Post Grouting Drilled Shaft Tips

Phase II

Principal Investigator: Gray Mullins

Research Associate: Danny Winters
Department of Civil and Environmental Engineering

June 2004

DISCLAIMER

The opinions, findings and conclusions expressed in this publication are those of the authors and not necessarily those of the State of Florida Department of Transportation.

CONVERSION FACTORS, US CUSTOMARY TO METRIC UNITS

Multiply	***by***	***to obtain***
inch	25.4	mm
foot	0.3048	meter
square inches	645	square mm
cubic yard	0.765	cubic meter
pound (lb)	4.448	Newtons
kip (1000 lb)	4.448	kiloNewton (kN)
Newton	0.2248	pound
kip/ft	14.59	kN/meter
pound/in^2	0.0069	MPa
kip/in^2	6.895	MPa
MPa	0.145	ksi
kip-ft	1.356	kN-m
kip-in	0.113	kN-m
kN-m	.7375	kip-ft

PREFACE

This research project was funded as a supplemental contract awarded to the University of South Florida, Tampa by the Florida Department of Transportation. Mr. Peter Lai was the Project Manager. Again, it is a pleasure to acknowledge his contribution to this study.

This project was carried out in part with the cooperation and collaboration of Auburn University and the University of Houston. The contributions provided by these institutions are greatly appreciated with particular acknowledgment to Dr. Dan Brown and Dr. Michael O'Neill, respectively.

Interest expressed by State and Federal Agencies such as Georgia DOT, Texas DOT, Mississippi DOT, South Carolina DOT, Arkansas DOT, Alabama DOT, Cal Trans, and the FHWA Eastern Federal Lands Bureau is sincerely appreciated.

Likewise, the principal investigator is indebted to the vast interaction afforded by Applied Foundation Testing, Beck Foundation, and Trevi Icos South. Therein, the opportunity to interact with prime contractors and provide input regarding post-grouted shaft alternatives was made possible by: Michael Muchard and Don Robertson; Keith Anderson and Gus Beck; and Bud Khouri and Michael Rossie, respectively.

Special thanks are extended to the College of Engineering's Machine Shop staff of Mr. Bob Smith, Mr. Bryan O'Steen, Mr. James Christopher, and Mr. Tom Gage for their unending support. The assistance of graduate researchers (*not in any order*) Mr. Byron Anderson, Mr. Michael Stokes, Mr. Van Wagner and Mr. Kevin Johnson as well as undergraduate assistant Mr. Ryan Florence is duly recognized.

Further, those companies that provided potential sites/projects for review as well as those companies that seriously considered the use of a post grouted shaft alternative are also appreciated. Those firms include: Archer Western, Astaldi Construction, Bauer / Coastal Caisson, Bechtel Engineering and Testing, Case Atlantic, Case Foundation, HDR Engineering, Kewitt Construction, Law Engineering, Melick-Tully & Associates, Morris Shea Bridge Company, PSI, Rossi, Schnabol Engineering, Traylor Bros., TreviIcos, Union Pacific Railroad, Williams Bros., Zachary Construction.

EXECUTIVE SUMMARY

This report presents results from a two-year supplemental study to further the understanding of the effects of post-grouting drilled shaft tips. As the project was an extension of previously concluded research, this document is differentiated by the denotation *Phase II.* The objectives of this phase of the study included: (1) review of construction sites where post grouted shafts may be a plausible alternative, (2) instrument and monitor grout tests and load tests of grouted and ungrouted shafts, (3) develop design software for shafts with post grouted tips, and (4) update database of post grouted shaft performances.

Review of Sites. Twenty five sites were reviewed for the applicability to post grouted drilled shaft tips. These sites spanned 7 states and involved 8 State or Federal agencies. In each case, boring logs were made available by either the contractor or the overseeing agency from which design curves were generated. All design curves showed both the ungrouted and grouted capacity of drilled shafts. In many cases, multiple diameter options were requested. Predicted improvement from using post grouted over conventional shafts ranged from 195 to 1266% (avg. 578%) for sites with shafts tipped in sand and 3 to 91% (avg.37%) for sites with shafts tipped in clay. These improvements were determined at a permissible displacement.

Post Grouted Shaft Projects. The entire research project involved the instrumentation, grouting, and load testing of 8 full scale test programs. These programs involved 26 test shafts tipped in sand, clay, and silt. Improvements ranged from 41 to 743% in various soil types and consistencies. Further, the response of 174 production shafts were recorded via field survey notes and reviewed for quality assurance. Therein, the satisfactory performance of every post grouted shaft was verified.

Design Software. A software capable of predicting the capacity of both conventional and post grouted shafts was developed called *Shaft 1-2-3.* The software makes use of the correlations developed in Phase I to determine the post grouted end bearing. Those correlations were subsequently updated with new data sets made available by the Phase II efforts. The software user has the option of several methods of determining side shear as well as convention end bearing capacity. All predicted capacity values (both grouted and ungrouted) are displacement dependent and not ultimate.

Updates and Recommendations. Information obtained from post grouted shafts tipped in clay and silt provided rationale for capacity predictions in those soils. Likewise, information obtained from additional tests in sand (over and above Phase I) was used to update the correlations developed in Phase I. Given the variance in end bearing that can be observed for conventional shafts, it was recommended that the design approach developed in Phase I be updated to not only include the new data (a primary objective of the supplemental study) but also to re-evaluate the use of the control shaft data as the basis for the improvement ratio (TCM). Thereby, the grouted shaft response which is relatively unaffected by construction variations would be compared to the predicted end bearing based on boring log information (preferably the Reese and O'Neill, 1988 method). This increases the breadth of the sandy soils encountered while also better representing various construction practices.

This page is intentionally blank.

TABLE OF CONTENTS

This page is intentionally blank.

LIST OF TABLES

This page is intentionally blank.

LIST OF FIGURES

This page is intentionally blank.

1. INTRODUCTION

1.1 Background

In 1999, the Florida Department of Transportation (FDOT) contracted the University of South Florida to embark on a two year study (herein denoted as Phase I) to assess the viability of using pressure grouting at the tip of drilled shafts. This end bearing modification technique, also called *post-grouting* or *base grouting*, had been used previously worldwide yet literature on its use lacked a rational design approach. As a consequence, there had been little use in the United States. The technique involves casting drilled shafts with tubing or sufficient plumbing incorporated that could deliver high pressure grout to the tip of the shaft (after the shaft concrete develops strength capable of withstanding the grouting pressure). Additionally, the literature suggested that pressure-grouting shafts tipped in loose to medium dense sand provided the most benefit, but improvement was observed in all soil types cited. Therefore, this method would be applicable to projects in urban areas where vibrations associated with pile driving are not well tolerated and/or where the soil strata would have required excessively long drilled shaft lengths without considerable end bearing contribution.

The primary focus of Phase I was to conduct full-scale load tests on pressure-grouted drilled shafts and evaluate the results of the test data to develop design recommendations for the use of pressure grouting drilled shaft tips. Therein, full-scale shafts were constructed, grouted, and load tested. The response of the grouted shafts was compared to un-grouted control shafts constructed without provisions for grouting. A thorough evaluation of the comparative data revealed strong trends that were developed into a design methodology in cooperation with the needs and advice of the FDOT.

The promising results of Phase I led to supplemental research (herein denoted as Phase II) with the primary goal of strengthening the data base of load test programs conducted on grouted versus ungrouted drilled shafts. As such, a target of five additional load test programs were sought to fulfill this objective. Further, the development of software for the design of grouted drilled shafts was mutually agreed upon by both the FDOT monitor and USF principal investigator as an extension of the proposed supplemental research. This software development was felt necessary to expedite the selection process of load test program sites by quickly evaluating the applicability and/or cost effectiveness of a post grouting alternative. To this end, the effort of the project was divided into concurrent tasks outlined in each of the subsequent Chapters.

1.2 Report Organization

Chapter 2. The development of design software capable of performing the analysis of drilled shafts both with and without post grouted tips is discussed in Chapter 2. The phases of software development are discussed with emphasis on user feedback and requested features, as well as revisiting the basis of the computations performed that were established in Phase I.

Chapter 3. Numerous sites were reviewed as possible candidate locations for this study. Chapter 3 provides a summary of these sites including details of soil profile, project information, and the anticipated improvement for the given site. Further, a listing of the presentations made to various prospective users in an effort to solicit interest in this technology is also included.

Chapter 4. Five sites from Chapter 3 went to full-scale implementation of post grouting drilled shaft tips. Those sites involved grouted drilled shafts tipped in sand, clay, and silt soil types. The instrumentation, construction, grouting, and testing of these shafts are presented in detail. The load test results of each of the projects including both side shear and end bearing performance are also presented. The performance of the shaft tips is discussed with regards to the various soil types encountered. Due to the long maturation time associated with construction, other on-going projects from Chapter 3 could not be included in this report.

Chapter 5. Conclusions from the research findings including how the results match worldwide experience as well as recommendations for a new design basis are discussed in Chapter 5.

2. DEVELOPMENT OF DESIGN SOFTWARE FOR POST GROUTED SHAFTS

Software capable of performing analysis to compute post grouted end bearing capacity was developed on the basis of the Phase I recommendations for design. This software was designed using macro-driven worksheets.

2.1 Visual Basic, Microsoft Excel and Shaft 1-2-3

Visual Basic is a user-friendly programming package which uses graphical user interfaces (GUIs) to develop programs (Schneider, 1999). Visual Basic is a relatively new programming language for creating and controlling elements in a Windows program through the use of dialog boxes, drop-down lists, command buttons, menu bars, etc.. Microsoft incorporated programming language into their products and further developed a new version of Visual Basic called Visual Basic for Applications, VBA (Harris, 1999). Microsoft Excel is one of the products which utilizes VBA. A convenient difference, VBA code for Microsoft Excel is stored in the workbook whereas original Visual Basic code is stored in text files.

The development of VBA has further advanced the ability to quickly analyze drilled shaft capacities based on standard penetration tests (SPT) using the software discussed herein, *Shaft 1-2-3*. Shaft 1-2-3 is a macro-driven Excel spreadsheet which utilizes VBA programming for calculating ungrouted and grouted drilled shaft capacities. It includes different analysis methods for sand, clay, silt, and limestone. The organization of this data is broken into four main worksheets in Shaft 1-2-3: (1) *General* worksheet, (2) *Boring Log*, (3) Capacity worksheet(s), and (4) Plot(s). The following section discusses the analysis procedure for Shaft 1-2-3. A detailed commentary for this program is provided in Appendix A as well as in the back-most sheet of the program.

2.2 Analysis Procedure

Shaft 1-2-3 calculates side shear as well as the ungrouted and grouted end bearing for drilled shafts in sand, clay, silt, and limestone. The procedure involves inputting general information and analysis procedures (*General* worksheet) as well as boring log information (*Boring Log* worksheet).

2.2.1 General Worksheet

The *General* worksheet (Figure 2-1) is the platform for the user to define the job specifications, analysis parameters, and to calculate shaft capacities. The job specifications include job name, location, and engineer performing the analysis. The job name will be transferred to the capacity worksheet(s) and plot(s) for ease of identification of the current project. The analysis parameters are broken into two sections. The first section includes the shaft diameter size (up to three diameters can be analyzed at one time), cut-off / scour elevation, displacement criteria, end bearing influence zone, and grout pressure limit. Within this worksheet the input values can be toggled between SI and English units (but all information will be converted and stored as English units for analysis). No capacity will be provided in the scour zone. A displacement criteria is required for determining the mobilized end bearing (Reese and O'Neill, 1988). Figure 2-2 and 2-3 show the load transfer in end bearing versus settlement in cohesionless and cohesive soils, respectively. For the SPT97 analysis of driven piles, data three diameters below the tip elevation is required. Shaft 1-2-3 uses the same procedure for analysis, but is user defined in the end bearing influence zone parameter. The grout pressure limit is based on the grout pump mechanical limitations. The second section defines the shaft capacity design method for each soil type.

The shaft capacity design method is divided into four general soil types with analysis methods for both end bearing and side shear. The four soil types: Type 1, clay; Type 2, silt; Type 3, sand; and Type 4, limestone. Clay and Sand calculations are based on AASHTO (1998) design procedures for both side shear and end bearing. Silt does not have a specified design procedure. Therefore, depending on the properties of the material, silt can be analyzed as a clay or a sand. Recommendations from Dr. Dan Brown, Auburn University, take the most conservative design value from the two methods if the properties are not known. Limestone uses design methods specified by McVay and Townsend (1990) or AASHTO (1998) for side shear and FHWA (1998) for end bearing. The default design methods are the recommended design methods for each soil type.

2.2.2 Boring Log Worksheet

The *Boring Log* worksheet (Figure 2-4) contains information obtained from standard penetration testing (SPT) logs. The required information for the *Boring Log* worksheet includes the boring number (boring identification), ground surface elevation, water table elevation, and the SPT log which includes depth, blow count (SPT N), soil type (Figure 2-5), and rock coring information (if applicable). As with the job name, the boring number will be transferred to the capacity worksheet(s) and plot(s) for ease of identification of the current project. Updating the boring log will store soil parameters for the analysis.

Soil parameters include soil unit weight ( ), undrained shear strength (S_u), and the angle of internal friction ( ) and are all correlations from SPT N values. Figure 2-6 shows the relationship with soil unit weight and SPT N values. The undrained shear strength can be estimated by the following equation:

$$S_u = 125 * N \quad \text{(Kulhawy and Mayne, 1990)} \qquad \text{Equation 2-1}$$

where, Su is the undrained shear strength (psf) and N is the standard penetration number (blows/ft). Figure 2-7 shows the correlations between the angle of internal friction and SPT N values.

2.2.3 Capacity Worksheets and Plots

Calculations for each shaft diameter analyzed are placed in separate worksheets. Figure 2-8 is a typical capacity worksheet which includes job name, shaft diameter, boring identification, elevations, ungrouted and grouted shaft capacities, and grout pressures. Each capacity worksheet has an associated plot (Figure 2-9).

2.3 Shaft 1-2-3 Program Updates and Revisions

As part of the development of the program Shaft 1-2-3 , the program was distributed to various organizations and persons for review and feedback. These organizations and persons included the Florida Department of Transportation State Geotechnical Engineers and the State Materials Office, the Federal Highway Administration, Dr. Mike O'Neill of the University of Houston, Dr. Dan Brown of Auburn University, Dr. Steve Dapp of Geosyntec. The following is a list of revisions based on comments from the reviewers mentioned above:

- Side shear surface area was being calculated incorrectly when a scour depth was introduced other than the existing ground surface. Surface area was adjusted to reflect inputted scour conditions.

- Influence zone beneath the shaft tip used too few data points to reflect the user-requested zone. The diameters below shaft tip calculation was adjusted to keep from deleting one extra data point from the final results.

- A grout pressure limit of zero was found to give an error statement. Therefore, the software was updated to instruct the user to input a non-zero grout pressure value before proceeding.

- Shafts tipped in clay or silty soils where found to predict unreasonably high grout pressures. This conclusion was based on the performance of shafts grouted in both soil types. In such cases, grout pressure above the ungrouted end bearing could not be achieved. This finding was in keeping with that observed by Reese and O'Neill (1988) for shafts displaced beyond 2.5% the shaft diameter. Therefore, the design grout pressure for shafts tipped in clay was limited to the smaller of either the calculated grout pressure (via available side shear) or the ultimate end bearing stress.

- Numerical evaluations showed the grouted end bearing of shafts tipped in clay to be less than those ungrouted when using low grout pressure. Therefore, the grouted end bearing in clay was set as to the larger of either the mobilized end bearing or the applied grout pressure.

- When the applied grout pressure approached or exceeded 700-800 psi, the heat generated by the high pressure accelerated set times of the grout. Although an admixture can be used to combat this condition, the default maximum grout pressure was adjusted to 750 psi to reflect most practical construction limitations during grouting. However, the user can still set this value based on anticipated equipment performance.

- Most conservative design method was added to silty soils since there is no recommended design procedure. Therein, when selecting this option both clayey and sandy soil types are assumed and capacity is calculated for both. The lower of all sand and clay end bearing capacity is then selected as "most conservative."

- The undrained shear strength (Su) was inadvertently limited to 5000 psf. This restriction was removed.

- When calculating side shear for clay in the scour zone, capacity was being assigned inappropriately. Load carrying capacity within the scour zone was adjusted to reflect the absence of this material.

- The opening screen was updated to reflect the current revision date.

- AASHTO requires that the end bearing be reduced if the diameter is greater than 75 inches and tipped in clay. This requirement had been overlooked and was subsequently included.

- The Commentary was revised to provide more clarifications to the user.

- When the user input boring log information in intervals larger than the depth assigned to the end bearing influence zone, an error statement was obtained. The algorithms associated with capacity calculations were adjusted to handle larger boring log intervals.

- At the conclusion of the project, the database used to develop the correlations between the Tip Capacity Multiplier (TCM) and the Grout Pressure Index (GPI) was updated to reflect new information. This will be discussed in greater detail in Chapter 5.

2.4 Recommended Shaft 1-2-3 Revisions

Shortcomings of the *Shaft 1-2-3* program include the inability to import cone penetration test (CPT) data, to adjust side shear and end bearing values based on load test data, and to subtract shaft mass from the shaft capacity due to different load factors. It is envisaged that the use of CPT data for grouted shafts will be highly applicable given the improvement that is obtainable in softer, CPT-friendly soils. As such, a subroutine capable of incorporating CPT soundings and various methods of analysis with CPT data will be helpful.

In many cases, local knowledge of soil properties and shear strengths are not reflected by AASHTO design methods (especially in stiff silts or clays). Although the software can be massaged to reflect this information it is not particularly user-friendly in this area. As such the software should be revised to include user-defined values for side shear. Likewise, as more information becomes available for grouted shafts in site-specific soils, a user-defined feature for end bearing (both grouted and ungrouted) should be incorporated.

2.5 Summary

The above recommendations to *Shaft 1-2-3* have not been incorporated into the latest version of the software. However, it does make use of all post-grouted shaft data collected to date. Further, the correlations between TCM and GPI are based on ungrouted tip performance as predicted by Reese and O'Neill (1988). The details and effects of these changes in the design approach from that outlined in Phase I are discussed in Chapter 5.

Finally, the program makes no attempt to subtract the dead weight of the shaft from the capacity. Therein, the shaft capacity is calculated as ultimate and the user is required to apply the appropriate load factors for dead load (i.e. shaft weight) as necessary. This is based on the variety of design approaches presently in use (e.g. ASD, LFD, or LRFD).

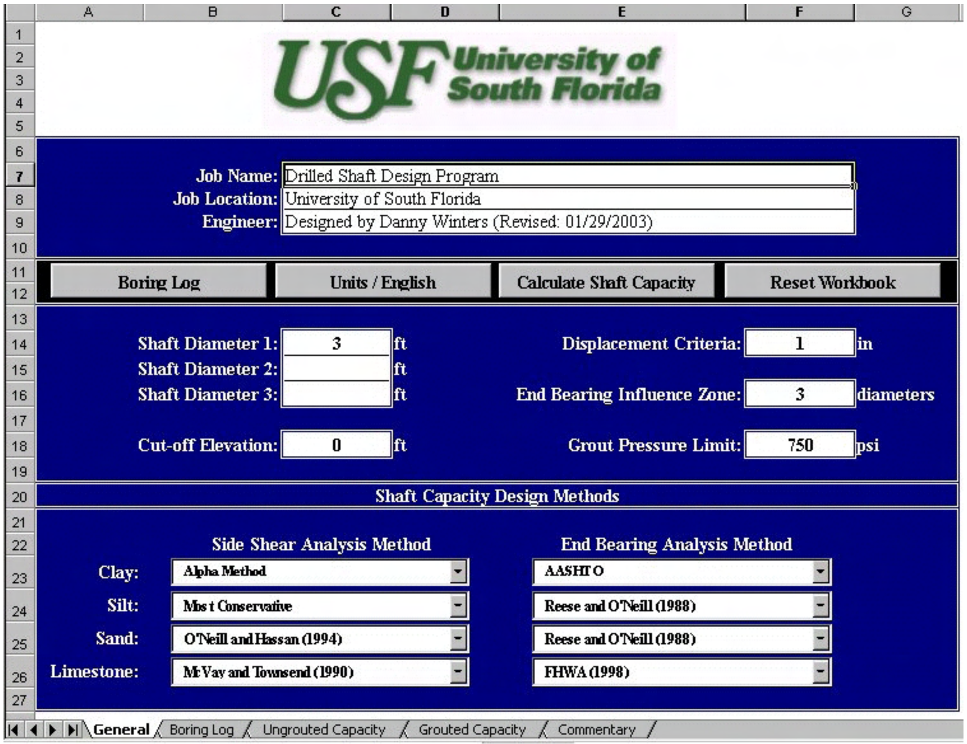


Figure 2-1 General Worksheet.

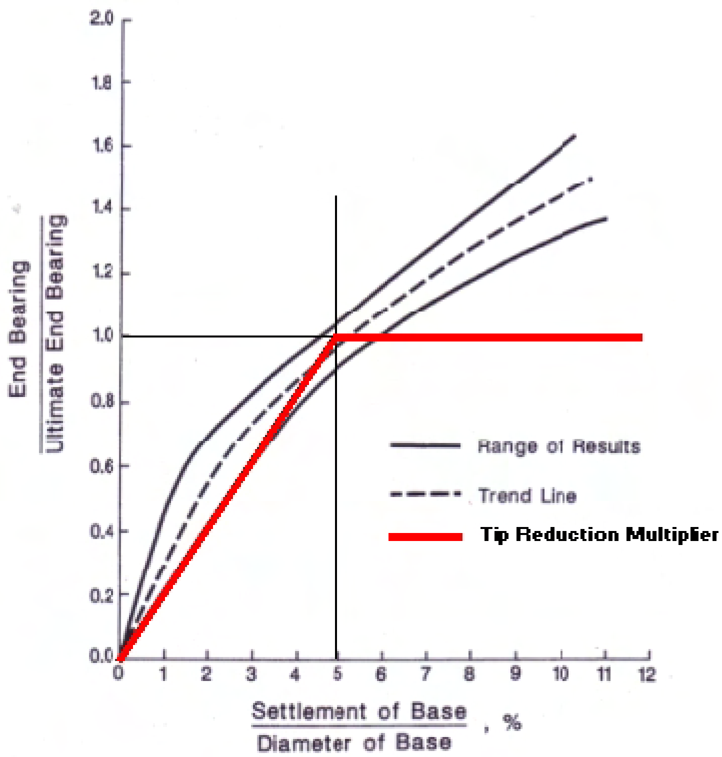


Figure 2-2 Tip Reduction Multiplier for cohesionless soils.

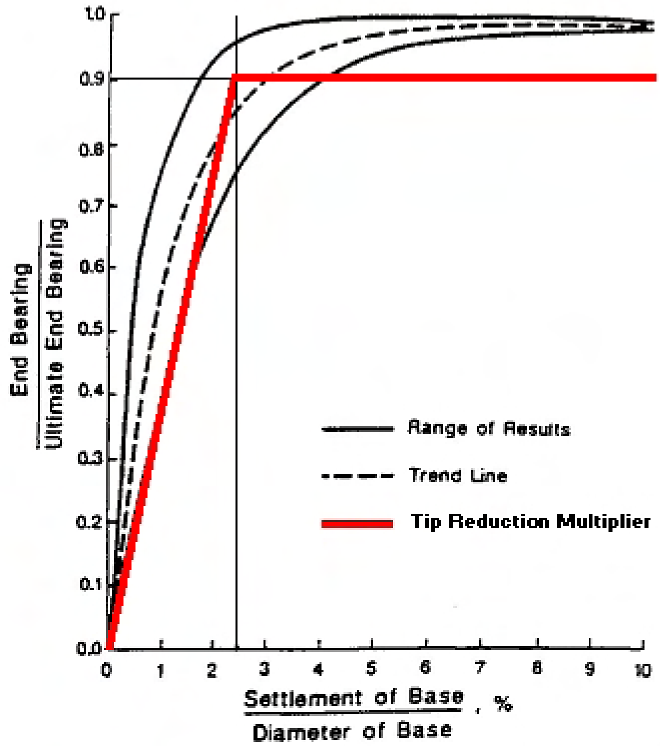
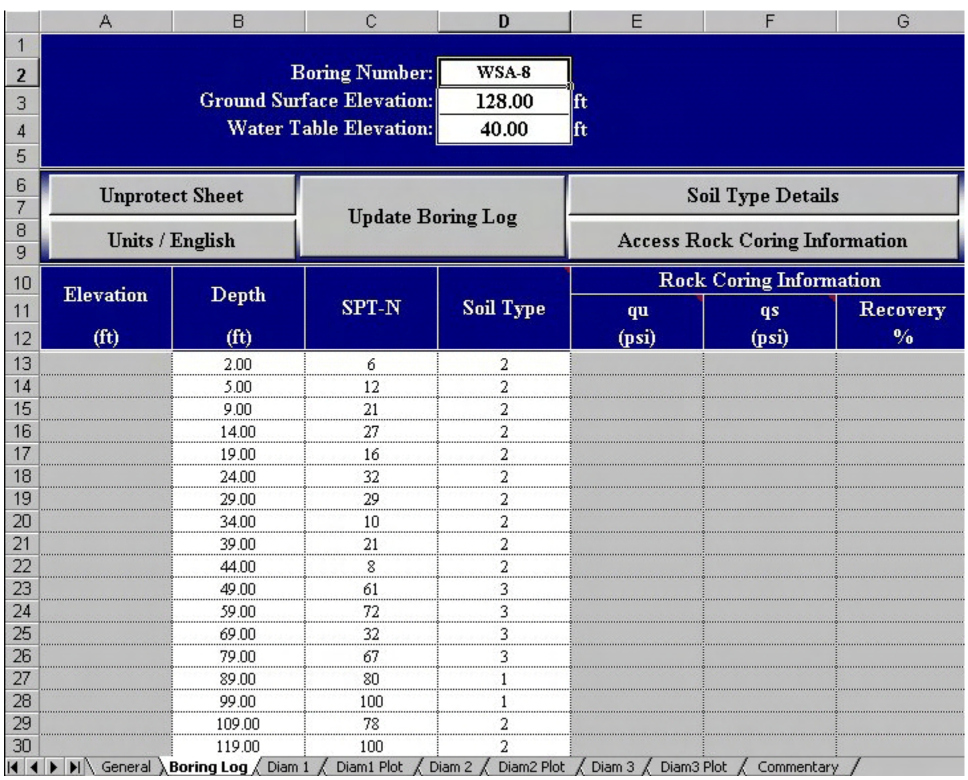


Figure 2-3 Tip Reduction Multiplier for cohesive soils.



Boring Number: WSA-8
Ground Surface Elevation: 128.00 ft
Water Table Elevation: 40.00 ft

Unprotect Sheet | Update Boring Log | Soil Type Details
Units / English | Access Rock Coring Information

Elevation (ft)	Depth (ft)	SPT-N	Soil Type	Rock Coring Information qu (psi)	qs (psi)	Recovery %
	2.00	6	2			
	5.00	12	2			
	9.00	21	2			
	14.00	27	2			
	19.00	16	2			
	24.00	32	2			
	29.00	29	2			
	34.00	10	2			
	39.00	21	2			
	44.00	8	2			
	49.00	61	3			
	59.00	72	3			
	69.00	32	3			
	79.00	67	3			
	89.00	80	1			
	99.00	100	1			
	109.00	78	2			
	119.00	100	2			

General | Boring Log | Diam 1 | Diam1 Plot | Diam 2 | Diam2 Plot | Diam 3 | Diam3 Plot | Commentary

Figure 2-4 Boring Log Worksheet.

Figure 2-5 Soil Type Form.

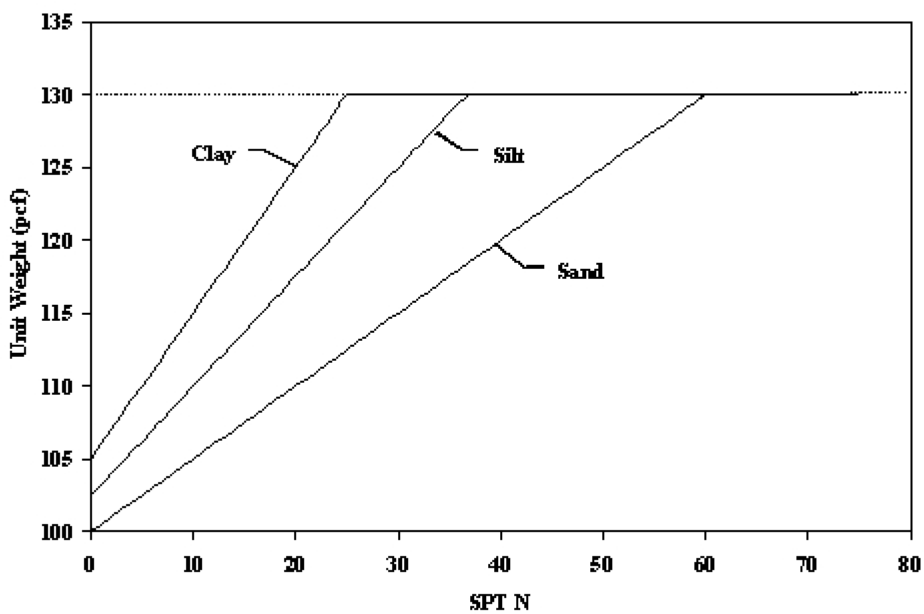


Figure 2-6 Soil unit weight and standard penetration test (SPT N) relationships.

SPT - N	ϕ^o
0 - 2	26
3 - 4	28
5 - 10	29
11 - 20	30
21 - 30	32
31 - 40	33
>40	34

Figure 2-7 Values for based on SPT N.

Natchez Trace Parkway

5 ft Diameter, Boring WSA-7

Elevation	Ungrouted Shaft Capacities				Grout Pressure	Grouted Shaft Capacities
	Ultimate Side Shear	Ultimate End Bearing	Ultimate Shaft Capacity	Ungrouted Capacity @ 1.00 in Disp		Grouted Capacity @ 1.00 in Disp
ft	tons	tons	tons	tons	psi	tons
97.00	0.00	0.00	0.00	0.00	0.00	0.00
94.00	0.00	0.00	0.00	0.00	0.00	0.00
90.00	0.00	0.00	0.00	0.00	0.00	0.00
85.00	0.00	0.00	0.00	0.00	0.00	0.00
80.00	0.00	0.00	0.00	0.00	0.00	0.00
75.00	0.00	0.00	0.00	0.00	0.00	0.00
70.00	0.00	0.00	0.00	0.00	0.00	0.00
65.00	8.27	422.49	430.76	149.10	5.85	263.91
60.00	24.10	415.94	440.04	162.74	17.05	293.48
55.00	45.50	368.45	413.95	168.32	32.18	310.93
50.00	71.15	343.89	415.04	185.78	50.33	350.73
40.00	263.57	441.79	705.36	528.65	186.44	705.36
30.00	456.00	441.79	897.78	721.07	322.55	897.78
20.00	648.42	441.79	1090.21	913.49	458.66	1090.21
10.00	840.84	441.79	1282.63	1105.91	594.77	1282.63
0.00	1033.26	441.79	1475.05	1298.34	730.89	1475.05

General / Boring Log / Diam 1 / Diam1 Plot / Diam 2 / Diam2 Plot / Diam 3 / Diam3 Plot / Commentary

Figure 2-8 Shaft Capacity Worksheet.

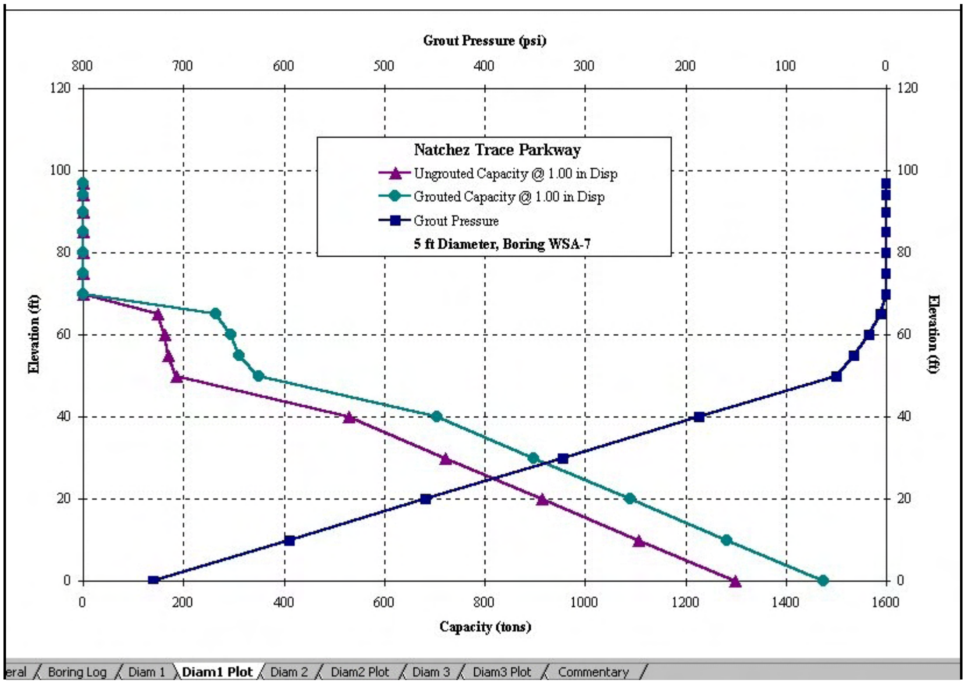


Figure 2-9 Shaft Capacity Design Curves.

3. PROJECTS REVIEWED FOR POST GROUTING

This chapter outlines the steps taken to secure information regarding post grouted shaft performance in a variety of soil types. Numerous State and Federal Agencies as well as contractors, consultants, and professional organizations were contacted and subsequently involved in this project. Collaboration with these entities was in-part the outcome of presentations made throughout the State and the Southeastern United States.

3.1 Technology Transfer Presentations

In an effort to increase awareness and stir interest in using post grouted shafts, numerous presentations were made to potential users. In most cases, the presentations were sculpted to address parameters, such as local soil type, specific to a given project. At a minimum, the presentations provided a pathway for subsequent correspondence concerning potential sites for data collection. A list of the various forums in which the FDOT post-grouted design approach was presented is given below. This listing does not include those presentations given by FDOT personnel or those given prior to Phase II of this research.

Designing for Post-Grouted Drilled Shaft Tips in Sand, GMEC Conference, Orlando, FL, April, 2002.

Value Engineering Cost Proposal for Post-grouting Drilled Shafts, Astaldi Construction Office, Davey, FL, June, 2002, presented to Tri-Rail Design Team.

Post-Grouting Drilled Shafts in Delta Deposits, Presented to MDOT / Arkansas DOT, Bermingham, AL, June, 2002, presented to State Engineers from both states.

Value Engineering Cost Proposal for Post-grouting Drilled Shafts, FDOT , Pensacola, FL, July, 2002, presented to Davis Highway Project Managers.

The Effect of Pressure-Grouting Drilled Shaft Tips on Bearing Capacity, Geotechnical Research in Progress Meeting, Tallahassee, FL, August 3, 2002.

Designing for Post-Grouted Drilled Shaft Tips in Sand, Presented to Alabama DOT, Montgomery, AL, October, 2002.

Value Engineering Options for New Bayfront Arena, Presented to Consultants and TexDOT Engineers, Corpus Criste, TX, December, 2002.

Designing Post Grouted Drilled Shaft Tips, Presented to FHWA Eastern Federal Lands Bureau, Washington, D.C., March, 2003.

Designing Post Grouted Drilled Shaft Tips, Presented to TexDOT and University of Houston Faculty, Houston, TX, April, 2003.

Post Grouting Drilled Shafts in Houston Area Soils, Presented to TexDOT and University of Houston Faculty, Houston, TX, May, 2003.

Post Grouting Drilled Shafts - Phase II, Presented at the Geotechnical Research in Progress Meeting, Orlando, FL, July, 2003.

Post Grouting Drilled Shaft Tips, Southeastern Transportation Geotechnical Engineering Conference, North Charleston, South Carolina, October, 2003.

Post Grouting Drilled Shaft Tips, Presented to ASCE Ridge Branch Chapter, Lakeland, FL, October, 2003, presented to local civil engineers.

Post Grouted Drilled Shafts: A Case History of the PGA Boulevard Bridge Project, DFI Annual Conference, Miami, FL, October, 2003.

3.2 Collaborators / Data Sources

The interest spurred by the above presentations led to the eventual involvement of over twenty contractors and design firms. In most cases, these entities requested a review of a specific site in the form of design curves presented by the investigators. Those parties involved at the time of this report are cited below:

Applied Foundation Testing
Archer Western
Astaldi Construction
Bauer / Coastal Caisson
Bechtel Engineering and Testing
Beck Foundation
Case Atlantic
Case Foundation
HDR Engineering
Kewitt Construction
Law Engineering
Melick-Tully & Associates
Morris Shea Bridge Company
PSI
Rossi
Schnabel Engineering
Traylor Bros.
TreviIcos
TreviIcos South
Union Pacific Railroad
Williams Bros.
Zachary Construction

Several State and Federal agencies were also interested in the use of post grouting. In most cases, these agencies eventually were involved in a project in their respective area. These agencies included:

ALDOT
Arkansas DOT
Cal Trans
CSX Railroad
FDOT
FHWA Eastern Federal Lands Bureau
GDOT
MDOT
Tex DOT

3.3 Evaluation of Potential Post Grout Sites

Over twenty foundation construction projects were reviewed for possible cost savings and/or applicability to a post-grouted drilled shaft option. As a result, over 250 design curves (Appendix C) were prepared based on individual boring logs that were supplied to contractors or owners for review, and in most cases, the post grouting option was competitive.

The performance improvement can be evaluated either on the basis of load carrying capacity at a specific depth or on the basis of shaft length reduction for a given load. Figure 3-1 shows a typical design curve annotated to show these comparisons. The un-annotated curve was generated from the design software and provided to the contractors.

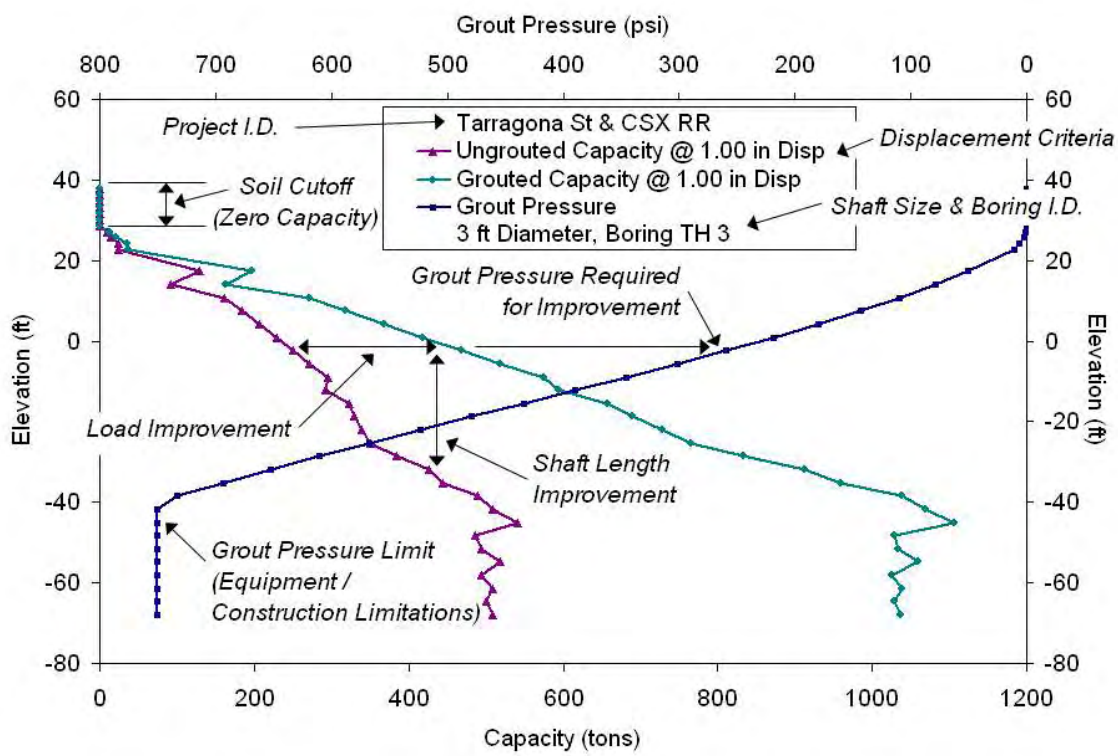


Figure 3-1 Detailed shaft capacity and grout pressure design curves.

For each project reviewed, the average end bearing improvement is reported. This average was derived from all lengths of shaft reported in a given design curve. This improvement was calculated as the difference between the grouted and ungrouted end bearing divided by the ungrouted end bearing. However, this average value is not truly representative of the actual improvement that would be realized but is an indicator of general improvement in that soil type. The actual improvement is dependent on the attainable grout pressure which is directly related to available side shear. The deeper a shaft is embedded, the more available side shear capacity and hence higher attainable grout pressure. Further, loose sands are more improvable than dense sands. Figure 3-2 shows the improvement as a function of depth for Boring PGAB-1 from the PGA Blvd Project in West Palm, FL. In general, the end bearing improvement increases with depth and decreases inversely proportional to relative density.

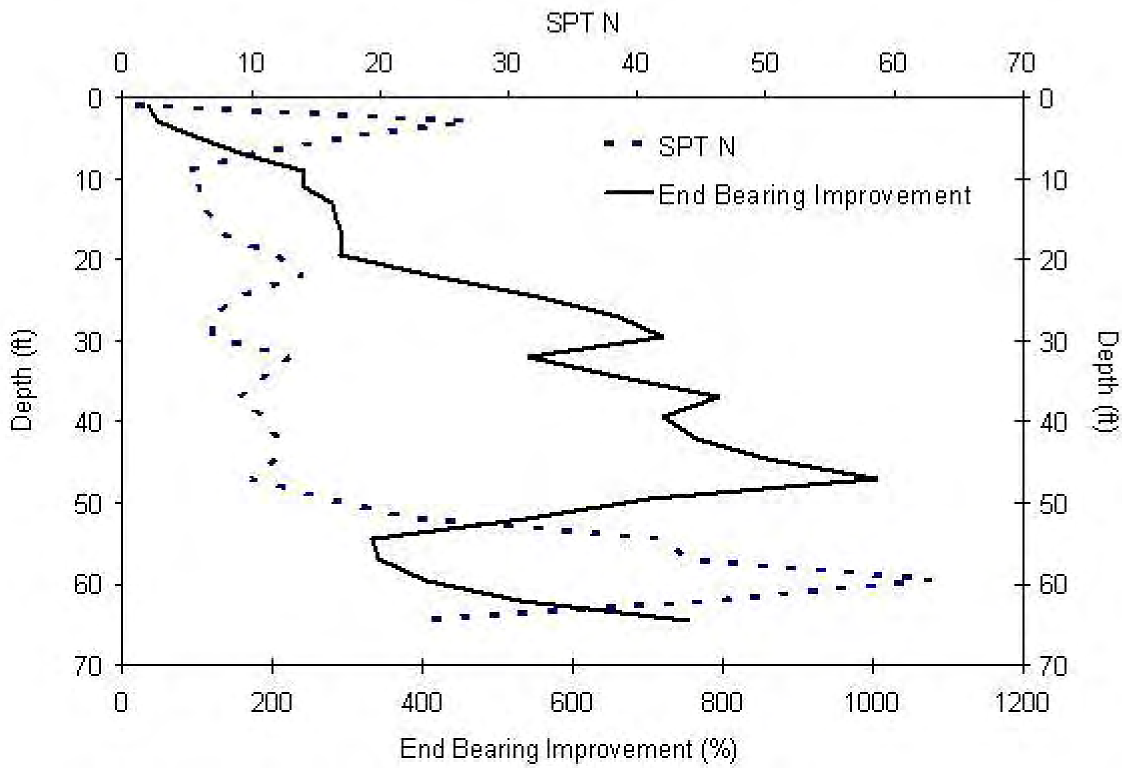


Figure 3-2 End Bearing Improvement with respect to depth and SPT N value.

A listing of the projects reviewed and the outcome status is provided below. Those projects where post grouting was implemented for this study are discussed first and are denoted with an asterisk. Those projects where post grouted shafts were not selected or were pending at the time of this report follow.

3.3.1 PGA Boulevard Grade Separation Project*

This project consisted of four grade separation bridges at the interchange of I-95 and PGA Blvd. The use of post grouted shafts was presented as a Value Engineering Change Proposal by the general contractor to circumvent the vibrations associated with pile driving around an Eye Surgery Clinic. Standard Penetration Test boring logs were provided to determine the effectiveness of post grouting drilled shafts. Although several boring logs were analyzed, Appendix B, Figures B-29, B-30, and B-51 show the general soil profile for this project. Design curves for 3 foot diameter drilled shafts were produced using the current prediction methods. Appendix C, Figures C-252 through C-256 show the design curves produced. Predicted end bearing capacity improvement for grouted shafts was 474%. Therefore, 234, 24" piles ranging between 60' and 70' in length could be replaced with 108, 36" diameter post grouted shafts 30' to 50' long. The VECP was accepted and construction of post grouted shafts commenced. Measured end bearing improvement was 263%. Full details of this project are included in Chapter 4.

3.3.2 Auburn, National Geotechnical Experimentation Site*

The University of South Florida (USF) and Auburn University (AU) jointly installed five drilled shafts for a multi-purpose research project at the National Geotechnical Experimentation Site (NGES), Alabama. One of the goals of the project was to determine the effectiveness of post grouting in silty soils. The drilled shafts were 3.5ft in diameter and 24ft in length. Four of the five shafts were constructed with post grouting apparatuses with the fifth shaft constructed as a control shaft. The soil was tested at each shaft location using a mini-Cone Penetration Test (mini-CPT). The soundings for each shaft are in Appendix B, Figures B-1 through B-5. In silts and clays only modest improvements are anticipated. In this case for the shafts tipped in silt a 16% improvement was predicted, the measured improvement was 77%. The entire construction, grouting, and load testing of these shafts is discussed in Chapter 4.

3.3.3 University of Houston, TX*

The University of Houston (UH) requested the University of South Florida (USF) to collaborate on a load test program to demonstrate to TexDOT the effectiveness of post grouting drilled shafts in soils indigenous to the Houston, TX area. A total of four 4ft diameter drilled shafts were constructed. The soil was tested at the location of each test shaft using the Cone Penetration Test. A total of six CPT soundings, one SPT, and one Texas Cone Penetrometer sounding was performed across the site. Appendix B, Figures B-41 through B-50 show the soil boring logs and CPT soundings. Two shafts were tipped in sandy soil while the other two shafts were tipped in clayey soil (more representative of Houston-area soils). Each pair of test shafts included a control shaft and a grouted shaft. A target load of 2000 tons was used in determining the shaft lengths. The design curve is shown in Appendix C, Figure C-261. Predicted end bearing improvement (using the Phase I design approach) for the shaft tipped in sand was 214%. The measured improvement in the dense sand was 41%. Although only modest improvement was anticipated for the shaft tipped in clay (33%), the measured improvement was 71%. Full details on this project are discussed in Chapter 4.

3.3.4 Natchez Trace Parkway*

This project was a Federal Highway Administration, Eastern Federal Lands Bureau program involving 26, 72" diameter drilled shafts approximately 75' in depth. Wilbur Smith Associates placed this project out for bid as a design-build project. The project location was in Natchez, MS. Over 14 soil borings were submitted for review to determine the validity of the post grout option. The soil profile consisted of a medium stiff to stiff clay for 35' followed by very dense sand for 40' underlaid by stiff silt. Out of the 14 soil boring logs provided 9 were analyzed. Appendix B, Figures B-18 through B-20 show the soil boring log values used for analysis. Design curves for 5', 5.5', and 6' diameter drilled shafts were provided to the contractor for review. Appendix C, Figures C-62 through C-83 show the design curves produced. The preliminary design curves showed an average increase in end bearing capacity from ungrouted to grouted drilled shafts of 195% in the sand layers and 67% in the silt layers. The measured end bearing improvement in stiff silt was 109%. Full details of this project are discussed in Chapter 4.

3.3.5 Farm to Market Rd 507 (FM 507)*

Shortly after a Technology Transfer Presentation to the Texas DOT, the State engineers opted to use two 30" diameter, 46' deep post grouted drilled shafts to increase the capacity of a failing bridge pier. The magnitude of the capacity shortfall caused 10" settlement with no signs of slowing. As such, the bridge was fully dismantled so that the post grouted shafts could be installed and the vertical alignment could be restored. The two post grouted drilled shafts were tipped in sandy clay soil. The project has been finished and traffic flow restored. Full details of this remedation program are presented in Chapter 4.

3.3.6 Bangkok, Thailand

Although it was improbable that the instrumentation and grouting of this project could be in some way controlled by the USF/FDOT research project, the principle investigator agreed to review this site in hopes of obtaining the field data in return. For this site, the soil profile consisted of soft to medium stiff clay with layers of medium dense to dense sand. Appendix B, Figure B-6 shows the soil boring log values used for analysis. Design curves for 2m (6.5ft) diameter drilled shafts provided to the contractor for review. Appendix C, Figure C-251 shows the design curves produced. The preliminary design curves showed an average increase in end bearing capacity from ungrouted to grouted drilled shafts of 854% in the sand layers and 91% in the clay layers. No further information was available at the time this report was published.

3.3.7 Bayway Bridge

Professional Services Industries, Inc. provided lengths for both grouted and ungrouted drilled shafts as alternatives to the driven pile option for SR 682 (Pinellas Bayway) from West Toll Plaza to West of SR 679 Bayway. Design for the grouted shaft option was based on Phase I results presented at the GMEC Conference by Peter Lai. PSI's preliminary calculations were based on four SPT borings. Appendix B, Figures B-52 and B-53 show the soil boring log values used for analysis. The PSI recommendations were verified using Shaft 1-2-3. Appendix C, Figures C-257 through C-260 show the design curves produced from Shaft 1-2-3. Further evaluation showed an average improvement in end bearing capacity of 322% for grouted shafts over conventional ungrouted shafts for a given shaft length. The latest FDOT specifications (Appendix D) will need to be incorporated should the post grouting option be selected.

3.3.8 Beau Rivage Condominiums

LAW Engineering requested design curves for a future condominium in Ft. Myers, Florida. To determine the soil profile for the site five soil borings were performed. The soil profile for the site consisted of a loose sand for 35ft underlain by a dense to very dense sand. One of the five soil boring logs provided was analyzed to verify the validity of the post grout option. Appendix B, Figure B-7 shows the soil boring values used for analysis. Design curves for 3', 3.5', 4', 4.5', 5', 5.5', and 6' diameter drilled shafts were provided to the contractor for review. Appendix C, Figures C-1 through C-7 show the design curves produced. The preliminary design curves showed an average increase in end bearing capacity from ungrouted to grouted drilled shafts of 413%. No further information was available at the time this report was published.

3.3.9 Benicia-Martinez Bridge

The Benicia-Martinez Bridge project is located in San Francisco Bay Area of California where I-680 crosses over the Carquinez Strait Bridge. Correspondence with CalTrans Engineers to establish the merits and logistics of post-grouting drilled shafts was initiated in collaboration with Dr. Dan Brown of Auburn University. Therein, concerns over the excavation cleanliness were the driving forces toward grouting; but the intent was to mitigate soft toe conditions that were anticipated due to extreme depths. USF researchers provided recommended specifications for construction and grouting apparatuses illustrations have been supplied. Serious doubts of whether the contractor could construct drilled shafts grouted or ungrouted has slowed progress. No further information was available at the time this report was published.

3.3.10 Bolling Airforce Base

Schnabel Engineering requested design curves for an Air Force base in the Washington D.C. area. To determine the soil profile for the site six Standard Penetration Tests (SPT) were performed across the site. The soil profile for the site consisted of loose to medium dense sand for 40ft underlaid by a medium dense to medium dense clay. Out of the six soil boring logs provided three were analyzed to determine the applicability of a post grouting option. Figure B-8 shows the soil boring log values used for analysis (Appendix B). Design curves for 2.5', 3', 3.5', 4', 4.5', 5', 5.5', and 6' diameter drilled shafts were provided to the contractor for review. Appendix C, Figures C-8 through C-31 show the design curves produced. The preliminary design curves showed an average increase in end bearing capacity from ungrouted to grouted drilled shafts of 370% for the sand layers and 33% for the clay layers. No further information was available at the time this report was published.

3.3.11 Cervantes Street

Florida Department of Transportation (FDOT) requested design curves for bridge foundations in Escambia County, FL. The project involves the replacement of US 90 Cervantes Street bridge over CSX railroad and city streets in Pensacola, FL. To determine the soil profile for the site a total of 23 Standard Penetration Test (SPT) soil borings and 4 auger borings were performed. The soil profile consisted of loose sand for 16ft followed by dense sand for 60ft underlaid by loose sand. Out of the 23 very similar soil borings two were used to verify the validity of the post grout option. Appendix B, Figure B-9 shows the soil boring log values used for analysis. Design curves for 3.5ft diameter drilled shafts were provided to the contractor for review. Appendix C, Figures C-32 and C-33 show the design curves produced. The preliminary design curves showed an average increase in end bearing capacity from ungrouted to grouted drilled shafts of 787% for the two soil boring logs. This site was deemed to be an ideal site for post grouted shafts by both USF and FDOT personnel. Although a drilled shaft contractor submitted a competitive cost proposal, the prime contractor opted to stay with the driven pile option.

3.3.12 I-10 / I-110 Interchange

Florida Department of Transportation (FDOT) requested design curves for foundations in Escambia County, FL for I-10 / I-110. To determine the soil profile, 132 soil boring logs were performed across the site. The soil profile consisted of medium dense to medium dense sands for 45ft followed by loose to medium dense sand for 30ft underlaid by very dense sand. Out of the 132 soil borings provided, 23 were analyzed to determine the validity of the post grout option. Appendix B, Figures B-10 through B-16 show the soil boring log values used for analysis. Design curves for 3' and 4' diameter drilled shafts were provided to the contractor for review. Appendix C, Figures C-34 through C-59 show the design curves produced. The preliminary design curves showed an average increase in end bearing capacity from ungrouted to grouted drilled shafts of 1266%. However, this foundation type was not used.

3.3.13 I-16 over Ogeechee River

Georgia Department of Transportation requested design curves for bridges on I-16 over Ogeechee River in Bryan and Effingham Counties, GA. Over 20 soil boring logs were performed across the site to determine the soil profile. The soil profile consisted of loose to medium dense sand for 40ft underlaid by dense to very dense silt. To determine the validity of the post grout option for this project two of the soil boring logs were analyzed. Appendix B, Figure B-17 shows the soil boring log values used for analysis. Design curves for 3ft diameter drilled shafts were provided to the contractor for review. Appendix C, Figures C-60 and C-61 show the design curves produced. The preliminary design curves showed an average increase in end bearing capacity from ungrouted to grouted drilled shafts of 464%. No further information was available at the time this report was published.

3.3.14 New Bayfront Arena

Texas Department of Transportation (TexDOT) requested design curves for the New Bayfront Arena in Corpus Christi, TX. The soil profile for the site consisted of loose to medium silt for 25ft underlain by soft to stiff clay. Two of the nine soil boring logs provided were used to determine the validity of the post grout option. Appendix B, Figure B-21 shows the soil boring log values used for analysis. Design curves for 2.5', 3', 3.5', and 4' diameter drilled shafts were provided to the contractor for review. Appendix C, Figures C-84 through C-90 show the design curves produced. The preliminary design curves showed an average increase in end bearing capacity from ungrouted to grouted drilled shafts of 25% for silt and clay. The post grout option was not selected by the contractor. No further information was available at the time this report was published.

3.3.15 New River Bridge

Florida Department of Transportation (FDOT) requested design curves for New River Bridge Corridor Project in Broward County, FL. The site investigation was performed by GEOSOL, Inc. which included 36 soil borings to determine the soil profile for the site. The soil profile for the site consists of loose to medium dense sand with layers of limestone throughout the site. Eighteen of the 36 soil boring logs provided were analyzed to determine the validity of the post grout option. Appendix B, Figures B-22 through B-27 show the soil profile used for the analysis. Design curves for 3', 4', 5', and 6' diameter drilled shafts were provided to the contractor for review. Appendix C, Figures C-91 through C-162 present those design curves. The preliminary design curves showed an average increase in end bearing capacity from ungrouted to grouted drilled shafts of 422% in the sand layers. However, due to the enormous magnitude of the rail loading, the shafts could not be founded in the shallower sands and required rock-sockets. The post grout option was not selected.

Review of this project lead the investigators to evaluate the potential capacity gain from post grouting rock-socketed shafts. Therein, it was determined that even a modest assumption for end bearing equaling the grout pressure could provide considerable improvement. Presently, no grout programs in rock have been conducted, but the physics is promising.

3.3.16 Newark Legal Center

Melick-Tully & Associates requested design curves for the Newark Legal Center in New Jersey. The soil profile for the site consisted of loose to dense sand for 60ft underlaid by stiff clay. The contractor provided two soil boring logs to be analyzed for the validity of the post grout option. Appendix B, Figure B-28 shows the soil boring log values used for design. Design curves for 3', 4', 5', and 6' diameter drilled shafts were provided to the contractor for review. Appendix C, Figures C-163 through C-170 show the design curves produced. The preliminary design curves showed an average increase in end bearing capacity from ungrouted to grouted drilled shafts of 624%. No further information was available at the time this report was published.

3.3.17 Opelika Parking Garage

This project consisted of a 6-level parking garage for the East Alabama Medical Center in Opelika, Alabama. Dr. Dan Brown, Auburn University, proposed post grouted drilled shafts to the engineering firm Christian Testing Laboratories, Inc as a cost saving alternative to drilling to the depth of rock (65 to 90+ feet). The maximum column design loads ranged from 1000 to 1600 kips. The current shaft design is based on an end bearing pressure of 50 ksf in sound rock. The soil profile was determined from 11 SPT soil borings across the site. The soil consisted of dense sandy silt for 4.5ft followed by stiff to firm saprolite for 50ft to 80ft underlaid by rock. No further information was available at the time this report was published.

3.3.18 SR 80 - Palm Beach County

Due to the success of the PGA Blvd project, a drilled shaft contractor requested design curves for SR 80 in Palm Beach County, FL. To determine the soil profile for the site 14 soil borings were performed. The soil profile consisted of medium dense to dense sand for 60ft. Out of the 14 boring logs provided 2 were analyzed to determine the validity of the post grout option. Appendix B, Figure B-31 shows the soil boring log values used for analysis. Design curves for 3', 3.5', and 4' diameter drilled shafts were provided to the contractor for review. Appendix C, Figures C-171 through C-176 show the design curves produced. The preliminary design curves showed an average increase in end bearing capacity from ungrouted to grouted drilled shafts of 418%. No further information was available at the time this report was published.

3.3.19 Towers Eleven

Bechtel Engineering and Testing requested design curves for the Towers Eleven Condominium in Daytona Beach, FL. To determine the soil profile 9 soil borings were performed. The soil profile consisted of loose sand for 16ft followed by very dense sand for 10ft underlaid by loose to medium dense sand. Three out of the nine soil boring logs provided were analyzed to determine the validity of the post grout option. Appendix B, Figure B-32 shows the soil boring log values used for the analysis. Design curves for 2', 2.5', 3', and 4' diameter drilled shafts were provided to the contractor for review. Appendix C, Figures C-177 through C-188 show the design curves produced. The preliminary design curves showed an average increase in end bearing capacity from ungrouted to grouted drilled shaft of 997%. No further information was available at the time this report was published.

3.3.20 Union Pacific Railroad

Texas Department of Transportation (TexDOT) requested design curves for an upgrade of an existing bridge over Union Pacific Railroad. Seven Texas Cone Penetration Test (TCP) soil borings were performed across the site to determine the soil profile. The soil profile consisted of stiff clay with layers of medium dense to dense sand. Out of the seven soil boring logs provided two were analyzed to determine the validity of the post grout option. The TCP values had to be converted to SPT N values in order to analysis the project using the FDOT post grout design procedures. Appendix B, Figure B-33 shows the soil boring log values used for the analysis. Design curves for 2.5', 3', and 3.5' diameter drilled shafts were provided to the contractor for review. Appendix C, Figures C-189 through C-198 show the design curves. The preliminary design curves showed an average increase in end bearing capacity from ungrouted to grouted drilled shafts of 712% in the sand layers and 33% in the clay layers. No further information was available at the time this report was published.

3.3.21 US 82 Mississippi River Bridge

Mississippi and Arkansas Department of Transportation requested design curves for US 82 in Washington County, Mississippi and Chicot County, Arkansas. This request was based on the positive response from a presentation given by the Principle Investigator. The contractor provided one soil boring log for analysis to determine the validity of the post grout option. The soil profile consisted of loose silt for 30ft followed by loose to very dense sand. Appendix B, Figure B-34 shows the soil boring log values used for the analysis. Design curves for 3', 4', 5', and 6' diameter drilled shafts were provided to the contractor for review. Appendix C, Figures C-199 through C-202 show the design curves produced. The preliminary design curves showed an average increase in end bearing capacity from ungrouted to grouted drilled shafts of 719%. No further information was available at the time this report was published.

3.3.22 US 98 Bay County, Florida

Design curves were prepared to evaluate the applicability of post grouted shafts for a project on US 98 in Bay County, Florida. To determine the soil profile across the site 26 soil borings performed. The soil profile consisted of loose to dense sand. Out of the 26 soil boring logs provided 24 were analyzed. Appendix B, Figures B-35 through B-40 show the soil boring log values used for analysis. Design curves for 3' and 4' diameter drilled shafts provided to the contractor for review. Appendix C, Figures C-203 through C-250 show the design curves produced. The preliminary design curves showed an average increase in end bearing capacity from ungrouted to grouted drilled shafts of 576%. No further information was available at the time this report was published.

3.4 Site Review Summary

Of the twenty two sites reviewed, 12 had sandy profiles, 5 sand and clay, 2 silty soils, 1 clay only, 1 sand and silt, and 1 rock. Shafts tipped in sand were predicted to exhibit an average end bearing capacity improvement of 578% where values ranged from 195% to 1266%. Those tipped in clay and silt were predicted to exhibit an average capacity improvement of 37% where values ranged from 3% to 91%. The five sites where post grouted shafts were constructed and investigated for this study are discussed in detail in Chapter 4.

4. FULL-SCALE LOAD TEST PROGRAMS

The primary focus of this research project was to conduct full-scale load tests on pressure-grouted drilled shafts and evaluate the test data to enlarge the data base of post grouted shafts. In conjunction with the tasks of this project, five test sites were evaluated for post grouted drilled shafts where shafts were constructed, post-grouted and/or load tested, and finally analyzed. These sites included: (1) State Road 786 over State Road 811 (PGA Blvd) in West Palm Beach, Florida, (2) the National Geotechnical Experimentation Site in Opelika, Alabama, (3) TexDOT Post Grout Demonstration in Houston, Texas, (4) Natchez Trace Parkway in Natchez, Mississippi, and (5) Farm to Market Road 507 (FM 507), Willacy County, Texas. The following sections discuss the load test / grouting program for each site. In addition to these five programs, the influence of grouting on pulse-echo integrity test results is discussed for those sites where such tests were conducted.

4.1 PGA Blvd

This grouted shaft test program was part of the PGA Boulevard Grade Separation Construction Project located in West Palm Beach, Florida and consisted of improvements to its intersection with Alt A1A (Old Dixie Highway). In all, the project had three bridge structures including PGA Boulevard Alt A1A, A1A SW Ramp to I-95 and Alt A1A SW Ramp over FEC Railway. The foundations for these bridges were originally designed using a LRFD design approach on 24” square prestressed concrete piles arranged in bents and groups. Factored design loads were 135 to 220 tons with a resistance factor of 0.65. Pile lengths were up to 70 feet. Because nearby structures were sensitive to vibration, pile driving hours were restricted to a four hour window at night. To circumvent these restrictions, post grouted drilled shafts were submitted as a Value Engineering Cost Proposal (VECP). Pressure grouting of the drilled shaft tips was used to increase end bearing capacity allowing the drilled shaft option to be economically viable. Other benefits included reducing the project schedule due to increased work hours, reduction of pier cap size, reducing risks associated with vibrations, and increased drilled shaft quality assurance.

A total of 108 drilled shafts replaced 234, 24 inch driven piles. The shafts design diameter was 36 inches with lengths between 30 and 50 feet. Factored design loads ranged from 195 to 490 tons. A resistance factor of 0.75 was used due to the added quality assurance from load testing. Although not considered at the design phase, additional confidence could have been incorporated in the form of an increased resistance factor due to post grouting every shaft. To verify the VECP design and demonstrate the construction method, a load test program was undertaken. Typically, a “methods shaft” is required to demonstrate the contractor’s ability to construct a drilled shaft (i.e. excavation and concreting). In this case, the State agreed to allow one of the two test shafts to be considered the “methods shaft.” Therein, only one additional shaft was required for the test program.

The load test program revolved around the relative end bearing performance of two 3 ft diameter test shafts denoted as LT-1 and LT-2 tipped in shelly sand. The test shafts were embedded 60 ft deep. LT-2 was constructed for post grouting with LT-1 constructed as the control shaft. The entire post grout program timeline is summarized below:

- Instrumentation of shaft LT-1 on April 8, 2002.
- Instrumentation of shaft LT-2 on April 9, 2002.
- Completion of shaft LT-1 on April 9, 2002.
- Completion of shaft LT-2 on April 10, 2002.
- Post grout shaft LT-2 on April 17, 2002.
- Axial Statnamic testing of LT-1 and LT-2 on April 25, 2002.
- Phase I production began July, 2002.
- Phase I production completed in November 2002.
- Phase II production began in August, 2003.
- Phase II production completed in December, 2003.

4.1.1 Soil Exploration and Site Layout

Three series of soil explorations were conducted throughout the duration of the project which encompass 18 SPT borings: (1) Design Phase, 13 borings denoted by the prefixes PGAB-1 through 6, FOB-1 through 6, and FOW-7; (2) Load Test Program, Boring B-1; and (3) Production Phase II (PGA Blvd left bridge) Borings B -1 through 4. A Standard Penetration Test boring was performed at each test shaft location. The SPT soil borings are located in Appendix B. The as-built location of Test Shaft LT-1 corresponded to Load Test Boring B-1 which was conducted within the time frame of test program. Boring PGAB-1, which was conducted at the design phase, coincided with the Test Shaft LT-2 location. NOTE: as a consequence of the naming scheme of the various boring series, there is the potential to confuse B-1 from the load test with B-1 of the Phase II production. The soil profile generally consisted of very loose to dense sand (SP) with intermittent layers of shelly sand for the entire depth of the borings. Generally, the upper 30 feet were very loose to medium dense (N= 2 to 20) and the lower 50 feet were medium dense to very dense sands (N= 9 to 41). Large amounts of rainfall following several years of near drought conditions in Florida produced unusually large fluctuations in groundwater levels between 3 to 10 feet in depth during the course of the project. The watertable was located at a depth of 8 feet at the time of construction.

4.1.2 Construction and Instrumentation

The two test shafts, designated as LT-1 and LT-2, were constructed with nearly identical geometric properties and subsurface conditions except LT-2 was equipped with a post grout apparatus at the tip. Each of the test shaft reinforcement cages were instrumented with two levels of four strain gages, one level of two strain gages, and one toe accelerometer. The strain gages for the test shafts were placed at a depth of 6' and 58' for the two levels of four gages and 30' for the level of two gages. The toe accelerometer was place at a depth of 58 feet. LT-2 also included telltales at three different levels. Figure 4-1 shows the installation of the strain gages and toe accelerometer. The data obtained from the strain gages was used to delineate the load contribution from the end bearing and side shear. The toe accelerometer was used to determine the bottom displacement of the test shafts during statnamic load testing.

Shaft LT-1 was an out of position non-production shaft constructed in accordance with the FDOT Standard Specification 455. An oversized diameter temporary casing was installed to a depth of 13 feet to stabilize the upper soils. This casing which was removed immediately after pouring concrete had an outer diameter of 42 inches and a ½ inch wall thickness. The remaining shaft excavation was drilled under a bentonite slurry with a 34 inch diameter drilling tool. The total length of LT-1 was 60.8 feet including 1 foot formed and poured above ground. Shaft LT-2 was an out of position non-production shaft constructed nearly identical to LT-1 but had the Flat Jack type post grout apparatus installed in the tip. The grout plate was a 24 inch diameter flat-jack system as used in Phase I in Clearwater, Florida. A sleeve-port system was incorporated into the flat-jack system as a backup system. Figures 4-2 through 4-4 show the construction of the flat-jack / sleeve-port system, installation of rubber membrane, and installation of the scuff ring, respectively. Figure 4-5 shows the installation of the grout plate for test shaft LT-2. Because an oversized diameter upper temporary casing was used, the upper 13 feet of the shaft had a section diameter of 42 inches. The remaining shaft below the temporary casing depth had a diameter of 34 inches. The total length of LT-2 was 60.5 feet including 1 foot formed and poured above ground. Figures 4-6 through 4-10 show the construction of the test shafts.

4.1.3 Post Grouting

The setup for a verification grout test is similar to that of a static load test in that a series of high accuracy displacement gages are used to measure foundation movement. Displacement transducers were mounted on the reference beam to measure the shaft top upward displacement and the upward displacement from toe-level telltales. An electronic pressure transducer was used to measure grout pressure. Figure 4-11 shows these gages attached to a reference beam that is founded outside the radial zone of influence developed by downdrag (or uplift) of the soil surrounding the foundation. Further, this reference beam was shaded to minimize effects of solar heating which can cause non-uniform deformations of the reference beam (Figure 4-12). A survey level was back-sighted to a known reference elevation and was used to monitor the foundation movement as well as the reference beam movement throughout the duration of the test. These conditions are in general accordance with ASTM recommended procedures for static load tests (D-1143) and such conditions were observed for this program.

During the base grouting, the bi-directionally acting grout pressure is resisted by the skin friction of the shaft. Thus, the available end bearing improvement is bounded by the shaft friction. Inherently, the grouting process then provides quantitative data on the skin friction and a lower limit of end bearing capacity of each shaft. Post grouting was performed with a high pressure high efficiency mixing grout plant as shown in Figure 4-13. The plant included a colloidal mixing tank and an agitated holding tank. The pump was a single stage hydraulic actuated piston type capable of 1500 psi grouting pressure. A neat cement grout consisting of Type I/II Portland cement at a water/cement ratio of 0.50 was used. Figures 4-14 shows the grouting process.

The general procedure consisted of first flushing the grout lines with fresh water until clear water was observed out the two return lines. Grouting began with the return line valves open. When grout return was observed, the return line valves were closed and pressurization started. The grout was injected at low flow rates and stopped periodically for various measurements. Grouting of test shaft LT-2 was terminated at 0.7 inches of upward displacement at a grout pressure of 760 psi.

4.1.4 Post Grouting Test Results

Grout Pressure vs. Disp. During the grouting of LT-2, the displacement of the top of the shaft was measured with displacement transducers and the displacement at the toe of the shaft was measured with string line transducers attached to tension telltales. Grout pressure was measured by an in-line pressure transducer at the grout pump. Field surveying of the top of shaft during grouting was recorded to confirm the transducers measurements. Figure 4-15 shows the grout pressure versus displacement from both computer-acquired grout test data and field survey notes. The results show a sustained grout pressure of 760 psi at 0.7 inches. The grout test was interrupted three times due to insufficient Portland cement supplies therein causing three unloading cycles as well as associated delays (also shown in Figures 4-16 and 4-17).

Figure 4-16 shows the difference between the grout pressure as measured at the grout pump and that registered at the toe of the shaft via strain gages (denoted as concrete stress). This is due to head loss in the grout tubes and/or thickening of the grout during delay times. With the exception of the transient spikes associated with the pump strokes the concrete stress at the toe is virtually identical to grout pressure at the pump during times of continuous pumping. Divergence shown here is due to prolonged waiting as the contractor did not have enough grout making materials on hand.

Side Shear. An important aspect of all grout tests (both pilot and production) is the side shear response that can be verified during grouting. Figure 4-17 shows the upward displacement versus developed side shear for LT-2 as determined from strain gage data and displacement transducers. The maximum mobilized side shear of the shaft was 124 tons for the upper 30 ft (average SPT N = 9) and 212 tons for the lower 30 ft (average SPT N = 17) which provided a total side shear of 336 tons upward resistance.

Grout Volume. The volume of grout pumped to the toe of test shaft LT-2 was also recorded manually in the survey notes. Figure 4-18 shows the grout volume versus grout pressure which indicates an increasing grout volume for a given increase in grout pressure beyond

750 psi. A net volume of 6.25 ft^3 was used to reach a grout pressure of 760 psi (over and above that volume required to achieve grout return). Likewise, the relationship between the grout volume and uplift provides further insight into the anticipated grout volume usage. Figure 4-19 shows that approximately 1 ft^3 was required to initiate uplift and 2 ft^3 to establish notable uplift. Above 5 ft^3, an increased volume was required for a given uplift.

Production Grouting Criteria. From the post grout test results, it is appropriate to set the field grout criteria based on side shear response and grout volume usage logs. The design grout pressure is predetermined, but is dependent on anticipated side shear resistance. In cases where the side shear response of the shaft does not reflect the anticipated capacity (during grouting), adjustments should be made to reflect the observed / measured capacity. At the design grout pressure (450 psi in this case), two values should be examined by the design engineer: the volume of grout placed and the uplift. At 450 psi, the net grout volume was 2.5 ft^3 and the uplift was 0.15 inches. The grout test, however, was permitted to exceed the design grout pressure in order to obtain an upper limit on both available side shear and the respective uplift displacement. These upper limits can then be used to sculpt the grout criteria to minimize the adverse effects of excessive shear strain and to set a reasonably attainable grout volume. The grout volume criterion verifies grout flow prior to achieving the design grout pressure. This criteria should give the grouting contractor as much tolerance as possible with each of the parameters to account for variations in soil strength or construction difficulties encountered throughout the site.

If an ultimate side shear can be defined by the grout test, the corresponding top-of-shaft displacement should be noted so that the uplift displacement criterion would not permit that much displacement. In this case, the grout test was terminated when the grout pressure reached the grout pipe capacity of 1000 psi. At which time no ultimate side shear could be defined with 0.7 inches of uplift (ult disp > 0.7 in). Downward load testing (discussed later) showed side shear resistance still increasing up to about 0.9 inches (Figure 4-20). However, a conservative production uplift limit was set at 0.75 inches which was more in line with that tested during grouting.

A minimum grout volume criterion should be set to confirm a reasonable flow of grout to the soil beneath the tip. As it is conceivable that well constructed shaft tips in dense sand may require less grout volume than in loose sands, this minimum grout volume criterion should be set as low as practicable. Forcing excessive grout volume beneath the tip does not necessarily increase the improvement (more does not mean better). The volume of soil displaced by a conventional shaft when it achieves a displacement of 5% of the diameter is a good lower bound for this value. In this case, 5% of 3 ft times the cross sectional area yields 1.06 ft^3 (where uplift initiated). At the design grout pressure, the grout test had used 2.5 ft^3. Therefore a minimum grout volume of 2 ft^3 was set for this criterion.

4.1.5 Axial Compressive Load Test

Axial compressive load testing was accomplished with a 16 MN statnamic device. During the statnamic load tests, many instruments were measured to obtain the foundation response. Directly mounted to the shaft top was the calibrated load cell to register the applied load. Top of shaft movement was recorded via the output of three capacitive accelerometers placed equi-distant around the shaft at 120 degrees. Strain gages embedded in the shaft at the top bottom and mid-depth were also continuously monitored during the statnamic tests. A brief description of the instrumentation used during the statnamic tests is given below.

- Axial Statnamic Device - This device uses a controlled burn of fuel to generate pressure which is transferred to the test shaft through a load cell. The statnamic device produces a time dependent load on the order of 1/2 second or less. The load was measured with a ring type electronic resistance load cell, located between the shaft top and the loading piston.

- Top-of-Shaft Accelerometers - Three capacitive accelerometers were arranged across the top of the shaft approximately 120 degrees apart during statnamic testing. Little to no eccentricity was shown during the loading. The capacitive accelerometers were manufactured by PCB Piezotronics, Inc. From the measured accelerations, pile displacements at each accelerometer location were calculated. Past experience with these gages provides a high level of confidence in the reliability and accuracy of displacement data from the acceleration measurements when compared with the direct displacement measurements.

- Strain Gages - Full bridge electronic resistance sister bar strain gages using Vishay gage type CEA-06-125UT-350 gages.

- Data Acquisition Systems - MEGADAC Data Acquisition System, manufactured by Optim, Inc. This system monitored the strain gages, load cell, and accelerometers. Data was recorded at 5,000 samples per second per channel.

The elevation was surveyed performed before and after each test to provide a check of permanent displacements. The derived static capacity from the statnamic tests was evaluated using the Segmental Unloading Point Method (SUP) developed at the University of South Florida (Mullins, et al., 2002a). The SUP method, discretizes a foundation into segments. The number of segments and their lengths are defined by the locations of the embedded strain gages. This allows the standard Unloading Point Method (UPM) to be applied to each segment. Then the total derived static response is calculated as the sum of the derived static response from the individual segments. The analysis of the statnamic load test data was performed using the segmental unload point software *SUPERSAW* (Winters, 2002). The following sections discuss the results from post grouting and axial compressive load testing program as well as the results from two phases of construction and production grout testing.

4.1.6 Side Shear Results

The strain gages at the toe of the shaft and the displacement of the top of shaft provide information on the mobilized side shear resistance. The load obtained from the strain gages assumed a uniform cross-sectional area at the toe based on a 3 ft diameter shaft. The modulus used for the analysis regression was provided from 6" x 12" cylinder testing. Figures 4-20 and 4-21 show the total shaft and end bearing capacity of LT-1 (ungrouted) and LT-2 (grouted), respectively. The algebraic difference between the two curves at a given displacement represents the mobilized side shear. Recall the grout test results of LT-2 (Figure 4-17) showed 336 tons of side shear developed during the uplift test. When subsequently tested in downward compression, the shear strain in the soil surrounding the shaft reversed at approximately 0.25 inches (Figure 4-21) and side shear then began to develop resistance to the downward movement. From that point of shear reversal an equivalent shear displacement (0.25 + 0.7 = 0.95 inches) was then adopted for comparison between the two test methods (upward vs. downward). The ratio of upward versus downward side shear of 0.75 was found for this test case which is comparable to cited literature values for sand (O'Neill, 2002). Figure 4-22 shows the side shear load of both test shafts, LT-1 and LT-2, during axial compressive loading (downward loading) and the side shear load during grouting (upward loading) of LT-2.

The maximum side shear load for LT-1 and LT-2 was 725 tons and 720 tons at displacements of 1.25 and 1.5 inches, respectively. Minimal difference was observed in the borings at LT-1 and LT-2 (Navg = 13 and 12, respectively) which was corroborated by the similar side shear response when at a similar state of side shear strain. This is an important finding for a post grout test program as it indicates the sensitivity of soil to strain reversal. It is also helpful in assessing how strictly the production uplift criteria during grouting must be upheld.

4.1.7 End Bearing Results

The end bearing load from the strain gages assumed a uniform cross-sectional area based on a 3 ft diameter drilled shaft and a concrete modulus from cylinder testing as discussed earlier. Figure 4-23 shows the end bearing load from both LT-1 and LT-2, the predicted AASHTO end bearing, and the applied grout pressure. The end bearing load for LT-1 and LT-2 were 132 tons and 466 tons at 1.6 inches, respectively. The load test program was intended to produce the AASHTO 5%D displacement (1.8 inches) to achieve ultimate end bearing (Reese and O'Neill, 1988). The two tests on LT-1 and LT-2 achieved maximum displacements of 1.63 and 1.80 inches, respectively. Although the test on LT-1 did not reach the target displacement, it was felt to be close enough for reasonable extrapolation. The measured end bearing at 5%D displacement for LT-1 and LT-2 were 168 and 482 tons, respectively. The predicted end bearing values (0.6 x SPT(N) x A_{tip}) were determined to be 148 tons and 89 tons for LT-1 and LT-2, respectively. The tip load applied from the grouting test of LT-2 was calculated to be 387 tons (Figure 4-23).

It is interesting to note that the predicted end bearing value of LT-1 was commensurate with that measured with a predicted / measured ratio of 0.9. Although LT-1 and LT-2 were intended to be identical shafts the natural variation in the end bearing from the two different borings (N = 35 and 21, respectively) introduced unavoidable variations when comparing grouted and ungrouted end bearing responses. At a displacement of 5%D, the grouted shaft end bearing was 2.8 times higher than the ungrouted. However, the grouted shaft end bearing was 5.4 times greater than the predicted for that exact soil boring. Even if predicted value is assumed to be 90% of measured, the grouted shaft could be assumed to have been 4.9 times greater than an ungrouted shaft in that exact location.

4.1.8 Production Phase Grouting

The shaft construction and grouting for the PGA Blvd project was conducted in two phases whereby 76 of the 108 total shafts were constructed in Phase I and the remaining 32 shafts in Phase II. Figures 4-24 through 4-33 show the construction of some of the production shafts. The grouting procedure for all shafts was conducted in keeping with the protocol set forth by the grout test program. That program provided a minimum grout volume (2 ft^3 or 57 liters), the maximum permissible uplift displacement (0.75 in or 19mm), and an estimated upper limit of expected grout (8 ft^3 or 250 liters). The design grout pressure was assigned on a pier by pier basis depending on the specific axial load requirements. The minimum grout volume criterion is and was intended to assure flow into the toe area prior to achieving the design grout pressure. The maximum anticipated grout volume was provided to aid the contractor in estimating grout volume that would be likely needed.

Uplift movement was required to be monitored by survey level and recorded in conjunction with periodic pressure and volume measurements.

Phase I. Phase I shaft construction spanned from June 21, 2002 to November14, 2002. On an active day, an average of 1.3 shafts were constructed per day (at most 4 per day). In most cases, multiple shafts were grouted in a single day to optimize the grouting contractor's labor and mobilization costs. On average, four shafts per day were grouted (ranging from 1 to 9 shafts per day).

During Phase I, three construction related mishaps required some form of grout protocol review or intervention. The first of which involved drilling one shaft 1 meter too shallow. Therein, the potential for side shear reduction and hence overall shaft capacity shortfall was of concern. Although the design grout pressure relies heavily on developing significant amount of side shear, some reserve side shear capacity was likely to be available. This was verified by grouting that shaft to a higher grout pressure than originally specified. The resultant increase in grout pressure both verified higher side shear as well as supported the use of a higher TCM for end bearing.

The second mishap was a direct consequence of torrential rains that raised the ground water elevation almost 10 feet. This in conjunction with lowering the work elevation led to difficulties in maintaining excavation stability. One excavation became so unstable that the excavation was halted, refilled, and subsequently excavated only after installing full length temporary casing. This happening was highlighted at the time of grouting when that shaft exhibited the highest uplift displacement approaching the grouting protocol limit of 19mm. In comparison with all other shafts, the next highest displacement was 9.5 mm; the smallest was less than 0.1mm.

Finally, some problems with the concrete mix design (% max aggregate) would occasionally cause plugging of the tremie pipe at the onset of concrete placement. In one instance, concreting was delayed to ascertain the cause of the problem. This delay led to difficulties extracting the full length temporary casing without also extracting the cage and shaft concrete (full length casing had been systematically implemented as a result of the higher than expected ground water elevation). The full length casing was left in place which raised concerns over loss of side shear from the relatively smoother steel surface (estimated net loss of 120 tons of side shear). To mitigate this condition, full length / depth compaction grouting was conducted up the sides at three equidistant locations around the shaft at a radial distance of approximately 6 inches from the casing. This procedure was conducted prior to base grouting. Therein, when the regularly scheduled post grouting was conducted, it served to verify sufficient side shear development (upward displacement was minimal, 2 mm).

Phase II. Phase II shaft construction spanned from November 10, 2003 to December 15, 2003 where on an active day an average of 2 shafts were constructed per day (at most 4 per day). Again, multiple shafts were grouted in a single day to optimize the grouting contractor's labor and mobilization costs. On average, four shafts per day were grouted (at most 7 per day).

Only two significant occurrences were documented during the construction and grouting of the 32 Phase II shafts. First, the general contractor requested that grouting be conducted after the columns on that End Bent were constructed on the yet to be grouted shafts. Interestingly enough, in some case studies grouting has been conducted with the additional reaction load from the structure (Bruce, et al., 1995). In this case the proposed procedure would have no adverse effects on the shaft capacity and was approved. The second occurrence was similar to what was experienced during Phase I where one of the shafts was inadvertently drilled short by 0.8 meters. Therein, the same protocol was used to verify the adequacy of the shaft capacity via increasing the grout pressure. Somewhat like the Phase I happening, this shaft was not adversely affected by the slightly reduced length and the increased grout pressure provided a proof of reserve capacity.

Throughout both Phase I and II of construction, the production grouting at each shaft tip provided tangible verification of shaft capacity. All shafts were grouted in accordance with the protocol established as a result of the grout test / load test program.

4.1.9 Production Measurements and Results

A total of 108 production drilled shafts with post grouting were constructed in two phases of this project. Phase I consisted of 76 production shafts while Phase II consisted of 32 production shafts. A post grout field record log was developed for each production shaft which includes general construction data for the shaft, surveyed top of shaft displacement, grout volume, and grout pressure. Figure 4-34 shows a typical field record for a post grouted drilled shaft (Pier 2 - Footing 3 Right - Shaft 1). All 108 production shaft field records are included in Appendix E. From the grouting field logs, various quality assurance curves can be produced. Figures 4-35 and 4-36 show two typical quality assurance curves produced from the field record log in Figure 4-34. Figure 4-35 shows grout pressure versus top of shaft displacement. Figure 4-36 shows grout volume versus grout pressure.

The production rates for construction and grouting of the drilled shafts are documented in Figures 4-37 and 4-38 for Phases I and II, respectively. In general one to two shafts were constructed in a given excavation day. Base grouting was typically conducted when a sufficient number of shafts could be accessed in a given day as shown by the large numbers of shafts per day that were grouted.

Uplift. While using base grouting, the capacity of each shaft constructed was in fact individually proof tested. Therein, field records like those shown in Figure 4-34 can provide the information necessary to produce a load - displacement curve by multiplying the tip area and the grout pressure to produce load. The ability to withstand the design grout pressure without significant uplift is indication of adequate shaft performance. By looking at the field data for all shafts, 100% quality assurance can assigned to the overall project. Figures 4-39 and 4-40 provide this type of project review for Phases I and II, respectively. Interestingly, those shafts grouted in Phase I exhibited an average uplift of 0.11 inches while those grouted in Phase II only showed 0.03 inches on average. Both Phases were designed using the same design approach but Phase II used updated borings obtained after Phase I completed. Although all borings were conducted by the same consultant, the company had switched all of their SPT hammer from manual to automatic in the time that elapsed from the design phase drilling to Phase II construction (1999 - 2003). As a result those shafts constructed in Phase II (left bridge) were considerably stiffer when grouted. Figure 4-41 shows the variation in SPT values from the right bridge and the left. Boring PGAB-4 was conducted with a manual hammer, while B-3 with an automatic. Assuming they are actually more similar than different, the variation corroborates those studies conducted for FDOT by Davidson and Maultsby (1998). However, even in that study, manual hammers ranged in energy efficiency from 39 to 93% while various automatic hammer manufacturers varied in energy efficiency from 52 to 98%. In this instance, there was no way of knowing how well the newer data taken with an automatic hammer would correlate to the design calibrated by load testing and the blow counts from a manual hammer. Hence, a more conservative approach was taken where no attempt to correct for an unknown change in hammer efficiency. The effect was manifested in lower uplift values during grouting of Phase II.

When the design grout pressure is appropriately assigned to optimize full use of available side shear, uplift is a direct indication of shaft performance. For each shaft on the PGA Blvd project, acceptable uplift values were observed (less than the uplift criterion). However, uplift values were noted to be somewhat higher in those shafts where concrete over-pours were experienced. Figure 4-42 provides a glimpse of this trend where shafts with higher than theoretical concrete volumes exhibited greater than average uplift values (> 0.11 inches). This is attributed to degradation of the initial soil structure due to slight instabilities in the excavation walls perhaps caused when the drill slurry head fluctuates below good construction values. This instability causes soil sloughing, greater excavation volume, and radial relaxation of the soil structure. The greater excavation volume is registered by the concrete over-pour, while the loss of soil structure integrity is registered by higher uplift.

Grout Volume. Although the cement material required for grouting constitutes only a small portion of the expense of post grouting shafts, it is imperative that there be an understanding of the grout volumes required to circumvent costly construction delays. On average, approximately 4 ft^3 of grout per shaft was required to flush the grout tubes and then pre-compress the soil beneath the tip. However, the grouting contractor must be prepared for the unusual occurrence where more is need. The largest total volume of grout required for any of the shafts grouted at PGA Blvd was 13.7 ft^3. In general, the grouting contractor started grouting a given shaft with approximately 6 ft^3 pre-mixed and was capable of mixing about 2 ft^3 every 5 to 10 minutes (which in most cases kept up with the placement rate in ft^3/min). Some of this grout is used to flush the grout tubes (return volume) and rest is used to pre-compress the soil (net volume).

The return volume required to flush the grout tubes is obviously dependent on length of shaft as well as the diameter and number of grout tubes. However, the depth to the shaft tip also affects the open tube grout pressure due to hydrostatic effects. Therein, deeper shaft tips start the pressurized grouting at a higher initial grout pressure (at toe). The higher grout pressure with depth causes an increase in grout volume over and above that required to flush the grout access tubes. Figure 4-43 shows the return volume for all 108 shafts as a function of the shaft tip depth. This curve shows the general trend of increased initial volume proportional to depth. In several instances, one or two of the three 1" SCH 80 grout tubes had become blocked prior to grouting which is indicated by return volumes smaller than the theoretical tube volume. In general, however, a significant volume of grout (up to 1.5 ft^3) was placed prior to initial grout pressurization (1.06 ft^3 corresponds to the volume beneath the tip at a 5%D displacement).

As the grout pressure of shafts tipped in sand is dependent on the side shear, the actual blow counts beneath the shaft tip has almost no effect on the end bearing. Therein, loose sands with low blow counts are highly improvable and dense sands with high blow counts are only moderately improvable. However, the volume of grout required to provide improvement is dependent on the initial relative density of the sand. Loose sands which are highly improvable require higher grout volumes to affect the densification. Figure 4-44 shows the net grout volume introduced beneath the shaft tips during the pressure grouting as a function of the soil density beneath the tip (as reflected by the SPT-N). Although the data is somewhat scattered, a clear trend of lower net volume with increased relative density can be seen. Therein, some of the scatter can be attributed to various design loads, shaft lengths, and available side shear reaction.

Figure 4-1 PGA Blvd: Installation of strain gages and toe accelerometer.

Figure 4-2 PGA Blvd: Grout plate construction with backup system.

Figure 4-3 PGA Blvd: Installation of rubber membrane onto grout plate.

Figure 4-4 PGA Blvd: Installation of scuff ring on grout plate.

Figure 4-5 PGA Blvd: Installation of grout plate.

Figure 4-6 PGA Blvd: Drilling of test shafts.

Figure 4-7 PGA Blvd: Clean-out of bottom to ensure cleanliness of bottom prior to cage placement.

Figure 4-8 PGA Blvd: Reinforcement cage placement.

Figure 4-9 PGA Blvd: Concrete pour of test shafts.

Figure 4-10 PGA Blvd: Control test shaft.

Figure 4-11 PGA Blvd: Post grout instrumentation setup.


Figure 4-12 PGA Blvd: Post grouting setup.

Figure 4-13 PGA Blvd: Grout pump.

Figure 4-14 PGA Blvd: Post grouting using neat cement with w/c ratio of 0.50.

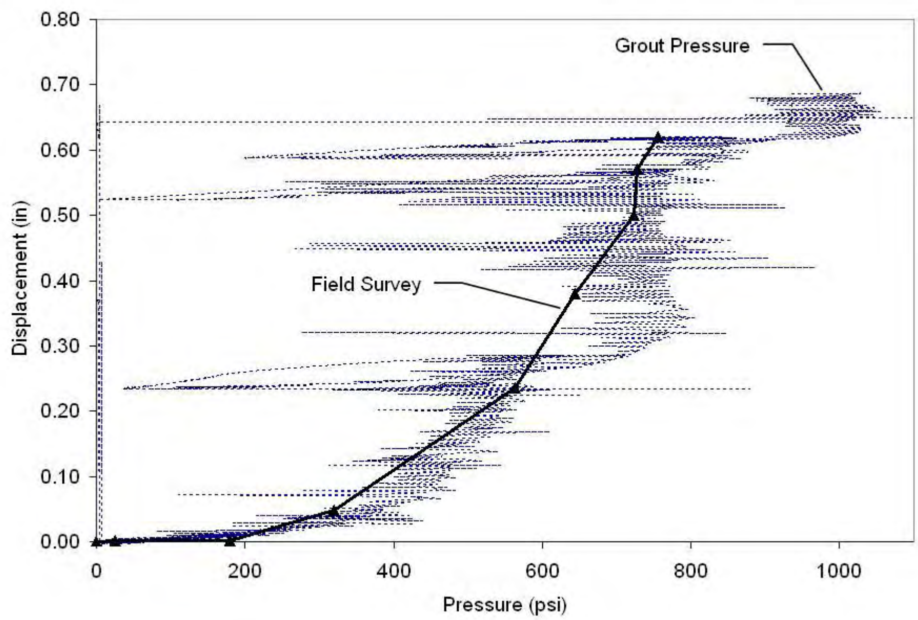


Figure 4-15 PGA Blvd: LT-2 grout pressure versus displacement during grouting.

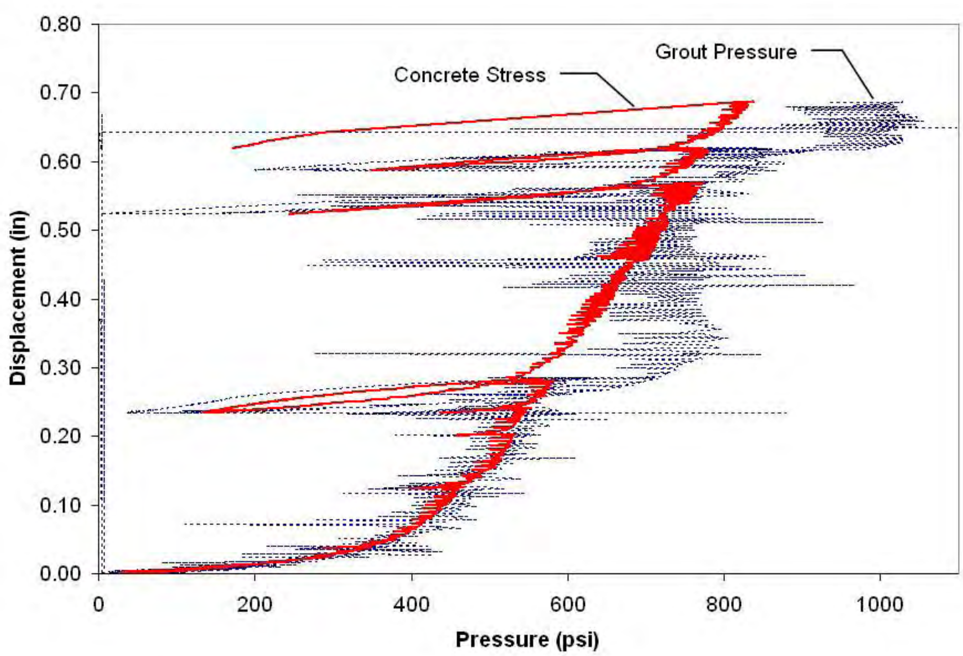


Figure 4-16 PGA Blvd: Shaft tip stress comparison with grout pump pressure.

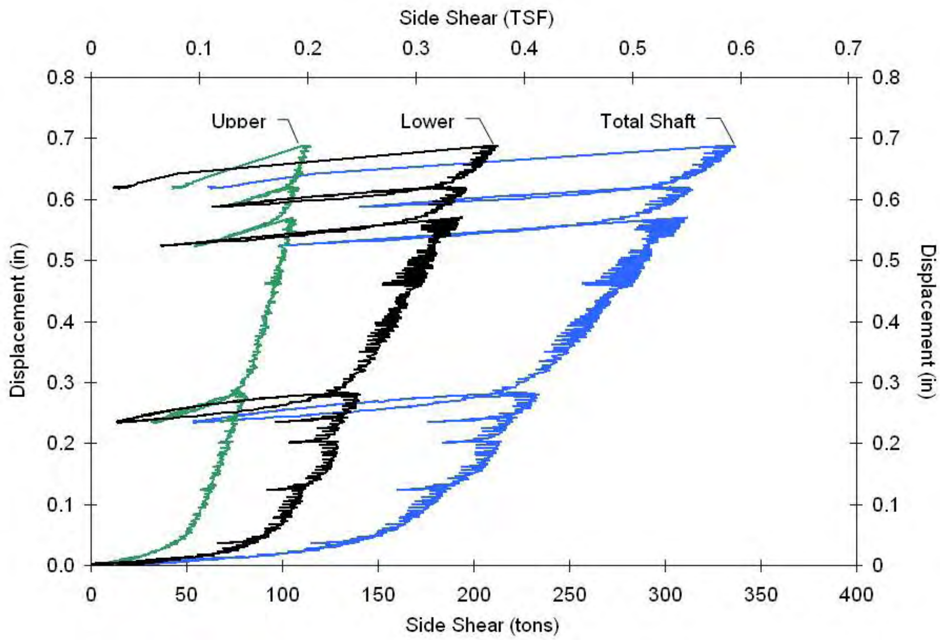


Figure 4-17 PGA Blvd: Side shear versus displacement during grouting.

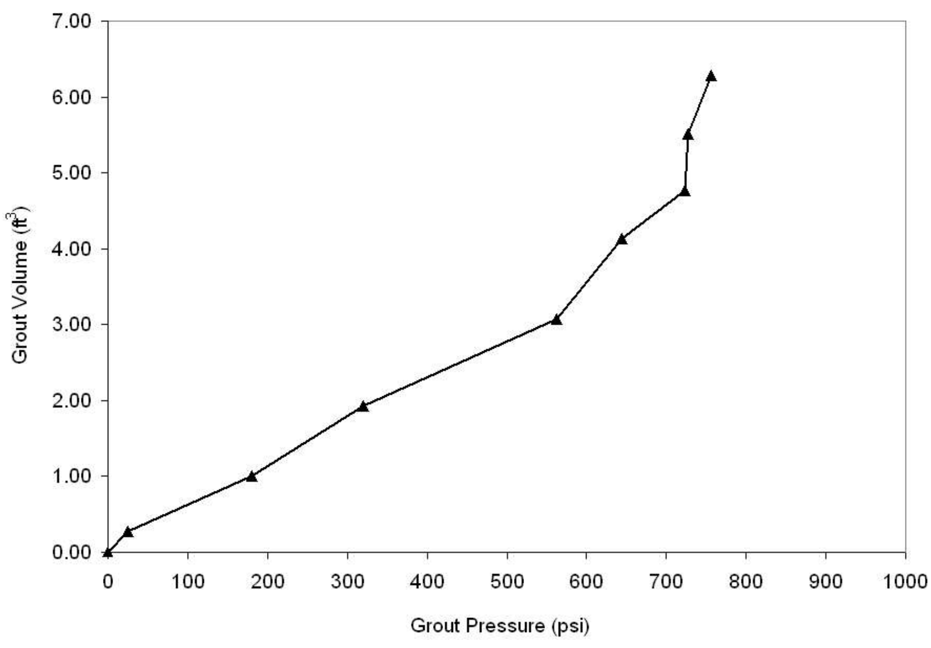


Figure 4-18 PGA Blvd: Grout volume versus grout pressure (LT-2).

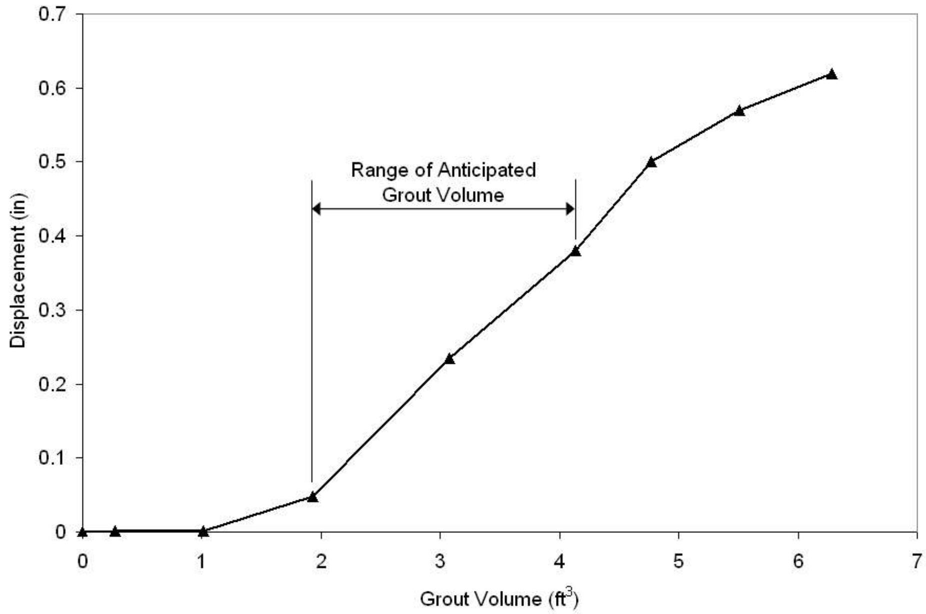


Figure 4-19 PGA Blvd: Grout volume versus displacement (LT-2).

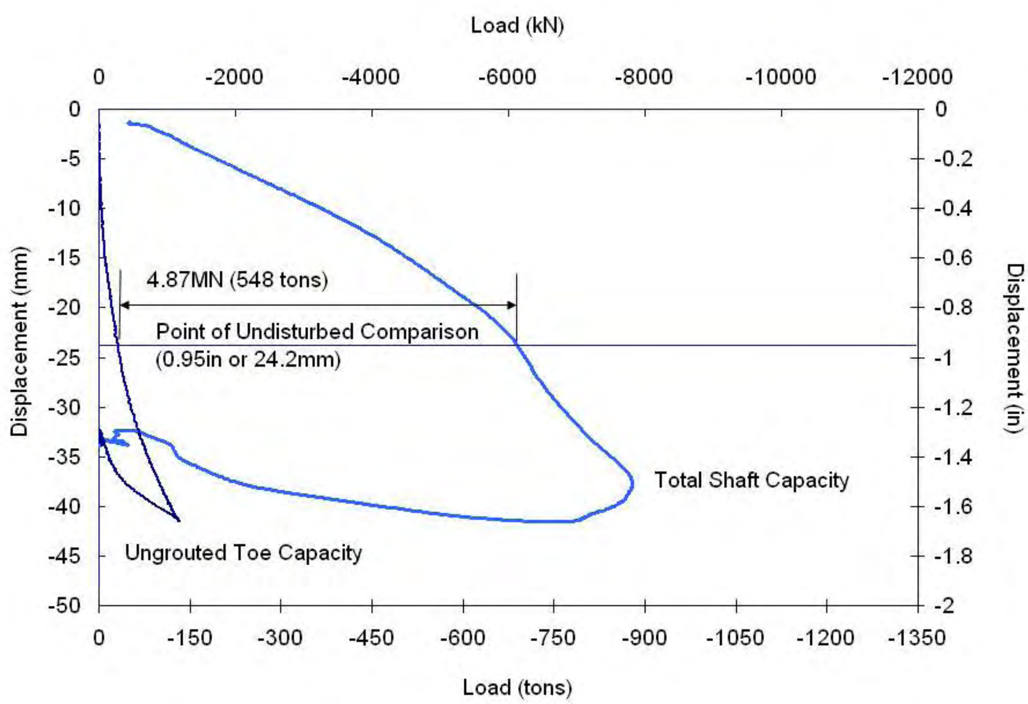


Figure 4-20 PGA Blvd: LT-1 total shaft and toe capacity curves.

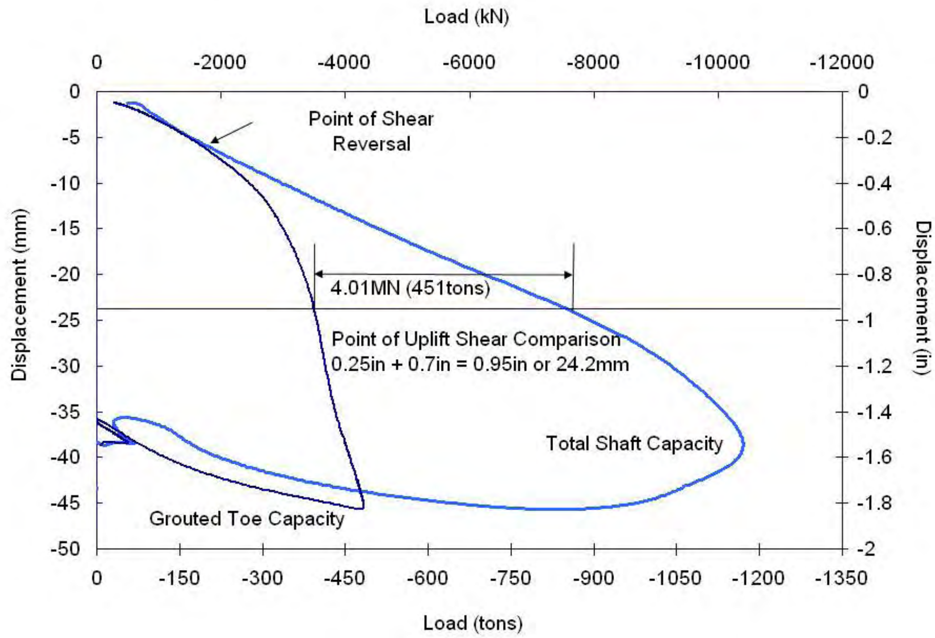


Figure 4-21 PGA Blvd: LT-2 total shaft and toe capacity curves.

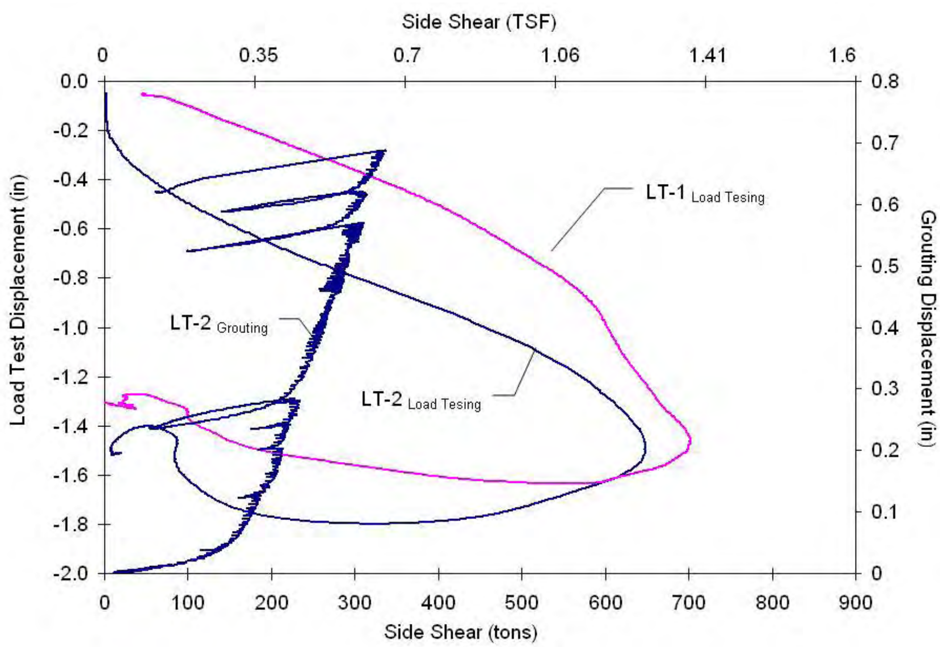


Figure 4-22 PGA Blvd: Side shear during load testing and grouting.

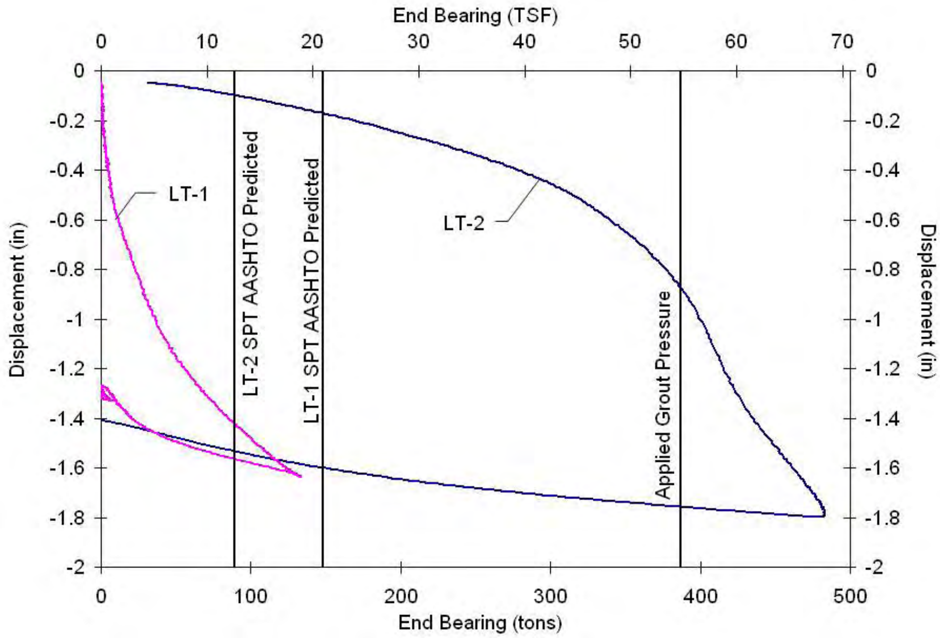


Figure 4-23 PGA Blvd: End bearing load plots with applied grout pressure and predicted AASHTO end bearing.

Figure 4-24 PGA Blvd: Post grouted production shafts.

Figure 4-25 PGA Blvd: Pier 3 shafts prior to post grouting.

Figure 4-26 PGA Blvd: Pier 2 prior to post grouting.


Figure 4-27 PGA Blvd: Pier 2 pile cap formation.


Figure 4-28 PGA Blvd: Pier 2 pile cap reinforcement.

Figure 4-29 PGA Blvd: Pier 2 pile cap reinforcement with column reinforcement.

Figure 4-30 PGA Blvd: Pier 2.

Figure 4-31 PGA Blvd: Southwest ramp Pier 2 production shafts.

Figure 4-32 PGA Blvd: End bent 4 columns.


Figure 4-33 PGA Blvd: Aerial photo.

Drilled Shaft Tip Post Grout Field Record	
Project Name: SR 786 PGA Blvd over SR 811 Grade Separation - Palm Beach County , Florida	
FIN Project No.: 229771-1-52-01	Shaft Designation: Pier 2 / 3 Right - Shaft -1
Contractor: Treviicos South	Post Grout Date: 8/6/02
Post Grouting By: Applied Foundation Testing, Inc.	Technicians: Damon Willis
	Engineer: Mike Muchard
Drilled Shaft Information	
Drilled Shaft Tip Diameter: 914 mm	Drilled Shaft Top Elevation: 1.8 m
Drilled Shaft Length: 14.39 m	Drilled Shaft Tip Elevation: -12.59 m
Shaft Construction Date:	Concrete Strength at Time of Grouting: Yes
Post Grouting Information	
Flat Jack Diameter: 610 mm	Grout Plant Type: HANY IC 310
Grout Tube Diameter: 20 mm I.D.	Pump Type: Single Stage Piston
Grout Tube Length: 15.39 m	Mixer Type: Colloidal Mixing w/Agitator Holding
No. of Grout Tubes: 3	Grout Type: Type II Portland Cement
Volume of Grout Tubes: 23.1 L	Water / Cement Ratio: 0.5 (22 liters water per bag)
Total Volume in Pump and Lines (L): 24.1	Yield: 36 Liters per bag (1.256 ft^3)
Post Grouting Criteria	
Maximum Permissible Displacement: 19 mm	Maximum Required Grout Pressure: 30.6 bar
Grout Volume Reset Value: 250 L (0.25 $m^{3)}$	

Post Grouting Data

Jetted tubes - clean water return 2:10 PM

Mix Grout 2:17PM

Time	Grout Pressure (bar)	Upward Shaft Displacement (mm)	Grout Volume (Liters)	Notes
2:22	0	0.0		
2:30	0	0.0	32.5	grout return
2:33	7	1.0	10.0	
2:38	18	5.0	20.0	
2:41	26	11.0	10.0	
2:43	31	19.0	10.0	
Totals	**31**	**19.0**	**82.5**	

Additional Comments

Final survey 2:48 PM no change

Conversions : Grout plant holding tank volume calibration = 1 inch in tank = 5 liters grout

(1 cubic meter = 1.307 cubic yards = 35.3 cubic feet) (1 Bar = 100 kPa = 14.7 psi)

(28.32 Liters = 1 cubic foot) (3.785 Liters = 1 Gallon) (25.4 mm = 1 inch) (1 kg = 2.21 lbs)

Figure 4-34 PGA Blvd: Typical drilled shaft post grout field record; Pier 2 / 3 Right - Shaft 1.

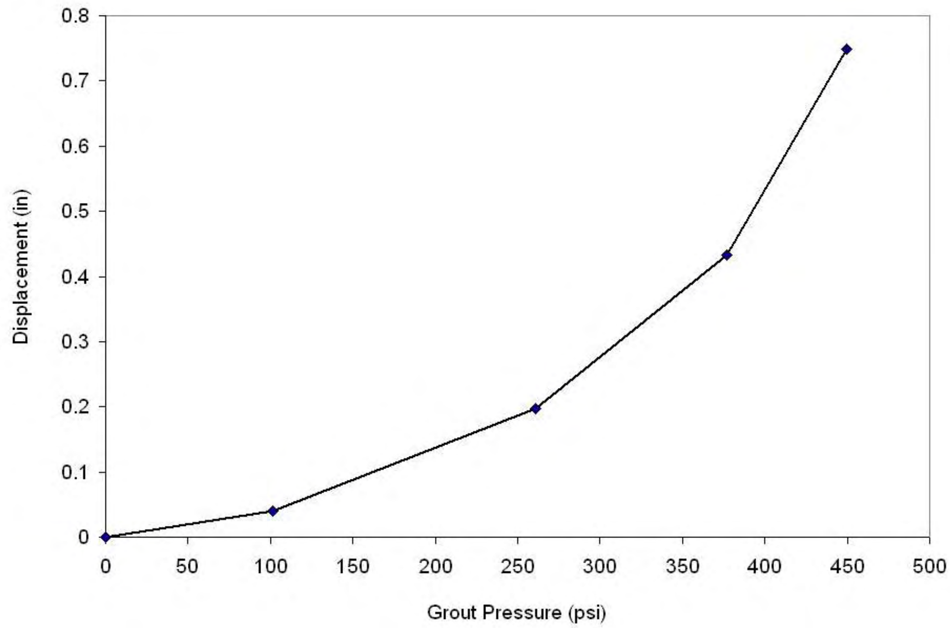


Figure 4-35 PGA Blvd: Typical grout pressure versus displacement from field grouting logs; Pier 2 / Footing 4 Right - Shaft 2.

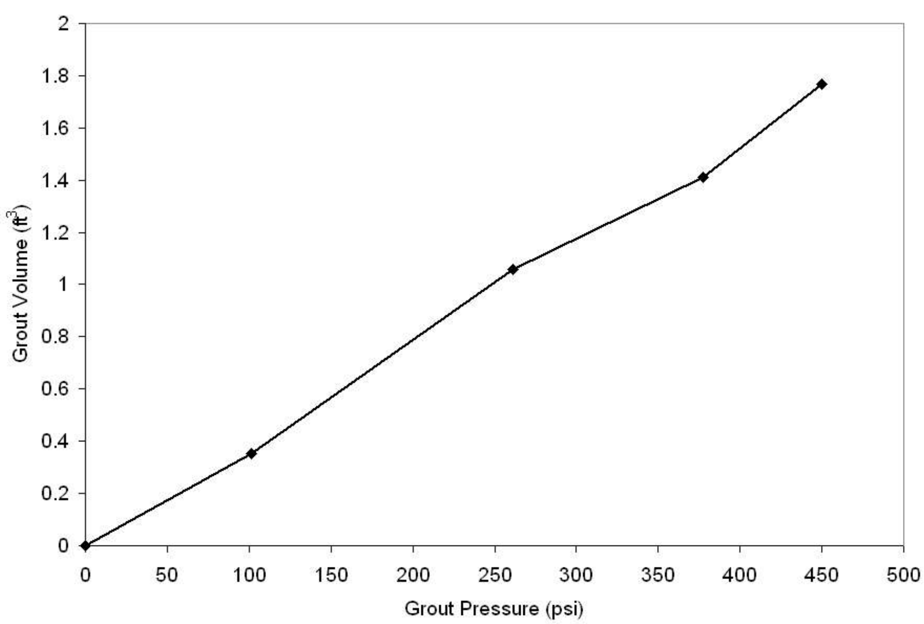


Figure 4-36 PGA Blvd: Typical grout pressure versus grout volume from field grouting logs; Pier2 / Footing 4 Right - Shaft 2.

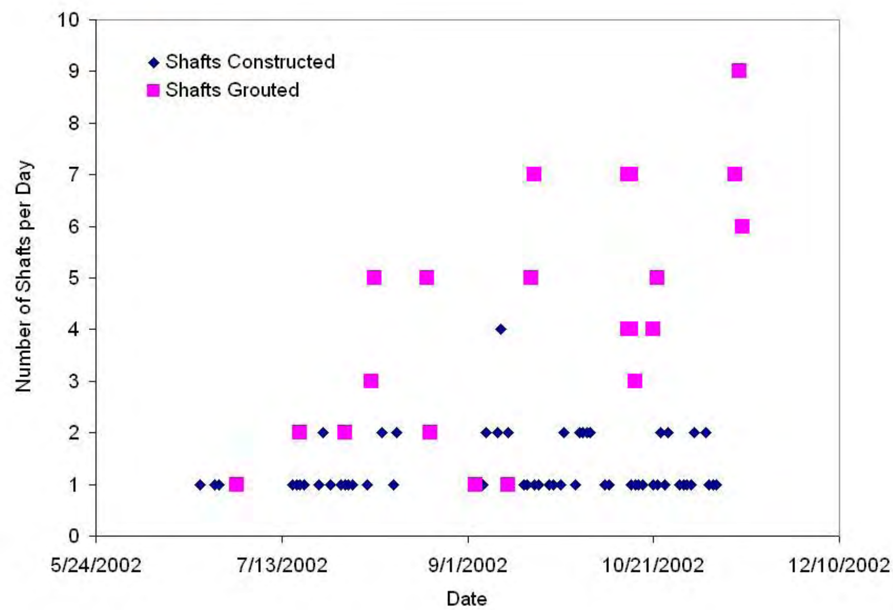


Figure 4-37 PGA Blvd: Production rates for shaft construction and grouting (Phase I).

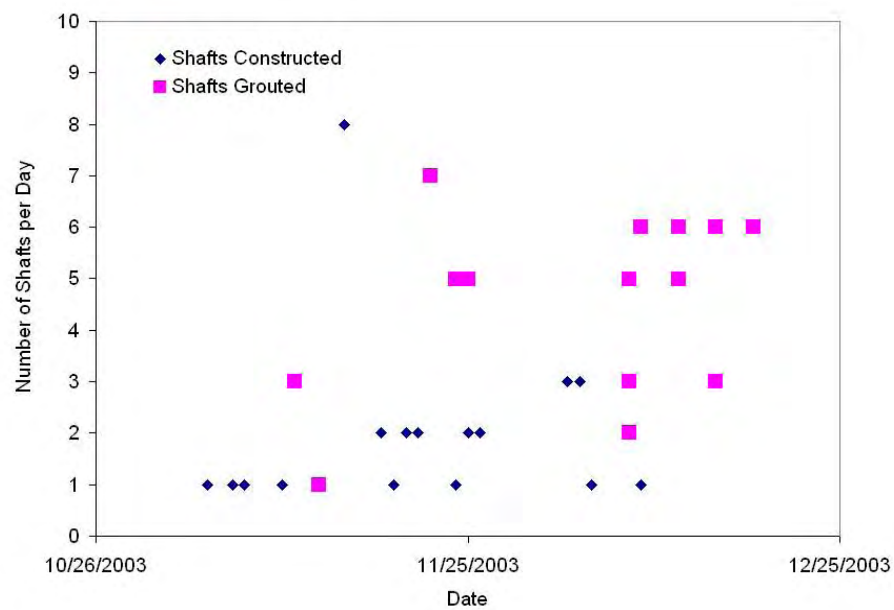


Figure 4-38 PGA Blvd: Production rates for shaft construction and grouting (Phase II).

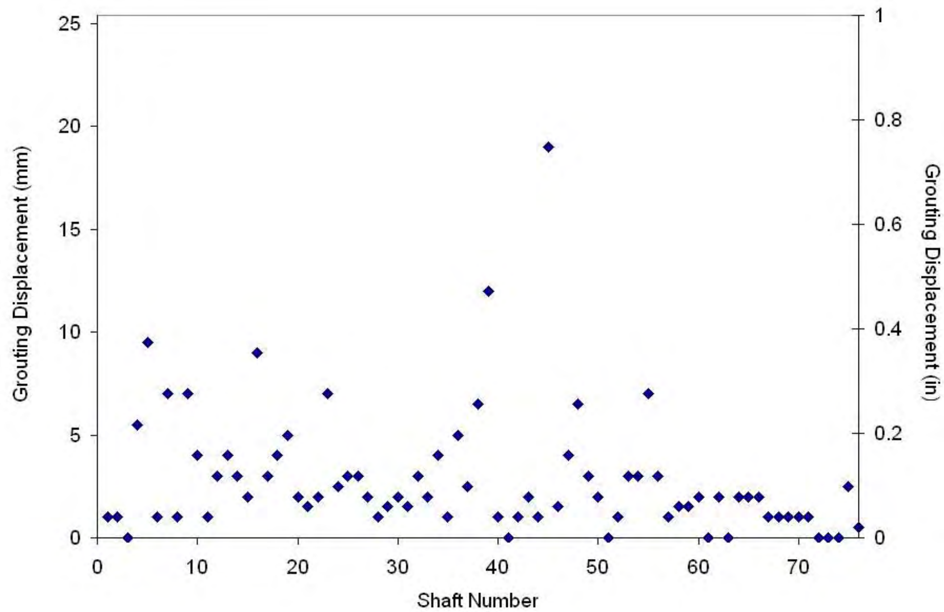


Figure 4-39 PGA Blvd: Phase I production QA data showing uplift for every shaft.

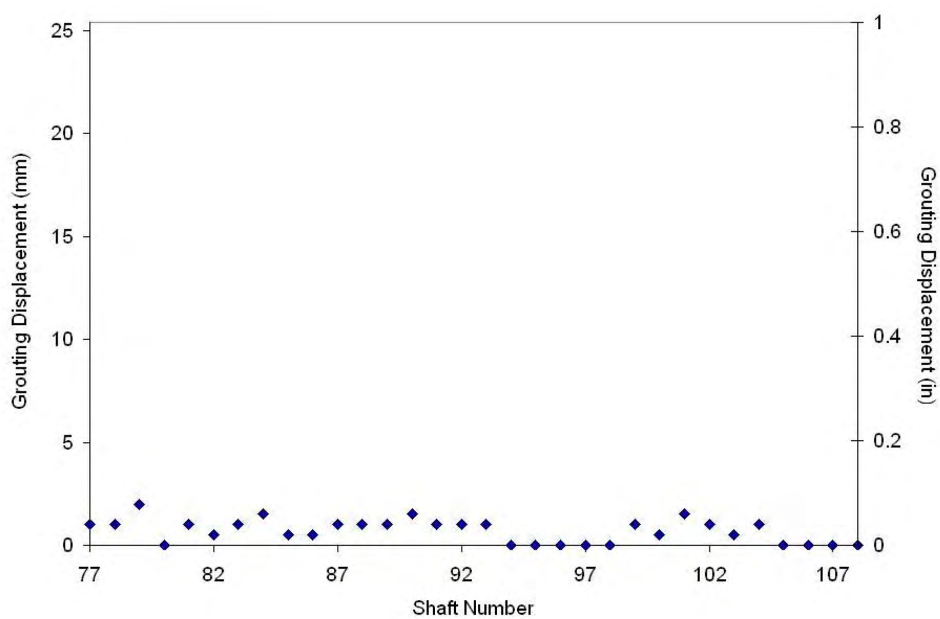


Figure 4-40 PGA Blvd: Phase II production QA data showing uplift for every shaft.

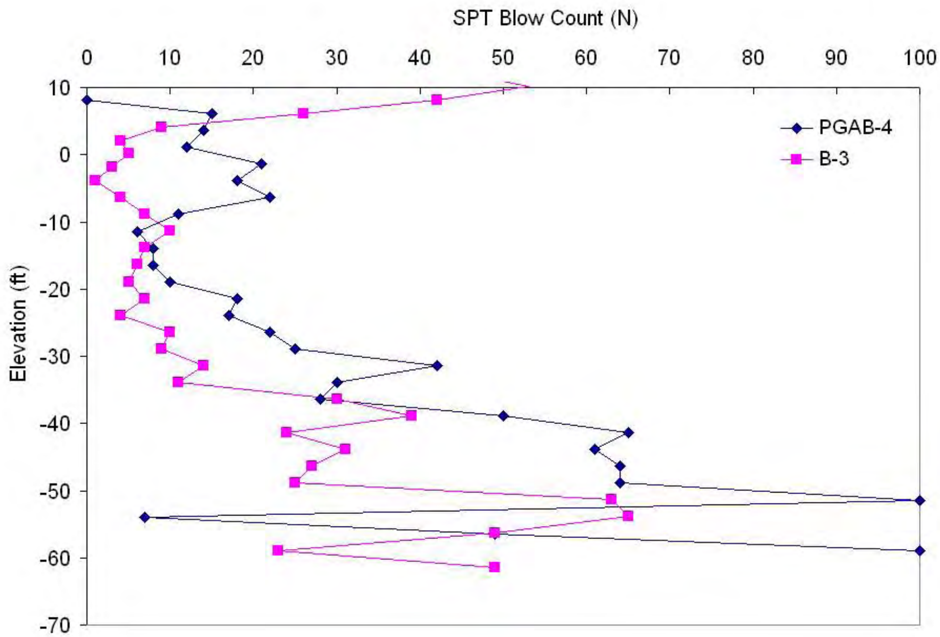


Figure 4-41 PGA Blvd: Variations in blow counts between manual and automatic hammers.

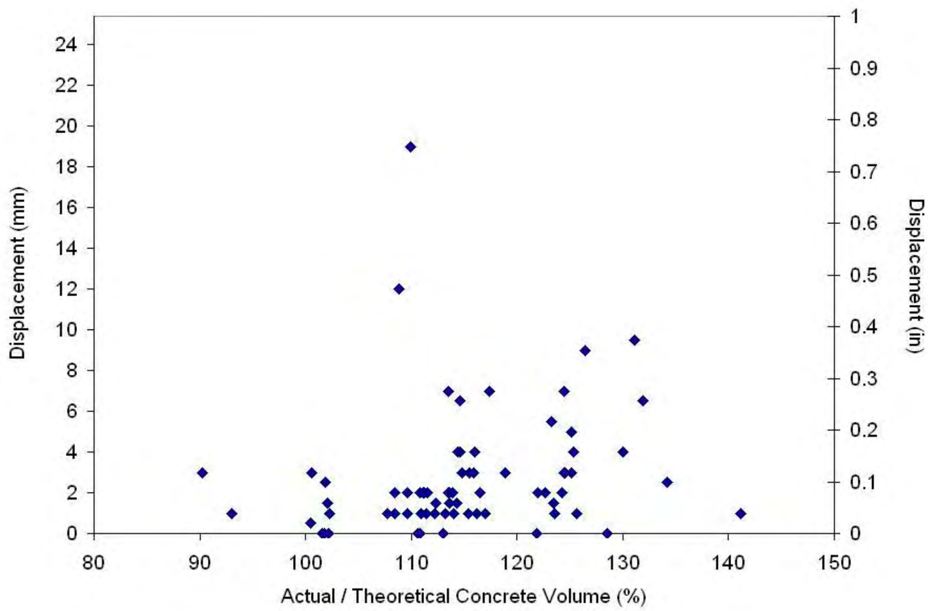


Figure 4-42 PGA Blvd: Concrete takes versus displacement for Phase I.

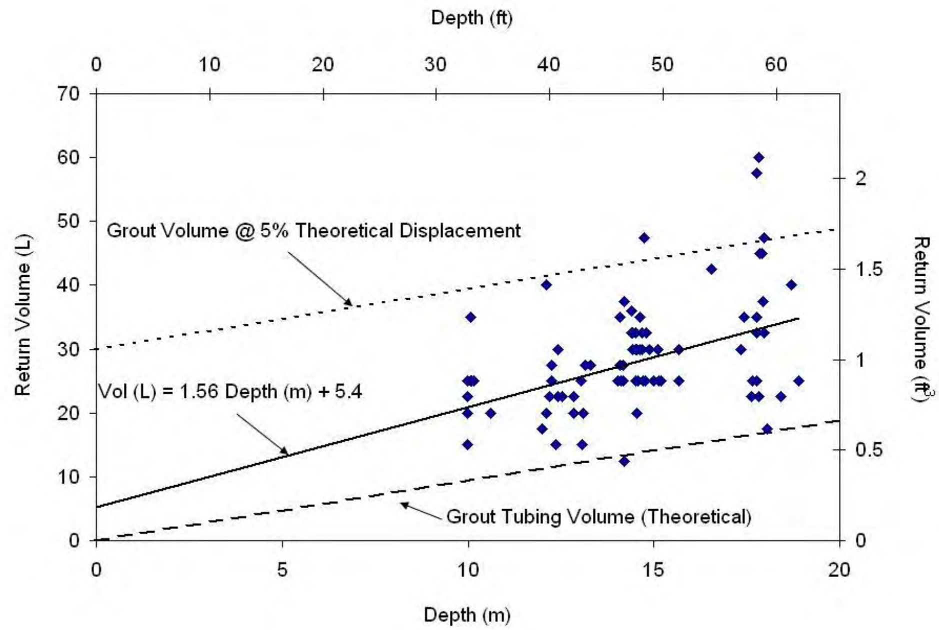


Figure 4-43 PGA Blvd: Volume of grout required to obtain return (no pressure).

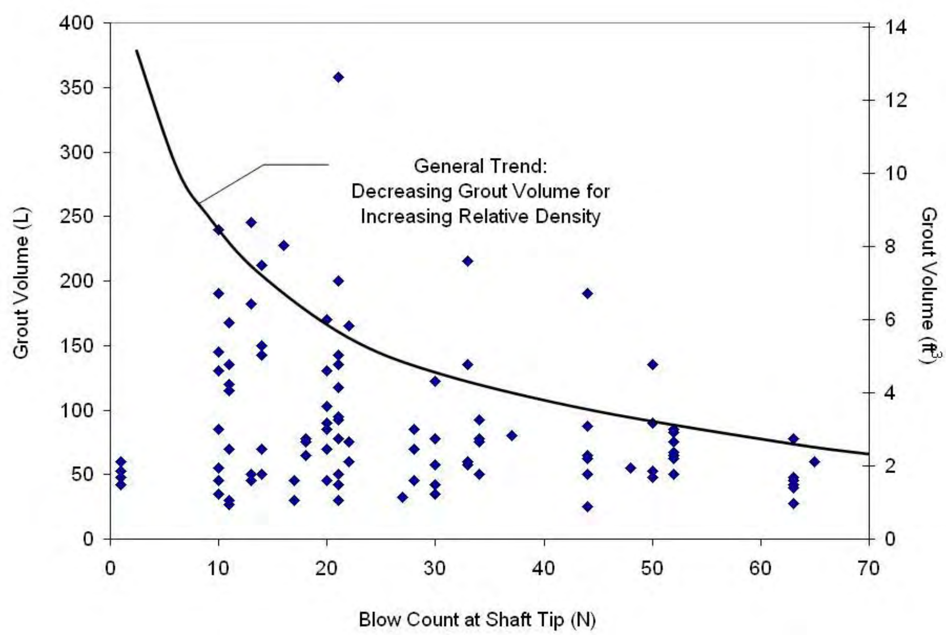


Figure 4-44 PGA Blvd: Net volume of grout placed beneath shaft tips.

4.2 The National Geotechnical Experimentation Site

The University of South Florida (USF) and the University of Auburn (UA) collaborated on a multi-purpose research project revolving around the construction and testing of five drilled shafts. The scope of this research project involved the investigation of concrete flow through reinforcement cages, various quality assurance tests, and the effectiveness of post grouting in silty soils. Although not the primary focus of this report, a brief description of each task is discussed below.

Due to the increasingly tight reinforcement cage spacings for heavy loads and seismic areas, concrete flow through these cages has been restricted and large differentials inside and outside the cages can be detected during construction. The first task of this research project was to vary the construction method, cage spacing, aggregate size, and slump and monitor the placement during construction and determine the quality post construction (i.e. permeability and aggregate distribution). A full description of this testing can be found in Mullins, et al., 2003.

The second task of this research project was to compare quality assurance tests in their ability to detect anomalies post construction. The various QA tests included Sonic Integrity Testing (SIT), Cross-hole Sonic Logging (CSL), and Thermal Integrity Testing. Each test shaft, with the exception to TS-4 (control), was constructed with known anomalies inside and outside the reinforcement cages. Similarly, a full discussion of this test program is available elsewhere (Mullins, et al., 2003).

The main focus of this test program with regard to this report was to examine the relative end bearing performance of five 3.5 ft diameter test shafts denoted as TS1, TS2, TS3, TS4, and TS5 in silty soils. The test shafts were embedded 24 ft deep. TS1, TS2, TS3, and TS5 were constructed for post grouting with TS4 constructed as a control shaft. The instrumentation and testing sequence is summarized as follows:

- Site exploration performed on October 16, 2002.
- Instrumentation and construction of test shafts on October 17, 2003.
- Post Grouting of TS1, TS2, TS3, and TS5 on October 29, 2002.
- Axial Statnamic testing of TS3 and TS5 on December 12, 2002.
- Axial Statnamic testing of TS1, TS2, and TS4 on December 13, 2002.
- Exhumed test shafts on February 28, 2003.
-

4.2.1 Soil Exploration and Site Layout

This grouted shaft test program was conducted at the National Geotechnical Experimentation Site for Silty Soils in Spring Villa, Alabama. This is a well-documented site showing relatively consistent soils throughout. The soil is typically classified as ML-SM. The general soil classification data are as follows: water content = 34%; grain size: 47% sand, 33% silt, 10% clay; LL=46, PI=10. Standard penetration test values are typically 8 to 14 blows/ft over the depth range of 6 to 50 ft. Lab test data show an effective cohesion of 17 kPa , an effective friction angle of 32 degrees, and an effective undrained shear strength of 92 kPa. Full documentation of the various geotechnical investigations can be found in Brown and Drew (2000), Mayne, et al. (2000), and Brown and Vinson (1998).

The subsurface investigation of the test site was performed by the University of South Florida using a mini-cone penetrometer. A CPT sounding was performed at the centerline of each test shaft location. The results of five CPT soundings are located in Appendix B. The site can be confirmed to have little variability when reviewing the CPT soundings. The location of the CPT soundings corresponded to the exact test shaft locations as shown in Figure 4-45.

4.2.2 Construction and Instrumentation

The 30 inch reinforcement cages were pre-assembled with 16 #9 bars and two DWYIDAG bars with #4 shear reinforcement (Figure 4-46). Each of the five drilled shaft reinforcement cages were instrumented with one level of four strain gages at the toe of the shaft. The strain gages for the test shafts were placed at a depth of 23 feet. The data obtained from these gages was used to delineate the load contribution from the end bearing and side shear during downward load testing. Each test shaft was also constructed with three Cross-hole Sonic Logging (CSL) tubes for various post construction quality assurance tests. The CSL tubes extended the full length of the test shafts. The ends of the CSL tubes were protected from soil, slurry, and concrete intrusion by placing thick layers of duct tape around the ends. Production grout plates (24 inch diameter) from PGA Blvd were modified for this testing (Figure 4-47). Figure 4-48 shows the strain gages, grout plate, and the CSL tubes at the bottom of the reinforcement cage.

The drilling of each test shaft was performed by a truck mounted, diesel drill rig equipped with a double flight auger. Figure 4-49 shows the drilling of TS-1. A borescope was used to inspect the cleanliness of each shaft and to monitor the concrete placement (Figure 4-50). Figures 4-51 through 4-54 show the construction of the test shafts. Quality assurance testing included CSL testing, Sonic Integrity Testing (SIT), and Thermal Integrity Testing prior to and after post grouting. SIT testing was performed on each shaft before grouting, after grouting, after load testing, and after exhuming the shafts. The goal was to capture characteristic changes in the toe reflection after post grouting the shafts. Figure 4-55 shows SIT testing on TS1. A brief discussion of the SIT test results are presented at the end of this Chapter.

4.2.3 Post Grouting

Post grouting of the four test shafts was performed in the same procedure as that of PGA Blvd. Figures 4-56 through 4-58 show the grouting of the test shafts. A brief summary of the post grouting is discussed below.

Grouting of four shafts was conducted in one day, first starting with test shaft TS3. The initial water / cement ratio was 0.5 which is within typical values of 0.4 to 0.6 when pumping neat cement grout through small diameter access lines. During the grouting procedure, the grout found a pathway through the end of the CSL tube and migrated inside the tube to the ground surface. The duct tape capping the CSL tubes was not sufficient to withstand the grout pressures; a serious oversight. Grouting was then stopped and the CSL tubes on all grouted test shafts were plugged to prevent the grout from migrating inside the tubes. Grouting continued until no additional pressure could be developed. The shaft was grouted to a sustained grout pressure of 100 psi (7.2 TSF).

The second shaft grouted was test shaft TS1. The shaft was grouted to a sustained grout pressure of 78 psi (5.6 TSF). No additional pressure could be developed. Upon completing the TS1 grouting, test shaft TS2 was then grouted. The shaft was grouted to a sustained grout pressure of 79 psi (5.7 TSF). The final test shaft, TS5, was then grouted to a sustained grout pressure of 105 psi (7.6 TSF). The summary of the grouting of the four test shafts is given below:

- Ten cubic feet of grout was pumped into each grouted test shaft.
- TS3 was grouted to a maximum sustained pressure of 100 psi and uplifted 0.132 inches.
- The maximum uplift for test shaft TS1 was 0.105 inches at 82 psi.
- The maximum sustained grout pressure for test shaft TS2 was 88 psi and the maximum uplift was 0.074 inches.
- The maximum uplift for test shaft TS5 was 0.075 inches at 109 psi.

4.2.4 Post Grouting Results

Grout Pressure vs. Disp. During the grouting of the four grouted test shafts, the displacement of the top of the shaft was measured with displacement transducers and the grout pressure was measured by an in-line pressure transducer at the grout pump. Figures 4-59 through 4-62 show the grout pressure and concrete stress over the duration of the grouting for TS-1, TS-2, TS-3, and TS-5 respectively. As expected, the concrete stress shows a similar magnitude to the grout pressure at the pump without the pump stroke response. Figures 4-63 through 4-66 show the grout pressure and concrete stress versus displacement for all four of the grouted shafts. Although only small uplift values were realized during grouting, the beginnings of a load-displacement response can be developed. The post grouting results are summarized as follows:

- TS-1 maximum grout pressure of 82 psi and uplift of 0.105 inches at 10 ft^3 of grout.
- TS-2 maximum grout pressure of 88 psi and uplift of 0.074 inches at 10 ft^3 of grout.
- TS-3 maximum grout pressure of 100 psi and uplift of 0.132 inches at 10 ft^3 of grout.
- TS-5 maximum grout pressure of 109 psi and uplift of 0.075 inches at 10 ft^3 of grout.

In each case, the maximum grout pressure that could be resisted by the bearing stratum (end bearing resistance) was insufficient to fully overcome the side shear and cause significant uplift. This silty soil fails in end bearing in a plunging fashion similar to clays and therefore is incapable of developing further capacity beyond the ultimate displacement (2.5%D for clay, Reese and O'Neill, 1988).

Side Shear. Unlike the PGA Blvd project where grout pressure limitations were dictated by practical construction limits, in silty/clayey soils the upper grout pressure limit is in line with the soils end bearing capacity as dictated by the consolidation state and the respective shear strength. As a consequence, grout testing of shafts tipped in clays will not always be able to verify the full side shear capacity, but rather may only be able to proof test the shaft up to the level permitted by the ultimate capacity of the end bearing strata. The proof load can be determined in those cases to be twice the tip area times the achieve grout pressure. Figures 4-69 through 4-73 show the side shear results for all the grout tests along with the downward load test results (except TS-5). A discussion of the downward load test follows this grout test section.

Grout Volume. As part of one of the other focuses of the NGES test program, each of the four grouted shafts were grouted in the same fashion to maintain experimental procedure. A uniform grout volume was adopted for all shafts after the completion of the first shaft. A net volume of grout between 1 and 3 cubic feet was anticipated for each shaft based on 2.5%D end bearing soil displacement and a cavity expansion multiplier of 3. However, uplift continued (although slowly) during the entire grouting process up to 10 ft^3 of grout. At approximately 5 ft^3 of grout volume, the grout pressure stopped increasing even after mild reductions in the w/c ratio. The remaining 5 ft^3 of grout was used in an effort to define full mobilization of the side shear. Upon exhuming the shafts, it could be seen that the grout had progressively migrated up the sides of the shafts (3 to 4 ft) as well as forming the anticipated grout bulb / layer beneath the shaft tip. Perhaps, the continuous uplift that had been noted during grouting was not the continuous / plastic failure of the side shear, but rather the progressive separation of the shaft sides from the surrounding soil. While the grout was fresh, this surface area contributed near-zero side shear to the shaft uplift resistance. The more grout placed, the less side shear and consequently progressive uplift. However, the amount of temporarily compromised side shear area is relatively small and is in the zone adjacent the tip which is discounted when computing the side shear for downward capacity. In any case, in silts and clays, grouting should be terminated when no increase in grout pressure can be obtained and / or when the grout volume exceeds the anticipated value of 3 times the 2.5%D displacement volume ($3 \times 0.025D \times \pi D^2/4$).

4.2.5 Axial Compressive Load Test

The axial Statnamic tests were performed using the University of South Florida 4MN Statnamic hydraulic catching device. Figures 4-67 and 4-68 show the statnamic test setup.

The top of each shaft was prepared for testing with a high-strength, fast-set grout mix. This allowed for a level surface for the load cell placement and to minimize eccentric loading of the test shafts.

The load test program had to demonstrate the full ultimate capacity of the shaft which in most cases is mobilized at a displacement of 2.5% of the shafts diameter. For the 42 inch diameter shafts on this site, a minimum of 1 inch displacement was required. Two cycles of loading were applied to each foundation to reach the target displacement. During the Statnamic load tests, many instruments were measured to obtain the foundation response. Directly mounted to the shaft top was the calibrated load cell to monitor the actual applied load. Shaft top displacement measurements were made with two capacitive accelerometers mounted 180 degrees from each other. The four embedded strain gages at the toe of each shaft were also monitored during loading.

4.2.6 Side Shear Results

The strain gages located at the toe of each shaft were used to calculate the side shear. A uniform cross-sectional area was assumed for all test shafts and modulus for the concrete obtained from cylinder testing. The load applied to the shaft was subtracted from the load calculated at the strain gages and divided by the surface area from ground surface to the strain gage location to provide the side shear results. Figures 4-69 through 4-73 show the side shear results of the downward load tests as well as the grout tests. In the case of TS-4 only downward data is presented, and in the case of TS-5 only uplift. Downward test data shows the two load cycles performed to achieve a target displacement of 2.5%D. Although uplift values were small during grouting, the side shear developed in those tests are similar to that shown by the downward testing at similar displacements.

The side shear results from the first load cycle of the downward testing of TS-1 through TS-4 are shown in Figure 4-74. This graph shows the relative performance of the grouted shafts with respect to the ungrouted TS-4. Unlike the PGA Blvd test results in sand where the uplift grout test caused negative side shear around the shaft (grouting uplift = 0.7 inches), the NGES shafts exhibited no shear stress reversal due to the small uplift (average uplift = 0.1 inches). However, similar to the PGA Blvd test results, the effect of uplift strain on ultimate side shear is not appreciable. The magnitude of side shear variation between each shaft could easily be more appropriately attributed to natural soil variation.

4.2.7 End Bearing Results

The end bearing calculated from the stain gages and the applied grout pressures for each test shaft are shown in Figure 4-75. During load testing, TS-3 did not reach the targeted 2.5%D displacement but rather only 1.9%D. The end bearing at 2.5%D displacement was 6.8 tsf, 6.8 tsf, 8.2 tsf and 3.5 tsf for shafts TS-1, TS-2, TS-3 (1.9%D) , and TS-4, respectively. The grouted shafts developed between 1.9 and 2.4 times more end bearing than the ungrouted control shaft (TCM). In each case, the end bearing mobilized during downward load testing was closely related to the applied grout pressure. The ratio of mobilized end bearing (at 2.5%D) to the applied grout pressure was 1.15, 1.07, and 1.14 for shafts TS-1, TS-2, and TS-3, respectively. Although these ratios are similar to those for shafts tipped in sands (Phase I, Mullins, et al., 2001), the ability to develop almost unbounded grout pressures was not possible in the silty soil. A recommended approach for silty soils will be discussed in Chapter 5.

4.2.8 Exhuming Test Shafts

The final phase of the multi-purpose research project at the NGES facility was to exhume the five test shafts for inspection. A 70 ton crane was brought in with a soil excavator to extract the five test shafts (17.3 ton deadload). Figure 4-76 through 4-79 show extraction of the test shafts. Each grouted test shafts showed the formation of a "grout-bulb" and grout migration up the side of the shafts. The ungrouted shaft, TS4, had reinforcement exposed at the bottom of the shaft. Figure 4-80 shows a comparison of an (a) ungrouted shaft versus a (b) grouted shaft from this test site.

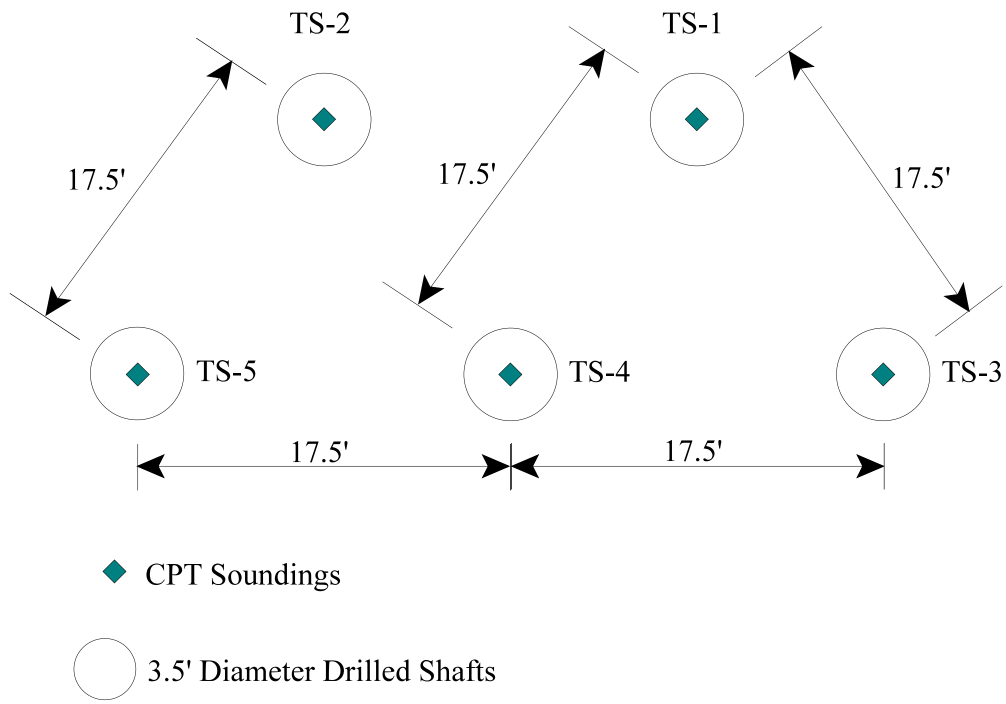


Figure 4-45 NGES Auburn test site layout.

Figure 4-46 NGES Auburn: Reinforcement cages.

Figure 4-47 NGES Auburn: 24" diameter grout plates.

Figure 4-48 NGES: Grout plate, lower strain gages, and CSL tubes installation.

Figure 4-49 NGES Auburn: Drilling of 42" diameter test shafts.

Figure 4-50 NGES Auburn: FDOT Borescope inspection.

Figure 4-51 NGES Auburn: Reinforcement cage installation.

Figure 4-52 NGES Auburn: Drilled shaft concrete pour.


Figure 4-53 NGES Auburn: Top of shaft finishing.


Figure 4-54 NGES Auburn: Test shafts.

Figure 4-55 NGES Auburn: SIT testing.

Figure 4-56 NGES Auburn: Grout pump.

Figure 4-57 NGES Auburn: Flushing grout lines.

Figure 4-58 NGES Auburn: Post grouting test setup.

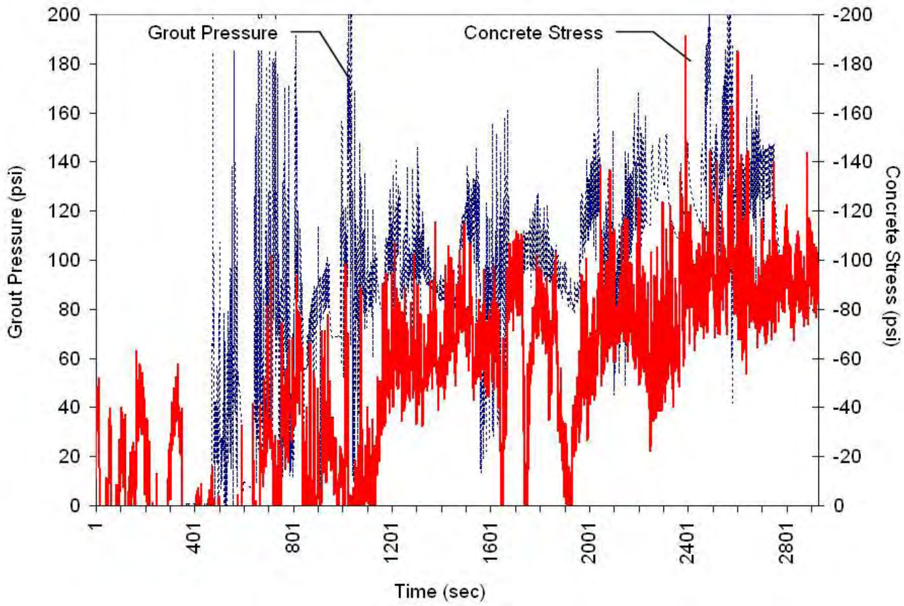


Figure 4-59 NGES Auburn: TS-1 Grout pressure and toe stress plots versus time.

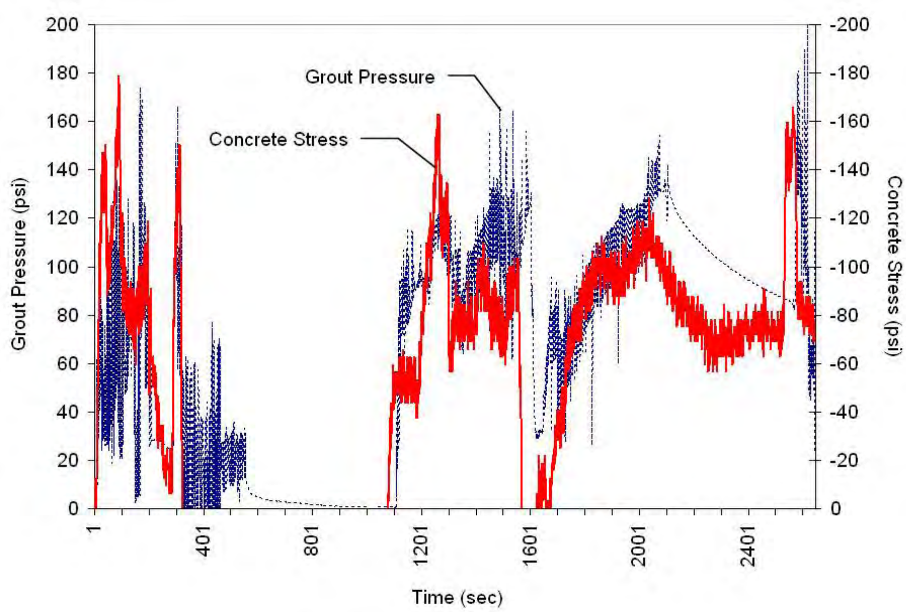


Figure 4-60 NGES Auburn: TS-2 Grout pressure and toe stress plots versus time.

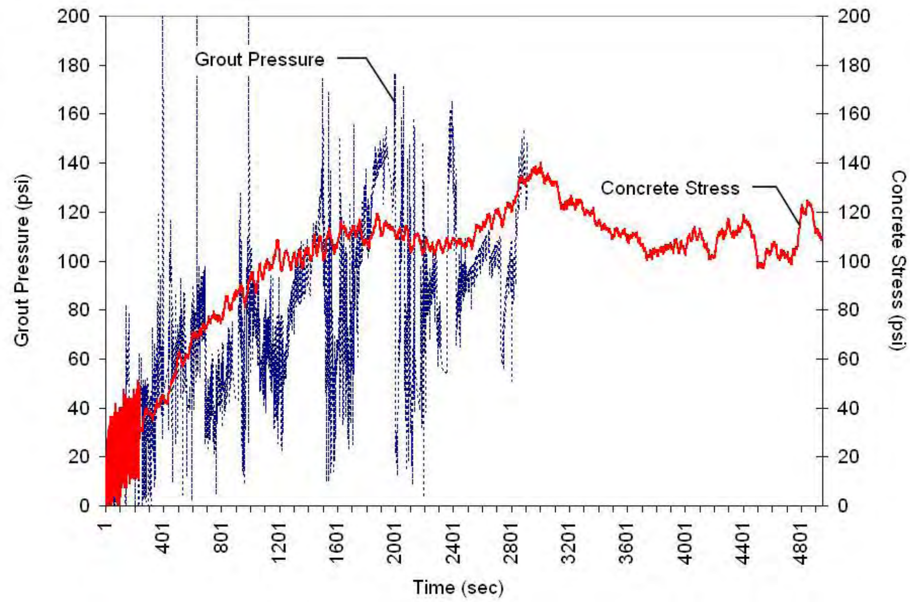


Figure 4-61 NGES Auburn: TS-3 Grout pressure and toe stress plots versus time.

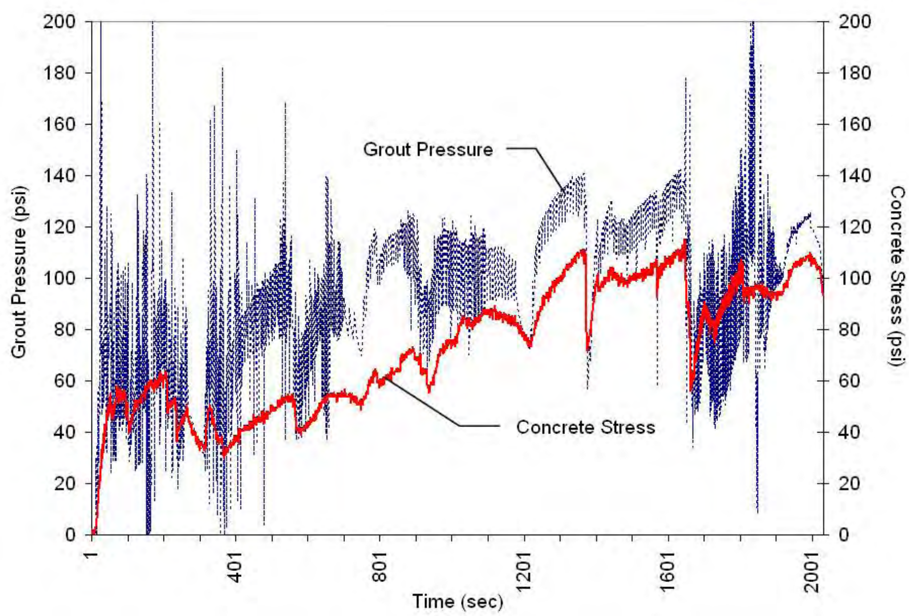


Figure 4-62 NGES Auburn: TS-5 Grout pressure and toe stress plots versus time.

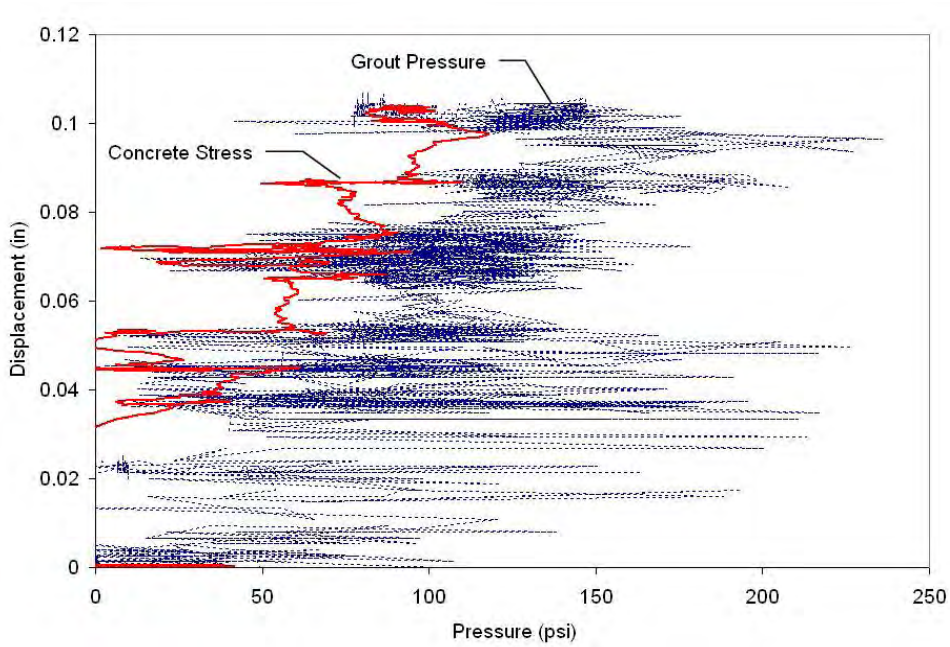


Figure 4-63 NGES Auburn: TS-1 grout pressure versus displacement.

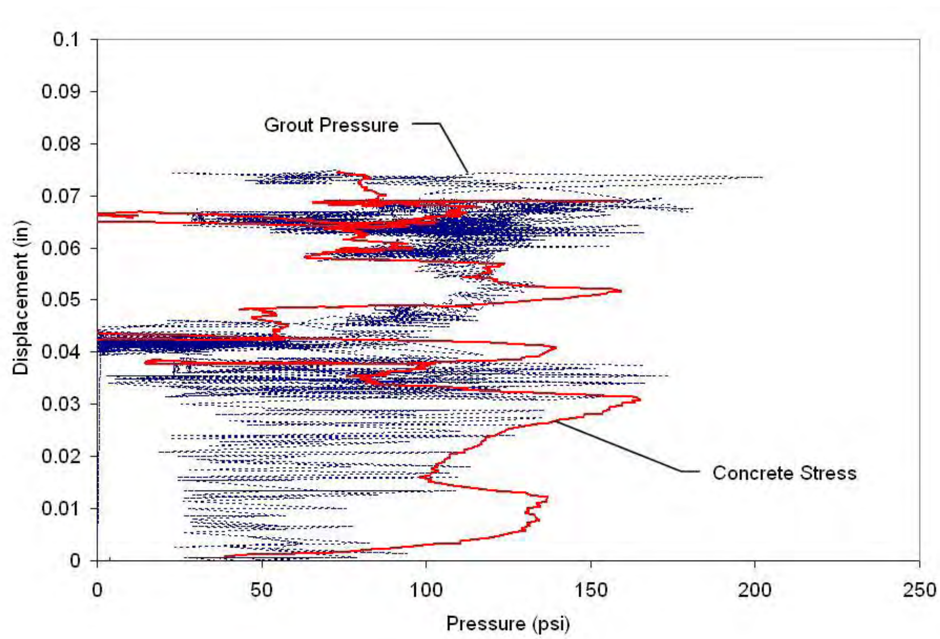


Figure 4-64 NGES Auburn: TS-2 grout pressure versus displacement.

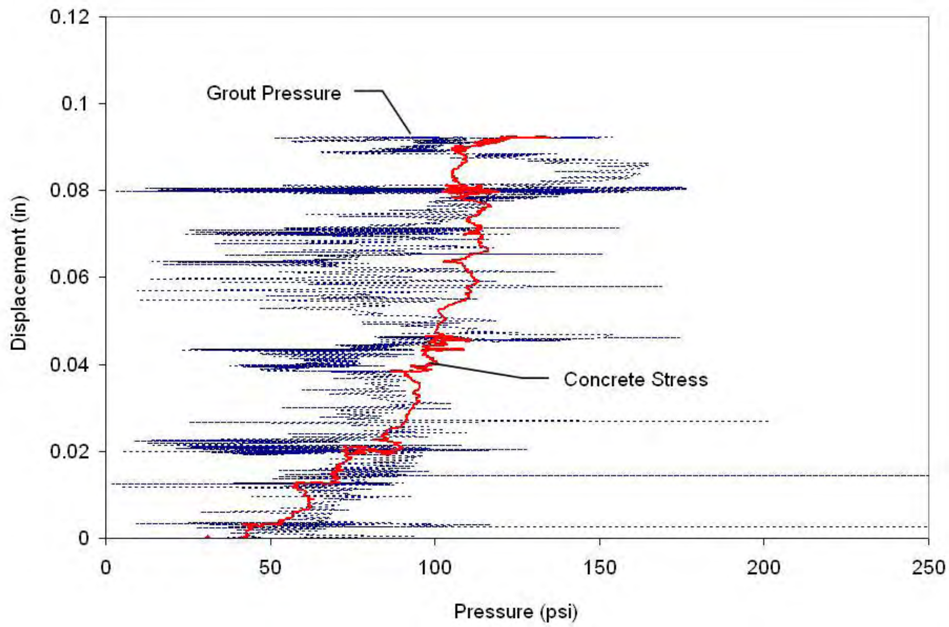


Figure 4-65 NGES Auburn: TS-3 grout pressure and concrete stress versus displacement.

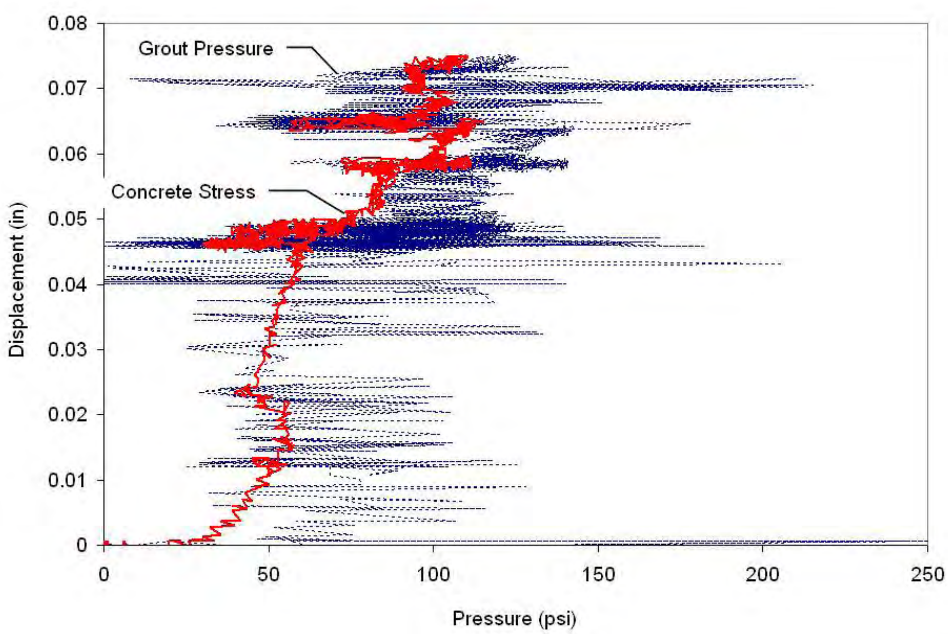


Figure 4-66 NGES Auburn: TS-5 grout pressure and concrete stress versus displacement.

Figure 4-67 NGES Auburn: Statnamic test setup.

Figure 4-68 NGES Auburn: USF & FHWA’s 4 MN Statnamic hydraulic catching device.

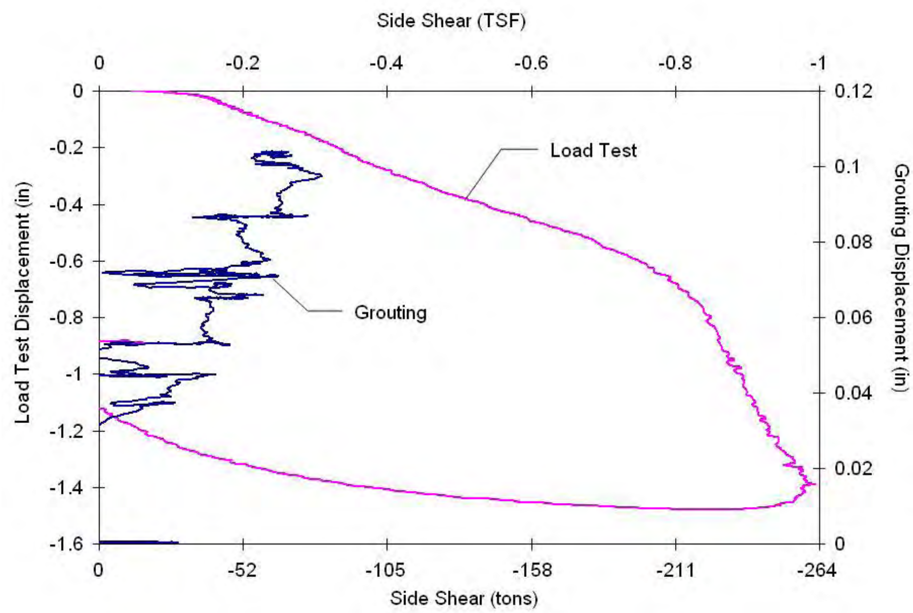


Figure 4-69 NGES Auburn: TS-1 Side shear plots.

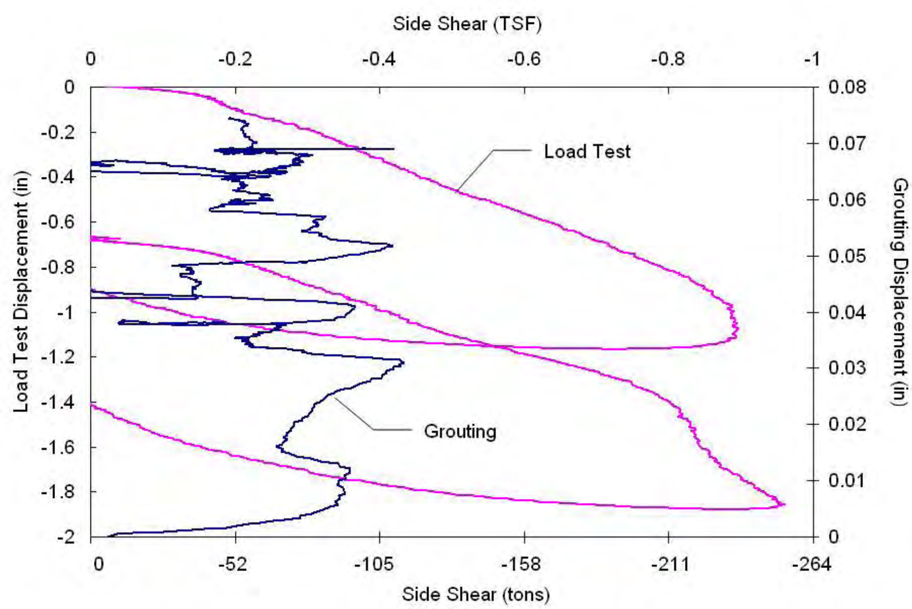


Figure 4-70 NGES Auburn: TS-2 Side shear plots.

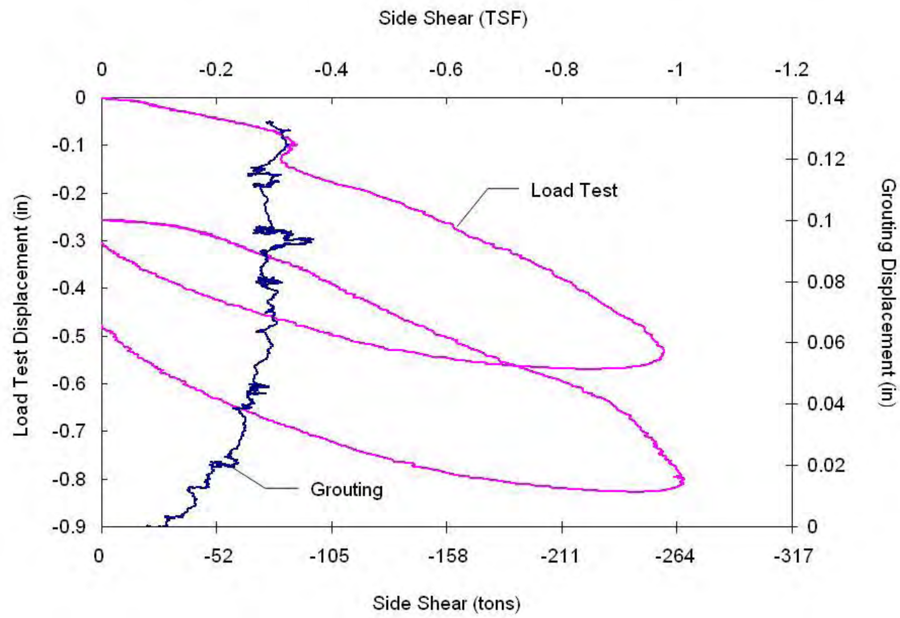


Figure 4-71 NGES Auburn: TS-3 Side shear plots.

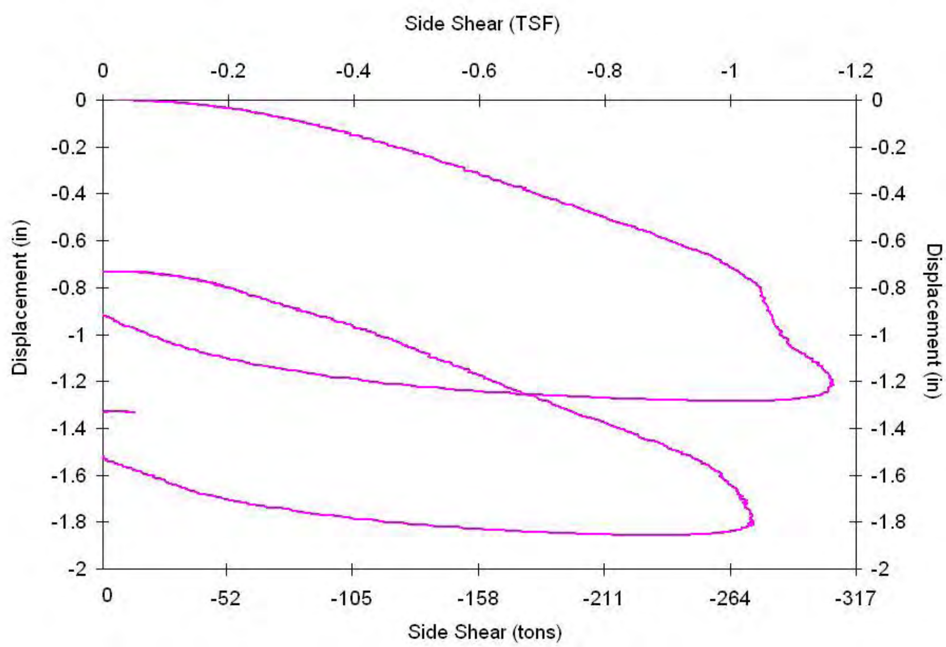


Figure 4-72 NGES Auburn: TS-4 Side shear plots during axial load testing.

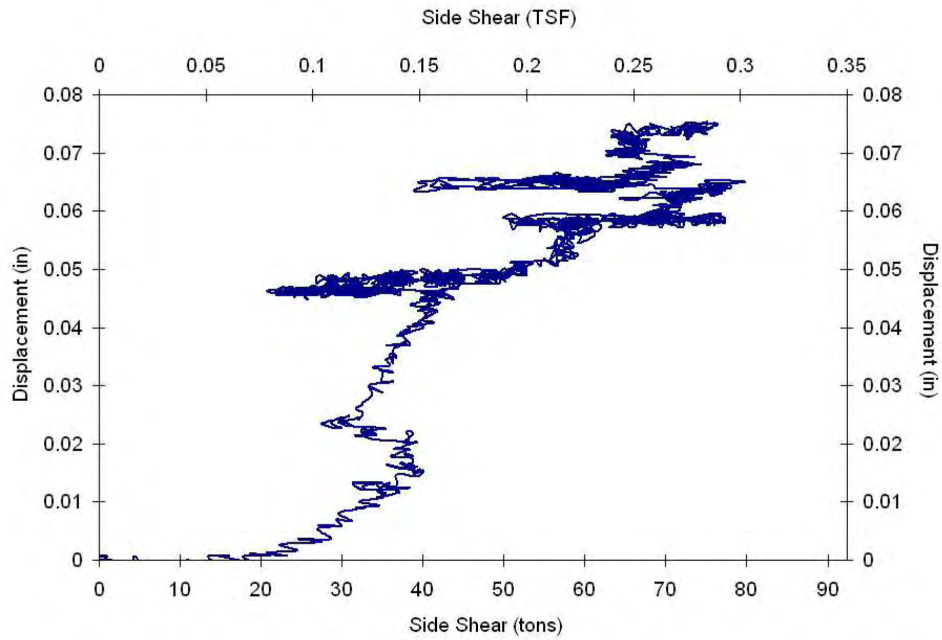


Figure 4-73 NGES Auburn: Side shear plot for TS-5 during grouting.

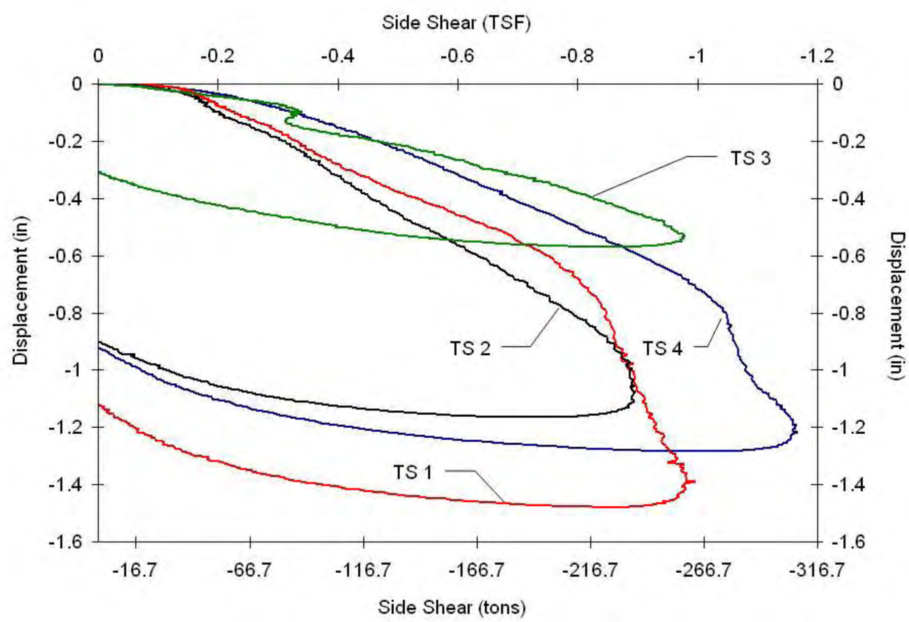


Figure 4-74 NGES Auburn: Side shear plots for load first load cycle.

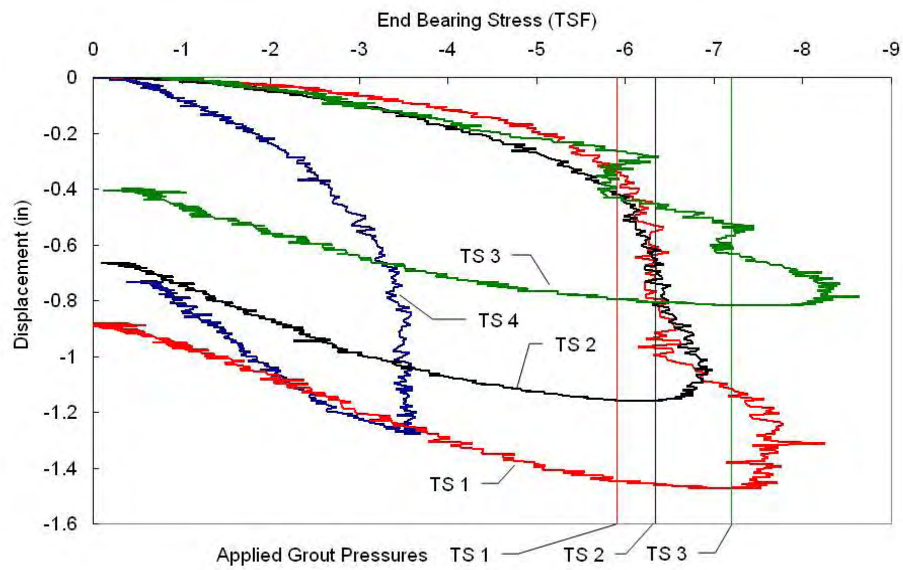


Figure 4-75 NGES Auburn: End bearing stress plots.

Figure 4-76 NGES Auburn: Exhuming of test shafts.

Figure 4-77 NGES Auburn: Soil excavator.

Figure 4-78 NGES Auburn: Shaft extraction.

Figure 4-79 NGES Auburn: Grouted shaft extraction.

Figure 4-80 NGES Auburn: Ungrouted shaft vs. Grouted shaft.

4.3 TexDOT Post Grout Demonstration

The University of South Florida (USF) and the University of Houston (UH) collaborated on this load test program to demonstrate the effectiveness of post grouting drilled shafts to the Texas DOT. This study revolves around the relative end bearing performance of four, 4' diameter test shafts denoted as S1, S2, C1, and C2. The "S" series shafts were embedded 21 ft and tipped in sand (i.e., S). The "C" series shafts were embedded 50 ft and tipped in clay. The 1 and 2 denotation refers to ungrouted and grouted, respectively. Figure 4-81 shows the test site layout. The comparison of their performance stems from downward compression load test results which are presented in an ensuing section. The instrumentation and testing sequence is summarized as follows:

- Instrumentation of shafts C-1, C-2, S-1, and S-2 on April 16, 2003.
- Completion of shafts C-1 and S-1 (Control Shafts) on April 17, 2003.
- Completion of shafts C-2 and S-2 (Grouted Shafts) on April 18, 2003.
- Post Grouting of S-2 and C-2 on April 25, 2003.
- Axial Statnamic testing of S-1 on April 29, 2003.
- Axial Statnamic testing of S-2, C-1, and C-2 on April 30, 2003.

4.3.1 Soil Exploration and Site Layout

The subsurface investigation of the test site was performed using three primary methods of exploration: Standard Penetration Tests, SPT; Texas Cone Penetration Tests, TCP; and Static Cone Penetration Tests, CPT. Figure 4-82 shows the CPT sounding for test shaft C-1. The results of six CPT soundings, two TCP tests, and two SPT borings are shown in Appendix B. Although some variation is noted between the two TCP profiles as well as the two SPT borings, the site can be confirmed to have little soil type variability when reviewing the CPT soundings. CPT soundings 2, 4, 5, and 6 are shown in Figure 4-83 along with the SPT and TCP results. The reproducibility of CPT data is well documented, when compared to SPT type tests. The location of the CPT soundings corresponded to the exact test shaft locations. A full geological discussion for the Houston area soils can be found elsewhere (Mullins and O'Neill, 2003).

4.3.2 Construction and Instrumentation

The test shafts reinforcement cages were constructed on site with 18 #9 deformed bars with #4 shear reinforcement at 6 inch centers (Figure 4-84). Each of the four reinforcement cages were instrumented with three levels of four strain gages. Figure 4-85 shows the installation of the strain gages. The location of these levels was selected based on the elevations at which a distinct soil strata change could be established from the CPT. The strain gages for the "S" series shafts were placed at depths of 4', 7', and 20'. The strain gages for the "C" series shafts were placed at depths of 10', 25' and 49'. The data obtained from these gages was used to delineate the load contribution from the various layers as well as to differentiate between end bearing and side shear. The grout plates used in this testing were 36 inch diameter. Figures 4-86 through 4-88 show the installation of the rubber membrane onto the grouting plates, the installation of the grout plates on the reinforcement cages, and the installed grout plates and strain gages, respectively.

The construction of the four test shafts was performed in a series of two days. The first day the control (ungrouted) shafts were drilled with the grouted shafts being drilled the second day. Figures 4-89 through 4-92 show the construction of the test shafts. SIT testing was performed on each shaft before grouting, after grouting, and after load testing to cooperate the results from the NGES SIT testing. Figure 4-93 shows SIT testing on test shaft C-2.

4.3.3 Post Grouting

The grouting (Figure 4-94) of two test shafts was conducted in one day, first starting with the shorter grouted shaft (S-2). The initial water/cement ratio was 0.55 which is within typical values of 0.4 to 0.6 when pumping neat cement grout through small diameter access lines. However, the higher the w/c ratio the higher the mobility of the grout through the surrounding soils. In this case, the 0.55 ratio migrated up one side of the shaft and found a pathway to the surface. Further, the rate of grout placement may have been too high to permit the grout bulb to squeeze some of the water from the outer boundary of the grout bulb into the surrounding soils (an advantageous mechanism when grouting in sandy soils). As a result, higher pump rates of a high mobility (high w/c ratio) grout can hydro-fracture the soil. When the design pressure cannot be obtained and the upward displacement is not excessive (as in this case), the contingency is to flush the grout lines with clean water and confirm that there is a clear pathway to conduct grouting at a later time. This technique is termed “staged grouting.” The second stage of grouting will typically use a lower w/c ratio and is conducted once the first stage grout has begun to set.

The second shaft grouted was test shaft C-2 using a lower w/c ratio of 0.45. Use of the lower w/c ratio in all probability minimized grout mobility thus eliminating the need for a second stage. The shaft was grouted to a sustained grout pressure of 150 psi (10.5 tsf) which was slightly in excess of the anticipated end bearing failure stress. No additional pressure could be developed. Upon completing the C-2 grouting, the same w/c ratio grout was then applied to test shaft S-2. Grouting continued until no additional pressure could be developed. The summary of the grouting of the two test shafts is given below:

- The first stage of S-2 was conducted to sustained pressures up to 90 psi and uplifted 0.125 inches. Grouting was terminated when pressure could not be sustained due to migration. Lines were flushed for subsequent grout stage.

- The maximum uplift for test shaft C-2 was 0.18 inches at 150 psi. Grouting was terminated when no additional pressure could be achieved while uplift continued. Grout pressure locked in by closing sacrificial in-line valves.

- The maximum sustained grout pressures for the second stage of S-2 was 220 psi and the maximum cumulative uplift was 0.48 inches. Grouting was terminated when uplift continued without an increase in grout pressure.

4.3.4 Post Grouting Results

Grout Pressure vs. Disp. The grout pressure versus displacement and survey field notes are shown in Figures 4-95 and 4-96 for S-2 and C-2, respectively. Shaft C-2 had a maximum grout pressure of 150 psi at a displacement of 0.18 inches. Shaft S-2 had a maximum grout pressure of 220 psi at a displacement of 0.48 inches. Figures 4-97 and 4-98 show the concrete stress and grout pressure at the toe for C-2 and S-2, respectively. The grout pressure is measured at the grout pump whereas the concrete stress is measured at the toe of the shaft. Differences in concrete stress versus grout pressure account for head loss in the grout tubes. However, when low flow rates are encountered, these two values are reasonably similar from the standpoint of using grout pressure at the pump to determine grout pressure at the toe (production grouting). This is the primary reason for requiring sustained grout pressure when production grouting to a design grout pressure. In load test programs such as those outlined in this chapter, the upper limit for grout pressure is left open to be determined. Therein, flow rates may not allow one-to-one correlation between these two values. This reinforces the need for toe strain and pump pressure measurements when performing a pilot grout study.

Grout Volume. Grout volume was also recorded in the field survey notes. Figure 4-99 shows the grout volume versus grout pressure for test shaft C-2. A total of 22 cubic feet of grout was pumped into C-2 at a slow rate which produced the appearance of full side shear development. Beyond 5 cubic feet of grout volume no increase in grout pressure could be developed which suggests a reasonable criterion for grouting termination. Although slight amounts of uplift continued during this time, the ultimate side shear displacement typical of this soil had not yet been achieved. When contrasting the displacement response with the grout volume (Figure 4-100) a relatively constant relationship between grout volume and displacement are observed beyond the 5 cubic feet (the point where no additional grout pressure could be achieved). When conducting a pilot grouting test, the relationships between pressure, volume, and uplift should be continuously reviewed to best evaluate the progress. Recommendations for this evaluation are provided in Section 5.4.

Shaft S-2 was grouted in two stages, the first stage was grouted with a relative high w/c ratio (0.55) which is within the recommended range of 0.4 - 0.6. However, this w/c ratio in conjunction with a relatively high pumping rate allowed the grout to migrate up the side of the shaft instead of "compaction grouting" the surrounding sands. Grout pressure could not be developed to the anticipated/design grout pressure (Figure 4-101). The rate of pumping is somewhat site/soil dependant but must be placed slowly enough to allow the grout at the grout/soil interface to squeeze out some of its water content thus forming a grout cake layer on which the more viscous grout can then press. This grout cake layer forms relatively quickly in free draining soils. The mechanism of grout cake formation in free draining soils should be understood by the grouting personnel to help assure that the design grout pressure is attained without needless grout volume or without requiring staged grouting. A second grouting stage (conducted later) was applied at slower rate using a lower w/c ratio of 0.45. The volume of grout shown in Figures 4-101 and 4-102 from 2 to 7 cubic feet is actually not contributing to the enhancement sought from grouting. Out of the total volume of 9 cubic feet used, only 3.5 cubic feet produced effective end bearing improvement (in stage 1, 1 - 2.5 ft^3 and in stage 2, 7 - 9 ft^3). The return volume is always required to fill the grout tubes although it provides no end bearing enhancement.

4.3.5 Axial Compressive Load Test

The axial Statnamic tests were performed using a 16MN mechanical catching device. Figure 4-103 shows the setup of the statnamic frame on test shaft C-2. The target of this load test program was to produce a 2.5% diameter displacement , 1.2 inches, for the shafts tipped in clay and a 5% diameter displacement, 2.4 inches, for the shafts tipped in sand. A load cell, three capacitive accelerometers mounted 120 degrees from each other, and the strain gages were monitored during the Statnamic load test. A survey level was also used to verify the final displacement of the test shafts. Figure 4-104 shows the statnamic load testing of C-2.

4.3.6 Side Shear Results

The side shear response from the grouting and axial load tests are shown in Figures 4-105 and 4-106. Shaft C-2 exhibited what appeared to be a full shearing response during grouting at 0.025 inches with an ultimate average side shear of 0.2 tsf (Figure 4-105). However, downward loading of the same shaft produced ultimate average side shear of 0.4 tsf. It is not uncommon for upward load tests to produce lower side shear than downward tests, but the difference is more commonly less drastic. The ungrouted shaft, C-1, developed an ultimate side shear of 0.8 tsf in downward loading. Shaft S-2 developed 0.9 tsf and 1.1 tsf in ultimate side shear (Figure 4-106) for upward and downward loading, respectively. This gives a side shear load direction ratio of 0.8. The ungrouted side shear for S-1 is 1.3 tsf in downward loading. Both Figures 4-105 and 4-106 include the computed side shear for various design methods as a reference.

Two scenarios that could have lead to this difference given that the site was reasonably uniform (i.e. soil boring logs and CPT soundings): the in-situ soil is sensitive to stress reversal or the construction varied slightly from day 1 (S-1 and C-1) and day 2 (S-2 and C-2). The “C” series tests were upwardly displaced the least, but yet exhibited the largest difference which may tend to minimize the concerns over reversal. Although the CPT tip stresses recorded for the two sets of shafts were similar throughout, the local friction values varied between C-1 and C-2 as well as S-1 and S-2. In both cases, the soundings at the location of the ungrouted shaft exhibited higher local friction values. However, it is not inconceivable that the excessive volumes of grout pumped into each of the grouted shafts dislodged and/or disrupted the interaction between the soil and shaft sides. If this is the case, it could have been avoided by more closely monitoring the grout volume, grout pressure, and uplift responses during grouting (recommended).

4.3.7 End Bearing Results

Figure 4-107 shows the end bearing stress for both C-1 and C-2, the predicted end bearings, and the applied grout pressure. The end bearing stress was calculated from the monitored strain gages assuming a uniform cross-sectional area based on a 4 ft diameter drilled shaft and a concrete modulus from cylinder testing. The end bearing stress for C-1 and C-2 were 11 tsf and 19 tsf at 2.5%D displacement, respectively. The predicted end bearings were calculated from the SPT-1, TCP-1, TAT (UU triaxial compression tests), and the associated CPT soundings. The predicted end bearing values were 7.2 tsf, 6.4 tsf, 8.0 tsf, and 3.5 tsf, respectively. The applied grout pressure across the base of the shaft was 10.8 tsf.

Similarly, Figure 4-108 shows the end bearing stress for both S-1 and S-2, the predicted end bearings, and the applied grout pressure. The end bearing stress for S-1 and S-2 were 25 tsf and 36 tsf at 5%D displacement, respectively. The predicted end bearings were calculated from the SPT-1, TCP-1, and the associated CPT soundings and were 16.2 tsf, 25 tsf, and 5.8 tsf, respectively. The applied grout pressure across the base of the shaft was 15.8 tsf.

The grouting beneath the tips produced end bearing improvement in both the clay and the dense sand. The grouted shafts were 1.7 and 1.4 times higher than the ungrouted shafts for clay and dense sand, respectively. In the clay, the maximum attainable grout pressure (10.8 tsf) was similar to the ultimate end bearing stress mobilized in the ungrouted control shaft (11 tsf). This type of grout pressure limit has been incorporated into design procedures and recommendations for grouted shafts tipped in clay. The improvement in ultimate end bearing should not be assumed to be entirely a manifestation of soil improvement but rather the by-product of a slight increase in the shaft tip diameter as well. If it is assumed that the ultimate end bearing resistance remained similar, then a radial expansion (grout bulb) of 7 inches could be responsible for this improvement. From a design standpoint, shafts tipped in clay should not be expected to develop grout pressures above the computed ungrouted ultimate end bearing capacity. Further, the usable end bearing of grouted shaft tipped in clay can be estimated to be only as high as the achievable grout pressure. This is still an improvement as the ultimate end bearing becomes the usable end bearing.

In the dense sand, the maximum attainable grout pressure was limited by the amount of available side shear and not the end bearing stress. Therein, the maximum grout pressure of 15.8 tsf was significantly less than the ultimate end bearing stress (25 tsf). Interestingly, the S-series shafts were merely shorter versions of the C-series shafts (essentially the same soil strata but tipped shallower). As such, more side shear resistance could be developed in the C-series than the S-series. However, more grout pressure was developed in the S-series due to the mechanism of the sand matrix to lock in the grout bulb from migrating in any direction. As side shear is the controlling parameter for grouted shafts tipped in sand, insitu loose sand beneath the tip can be expected to produce end bearing capacities commensurate with dense sand given the same side shear reaction. The difference lies in the amount of grout required to affect the soil modification to a similar density state. The dense sand beneath S-2 (N > 50) and the net volume taken to grout (3 ft^3) are in line with that observed at the PGA Blvd project (Figure 4-44).

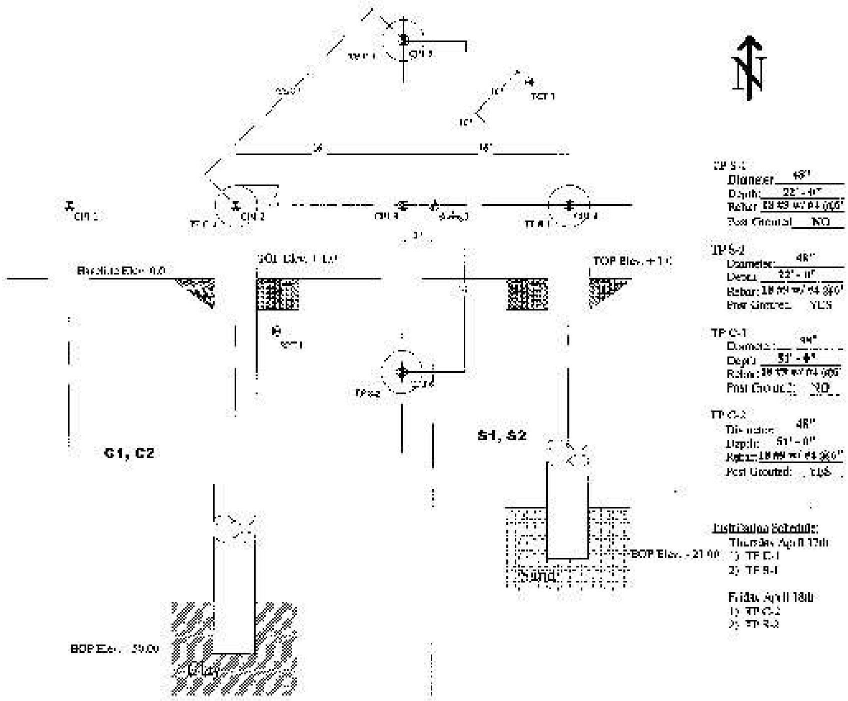


Figure 4-81 TexDOT Demonstration site layout.

Figure 4-82 TexDOT Demo: CPT testing performed by Fugro.

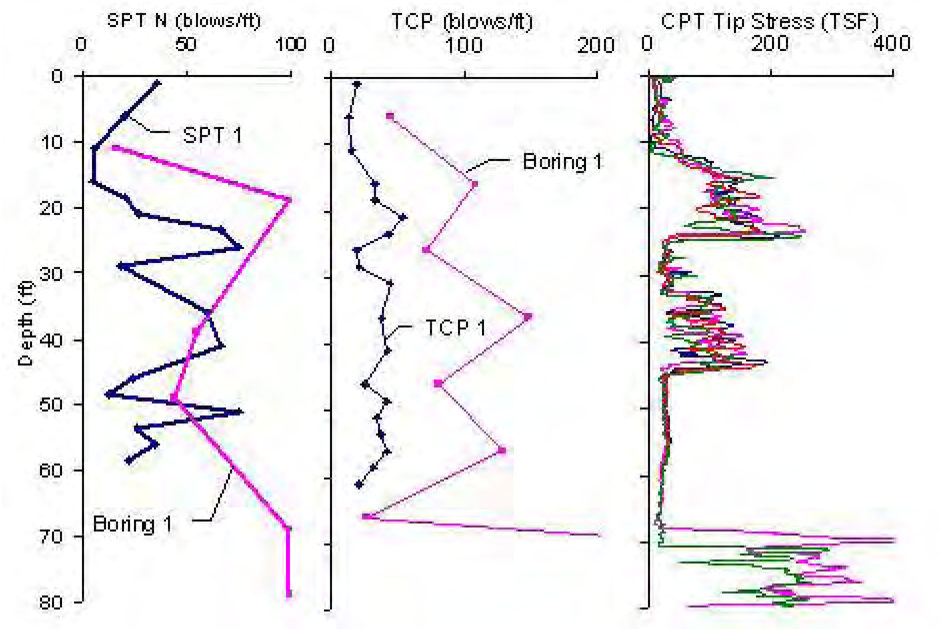


Figure 4-83 TexDOT Demo: SPT, TCP, and CPT testing results.

Figure 4-84 TexDOT Demo: Reinforcement cage construction.

Figure 4-85 TexDOT Demo: Strain gage installation.

Figure 4-86 TexDOT Demo: Installation of rubber membrane on grout plates.

Figure 4-87 TexDOT Demo: Grout plate installation.

Figure 4-88 TexDOT Demo: Grout plates and strain gages installed.

Figure 4-89 TexDOT Demo: Drilling of 48" diameter test shafts.

Figure 4-90 TexDOT Demo: Reinforcement cage placement.

Figure 4-91 TexDOT Demo: Concrete pour.

Figure 4-92 TexDOT Demo: Test site.

Figure 4-93 TexDOT Demo: SIT testing.

Figure 4-94 TexDOT Demo: Post grout setup and flushing of lines.

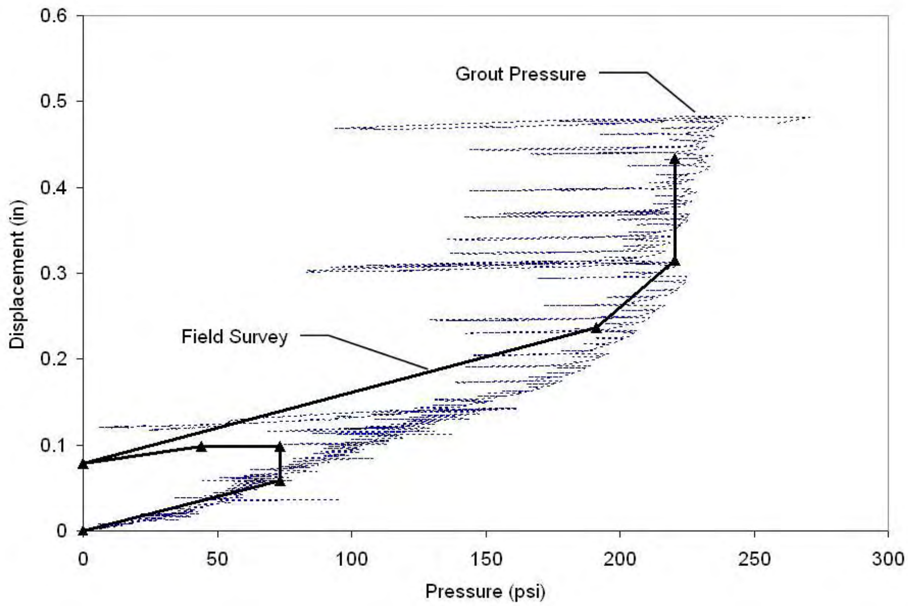


Figure 4-95 TexDOT Demo: Comparison between load test and production survey data (shaft S-2).

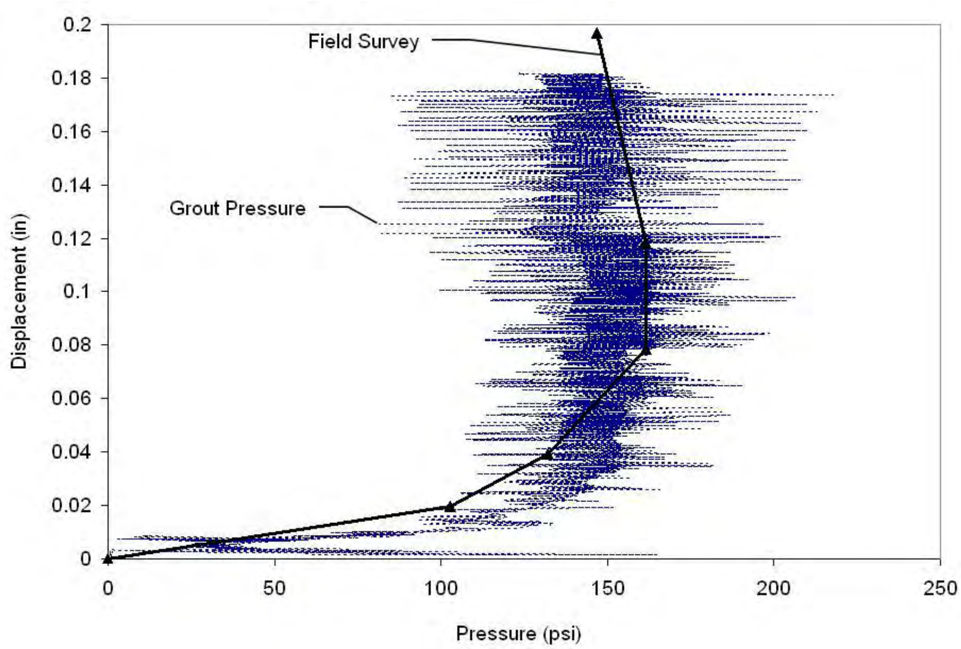


Figure 4-96 TexDOT Demo: Comparison between load test and production survey data (shaft C-2).

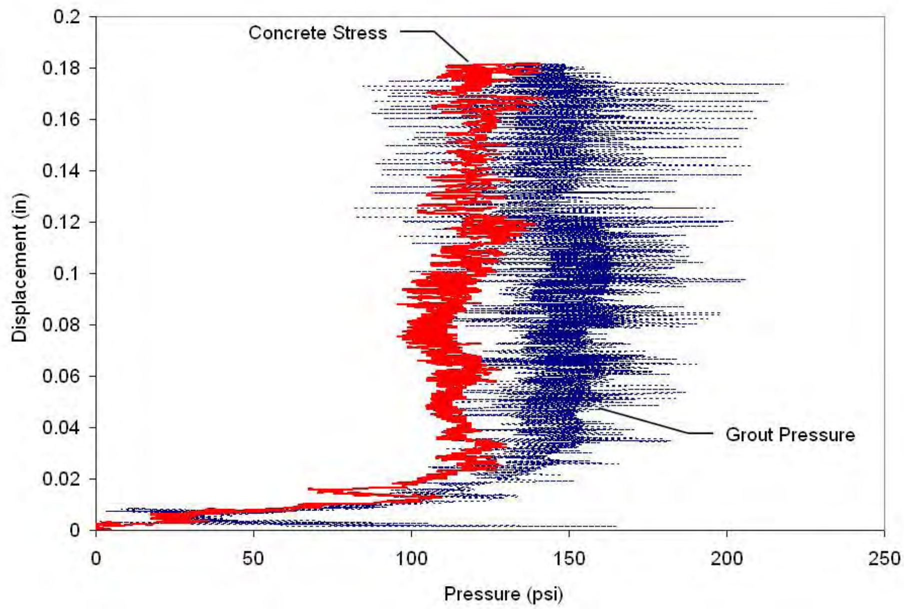


Figure 4-97 TexDOT Demo: Comparison of applied grout pressure and tip concrete stress (shaft C-2).

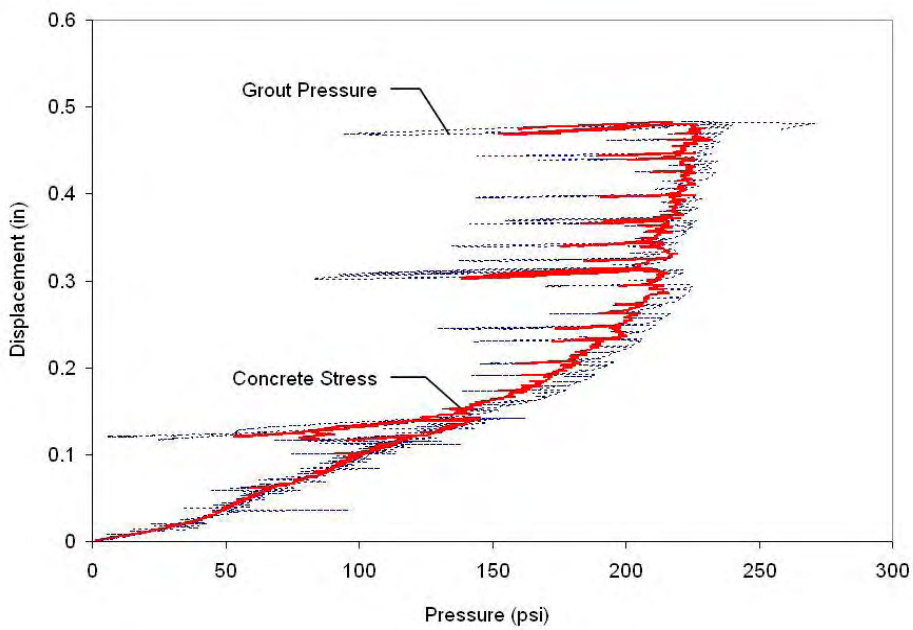


Figure 4-98 TexDOT Demo: Comparison of applied grout pressure and tip concrete stress (shaft S-2).

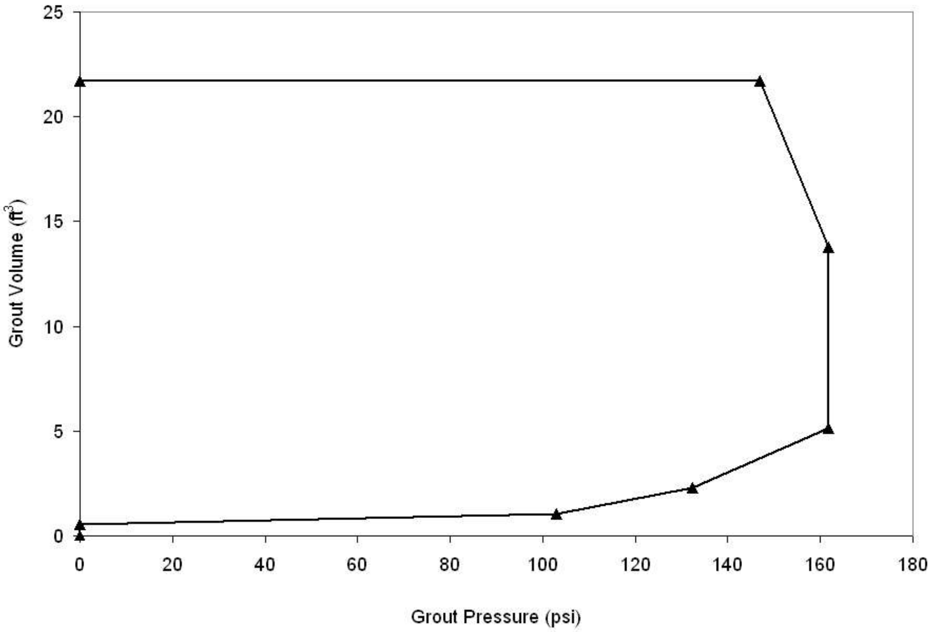


Figure 4-99 TexDOT Demo: C-2 grout volume versus grout pressure.

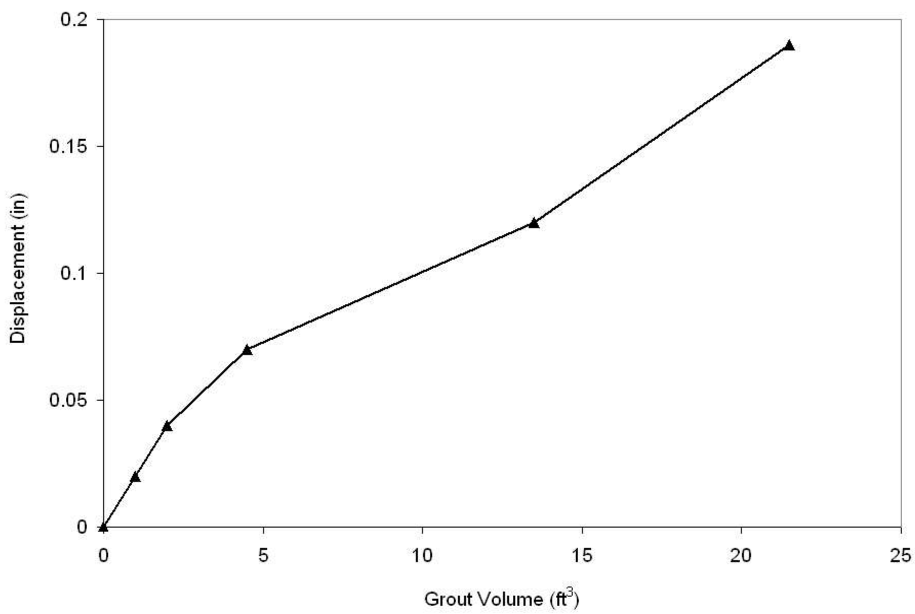


Figure 4-100 TexDOT Demo: C-2 displacement versus grout volume.

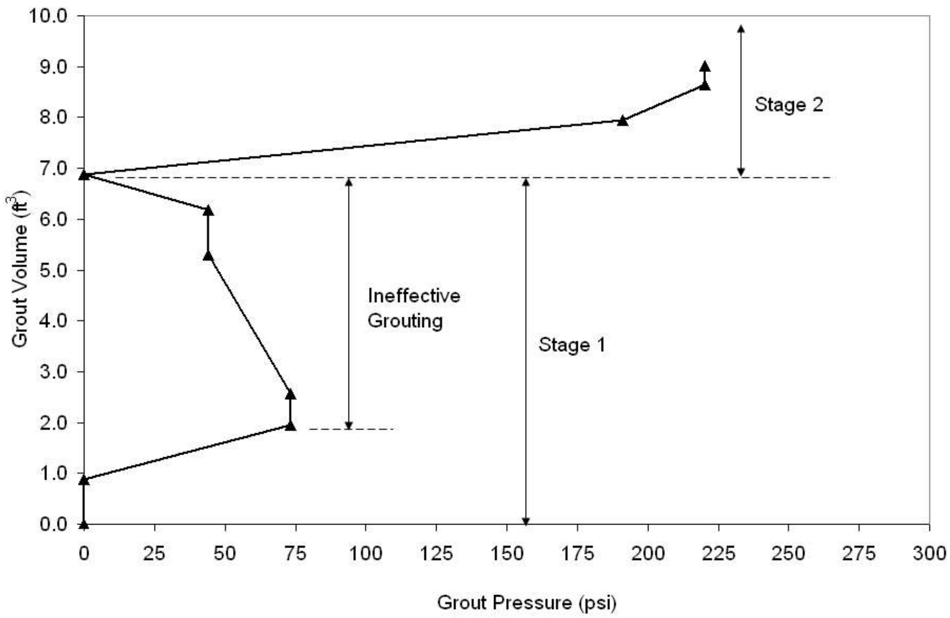


Figure 4-101 TexDOT Demo: S-2 grout volume versus grout pressure.

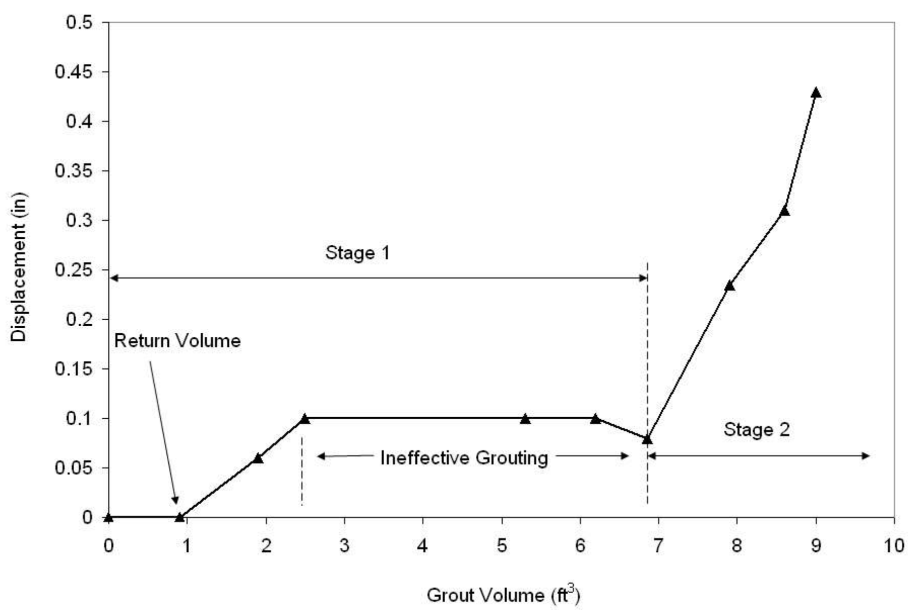


Figure 4-102 TexDOT Demo: S-2 displacement versus grout volume.

Figure 4-103 TexDOT Demo: Statnamic setup


Figure 4-104 TexDOT Demo: Statnamic load test

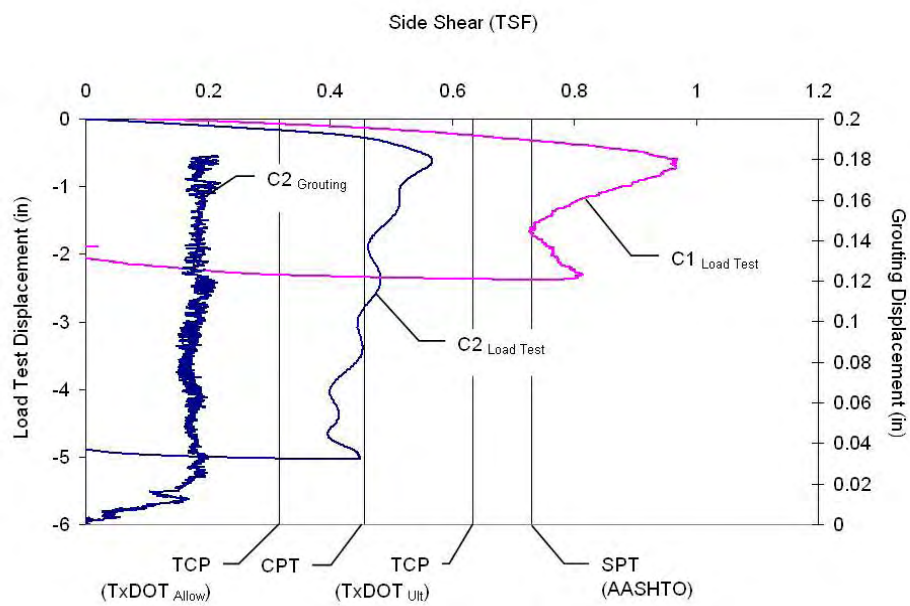


Figure 4-105 TexDot Demo: Side Shear Plots for Shafts C-1 and C-2.

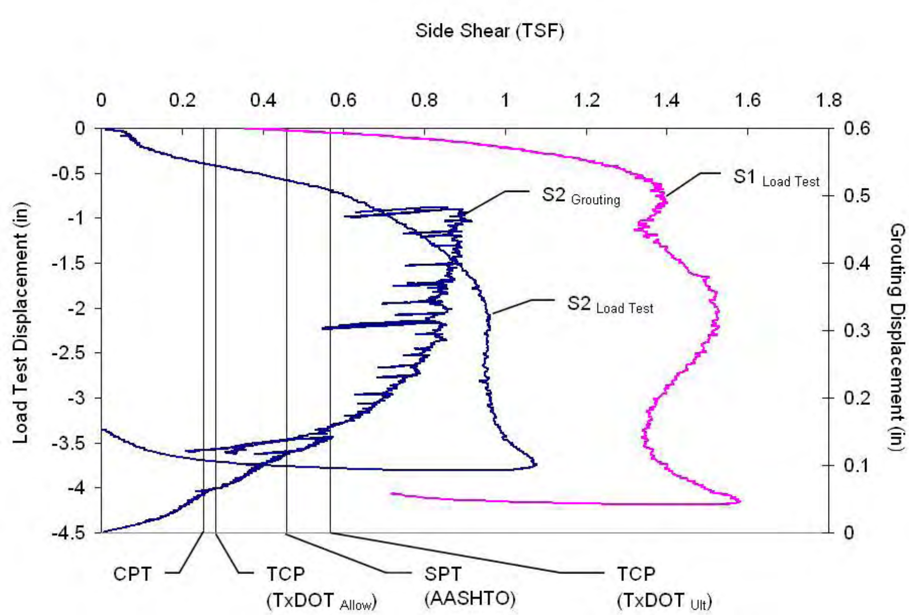


Figure 4-106 TexDOT Demo: Side Shear Plots for Shafts S-1 and S-2.

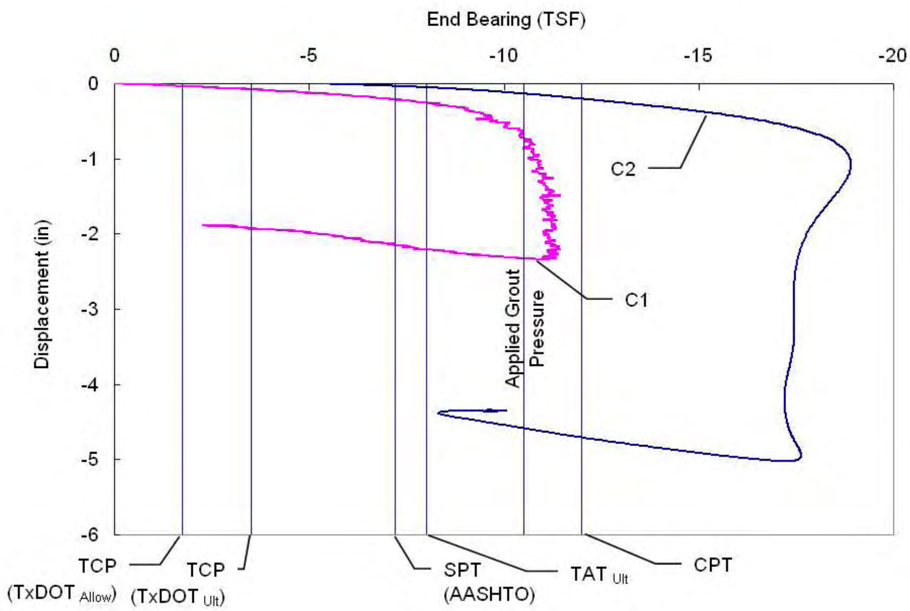


Figure 4-107 TexDOT Demo: End Bearing Stress Plots for Shafts C-1 and C-2.

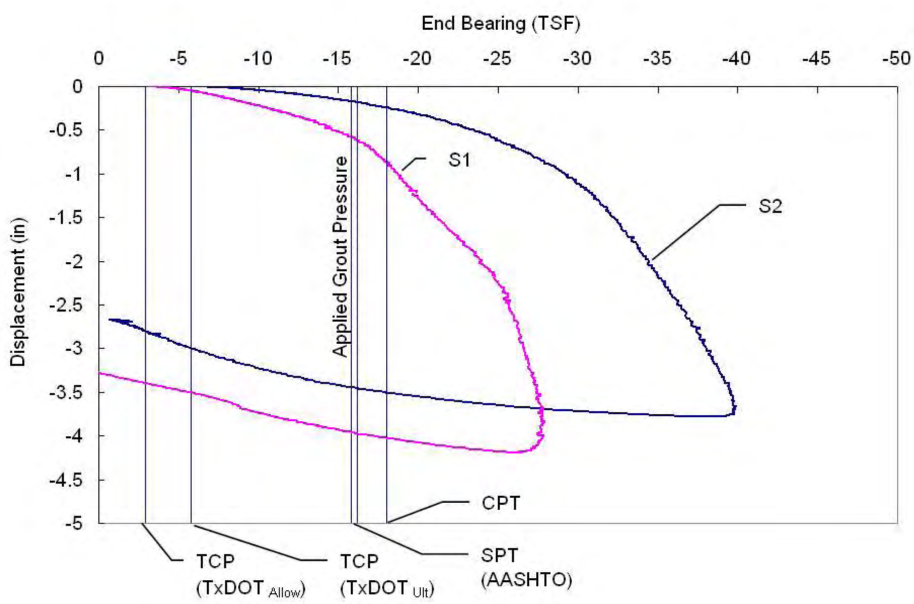


Figure 4-108 TexDOT Demo: End Bearing Stress Plots for Shafts S-1 and S-2.

4.4 Natchez Trace Parkway

The Natchez Trace Parkway was originally constructed in the 1930's as a scenic route from Natchez, MS to Nashville, TN. The two lane roadway was over 450 miles built and maintained by the National Park Service. The latest addition to this project consisted of 4.3 miles of new alignment incorporating 6 bridges under a design-built contract of $30,000,000. The six bridges include crossings over St. Catherine Creek, Melvin Bayou, and Perkins Creek. The bridge crossing St. Catherine Creek and Melvin Bayou is the longest of the six bridges at 1,700 feet. Post grouting drilled shafts was proposed and accepted for the bridge crossing St. Catherine Creek and Melvin Bayou.

The post grouting project for the St. Catherine Creek and Melvin Bayou bridge called for a load test of a non-production shaft and the monitoring of a instrumented production shaft. The entire shaft construction program involved 26, 6 ft diameter grouted shafts ranging in length from 70 to 98 feet. The success of this project revolved around the end bearing performance of a 6 ft diameter grouted drilled shaft in silty soil as determined by grout testing and downward load testing. The test shaft was embedded 75 ft deep. This non-production test shaft was constructed with an isolation casing to represent scour conditions. An instrumented production test shaft was also monitored to determine the relative performance of the two test shafts with and without isolation casing (non-production and production, respectively). Both test shafts were constructed with grouting apparatuses. The grouted end bearing performance was based on the predicted end bearing capacity of the test shafts. The instrumentation and testing sequence is summarized as follows:

- Construction of non-production test shaft on June 3, 2003.
- Post grout non-production test shaft on June 10, 2003.
- Axial Statnamic load testing of non-production test shaft on June 13, 2003.
- Production grouting began on July 10, 2003.
- Post grout instrumented production shaft on July 10, 2003.
- Production grouting was completed on August 20, 2003

4.4.1 Soil Exploration and Site Layout

The site conditions are mostly wooded with near vertical loess bluffs caused by erosion as shown in Figure 4-109. The loess are tall vertical cuts from wind blown silt and clay-size particles. Immediately below the loess are Catahoula formations which are over-consolidated clays and sands. ATV drill rigs were used for soil sampling due to the terrain. Figure 4-110 shows a Standard Penetration Test (SPT) being performed.

A total of 49 SPT soil borings were performed on 500 feet spacings along the alignment. The SPT soil borings for the two test shafts are located in Appendix B. The as-built location of Test Shaft 1 corresponded to Boring WSA-6 and Boring B-1 coincided with Test Shaft 2. The soil profile generally consisted of loose silt followed by a medium dense to very dense sand / silty sand underlaid by very dense silt. The average SPT N value located at the tip of the shafts were greater than 60. The watertable was located at a depth of 12 feet at the time of construction.

4.4.2 Construction and Instrumentation

The non-production test shaft was constructed with a 40 foot, 74 inch diameter permanent isolation casing and a total length of 76 feet with a diameter of 72 inches below the isolation casing. Figure 4-111 shows a crane mounted drill rig which was used for drilling the test shaft. An 84 inch auger was used to drill the upper 40 feet for the isolation casing. Figure 4-112 shows the 71 inch hollow stem auger used for drilling below the isolation casing. A bentonite slurry was used during drilling. Figure 4-113 shows the bentonite slurry tank with a de-sanding unit. The test shaft 60 inch diameter reinforcement cage was constructed with four levels of three strain gages and two telltale tubes for monitoring during grouting and load testing. The test shaft was also constructed with six CSL tubes for quality assurance testing. The strain gages were placed at depths of 35', 50', 60', and 72.5'. Figure 4-114 shows the installation of the strain gages and telltale tubes. The reinforcement cage was also constructed with a 60 inch diameter flat-jack grout plate as shown in Figure 4-115. The grout plate was attached to the reinforcement cage while the cage was vertical. This was done to insure that the grout plate and connections were not damaged during the lifting of the 76 foot long cage. Figures 4-116 shows the installation of the grout plate to the reinforcement cage. Figures 4-117 and 4-118 show the cage placement and the concrete pour for the test shaft, respectively.

The instrumented production shaft was constructed similar to the non-production but without the isolation casing. Also, the production test shaft was constructed with only two levels of three strain gages and two telltales. The two levels of strain gages were placed at depths of 40' and 72.5'. The data obtained from the strain gages was used to delineate the load contribution from end bearing, total side shear, and side shear contributed within the scour zone.

4.4.3 Post Grouting

The post grouting of both test shafts, non-production and production, were performed on separate days using a single stage hydraulic piston pump. The grouting procedure consisted of flushing the grout lines to ensure clear passage, pumping grout into until return was observed in the other lines and then locking the valve, and pumping grout to the toe until the recommended pressure, volume, or displacement was reached. During the grouting of the test shafts, the strain gages, pressure transducer, and displacement transducers for the top and toe of the shaft were monitored using a computer data acquisition system. Figures 4-119 through 4-121 show the setup of the reference beam and displacement transducers. Manual readings of the applied grout pressure and the surveyed top of shaft displacement were taken to verify the data obtained from the data acquisition system. A total of 3.5 cubic feet of grout was pumped to the toe at a upward displacement of 0.167 inches and a grout pressure of 310 psi for the non-production test shaft. The instrumented production shaft was grouted to 410 psi with a total volume of 10.6 cubic feet and displaced 0.177 inches upwards.

4.4.4 Post Grouting Results

Grout Pressure vs. Disp. The grout pressure and the field survey notes versus displacement are shown in Figure 4-122 for the non-production test shaft with isolation casing. The maximum grout pressure achieved was 300 psi at a maximum uplift of 0.155 inches. Figure 4-123 shows the grout pressure versus displacement from field survey notes of the instrumented production test shaft without isolation casing (Bent 5 - Shaft 2). The pressure transducer showed a maximum grout pressure of 424 psi while the pressure gage recorded in the field survey notes showed 464 psi. Note: Figure 4-123 shows displacement versus load, therefore, the load should be divided by the cross sectional area (4071.5 in^2) to provide a direct correlation to grout pressure (not shown). The field survey notes also showed a difference in displacement from the computer acquired displacement. The field survey notes showed a maximum displacement of 0.177 inches while the displacement transducers showed a displacement of 0.158 inches.

The two test shafts (non-production and production) were constructed similar with and without isolation casing and grout tested to show the effects for the scour zone on grouting. The non-production test shaft showed a fully developed side shear during grouting. Where as, the instrumented production shaft, at higher grout pressures, still showed an increasing side shear development. The difference in grout pressures was 164 psi between the two test shafts. The additional side shear contribution from the scour zone during grouting realizes the potential for higher grout pressures and a result of higher end bearing. The maximum applied grout pressure for a post grouted shaft is dependent on either the available side shear or the end bearing capacity (depending on soil type). The same effect can be accomplished by applying dead load to a post grouted shaft during grouting. The result from higher applied grout pressures produces a higher grout pressure index which produces a higher tip capacity multiplier.

Alternatively, one could set an uplift limit that would reflect the same amount of shaft capability as if the scour depth (isolation casing) were in effect. For instance, a 300 psi grout pressure (1200 kip uplift force) would cause approximately 0.1 inches of uplift movement (Figure 4-123). In either case, the uplift response to grout pressure should be notably different between the two scenarios.

Grout Volume. The field grouting survey notes also recorded the grout volume pumped to the toe of each test shaft. Figure 4-124 shows the grout volume versus grout pressure curve for the non-production test shaft. The grout pressure versus grout volume curve is a typical curve produced during grouting of all post grouted drilled shafts. The curve shows an increasing volume with no increase in grout pressure. This volume is the required volume for grout return in the tubes. A return volume of 1.3 ft^3 was required as shown as the intercept value in Figure 4-124. Once the grout tubes were locked off, a net volume of 2.3 ft^3 was pumped into the test shaft to reach the maximum grout pressure of 300 psi.

Figure 4-125 shows the field survey grout pressure versus grout volume for the production test shaft without isolation casing. The figure shows a required grout volume of 2.5 ft^3 for grout return. At a net grout volume of 8.2 ft^3, the curve shows an increase in grout pressure without a significant increase in volume. Figure 4-126 shows a similar trend in the displacement-volume curve. This indicates that some material beneath the tip was highly compressible, but once compressed, very small volumes of grout caused large increases in grout pressure due to low compliance. Although impossible to produce displacement without input volume, the field logs are rather crude with regards to volume measurements. Nevertheless, the overall trend is observable. In this case, a 0.1 inch uplift would require a theoretical volume of 0.23 ft^3 (with no cavity expansion). As such, the near vertical response in Figure 4-126 is reasonable when considering a change in volume of 1.1 ft^3 (7.1 to 8.2) over an uplift of 0.14 inches (0.04 to 0.18).

Side Shear. Figure 4-127 shows the side shear for the non-production test shaft (with isolation casing) from both grout testing and load testing. The developed upward side shear was 1390 kips at 0.155 inches. Figure 4-128 shows the developed side shear versus upward displacement for the instrumented production shaft (without isolation casing) as determined from strain gage data and displacement transducers. The maximum mobilized side shear of the shaft was 1778 kips at an uplift of 0.16 inches. Of which, 386 kips was contributed from the scour zone. The difference in side shears at a displacement of 0.16 inches is the additional side shear developed in the scour zone.

4.4.5 Axial Compressive Load Test

The axial compressive load test was performed using a 16MN statnamic mechanical catching device on the non-production test shaft. Figures 4-129 through 4-131 show the setup of the statnamic frame on the non-production test shaft. A load cell, three capacitive accelerometers mounted 120 degrees apart, and the strain gages were monitored during the statnamic load test. A survey level was used to verify the final displacement of the test shafts. Figure 4-132 shows the statnamic load testing of the non-production test shaft.

4.4.6 Side Shear Results

The developed side shear during axial compressive load testing (downward) and grout testing (upward) for the non-production test shaft are shown in Figure 4-127. The maximum developed side shear during load testing was 1200 kips at a displacement of 0.326 inches. The maximum side shear during grouting was 1390 kips at a displacement of 0.155 inches. At the same displacement during downward load testing, the developed side was 1000 kips.

4.4.7 End Bearing Results

The Natchez load test program did not include an ungrouted control shaft so no direct comparison to such a shaft was performed. However, the end bearing performance of the shaft as shown in Figure 4-133 indicates no ultimate capacity was achieved. At a displacement of 0.37 inches (0.5%D) 2000 kips of end bearing was mobilized (36 tsf). This was 1.6 times higher than the applied grout pressure of 21.6 tsf.

4.4.8 Production Phase

The shaft construction and grouting for the Natchez Trace Parkway project was conducted in two months. A total of 26 shafts were constructed. Figures 4-134 through 4-138 show the construction of production shafts. The grouting procedure for all shafts was conducted in keeping with the protocol set forth by the grout test program. That program provided a minimum grout volume (2 ft^3 or 57 liters), a maximum permissible uplift displacement (0.25 in or 6.35 mm), and a minimum required grout pressure (310 psi or 21 bars). The minimum grout volume criterion is and was intended to assure flow into the toe area prior to achieving the design grout pressure. Shaft grouting spanned from July 10, 2003 to August 20, 2003. In most cases, multiple shafts were grouted in a single day to optimize the grouting contractor's labor and mobilization costs. On average, three shafts per day were grouted (ranging from 2 to 4 shafts per day).

Uplift movement was required to be monitored by survey level and recorded in conjunction with periodic pressure and volume measurements. During grouting, 3 of 26 shafts did not respond as expected. The shafts which did not respond as expected were both shafts in Bent 11 and Shaft 1 in Bent 12. These shafts showed larger displacements at lower pressures.

4.4.9 Production Results

A total of 26 production drilled shafts with post grouted tip enhancements were constructed. A post grout field record log was developed for each production shaft which includes general construction data for the shaft, surveyed top of shaft displacement, grout volume, and grout pressure. An overview of the performance of all 26 shafts can be obtained by reviewing the grout uplift records. Figure 4-139 shows the uplift for each shaft grouted. Although the pilot grout test only displaced 0.16 inches at the point of ultimate shear, the production shafts were permitted to uplift as much as 0.6 inches (presumably based on the response of the uplift-grout pressure curve for each shaft). In such cases, a linear response would indicate reasonable performance.

The return grout volume required to flush the four 1" diameter SCH 80 grout tubes varied between 1 and 8 cubic feet (approx.). The theoretical volume to fill only the grout tubes varied from 1 to 1.5 cubic feet over the range of lengths/depths to which the shafts were constructed. Figure 4-140 shows the return volume as a function of shaft length/depth. With the exception of 3 shafts which were noted to have encountered tube blockages during grouting, each of the shafts took more than the theoretical tube volume. The difference between the theoretical and measured grout tube volume accounts for the volume/pathway the grout took to achieve communication between grout tubes beneath the shaft tip. The additional volume required to achieve a theoretical 2.5%D displacement is also noted (4.2 ft^3). This volume is the minimum target volume that the grouting should achieve to begin to pre-compress the soil for effective post grouting.

The uplift displacement as measured during grouting is often a good indicator of proper shaft performance provided the design grout pressure is optimized to make full use of the available side shear (as discussed earlier). The actual vs. predicted shaft concrete volume also serves as a good indicator of proper construction techniques, especially in soils in which it is difficult to maintain excavation stability. In this case, the site varied between hard silts and sands, so such a review may be difficult. Figure 4-141 shows the uplift for each shaft versus the respective concrete volume expressed as the percentage of theoretical. No clear trend is immediately discernible. In fact, the highest uplift was recorded for a 100% theoretical shaft. However, borings were not available at each pier location and in some instances did not reflect the stratigraphy encountered at the time of excavation. Therein, the estimated side shear values would not have reflected the actual values.

Figure 4-109 Natchez: Wind blown loess bluffs.

Figure 4-110 Natchez: SPT testing.

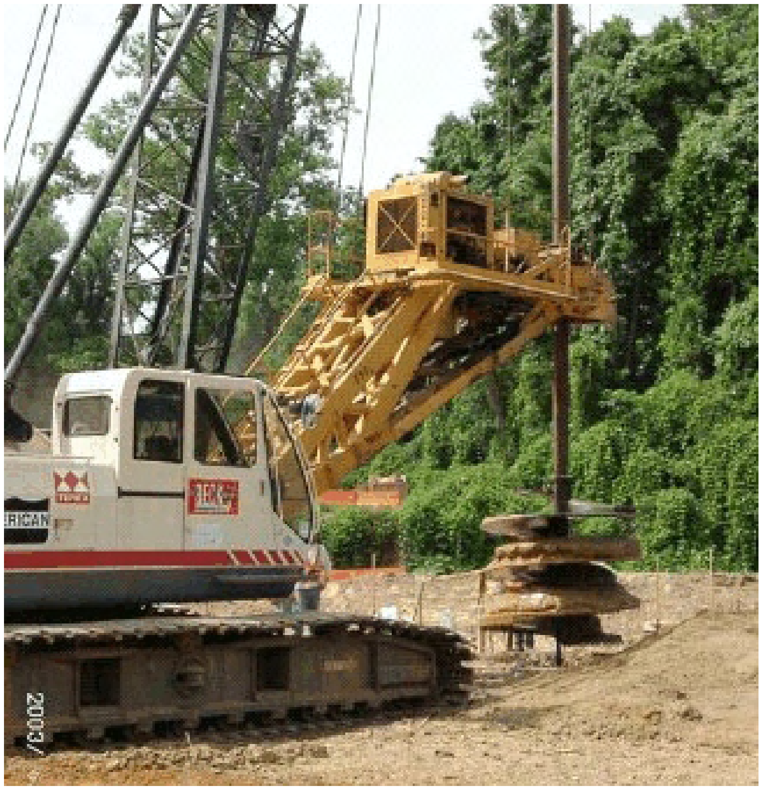


Figure 4-111 Natchez: Crane mounted drill rig.

Figure 4-112 Natchez: Drilling of test shaft with isolation casing.

Figure 4-113 Natchez: Slurry tank and de-sanding unit.

Figure 4-114 Natchez: Strain gage installation.

Figure 4-115 Natchez: 60 inch diameter grout plate.

Figure 4-116 Natchez: Grout plate installation.

Figure 4-117 Natchez: Reinforcement cage installation.

Figure 4-118 Natchez: Concrete placement with a pump truck.

Figure 4-119 Natchez: Instrumentation setup.

Figure 4-120 Natchez: Field survey during post grouting (quality assurance testing of post grouting).

Figure 4-121 Natchez: Post grout setup and testing.

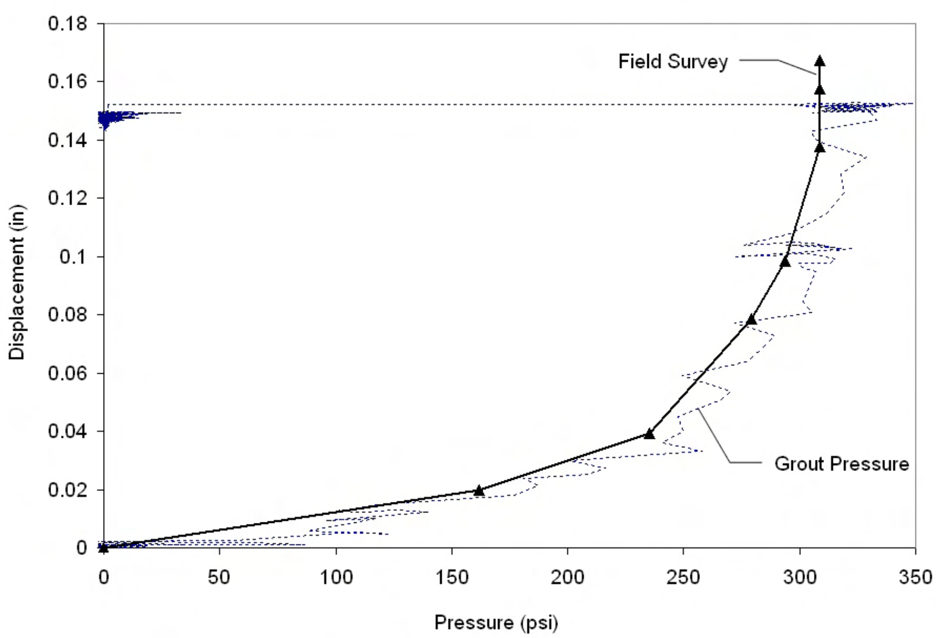


Figure 4-122 Natchez: Grout pressure versus displacement non-production test shaft.

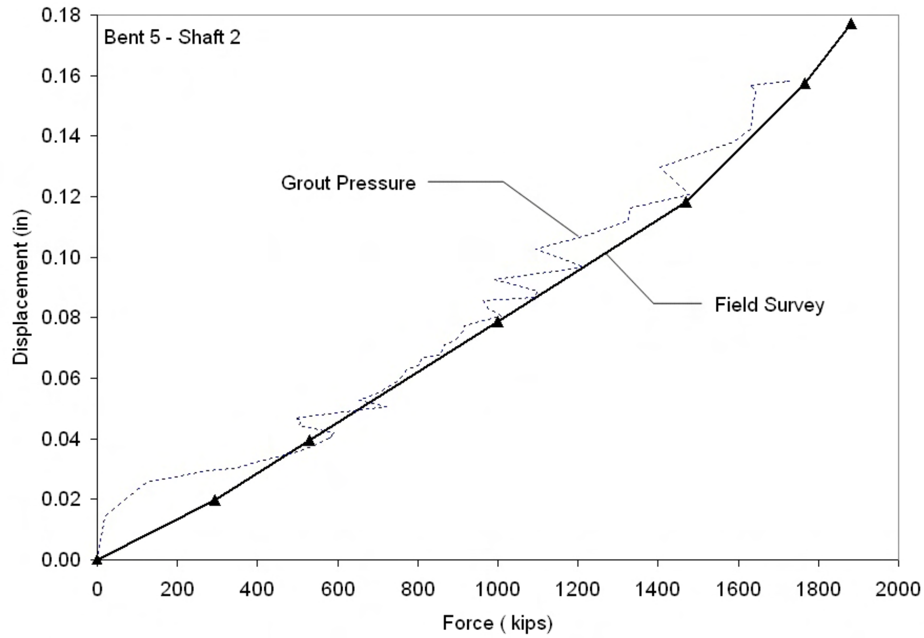


Figure 4-123 Natchez: Grout pressure versus displacement (instrumented production test shaft w/o isolation casing).

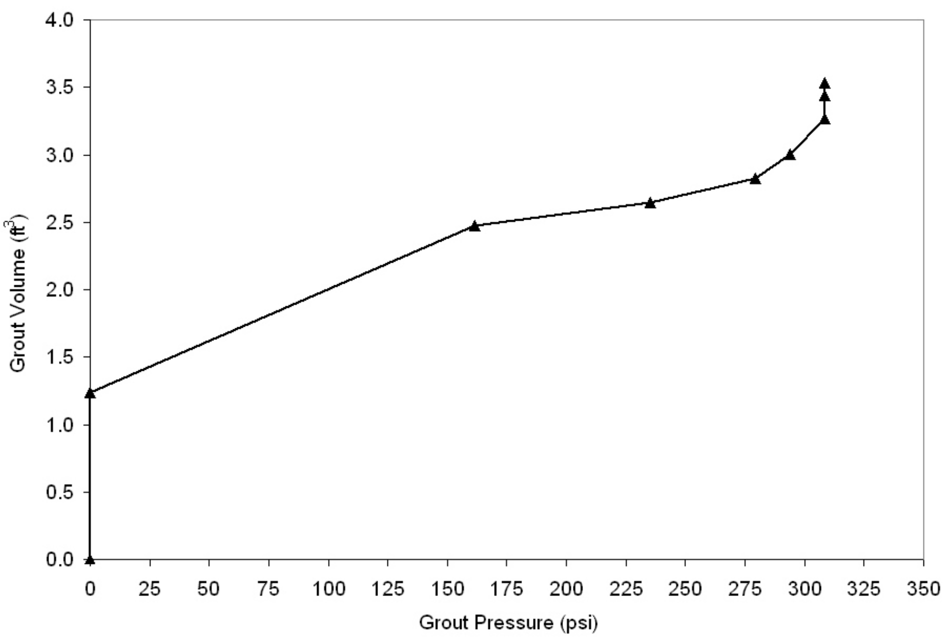


Figure 4-124 Natchez: Grout pressure versus grout volume (non-production test shaft).

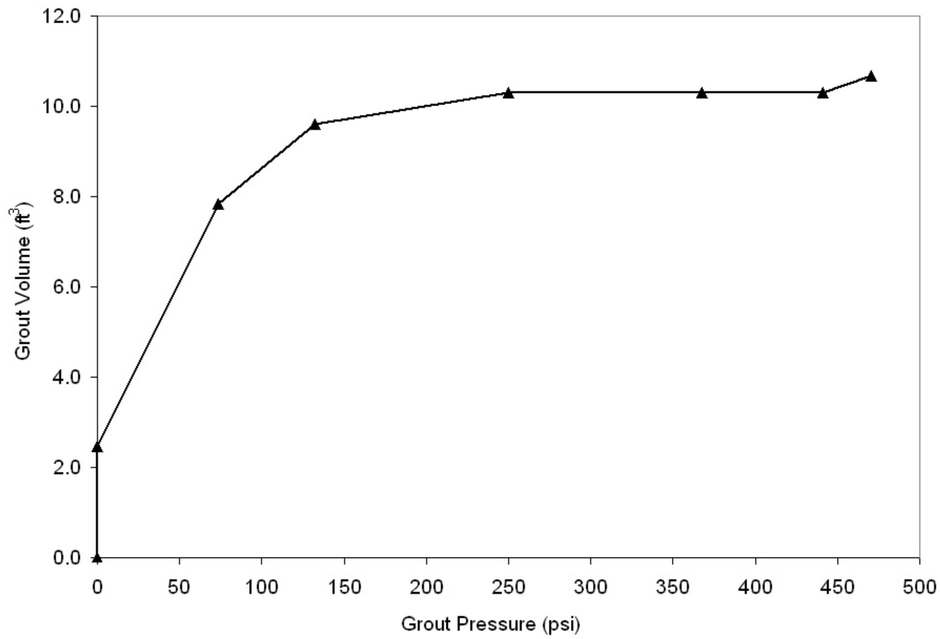


Figure 4-125 Natchez: Grout pressure versus grout volume (production test shaft).

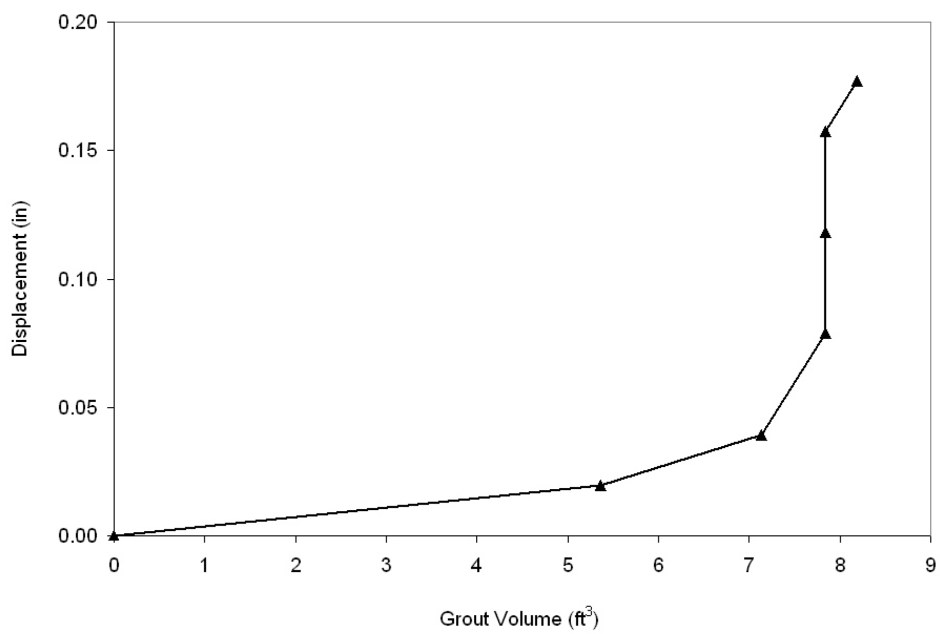


Figure 4-126 Natchez: Grout volume versus displacement (production test shaft).

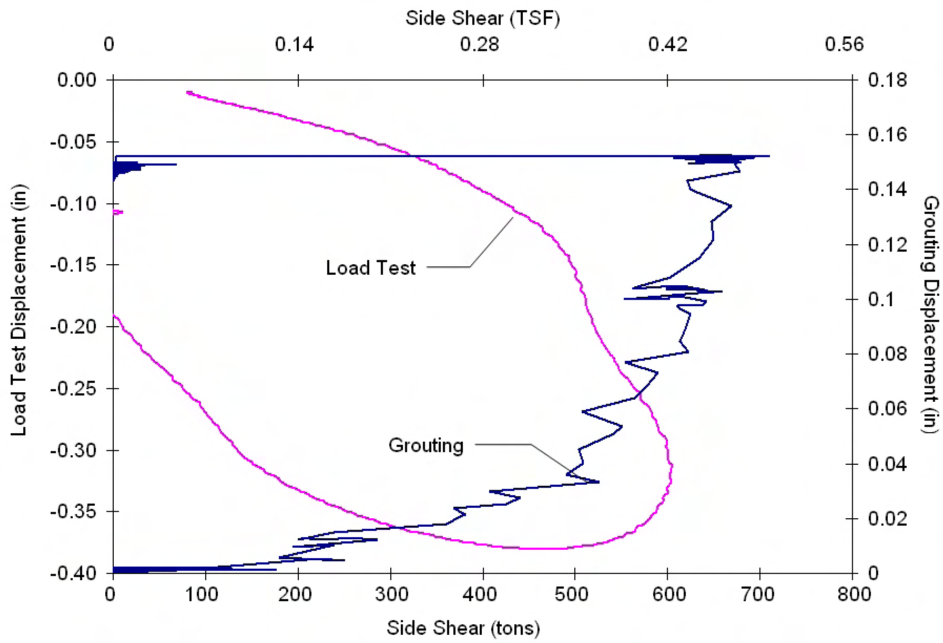


Figure 4-127 Natchez: Side shear for non-production test shaft with isolation casing.

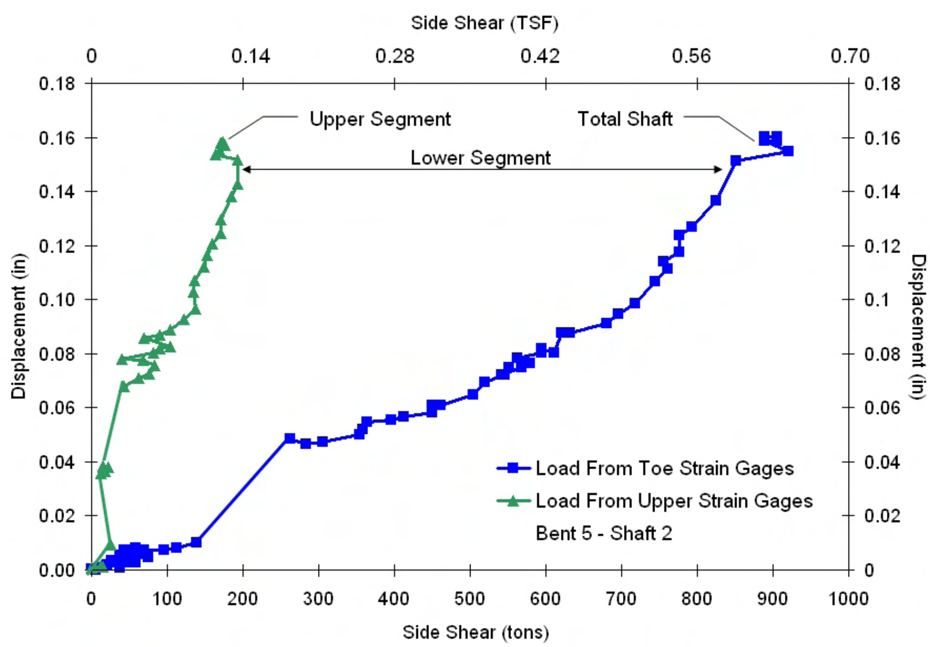


Figure 4-128 Natchez: Side shear for the instrument production shaft.

Figure 4-129 Natchez: Statnamic load test setup.

Figure 4-130 Natchez: Statnamic setup (continued).

Figure 4-131 Natchez: Statnamic setup (continued).

Figure 4-132 Natchez: Statnamic load testing.

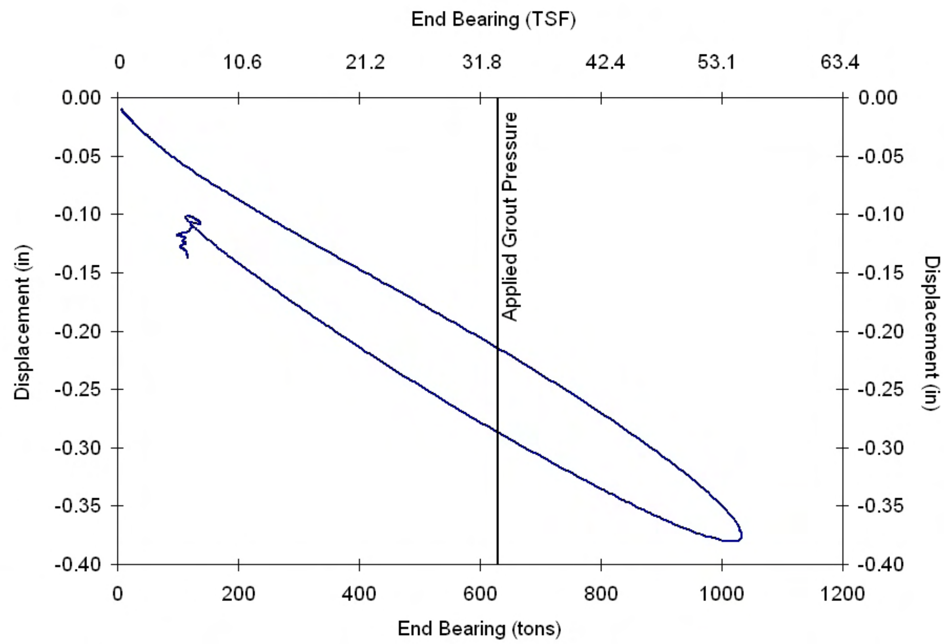


Figure 4-133 Natchez: End bearing (test shaft with isolation casing).

Figure 4-134 Natchez: Production site preparation.

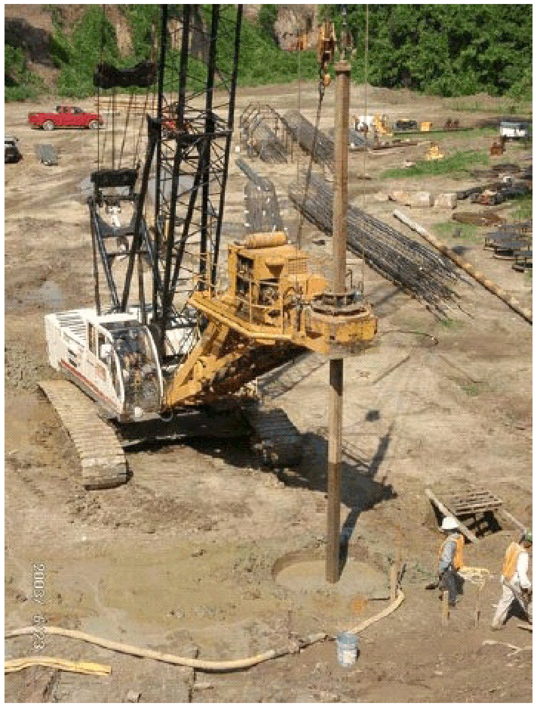


Figure 4-135 Natchez: Production drilling of Bent 5.

Figure 4-136 Natchez: Bent 8 cage placement.


Figure 4-137 Natchez: Production bridge bents.


Figure 4-138 Natchez: Bridge alignment.

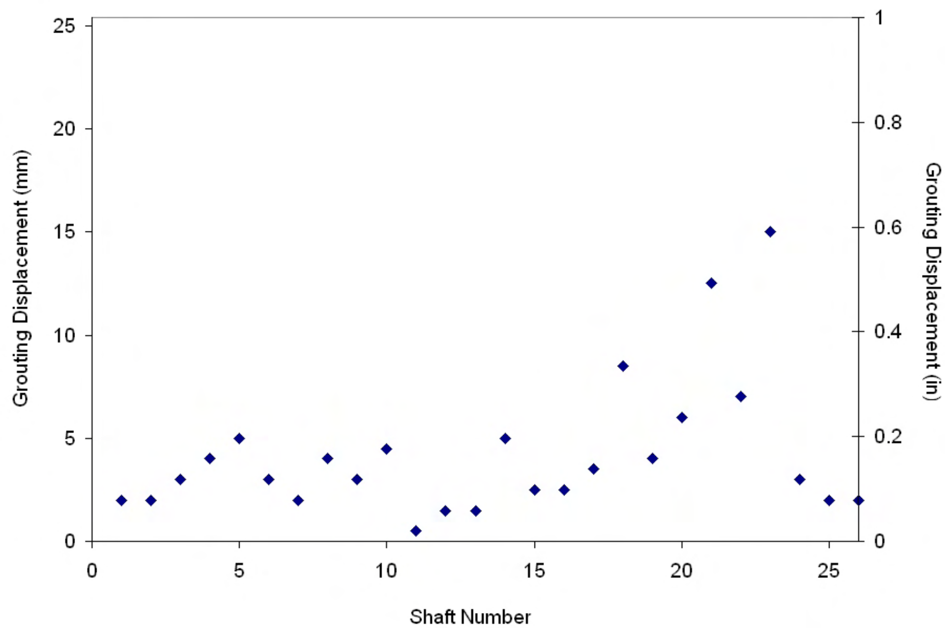


Figure 4-139 Natchez: Production QA data showing uplift for every shaft.

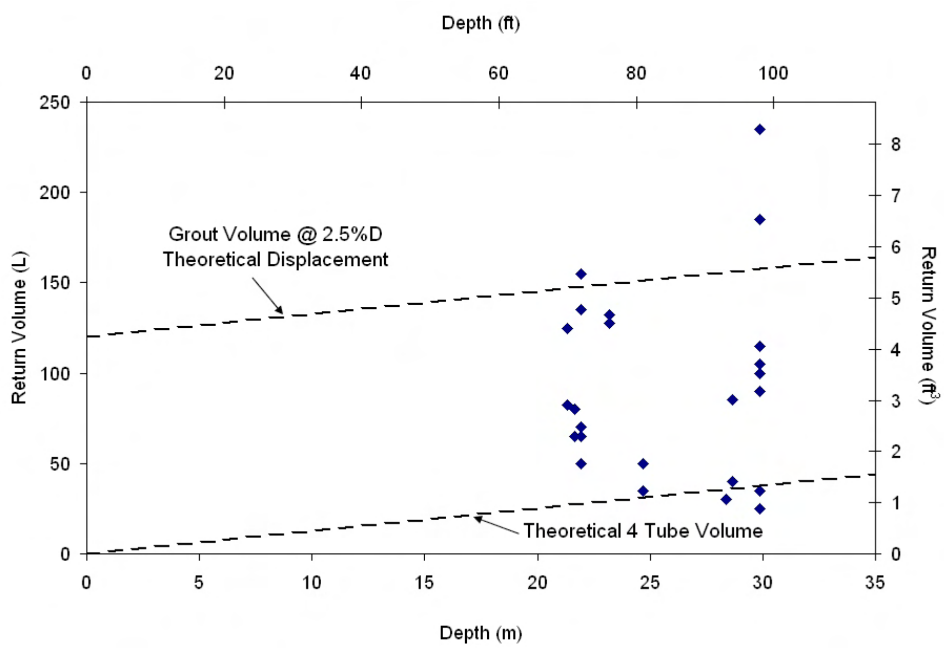


Figure 4-140 Natchez: Volume of grout required to obtain return (no pressure).

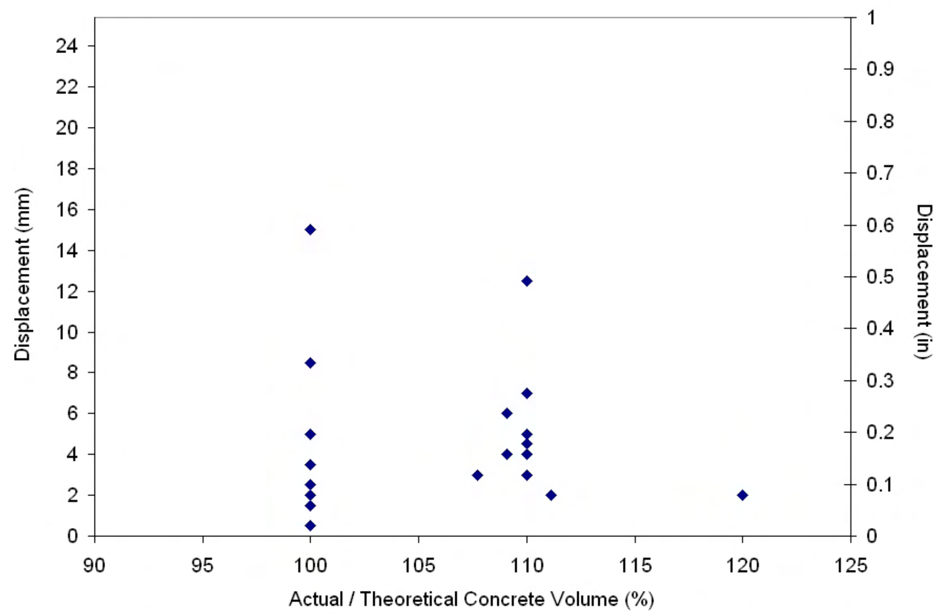


Figure 4-141 Natchez: Concrete takes versus displacement.

4.5 Farm to Market Road 507 (FM 507)

The Farm to Market Road 507 (FM 507) bridge project consisted of the replacement of the North Floodway Pilot Channel Bridge which had an existing two lane, 22 ft wide roadway. The replacement would increase the volume of traffic from a two lane to a four lane, 44 ft wide roadway. During the construction of the bridge, the increase in loading began failing the bridge pier. The magnitude of the capacity shortfall caused 10 inch settlement and showed no signs of slowing. The design engineers decided to retrofit the failing bridge bent with two post grouted drilled shafts to restore the capacity of the structure. As such, the bridge was fully dismantled so that two 30 inch diameter, 46 ft deep post grouted shafts could be installed and the vertical alignment could be restored. The two post grouted drilled shafts were tipped in sandy clay soil. The instrumentation and testing sequence is summarized as follows:

- Instrumentation of two production test shafts on July 1, 2003.
- Completion of two production test shafts on July 1, 2003.
- Post Grouting of two production test shafts on July 8, 2003.

4.5.1 Soil Exploration and Site Layout

The subsurface investigation of the test site was performed using the Texas Cone Penetration Test (TCP). A total of four TCP tests were performed to determine the soil profile. Appendix B, Figures B-55 and B-56 show the soil boring logs for this site. The soil boring log closest to the retrofit bridge Bent #2 was Test Hole No. 1, which was approximately 65 ft south of Bent #2. The soil consisted of stiff sandy clay for 20 ft followed by very dense clayey sand through the rest of the soil boring log. The average TCP value for the upper layer is 171 blows/ft and the average TCP value for the lower layer is 647 blows/ft.

4.5.2 Construction and Instrumentation

The construction of the two retrofit drilled shafts began by removing debris from each shaft location with an excavator to a depth of 13 feet. A 34 inch permanent steel casing was driven to an elevation of -16 feet for each drilled shaft to stabilize the upper soil during drilling. The permanent steel casing extended 2 feet above the existing waterline. A temporary 36 inch steel casing was placed around the permanent casing to allow for a 5 foot head of slurry during drilling. A bentonite slurry was used during drilling. The drilling was performed using a 30 inch diameter double cut single flight drill auger. The soil was removed to an elevation of -31 feet. Figure 4-142 shows the setup and drilling of the west shaft. A 4 inch hydraulic submersible pump attached to the end of a self-contained reel concrete pump rig was used to de-sand the bentonite slurry to a sand content of 2.5%. Figures 4-143 through 4-145 show the submersible pump, the de-sanding of the west shaft, and the de-sanding of the east shaft, respectively.

The reinforcement cages for both production test shafts were constructed with 8 - #9 bars with #3 spiral on 6 inch centers for shear reinforcement. The reinforcement cages were instrumented with four strain gages, placed 90 degrees around the cage, two feet above the bottom of the cage. The strain gages were used to determine the load applied to the toe of the shafts during grouting. To allow for the monitoring of the toe displacement, two full length PVC tubes were attached to the cage to allow for telltales during grouting. Figure 4-146 shows the installation of the strain gages and telltale tubes. The grout plates used on both shafts were 24 inch diameter flat jack type attached to the bottom of the reinforcement cage. Figures 4-147 through 4-149 show the grout plate, the grout plate attached to the reinforcement cage, and the placement of the reinforcement cage into the east shaft. Figures 4-150 and 4-151 show the concrete placement and finishing of the west shaft.

4.5.3 Post Grouting

The post grouting of the two drilled shafts were performed in one day using a single stage hydraulic actuated piston type pump. Figure 4-152 shows the grout pump. The grouting procedure consisted of flushing the grout lines with water until clear water was returned in the other lines. This insured a clear passage for the grout to the bottom of the shafts. Portland Type I / II cement with a w/c ratio of 0.4 to 0.5 was pumped into the grout lines with valves open until grout return was achieved in the other line. The valves were then closed and pumping continued until the recommended grout pressure, grout volume, or allowable displacement was achieved. Figure 4-153 shows the grouting of the west shaft.

During the grouting, the strain gages, grout pressure, and displacement of top and toe of shaft were measured using a data acquisition system. Figure 4-154 shows the setting up of the data acquisition system. Survey data along with grout pressure was also taken to verify the computer data shown in Figure 4-155 (standard practice for all post grout shafts as a form of quality assurance and quality control). The displacement transducers were attached to a reference beam which was attached to the existing adjacent columns. Figure 4-156 shows the setup of the instrumentation on the west shaft. The adjacent columns were 3 feet away from the post grouted shafts and tipped 15 feet above the grouted shaft tips. This proved to be a problem for the west shaft. Ground heave was noticed during grouting causing the adjacent shafts with the reference beam to move up. The survey data and computer data shows a difference in displacement and is discussed further in this chapter.

4.5.4 Post Grouting Results

West Shaft. The post grouting results showing grout volume versus grout pressure are shown in Figure 4-157. A total of 11.7 cubic feet of grout was pumped to the toe of the shaft to obtain 250 psi of grout pressure. The displacement versus volume relationship is shown in Figure 4-158. A significant difference was observed between the survey data and the computer recorded data. The shaft displacement was measured to be 0.170 inches from the displacement transducers and 0.295 inches from the survey level. The difference of 0.125 inches was attributed to the reference beam movement during grouting. Figure 4-159 shows the shafts displacement versus load. The displacement shows the difference in the two measurements. The calculated load assumes a cross sectional area of 707 in^2 and a concrete modulus of 3420 ksi. A total of 84 tons was applied to the toe of the shaft. The drilled shaft post grout field log is shown in Appendix E.

East Shaft. Similarly, the grout volume versus grout pressure obtained during grouting of the East shaft is shown in Figure 4-160. The maximum grout pressure was 191 psi at a grout volume of 4.9 cubic feet. The displacement - volume relationship is given in Figure 4-161. Figure 4-162 shows the displacement versus load during grouting. The maximum uplift of the shaft during grouting was 0.281 inches. The maximum load of 74 tons was calculated using a cross sectional area of 707 in^2 and a concrete modulus of 3420 ksi. The field record of grouting is shown in Appendix E.

Figure 4-142 FM 507: Setup and drilling of the west shaft.


Figure 4-143 FM 507: Submersible de-sanding pump.

Figure 4-144 FM 507: De-sanding of the west shaft.

Figure 4-145 FM 507: Reinforcement cage placement in west shaft and de-sanding of east shaft.

Figure 4-146 FM 507: Strain gages and telltale tubes installed.

Figure 4-147 FM 507: 24" grout plate.

Figure 4-148 FM 507: Grout plate attached to reinforcement cage prior to placement.

Figure 4-149 FM 507: East shaft cage placement.


Figure 4-150 FM 507: Concrete placement in west shaft.


Figure 4-151 FM 507: Finished west shaft.

Figure 4-152 FM 507: Grout pump.

Figure 4-153 FM 507: Grouting of the west shaft.

Figure 4-154 FM 507: Data acquisition setup.

Figure 4-155 FM 507: Field surveying during grouting.

Figure 4-156 FM 507: Instrumentation setup on the west shaft.

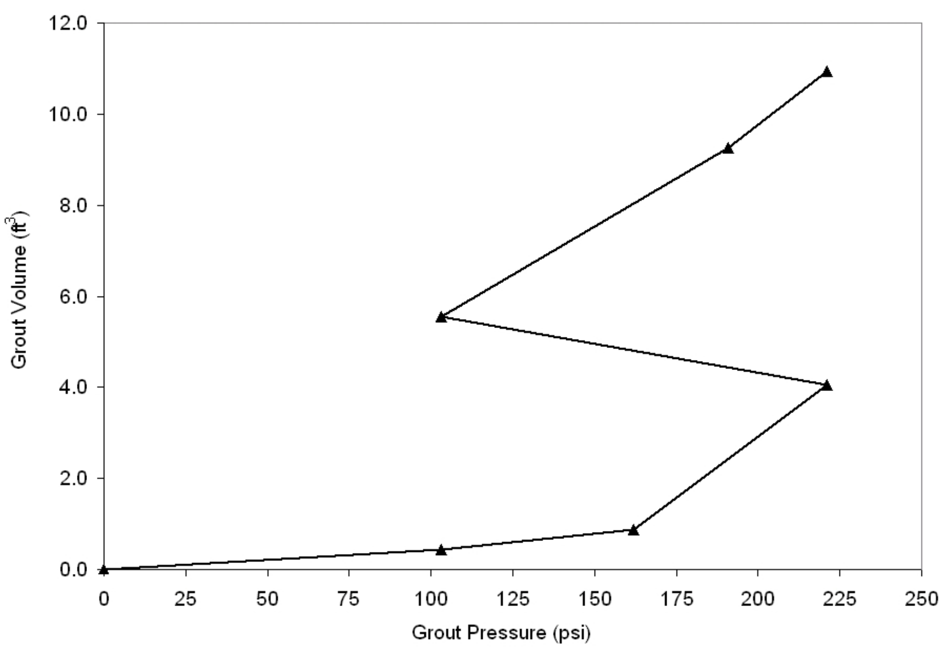


Figure 4-157 FM 507: West shaft grout pressure versus grout volume.

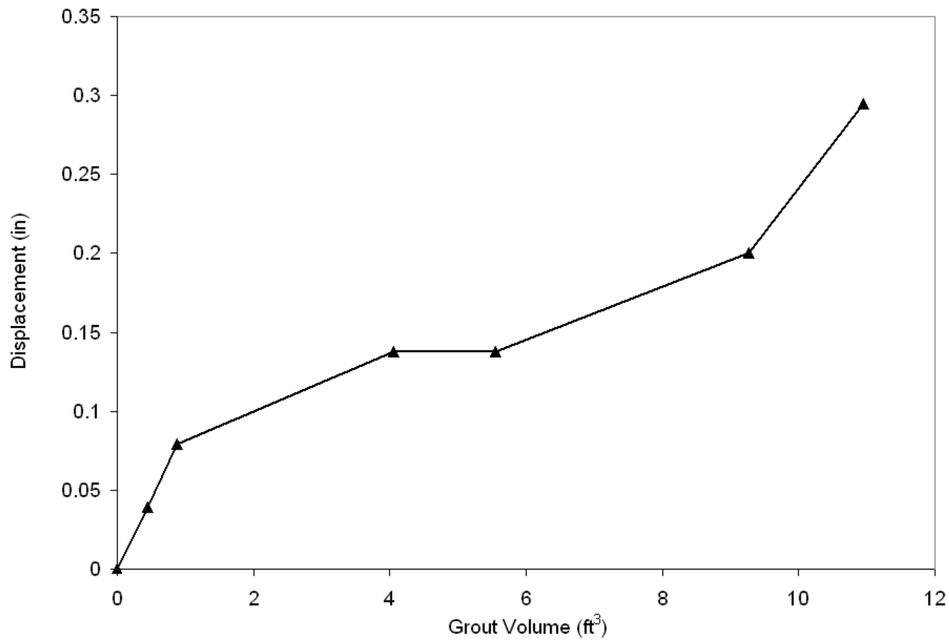


Figure 4-158 FM 507: West shaft grout volume versus displacement.

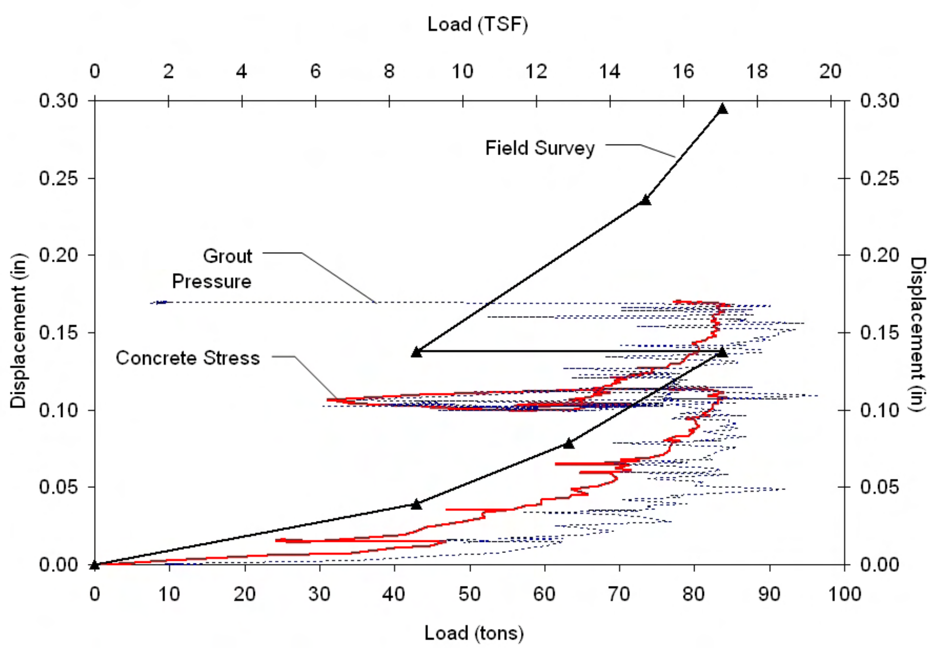


Figure 4-159 FM 507: West shaft load versus displacement.

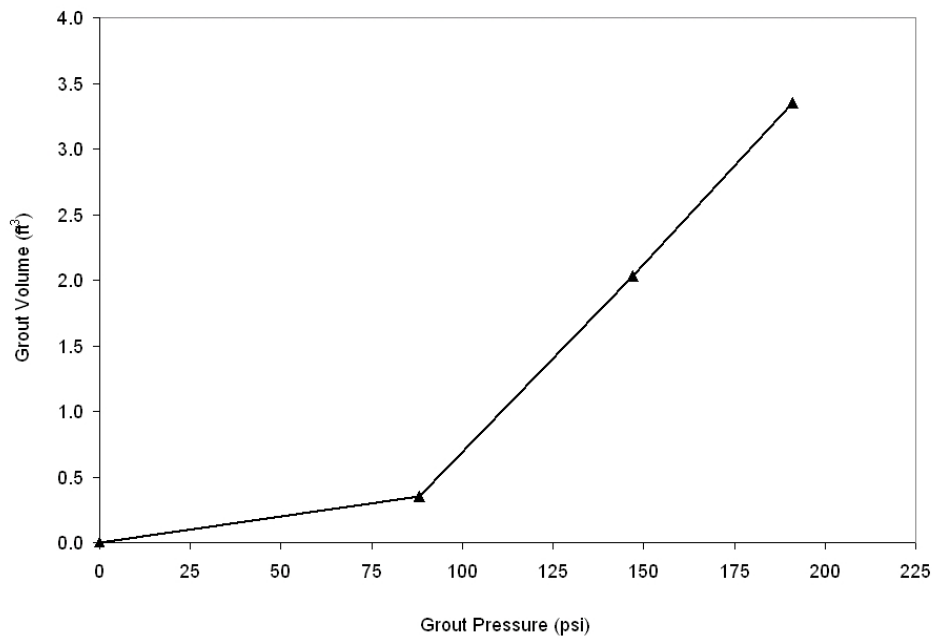


Figure 4-160 FM 507: East shaft grout pressure versus grout volume.

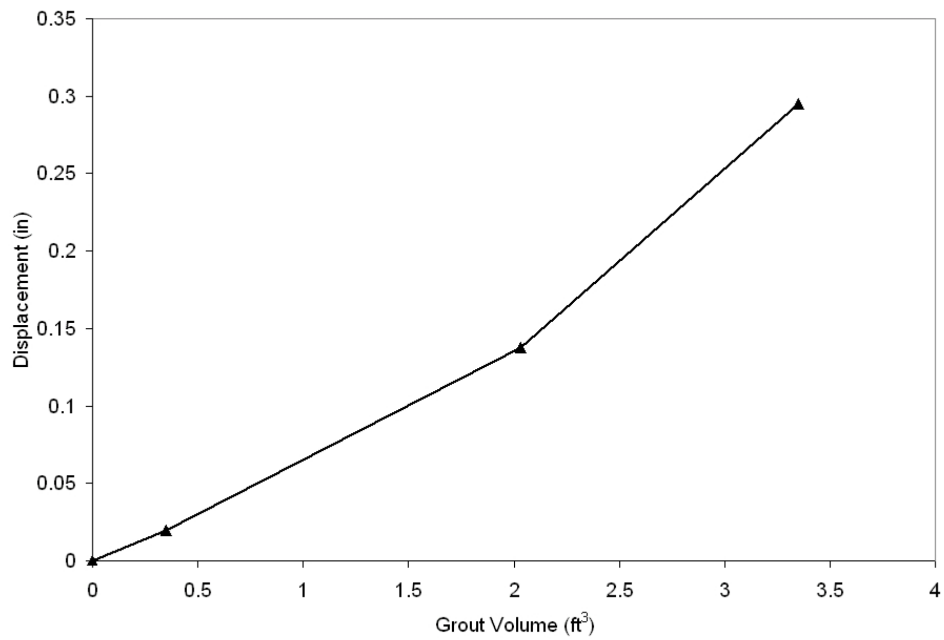


Figure 4-161 FM 507: East shaft grout volume versus displacement.

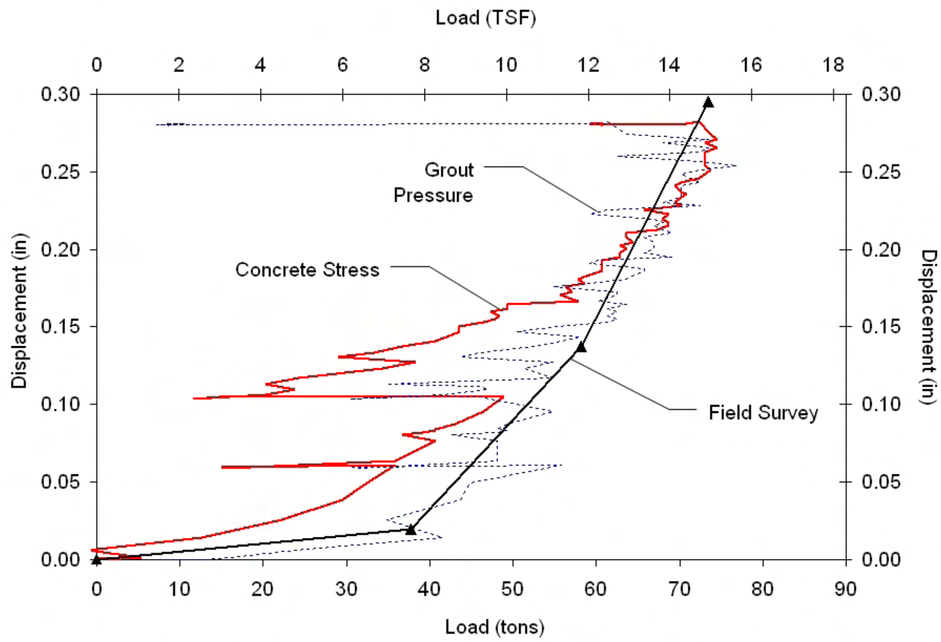


Figure 4-162 FM 507: East shaft load versus displacement.

4.6 Sonic Integrity Tests

Two of the five field programs contained additional quality assurance testing involving sonic integrity testing (SIT). This type of test is traditionally relied upon to verify length of piles or to indicate the presence of anomalous cross-sectional area. Although the results are somewhat subjective, the method is reasonably accepted to detect gross anomalies and the depth to said imperfections. This project experimented with SIT to ascertain whether it could register changes in a shaft tip reflection after post grouting and subsequent load testing.

Auburn / NGES. One of the two sites where SIT was employed included the NGES for silty soil (Auburn University). Figures 4-163 through 4-167 show the before grouting and after grouting SIT data for the five test shafts at the NGES test site in Opelika, Alabama in silty soil. Shafts TS-1, TS-2, and TS-3 (Figure 4-163, 164, and 165) show marked increase in the toe reflection after grouting with indications typical of a bulging cross section. This is consistent with a grout bulb as well as the noted shape of the exhumed shafts. Shaft TS-4 (Figure 4-166), the ungrouted shaft, showed no change in the sonic echo trace after testing. This would imply that the changes noted in the grouted shafts were due to the increase in tip area/volume and not the soil pre-compression which would have occurred during both grouting and load testing. Shaft TS-5 (Figure 4-167) shows no change.

TexDOT Demonstration. The second site where sonic echo tests were conducted was the Houston, Texas TexDOT demonstration site. Figures 4-168 through 4-171 show the results for the SITs conducted in Houston, Texas on post grouted shafts tipped in both sand and clay as well as those shafts that remained ungrouted. Figure 4-168 shows the before and after load testing results of SITs on test Shaft S-1 (ungrouted) tipped in dense sand. This shaft showed no change in the echo response after pre-compression of the toe area soil from load testing. However, its grouted counterpart, test Shaft S-2, showed marked improvement in the echo response (Figure 4-169) also tipped in dense sand.

The other two test shafts at the Houston project were tipped in hard clay ($N > 50$). These shafts showed the same change in echo response from before and after load testing. Figures 4-170 and 4-171 show these traces for test shafts C-1 and C-2, respectively.

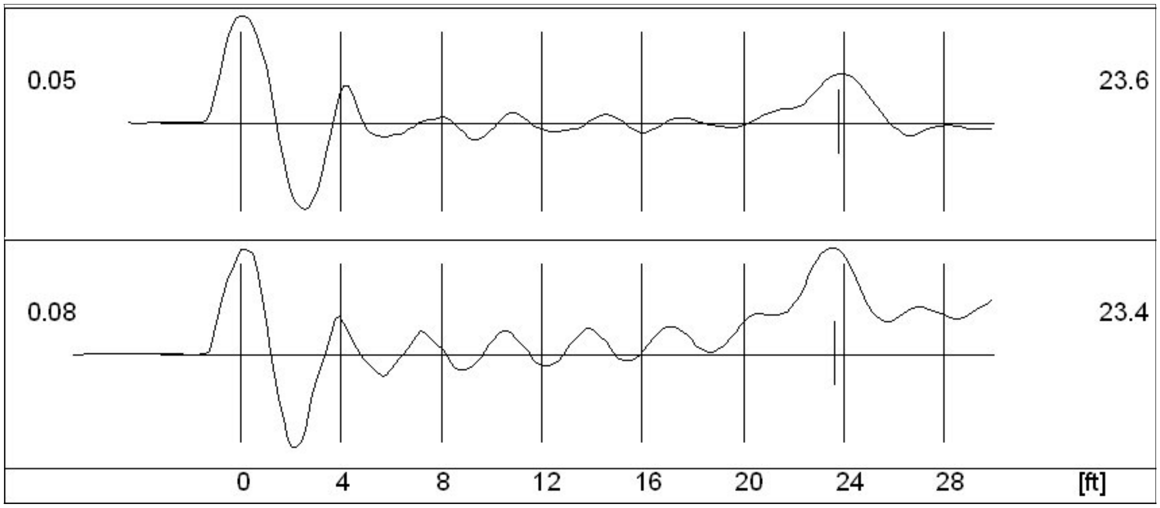


Figure 4-163 Auburn: SIT data for TS-1 before grouting (top) and after load testing (bottom).

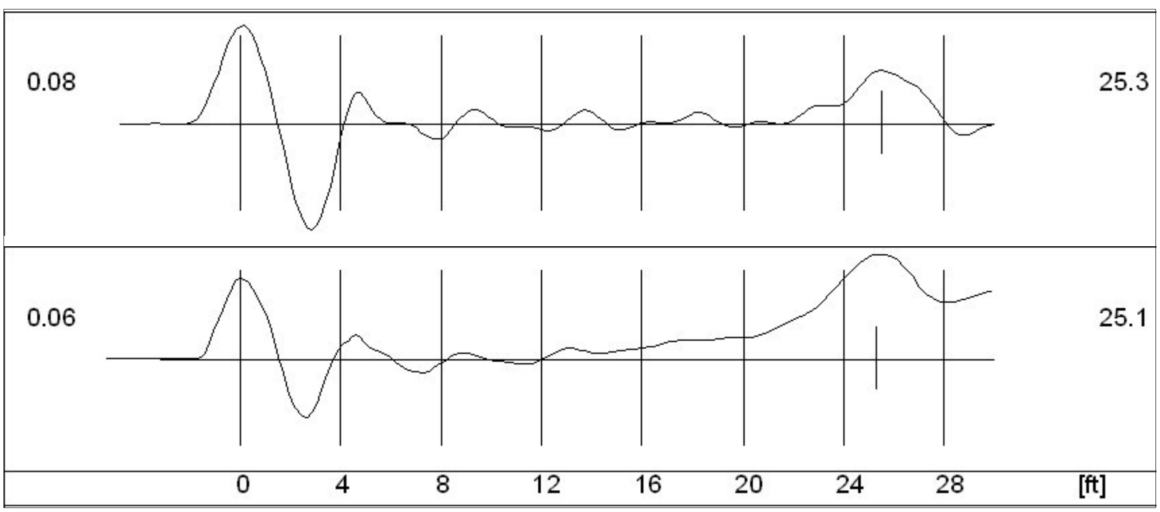


Figure 4-164 Auburn: SIT data for TS-2 before grouting (top) and after load testing (bottom).

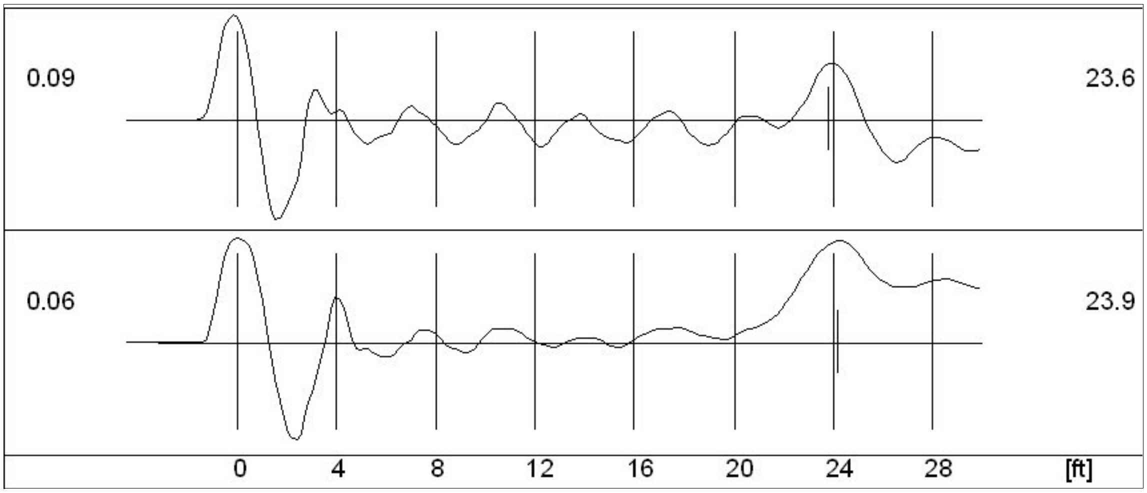


Figure 4-165 Auburn: SIT data for TS-3 before grouting (top) and after load testing (bottom).

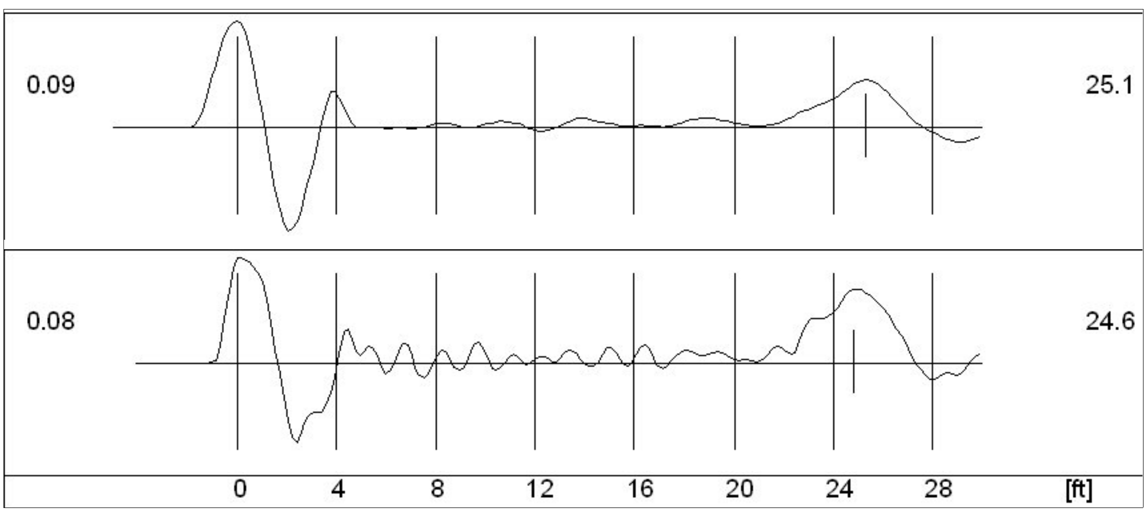


Figure 4-166 Auburn: SIT data for TS-4 day of grouting (top) and after load testing (bottom).

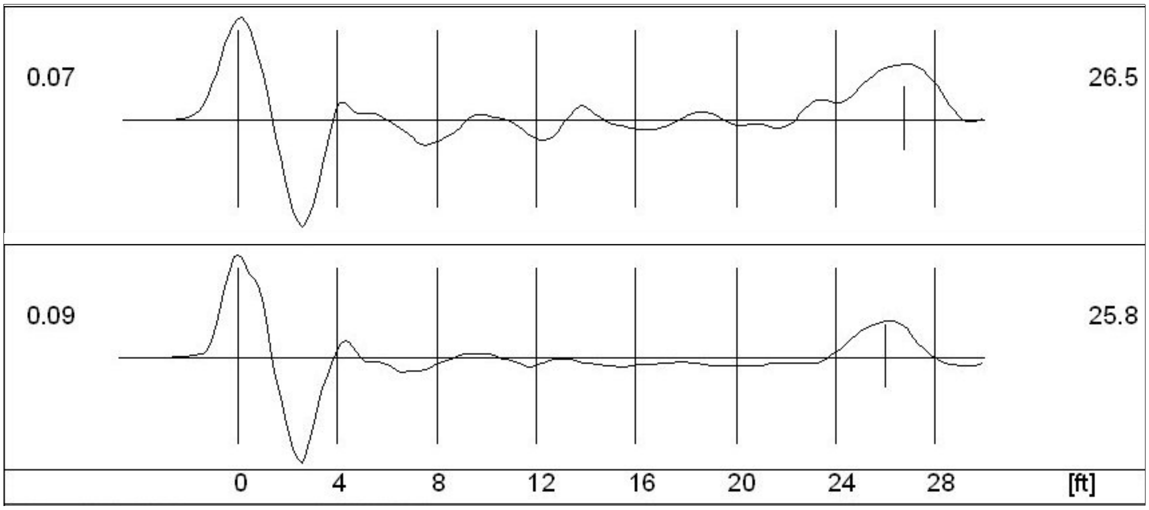


Figure 4-167 Auburn: SIT data for TS-5 before grouting (top) and after load testing (bottom).

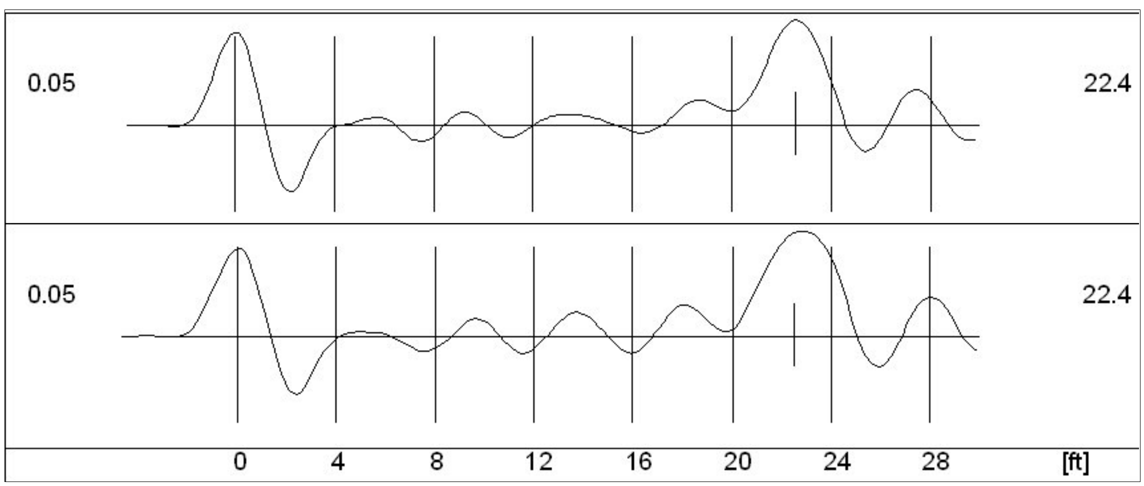


Figure 4-168 TexDOT Demo: SIT data for S-1 prior to grouting (top) and after load testing (bottom).

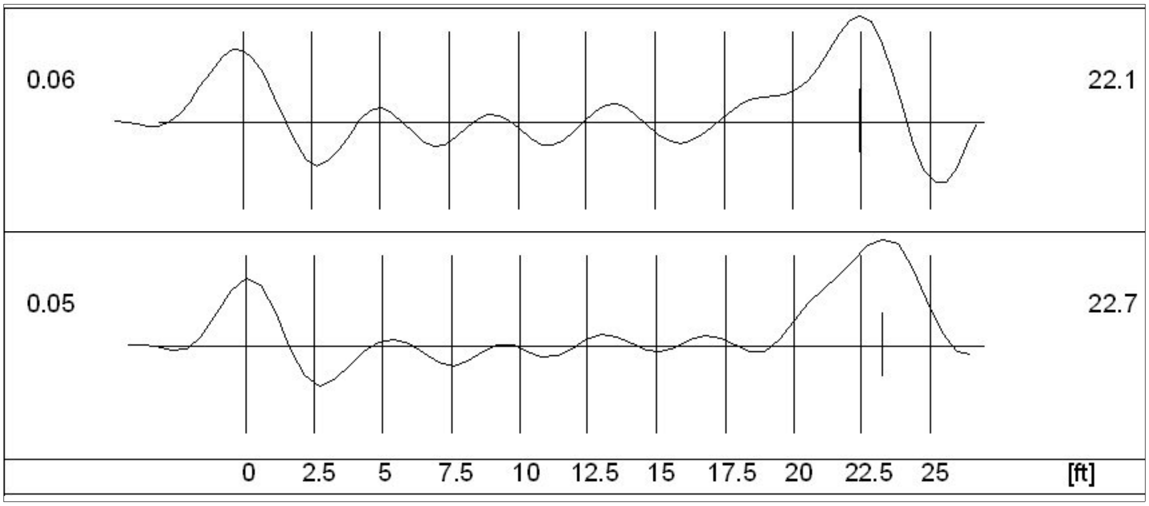


Figure 4-169 TexDOT Demo: SIT data for S-2 before grouting (top) and after load testing (bottom).

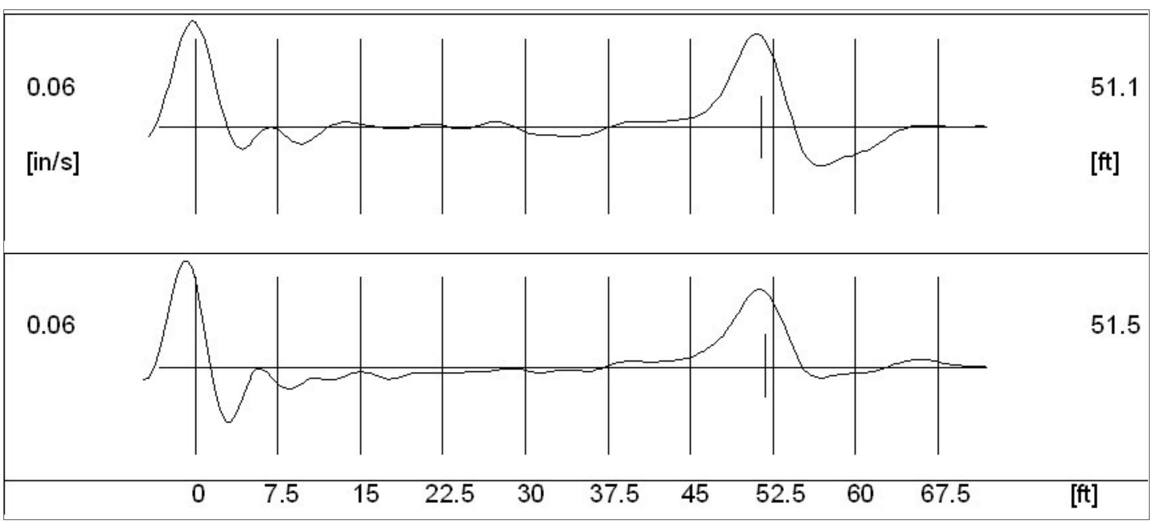


Figure 4-170 TexDOT Demo: SIT data for C-1 prior to grouting (top) and after load testing (bottom).

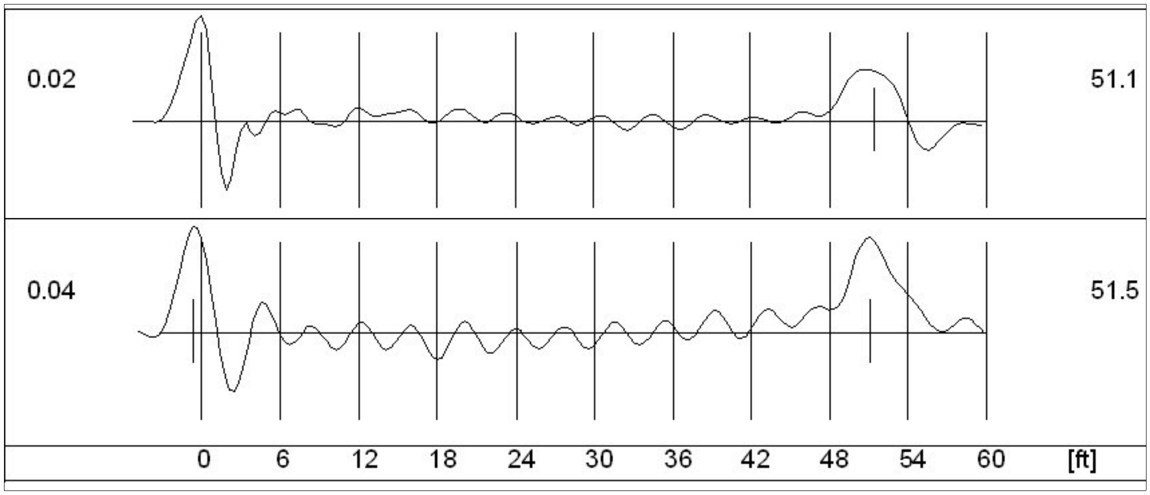


Figure 4-171 TexDOT Demo: SIT data for C-2 before grouting (top) and after load testing (bottom).

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 General Overview

At the onset of this project post grouting drilled shaft tips to either mitigate construct debris (soft toe) or to improve the end bearing response had been documented in numerous publications dating back as far as the early 1960's. These citations indicated that end bearing could be improved for sands and clays with ultimate capacities as much as two to three times ungrouted shafts. However, numerical procedures to predict the improvement were not available. In some instances, details concerning the diameter of the shaft, depth to the tip, and grout pressure employed could be extracted. Figure 5-1 shows the field of cited experiences prior to this study.

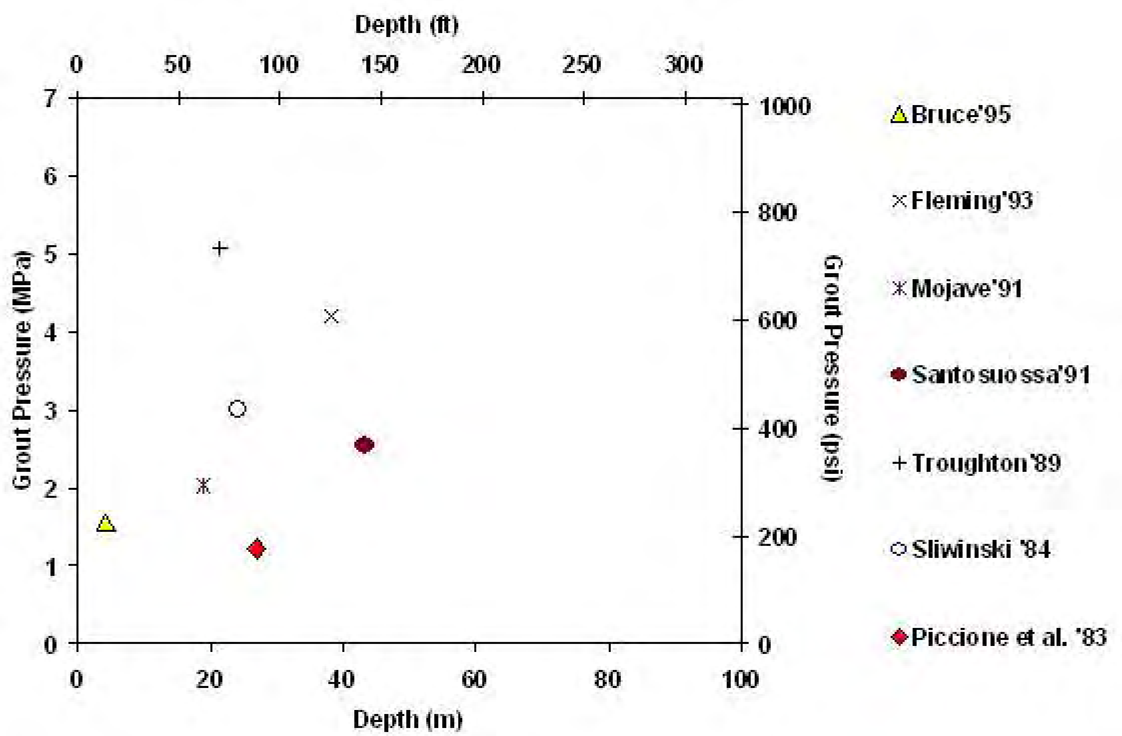


Figure 5-1 Published grout pressure versus depth prior to this research program.

This figure shows the trend of increasing applied grout pressure with depth which is in keeping with the understanding that the maximum grout pressure is dependent on the available side on which the grout pressure can react. This study has uncovered several other case studies that occurred after the original project inception as well as provided numerous experimental and production experiences that significantly add to this trend. Figure 5-2 shows the same figure (Figure 5-1) updated with the latest available data. This includes all the case studies outlined in Chapter 4 as well as unpublished international experiences.

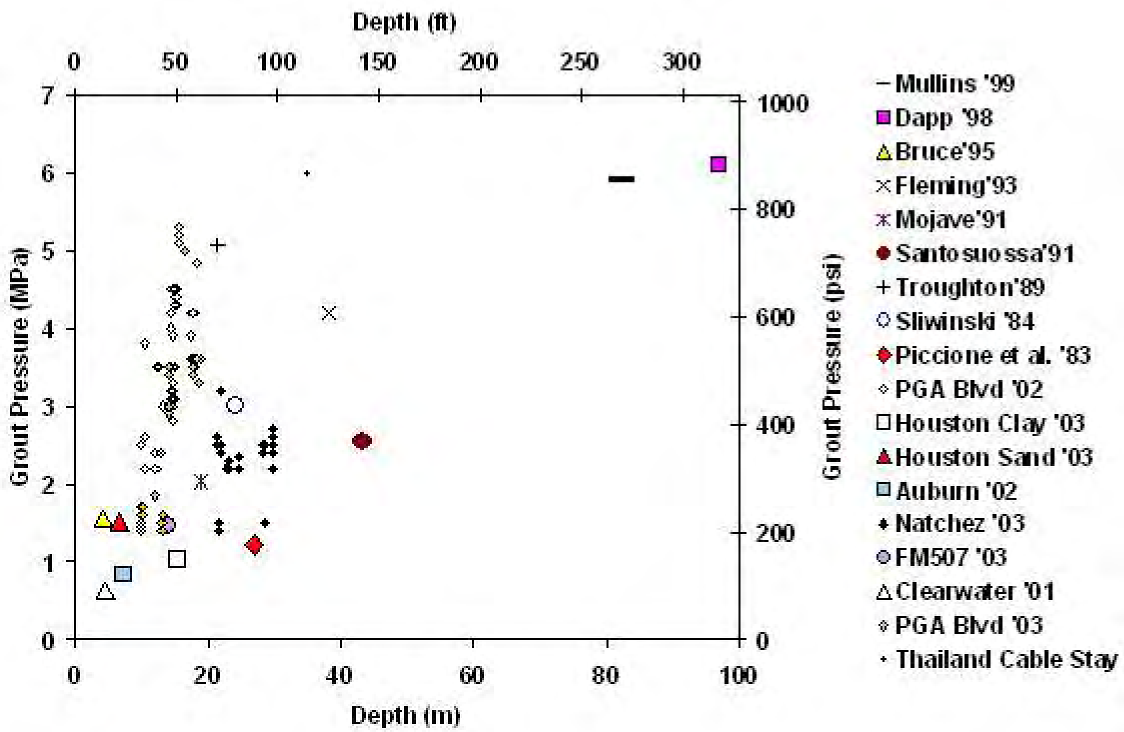


Figure 5-2 Grout pressure versus depth including Phase II data sets.

In general, the data from various sites tend toward groups of similar depth shafts. Those data points that were extracted from this study were mostly optimized designs that were grouted to the maximum permissible grout pressure. As such those points exhibit higher applied grout pressure for a given depth. In reality, the available side shear plays into the grout pressure as well as the length to diameter ratio. Therein, longer shafts of a given diameter and unit side shear have greater side shear and therefore can resist higher grout pressures. In concept, the anticipated grout pressure for a given site could be generalized with respect to the L/D ratio and a given average unit side shear. Figure 5-3 shows such a conceptual relationship.

As the grout pressure is a function of tip area, unit side shear area, and shaft length, the expression for anticipated grout pressure can be simplified as follows:

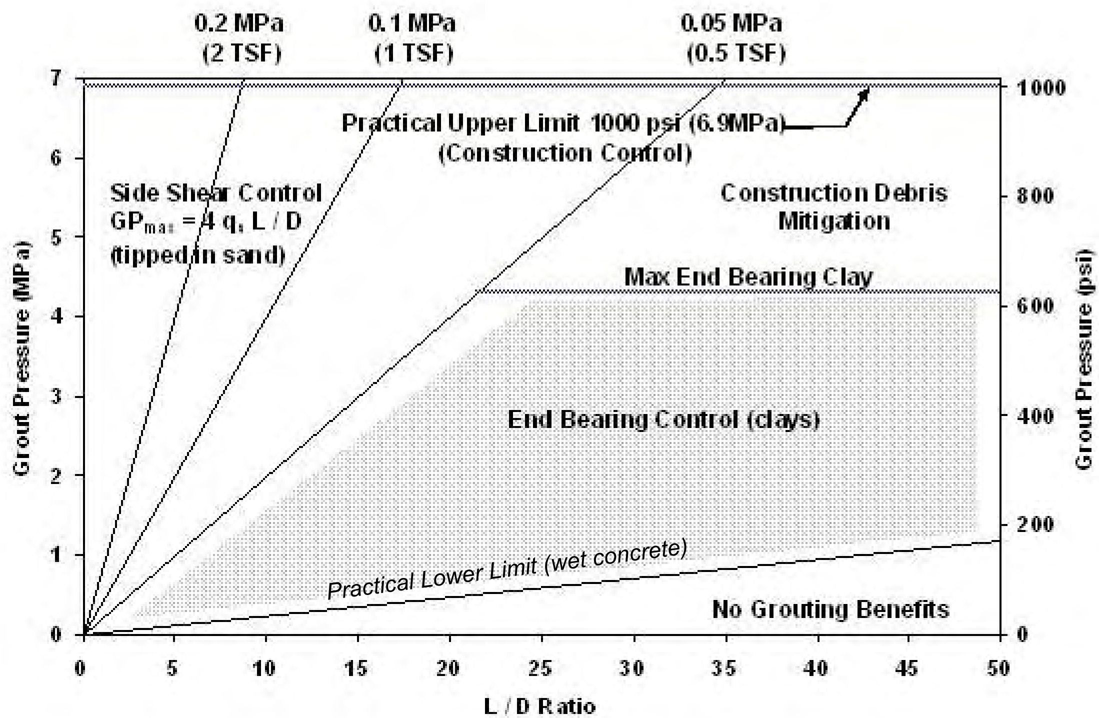


Figure 5-3 Pressure versus depth concept graph.

$$GP_{max} = Side\ Shear\ Force\ /\ Tip\ Area$$

$$GP_{max} = (qs\ \pi DL)\ /\ (\pi D^2\ /\ 4)$$

$$GP_{max} = 4qs\ L/D$$

Where GP_{max} represents the maximum predicted grout pressure, q_s the unit side shear, and L/D the length to diameter ratio.

Several ranges are also identified in Figure 5-3 that denote practical limits on grouting. The lines denoting unit side shear values present upper bounds on grouting for shafts constructed in soils with average unit side shear values of 0.5, 1.0 and 2.0 tsf. For all soils and L/D ratios, a practical upper limit on grout pressure should be applied that considers the practical construction limitations of the grout pump, grout tubes, or the working life of the neat cement grout. Although pressures as high as 1600 psi are attainable, a 1000 psi upper limit is more realistic without having to use specialized grout tubes. In environments more reflective of Florida's climate, grout pressures above approximately 750 psi may be problematic due to the additional heat generated in the grout by the pump (shortens the grout working life). Pressures higher than 750 are workable, but should be demonstrated with the entire grout system at hand. Further, admixtures may ease this condition.

A practical lower limit is also superimposed on to this relationship that represents the hydrostatic pressure of wet concrete. Assigning a grout pressure at or below this level does not provide a benefit worthy of the effort. Although in some instances the process of flushing the grout lines has shown grout volume taken into soft areas or unexpected voids, far more can be derived from a grouting protocol that makes full benefit from an optimized design.

Historically, those projects where grout pressure did not make full use of the available side shear (but were well above the hydrostatic wet concrete head) were used to mitigate soft toe conditions caused by deep excavations and/or prolonged cage placement and concreting processes. Finally, when grouting in clays, the grout pressure should not be expected to exceed the ultimate end bearing stress for conventional shafts tipped in the same soil. AASHTO indicates this upper bound to be 40 tsf (shown), however, a given site may not be capable of developing this value and would rather fall more in line with calculated values.

Using the side shear and L/D relationships rather than depth, Figures 5-2 and 5-3 are merged in Figure 5-4.

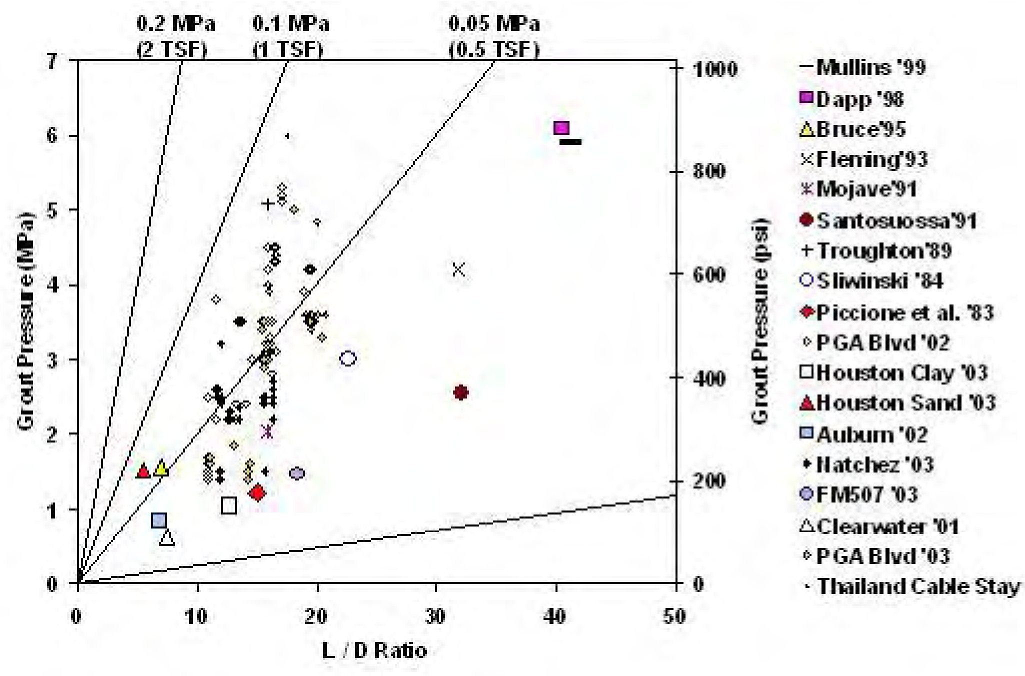


Figure 5-4 Grout pressures versus L/D ratio including Phase II data sets.

Note that in most cases, the PGA Blvd Phase I construction employs a higher unit side shear (> 0.5 tsf) than Phase II (approx. 0.5 tsf) which is in keeping with the Phase II production borings that showed lower blow counts. Likewise, note that the Natchez Trace Parkway project consistently employed average side shear values on the order of 0.5 tsf or less.

5.2 Design Approach Revisions

The design of post grouted drilled shafts as outlined in Phase I of the project has been maintained for shafts tipped in sandy soils. New data was added to the database which slightly modified the slope and intercept values of the TCM curves. Additionally, a new approach is presented that may provide more opportunities for database expansion in the future. Phase I of the project focused primarily on sandy soils as these soils were purported to produce the highest level of improvement. However, in Phase II, data became available in other soil types (i.e. silts and clay) that show merit in post grouting shafts tipped in those soil types.

Sand. The design approach developed in Phase I of this project was based on the performance of shafts tipped in shelly sands and silty sands. Further, the improvement was based on the relative performance gain of post grouted shafts over similarly constructed ungrouted shafts. In Phase II, several more sets of data were made available that slightly altered the slope of the TCM vs.GPI relationships. Further as the design approach is most aptly applied to shafts designed using a prescribed ultimate displacement (Reese and O'Neill, 1988), the most useful design curve is that which assumes a 5%D ultimate displacement. This conventional design of end bearing capacity was based on numerous full scale tests that fully defined the ultimate capacity as a function of permissible displacement. Figure 5-5 shows the range of results as well as the trend of all tests conducted on ungrouted shafts with regards to end bearing response. The values outlined for displacements 1, 2, and 5%D represent the most appropriate intercept values for the TCM vs GPI curve for 5%D ultimate designs. Thereby the intercept (where grout pressure equals zero) of the updated design of post grouted shafts reflects these values instead of the relatively few ungrouted shafts used to set these intercepts originally. Figure 5-6 shows the slight adjustments that were a result of both the new data sets and correcting the intercept values.

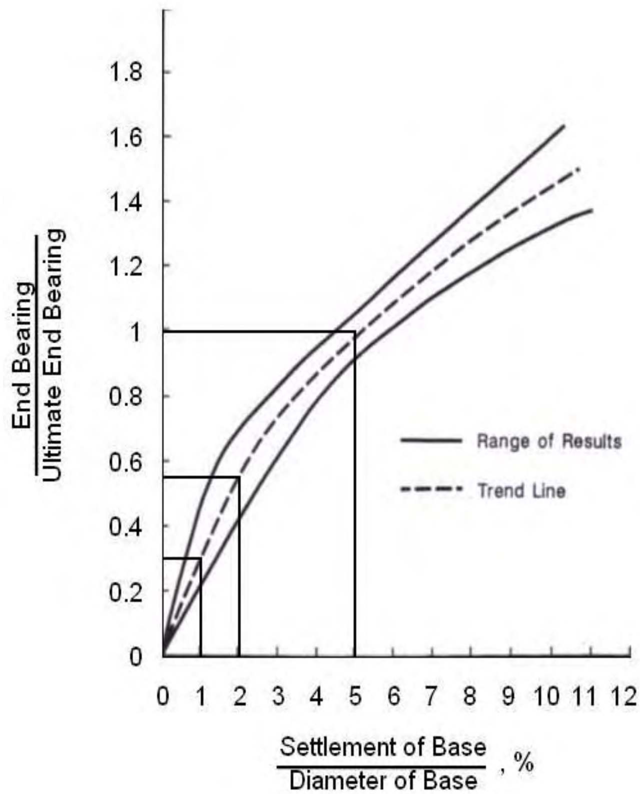


Figure 5-5 Intercepts for TCM/GPI curves for sand (after Reese & O'Neill, 1988).

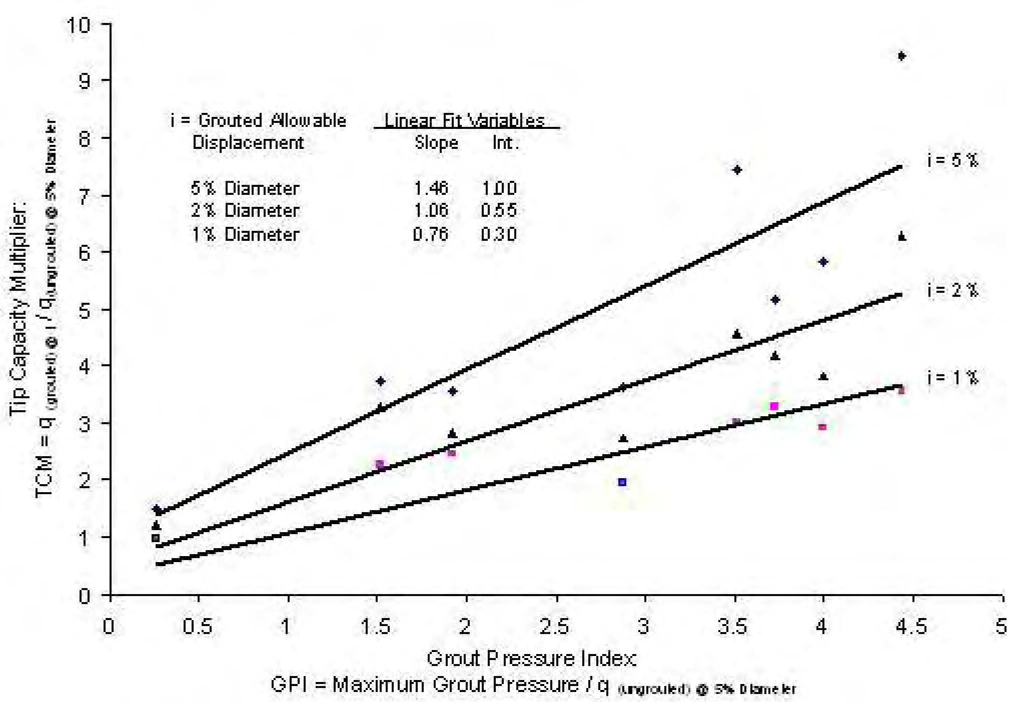


Figure 5-6 Phase I updated TCM and GPI 5% design graph for sand.

Upon re-evaluating the intercept values of the TCM curves, it became apparent that if the ungrouted shafts used to determine the relative improvement were artificially low, then the TCM values would be artificially high when applied to standard design values. This stems from the observation that most of the control shafts used in the study exhibited slightly lower end bearing values than the AASHTO predicted value of 0.6N (in tsf). Although the difference was slightly lower, it did not reflect the more usual observance that AASHTO predicted end bearing values are lower than measured. Further, ungrouted (conventional) shafts are more subject to large variations in end bearing associated with numerous construction techniques and mishaps. Conversely, grouted shafts tipped in sand will converge to a more reproducible end bearing based on the availability of adequate side shear. With this in mind, using the more exhaustive data set based on end bearing predicted by 0.6N would produce a more rational approach. This also allows more data sets to be included that do not have an ungrouted control shaft provided the soil boring data were available. Figure 5-7 shows a different TCM design curve based on the ratio of measured grouted end bearings and predicted end bearings. The revision affects the slope of the TCM curves but the intercepts are maintained reflecting the larger database presented by Reese and O'Neill (1988).

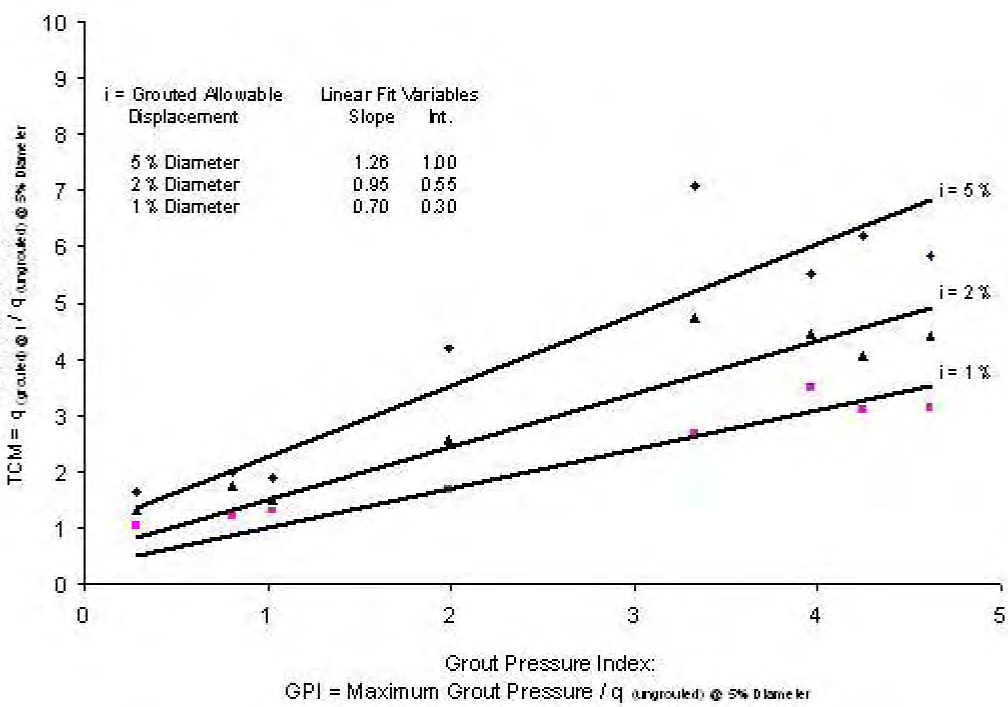


Figure 5-7 Phase II recommended TCM and GPI 5% design graph for sand.

To illustrate the magnitude of the revisions on predicted end bearing values, the following design example is provided using the various slope and intercept values developed over the duration of the two phases of this project (Table 5-1):

Table 5-1 Phase I & Phase II design slope and intercepts from grouting research program.

Displacement	Phase I		Phase $I_{updated}$		Phase $II_{recommended}$	
(%D)	Slope	Intercept	Slope	Intercept	Slope	Intercept
5%	1.86	1.00	1.46	1.00	1.26	1.00
2%	1.27	0.63	1.06	0.55	0.95	0.55
1%	0.79	0.49	0.76	0.30	0.70	0.30

Given: A 3 ft diameter drilled shaft tipped in sand (SPT N_{tip} = 30 and F_s = 200 tons).

* Calculate the available grout pressure:

Grout Pressure = Side Shear Force / Tip Area
$GP = 200 \text{ tons} / ((3 \text{ ft})^2 \quad /4)$
GP = 28.3 tsf

* Calculate the ultimate and allowable end bearing:

Ultimate End Bearing = 0.6 * SPT N (Reese & O'Neill, 1988)
$q_p = 0.6 * 30$
$q_p = 18$ tsf

* Calculate the grout pressure index (GPI):

Grout Pressure Index = Grout Pressure / Ultimate End Bearing
GPI = 28.3 tsf / 18 tsf
GPI = 1.57

Determine the tip capacity multiplier (TCM) at various displacements. The slope and intercepts are found in Table 5-1.

* TCM = GPI * Slope + Intercept

Table 5-2 TCM values for all phases of research program.

%D	Phase I	Phase $I_{updated}$	Phase $II_{recommended}$
5%	3.92	3.29	2.98
2%	2.62	2.21	2.04
1%	1.73	1.49	1.40

*	Calculate the grouted end bearing.

Grouted End Bearing = TCM * Ultimate End Bearing

Table 5-3 Grouted end bearing for all phases of research program.

%D	Phase I	Phase I $_{updated}$	Phase II $_{recommended}$
5%	70.6 tsf	59.3 tsf	53.6 tsf
2%	47.3 tsf	39.9 tsf	36.7 tsf
1%	31.1 tsf	26.9 tsf	25.2 tsf

The effect of changing from control shaft data to predicted ungrouted capacities is relatively minor although it does produce a more conservative prediction (as discussed earlier). The bulk of the changes are the result of introducing new data sets that showed more modest improvement in sand.

Clay. Designing for shafts tipped in clay should apply a relatively conservative approach as outlined herein. The performance of any post grouted shaft project (regardless of soil type) should not rely solely on office predictions of improvement, but rather the verified performance of a pilot grout test program. However, the designer needs to have some ability to predict field performance even if only for cost estimating.

The duration of the grout loading event is not sufficient to affect significant changes in the void structure or shear strength of clays beneath the tip. As a result, the end bearing material will begin to exhibit bearing capacity failure as the grout pressure approaches the ultimate end bearing. Therefore, the maximum attainable grout pressure can be predicted to be equivalent to the ultimate ungrouted end bearing capacity. End bearing improvement will still be realized. Although this study as well as others have shown ultimate end bearing capacity improvement of 100 to 200%, it is recommended that the ungrouted ultimate capacity be used as usable capacity that can be mobilized in relatively small displacements. However, a TCM > 1 may be determined when load testing is used for verification.

Silt. Designing for grouted tips in silty soils should not use sand criteria nor should it be restricted to clay criteria. The testing conducted in Phase II at the NGES for silty soils suggests that a TCM / GPI relationship may exist that could be used for design. Therein, silty soils are more free draining than clay and less than sands. The improvement in silts is more like clay than sand. The most notable difference between silt and clay is that the attainable grout pressure can be higher than the ungrouted end bearing. This is presumably due to the slight densification that is possible over the duration of the grouting. Figures 5-8 and 5-9 show a plausible design approach using a TCM approach. The limitation which has not been resolved at this time lies in defining an upper grout pressure limit.

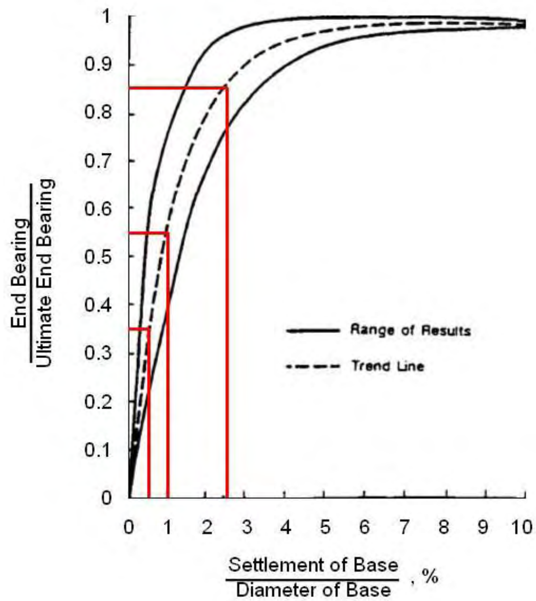


Figure 5-8 Phase II recommended intercepts for cohesive soils (after Reese & O'Neill, 1988).

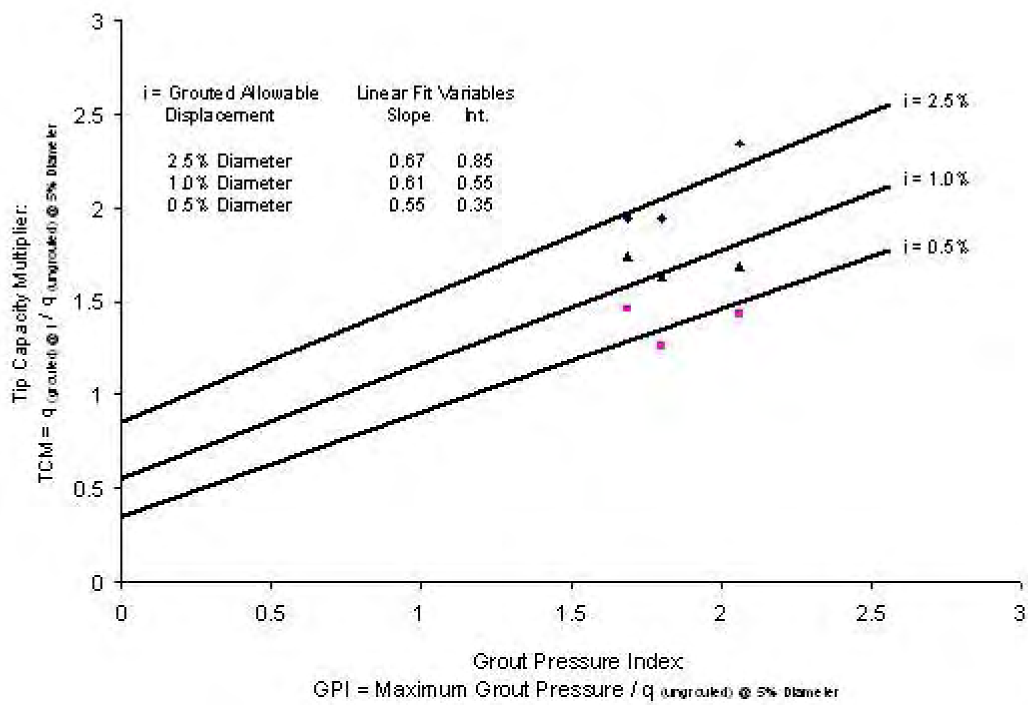


Figure 5-9 Phase II recommended TCM and GPI 2.5% design graph for silt.

Until such a limit can be defined, the authors recommend treating silts like clay until a pilot post grout test is conducted.

5.3 Soil Type Improvements

The amount of improvement that can be anticipated is understandably soil type dependent. Therein, the largest benefit is derived from readily compressible, free draining soil with a low relative density. However, all soil types show improvement. Table 5-4 shows the improvement cited for various soil types and consistencies investigated during this project.

Table 5-4 Summary of improvements in various soil types.

Site Location	Improvement (%)	Soil Type / Consistency
Clearwater Site II	743	Loose Silty Sand
Clearwater Site I	349	Loose to Medium Dense Shelly Sand
PGA Blvd.	263	Loose to Medium Dense Sand
Natchez Trace Parkway	109	Stiff Silt
Royal Park Bridge	83	Cemented Sands
NGES / Auburn	77	Medium Dense Silt
TexDOT Demo	71	Very Stiff Clay
TexDOT Demo	41	Dense Sand

5.4 Field Recommendations / Practical Considerations

Although post grouting drilled shafts is not a particularly difficult procedure, several recommendations are presented that may circumvent delays and help to provide a better overall foundation element. These recommendations relate to pilot grout test programs, grout distribution system installation, and the grouting process.

Pilot Programs. Pilot grout test programs should be conducted at the onset of any production post grouted shaft project. This program is intended to define the grout criteria as well as demonstrate that the grouting contractor is familiar with the process. Shaft construction including cage preparation and grout tube installation should follow the guidelines below as well as the grouted shaft specifications provided by the State (Appendix D). During the grouting process, the grout pressure, grout volume, and shaft uplift should be monitored and recorded at intervals capable of clearly delineating the following relationships: (1) grout pressure vs. uplift displacement, (2) grout volume vs. uplift displacement, and (3) grout pressure vs. grout volume. Relationships (1) and (3) should appear similar in shape. Clear increases in displacement or volume without increase in pressure are an indication of ineffective grouting. Figures 5-10 and 5-11 show field measurements of two scenarios where these three relationships were monitored to show grouting effectiveness (side shear and end bearing control). In all instances, if grout pressure is still increasing, end bearing enhancement from grouting is still effective.

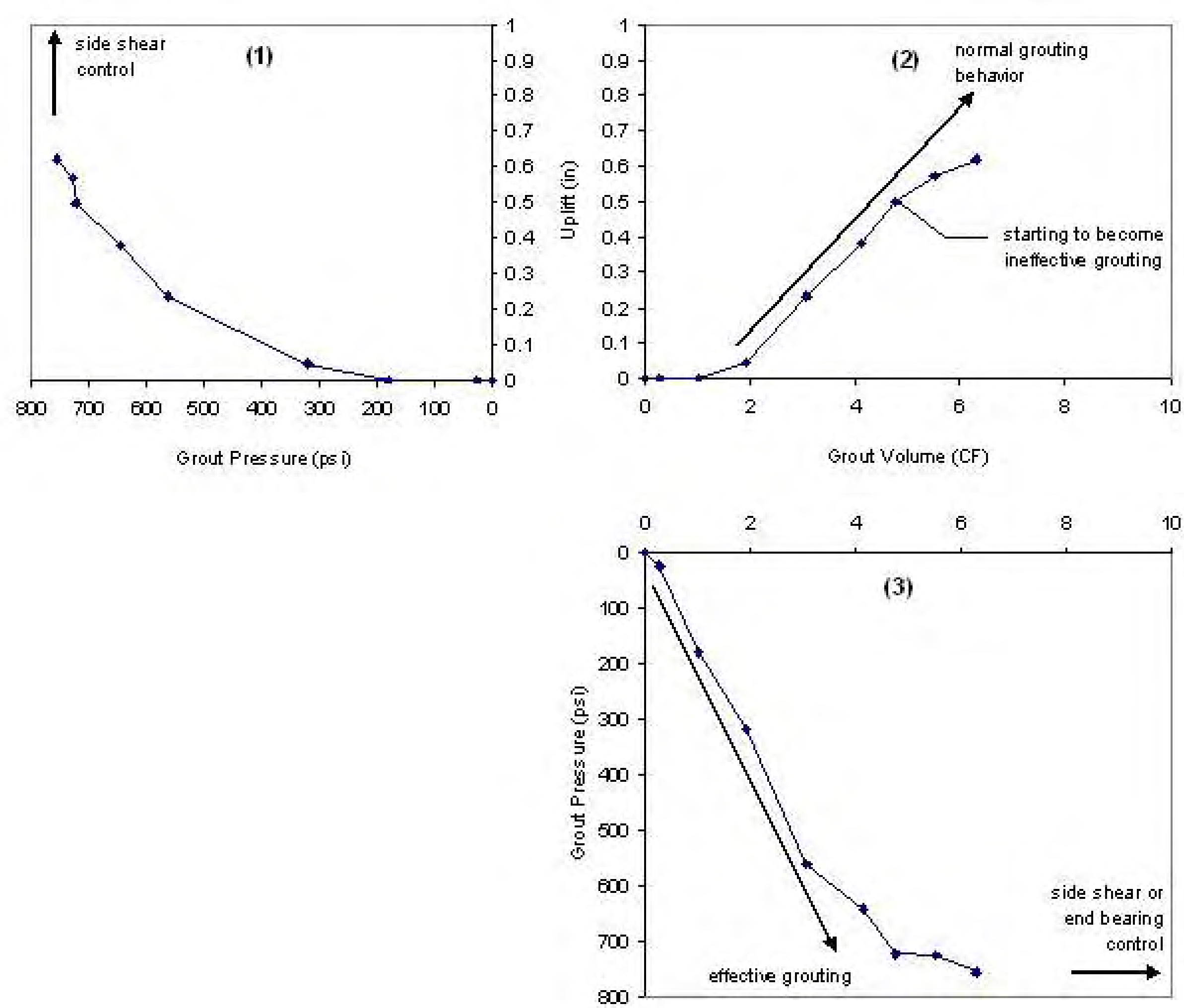


Figure 5-10 Normal behavior for side shear-controlled, post grouted shafts (sands).

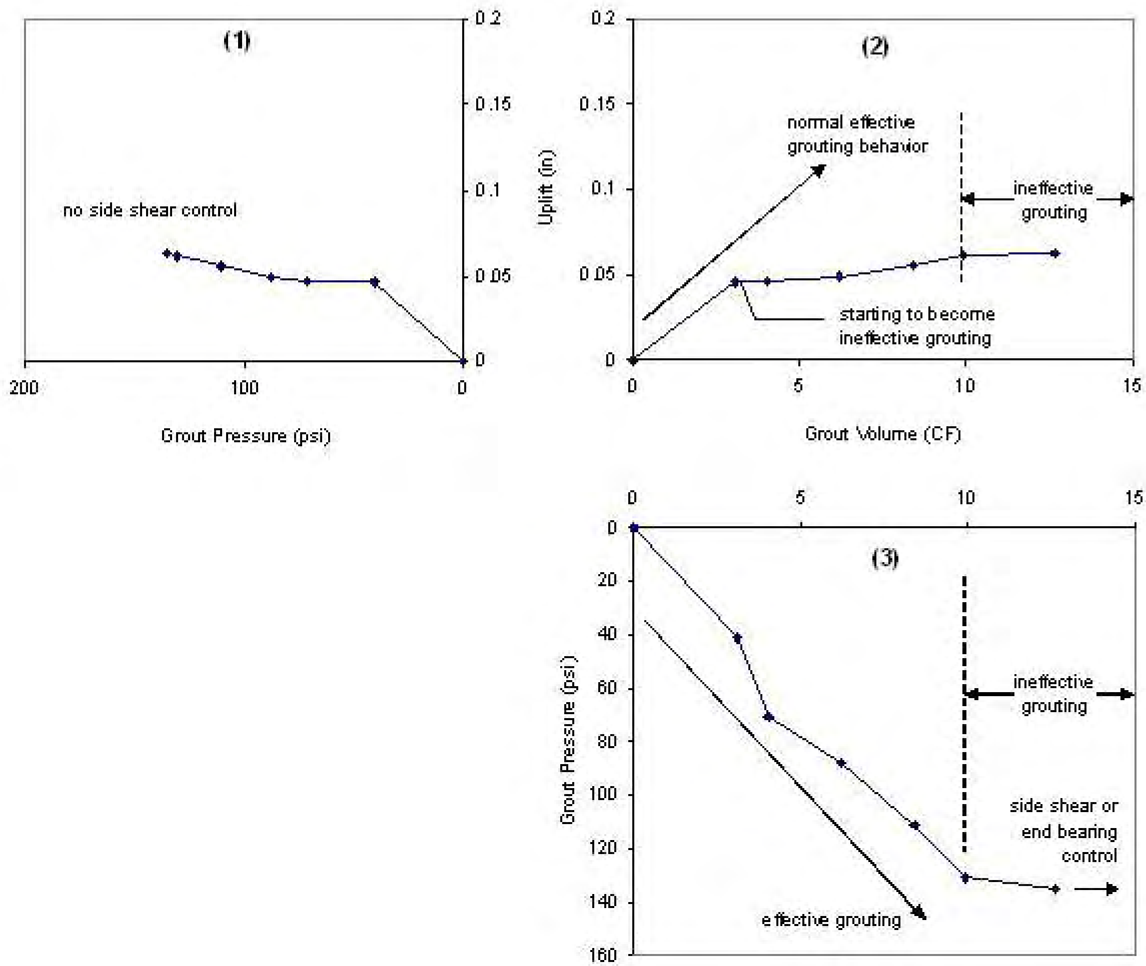


Figure 5-11 Normal behavior for end bearing-controlled, post grouted shafts (soft clays).

In most cases, a post grouted shaft should exhibit side shear control when tipped in sands (Figure 5-10). Exceptions may exist when the shaft design is governed by scour and the existing overburden provides additional side shear resistance during grouting. In contrast, shafts tipped in clay may only compress the bearing stratum without significant uplift displacement. In general, relationship (1) shows the side shear response to upward loading, a near vertical slope is clear indication of ultimate side shear and ineffective grouting; (2) identifies the grout volume usage trend, a zero slope is not in itself indication of ineffective grouting without corroboration from relationship (3), see Figure 5-11; and (3) can be used to show the bearing stratum compression as no uplift may be realized even though notable grout volume has been placed - a zero slope denotes either side shear or end bearing control. When used collectively, these relationships quickly identify the geotechnical mode governing the grouting program. The pilot grout program is conducted in order to verify the design and set the grouting procedure. It should not be preconceived as to the outcome of the test prior to its commencing.

Grout System Installation. Cage assembly and grout tube installation require some foresight to minimize unfortunate occurrences. In some instances, it may be necessary to install the grout distribution system (e.g. flat jack assembly) after the cage has been hoisted into a vertical position. This prevents needless damage to the cell caused by cage racking. When flat jack assemblies are used below water or slurry depths of 50 feet, a geo-grid or other protection should be layered between the flat jack upper plate and the rubber membrane. This measure is to prevent the rubber membrane from being forced into the grout tube from pressure associated with cage placement in the slurry or additional stress from concreting. The increased potential for membrane rupture from the additional weight of concrete (over and above hydrostatic pressure) can be minimized by filling the grout tubes with water prior to concrete placement.

Grouting tubes should be capable of withstanding the full design grout pressure. It is encouraged to use higher strength tubing than necessary especially for pilot grout programs exploring the capabilities of the shaft side shear and end bearing resistance to grout pressure. The inner diameter of the grout tubing should be large enough to permit the introduction of a smaller diameter tube for flushing. The exposed portion of the grout tubes should be adequately restrained against the lateral movement caused by the surging of the grout pump lines. For example, SCH 80 pvc pipe can usually provide adequate pressure resistance, but is not capable of resisting the bending moment caused by heavy grout pump hoses.

When cross-hole sonic logging (CSL) access tubes are to be used, they should be capped sufficiently at the bottom of the tube so as to withstand the full grout pressure. The close proximity of the CSL tubes with the grout distribution system increases the probability of high grout pressure being directly in contact with them. Likewise, embedded strain gages in the proximity of the grout plate are likely to experience fluid or grout pressure which exceed typical instrumentation conditions. In such cases, strain gages should be placed no closer than the larger of either 1 shaft diameter or 3 feet from the grout plate. Alternately, the hermitic sealing of the gages should be designed to withstand the full grout pressure.

Grouting. Although literature recommends w/c ratios of 0.4 to 0.6 as reasonable, a w/c ratio of 0.5 should be adopted at the onset from which slight variations can be made. In warm ambient conditions or where high grouting pressures (above 700 psi) are required, the w/c ratio may be increased to account for accelerated set times of the grout. A highly mobile grout is not desirable. Likewise, the pumping rate of the grout should be maintained as slow as practicable. High pumping rates and high w/c ratios have detrimental effects. When higher w/c ratios are necessary, pump rates should be decreased to permit expulsion of free water from the grout into the surrounding soils. Thereby, the grout bulb forms a filter cake at the grout-soil interface. Note: This consideration is also shaft diameter and soil type dependant whereby smaller diameter shafts and less pervious soils require slower pumping rates. In this study, the only complications associated with grouting were attributed to high w/c ratios (e.g. > 0.55). Figure 5-12 shows the range of pumping rates experience during this research program. The pumping rates are expressed as the grout volume per time per shaft tip area. In most cases, the high pressure, low volume pumps used for neat cement can only supply upwards of 1 cu ft / min and do not pose a significant threat to the success of a post grouted shaft project.

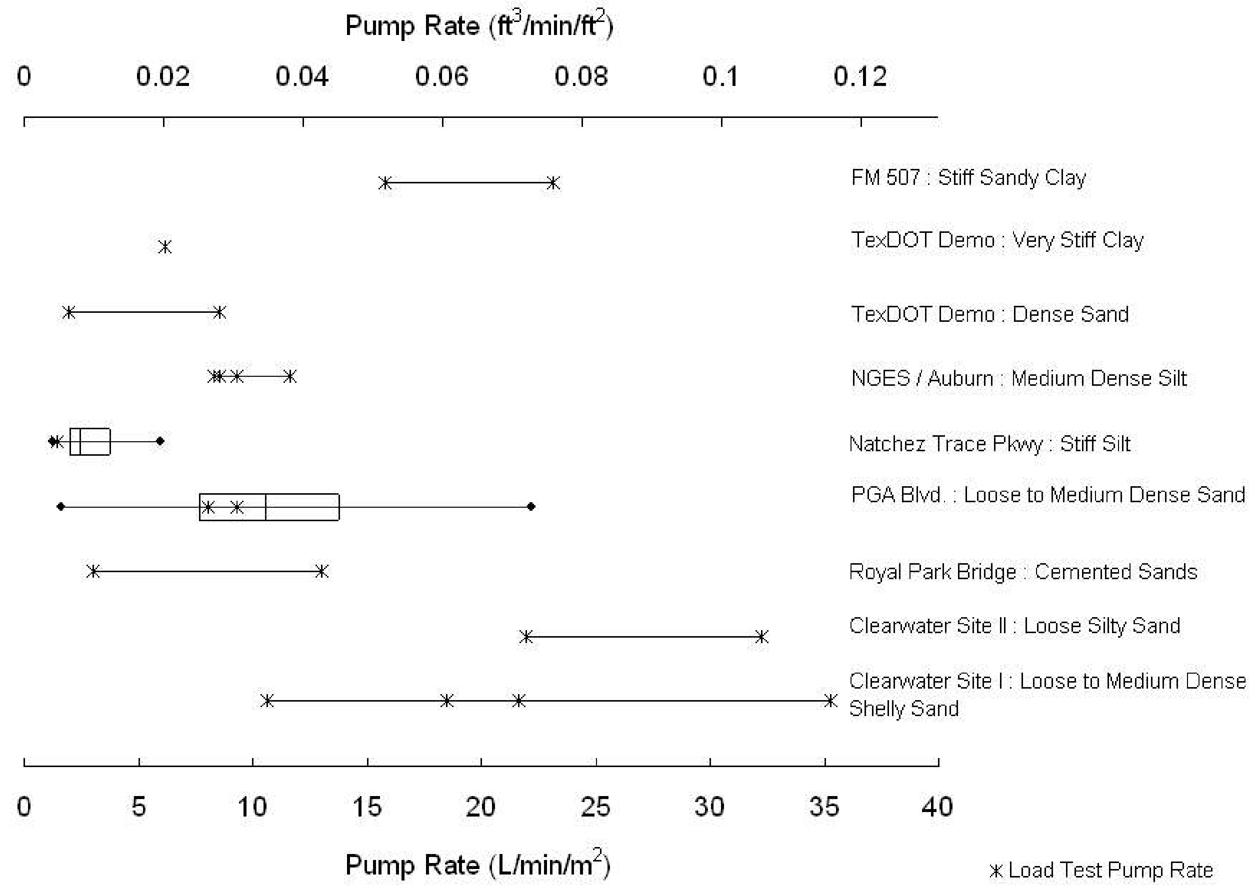


Figure 5-12 Grout pumping rates successfully used during this project.

If the grout becomes too mobile as a consequence of one of the above conditions, one of the relationships outlined and monitored in the pilot program will show ineffective grouting. In such a case, the grouting will need to be terminated, lines flushed with clear water, and grouting resumed later after the initial grout has set. The grouting contractor should be ready to flush lines and/or terminate and resume at any time as standard practice. In many instances, unforeseen weather may dictate this scenario. Should the grout tubes become blocked by soils or thickened grout, a small diameter tube should be readily available that can be inserted within the grout tube the full length of the shaft to flush out the blockage.

In cases where the design grout pressure cannot be achieved especially when the shaft uplift (as measured by survey) has not exceeded the recommended maximum, the following steps should be taken (Table 5-5): (1) grout tubes should be flushed with clean water and the w/c ratio should be reduced by 0.025 before resuming, (2) if step (1) fails to provide the desired outcome, then all grout tubes should be flushed and grouting should resume the next day. This type of staged grouting can be conducted as many times as necessary.

If the maximum uplift is encountered prior to achieving the design grout pressure, grouting should be terminated, grout tubes flushed, and remedial side shear enhancement will have to be implemented. Side shear can be enhanced using a series of full-length compaction grouting points conducted radially around the circumference of the shaft. Tip grouting can be resumed once remediation is completed.

Although a grout test program is helpful in determining the anticipate grout volume for a given site and shaft diameter, it is recommended that the materials required to mix three times that volume be available to account for unforeseen grout volume takes.

Table 5-5 Trouble Shooting Guide for Post Grouting

Problem	Observations / Conditions	Counter Measures	
		Design Phase	Production Phase
Unable to achieve design grout pressure	excessive uplift *vertical slope (1)*	re-evaluate side shear use more conservative side shear design	stage grout after compaction grouting around shaft
			re-evaluate shaft capacity with available side shear and use dead load reaction to allow a higher grout pressure
	excessive grout volume *zero slope (2) and (3)*	stage grout with reduced w/c ratio re-evaluate end bearing (clays)	stage grout with reduced w/c ratio
	excessive grout volume visible grout flow around shaft sides	stage grout with reduced w/c ratio	stage grout with reduced w/c ratio
	excessive grout volume visible grout flow out CSL tubes	cap CSL tubes and stage grout	cap CSL tubes and stage grout
	shaft tipped in gravelly soil	plan to stage grout	plan to stage grout

REFERENCES

AASHTO, LRFD Bridge Design Specifications, Customary U.S. Units (2nd Edition), American Association of State Highway and Transportation Officials, Washington, D.C., 1998, *w/1999 interim revisions.*

ASTM D-1143-98, (1998), "Standard Test Method For Piles Under Static Axial Compressive Load," *Annual Book of ASTM Standards*, Part 20, Philadelphia, PA.

Brown, D. and Drew, C. (2000). "Axial capacity of auger displacement piles at Auburn University." GeoDenver 2000 'New Technological and Design Developments in Deep Foundations', Geotechnical Special Publication, No. 100, pp.397-403, Reston, VA.

Brown, D.A. and Vinson, J. (1998). "Comparison of strength and stiffness parameters for a Piedmont residual soil." *Geotechnical Site Characterizations*, Proceedings of the First International Conference on Site Characterization, Atlanta, GA, April.

Bruce, D. A., Nufer, P. J., and Triplett, R. E. (1995). " Enhancement of Caisson Capacity by Micro-Fine Cement Grouting - a Recent Case History" ASCE Special Publication 57, Verification of Geotechnical Grouting.

Carter, J.P. and Kulhawy, F.H. (1987). "Analysis and Design of Foundations Socketed into Rock." Research Report 1493-4, Geotechnical Engineering Group, Cornell University, Ithaca, New York.

Dapp, S. D. (2002). "Pressure Grouting of Drilled Shafts in Sands." Ph.D. Dissertation, University of South Florida, Tampa, Florida.

Dapp, S. and Mullins, G. (2002). "Pressure-Grouting Drilled Shaft Tips: Full-Scale Research Investigation for Silty and Shelly Sands." *Deep Foundations 2002: An International Perspective on Theory, Design, Construction, and Performance,* ASCE Geo Institute, GSP No. 116, Vol. I, pp. 335-350.

Dapp, S.D. (1998). "Interviews with engineers during load testing on the My Thuan Bridge." Mekong Delta, Vietnam.

Davidson, J.L. and Maultsby, J.P. (1998). "Standard Penetration Test (SPT) Energy Testing." Presentation given to the Florida Department of Transportation, October.

FHWA. (1998). "Load and Resistance Factor Design (LRFD) for Highway Bridge Substructures." U.S. Department of Transportation, Publication No. FHWA HI-98-032.

Flemming, W. G. K. (1993) "The Improvement of Pile Performance by Base Grouting" Proceedings of the Institution of Civil Engineers, London.

Harris, M. (1999). *Sams Teach Yourself Microsoft Excel 2000 Programming in 21 Days*. Sams Publishing, Indianapolis, Indiana.

Kulhawy, F.H. and Mayne, P.W. (1990). "Manual on Estimating Soil Properties for Foundation Design." Electric Power Research Institute, Palo Alto, California.

Mayne, P.W., Brown. D, Vinson, J., Schneider, J.A., and Finke, K.A. (2000). "Site characterization of Piedmont residual soils at the NGES, Opelika, Alabama." A2K Specialty Conference on Performance Verification of Constructed Geotechnical Facilities, Geotechnical Special Publication, No. 93, pp 160-185, Amherst, MA.

McVay, M.C. and Townsend, F.C. (1990). "Design of Socketed Drilled Shafts in Limestone."

Meyerhof, G.G. (1976). "Bearing capacity and settlement of piled foundations." Proceedings of the American Society of Civil Engineers, GT3, pp. 197-228.

Mojabi, M.S., and Duffin, M. J. (1991) "Large Diameter, Rock Socket, Base Grouted Piles in Bristol" Proceedings of the 4th International Conference on Piling and Deep Foundations, Stresa, Italy, April

Mullins, G., Ashmawy, A., and Garbin, E.J. (2003). "The Influence of Water Table in Drilled Shaft Construction - Phase I Final Report," *Draft Report* submitted to Florida Department of Transportation, July.

Mullins, G. and O'Neill, M. (2003). "Pressure Grouting Drilled Shaft Tips - A Full-Scale Load Test Program." Research Report, University of South Florida, Tampa, Florida, May.

Mullins, G., Lewis, C., and Justason, M., (2002). "Advancements in Statnamic Data Regression Techniques," *Deep Foundations 2002: An International Perspective on Theory, Design, Construction, and Performance,* ASCE Geo Institute, GSP No.116, Vol II, pp. 915-930.

Mullins, G., Dapp, S., Fredrerick, E. and Wagner, R. (2001). "Pressure Grouting Drilled Shaft Tips - Phase I Final Report." *Final Report* submitted Florida Department of Transportation, December.

Mullins, G., Dapp, S., and Lai, P. (2000). "New Technological and Design Developments in Deep Foundations, Pressure-Grouting Drilled Shaft Tips in Sands." American Society of Civil Engineers, Denver, Colorado.

Mullins, G. (1999). "Interviews with engineers during load testing on the Taipei Financial Center." Taipei, Taiwan.

O'Neill, M.W. (2002). "Discussion and Closure of 'Side Resistance in Piles and Drilled Shafts." Journal of Geotechnical and Geoenvironmental Engineering, May, Vol. 128, No. 5, pp.446-450.

O'Neill, M.W. and Hassan, K.M. (1994). "Drilled Shafts: Effects of Construction on Performance and Design Criteria." Proceedings of the International Conference on Design and Construction of Deep Foundations, December 1994, Vol. 1, pp. 137-187.

Piccione, M., Carletti, G., and Diamanti, L. (1984) “the Piled Foundations of the Al Gazira Hotel in Cairo” Proceedings of the International Conference on Advances in Piling and Ground Treatment for Foundations, Institution of Civil Engineers, London, UK.

Reese, L.C. and O'Neill, M.W. (1988). "Drilled Shafts: Construction and Design." FHWA, Publication No. HI-88-042.

Schneider, D.I. (1999). *An Introduction to Programming Using Visual Basic 6.0 Fourth Edition*. Prentice Hall, Upper Saddle River, New Jersey.

Sliwinski, Z. J., and Flemming, W. G. K. (1984) “the Integrity and Performance of Bored Piles” Proceedings of the International Conference on Advances in Piling and Ground Treatment for Foundations, Institution of Civil Engineers, London, UK.

Touma, F.T. and Reese, L.C. (1974). "Behavior of Bored Piles in Sand." Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol. 100, No. GT7, pp. 749-761.

Troughton, V. M. and Platis, A.(1989) “The Effects of Changes in Effective Stress on a Base Grouted Pile in Sand” Proceedings of the International Conference on Piling and Deep Foundations, London, UK, May.

Winters, D. (2002). “SUPERSAW Statnamic Analysis Software.” Master’s Thesis, University of South Florida, Tampa, Florida.