPa/m to pci: multiply by 3.684
The short shaft was instrumented with 6 vibrating wire strain gages attached to 1 m sister bars, the long shaft with 9 strain gages similarly attached to the reinforcement cage using sister bars. Both shafts had continuous slope inclinometer probes inserted into a precast plastic housing. The instrumentation scheme allowed for the measurement of both strain and deflection with depth.

3.3.4 I-85 Load Test Performance Predictions

Four class A performance predictions were developed for each shaft as described below:

1. “Predicted-Dilatometer” – P-y curves are computed based on $k_h$ from rock dilatometer test data and the Weathered Rock Model (Cho, 2002).
2. “Predicted-Geologic Based” – P-y curves are computed using the Weathered Rock Model with $k_h$ determined from empirical equations for the coefficient of subgrade reaction (Cho, 2002).
3. Reese’s Method for P-y curves in weak rock and engineering properties measured in the laboratory and field (Reese, 1997).
4. P-y curves using the Stiff Clay Model and standard material properties used by the NCDOT (Reese, Cox, and Koop, 1975).

3.3.4.1 I-85 Load Test Performance Predictions

Performance predictions for the I-85 load tests were calculated as described in Section 3.2.4.1 through 3.2.4.3 with the following exception; the reduced GSI concept was not utilized. Table 16 lists the parameters used in making the “Predicted-Dilatometer” and “Predicted-Geologic Based” predictions for both the short and long shaft. Table 17 lists the $k_h$ and $P_{ult}$ values used to construct P-y curves for the predictions of the short and long shaft. Table 18 gives the $k_h$ values calculated using the Weathered Rock Model with empirical equations and geologic parameters. These values were used to make the “Predicted-Geologic Based” predictions. The performance predictions for the short and long shaft are presented in Figures 56 and 57, respectively.
Table 16. Parameters for I-85 Performance Predictions – Dilatometer and Geologic Based

<table>
<thead>
<tr>
<th>Layer Number</th>
<th>Short Shaft</th>
<th>Long Shaft</th>
<th>Short Shaft</th>
<th>Long Shaft</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td>Layer Thickness (m)</td>
<td>1.2</td>
<td>0.7</td>
<td>0.76</td>
<td>1.5</td>
</tr>
<tr>
<td>$\gamma$ (kN/m$^3$)</td>
<td>15</td>
<td>15</td>
<td>15</td>
<td>15</td>
</tr>
<tr>
<td>$\sigma_{ci}$ (MPa)</td>
<td>29.1</td>
<td>24.8</td>
<td>45.5</td>
<td>25.0</td>
</tr>
<tr>
<td>RQD (%)</td>
<td>53</td>
<td>64</td>
<td>64</td>
<td>44</td>
</tr>
<tr>
<td>GSI</td>
<td>59</td>
<td>59</td>
<td>59</td>
<td>38</td>
</tr>
<tr>
<td>$m_i$</td>
<td>9</td>
<td>9</td>
<td>9</td>
<td>9</td>
</tr>
<tr>
<td>$E_s$ (GPa)</td>
<td>0.1079</td>
<td>0.092</td>
<td>0.3362</td>
<td>0.2242</td>
</tr>
<tr>
<td>$k_h$ (MPa/m)</td>
<td>107.9</td>
<td>92.0</td>
<td>336.2</td>
<td>224.2</td>
</tr>
<tr>
<td>Avg. $E_s$ (GPa)</td>
<td>0.1689</td>
<td>0.4405</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$K_R$</td>
<td>$1.082 \times 10^{-1}$</td>
<td>$6.805 \times 10^{-3}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Calculated $T_o$ (m)</td>
<td>2.5</td>
<td>2.55</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$K_h$ Number, $I_t$</td>
<td>3.76</td>
<td>6.35</td>
<td></td>
<td></td>
</tr>
<tr>
<td># P-y Curves Used</td>
<td>15</td>
<td>18</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: $k_h$ values presented are based on rock dilatometer testing, $k_h$ from empirical equations given in Table 18.

Table 17. $k_h$ and $P_{ult}$ Values for I-85 Load Test Predictions – Dilatometer

<table>
<thead>
<tr>
<th>Layer #</th>
<th>Curve #</th>
<th>Depth (m)</th>
<th>$k_h$ (MPa)</th>
<th>$P_{ult}$ (kN/m)</th>
<th>Layer #</th>
<th>Curve #</th>
<th>Depth (m)</th>
<th>$k_h$ (MPa)</th>
<th>$P_{ult}$ (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>0.47</td>
<td>82.2</td>
<td>3131.1</td>
<td>1</td>
<td>1</td>
<td>0.55</td>
<td>170.8</td>
<td>1444.3</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.85</td>
<td>82.2</td>
<td>3209.7</td>
<td>2</td>
<td>1.05</td>
<td>170.8</td>
<td>1574.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>1.2</td>
<td>82.2</td>
<td>3280.2</td>
<td>3</td>
<td>1.55</td>
<td>170.8</td>
<td>1689.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>1.35</td>
<td>82.2</td>
<td>3309.9</td>
<td>4</td>
<td>1.7</td>
<td>170.8</td>
<td>1721.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>1.52</td>
<td>70.1</td>
<td>2942.5</td>
<td>5</td>
<td>1.9</td>
<td>81.0</td>
<td>1915.9</td>
<td></td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>1.65</td>
<td>70.1</td>
<td>2967.5</td>
<td>6</td>
<td>2.05</td>
<td>81.0</td>
<td>1947.1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>1.8</td>
<td>70.1</td>
<td>2995.8</td>
<td>7</td>
<td>2.4</td>
<td>81.0</td>
<td>2017.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>1.9</td>
<td>70.1</td>
<td>3014.6</td>
<td>8</td>
<td>2.55</td>
<td>81.0</td>
<td>2046.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>2.05</td>
<td>70.1</td>
<td>3042.6</td>
<td>9</td>
<td>2.65</td>
<td>81.0</td>
<td>2065.3</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>10</td>
<td>2.22</td>
<td>256.2</td>
<td>4972.4</td>
<td>10</td>
<td>2.95</td>
<td>877.7</td>
<td>2301.6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>11</td>
<td>2.35</td>
<td>256.2</td>
<td>4997.7</td>
<td>11</td>
<td>3.05</td>
<td>877.7</td>
<td>2320.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>2.5</td>
<td>963.9</td>
<td>5026.7</td>
<td>12</td>
<td>3.2</td>
<td>877.7</td>
<td>2348.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>13</td>
<td>2.6</td>
<td>963.9</td>
<td>5045.9</td>
<td>13</td>
<td>3.35</td>
<td>877.7</td>
<td>2376.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>2.8</td>
<td>963.9</td>
<td>5084.2</td>
<td>14</td>
<td>3.55</td>
<td>460.7</td>
<td>4081.4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>2.9</td>
<td>963.9</td>
<td>5103.2</td>
<td>15</td>
<td>3.65</td>
<td>460.7</td>
<td>4099.1</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>16</td>
<td>3.88</td>
<td>2927.4</td>
<td>4139.6</td>
<td>16</td>
<td>3.88</td>
<td>2927.4</td>
<td>4139.6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>17</td>
<td>4.13</td>
<td>2927.4</td>
<td>4183.1</td>
<td>17</td>
<td>4.38</td>
<td>2927.4</td>
<td>4226.2</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>18</td>
<td>4.38</td>
<td>2927.4</td>
<td>4226.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

CONVERSIONS

MPa to psi: multiply by 145.04
kN/m to kips/inch: divide by 175.13

Note: Depth is referenced from the Point of Load, El. 97.83 m
Table 18. $k_h$ Values for I-85 Load Test Predictions – Geologic Based

<table>
<thead>
<tr>
<th>Layer #</th>
<th>Curve #</th>
<th>Depth$^1$ (m)</th>
<th>$k_h$ (MPa)</th>
<th>Layer #</th>
<th>Curve #</th>
<th>Depth$^1$ (m)</th>
<th>$k_h$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td>1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>0.47</td>
<td>219.8</td>
<td>1</td>
<td>1</td>
<td>0.55</td>
<td>61.2</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>0.85</td>
<td>223.3</td>
<td>2</td>
<td>2</td>
<td>1.05</td>
<td>62.9</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>1.2</td>
<td>226.6</td>
<td>3</td>
<td>3</td>
<td>1.55</td>
<td>64.6</td>
</tr>
<tr>
<td>4</td>
<td>4</td>
<td>1.35</td>
<td>227.9</td>
<td>4</td>
<td>4</td>
<td>1.7</td>
<td>65.1</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>1.52</td>
<td>212.7</td>
<td>5</td>
<td></td>
<td>1.9</td>
<td>70.1</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td>1.65</td>
<td>213.9</td>
<td>6</td>
<td></td>
<td>2.05</td>
<td>70.6</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td>1.8</td>
<td>215.3</td>
<td>7</td>
<td></td>
<td>2.4</td>
<td>71.8</td>
</tr>
<tr>
<td>8</td>
<td></td>
<td>1.9</td>
<td>216.3</td>
<td>8</td>
<td></td>
<td>2.55</td>
<td>72.3</td>
</tr>
<tr>
<td>9</td>
<td></td>
<td>2.05</td>
<td>217.6</td>
<td>9</td>
<td></td>
<td>2.65</td>
<td>72.6</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>2.22</td>
<td>290.6</td>
<td>10</td>
<td></td>
<td>2.95</td>
<td>78.3</td>
</tr>
<tr>
<td>11</td>
<td></td>
<td>2.35</td>
<td>291.8</td>
<td>11</td>
<td></td>
<td>3.05</td>
<td>78.6</td>
</tr>
<tr>
<td>12</td>
<td></td>
<td>2.5</td>
<td>1495.7</td>
<td>12</td>
<td></td>
<td>3.2</td>
<td>79.2</td>
</tr>
<tr>
<td>13</td>
<td></td>
<td>2.6</td>
<td>1500.4</td>
<td>13</td>
<td></td>
<td>3.35</td>
<td>79.7</td>
</tr>
<tr>
<td>14</td>
<td></td>
<td>2.8</td>
<td>1509.9</td>
<td>14</td>
<td></td>
<td>3.55</td>
<td>243.3</td>
</tr>
<tr>
<td>15</td>
<td></td>
<td>2.9</td>
<td>1514.6</td>
<td>15</td>
<td></td>
<td>3.65</td>
<td>243.7</td>
</tr>
</tbody>
</table>

Note: Depth is referenced from the Point of Load, El. 97.83 m

MPa to psi: multiply by 145.04
Figure 56. I-85 Short Shaft Performance Predictions

Figure 57. I-85 Long Shaft Performance Predictions
3.3.5 I-85 Load Test Results

Using a hydraulic jack, lateral load was applied to both shafts in increments of 89 kN up to a maximum load of 1334 kN. At the maximum applied lateral load of 1334 kN, the short shaft deflected 0.0478 m and the long shaft experienced 0.0172 m of deflection.

3.3.5.1 Top Deflections and Inclinometer Readings

Deflections above the ground surface for both shafts were monitored with dial gages. Shaft head deflections for both the short and long shaft are presented in Figure 58. Unloading data was not obtained for the long shaft due to a malfunction with the dial gage. The short shaft yielded non-linear increments of deflection as the maximum load was approached; however the long shaft yielded nearly linear increments of deflection up to the maximum applied load.

![Figure 58. Top Displacements of the Short and Long Shaft Measured from Dial Gages](image-url)
A system of continuous slope inclinometers was used to measure deflection profiles with depth for both the short and long shaft. Inclinometer data are recorded as the cumulative sum of successive gage deflections beginning with the bottom-most gage. Since neither string of inclinometers extended below the shaft tip, the data must be adjusted to a known value of deflection. Shaft head deflections measured from dial gages were used to adjust this data. Deflection profiles before dial gage adjustment for both the short and long shaft are presented in the Appendix. Adjusted deflection profiles are given in Figures 59 and 60.

![Figure 59. Deflection Profiles after Dial Gage Adjustment – I-85 Short Shaft](image-url)
Figure 60. Deflection Profiles after Dial Gage Adjustment – I-85 Long Shaft

Figure 60 b. Deflection Profiles after Dial Gage Adjustment – I-85 Long Shaft

(Scale increased for clarity)
### 3.3.5.2 Predicted and Measured Test Shaft Performance

Based on predicted and measured shaft behavior, Figures 61 and 62 demonstrate the applicability of the Weathered Rock Model. The dilatometer predictions seemed to model the behavior of both the short and long shaft fairly well. Discrepancies between deflections predicted using geologic data and measured deflections could be attributed to the inherent problems associated with using an empirical equation to estimate in-situ properties. For the long shaft, the dilatometer prediction began to become more non-linear at 1000 kN; however the measured data remained linear.

![Graph showing deflection performance](image)

*Figure 61. I-85 Short Shaft Pile Head Deflection Performance*
3.3.5.3 Back Calculated P-y Curves

Using the strain measurements with depth, moment curves were developed for each load increment using Equations 12, 16, and 17. A fourth order equation was used to regress the moment data; the soil reaction (P, kN/m) with depth was calculated from the second derivative of the moment curves. Deflection (y, meters) was taken directly from the inclinometer readings. Back calculated P-y curves from strain gage and inclinometer data are shown in Figures 63 and 64.

The back calculated P-y curves show the $k_h$ increased with depth; however the decrease in $k_h$ from 2.6 m to 3.0 m for the long shaft should be noted. This could be attributed to changes in rock properties that were not discovered in the subsurface investigation or in error introduced in the back calculation process, partly due to small deflections around the point of rotation (Cho, 2002).
Transformed axes plots were used to curve fit the back calculated P-y data, as presented in Chapter 2. These plots were used to establish the values of $k_h$ and $P_{ult}$ for the back calculated P-y curves. Two examples are shown in Figures 65 and 66, the remainder is presented in the Appendix.

![Figure 63. Back Calculated P-y Curves for the Weathered Rock – I-85 Short Shaft](image-url)
Figure 64. Back Calculated P-y Curves for the Weathered Rock – I-85 Long Shaft

Figure 65. Curve Fitting Results – I-85 Short Shaft
3.3.5.4 Predicted and Back Calculated P-y Curves

Figures 67 through 73 are shown for comparison of P-y curves generated using the new model and those back calculated from measured strain and deflection data. Because there was little variation in $k_h$ and $P_{ult}$ in the divided subsurface profiles, one predicted P-y curve is shown for each layer. For clarity the graphs are plotted on differing scales.

As was realized in the I-40 load test results, the Weathered Rock Model seems to under predict available resistance near the ground surface and over predict at deeper depths. However, there seems to be an overall balance based on the reasonably good comparison between predicted and measured shaft deflections above the ground surface.
Figure 67. Predicted and Back Calculated P-y Curves – I-85 Short Shaft Layer 1

Figure 68. Predicted and Back Calculated P-y Curves – I-85 Short Shaft Layer 2
Figure 69. Predicted and Back Calculated P-y Curves – I-85 Short Shaft Layer 3

Figure 70. Predicted and Back Calculated P-y Curves – I-85 Long Shaft Layer 1
Figure 71. Predicted and Back Calculated P-y Curves – I-85 Long Shaft Layer 2

Figure 72. Predicted and Back Calculated P-y Curves – I-85 Long Shaft Layer 3
3.4 Distribution of the Subgrade Reaction ($k_h$)

The subgrade reaction ($k_h$) can be determined by evaluating the initial slope of a P-y curve; the coefficient of subgrade reaction ($k_{ho}$) is then calculated by dividing $k_h$ by the diameter of the test shaft. Cho (2002) presented Figure 74, the distribution of $k_{ho}$ for the field tests used in the development of the Weathered Rock Model. Large increases in $k_{ho}$ realized below the point of rotation were explained to be due to the resistance of the passive wedge behind the shaft. Figure 75 presents the distribution of $k_{ho}$ evaluated from the verification load tests.
Figure 74. Measured $k_{ho}$ Values from Field Tests (Cho, 2002)

Figure 75. Measured $k_{ho}$ from Verification Tests
Large increases in $k_{ho}$ below the point of rotation, as seen in Figure 75, were not realized from the verification load test results. This can be rationalized knowing that Triassic Weathered Rock will slake (degrade in strength) in the presence of water. Figure 76 combines data from the Weathered Rock Model development and verification testing in order to clarify the magnitude of increase in $k_h$ below the point of rotation realized in Triassic Weathered Rock. Again, lower strength magnitudes at the verification test sites as compared with the model development sites are hypothesized to be the reason for the values of the $k_h$ Number measured. Also, it is possible that these measurements are linked to some amount of strain softening as a result of large deflections measured at deeper depths for the verification tests.

Figure 76. $K_h$ Number for Depths below the Point of Rotation ($I_T$) – for Triassic Weathered Rock

Based on Figure 76, the following equation is provided for the estimation of the $k_h$ Number ($I_T$) when Triassic Weathered Rock is encountered.
\[ I_T = 1.5 - 8 \log \left( \frac{T_o}{L} \right) \]  \hspace{1cm} (41)

3.5 Proposed Design Procedures

Based on the new P-y curve model and the verification testing presented in this chapter, the following procedures for the analysis of drilled shafts embedded in Triassic Weathered Rock are advanced.

3.5.1 Design of Laterally Loaded Drilled Shafts using Dilatometer Data

The following recommendations and procedures are based on the Weathered Rock Model published by Cho (2002), and the verification testing described in this thesis report. The recommended design parameters (for the Triassic Weathered Rock) are proposed on the basis of the results of the verification testing and analyses. This procedure utilizes data from rock dilatometer testing.

Step 1: Calculation of GSI value

GSI is the summation of the ratings for the five parameters outlined in Table 2. Each parameter: strength of intact rock material, RQD, spacing of joints, condition of joints, and groundwater level, is given a rating based on available in-situ data. If sufficient data are unavailable, especially for spacing and condition of joints, ratings on the basis of measured RQD values can be used (for example, if RQD = 80%, RQD rating = 17, Spacing of Joints rating = 25, Condition of Joints rating = 20). A groundwater rating of 10 was always used for the verification testing predictions. GSI values used for the predictions for both I-40 and I-85 load tests are listed in Tables 9 and 16.
Step 2: Calculation of Weathered Rock Modulus of Elasticity

The modulus of elasticity is expressed as follows:

$$E_s(GPa) = \frac{\sigma_{ci}}{100} \times 10^{(GSI-10)/40}$$  \hspace{1cm} (42)

where, $\sigma_{ci}$ = compressive strength of rock (GPa).

When multiple layers of weathered rock are encountered, the modulus of elasticity for each layer should be calculated, and then a representative value for the entire profile can be determined from a weighted average.

Step 3: Calculation of Flexibility Factor

A flexibility factor is computed as follows:

$$K_R = \frac{E_p I_p}{E_s L^3}$$  \hspace{1cm} (43)

where, $E_p$ = modulus of elasticity of pile,
$I_p$ = moment of inertia of pile,
$L$ = length of pile embedded in weathered rock.

Step 4: Calculation of the Point of Rotation

The following equation is used to define the turning point as a function of the embedded shaft length:

$$\frac{T_o}{L} = 1 + 0.18 \log K_R$$  \hspace{1cm} (44)

where, $T_o$ = turning point,
$L$ = embedded length of shaft.

Step 5: Calculation of the $k_h$ Number

Once $T_o$ is estimated from step 4, the $k_h$ Number for depths below the point of rotation can be determined as follows:

$$I_T = -28 - 383 \log \left( \frac{T_o}{L} \right)$$  \hspace{1cm} (45)
Available data suggests a separate relationship for $k_h$ Number in Triassic Weathered Rock. Equation 41 may be used when designing for Triassic Weathered Rock; however, caution should be used due to the amount of data available for the development of this equation.

**Step 6: Calculation of Coefficient of Subgrade Reaction**

For rock dilatometer test data, the coefficient of subgrade reaction can be calculated as follows (another procedure is presented later if only geologic parameters are available):

$$k_{ho} = 2(1 + \nu_r) \times (v_o + v_m) \times \frac{1}{(\Delta p - \Delta p_i) - c} \text{ (kN/m}^3\text{)} \tag{46}$$

where, $v_o$ = normal initial or at rest volume of the deflated probe (1,950 cc; for the ROCTEST Model Probex 1)
$v_m$ = mean additional volume
$\mu_r$ = Poisson’s ratio of membrane (0.3)
$\Delta p_i$ = change of the pressure of the dilatable membrane (kPa)
$\Delta p$ = applied pressure increment (kPa)
$c$ = volume correction factor ($7.878 \times 10^{-4}$ cc/kPa; for the ROCTEST Model Probex 1)

By performing multiple dilatometer tests within the weathered rock profile a distribution of $k_{ho}$ with depth can be generated.

**Step 7: Calculation of the Subgrade Reaction**

$$k_h = (k_{ho})B \ (0 \leq z \leq T_o) \tag{47}$$
$$k_h = (k_{ho})T_zB \ (T_o < z \leq L) \tag{48}$$

**Step 8: Calculation of the Normal Limit Stress**

$$p_L = \gamma^' z + \sigma_{ci} \left( m_b \frac{\gamma^' z}{\sigma_{ci}} + s \right)^a \tag{49}$$
where, $\gamma' = \text{effective unit weight of the rock mass, kN/m}^3$,

$z = \text{depth from the rock mass surface, m},$

$\sigma_{ci} = \text{compressive strength of the rock (kPa),}$

$m_b, S, \text{and } a = \text{coefficients based on GSI from Table 3}.$

**Step 9: Calculation of the Shearing Resistance along the sides of a Drilled Shaft**

The side shear resistance is calculated based on the compressive strength of rock as follows:

$$\tau_{\text{max}} = 0.20\sqrt{\sigma_{ci}} \text{ (MPa)}$$  \hspace{1cm} (50)

**Step 10: Calculation of the Ultimate Resistance**

Based on $p_L$ and $\tau_{\text{max}}$, the $P_{\text{ult}}$ is computed as:

$$P_{\text{ult}} = (p_L + \tau_{\text{max}})B$$  \hspace{1cm} (51)

**Step 11: Construction of the P-y Curve**

Once, $k_h$ and $P_{\text{ult}}$ are evaluated, the P-y curves are constructed using the following hyperbolic equation:

$$P = \frac{y}{\left(\frac{1}{k_h}\right) + \left(\frac{y}{P_{\text{ult}}}\right)}$$  \hspace{1cm} (52)

Any number of P-y curves can be developed throughout the profile depending on the density of curves desired for an analysis.

### 3.5.2 Design of Laterally Loaded Drilled Shafts using Geologic Data

Geologic data are used together with a set of empirical equations to calculate the coefficient of subgrade reaction of weathered rock. The geologic method can be used in place of dilatometer data; however, results of the verification testing suggest that the empirical equations do not model the insitu properties as accurately as the rock dilatometer. For the design of laterally loaded drilled shafts using geologic data, Steps 1
through 5 are carried out as described above, followed by Steps 6 through 8 presented below.

**Step 6: Calculation of the Coefficient of Subgrade Reaction**

In this case, \( k_{ho} \) is computed as a function of \( \sigma_{ci} \) and GSI as follows:

\[
k_{ho} (kN/m^3) = \sqrt[40]{\frac{GSI}{\sigma_{ci}x10^3}}10^{GSI-10}
\]

Note: GSI reduction factor, \( \alpha_{GSI} \), should be used for Triassic Weathered Rock; rational is presented below.

**Step 7: Calculation of the Distribution of the Coefficient of the Subgrade Reaction**

\[
n_\delta = \frac{2E_pL_p}{k_{ho}L^3} \times 10^5
\]

where \( k_{ho} = \) coefficient of subgrade reaction for weathered rock at surface (kN/m^3)

\( L = \) embedded shaft length, m

**Step 8: Calculation of the Modulus of Subgrade Reaction**

The magnitude of the modulus of subgrade reaction is estimated based on the location of the turning point as follows:

\[
k_h = (k_{ho} + n_\delta z)B \quad (0 \leq z \leq T_o)
\]

\[
k_h = [(k_{ho} + n_\delta T_o) + n_\delta(z - T_o)]B \quad (T_o < z \leq L)
\]

Equations 49, 50, and 51 are proposed to calculate the ultimate resistance of the weathered rock. Equation 52 is proposed to construct P-y curves for any values of \( k_h \) and \( P_{ult} \).

For the Triassic Weathered Rock tested in this research program, the geologic model, as described above, generally under predicted pile head deflections with exception of the I-85 Long Shaft. Based on these results it is proposed to adjust GSI values by a reduction factor such that the geologic model matches or consistently and conservatively predicts pile head deflections. Table 19 presents GSI values for each verification test.
shaft as determined using the method described in Section 3.5.1 (Step 1); to the left of these values, in parenthesis, are the GSI values used to estimate shaft deflections that closely represented those measured. GSI reduction factors ($\alpha_{\text{GSI}}$) are determined from the ratio between the two values and are presented in Figure 77. Figures 78 through 81 are presented for comparison of measured test results and recommended design procedures. The “Recommended-Dilatometer” curves are the same as those presented as “Predicted-Dilatometer” in previous sections. “Recommended-Geologic Based (Class B)” were developed using the GSI reduction factor, $\alpha_{\text{GSI}}$.

Table 19. GSI Values for the Verification Load Tests

<table>
<thead>
<tr>
<th>Layer No.</th>
<th>Parameter</th>
<th>I-40 Load Tests</th>
<th>I-85 Load Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Short Shaft</td>
<td>Long Shaft</td>
</tr>
<tr>
<td></td>
<td>Strength</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>1</td>
<td>RQD</td>
<td>20</td>
<td>13</td>
</tr>
<tr>
<td></td>
<td>Spacing</td>
<td>30</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>Condition</td>
<td>25</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>Groundwater</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td><strong>Total GSI</strong></td>
<td>(77) 87 (0.89)</td>
<td>(41) 57 (0.72)</td>
</tr>
<tr>
<td>2</td>
<td>Strength</td>
<td>2</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>RQD</td>
<td>17</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>Spacing</td>
<td>25</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>Condition</td>
<td>20</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>Groundwater</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td><strong>Total GSI</strong></td>
<td>(61) 74 (0.82)</td>
<td>(79) 89 (0.89)</td>
</tr>
<tr>
<td>3</td>
<td>Strength</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>RQD</td>
<td>17</td>
<td>17</td>
</tr>
<tr>
<td></td>
<td>Spacing</td>
<td>25</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>Condition</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>Groundwater</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td><strong>Total GSI</strong></td>
<td>(63) 76 (0.83)</td>
<td>(63) 76 (0.83)</td>
</tr>
<tr>
<td>4</td>
<td>Strength</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>RQD</td>
<td>17</td>
<td>17</td>
</tr>
<tr>
<td></td>
<td>Spacing</td>
<td>25</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>Condition</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>Groundwater</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td><strong>Total GSI</strong></td>
<td>(61) 74 (0.82)</td>
<td>(61) 74 (0.82)</td>
</tr>
</tbody>
</table>
Figure 77. GSI Reduction Factor, $\alpha_{GSI}$, for Triassic Weathered Rock
Figure 78. I-40 Short Shaft Pile Head Deflections with Recommendations

Figure 79. I-40 Long Shaft Pile Head Deflections with Recommendations
Figure 80. I-85 Short Shaft Pile Head Deflections with Recommendations

Figure 81. I-85 Long Shaft Pile Head Deflections with Recommendations
The results of the verification testing discussed in the preceding sections have proven that the rock dilatometer provides the most accurate means of predicting insitu modulus when estimating drilled shaft behavior with the Weathered Rock Model. While the proposed geologic parameters produced conservative results, the designing engineer should use good judgment based on all of the material presented in this report when analyzing laterally loaded drilled shafts with geologic data.

3.6 Potential Cost Savings

A significant cost saving can be realized by the NCDOT and other agencies by implementing the Weathered Rock Model for design analysis. As described previously, the NCDOT has standard material properties for analyzing drilled shafts embedded in weathered rock profiles. NCDOT uses the Stiff Clay Model (Reese, Cox, and Koop, 1975) with the following engineering properties representing weathered rock: Cohesion = 200 kPa, $k = 543,000$ kN/m$^3$, and $\varepsilon_{50} = 0.004$. As a means for quantifying the potential cost savings, the I-85 Long Shaft was reanalyzed with the Stiff Clay Model to see how long it needed to be to match the measured top deflection (0.0172 m) at the maximum applied load of 1334 kN. Using the Stiff Clay Model and the material properties listed above, the shaft would need to be embedded over 30 m to approximately match the measured deflection. As mentioned before, the actual embedded length of the I-85 Long Shaft was 4.21 m. Obviously, with a difference in length of 25.79 m a very significant cost savings could be realized by the implementation of the Weathered Rock Model.

3.7 Inclusion of the Weathered Rock Model in the Computer Program

LTBASE (Borden and Gabr, 1987)

The computer program LTBASE was developed for the analysis of laterally loaded drilled shafts with slope and base effects at North Carolina State University by Borden and Gabr (1987). With the development of the Weathered Rock Model, a subroutine was added to the computer code to allow engineers to use the this method when designing for weathered rock profiles. The program allows for layered profiles of soil and weathered rock. In addition to the Weathered Rock Model, the program code
contains many of the other popular design models for soils. Table 19 presents the form of input file used in LTBASE; steps for analysis when using the Weathered Rock Model follow along with a description of the input variables presented in the Appendix.

### Table 20. LTBASE Input File Format

<table>
<thead>
<tr>
<th>Input File Format</th>
<th>Input Variables</th>
</tr>
</thead>
<tbody>
<tr>
<td>ANALYSIS OF SHAFT</td>
<td>TITLE</td>
</tr>
<tr>
<td>NCDOT Example</td>
<td>PROJECT NUMBER</td>
</tr>
<tr>
<td>NCDOT</td>
<td>PROJECT LOCATION</td>
</tr>
<tr>
<td>Initials</td>
<td>OPERATOR NAME</td>
</tr>
<tr>
<td>10/11/02</td>
<td>DATE</td>
</tr>
<tr>
<td>0</td>
<td>NOPTION</td>
</tr>
<tr>
<td>20. 0.0 1 1.5</td>
<td>PT,BC2,KODE,FSCR</td>
</tr>
<tr>
<td>30. 0.2 .001 4.0 100 5</td>
<td>D,H,TOL,DEFCR,N,NU,NTYPE*,NCHOICE,IPRINT,IOUT</td>
</tr>
<tr>
<td>15.4 .319E+12</td>
<td>TP, EIP</td>
</tr>
<tr>
<td>0. 0.</td>
<td>THETA,THETAU</td>
</tr>
<tr>
<td>1</td>
<td>NX</td>
</tr>
<tr>
<td>30.0 30. 159.1 35.0 9.</td>
<td>TH(1),DIA(1),GAM(1),FPHI(1),SK(1),CSHO(1),EP50(1),NPC(1)</td>
</tr>
<tr>
<td>1</td>
<td>I</td>
</tr>
<tr>
<td>.319E+12 20.0</td>
<td>RR(J), XX(J)</td>
</tr>
</tbody>
</table>

A description of the input variables is presented in the Appendix.

### 3.7.1 Steps for LTBASE Analysis

Once the input file has been created, the following steps should be used when analyzing laterally loaded drilled shafts with the Weathered Rock Model.

1. Determine the initial depth to the point of rotation using the concepts of the Weathered Rock Model or simply assume.
2. Perform an initial run of the LTBASE program using the input file with the initial depth to the point of rotation.
3. Evaluate the depth to the point of rotation from output file (for the load increment of interest).
4. Update the input file with the new depth to the point of rotation.
5. Repeat process until depth to the point of rotation from the output matches that in the input file.
### 3.8 Summary of Verification Testing

The Weathered Rock Model for designing drilled shafts embedded in weathered rock profiles published by Cho (2002) was used to develop class A performance predictions of four drilled shafts in weathered rock of the Durham Triassic Basin. Performance predictions were also created using two other popular models for the design of laterally loaded drilled shafts, Reese’s Method for P-y Curves for Weak Rock (Reese, 1997) and the Stiff Clay Model (Reese, Cox, and Koop, 1975). Results of the verification testing show that the Weathered Rock Model can predict the behavior of laterally loaded drilled shafts fairly reasonably. A new equation for the estimation of the $k_h$ Number ($I_T$) in Triassic Weathered Rock is suggested; however, further data are needed to establish its validity. Two recommended design procedures based on the Weathered Rock Model (Cho, 2002) are revised and presented based on results of verification testing, one utilizing rock dilatometer data and another using empirical equations and geologic parameters.
CHAPTER 4. SUMMARY AND CONCLUSIONS

The literature review presented two popular methods for the design of laterally loaded drilled shafts in weathered rock profiles. Reese’s Method for the development of P-y curves for weak rock (Reese, 1997) was termed “interim” due to the limited amount of data available for development. The Stiff Clay Model (Reese, Cox, and Koop, 1975) is a popular method used when weathered rock is encountered and is known to produce conservative results when material properties standardized by the NCDOT are utilized. The Weathered Rock Model published by Cho (2002) utilizes either rock dilatometer data or a set of empirical equations with geologic markers to establish the subgrade reaction ($k_h$).

Class A performance predictions of four full-scale test shafts were used to verify the Weathered Rock Model. Performance predictions were developed using the three methods mentioned above, and compared with the measured results of the four load tests. The verification test sites were located within the Durham Triassic Basin of North Carolina. The weathered rock found within the basin is known for slaking and degradation of strength in the presence of water. Based on the results of the load tests the following conclusions are advanced:

1. The Weathered Rock Model predicts the behavior of laterally loaded drilled shafts embedded in Triassic Weathered Rock more accurately than the other methods presented in this report. (Note: Reese’s Method was used with laboratory and field measured properties, the Stiff Clay Model was used with material properties standardized by the NCDOT)
2. The rock dilatometer provides the most accurate estimation of the subgrade reaction.
3. When evaluating the coefficient of subgrade reaction with empirical equations and geologic markers, for Triassic Weathered Rock, GSI values should be multiplied by the GSI reduction factor, $\alpha_{GSI}$.
4. The increase in $k_h$ below the point of rotation for drilled shafts embedded in Triassic Weathered Rock is not as significant as that realized in the types of
material encountered during field testing for the development of the Weathered Rock Model.
CHAPTER 5. RECOMMENDATIONS FOR FUTURE RESEARCH

Based on the results of the research present in this report, the following recommendations for future research are advanced:

1. Perform rock dilatometer testing to increase the database created by Cho (2002) of the coefficient of subgrade reaction for various types of weathered rock. This would greatly reduce the cost associated with designing laterally loaded drilled shafts when the Weathered Rock Model is utilized.

2. Explore the relationship between the $k_0$ Number and the ratio $T_o/L$ for Triassic Weathered Rock.

3. Explore the possibility of adapting the new P-y curve model to other types of materials, e.g. more competent rock.

4. As a result of the increased capacity of weathered rock as related to lateral loaded realized by implementation of the Weathered Rock Model, it is a possibility that axial limitations placed on the material may be design-limiting factors. Therefore, research into the axial capacity of weathered rock may be warranted.
REFERENCES


APPENDIX

Figure A-1. Curve Fitting Result for Field Tests  – I-40 Short Shaft

Figure A-2. Curve Fitting Result for Field Tests  – I-40 Short Shaft
Figure A-3. Curve Fitting Result for Field Tests – I-40 Short Shaft

Figure A-4. Curve Fitting Result for Field Tests – I-40 Short Shaft
Figure A-5. Curve Fitting Result for Field Tests – I-40 Short Shaft

Figure A-6. Curve Fitting Result for Field Tests – I-40 Short Shaft
Figure A-7. Curve Fitting Result for Field Tests – I-40 Short Shaft

Figure A-8. Curve Fitting Result for Field Tests – I-40 Short Shaft
Figure A-9. Curve Fitting Result for Field Tests – I-40 Short Shaft

Figure A-10. Curve Fitting Result for Field Tests – I-40 Long Shaft
Figure A-11. Curve Fitting Result for Field Tests – I-40 Long Shaft

Figure A-12. Curve Fitting Result for Field Tests – I-40 Long Shaft
Figure A-13. Curve Fitting Result for Field Tests – I-40 Long Shaft

Figure A-14. Curve Fitting Result for Field Tests – I-40 Long Shaft
Figure A-15. Curve Fitting Result for Field Tests – I-40 Long Shaft

Figure A-16. Curve Fitting Result for Field Tests – I-40 Long Shaft
Figure A-17. Curve Fitting Result for Field Tests – I-40 Long Shaft

Figure A-18. Curve Fitting Result for Field Tests – I-40 Long Shaft
Figure A-19. Curve Fitting Result for Field Tests – I-40 Long Shaft

Figure A-20. Curve Fitting Result for Field Tests – I-40 Long Shaft
Figure A-21. Curve Fitting Result for Field Tests – I-40 Long Shaft

Figure A-22. Curve Fitting Result for Field Tests – I-85 Short Shaft
Figure A-23. Curve Fitting Result for Field Tests – I-85 Short Shaft

Figure A-24. Curve Fitting Result for Field Tests – I-85 Short Shaft
Figure A-25. Curve Fitting Result for Field Tests – I-85 Short Shaft

Figure A-26. Curve Fitting Result for Field Tests – I-85 Short Shaft
Figure A-27. Curve Fitting Result for Field Tests – I-85 Short Shaft

Figure A-28. Curve Fitting Result for Field Tests – I-85 Short Shaft
Figure A-29. Curve Fitting Result for Field Tests – I-85 Short Shaft

Figure A-30. Curve Fitting Result for Field Tests – I-85 Short Shaft
Figure A-31. Curve Fitting Result for Field Tests – I-85 Long Shaft

Figure A-32. Curve Fitting Result for Field Tests – I-85 Long Shaft
Figure A-33. Curve Fitting Result for Field Tests – I-85 Long Shaft

Figure A-34. Curve Fitting Result for Field Tests – I-85 Long Shaft
Figure A-35. Curve Fitting Result for Field Tests – I-85 Long Shaft

Figure A-36. Curve Fitting Result for Field Tests – I-85 Long Shaft
Figure A-37. Curve Fitting Result for Field Tests – I-85 Long Shaft

Figure A-38. Curve Fitting Result for Field Tests – I-85 Long Shaft
Figure A-39. Curve Fitting Result for Field Tests – I-85 Long Shaft

Figure A-40. Curve Fitting Result for Field Tests – I-85 Long Shaft
Figure A-41. Curve Fitting Result for Field Tests – I-85 Long Shaft

Figure A-42. Curve Fitting Result for Field Tests – I-85 Long Shaft
Figure A-43. Deflection Profiles before Dial Gage Adjustment – I-40 Short Shaft

Figure A-44. Deflection Profiles before Dial Gage Adjustment – I-40 Long Shaft
Figure A-45. Deflection Profiles before Dial Gage Adjustment – I-85 Short Shaft

Figure A-46. Deflection Profiles before Dial Gage Adjustment – I-85 Long Shaft

130
Description of Input Variables for LTBASE Computer Program (Borden and Gabr, 1987) with inclusion of the Weathered Rock Model (Cho, 2002)

<table>
<thead>
<tr>
<th>LTBASE input file</th>
<th>Description:</th>
</tr>
</thead>
<tbody>
<tr>
<td>ANALYSIS OF SHAFT</td>
<td>TITLE</td>
</tr>
<tr>
<td>NCDOT Example</td>
<td>PROJECT NUMBER</td>
</tr>
<tr>
<td>NCDOT</td>
<td>PROJECT LOCATION</td>
</tr>
<tr>
<td>Initials</td>
<td>OPERATOR NAME</td>
</tr>
<tr>
<td>10/11/02</td>
<td>DATE</td>
</tr>
<tr>
<td>0</td>
<td>NOPTION</td>
</tr>
<tr>
<td>20. 0.0 l 1.5</td>
<td>PT,BC2,KODE,FSCR</td>
</tr>
<tr>
<td>30. 0.2 .001 4.0 100 5 -1 l l 1</td>
<td>D,H,TOL,DEFCR,N,NU,NTYPE*,NCHOICE,IPRINT,IOUS</td>
</tr>
<tr>
<td>15.4 .319E+12</td>
<td>TP, EIP</td>
</tr>
<tr>
<td>0. 0.</td>
<td>THETA,THETAU</td>
</tr>
<tr>
<td>l</td>
<td>NX</td>
</tr>
<tr>
<td>30.0 30. 159.1 35.0 9. 3626. .000 -4</td>
<td>TH(1),DIA(1),GAM(1),FPHI(1),SK(1),CSHO(1),EP50(1),NPC(1)</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>.319E+12 20.0</td>
<td>RR(J), XX(J)</td>
</tr>
</tbody>
</table>

Lines 1-5: General Information

<table>
<thead>
<tr>
<th>ANALYSIS OF SHAFT</th>
<th>TITLE</th>
</tr>
</thead>
<tbody>
<tr>
<td>NCDOT Example</td>
<td>PROJECT NUMBER</td>
</tr>
<tr>
<td>NCDOT</td>
<td>PROJECT LOCATION</td>
</tr>
<tr>
<td>Initials</td>
<td>OPERATOR NAME</td>
</tr>
<tr>
<td>10/11/02</td>
<td>DATE</td>
</tr>
</tbody>
</table>

Line 6: Analysis Option

| 0                 | NOPTION, =1 Length is internally incremented |
|                   | =0 Single run analysis |

Line 7: Loading Conditions

| 20. 0.0 l 1.5     | PT,BC2,KODE,FSCR |
|                   | PT = Initial lateral load to be applied at top of shaft, (kips) |
|                   | BC2 = Moment from shear force, (kip-ft) if KODE = 1 |
|                   | Slope, (in/in) if KODE = 2 |
|                   | Moment/slope, (kip-ft) if KODE = 3 |
|                   | KODE = Code to control boundary condition at top of shaft |
|                   | FSCR = Limiting factor of safety criterion. |
Line 8: Shaft dimensions and analysis control

<table>
<thead>
<tr>
<th>30. 0.2 .001 4.0 100 5 -1 1 1</th>
<th>D,H,TOL,DEFCR,N,NU,NTYPE,NCHOICE,IPRINT,IOUT</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D = Shaft diameter at the ground surface. (inches)</td>
</tr>
<tr>
<td></td>
<td>H = Total length of the pier / No. of increments (N), (feet) (100 maximum)</td>
</tr>
<tr>
<td></td>
<td>TOL = Tolerance of solution convergence, recommended value 0.001</td>
</tr>
<tr>
<td></td>
<td>DEFCR = Allowable deflection value at the top of the shaft, (inches)</td>
</tr>
<tr>
<td></td>
<td>N = No. of increments into which the shaft is divided.</td>
</tr>
<tr>
<td></td>
<td>NU = No. of pier increments above the ground surface</td>
</tr>
<tr>
<td></td>
<td>NTYPE = Analysis option, 0 for SOIL case and -1 for Weathered Rock Model</td>
</tr>
<tr>
<td></td>
<td>NCHOICE = P-y curve generation option:</td>
</tr>
<tr>
<td></td>
<td>= 1, the program generates P-y curves initially.</td>
</tr>
<tr>
<td></td>
<td>= 0, user inputs P-y curves.</td>
</tr>
<tr>
<td></td>
<td>IPRINT = 1, P-y curves are printed internally by the program</td>
</tr>
<tr>
<td></td>
<td>= 0, printing of P-y curves is suppressed.</td>
</tr>
<tr>
<td></td>
<td>IOUT = 1, Output file &quot;*.OUT&quot; is printed.</td>
</tr>
<tr>
<td></td>
<td>= 0, Printing of &quot;*.OUT&quot; is suppressed</td>
</tr>
</tbody>
</table>

Line 9: Input depth to point of rotation and EI

<table>
<thead>
<tr>
<th>15.4 .319E+12</th>
<th>TP, EIP</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP = Input depth to point of rotation from ground surface (feet)</td>
<td></td>
</tr>
<tr>
<td>EIP = Shaft stiffness, (psi)</td>
<td></td>
</tr>
</tbody>
</table>

Line 10: Slope analysis option

<table>
<thead>
<tr>
<th>0. 0.</th>
<th>THETA,THETAU</th>
</tr>
</thead>
<tbody>
<tr>
<td>THETA = Slope angle of the ground surface in the front of shaft, (degrees).</td>
<td></td>
</tr>
<tr>
<td>THETAU = Slope angle of the ground surface in the back of shaft, (degrees).</td>
<td></td>
</tr>
</tbody>
</table>

Line 11: Input and generation of P-y curves

| 1 | NX = Number of the layers in the subsurface profile to be analyzed, if NCHOICE = 1 |
### Line 12: Soil/rock properties

| 30.0 30.1 159.1 35.0 9. 3626 .000 -4 | TH(1),DIA(1),GAM(1),FPHI(1),SK(1),CSHO(1),EP50(1),NPC(1)  
| | TH(2),DIA(2),GAM(2),FPHI(2),SK(2),CSHO(2),EP50(2),NPC(2)  
| | TH(3),DIA(3),GAM(3),FPHI(3),SK(3),CSHO(3),EP50(3),NPC(3)  
| **For soil:** |  
| *TH(K)* = Distance from ground surface to the end of the layer (feet)  
| *DIA(K)* = Diameter of shaft at the mid-height of the layer (inches)  
| *GAM(K)* = Effective or total unit weight of soil at the mid-height of the layer (pcf)  
| *FPHI(K)* = Angle of internal friction soil at the mid-height of the layer, (degrees).  
| *SK(K)* = Coefficient of lateral subgrade reaction at the mid-height of the layer, (pci).  
| *CSHO(K)* = Undrained shear strength of the soil at the mid-height of the layer, (psi)  
| *EP50(K)* = Strain corresponding to 50% stress level at the mid-height of the layer  
| *NPC(K)* = Code to control the type of P-y curves to be generated:  
| **For rock:** |  
| *TH(K)* = Distance from ground surface to the end of layer (feet).  
| *DIA(K)* = Diameter of shaft at the mid-height of the layer (inches).  
| *GAM(K)* = Effective or total unit weight of rock at the mid-height of the layer (pcf).  
| *FPHI(K)* = GSI value of rock at the mid-height of the layer.  
| *SK(K)* = m value of rock at the mid-height of the layer.  
| *CSHO(K)* = Unconfined compressive strength of rock at the mid-height of the layer, (psi).  
| *EP50(K)* = 0.00 for the rock sub-layer  
| *NPC(K)* = Code to control the type of P-y curves to be generated:  

#### Line 13: Pier stiffness

1  

1 = Number of different shaft cross-sections

#### Line 14: Pier stiffness (2)

| .319E+12 20.0 | RR(J), XX(J)  
| | RR (J) = EI value (psi)  
| | XX (J) = Depth from top of shaft to point where cross-section changes (feet)
IMPORTANT NOTE: In the output file depth is referenced from the top of the shaft. However, in the input file depth to the point of rotation is referenced for the ground surface. Save input file with .dat extension. Load vs. deflection results are given in the *.prn file.