Final Report

STATIC AND DYNAMIC FIELD TESTING OF DRILLED SHAFTS: SUGGESTED GUIDELINES ON THEIR USE FOR FDOT STRUCTURES

UF Project No.:	4910-4504-725-12
WPI No.:	BC354-08
Contract No.:	99052794
FDOT No.:	99052794

Submitted to:

Mr. Richard Long
Research Center
Florida Department of Transportation
605 Suwannee Street, MS 30
Tallahassee, FL 32399-0450

Dr. Sastry Putcha, PE
Florida Department of Transportation
605 Suwannee Street, MS 5L
Tallahasseee, FL 32399

Principal Investigator: Michael C. McVay
Co-Principal Investigator: Ralph D. Ellis, Jr.
Researchers: Michael Kim
Juan Villegas
San-Ho Kim
Sam Lee

UNIVERSITY OF FLORIDA

Department of Civil and Coastal Engineering, College of Engineering
365 Weil Hall, P.O. Box 116580, Gainesville, FL 32611-6580

October 2003

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
WPI No. BC354-08		

4. Title and Subtitle	5. Report Date
Static and Dynamic Field Testing of Drilled Shafts: Suggested Guidelines on Their Use for FDOT Structures	July 9, 2003
	6. Performing Organization Code
7. Author(s)	8. Performing Organization Report No.
Drs. McVay, Ellis, Mssrs. M. Kim, J. Villegas, S-H. Kim, S. Lee	4910-4504-725-12
9. Performing Organization Name and Address	10. Work Unit No. (TRAIS)
University of Florida Department of Civil and Coastal Engineering 365 Weil Hall / P.O. Box 116580 Gainesville, FL 32611-6580	11. Contract or Grant No. 99052794, BC354-08
12. Sponsoring Agency Name and Address	13. Type of Report and Period Covered
Florida Department of Transportation Research Management Center 605 Suwannee Street, MS 30 Tallahassee, FL 32301-8064 Program Manager: Dr. Sastry Putcha	Final Report September 1999 – July 2003
	14. Sponsoring Agency Code

15. Supplementary Notes

Prepared in cooperation with the Federal Highway Administration

16. Abstract

The Geotechnical, Construction, and Design information from eleven major FDOT bridge sites involving drilled shaft foundations with field load testing was collected and analyzed. From the 27 Osterberg and 11 Statnamic field tests, it was found that the shaft capacities compared favorably to one another and did not differ by more than the site variability. Also, comparison of measured and predicted (FDOT) unit skin friction in limestone was very favorable as represented by the proposed LRFD resistance factors, ϕ, and ASD factors of safety.

Using the Osterberg field results, which did achieve FDOT failure (elastic shortening plus settlement equal to shaft diameter divided by 30), tip resistance vs. tip displacements for Florida limestone was compared to the FHWA predicted response (O'Neill) implemented in FB-Pier. Agreement was quite good as represented by LRFD resistance factors, ϕ, and ASD factors of safety vs. reliability. However, the predicted response did require an additional rock property, i.e., mass modulus of the rock, which may be obtained from Young Modulus of intact rock samples and average RQD from the nearest borings.

From the construction information, it was identified that the typical cost of an Osterberg or Statnamic test varied from $100,000 to $120,000, depending on shaft size, with a typical test requiring two days to complete.

To assist the design engineer in assessing quantities of field load testing, as well as comparing different foundation types (i.e. drilled shafts, driven piles, etc.), it is proposed that a cost (deep foundation) vs. reliability (or risk) plot be generated for the site. Based on the steepness of the plot, as well as the cost of field load tests ($100,000 to 120,000), an estimate of the number of field load tests may be determined along with acceptable risk or reliability.

17. Key Words	18. Distribution Statement
Drilled Shafts, Osterberg, Statnamic, Skin Friction, End Bearing, LRFD Resistance Factors, Benefit vs. Cost	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA, 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	303	

Form DOT F 1700.7 (8-72)

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1
INTRODUCTION

1.1 GENERAL

Before the 1980s most Florida Department of Transportation (FDOT) bridges were supported on small diameter, driven prestressed concrete piles. With the advent of AAHSTO's ship impact code (i.e., Sunshine Skyway), as well as cost, foundation elements have been getting larger (diameter) and larger.

A foundation element, which has seen tremendous growth in Florida as a result of lateral capacity/cost, is drilled shafts. Diameters from three to ten feet with lengths ranging from twenty to one hundred and fifty feet have been installed in Florida. In the case of Florida's Karst limestone, which varies laterally and vertically, both capacity and construct ability (i.e., drilling vs. driving) of shafts have a number of advantages in limestone over driven piles.

As with any foundation design, AASHTO, FHWA, and FDOT, savings in design through lower F.S., or higher resistance factor, ϕ, through field-testing drilled shafts are recommended. The latter has two benefits: 1) validate design assumptions (i.e., skin friction), and 2) ensure appropriate construction process and control. Unfortunately, there presently exist no guidelines (design or construction) on the number of load tests, their location, or type (conventional, Osterberg, or Statnamic). For instance, prior testing history shows many of one type of test (i.e., six Osterbergs), or multiple types (four Osterbergs and six Statnamic) of tests being performed at a site. Complicating the process, the designer has little information on the cost of load testing versus the risk of shaft failure (i.e., assessing benefit vs. cost) for design. Finally, due to the spatial variability (vertical and horizontal) of the limestone, end-bearing design has been spotty or nominal for shafts.

Recently, the FDOT has implemented Load Resistance Factor Design (LRFD) for drilled shafts. The latter is based on a probabilistic approach and determines the LRFD resistance factor, ϕ, based on the risk (probability) of failure. Since, LRFD resistance factor, ϕ, and Allowable Stress Design (ASD) factor of safety, F.S., are directly related, each may be assessed for different reliabilities. Using LRFD, spatial variability may be handled one of two ways: 1) establishment of standard deviation for all sites, i.e., measured vs. predicted response with the associated COV (coefficient of variation) and LRFD, resistance factor, ϕ determined for the whole state; or 2) based on nearest boring, determine measured vs. predicted response with associated COV, and establish resistance factors and factors of safety for given reliabilities. Both may be employed for modeling skin and tip resistance of drilled shafts founded in Florida Limestone.

Standard practice in deep foundation design and construction involves field-testing. However, since the cost of testing various foundation types is quite variable, it is expected that the number of field-test for any site should vary with foundation type. Also, since the final foundation selection is determined in the design phase based on overall cost, the number of field-tests and associated cost needs to be optimized. To optimize shaft design lengths (i.e., axial loads), an accurate assessment of resistance factors or safety factors with associated risk of failure (including spatial variability), as well as the cost of a drilled shaft per foot, and cost of load testing must be obtained. From the latter, and specific risks or probabilities, with the associated resistance factors or safety factors, different foundation element lengths may be computed for a site. From the computed lengths, and cost per foot of shaft, the total foundation cost vs. risk for a site may be established. From this plot, the design engineer may estimate the number of field load tests for a site. For instance, with a steep cost vs. risk plot, significant economical savings may be viable by using the FDOT or AASHTO reduced safety-factor with field-load test (i.e., F.S. of 2 versus 3.5). In the case of a flat cost vs. risk plot, significant number of field-load tests may not be economically warranted vs. cost of lengthening the shafts.

The study begins with a collection of project information from all of the major FDOT drilled shaft supported bridges (11 sites) over the past five to seven years. The latter includes plans (shaft size, lengths, and loads), load test reports, drilled shaft cost per foot, as well as load test cost (Osterberg and Statnamic) information, along with project schedules from the contractors. The rock laboratory properties were obtained from the plans, load test reports, as well as later samples obtained by the districts and tested at the State Materials Office. Next, all of the measured unit skin friction (T-Z) curves, as well as end bearing (Q-Z) curves for the shafts tested (28) were back computed for the sites. Based on the laboratory strength (qu and qt), and stiffness (Young's Modulus) data, both the average unit skin friction and end bearing were computed from current FDOT design practice. From the latter, the ratio of measured and predicted skin friction and end bearing for all of the load tests were found along with the standard deviation. Using the statistics (bias, and standard deviation), both the LRFD resistance factors, as well as ASD factors of safety were computed for different risk or reliability values. Next, from the FDOT cost records, the contractor cost and construction records, as well as load testing equipment manufacturer costs, the true cost of field load testing along with drilled shaft cost per foot were established. Based on the latter, the benefit-cost analysis for field load testing for each of the eleven investigated sites, as well as the establishment of the approach for future sites was performed.

1.2 BRIEF HISTORY OF DRILLED SHAFTS

Early versions of drilled shafts originated from the need to support higher and heavier buildings in cities such as Chicago, Cleveland, Detroit, and London, where the subsurface conditions consisted of relatively thick layers of medium to soft clays overlying deep glacial till or bedrock. In 1908 hand-dug caissons were replaced by machine excavation equipment, capable of boring a 12-inch diameter holes to a depth of 20 to 40 feet. Hugh B. Williams of Dallas developed the first truck-mounted rigs in 1931, which rapidly progressed the drilled-shaft industry.

Prior to World War II, more economical and faster constructed drilled shaft foundations were possible with the development of large scale, mobile, auger-type and bucket-type, earth-drilling equipment. In the late 1940's and early 1950's, drilling contractors had developed techniques for making larger underreams, larger diameters, and cutting into rocks. Large-diameter, straight shafts founded entirely in clay, which gained most of their support from the side resistance, became common usage in Britain. Many contractors also began introducing casing and drilling mud into boreholes for permeable soils below the water table and for caving soils.

A bridge project in the San Angelo District of Texas is believed to be the first planned use of drilled shafts on a state department of transportation projects (McClelland, 1996). While "drilled shaft" is the term first used in Texas, "drilled caisson" or "drilled pier" is more common in the Midwestern United States.

It wasn't until the late 1950's and early 1960's, that analytical design methods were developed for drilled shaft design. Further extensive full-scale load-testing programs were carried out from the 1960's and into the 1980's. Due to improved design methods and construction procedures, drilled shafts became regarded as a reliable foundation system for highway structures by numerous state DOT's (Reese and O'Neil, 1999). A principle motivation for using drilled shafts over other types of deep foundations (e.g., piles) is that a single large high capacity drilled shaft can replace a group of driven piles with cap, resulting in lower costs.

1.3 PURPOSE AND SCOPE

The FDOT currently considers either drilled shafts or piles as the main foundation elements underneath all bridges. For any foundation, both capacity and construct ability are evaluated during installation through field tests. In the case of drilled shafts and associated load tests, the research was to initiate with the collection of a large database (plans, load test results, soil reports, and as built data) for the major FDOT bridges over the past ten years. From the latter data the following analysis was to be undertaken:

1) Reduce and compare both the unit skin friction and end bearing measured from both Osterberg and Statnamic field tests.
2) Evaluate/improve current FDOT design practices for both unit skin friction and end bearing for drilled shafts installed in Florida Limestone.
3) Develop a procedure to estimate the number of field load tests based on a benefit/cost analysis.

CHAPTER 2
SITE DESCRIPTION

2.1 GENERAL

To complete the purpose and scope of this project, the following information had to be collected: 1) as built design plans, 2) Osterberg & Statnamic load test reports, 3) Geotechnical reports, and 4) construction schedule & cost data. The following eleven (11) bridge projects had the required information.

17th Causeway Bridge, State Job #86180-3522

Acosta Bridge, State Job #72160-3555

Apalachicola River Bridge (SR20), State job #47010-3519/56010-3520

Christa Bridge, State job #70140-3514

Fuller Warren Bridge Replacement Project, State Job #72020-3485/2142478

Gandy Bridge, State Job #10130-3544/7113370

Hillsborough Bridge, State Job #10150-3543/3546

McArthur Bridge, State job 87060-3549

Venetian Causeway (under construction), State job #87000-3601

Victory Bridge, State job #53020-3540

West 47th over Biscayne Water Way, State job #87000-3516

The location of each project is shown in Figure 2.1. All of the projects are located in coastal areas of Florida, and all of the shafts were constructed in Florida limestone. A description of each site along with field and laboratory tests follows.

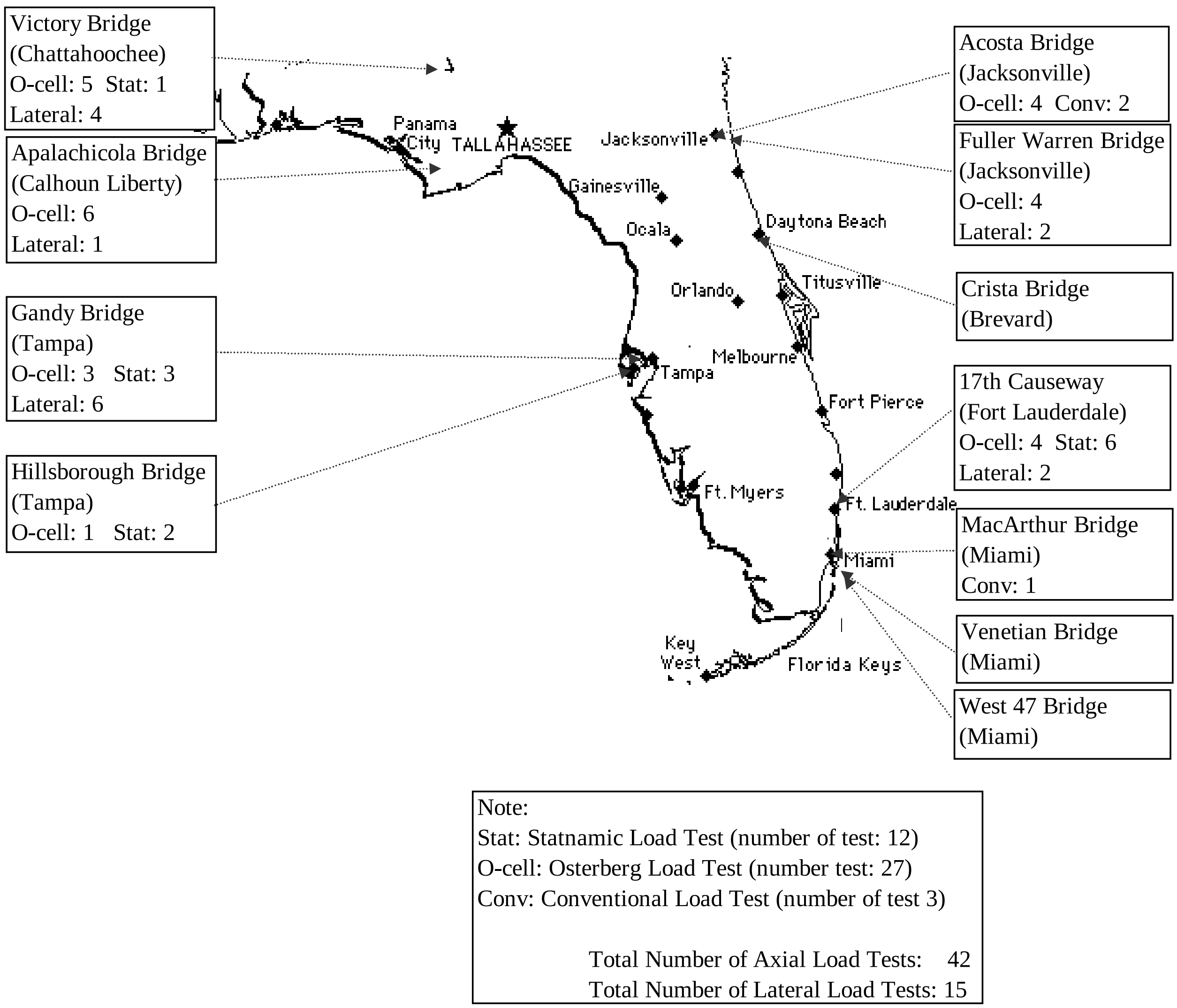


Figure 2.1 Project Locations with Number of Load Tests

2.2 17TH STREET BRIDGE

2.2.1 Site Description

This is a bascule replacement bridge for the old movable bridge on S.E. 17th Street Causeway over the intercoastal waterway in Fort Lauderdale, located in Broward County. The new bascule bridge provides about 16.76 meters of clearance over the navigation channel of the intercoastal waterway when closed.

The construction started on the west end at Station 28+73, which is approximately 127 meters west of the intersection between Eisenhower Boulevard and S.E. 17th Street Causeway. The end of construction was on the east at Station 41+60, which is approximately 540 meters east of the intersection between S.E. 23rd Avenue and S.E. 17th Street Causeway.

2.2.2 General Soil Profile

The general topography on the west end of the S.E. 17th Street project was level outside of the area of the embankments, i.e., elevation in the range of +1.5 to +2.0 meters (NGVD). The project alignment from the west end to the intercoastal waterway, the elevation of the ground surface increases smoothly to elevations of +8 meters (NGVD) in the vicinity of the west abutment of the bridge. The average elevation of the ground surface on the N.W. and S.W. frontage roads ranges from approximately +1.5 to +2.0 meters (NGVD).

As the bridge approaches the intercoastal waterway, the elevation of the bottom of the bay drops smoothly to elevations in the vicinity of 4.6 meters (NGVD). From the Navigation Channel, the elevation of the bottom of the bay increases smoothly until the ground surface is encountered at the east side of the intercoastal Waterway. The average elevation of the ground surface on the N.E. and S.E. Frontage Roads ranges from approximately +1.5 to +2.0 meters (NGVD). The elevation of the project alignment on S.E. 17th Street on the east approach embankment starts approximately at elevations of as much as +6.5 meters (NGVD). As the project alignment proceeds to the east, the ground surface elevation drops to approximate elevations between +1.5 and +2.0 meters around stations 40+50 to 41+00. At this point, the ground surface elevation starts to increase again as the project alignment approaches the Mercedes Bridge on S.E. 17th Street.

2.3 ACOSTA BRIDGE

2.3.1 Site Description

The newly elevated 4-lane Acosta Bridge crosses the St. Johns River in the downtown district of Jacksonville, FL. It replaces a 2-lane lift span bridge (completed in 1921) and carries the Automated Skyway Express (ASE), a light-rail people mover, for the Jacksonville Transportation Authority (JTA).

2.3.2 General Soil Profile

The average elevation of the ground surface of the project ranges from +3.0 to +15.0 feet (NGVD). In the shallow areas of the river crossing, there is a 2’ to 10’ thick layer of sand. This thin upper sand layer is very susceptible to scour. The upper sand layer is underlain by a layer of limestone varying in thickness from 10’ to 20' thick, which is underlain by a thick deposit of overconsolidated sandy marl. The limestone layer is much more resistant to scour.

2.4 APALACHICOLA BRIDGE

2.4.1 Site Description

The Florida Department of Transportation built a new concrete/steel bridge on State Road (SR) 20 over the Apalachicola River between the towns of Bristol and Blountstown in Calhoun County, parallel to the existing 2-lane structure. The existing steel truss bridge was constructed in the 1930's and had been designated as an historic monument. The construction involved building a new 2-lane concrete-steel bridge, and renovating the old bridge. The final crossing consists of two lanes traveling east-west (new bridge) and two lanes traveling west-east (renovated old bridge).

Each of the structures consists of a trestle portion crossing the surrounding flood plain as well as a high-level portion spanning the river itself. The trestle portion of the new structure is

4,464 feet long while the approaches and main span comprise 3,890 feet, resulting in a total structure length of 8,362 feet. The main span provides a vertical clearance of 55 feet from the normal high water level of the river. The river is about 700 feet wide at the crossing.

2.4.2 General Soil Profile

The new bridge alignment runs approximately parallel to the existing structure just to its south. Natural ground surface elevations in the flood plain generally range from about elevation +41 feet to +47 feet on the West Side of the river and from elevation +44 feet to +48 feet on the East Side of the river. Mud line elevations of the bridge at the new pier locations ranged from about + 17 feet to + 18 feet in the river. According to the project plans, the mean low river water elevation was + 32. 0 feet and the normal high river water elevation was +46.5 feet.

Subsurface stratigraphy consists of soft to very stiff sandy clays, sandy silts, along with some clayey sands of 10 to 20 feet in thickness underlain by sands to silty clayey sands ranging in density from loose to dense with thickness from a few feet to a maximum of 30 feet. Beneath the sands, calcareous silts, clays, sands and gravels, with layers of inter bedded limestone, generally extend from about elevation zero feet to about elevation -50 feet to -60 feet. The calcareous material is limestone that is weathered to varying degrees. While the upper 10 to 15 feet of the material generally ranges from stiff to medium dense, it transitions to dense and strong limestone with increasing depth. At approximately elevation -50 feet to -60 feet, very well cemented calcareous clayey silt with sand is encountered that extended to elevation -65 feet to -75 feet. This material is generally underlain by very hard limestone that extends to the maximum depth of 135 feet (elevation -94 feet).

2.5 FULLER WARREN BRIDGE

2.5.1 Site Description

The new Fuller Warren Bridge replaces the old Gilmore Street Bridge in Jacksonville, Florida. The new bridge spans Interstate Highway 95 (I-95) across the St. Johns River in downtown Jacksonville. The old bridge was a four-lane concrete structure with steel, drawbridge bascule extending across the channel. The new concrete high span bridge has a total of eight travel lanes and was constructed parallel to the old bridge, 120 feet offset to the south.

2.5.2 General Soil Profile

The average elevation of the ground surface for this project ranges from +4.0 to +20 feet. Overburden soils are encountered from the surface elevations down to the limestone formation at elevations -12 to -27 feet. The overburden soils generally consist of very loose to very dense fine sands with layers of clayey fine sands and/or layers of very soft clay. A variably cemented sandy limestone formation is encountered between elevations of -12 to -45 feet (MSL). The limestone formation is typically 10 to 20 feet thick.

2.6 GANDY BRIDGE

2.6.1 Site Description

The Gandy Bridge consists of two double lane structures across Old Tampa Bay between Pinellas County to the west and Hillsborough County to the east in west central Florida. The new bridge replaces the westbound structure of the existing Gandy Bridge across Old Tampa Bay. Age, deterioration, and other factors associated with the old bridge warranted its replacement.

2.6.2 General Soil Profile

The average elevation of the ground surface of the project ranges from +0.0 to -22 feet. The surface soils consist of approximately 45 feet of fine shelly sand and silt. Underlying the sands and silts are highly weathered limestone. The limestone is encountered at depths varying from 58 to 65 feet below existing grade. The elevation of the top of the limestone varies from approximately -4 feet (NGVD) to -53 feet (NGVD) along the axis of the bridge across the bay. Four-inch rock cores were taken in selected borings. The recovered rock samples are generally tan white shelly calcareous slightly phosphatic limestone, with chert fragments. Much of the limestone is weathered and due to the solution process has pockets of silts and clays embedded in the matrix.

2.7 HILLSBOROUGH BRIDGE

2.7.1 Site Description

The project consisted of the construction of a new bridge, as well as the rehabilitation of the existing bridge on State Road 600 (Hillsborough Avenue) across the Hillsborough River in northern Tampa. The old bridge was designed and constructed in the late 1930's. It was 358 linear feet long with a 93.5-foot vertical lift span. The four 10-foot traffic lanes were not able to accommodate the heavy traffic. Due to its historic significance, the old bridge was identified as a historic monument and was rehabilitated. The new structure is 436 linear feet in length with a vertical bascule-type moveable span lift.

2.7.2 General Soil Profile

The average elevation of the ground surface of the project ranges from +1.8 to -10.5 feet, with the limestone formation between elevations -15 to -40 feet. The overburden soils ranged from a very loose/dense fine sands to clayey fine sands. A variably cemented limestone formation is encountered between elevations of -15 to -40 feet. The limestone formation is typically 10 to 50 feet thick over the site.

2.8 MACARTHUR BRIDGE

2.8.1 Site Description

The former MacArthur Causeway Draw Bridge served as one of a few means of transit between Miami and Miami Beach. Due to traffic congestion, especially when the drawbridge was up, the Florida Department of Transportation elected to construct a new high-level fixed span bridge to improve traffic conditions.

The new bridge begins at Station 1039+00 (interstate I-395) and extended eastward along the MacArthur Causeway, to Station 225 + 80 (Watson Island).

The west approach of the existing bridge is located within a man-made fill area adjacent to Biscayne Bay. The east approach is located on a partially man-made fill area. Watson Island is hydraulically fill material obtained from both Turning Basin and the Port of Miami main channel.

2.8.2 General Soil Profile

The average elevation of the ground surface of the project ranges from +2.0 to -11 feet. The overburden soils generally occurred down to an elevation of -12 to -27 feet (MSL), which is the top of the limestone formation. The overburden soils generally consist of very loose to very dense fine sands or clayey fine sands with zones of very soft clay. The limestone was highly variable, cemented and sandy, as well as fossiliferous. The limestone formation is typically 10 to 20 feet thick.

2.9 VENETIAN CAUSEWAY BRIDGE

2.9.1 Site Description

Spanning some 2-1/2 miles, and joining 11 islands, the Venetian Causeway is an important link between the cities of Miami and Miami Beach, in Dade County. The Causeway serves as an evacuation route for residents of Miami Beach and the islands during a hurricane. The existing Causeway includes some 12 bridges and is open to 2-lane traffic with one sidewalk running along the north site. The existing roadway was completed in 1926, is 36 feet wide with a 4-foot sidewalk, and is on the National Register of Historic Places.

2.9.2 General Soil Profile

The elevation of the bottom of Biscayne Bay ranges from -1.4 to -10.3 feet. The upper soils consist mostly of sands down to an elevation -10 to -20 feet. Beneath the sands is a transition zone of limestone and calcareous sandstone layers, frequently combined with pockets of sands down to an elevation -28 to -31 feet. Underlying this are harder layers of limestone and calcareous sandstone layers with sporadic sand pockets down to an elevation -31 to -55 feet.

2.10 VICTORY BRIDGE

2.10.1 Site Description

The Victory Bridge crosses the flood plain of the Apalachicola River about one mile west of Chattahoochee on U.S. 90. The Jim Woodruff dam is located approximately 0.6 miles north (upstream) of the bridge. The original bridge was completed shortly after the end of World War I and is supported on steel H-piles. The bridge was subsequently designated as an historic structure to prevent its demolition. The new bridge is located approximately 50 feet south of the old bridge and supported on drilled shafts.

2.10.2 General Soil Profile

The soil profile at the Victory Bridge is quite variable, ranging from silt and clay to sand with shell over limestone. The ground surface occurs at an elevation of +48 to +58 feet with weathered limestone at surface at some locations. The overburden soils generally consist of very loose to very dense fine sands and clayey fine sands with some zones of very soft clay. A cemented limestone formation is encountered between elevations of +40 to –20 feet. The limestone formation was typically 10 to 50 feet thick.

CHAPTER 3
DESCRIPTION OF FIELD LOAD TESTING

3.1 GENERAL

Load tests are generally performed for two reasons: 1) as a proof test for design verification, i.e., ensure that the test shaft is capable of sustaining the design loads; and 2) validate that the contractor's construction approach is appropriate. Generally, the shafts are instrumented (strain gauges along length of shaft) to assess skin and tip resistances for various shaft head displacements.

It is critical that the test shaft be founded in the same formation and by the same construction procedures as the production shafts. Generally more than one load test is scheduled for a major bridge projects.

According to the FDOT, the failure of a drilled shaft is defined as either 1) plunging of the drilled shaft, or 2) a gross settlement, uplift or lateral deflection of 1/30 of the shaft diameter in an axial loading test.

Until the nineteen eighties, the only feasible way of performing a compressive load test on a drilled shaft was through conventional method, or large reaction frames. Unfortunately, the conventional methods had a limited capacity (approx. 1500 tons, Justason et al., 1998) and require significant installation and testing time. Recently, two new alternative methods for field testing drilled shaft load tests have been developed which don't require a reaction system. Moreover, these latter methods (Osterberg and Statnamic load tests) have higher capacity (approx. 3000 to 6000 tons) and require less setup time than the conventional load tests, which makes them less expensive than conventional testing.

While the Osterberg test is a statically loaded system, the Statnamic is considered to be a semi-dynamic system. The following section describes the difference between static, Statnamic, and dynamic load testing followed by sections describing each test in detail.

3.2 COMPARISON of AXIAL LOAD TESTING: STATIC, STATNAMIC, and DYNAMIC LOAD TESTING

The main differences between static, Statnamic, and dynamic load testing (i.e., pile driving) can be seen from the comparisons of stresses, velocities and displacements along the pile/shaft. The comparisons between these factors are shown Figure 3.1 (Middendorp and Bermingham, 1995).

For the dynamic loading, a short duration impact is introduced to the pile head by a drop hammer or a pile-driving hammer (shown in Figure 3.1). A stress wave travels along the pile resulting in large differences in stresses from pile level to pile level. While some pile levels experience compression, other pile levels experience tension. This pattern is constantly fluctuating during the test. The same pattern occurs in the velocities and the displacements along the pile.

In Statnamic load testing, the load is gradually introduced to the pile (shown in Figure 3.1). Compression stresses change gradually along the pile, and all pile parts remain under compression. From the top of the pile/shaft down, the skin resistance of the soil reduces the compression stresses. All points along the pile move at almost same velocities, and displacements change gradually along the length of the pile/shaft.

In static load testing, the load is introduced to the pile in successive steps (shown in Figure 3.1). Each step is maintained over a period of time ranging from minutes to hours. Compression stresses change gradually along the pile and all pile segments remain under

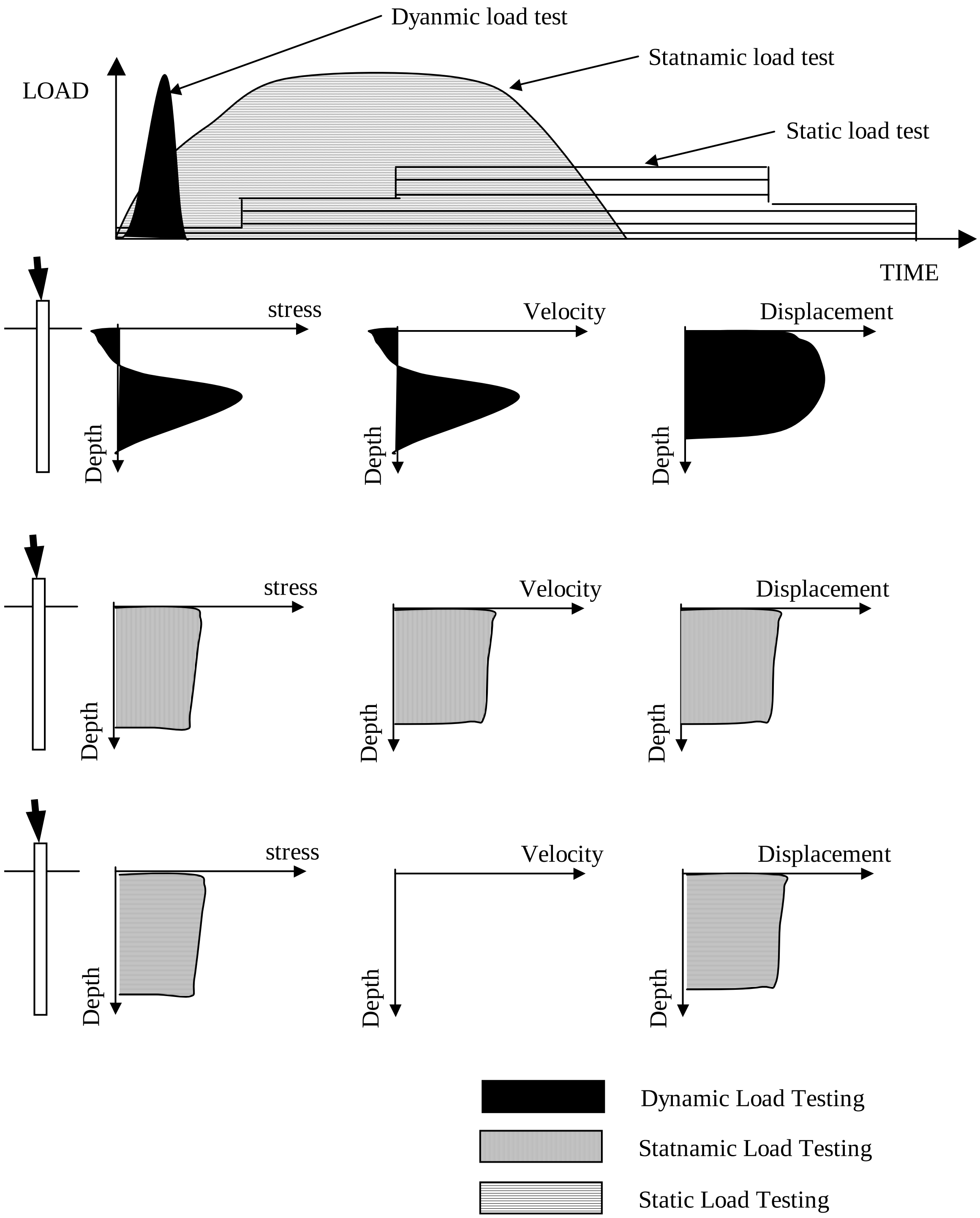


Figure 3.1 Comparison of Stresses, Velocities and Displacements for Dynamic, Statnamic, and Static Load Testing (after Middendorp and Bielefeld, 1995)

compression. Along the length of the pile/shaft, the skin resistance reduces the compression stresses. All pile segments move with almost zero velocity, and displacements change gradually along the pile/shaft.

Evident from Figure 3.1, the Statnamic test is closer to conventional static load testing than the dynamic load test (i.e., pile driving). The major difference between the Statnamic and static load testing is the velocities. While the velocities are considered close to zero for the static test, they can be in the range of 0.1 to 2 m/s for the Statnamic test. The latter results in the consideration of both inertia, and damping forces when estimating static resistance from the Statnamic results (Middendorp et al., 1992, Matsumoto and Tsuzuki, 1994).

The following sections briefly explain each axial test (Conventional, Osterberg, and Statnamic load tests).

3.3 CONVENTIONAL LOAD TESTING

Static load is applied to the top of the shaft by means of a hydraulic jack. Several arrangements can be used to support the jack, including reaction piles/shafts, load platforms, or high-strength anchors. The most frequently used arrangement is the reaction shafts. The load is applied with a hydraulic ram against a reaction beam supported by two reaction pile/shaft spaced generally five diameters from the test shaft. A typical setup is shown in Figure 3.2. The ASTM D 1143-81 procedure for static slow testing is usually followed.

The load is applied in successive load increments/steps. Each step is maintained over a period of minutes to hours (generally 10 minutes). In every step, the load, settlement, and time are recorded. The test continues until a settlement of at least 5 percent of the base diameter is achieved or the shaft plunges with no additional load applied (Reese and O’Neil, 1999).

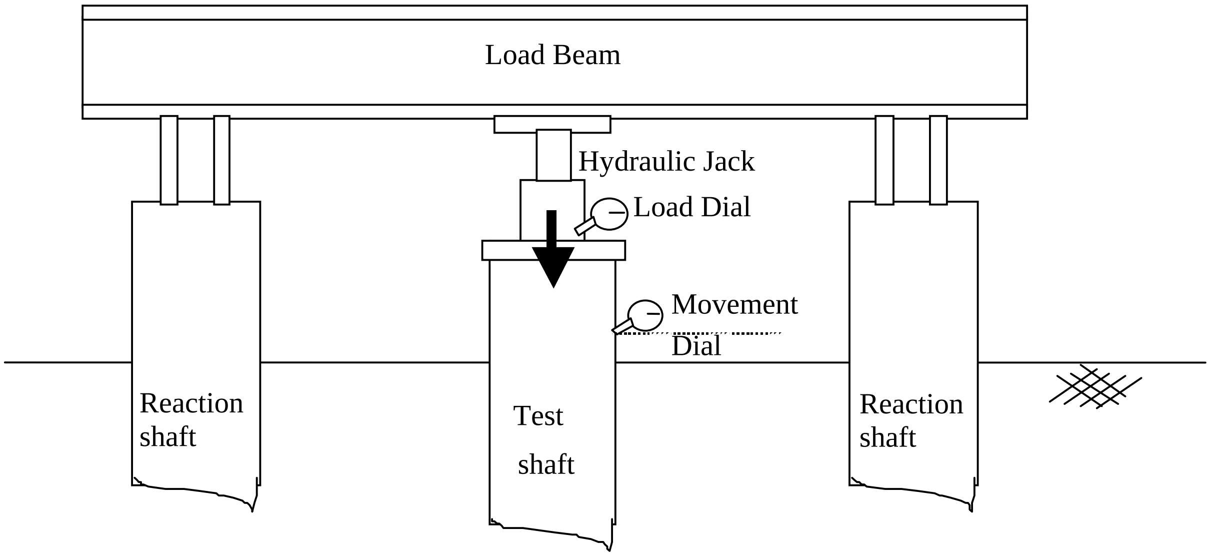


Figure 3.2 Schematic of Typical Conventional Load Test

The advantage of the static load test is that it simulates the conventional dead loading. However, the disadvantages of the static test are:

1. Reaction shafts spaced closer that five diameters impact test results.
2. The reaction frame and reaction anchors are significant structures.
3. The maximum capacities are limited, generally limited to 1000 tons.
4. The standard procedure might take several days to complete.
5. The test is generally more expensive than the Osterberg or Statnamic load tests.

3.4 OSTERBERG LOAD TESTING

The Osterberg cell, developed by Osterberg (1989), is basically a hydraulic jack that is cast into a shaft. Since the O-cell (Osterberg cell) can produce up to 3,500 tons of force acting in both the upward and downward directions, the Osterberg cell automatically separates the skin friction from the bearing resistance. As mentioned above, the Osterberg test does not need a conventional jack, reaction frame or reaction anchor system. As a result, the Osterberg test requires much less time to complete than a conventional test. A schematic diagram of the Osterberg cell loading system is shown in Figure 3.3.

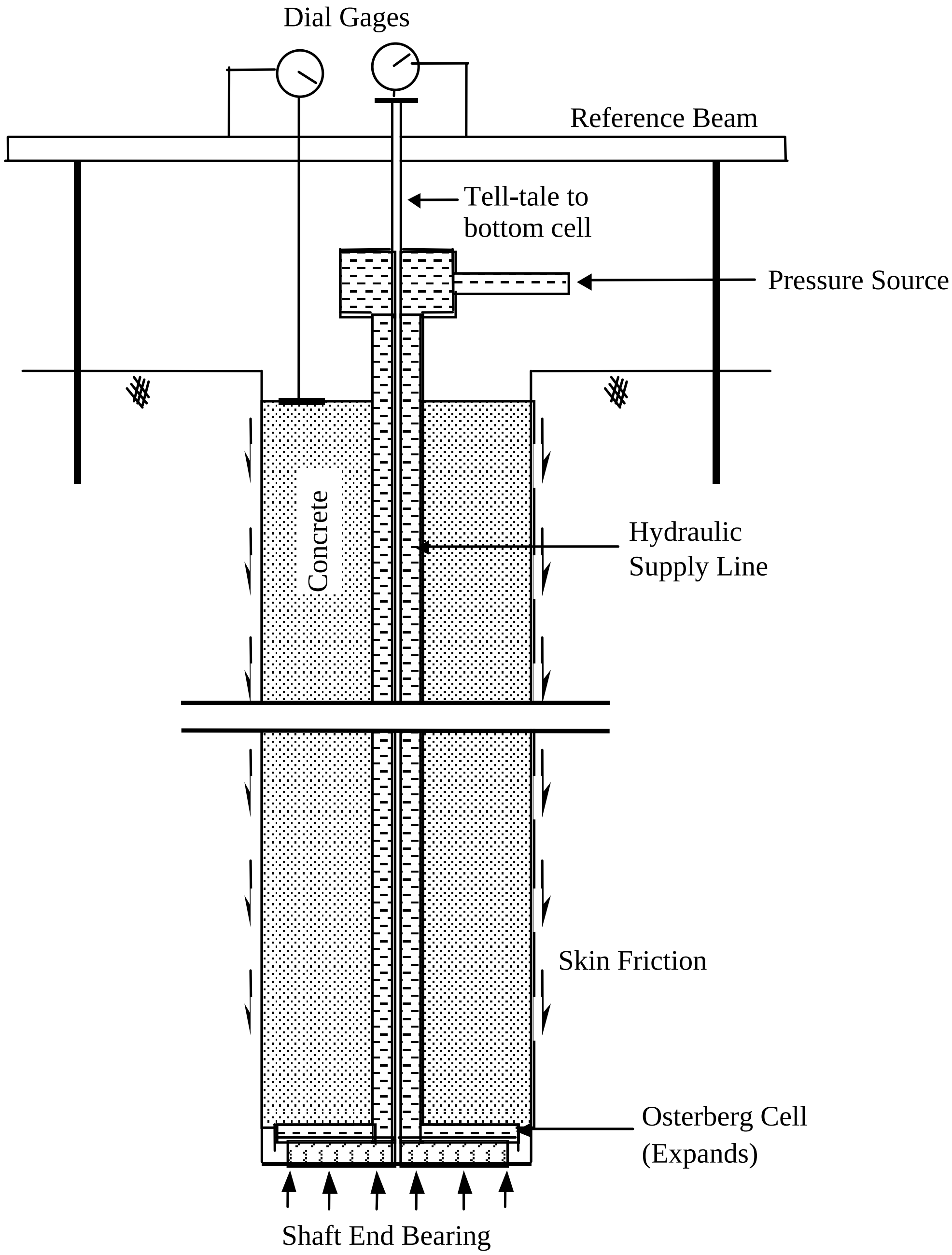


Figure 3.3 Schematic of Osterberg Load Test (after Reese and O'Neil, 1999)

The cell, Figure 3.3, consists of two plates of prescribed diameter. Between the plates, there is an expandable chamber that can hold pressurized fluid. The upper and lower plates on the cell can be field welded to the steel plates. The diameters of the steel plates are approximately equal to that of the test shaft. The Osterberg cell is calibrated in a test frame so that the load versus applied pressure relationship is obtained. When the load is applied to the cell, the load is equally distributed at both top and bottom.

Dial gauges connected to telltales measure the movement of top cell and bottom cell. Electronic LVDT has been used to measure vertical movement between the top and bottom plates. With such an arrangement, it is possible to obtain relations of side resistance versus side movement and base resistance versus base movement until either the base or side resistance reach failure.

Generally, test shafts are also instrumented with pairs of strain gauges, placed from just above the top of the load cell to the ground surface. By analyzing load distribution from the stain gauges, the load transfer can be calculated for the various soil and rock layers.

End bearing provides reaction for the skin friction, and skin friction provides reaction for the end bearing. This unique interrelationship makes the placement of the cell critical. If the cell were to be placed too high (see Figure 3.4), the shaft would most likely fail in skin friction above the O-cell. If the O-cell is placed too deep in the shaft, the portion of shaft below the cell will likewise fail early. If either occurs too soon, the information about the other is incomplete. As a consequence, it is not easy to get both the ultimate side and tip resistances with just one Osterberg cell. If only the ultimate tip resistance is desired, the cell should be installed at the bottom of the shaft. On the other hand, if the ultimate side resistance is needed, the cell should be installed upward from the tip of the shaft.

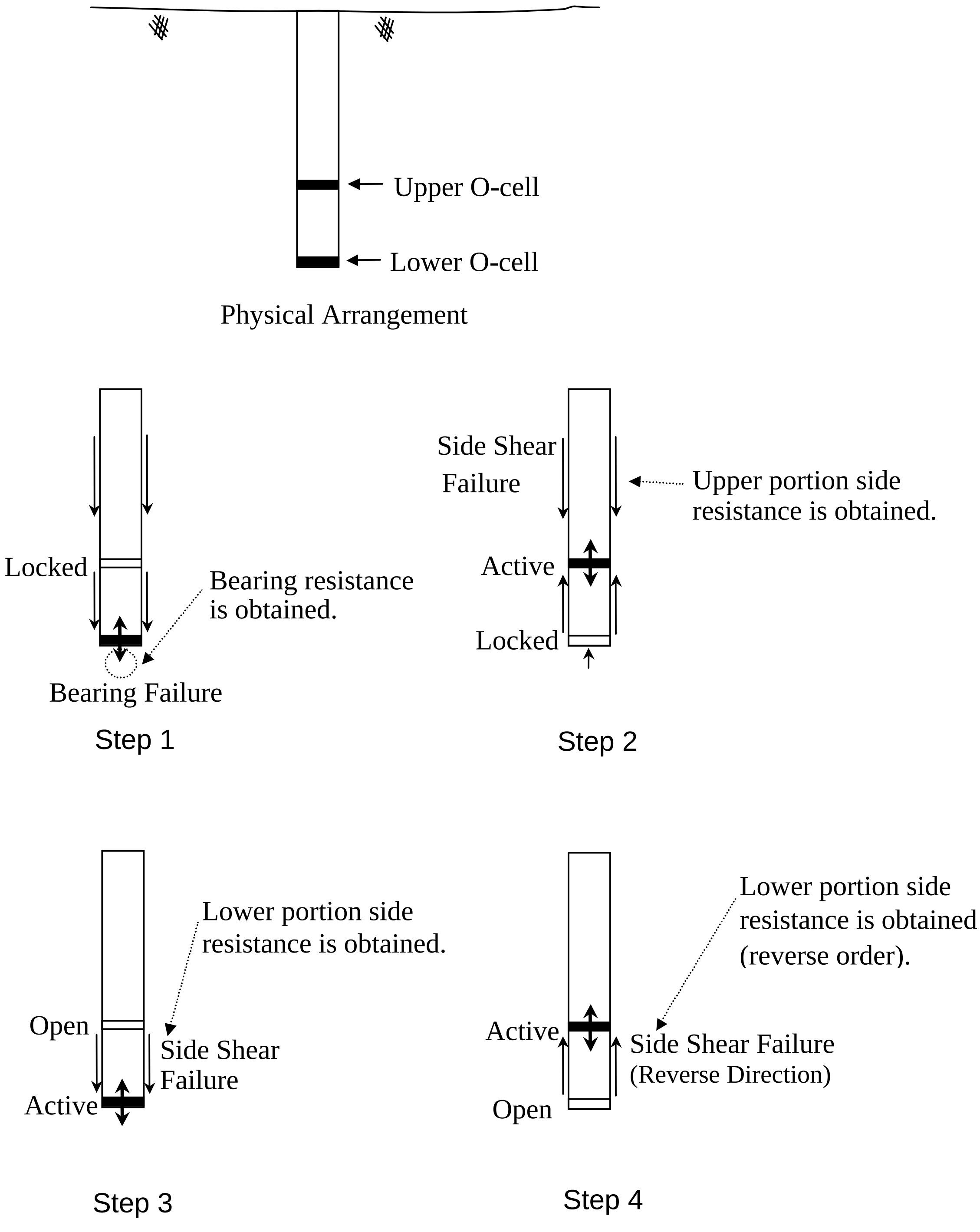


Figure 3.4 Multi-level Osterberg Testing Setup Configuration (after Reese and O'Neil, 1999)

Osterberg tests are typically performed in accordance with ASTM D1143 (Quick Load Test Procedures). The loads are applied during each stage in increments of 5% of the estimated maximum applied load. The shafts are unloaded in increments of about 25% of the maximum applied load.

Numerous other configurations are possible including a multi-level setup (see Figure 3.4: used in Apalachicola Bridge and Fuller Warren Bridge), which is capable of fully mobilizing both side and tip resistances. Nine thousand tons of combined side and base resistances have been achieved with this arrangement. Obviously, this configuration permits significantly higher loads than the conventional test.

In summary, the Osterberg load test provides the following advantages when compared to the conventional static load test and the Statnamic load test:

1. Requires no external frames and is constructed in conjunction with the shaft, reducing setup time.
2. Multiple load cells placed at the base of the shaft can be used to test shafts to capacities above 9000 tons.
3. Load cells placed at different levels in the shaft can be used to test both end bearing and skin friction, separately.
4. Ability to reload test shafts to obtain residual side shear strength.
5. Ability to perform on shafts in water (river or channel).
6. Ability to perform on inclined piles or shafts on land or over water.
7. Cost effective compared to Conventional test (about 50 – 60% for situations in which conventional loading tests can be used)

The Osterberg test has the following disadvantages:

1. Single cell tests generally fail by either mobilizing the full skin friction or end bearing which limits the information on the other component.
2. Since the shaft is being pushed upward, several physical effects may be different as compared to conventional loads.

3.5 STATNAMIC LOAD TESTING

Statnamic load testing was jointly developed in both Canada and the Netherlands (Middendorp et al., 1992) to measure both skin and end bearing resistances simultaneously. Statnamic devices have been constructed capable of applying loads in excess of four thousand tons. The cost of a Statnamic test is usually comparative to an Osterberg cell for similar load magnitudes.

The principle of the Statnamic test is shown in Figure 3.5. Dead weights (reaction masses) are placed upon the surface of the test shaft. Fuel propellants and a load cell are placed underneath the dead weights. The solid fuel within the combustion chamber burns, developing gas pressure, which act downward against the shaft and upward against the dead weights (reaction masses). The pressure acting on top of the shaft, induce a load-displacement response, which is measured with laser and load cell devices. The pre-determined load is controlled by the size of the reaction mass and quantity of propellant. The duration of the applied load is typically 120 to 250 milliseconds. Pile/shaft acceleration and velocity are typically on the order of 1g and 1m/s respectively. Displacement is monitored directly using a laser datum and an integrated receiver located at the center axis of the pile/shaft. In addition, displacement may be calculated by integrating the acceleration measured at the top of the shaft. Force is monitored directly using a calibrated load cell.

A Statnamic graph of settlement (movement) versus load is shown in Figure 3.5. Since there are some dynamic forces (i.e., damping and inertia), some analysis is necessary. Currently, the Unloading Point Method (UPM) (Middendorp et al., 1992) is the standard tool for assessing the damping inertial forces and determining the static capacity as shown in Figure 3.5.

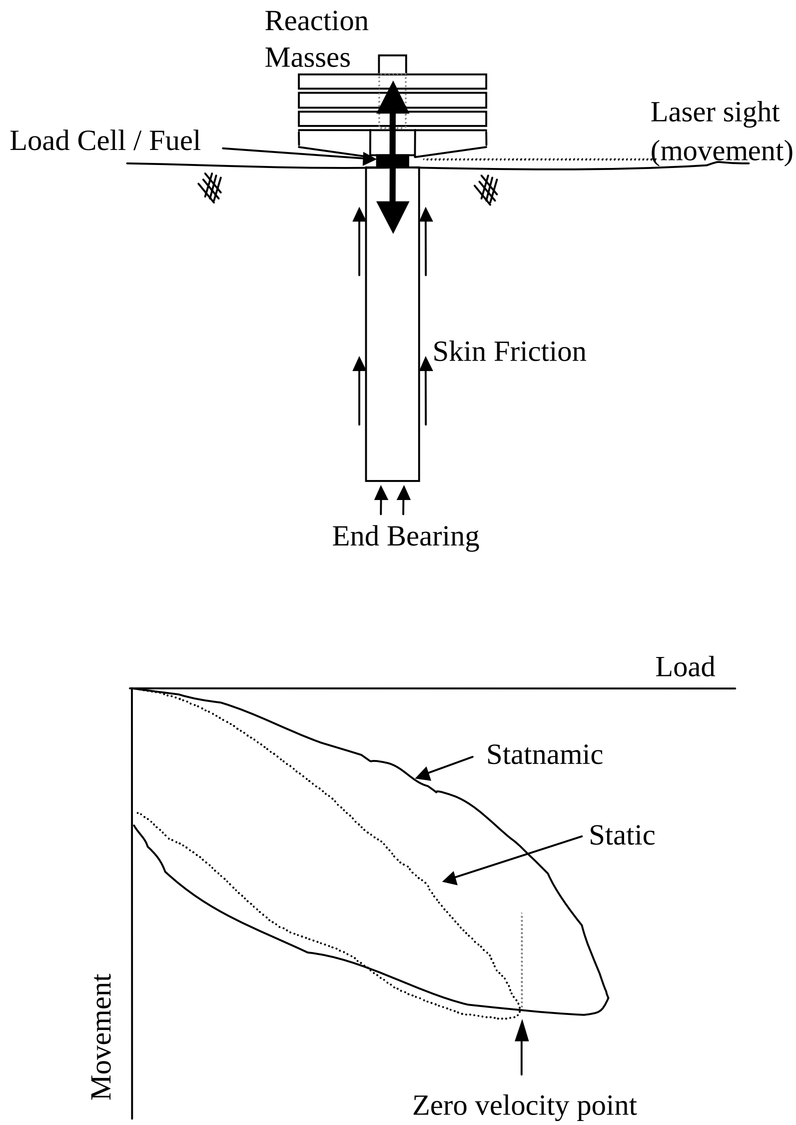


Figure 3.5 Schematic of Statnamic Load Test

The UPM analysis consists of two parts: 1) determination of static resistance at maximum displacement (Unloading Point), and 2) construction of the static load-displacement diagram. The following describes the step-by-step procedures for the UPM method. The Schematic of Unloading point method is shown in Figure 3.6.

1. It is assumed that the pile can be modeled as a Single Degree of Freedom System due to the long duration of Statnamic loading.

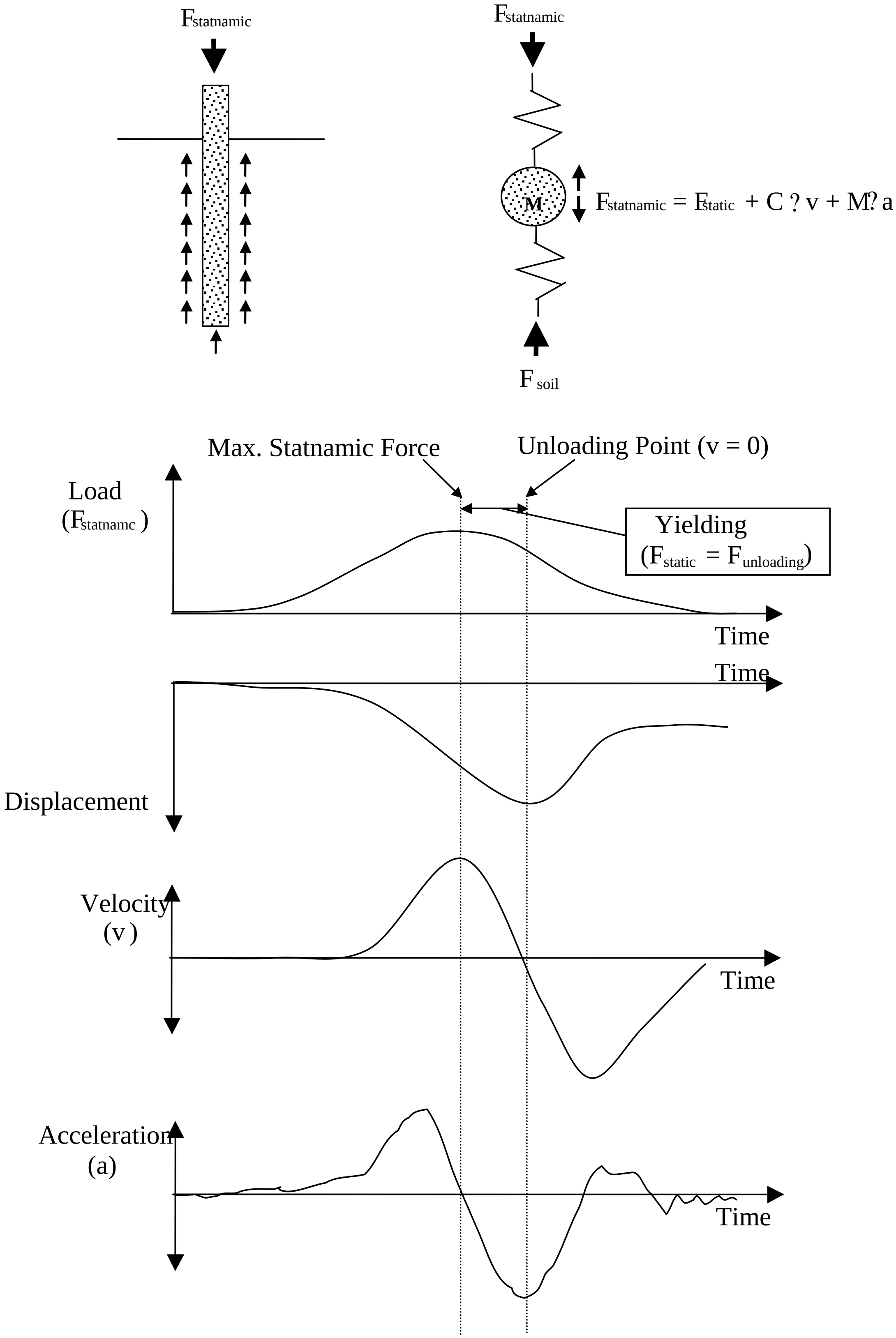


Figure 3.6 Schematic of Unloading Point Method (after Middendorp et al., 1992)

2. $F_{statnamic} = F_{static} + C\times v + M\times a$ (C: damping factor, v: velocity, M: mass, a: acceleration, therefore $C\times v$: damping force, $M\times a$: inertial force), see Figure 3.6.
3. At the maximum displacement (Unloading Point), the velocity equals zero (v = 0).
4. At the Unloading Point, since $F_{statnamic}$ and the acceleration are known (measured by devices), F_{static} can be calculated.

   $F_{static} = F_{statnamic} – M\times a$, (v = 0 at the Unloading Point)
5. It is assumed that the soil is yielding over the range $F_{statnamic(max)}$ to $F_{unloading}$, so $F_{static} = F_{unloading}$.

   $F_{unloading} = F_{statnamic} – M\times a$ -------------------------------- from step 4
6. Since F_{static} is known ($F_{static} = F_{unloading}$) over the range $F_{statnamic(max)}$ to $F_{unloading}$, a mean C (damping factor) can be calculated over this range.

   $C\times v = (F_{statnamic} - F_{static} - M\times a)$ ----------------------------- from step 2

   $C_{mean} = (F_{statnamic} – F_{unloading} - M\times a) / v$ -------------------- from step 5
7. Now static resistance F_{static} can be calculated at all points.

   $F_{static} = (F_{statnamic} - C_{mean}\times v - M\times a)$
8. Draw the static load-movement diagram, shown in Figure 3.5.

In summary, the Statnamic load test provides the following advantages when compared to both conventional and Osterberg load testing:

1. Propellants are safe and a reliable way to produce a predetermined test load of desired duration. Loading is repeatable and is unaffected by weather, temperature, or humidity.
2. Since the Statnamic requires no equipment to be cast in the shaft, it can be performed on a drilled shaft for which a loading test was not originally planned.
3. The Statnamic device can be reused on multiple piles/shafts.
4. The Statnamic test has little or no effect on the integrity of the shaft (non-destructive).
5. The Statnamic is a top-down test simulating a real load case, while the Osterberg generates an up-lifting force.

The main disadvantage of the Statnamic test is its dynamic nature and the need to assess the dynamic forces (inertia and damping) that are developed during the test. The dynamic forces can be computed using the Unloading Point Method (UPM) (Middendorp et al., 1992). AFT (Applied Foundation Testing, Inc.) has recently developed the segmental approach (Segmental

Unloading Point Method) based on variable instrumentation placed along the side of the pile/shaft. The derived static loads presented in this report were calculated using the UPM method since this method was the only available method at the time of testing.

3.6 LATERAL LOAD TESTING

Large diameter drilled shafts, are especially good at resisting lateral forces (hurricanes, ship impact, etc.). Generally, lateral load tests on are performed to validate design, as well as assessing tip cut-off elevation. Hydraulic jacks (conventional), Osterberg cells, or Statnamic devices can be used to apply the lateral load. For lateral load testing, the size of the pile/shaft influences the depth load is transferred (i.e., test production size pile/shafts). The pile/shaft are cast with pairs of strain gauges along their length from which moments and shears can be computed. From the latter, the soil's resistance (p-y curves) can be computed (i.e., change in shear between any two points give p) to validate design.

Since the data collected (Figure 2.1) includes only conventional lateral load tests, the latter will be briefly described. The two common configurations are shown in Figure 3.7, and are conducted by either pushing two piles against each other or by pulling two piles/shafts toward each other. The load is applied by either a jack that pushes (compression) two piles or by a jack that pulls (tension) two test piles/shafts using an attached cable or tie. The applied load is measured through a load cell, which is installed near the jack.

The nature of loading employed in the loading test should duplicate the loading in service as closely as possible. For example, if the primary design is static, the applied load should be increased slowly. If the primary loadings were wind loading, one-way cyclic loading would be appropriate. If the primary loading is wave or seismic loading, two-way cyclic loading may be appropriate. Repeatedly pushing and pulling the shaft through its initial position can simulate

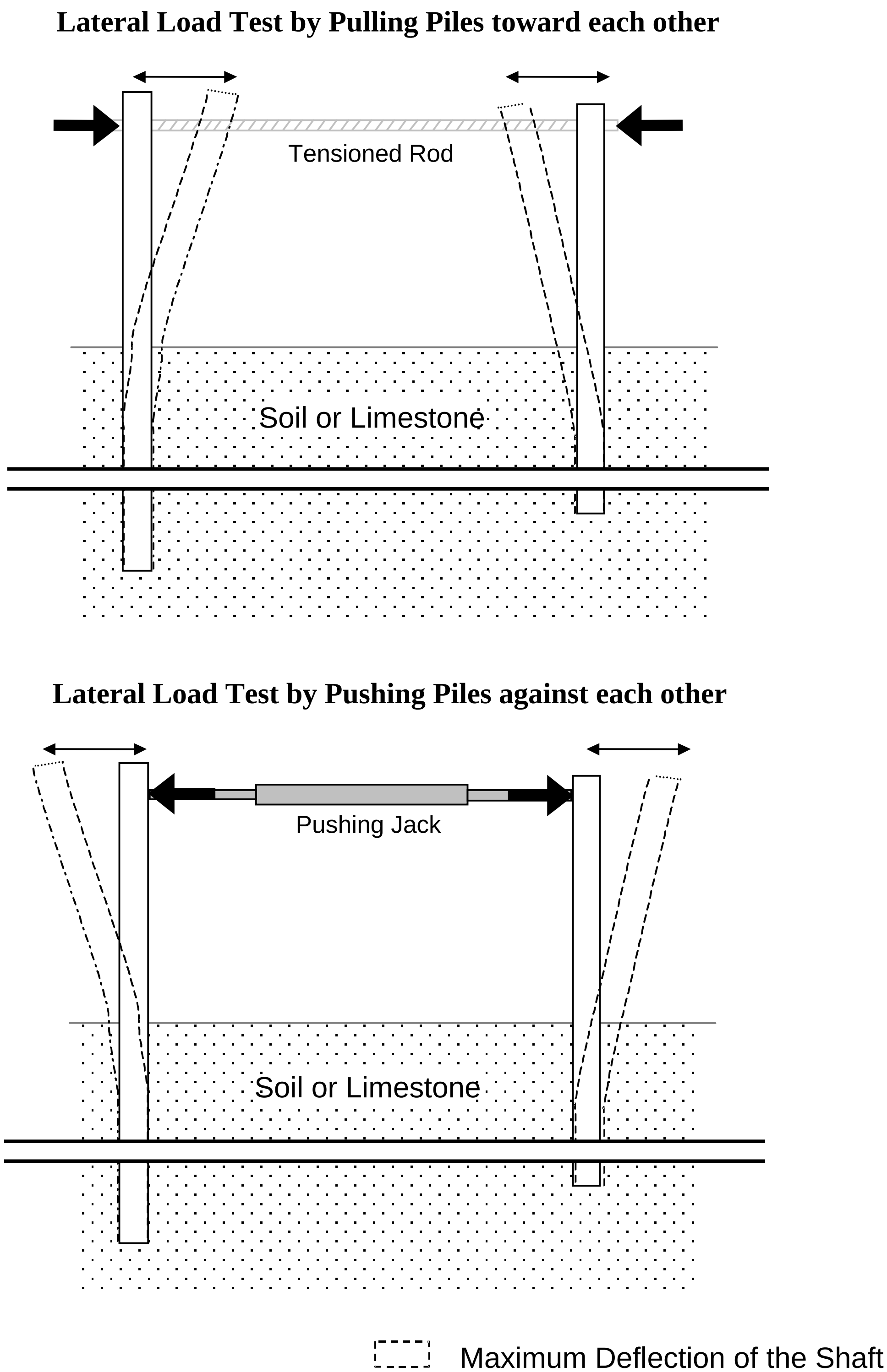


Figure 3.7 Schematic of Conventional Lateral Load Test

the two-way cyclic load. The latter would be used to develop cyclic p-y curves, which are known to degrade with cycles.

CHAPTER 4
MEASURED SKIN and TIP RESISTANCE ANALYSIS

4.1 GENERAL

A total of forty shafts were analyzed: 25 Osterberg, 12 Statnamic and 3 Conventional tests. As shown in Figure 2.1, all the projects are located near coastal areas of Florida. All the reported data were obtained from the load test reports sent to the FDOT by the consultants who performed the tests.

Since the failure states for both skin and end bearing resistance were generally unknown, the T-Z curves (skin friction vs. displacement) and Q-Z curves (tip resistance vs. tip displacement) had to be generated. Based on T-Z and Q-Z curves, the ultimate (where displacement become excessive without increasing load) and mobilized (skin friction and end bearing less than ultimate) skin friction could be established.

For Osterberg tests, the skin friction distribution along the shaft was evaluated using strain gauge data, while the end bearing was evaluated using the load-movement response at the O-cell's bottom plate, Figure 4.1. To evaluate skin friction along the shaft, the measured strain was used to calculate the axial loads at each gauge's elevation based on Hook's Law:

$$\sigma = E\varepsilon \qquad \text{(Eq. 4.1)}$$

$$P = AE\varepsilon \text{ ------- from } P = A\times\sigma \qquad \text{(Eq. 4.2)}$$

$$fs = \Delta P/(\Delta L\times\pi D) \qquad \text{(Eq. 4.3)}$$

where:

σ: compressive stress
P: compressive load
A: cross sectional area of the pile
E: elastic modulus of the pile material

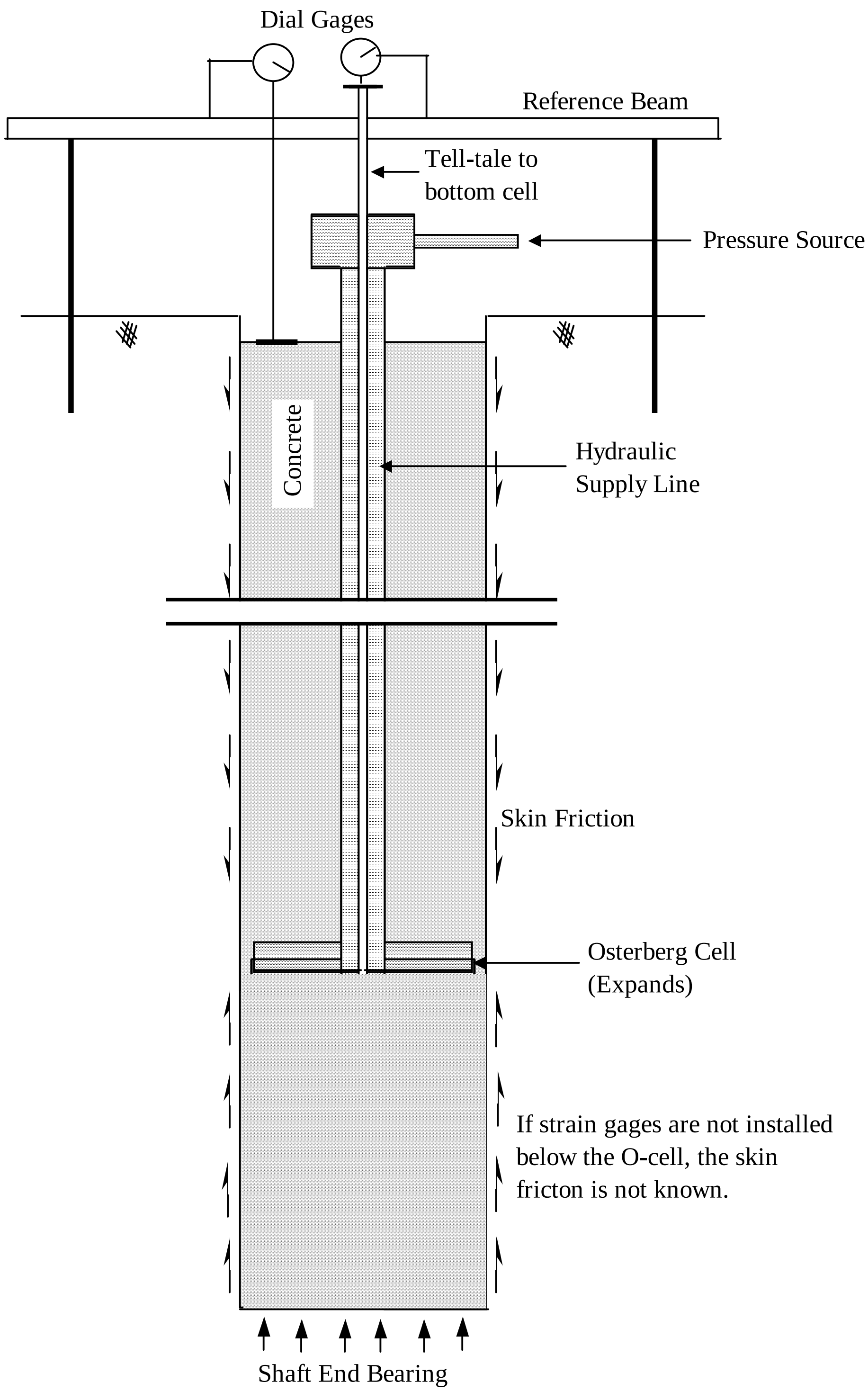


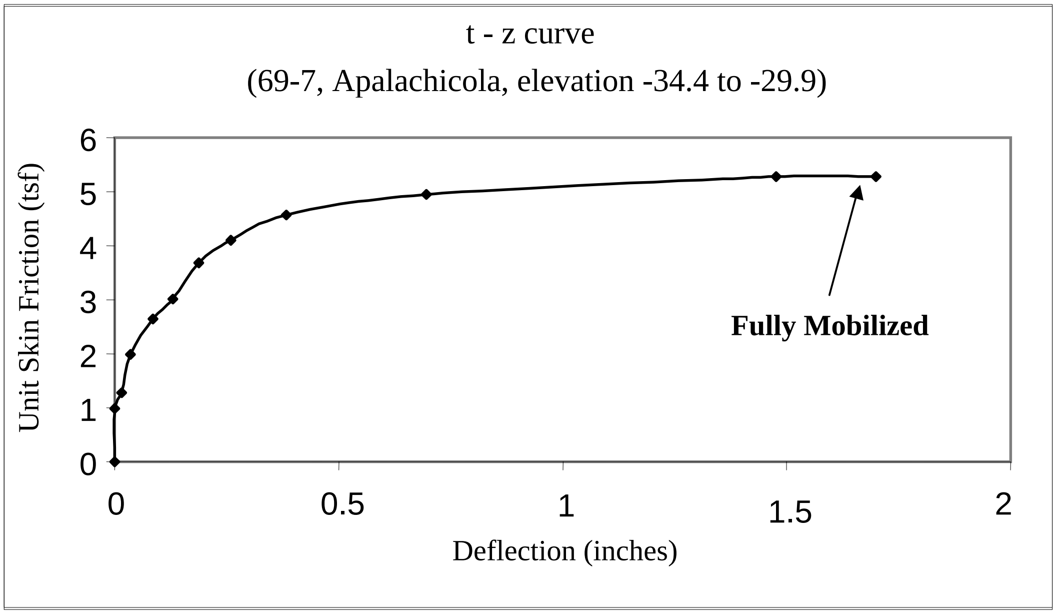
Figure 4.1 Osterberg Setup When the O-cell is Installed Above the Tip

ε: compressive strain
ΔL: distance between two adjacent strain gauges
ΔP: load difference between two adjacent strain gauges
fs: unit skin friction between two adjacent strain gauges

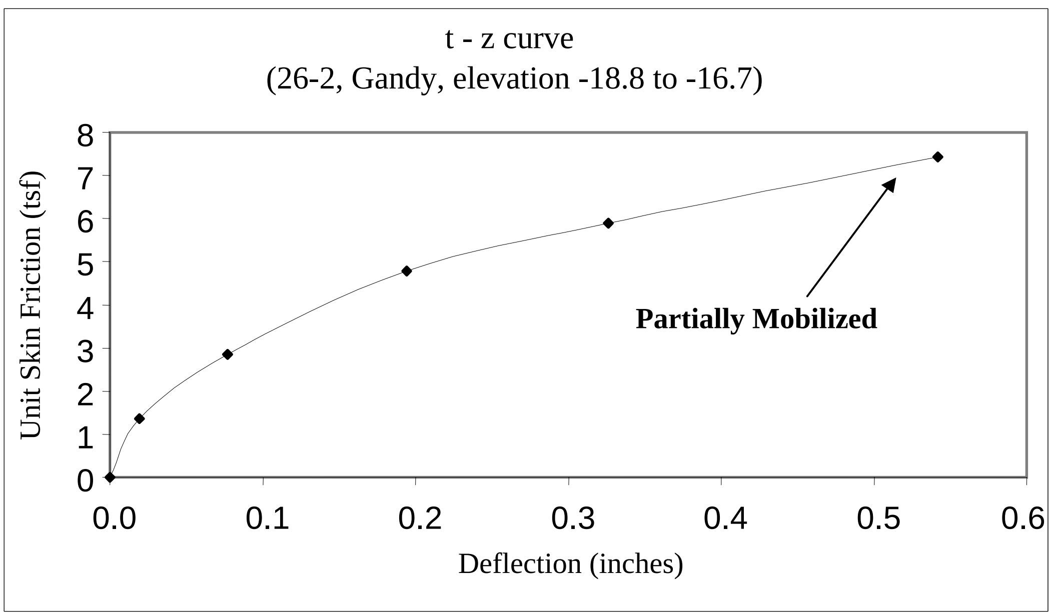
The load transferred to the soil between any two gauges is the difference in the compressive loads between the two gauges. The unit skin friction, i.e., T in a T-Z curve is obtained by dividing the transferred load by the surface area of the shaft within the two gauges' locations. As shown in Equations 4.2 and 4.3, the calculated unit skin friction is dependent on the diameter and modulus of elasticity of the shaft. The assumed diameter was determined from the measured concrete volume used to construct the shaft. The modulus of elasticity for the shaft was calculated in the portion of the shaft above the ground surface using a composite area of both the steel and concrete in the shaft. The composite modulus of elasticity used for the equation was generally about 4,000,000 psi.

Typical skin friction T-Z curves are shown in Figure 4.2. As seen in the top figure, an ultimate skin friction is found where the curve flattens. However, in the lower figure, the shaft has not settled enough for the latter to occur, and the mobilized skin friction is recorded, which is less than the ultimate skin friction.

For the end bearing, failure was usually defined using the FDOT settlement criteria (i.e., FDOT defines failure as settlement equal to 1/30 of the diameter of the shaft), which may occur prior to excessive settlement. However some shaft tip settlements never reached the FDOT value, others were loaded beyond the FDOT criterion. Consequently, the following different unit end bearing categories were assigned: mobilized (settlement less than 1/30 of shaft diameter), FDOT failure (settlement 1/30 of shaft diameter), and Maximum failure (settlement larger than 1/30 of shaft diameter). The difference between them is shown in Figure 4.3. The end bearing for Osterberg was calculated using the following equation:

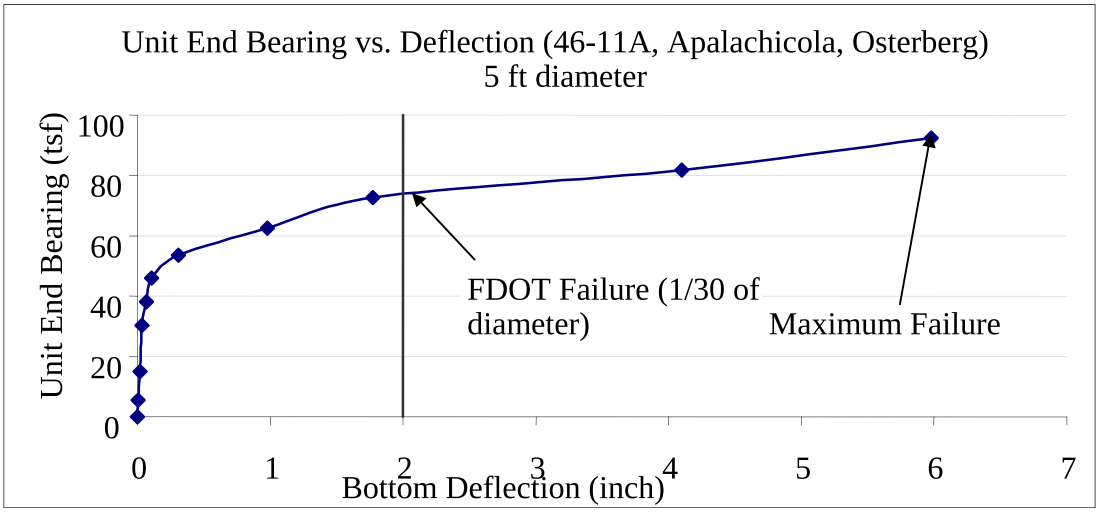


An example of fully mobilized t -z curve



An example of partially mobilized t -z curve

Figure 4.2 Examples of Fully and Partially Mobilized Skin Frictions



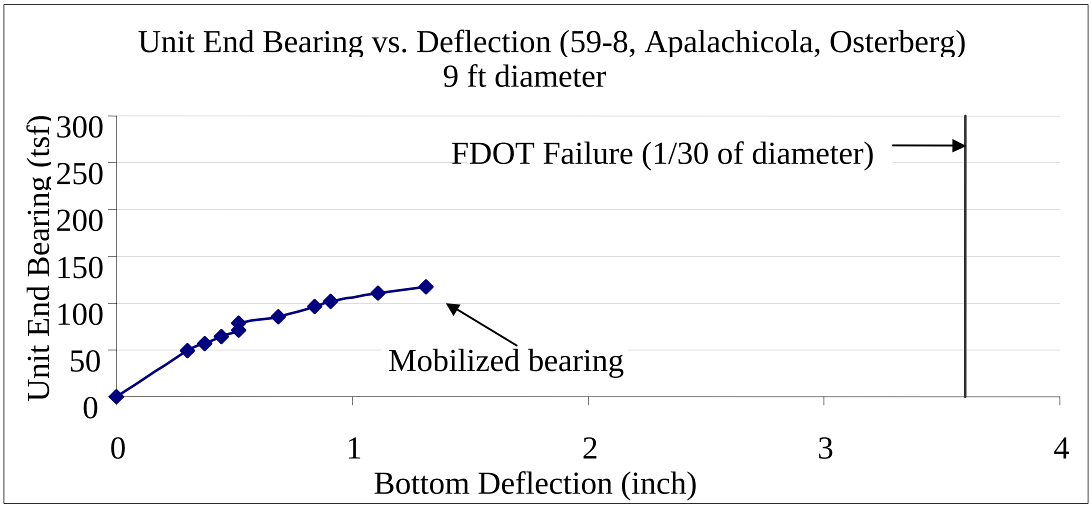


Figure 4.3 Examples of Mobilized, FDOT Failure, and Maximum End Bearing

$$Qs = P / A \qquad \text{(Eq. 4.4)}$$

Qs: unit end bearing

P: applied load at O-cell

A: cross sectional shaft area at the tip

Note: the above equation is only valid when the O-cell is installed at the tip of the shaft.

If there is a certain distance between the O-cell and the tip of the shaft (noted as Unknown Friction in Table 4.1) as displayed in Figure 4.1, the skin friction values for the zone beneath the Osterberg cell must be computed (only possible when extra stain gauges are installed beneath the O-cell) or it must be assumed to be equal to the skin friction value above the O-cell.

Table 4.1 Summary of Unit End Bearing from Osterberg Load Tests

	Shaft Name	Shaft Length (ft)	*Unknown Friction (ft)	Tip Movement (in)	Failure Status	Mobilized Bearing (tsf)	FDOT Failure (tsf)	Maximum Failure (tsf)
17th Street Bridge	LTSO 1	119.4	5.2	0.624	Both	x	x	x
	LTSO 2	142.0	9.1	1.95	Tip Failure	x	x	x
	LTSO 3	100.1	11.1	1.89	Both	41.5	x	x
	LTSO 4	77.5	2.6	3.53	Tip Failure	x	x	66.4
Acosta Bridge	Test 1	64.2	0	4.41	Tip Failure	x	61.7	90.3
	Test 2	101.2	0	2.97	Tip Failure	x	28	39
	Test 4	113.9	0	3.2	Tip Failure	x	22.4	30.2
	Test 5A	87.8	0	5.577	Tip Failure	x	18.5	29.4
Apalachicola Bridge	46-11A	85.0	0	5.977	Both	x	72.6	92
	53-2	72.0	0	2.1	Both	70	x	x
	57-10	84.0	0	1.7	Both	60**	x	x
	59-8	134.0	9	1.3	Both	65	x	x
	62-5	89.2	0	2.69	Both	x	38	40
	69-7	99.1	0	4.46	Both	x	36	44
Fuller Warren Bridge	LT-1	41.0	0	0.23	Skin Failure	87	x	x
	LT-2	27.9	0	2.56	Both	x	80.8	89.5
	LT-3a	120.7	0	2.94	Both	x	34	34
	LT-4	66.8	0	3.12	Both	x	54	70
Gandy Bridge	26-2	38.4	9.8	0.4	Skin Failure	x	x	x
	52-4	54.5	4.33	2.9	Both	x	139.2	x
	91-4	74.7	6.7	2.5	Both	x	42.9	x
Hillsborough Bridge	4-14	70.8	7.33	1.74	Both	x	x	x
Victory Bridge	3-1	33.2	0	0.5	Both	109	x	x
	3-2	38.6	9.66	0.4	Skin Failure	x	x	x
	10-2	46.6	7.7	2.367	Both	x	45	x
	19-1	45.0	0	0.528	Both	124.4	x	x
	19-2	50.7	12.14	0.4	Skin Failure	x	x	x

Note:
1) *Unknow Friction: Distance from the tip to the lowest strain gage.(Side skin friction is unknown)
2) **: Ultimate unit end bearing due to plunging.
3) x: Not determined.
4) Shaft Length: Distance from the top to the tip of the shaft.
5) Failure Status: Both means that the shaft fails in both side and end resistance.
6) Mobilized: The mobilized unit end bearing when the bottom movement is less than 1/30 of the shaft diameter.
7) FDOT Failure: The unit end bearing when the bottom movement is 1/30 of the shaft diameter.
8) Maximum Failure: The unit end bearing when the bottom movement is larger than 1/30 of the shaft diameter.

The load at the tip can be calculated using the following equation:

$$Qs = (P - fs \times A_{fs}) / A \quad \text{(Eq. 4.5)}$$

Qs: unit end bearing

P: applied load at O-cell

fs: assumed or calculated skin friction below the O-cell

A_{fs}: surface area of the shaft below the O-cell

A: cross sectional shaft area at the tip

The unit end bearings for Osterberg and Statnamic tests are summarized in Tables 4.1 and 4.2. A summary of test type, location, dimensions, elevations and other configurations are provided in Appendix A. Appendix B tabulates corresponding unit skin frictions, Appendix C shows the generated T-Z curves for each level along the shaft, and Appendix D tabulates the information used in lateral load tests.

Table 4.2 Summary of Unit End Bearing from Statnamic Load Tests

	Shaft Name	Shaft Length (ft)	Max Statnamic Load (ton)	Derived Static Load (ton)	Top Movement (in)	Middle Rock Movement (in)	Tip Movement (in)	Skin Failure Status	Mobilized Bearing (tsf)	FDOT Failure (tsf)
17th Street Bridge	LTSO 1	119.4	3450	3912	0.87	0.27	0.22	81%	52.6	x
	LTSO 2	142.0	3428	3765	0.94	0.58	0.39	90%	2.6	x
	LTSO 3	100.1	3512	1353	1.26	0.78	0.73	95%	27	x
	LT 1	72.1	3718	2665	1.49	1.27	1.04	100%	65	x
	LT 2	61.3	3584	3765	0.55	0.19	0.11	79%	0	x
Gandy Bridge	26-1	33.4	3375	3307	0.53	0.44	0.42	87%	46.9	x
	52-3	55.6	3430	3372	0.52	0.46	0.44	88%	x	x
	91-3	70.7	3436	3068	0.74	0.65	0.62	92%	103.5	x
Hillsborough Bridge	4-14	70.8	3287	3125	1.63	1.24	1.16	100%	x	x
	5-10	75.3	3243	3573	0.57	0.22	0.11	80%	x	x
Victory Bridge	TH 5	43.1	3600	3430	0.54	0.28	0.25	83%	45	x

Note:
1) x: Not determined.
2) Derived Static Load: Equivalent static soil resistance calculated using the Unloading Point Mehod (UPM).
3) Top Movement: The movement at the top of the shaft.
4) Middle Rock Movement: The movement at the middle of the rock socket.
5) Failure Status: Percentage of ultimate skin failure based on the "Normalized T-Z Curve" in Figure 4.6.
6) Mobilized: The mobilized unit end bearing when the bottom movement is less than 1/30 of the shaft diameter.
7) FDOT Failure: The unit end bearing when the bottom movement is 1/30 of the shaft diameter.

4.2 STATIC (OSTERBERG & CONVENTIONAL) LOAD TESTING

4.2.1 General

A total of 27 Osterberg and 3 conventional load tests were analyzed from the 42 axial load tests (conventional, Osterberg and Statnamic tests) collected. All the projects are located in coastal areas of Florida, and all the shafts were installed in Florida limestone. Of the 27 Osterberg tests, 10 failed in either skin or end bearing resistance, the other 17 tests failed in both skin and end bearing resistances. Table 4.1 shows how each shaft failed.

Q-Z (tip resistance vs. tip displacement) curves are generated to analyze the unit end bearing for each shaft. When the O-cell was located at the tip of shaft, the end bearing resistance was easily calculated: the applied load on the O-cell divided by the area of the bottom. However, when the O-cell was located a certain distance from the tip, the end bearings could not be calculated unless the skin frictions below the O-cell was assumed (see Figure 4.1). For those cases which were not instrumented below the O-cell, the unit skin friction above the cell were used to separate out skin from tip resistance for O-cell data.

Unit skin frictions (ultimate and mobilized when not failed) are computed along the shaft and displayed for each project. A brief discussion about test results for each site follows.

4.2.2 17th Street Bridge

A total of four Osterberg tests were performed at 17th Street Bridge: Pier 6, Shaft 10 (LTSO1); Pier 7, Shaft 3 (LTSO 2); Pier 5, Shaft 3 (LTSO 3); and Pier 10, Shaft 1 (LTSO 4). All four shafts were constructed using the “wet method” with water. As described below, all load-tested shafts were four feet in diameter.

A breakdown by Pier, Shaft tested, and type of test is shown below:

Pier	Shaft	Type of Test
Pier 6 (*4ft. diameter*)	Shaft 10 (LTSO1)	Osterberg & Statnamic
Pier 7 (*4ft. diameter*)	Shaft 3 (LTSO2)	Osterberg & Statnamic
Pier 5 (*4ft. diameter*)	Shaft 3 (LTSO3)	Osterberg & Statnamic
Pier 10 (*4ft. diameter*)	Shaft 1 (LTSO4)	Osterberg & Statnamic
Pier 2 (*4ft. diameter*)	Shaft 1 (LT1)	Statnamic
Pier 8 (*4ft. diameter*)	Shaft 3 (LT2)	Statnamic

From the test results, LTSO1 and LTSO3 failed in both side and end resistances, whereas, LTSO2 and LTSO4 failed in end bearing. Since almost no movement was measured above the O-cell in LTSO2 and LTSO 4, the information regarding unit skin friction was considered of minimal use. Even though all four Osterberg tests failed in end bearing (LTSO 1 and LTSO 3 failed in both skin and tip), the unit end bearing was difficult to assess. This is due to the unknown magnitude of side friction below the Osterberg cell (no instrumentation to back compute skin friction below the O-cell). When the O-cell is located above the tip of the shaft (shown in Figure 4.1), it is assumed that the side resistance below the cell is equal to the unit resistance just above the cell. This side resistance below the O-cell was subtracted (force) from the applied load in the O-cell since the applied load consists of side and end bearing resistances. For comparison purposes, the end bearing values obtained from the load test report is presented in Table 4.1.

Based on both the Geotechnical report and the drilled shaft boring logs, it was observed that the elevation of the top of the limestone formation varied considerably within the site. The skin friction also varied considerably along the lengths of the shafts and from one shaft to another. The variability of the limestone formation and unit skin friction along the shafts is shown in Figure 4.4. Since the strength of the limestone was very different on each side of the channel, the site was divided into soft limestone and hard limestone areas. The soft limestone area includes LT1 (Statnamic) and LTSO3 (Statnamic and Osterberg), and the hard limestone

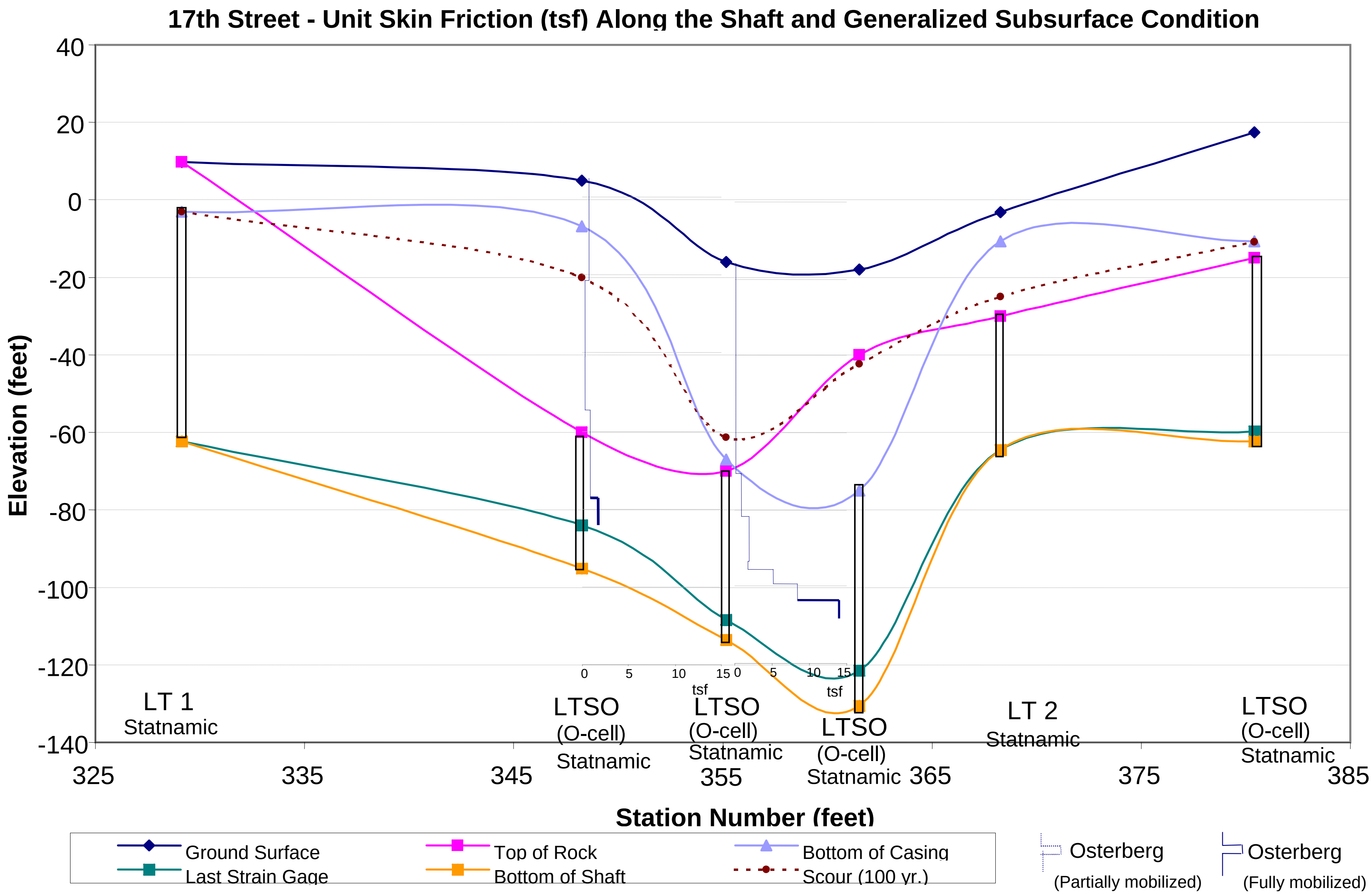


Figure 4.4 Unit Skin Friction Along Each Shaft for Osterberg Load Test, 17th Bridge

area includes the remaining load test shafts. Note that both ultimate and mobilized unit skin frictions are presented in solid lines and dashed lines, respectively.

4.2.3 Acosta Bridge

A total of four Osterberg tests (provided by Schmertmann & Crapps, Inc.) were performed at the site: Test 1, 2, 4, and 5A. All of the tested shafts were three feet in diameter, and all failed in bearing with at least 3 inches of bottom movement.

A breakdown by Pier, Shaft tested, and type of test is shown below:

Test Site 1	(*3 ft. diameter*)	Osterberg
Test Site 2	(*3 ft. diameter*)	Osterberg & Conventional
Test Site 4	(*3 ft. diameter*)	Osterberg & Conventional
Test Site 5A	(*3 ft. diameter*)	Osteberg

Test Sites 2 and 4 had conventional top down tests performed after running the Osterberg test on each shaft. For these conventional tests, the tests were conducted with the Osterberg cells open to eliminate end-bearing resistance in an attempt to mobilize the full skin friction. For the conventional top down tests, 250 tons were applied. The applied load was capable of fully mobilizing the skin resistance for Test 2, but not for Test 4.

Unfortunately, none of the Osterberg tests mobilized the ultimate skin friction on the shafts, all of the Osterberg tests reached failures in end bearing. Only Test 2 was very close to the ultimate skin resistance as reported in the company's report for the conventional top down test. The variability of the top of limestone formation and computed unit skin frictions (mobilized and ultimate) along each of the shafts are shown in Figure 4.5.

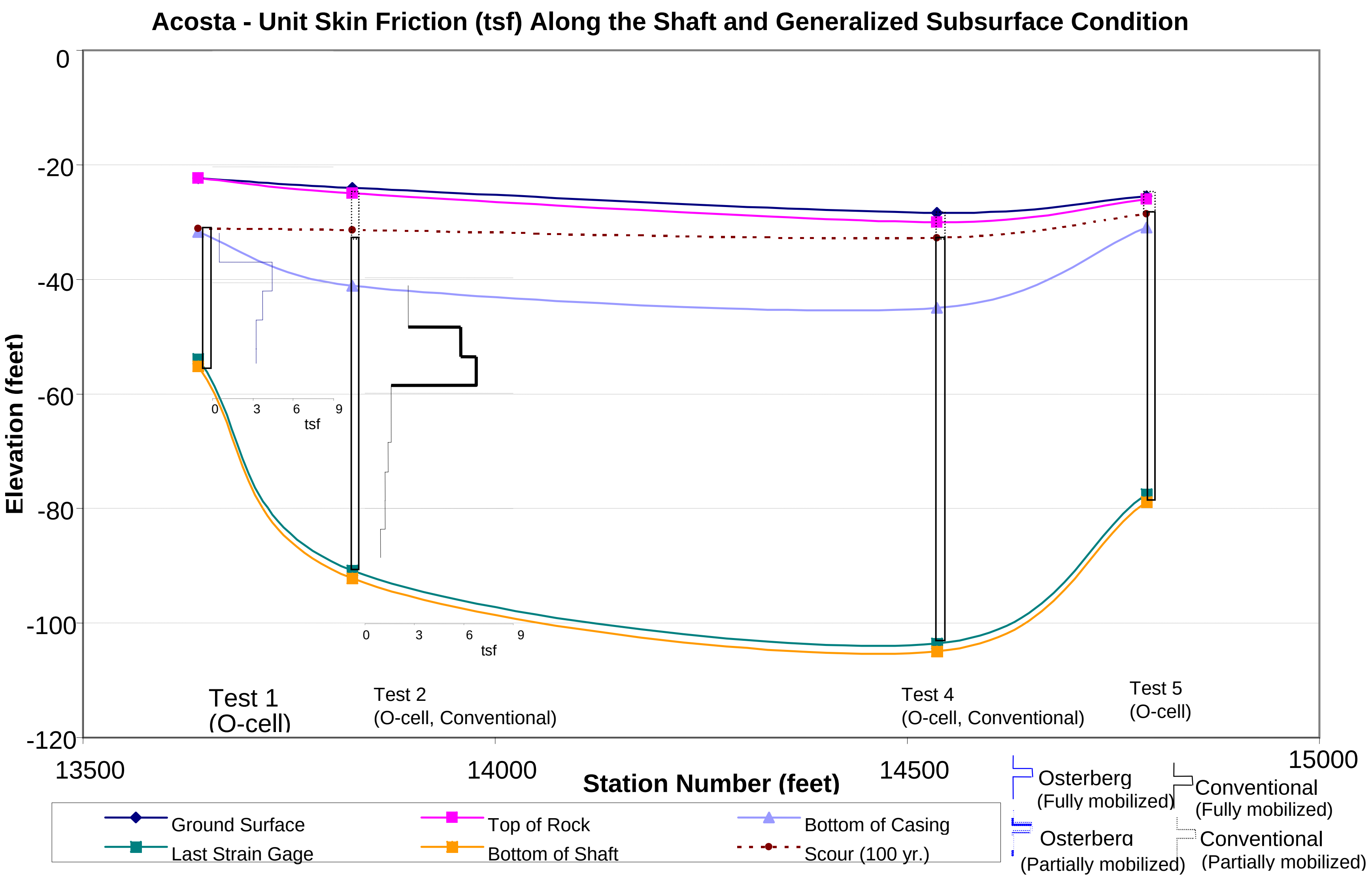


Figure 4.5 Unit Skin Friction Along Each Shaft for Osterberg Load Test, Acosta Bridge

Since all four test shafts failed in end bearing, both the FDOT failure and the maximum tip resistance were determined (see Table 4.1). Note that the reduced FDOT and maximum tip resistances differ based on tip settlement as explained in Figure 4.3.

4.2.4 Apalachicola Bridge

A total of six Osterberg tests (provided by Dames & Moore) were performed: Pier 46, Shaft 11A (46-11A); Pier 53, Shaft 2 (53-2); Pier 57, Shaft 10 (57-10); Pier 59, Shaft 8 (59-8); Pier 62, Shaft 5 (62-5); and Pier 69, Shaft 7 (69-7). For the six test shafts, two were 5-foot diameter shafts, two were 6-foot diameter shafts, one was a 7-foot diameter shaft, and one was a 8-foot diameter shaft.

A breakdown by Pier, Shaft tested, and type of test is shown below:

Pier	Shaft	Type of Test
Pier 46 (*5ft. diameter*)	Shaft 11A (41-11A)	Osterberg
Pier 53 (*6ft. diameter*)	Shaft 2 (53-2)	Osterberg
Pier 57 (*7ft. diameter*)	Shaft 10 (57-10)	Osterberg
Pier 59 (*7ft. diameter*)	Shaft 8 (59-8)	Osterberg & Lateral
Pier 62 (*6ft. diameter*)	Shaft 5 (62-5)	Osterberg
Pier 69 (*5ft. diameter*)	Shaft 7 (69-7)	Osterberg

For two of the six tests, shafts 57-10 and 69-7, multi-level Osterberg testing was performed in order to effectively isolate both end bearing and side resistance. The sequence of loading in multi-level Osterberg testing was described earlier in Figure 3.4. Both ultimate and mobilized unit skin friction for each shaft along with the limestone formation elevation are shown in Figure 4.6.

The end bearings computed at each shaft is tabulated in Table 4.1. These end bearings are relatively consistent over the whole site.

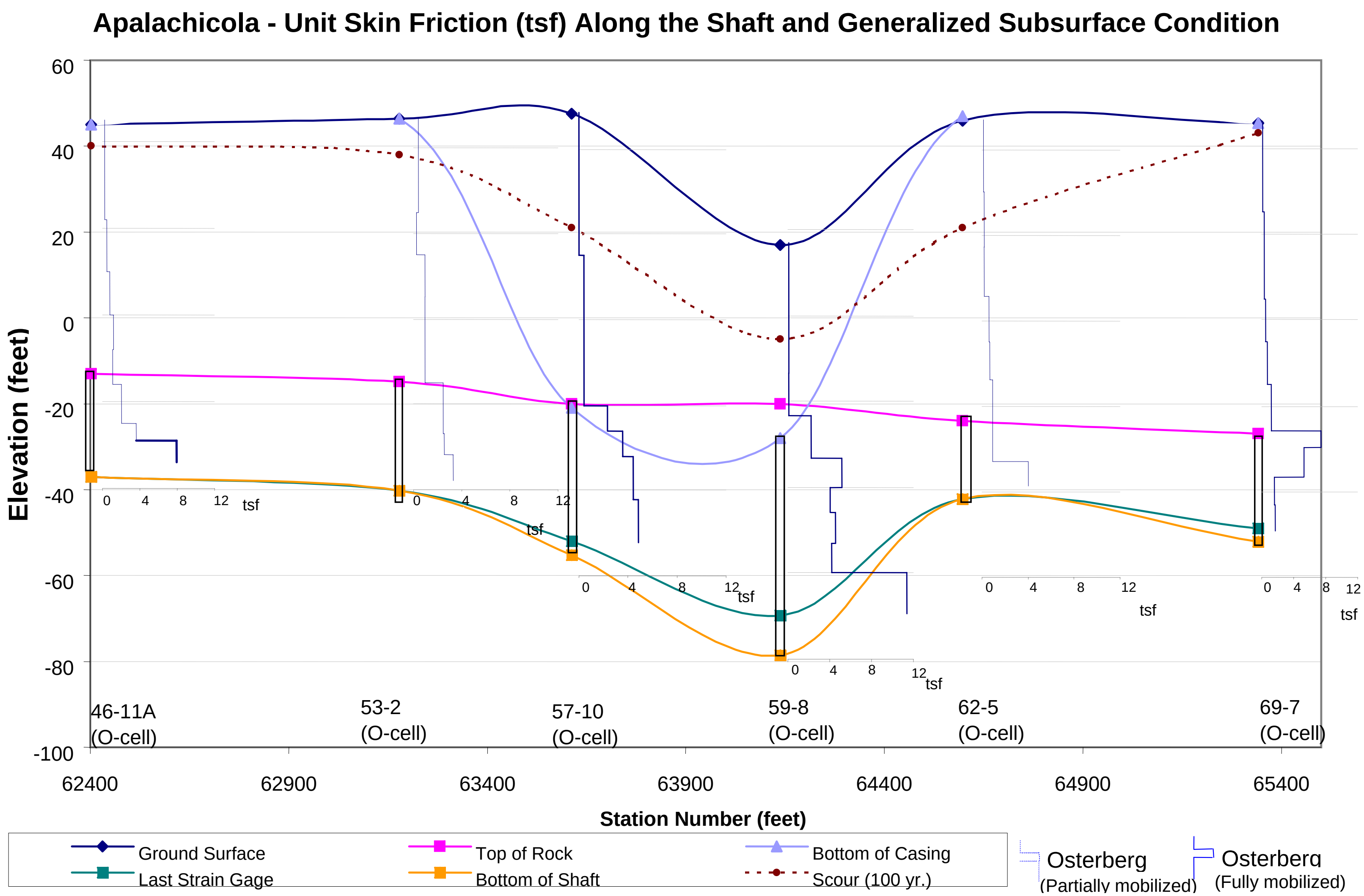


Figure 4.6 Unit Skin Friction Along Each Shaft for Osterberg Load Test, Apalachicola Bridge

4.2.5 Fuller Warren Bridge

A total of four Osterberg tests (provided by Law, Inc.) were performed: LT-1, LT-2, LT-3a, and LT-4. The tested shafts consist of one 3-foot diameter shaft, one 4-foot diameter shaft, and two 6-foot diameter shafts.

A breakdown by Pier, Shaft tested, and type of test is shown below:

Shaft Tested	Type of Test
Shaft LT-1 (*3ft. diameter*)	Osterberg
Shaft LT-2 (*6ft. diameter*)	Osterberg
Shaft LT-3a (*6ft. diameter*)	Osterberg
Shaft LT-4 (*4ft. diameter*)	Osterberg
Shaft LLT-1 (*6ft. diameter*)	Conventional Lateral
Shaft LLT-2 (*6ft. diameter*)	Conventional Lateral

While LT-1 and LT-4 were performed on land, LT-2 and LT-3 were conducted over water. Multi-level Osterberg testing was performed on LT-2, LT-3a and LT-4 in order to isolate both ultimate end bearing and side shear resistance (see Figure 3.4).

The unit skin frictions along the shafts are shown in Figure 4.7. It should be noted that the decrease in skin friction in the lower portions of the shafts is due to the presence of sandy clayey silt and clayey silty fine sand near the tip of the shafts. This material is locally termed "Marl" and has SPT blow counts in the 20s. Also note that the top of the limestone formation varies in the same fashion as the ground surface.

According to the end bearing data given in Table 4.1, LT-1 and LT-2 have the highest end bearings. These high end bearings well agree with the observed high skin frictions as shown in Figure 4.7; the unit skin friction at the bottom of LT-3a and LT-4 is clearly lower than those of LT-1 and LT-2 indicating that weaker geomaterials exist at the base of LT-3a and LT-4.

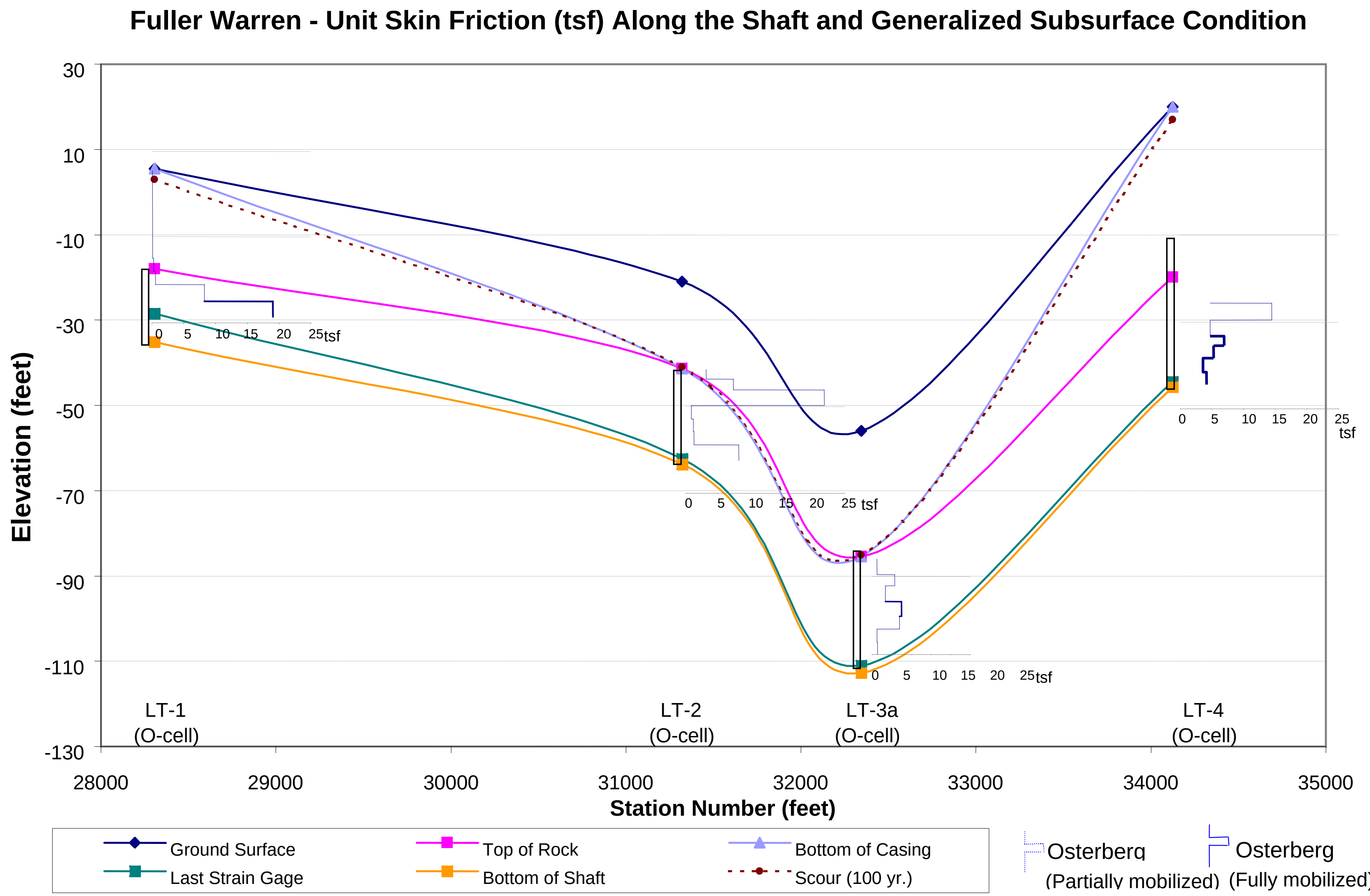


Figure 4.7 Unit Skin Friction Along Each Shaft for Osterberg Load Test, Fuller Warren Bridge

4.2.6 Gandy Bridge

A total of six shafts were load tested. One O-cell was installed at each test shaft. These tests were performed by Load Test, Inc.: Pier 26, shaft 2(26-2); Pier 52, shaft 4 (52-4); and Pier 91, shaft 4 (91-4).

A breakdown by Pier, Shaft tested, and type of test is shown below:

Pier	Shaft Tested	Type of Test
Pier 26 (*4ft. diameter*)	Shaft 1 (26-1)	Statnamic
Pier 26 (*4ft. diameter*)	Shaft 2 (26-2)	Osterberg
Pier 26 (*4ft. diameter*)	Shaft 1 & 2	Conventional Lateral
Pier 52 (*4ft. diameter*)	Shaft 3 (52-3)	Statnamic
Pier 52 (*4ft. diameter*)	Shaft 4 (52-4)	Osterberg
Pier 52 (*4ft. diameter*)	Shaft 3 & 4	Conventional Lateral
Pier 91 (*4ft. diameter*)	Shaft 3 (91-3)	Statnamic
Pier 91 (*4ft. diameter*)	Shaft 4 (91-4)	Osterberg
Pier 91 (*4ft. diameter*)	Shaft 3 & 4	Conventional Lateral

This project is located over the Tampa Formation and has had numerous foundation construction problems. The site is very karastic with very erratic surface elevations and variable strength characteristics. The top of the limestone formation and unit skin frictions along the shafts is presented in Figure 4.8.

The unit end bearing for the test shafts 51-4 and 91-4 was computed assuming that the unit skin friction below the O-cell was equal to the unit skin friction above the O-cell. The unit end bearing values presented in Table 4.1 are the values recommended by Load Test, Inc. in their Gandy load test report.

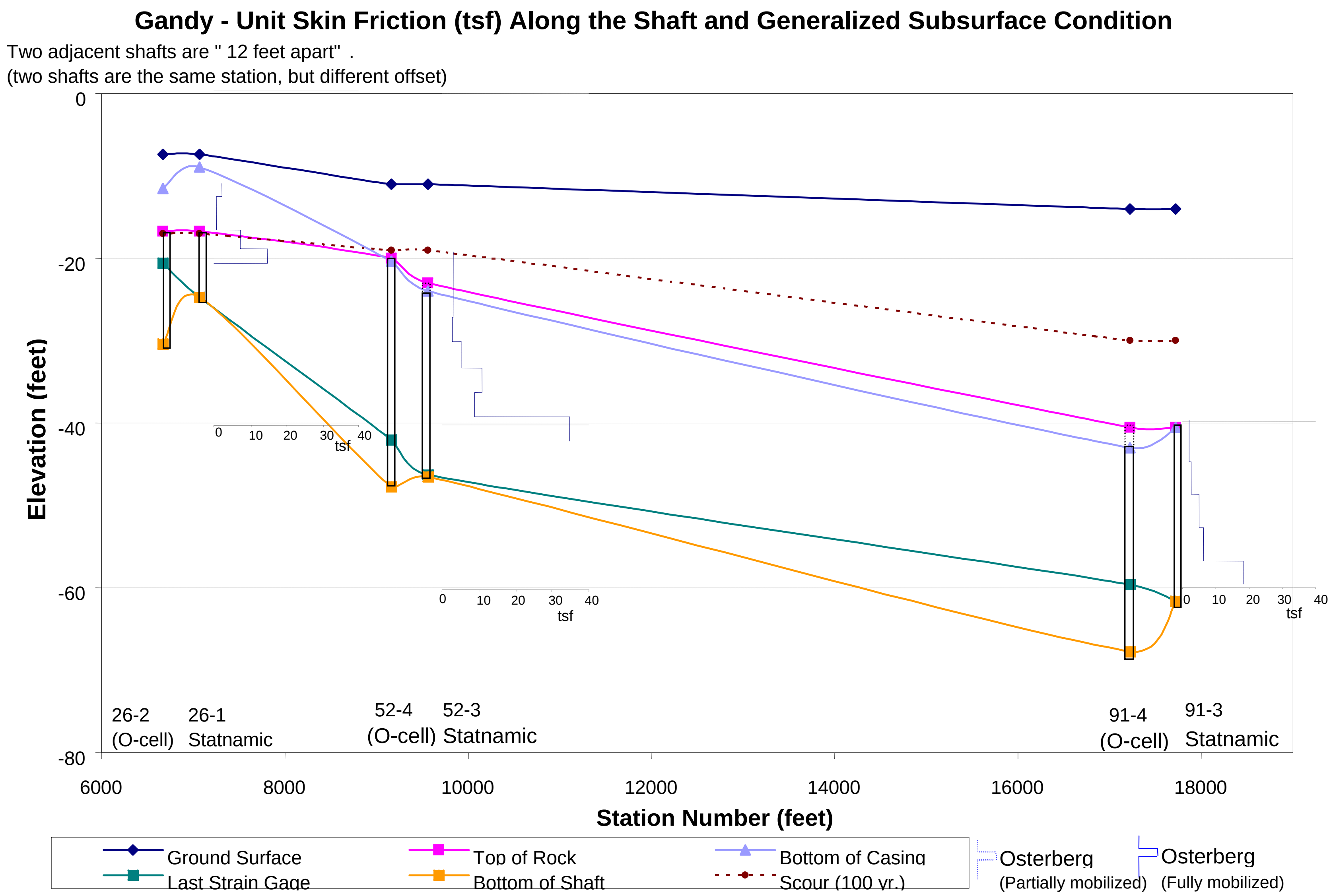


Figure 4.8 Unit Skin Friction Along Each Shaft for Osterberg Load Test, Gandy Bridge

4.2.7 Hillsborough Bridge

One Osterberg (provided by Load Test, Inc.) and two Statnamic (provided by Berminghammer Foundation Equipment) tests were performed at this test site: Pier 4, shaft 14 (4-14); and Pier 5, shaft 10 (5-10). All test shafts were four feet in diameter.

A breakdown by Pier, Shaft tested, and type of test is shown below:

Pier	Shaft Tested	Type of Test
Pier 4 (*4ft. diameter*)	Shaft 14 (4-14)	Osterberg & Statnamic
Pier 5 (*4ft. diameter*)	Shaft 10 (5-10)	Statnamic

The Osterberg test at 4-14 failed in skin friction above the cell and consequently, the end bearing and skin friction for the portion of the shaft below the O-cell were not fully mobilized. Moreover, since there was 9.8 feet of shaft beneath the cell and the lowest strain gauge was located 7.3 feet above the bottom of the shaft, it was very difficult to assess the unit skin friction value along the shaft below the last strain gauge. Consequently, due to the unknown skin friction below the cell, the end bearing could not be computed from the Osterberg test. The top of the limestone formation and unit skin frictions along the shaft are shown in Figure 4.9.

4.2.8 MacArthur Bridge

One conventional top down load test (provided by Law Engineering, Inc.) was performed on a 30-inch diameter shaft. Four 42-inch diameter reaction shafts were installed using steel casing and slurry. The capacity of the loading frame was 1250 tons. Unfortunately, the test shaft did not reach failure according to the load vs. deflection curve. For instance, at a top load of 1080 tons only 0.361 inch of head deflection was recorded. Given the magnitude of movement, and associated mobilized unit skin friction and end bearing (i.e. negligible), the results couldn't be used; however the construction information was considered.

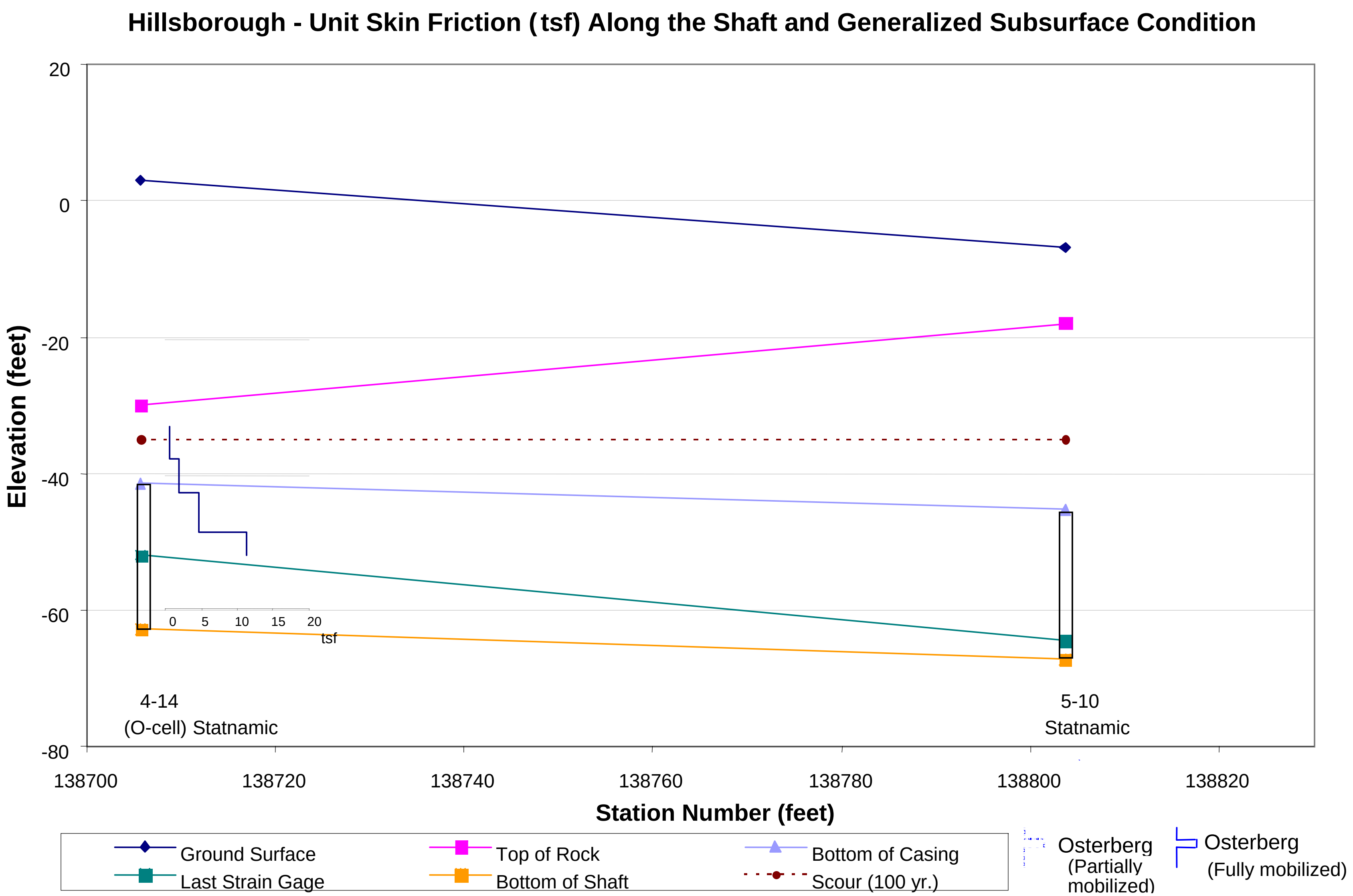


Figure 4.9 Unit Skin Friction Along Each Shaft for Osterberg Load Test, Hillsborough Bridge

4.2.9 Victory Bridge

A total of five Osterberg load tests (provided by Load Test, Inc.) and one Statnamic test (provided by Berminghammer Foundation Equipment) were performed at this site. The tested shafts were all four-foot diameter shafts.

A breakdown by Pier, Shaft tested, and type of test is shown below:

Pier	Shaft Tested	Type of Test
Bent 3 (*4ft. diameter*)	Shaft 1 (3-1)	Osterberg
Bent 3 (*4ft. diameter*)	Shaft 2 (3-2)	Osterberg
Bent 10 (*4ft. diameter*)	Shaft 2 (10-2)	Osterberg
Bent 19 (*4ft. diameter*)	Shaft 1 (19-1)	Osterberg
Bent 19 (*4ft. diameter*)	Shaft 2 (19-2)	Osterberg
Test Hole 1 (*4ft. diameter*)		Conventional Lateral Load
Test Hole 2 (*4ft. diameter*)		Conventional Lateral Load
Test Hole 5 (*4ft. diameter*)		Statnamic & Lateral

All of the test holes and test shafts were constructed using casings and wet methods (water). All the Osterberg tests failed in side shear, and one (10-2) reached the FDOT bearing failure criterion.

The top of the limestone formation and the unit skin friction on the shafts are shown in Figure 4.10. Note that solid lines are fully mobilized skin frictions, whereas the dashed are partially mobilized frictions. It is evident that the unit skin frictions are uniform over the site. This is reflected in the low standard deviation and the low coefficient of variability (29%), shown in Figure 4.5.

Three Osterberg cells at Shafts 3-2, 10-2 and 19-2 were placed above the tip to insure sufficient bearing resistance in order to fully mobilize the skin resistance above the cells. The other two tests (3-1 and 19-1) had the Osterberg cell at the bottom of the shaft to obtain a direct

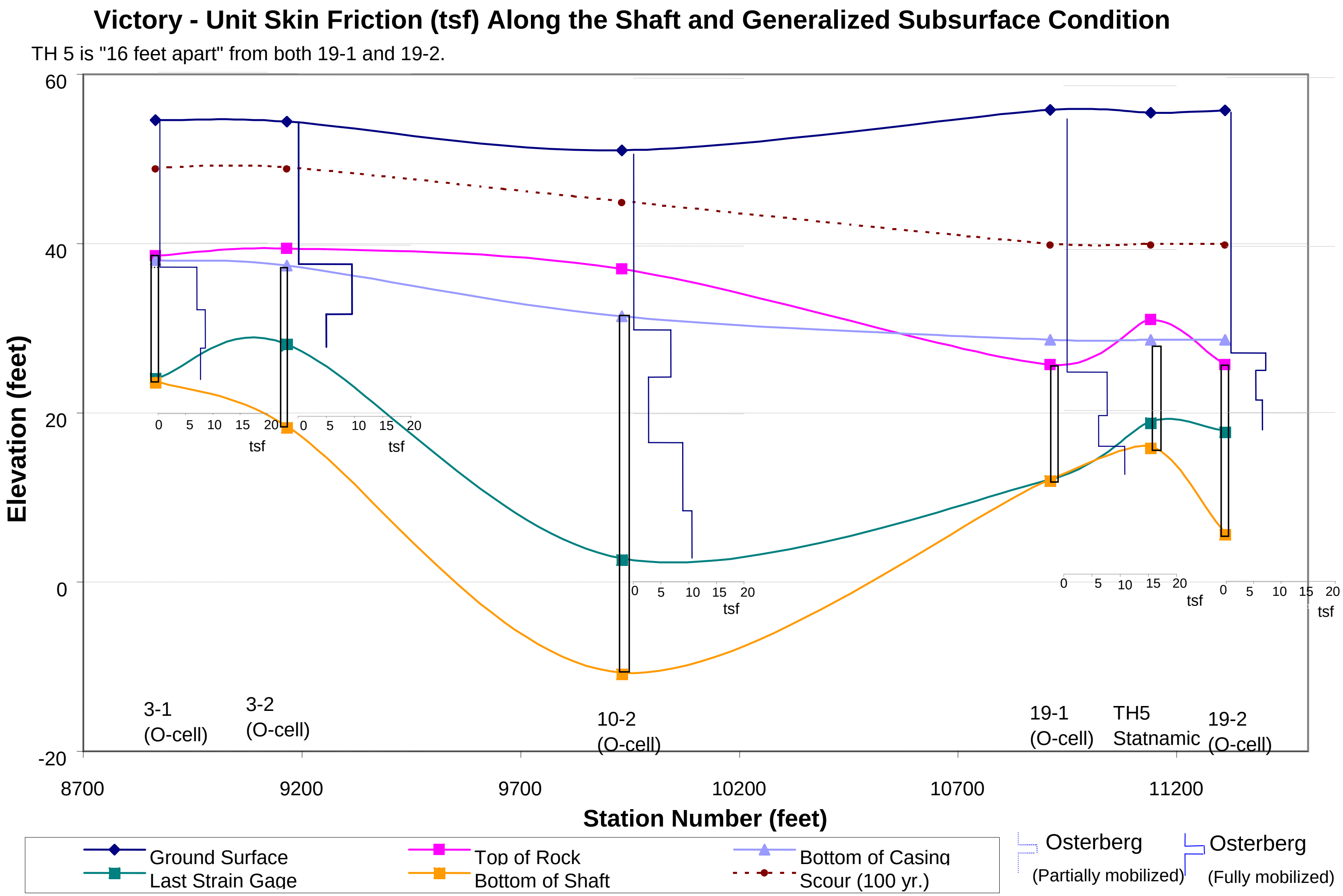


Figure 4.10 Unit Skin Friction Along Each Shaft for Osterberg Load Test, Victory Bridge

measure of end bearing. In the case of end bearing at shaft 10-2, it was computed assuming that the unit skin friction below the O-cell was equal to friction above the O-cell. The end bearing values presented in Table 4.1 are recommended values given in the Victory load test report.

4.2.10 Analysis and Summary

4.2.10.1 Skin friction analysis and summary

To investigate the range of mobilized versus ultimate unit skin frictions; a frequency distribution for every point on each shaft was generated for all sites in Figure 4.11. The figure shows two distributions: the fully mobilized (ultimate: solid lines Fig. 4.4-4.10) unit skin frictions and the partially mobilized (dashed lines: Fig. 4.4-4.10) unit skin frictions along all the shafts. As expected the peak of ultimate unit skin frictions distribution is to the right of the partially mobilized skin friction. Unexpectedly, the average of mobilized skin frictions is higher than that of ultimate skin frictions. But, the median of ultimate deviation is higher than that of mobilized deviation. The medians should be used for comparison purposes since the mean is highly affected by extreme values. The ultimate unit skin friction shows a mean range of 3 to 9 tsf and standard deviation of 3.8 tsf. A comparison between Statnamic and Osterberg unit skin friction will be presented in Chapter 5.

A breakdown of the average unit skin friction by site (Figures 4.4 to 4.10) is given in Figure 4.12. The mean values in this figure include the combination of both mobilized and ultimate values. The combination was necessary due to the insufficient ultimate values for some of the sites. Also, note the 17th Street Bridge is divided into two areas soft and hard. Unfortunately, there was only one load test in the soft area and limited number of laboratory strength data (i.e., qu and qt) in this area.

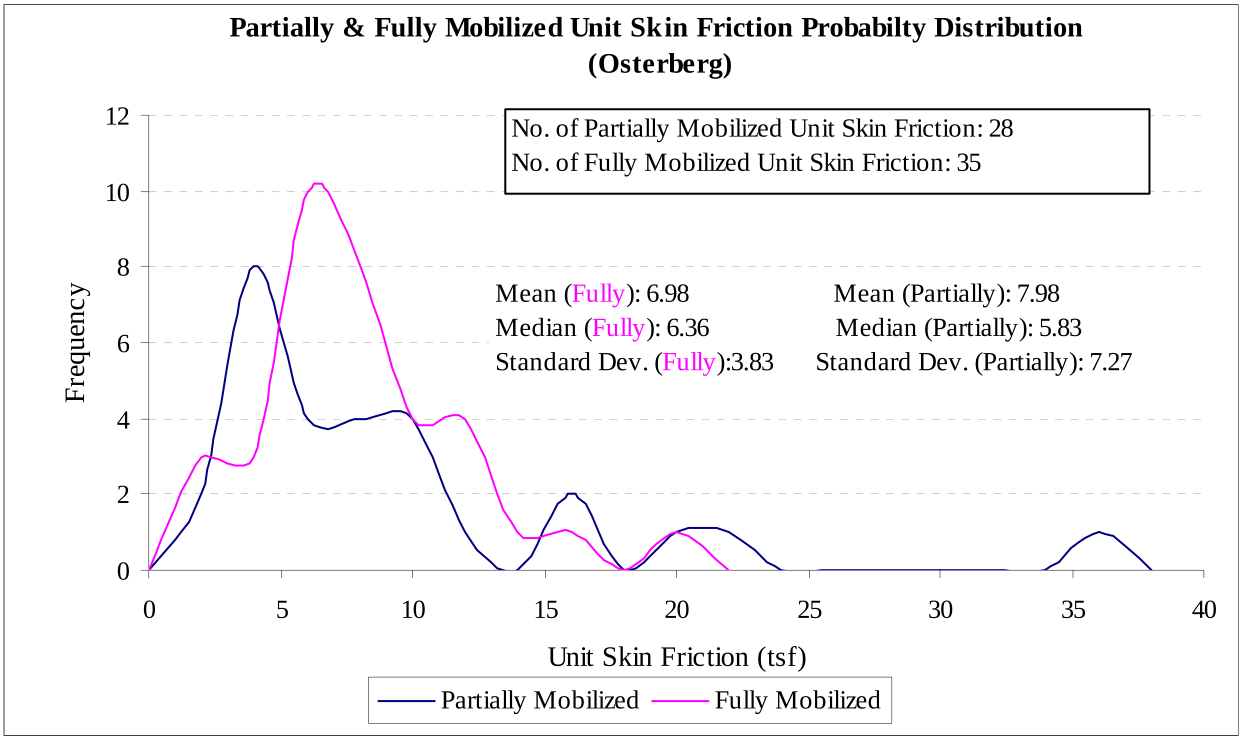


Figure 4.11 Osterberg Unit Skin Friction Probability Distribution

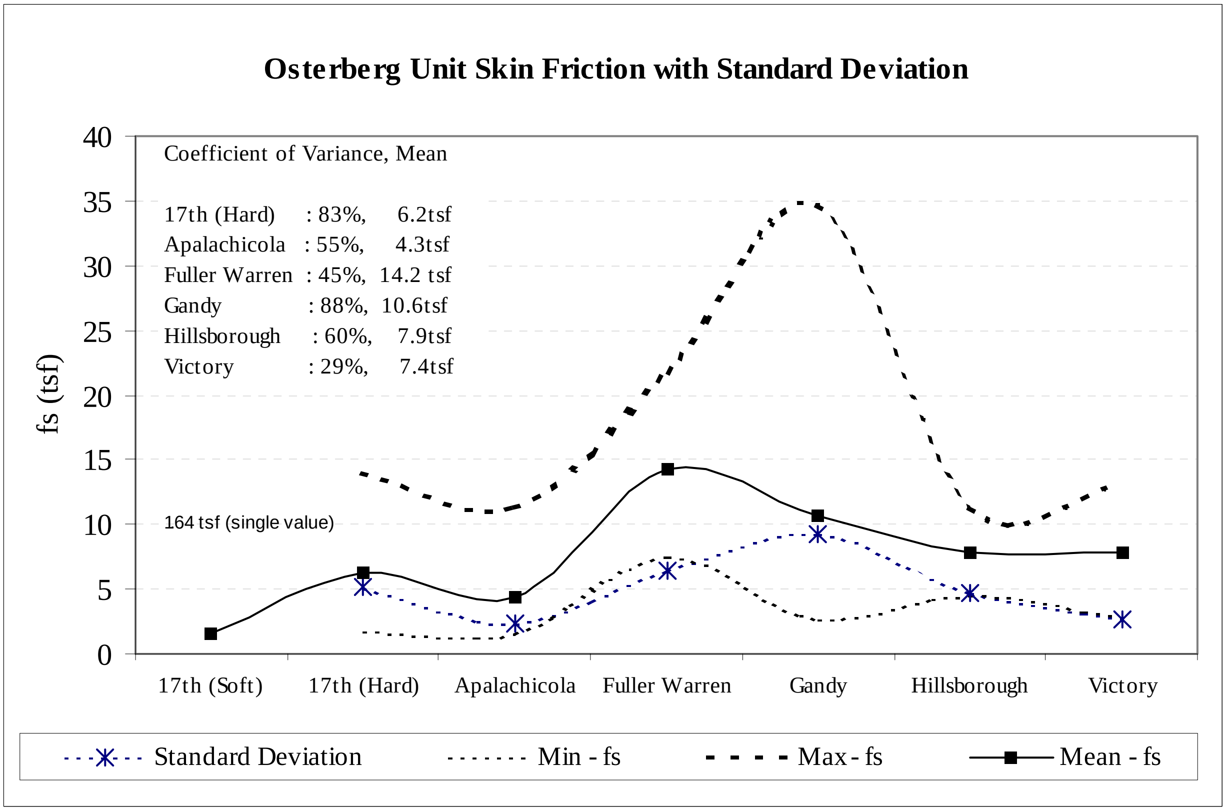


Figure 4.12 Osterberg Unit Skin Friction with Standard Deviation

The coefficient of variability of the unit skin friction at a point is also given in Figure 4.12. As expected, each site has a high variability in the range of 30 to 90%. However, the latter does not represent the variability that the designer faces, since he/she is interested in the average unit skin friction variability for a shaft anywhere on the site. The latter will be discussed extensively in chapter seven.

Of interest also are the normalized T-Z curves for Florida Limestone. Shown in Figure 4.13 are the fully mobilized T-Z curves back computed from all of the Osterberg tests. Both the x and y axes have been normalized. The unit skin friction (y axis) is divided by the ultimate skin friction (fully mobilized failure), and the vertical displacement, x-axis, was divided by the diameter of the shaft. The latter normalization had a significant influence on reducing the variability of the T-Z curves.

To use the curve, Figure 4.13, consider a 4-ft diameter shaft. For a vertical displacement of 0.5% of the diameter, or 0.25 inches, then approximately 80% of the ultimate skin friction is mobilized. The latter general T-Z curve is only a function of diameter of the shaft (controls displacement), and strength of rock (controls ultimate unit skin friction).

4.2.10.2 End bearing analysis and summary

The end bearings for each project are tabulated in Table 4.1. The unit end bearings are categorized into three different criteria: mobilized, FDOT failure, and Maximum failure. The difference between them is explained in Figure 4.3.

As discussed earlier, if the Osterberg cell is located at the shaft's tip, the tip resistance can generally be fully mobilized. However, if the O-cell is located a certain distance above the shaft's tip, the end bearing and skin friction for the portion of the shaft below the cell may not be fully mobilized, and the failure generally occurs on the portion of the shaft above the cell. In this

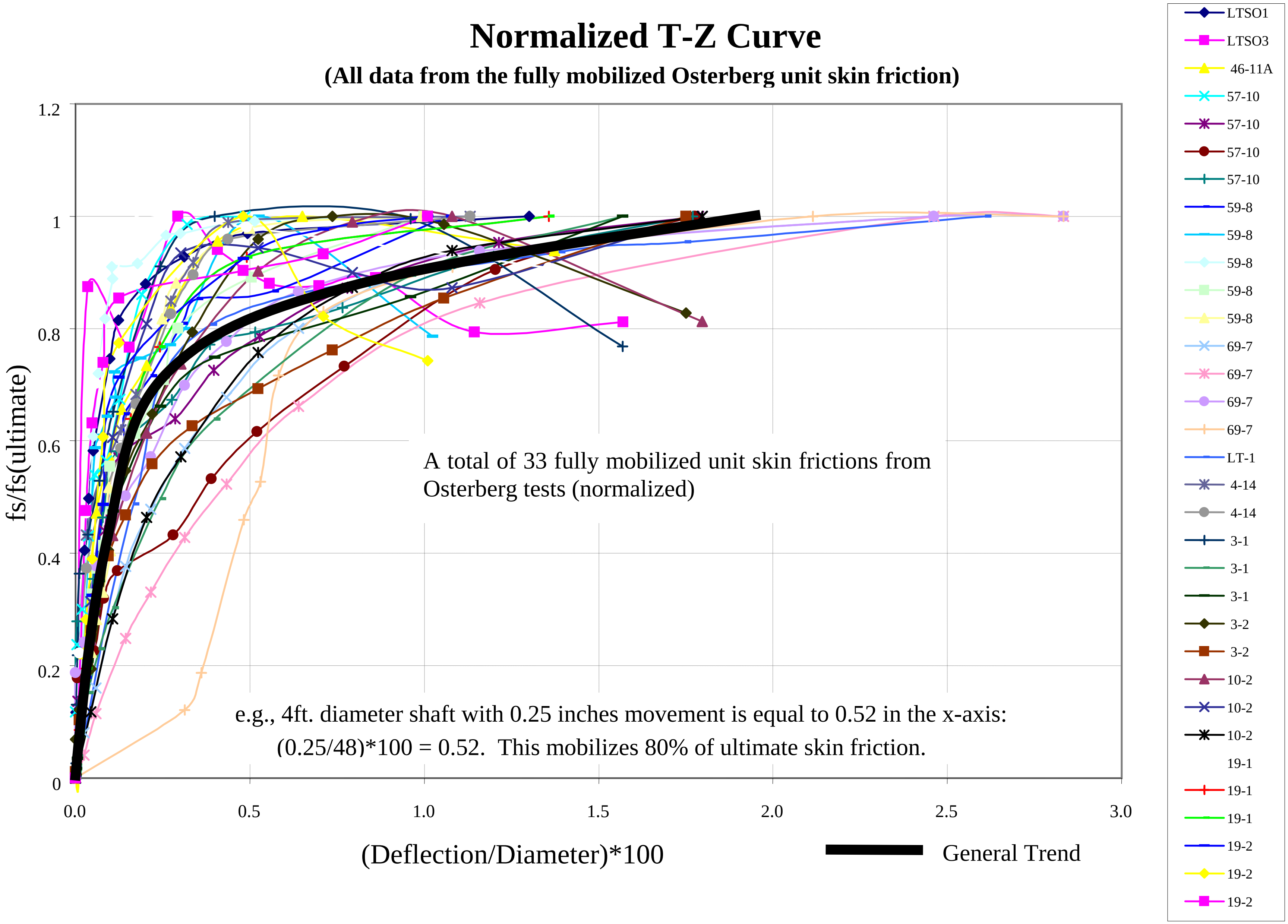


Figure 4.13 Normalized T-Z Curves with General Trend (Osterberg)

case, the skin friction value for the zone below the Osterberg cell has to be computed (if strain gauges were installed below O-cell) or assumed in order to estimate the amount of end bearing since the load applied at the O-cell includes both skin and end bearing resistances. If the length of the shaft below the cell is relatively long (more than 6 ft.), assuming the skin friction below the cell is not recommended. Moreover, many of the consultant test reports did not report end bearings, which is indicated as "x" in Table 4.1. Most of the values in Table 4.1 were quoted from load test reports and verified. Ten of the twenty-seven collected Osterberg tests failed in either skin or end bearing resistance instead of failing in both at the same time.

Based on the values given in Table 4.1, 13 out of 27 tests reached the FDOT failure criteria. The 10 test shafts that failed in end bearing failed by both the FDOT and maximum failure criteria. For these shafts, the mean of maximum unit end bearings was about 25% higher than the mean of FDOT failures. The average of all the FDOT failures is 44.6 tsf, and the average of the maximum failures is 55.8 tsf.

4.3 STATNAMIC LOAD TESTING

4.3.1 General

Twelve of the forty-two axial load tests collected were Statnamic tests. Based on the shape of the load vs. displacement curves obtained from the load test reports, most of the tests did not reach ultimate failure. That is, the load vs. displacement curves did not exhibit plunging at or near their peak, with the exception of Shaft 14-4 in Hillsborough.

Each Statnamic test was analyzed with the Unloading Point Method (UPM) (Middendorp et al., 1992) to determine the static load-displacement response of the shafts. The UPM method is described in Section 3.5 ($F_{statnamic} = F_{static} + C\times v + M\times a$).

Since most of the test shafts did not exhibit failure, the reported static capacity was relatively similar to the measured Statnamic force (see Figure 4.14). The latter was attributed to the reported small damping and inertial forces for the shafts tested in the stiff limestones. Based on the UPM method (section 3.5) there will be little difference between the Statnamic force and the

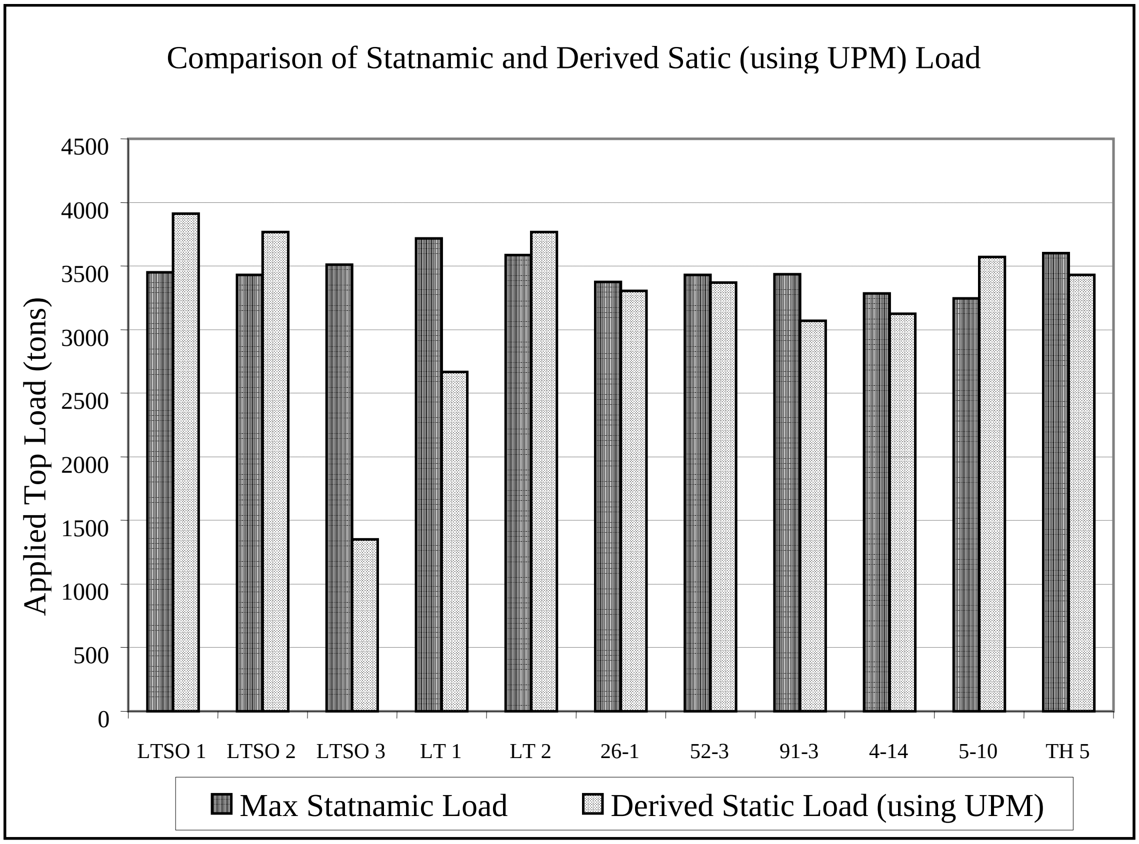


Figure 4.14 Comparison of Statnamic and Derived Static (using UPM) Load in Tons

derived static force, with the latter being higher than the measured dynamic force. Generally for such situations, load test reports generally used the lower capacity for conservatism, i.e., measured Statnamic force (see Figure 4.14). When shafts were installed in soft materials (LTSO3 and LT1 at 17^{th} bridge), there was a big difference between the Statnamic force and the derived static force due to high damping and inertial forces. The comparison of Statnamic and derived static load are shown in Figure 4.14.

Also of interest was the Q-Z or tip resistances vs. tip displacement curves from the Statnamic tests. As shown in Figure 4.3 from the Osterberg tests, the mobilized end bearing is a function of tip displacements. Shown in Table 4.2 are measured maximum displacements from the Statnamic results. Evident from the table, only two tests had tip displacements greater than one inch with the majority having displacement less ½ inch. Consequently, only two of the Statnamic tests reached the FDOT failure criterion at the shaft tip. It is recommended that future Statnamic tests evaluate the shaft capacity (skin and tip) prior to selecting the maximum applied dynamic force. A discussion of each site with Statnamic static results follows.

4.3.2 17th Street Bridge

A total of six Statnamic tests were performed at 17th Street Bridge: Pier 6, Shaft 10 (LTSO1); Pier 7, Shaft 3 (LTSO2); Pier 5, Shaft 3 (LTSO3); Pier 10, Shaft 1 (LTSO4); Pier 2, Shaft 1 (LT1); and Pier 8, Shaft 3 (LT2). The unit skin frictions along the shafts in Figure 4.15 are all drawn with dashed lines (all mobilized skin frictions). LTSO 4 is not drawn in this figure since the data were provided in the load test company's report. In the Statnamic load test report (provided by Applied Foundation Testing, Inc.), only LTSO3 and LT1 had a derived static capacity much smaller than its Statnamic capacity. It was believed that because the soil conditions at LTSO3 and LT1 were considerably softer than other shafts, high damping and inertial forces were generated.

The summary of maximum Statnamic load, derived static load, and movement are tabulated in Table 4.2. The end bearings are very low to the measured tip displacements with the exception of LT1 (soft rock).

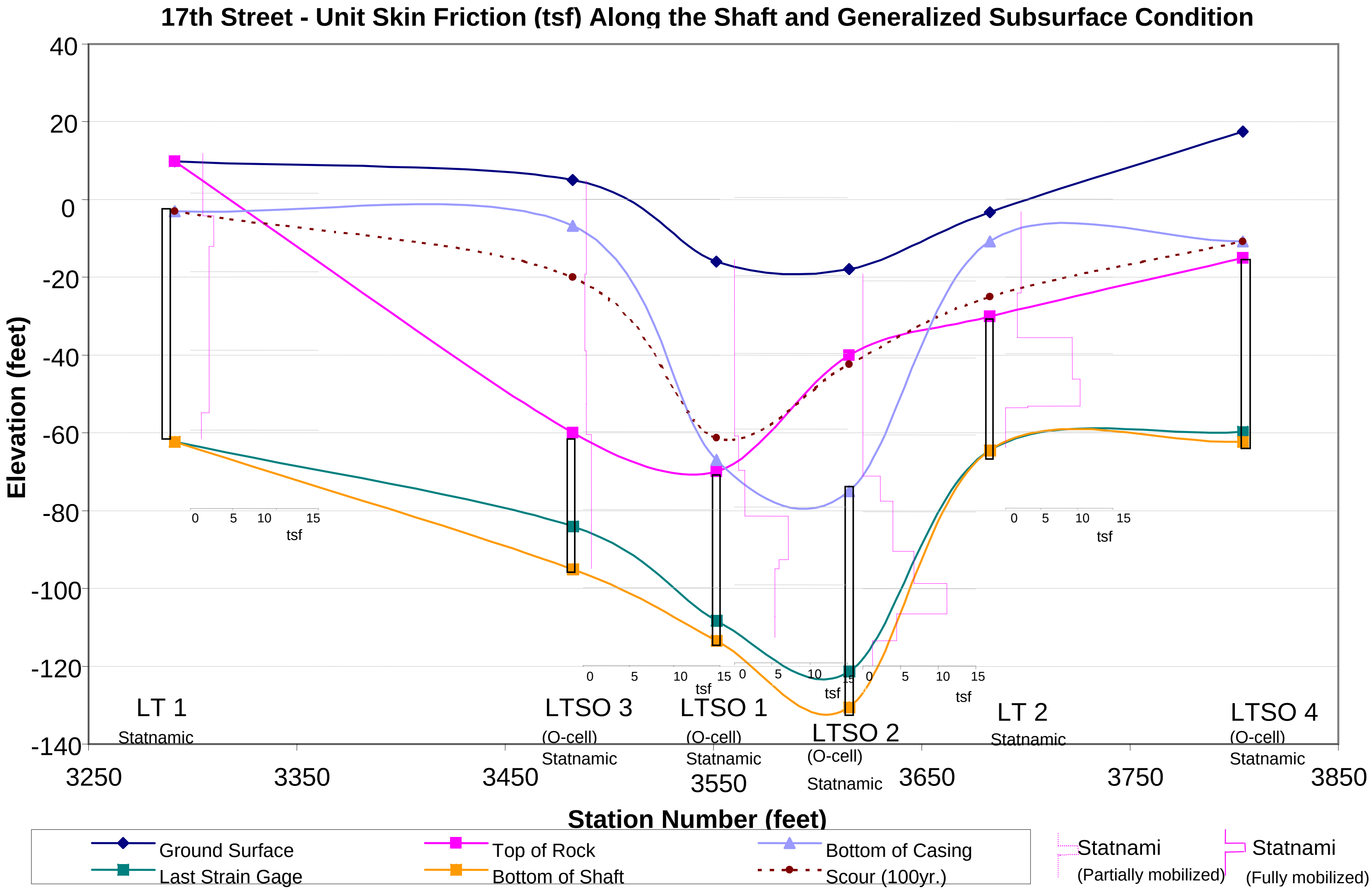


Figure 4.15 Unit Skin Friction Along Each Shaft for Statnamic Load Test, 17th Bridge

4.3.3 Gandy Bridge

A total of three Statnamic tests (Berminghammer Corporation Limited) were conducted to validate design and to compare to the Osterberg results. The tests were performed at the following locations: Pier 26, shaft 1 (26-1); Pier 52, shaft 3 (52-3); and Pier 91, shaft 3 (91-3). The unit skin friction along the shaft is shown in Figure 4.16. Since the data for 91-3 were not provided in the load test report, the unit skin frictions were not drawn in the figure.

As indicated in Table 4.2, the end bearing at 91-3 shows relatively high values compared to the other tested shafts. The summary of maximum Statnamic load, derived static load, and movement are also tabulated in Table 4.2.

The computed static loads from the unloading point method (UPM) were very similar to the Statnamic forces, shown in Table 4.2 and Figure 4.14.

4.3.4 Hillsborough Bridge

A total of two Statnamic tests were performed on Pier 4, Shaft 14 (4-14) and Pier 5, Shaft 10 (5-10). The Statnamic test at 4-14 had a maximum applied load of 3287 tons with a maximum top displacement of 1.63 inches, but it did not exhibit plunging (large displacement with little load increase). However, shaft 4-14 did fail in skin friction due to a large net displacement, 1.06 of the shaft. A maximum load of 3243 tons with a total top movement of 0.57 inches was measured at 5-10. The displacement of the tip was only 0.11 inches, which wasn't sufficient for FDOT failure.

The derived static forces using the UPM are shown in Figure 4.14. The unit skin friction along the shaft is shown in Figure 4.17.

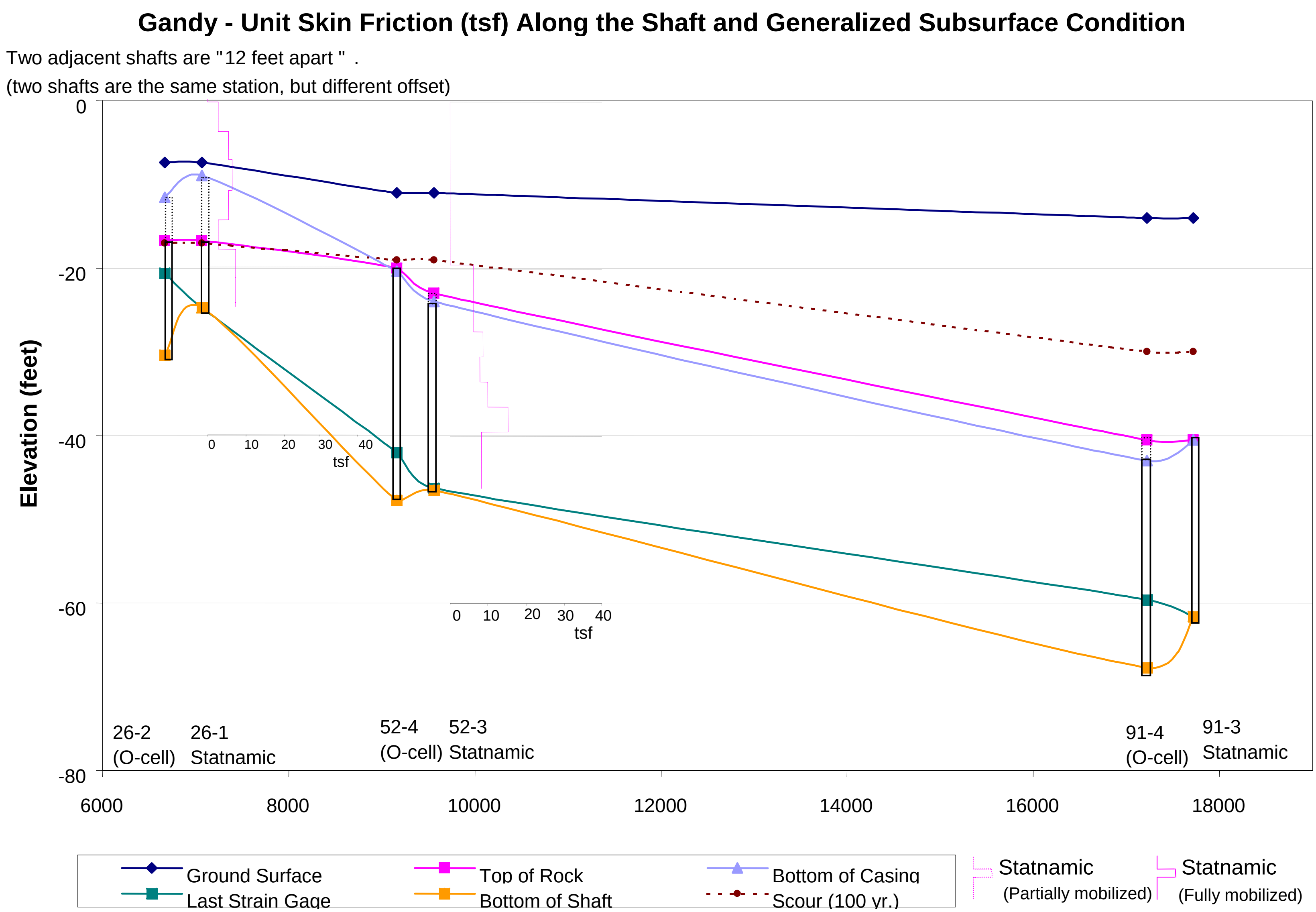


Figure 4.16 Unit Skin Friction Along the Shaft for Statnamic Load Test, Gandy Bridge

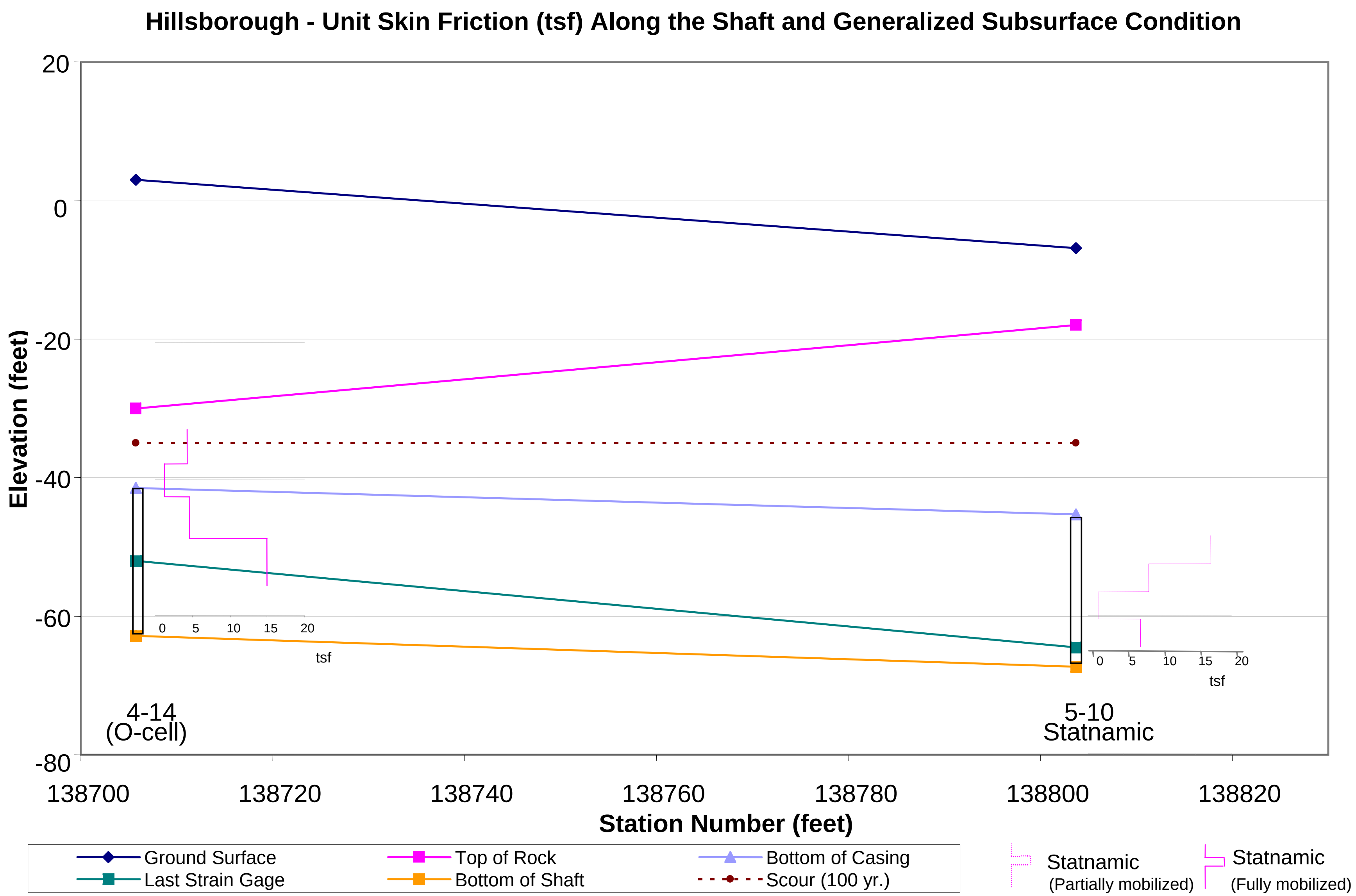


Figure 4.17 Unit Skin Friction Along the Shaft for Statnamic Load Test, Hillsborough Bridge

4.3.5 Victory Bridge

A single 30 MN Statnamic test was performed at Test Hole 5 (TH5) at Victory Bridge. The applied dynamic load, 3600 tons, was a world record at that time. The static resistance using the Unloading Point Method was 3,430 tons. Based on the load-settlement response, the test did not fully mobilize both skin friction and end bearing.

An average skin friction of 0.8 tsf in the upper soils (clay and sands) and a maximum of 18.5 tsf in the hard limestone formation were reported and shown in Figure 4.18. The average unit skin friction in the medium dense sand to weathered limestone was 2.8 tsf. The unit end bearing was estimated to be over 45 tsf based on tip displacements (Table 4.2).

4.3.6 Analysis and Summary

4.3.6.1 Skin friction analysis and summary

The statistical analyses of the Statnamic vs. the Osterberg data show similar average skin friction and standard deviation, as shown in Figures 4.11 and 4.19. However, the majority of the Statnamic unit skin frictions were slightly higher than that of the Osterberg test. The median of the Statnamic unit skin friction (partially mobilized) was 7.31 tsf, and the median of the Osterberg unit skin (partially mobilized) was 5.83 tsf. The median was used for comparison instead of mean, because means are highly affected by extreme values. The statistical analysis of the Osterberg result is shown in Figures 4.11 and 4.12.

Statnamic tests showed high variability (coefficient of variance) in unit skin frictions as evident in Figure 4.20. It shows the max, min, mean, and standard deviation of each project. It should be noted that most of the Statnamic load tested shafts were close to being fully mobilized in skin friction. The latter is based on the average vertical movement in the "Middle of the Rock," (Table 4.2) and the normalized T-Z curves given in Figure 4.13.

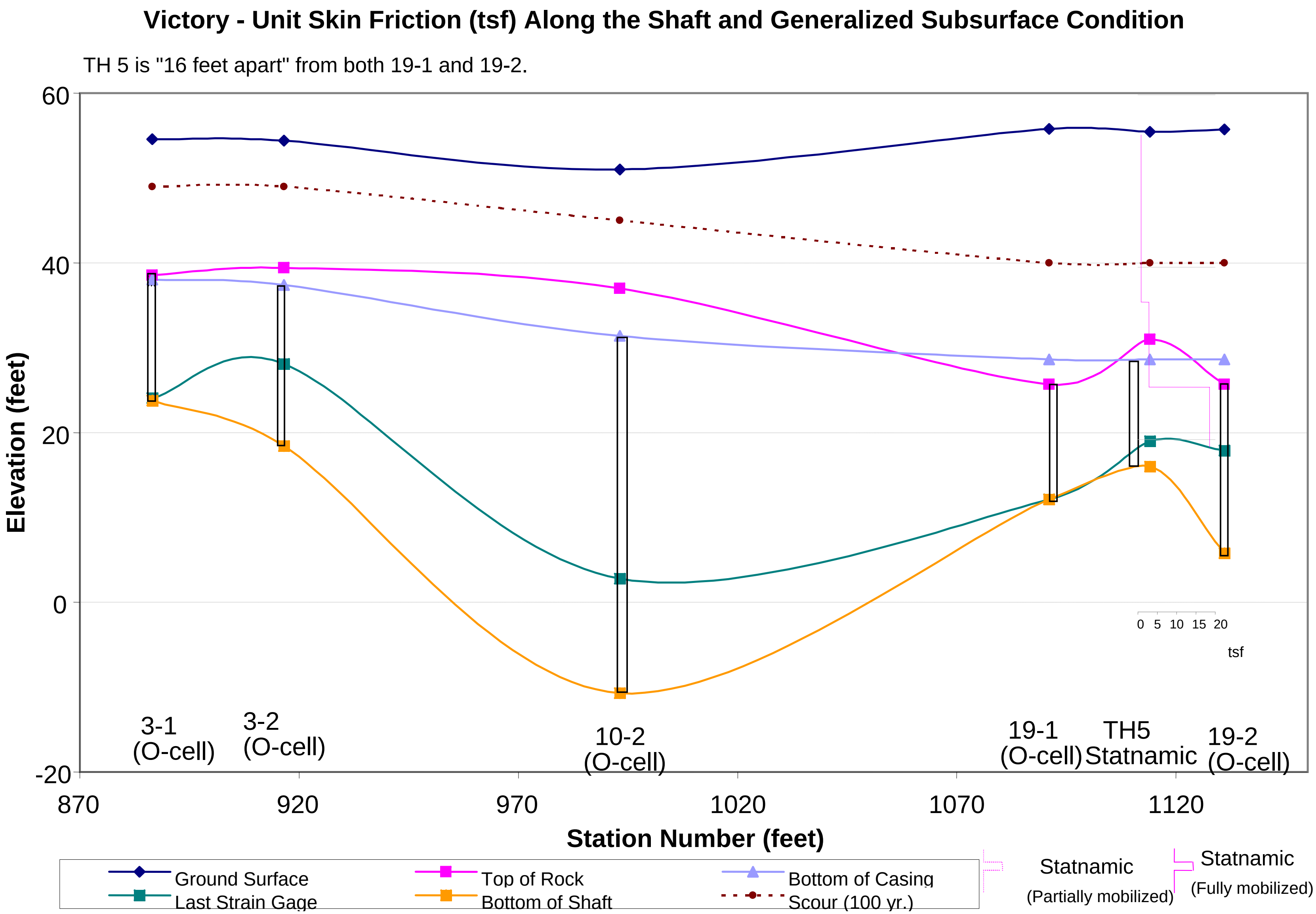


Figure 4.18 Unit Skin Friction Along the Shaft for Statnamic Load Test, Victory Bridge

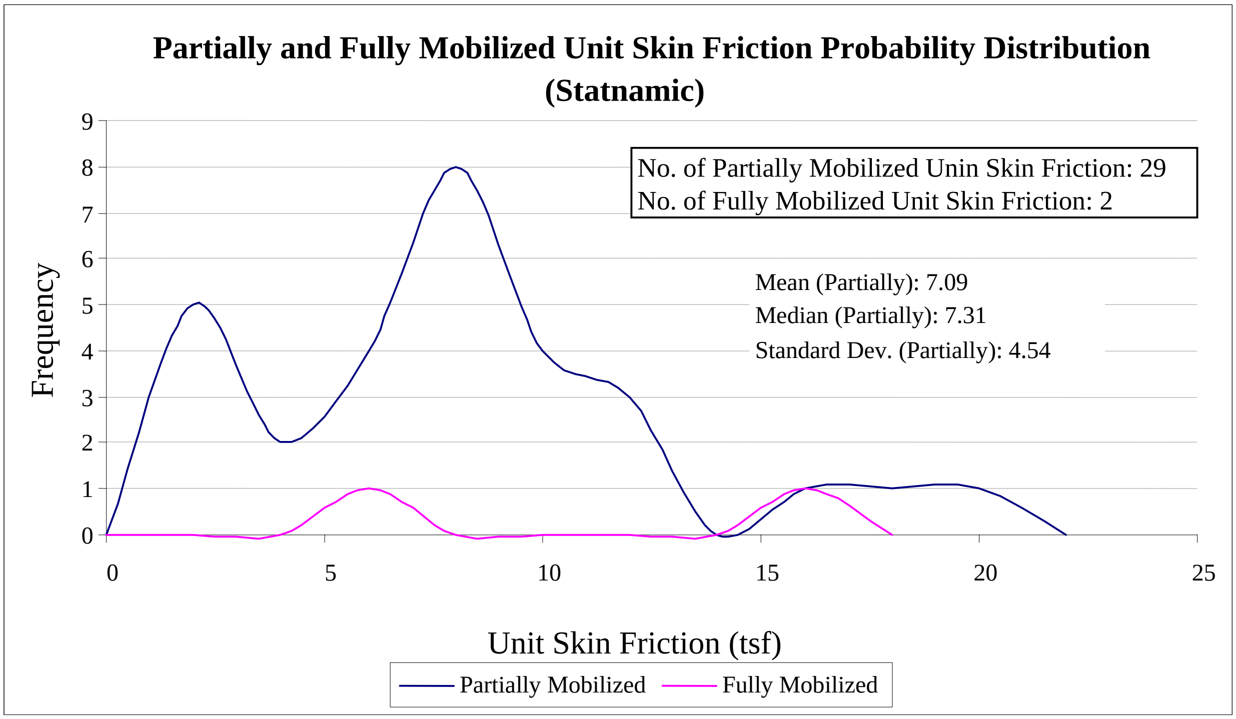


Figure 4.19 Statnamic Unit Skin Friction Probability Distribution

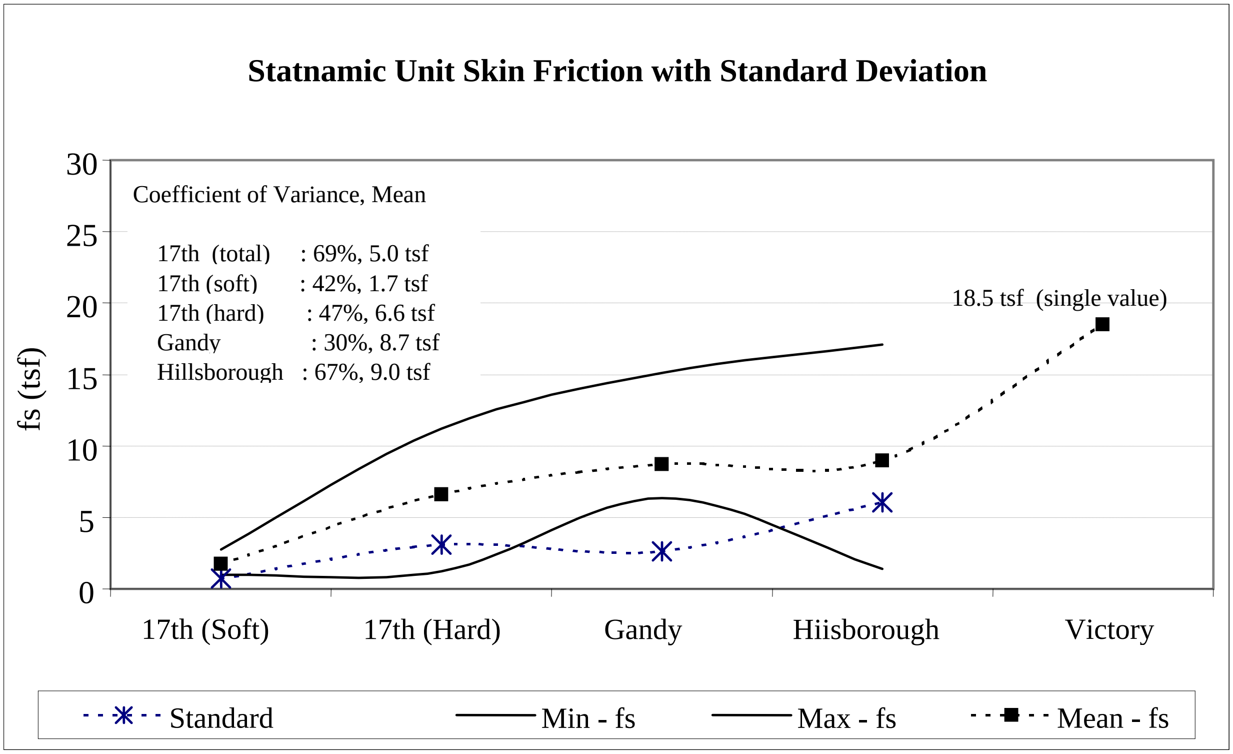


Figure 4.20 Statnamic Unit Skin Friction with Standard Deviation

The load test reports provided by the load test companies generally present both UPM static capacity as well as the dynamic loads (Statnamic loads). Figure 4.14 provides a comparison of both the Statnamic (dynamic) and derived static forces. In general, the derived static capacities are relatively similar to the measured Statnamic force. It is believed that when test shafts are installed in stiff soil/rock materials, small damping and inertial components are generated. However, there is a big difference between the Statnamic force and the derived static force when shafts are installed in soft soil/rock materials (LTSO3 and LT1 at 17^{th} bridge). This is likely due to high damping and inertial components.

4.3.6.2 End bearing analysis and summary

The unit end bearing from Statnamic testing, as given in Table 4.2, is typically small (exception of Gandy 91-3) due to the limited tip movement (also shown in Table 4.2). This small displacement is due to the test process. Since the load is applied at the top of the shaft (shown in Figure 3.5), a considerable shortening occurs along the shaft. Consequently, most of the applied load at the top is transferred to skin resistance with very little reaching the tip. Moreover, since the shafts did not move much (typically 0.5 to 1 inch), the FDOT failure criterion was not reached (see Table 4.2). It should be noted that the end bearing failure occurs only after the side resistance failure occurs; much more deflection is needed to fail the end bearing than side resistance.

It is concluded that the end bearing cannot be sufficiently generated using the current Statnamic test capacity (about 3000 tons).

4.4 COMPARISON of OSTERBERG and STATNAMIC TEST RESULTS

4.4.1 Skin Friction Analysis and Summary

The frequency distribution of the unit skin frictions for all the reduced Osterberg and Statnamic data is shown in Figure 4.21. The individual ultimate and mobilized unit skin frictions by site are given in Figures 4.22-2.28. It should be noted that the mean of fully mobilized unit skin frictions is biased toward the Osterberg data since the majority of data is from the Osterberg tests (number of Osterberg data: 35, number of Statnamic data: 2). However, the mean of partially mobilized unit skin frictions is equally influenced by both Osterberg and Statnamic data (number of Osterberg data: 28, number of Statnamic data: 29).

According to Figure 4.21, the mean and median of fully mobilized skin frictions are close to those of the partially mobilized skin frictions. However, the standard deviation for fully and partially mobilized is different by 2 tsf: the standard deviation for fully mobilized friction is 3.98 tsf and the standard deviation for partially mobilized friction is 6.00 tsf. The difference indicates that once skin friction reaches failure, the majority of ultimate unit skin frictions will likely be between 3 and 11 tsf (one standard deviation from the mean).

The mean unit skin friction and the standard deviation by site are shown in Figure 4.29. In this figure, six projects (17^{th} bridge is divided into two areas due to the variability on each side of the channel) are shown. Some of the sites had only Osterberg or Statnamic tests and others had both tests (see Figure 2.1). Due to lack of data, all the partially mobilized and fully mobilized unit skin frictions are combined and analyzed. Again, the mean in Figure 4.29 is biased toward the Osterberg results due to the number of data: 63 unit skin frictions values for Osterberg (including fully and partially mobilized frictions) and 31 Statnamic values (including fully and partially mobilized frictions).

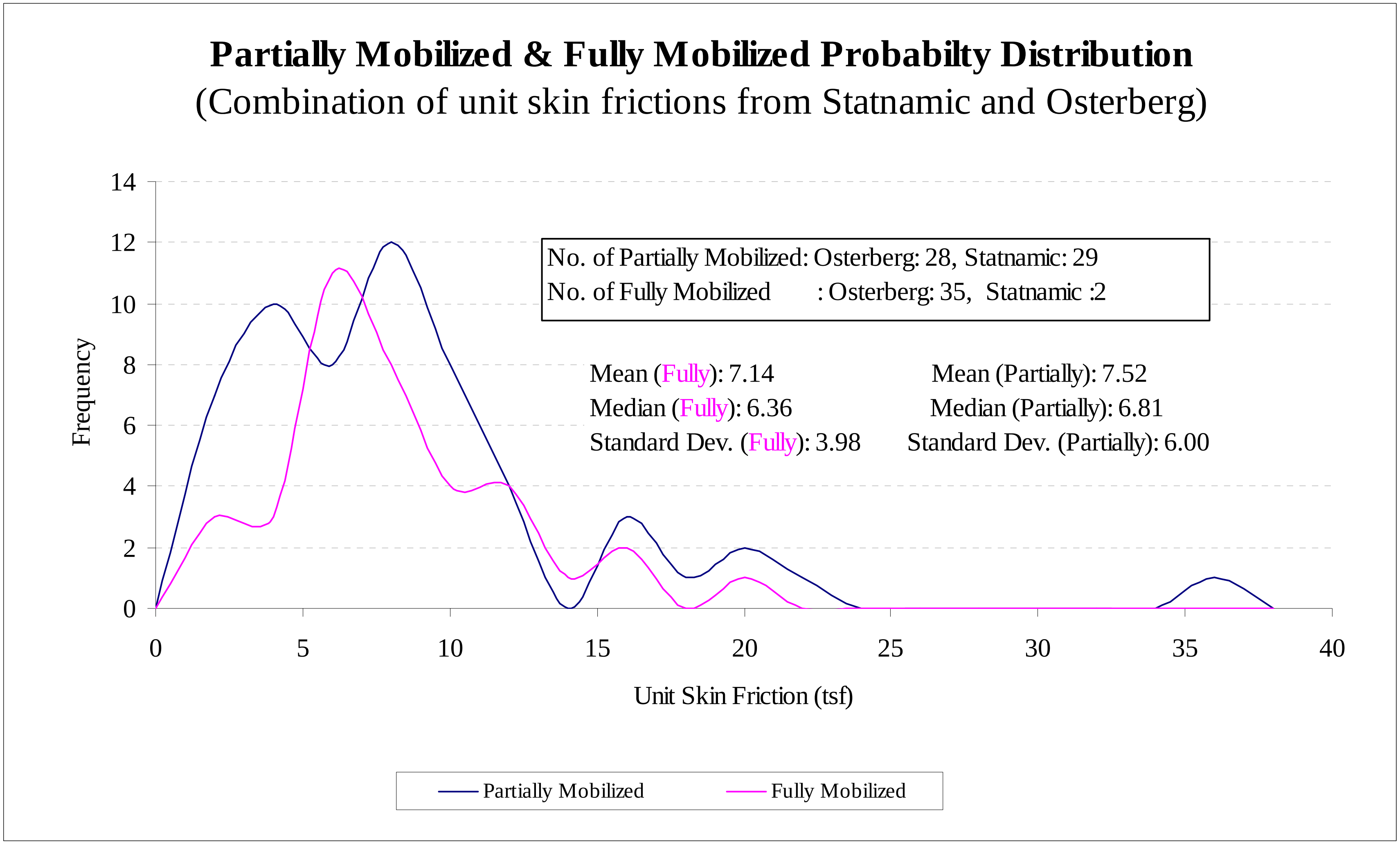


Figure 4.21 Combined Osterberg and Statnamic Unit Skin Friction Probability Distributions

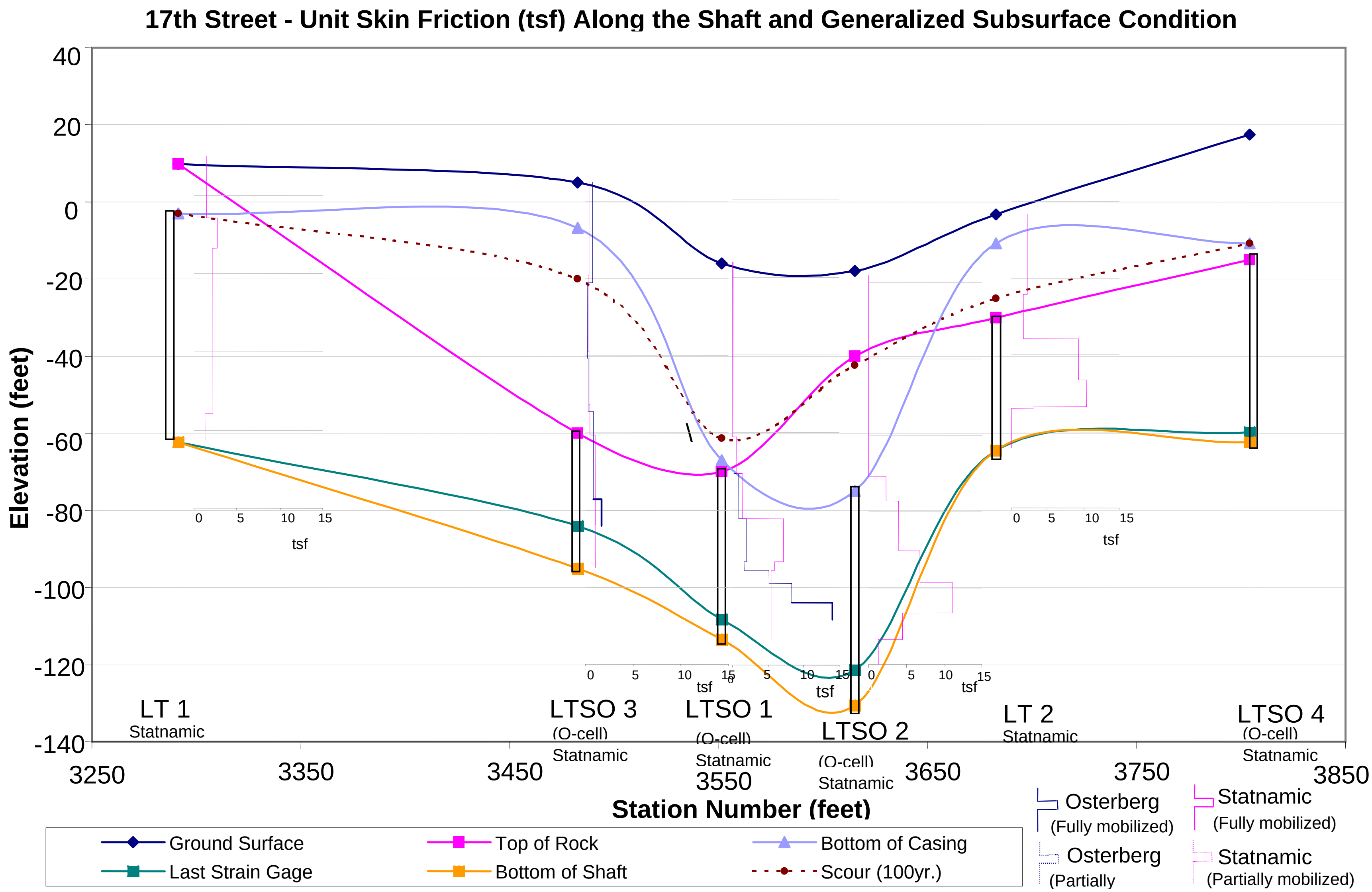


Figure 4.22 Unit Skin Friction Along Each Shaft from Osterberg and Statnamic Testing, 17th Bridge

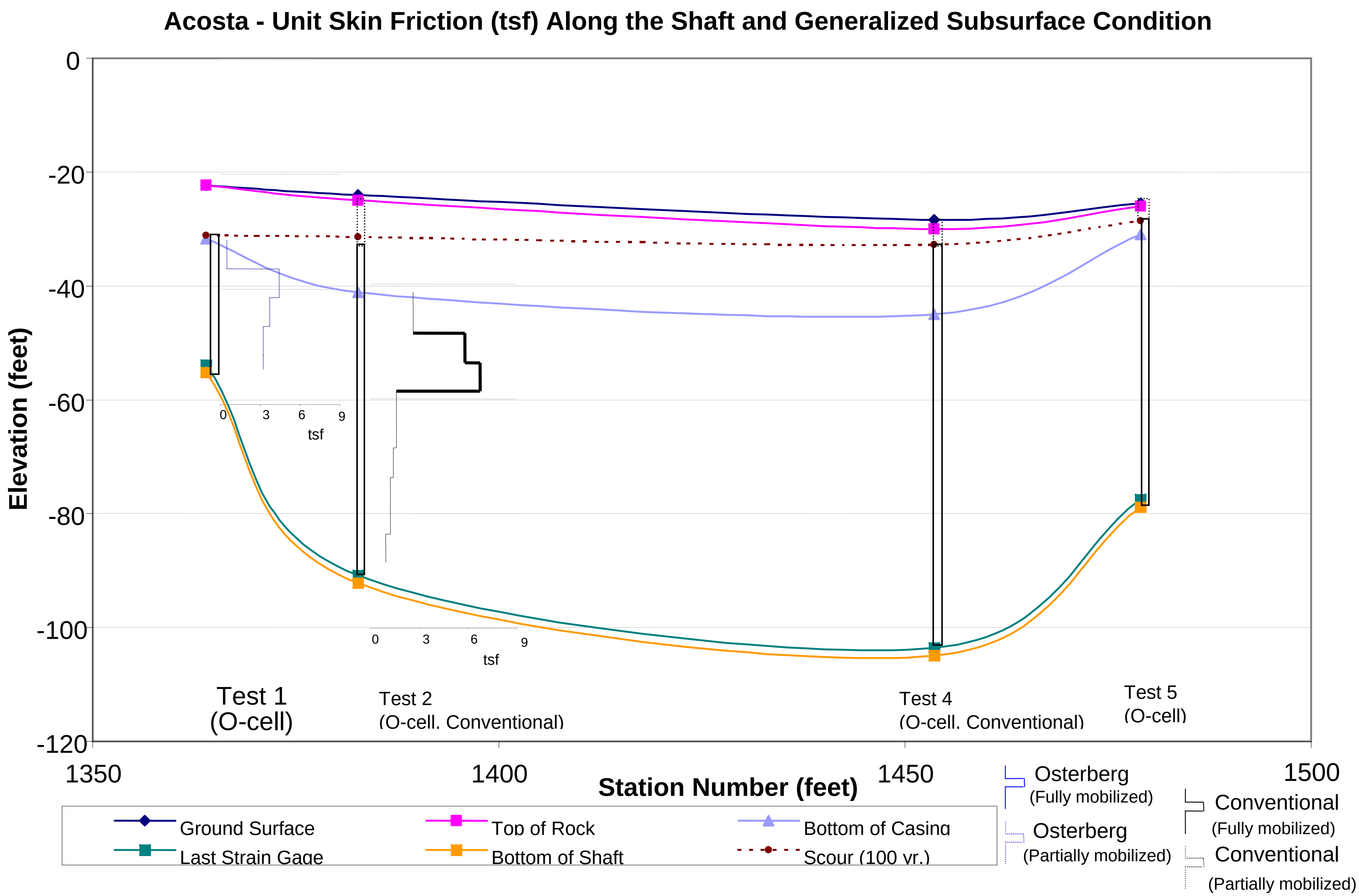


Figure 4.23 Unit Skin Friction Along Each Shaft from Osterberg and Statnamic Testing for Acosta Bridge

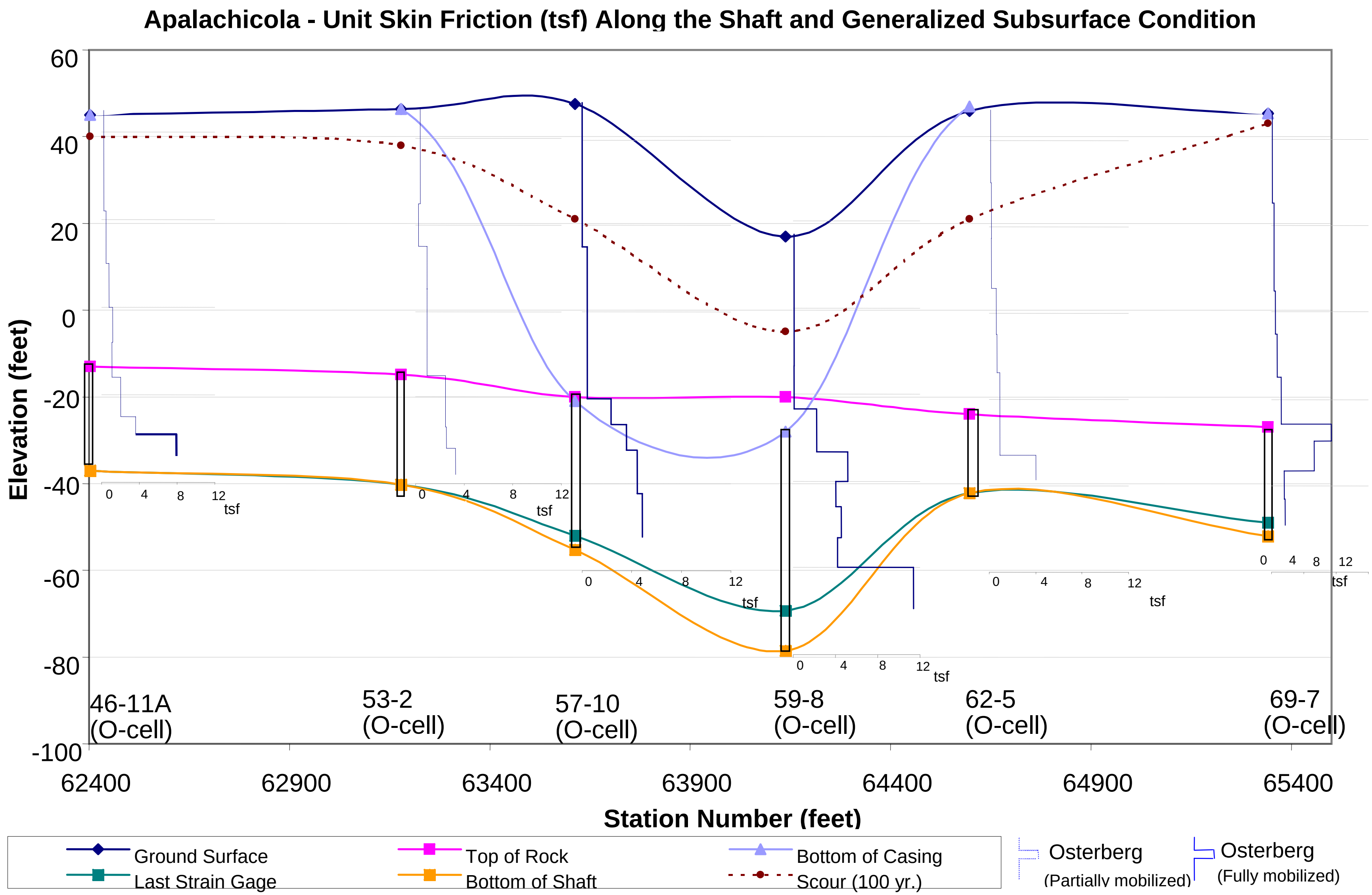


Figure 4.24 Unit Skin Friction Along Each Shaft from Osterberg and Statnamic Testing, Apalachicola Bridge

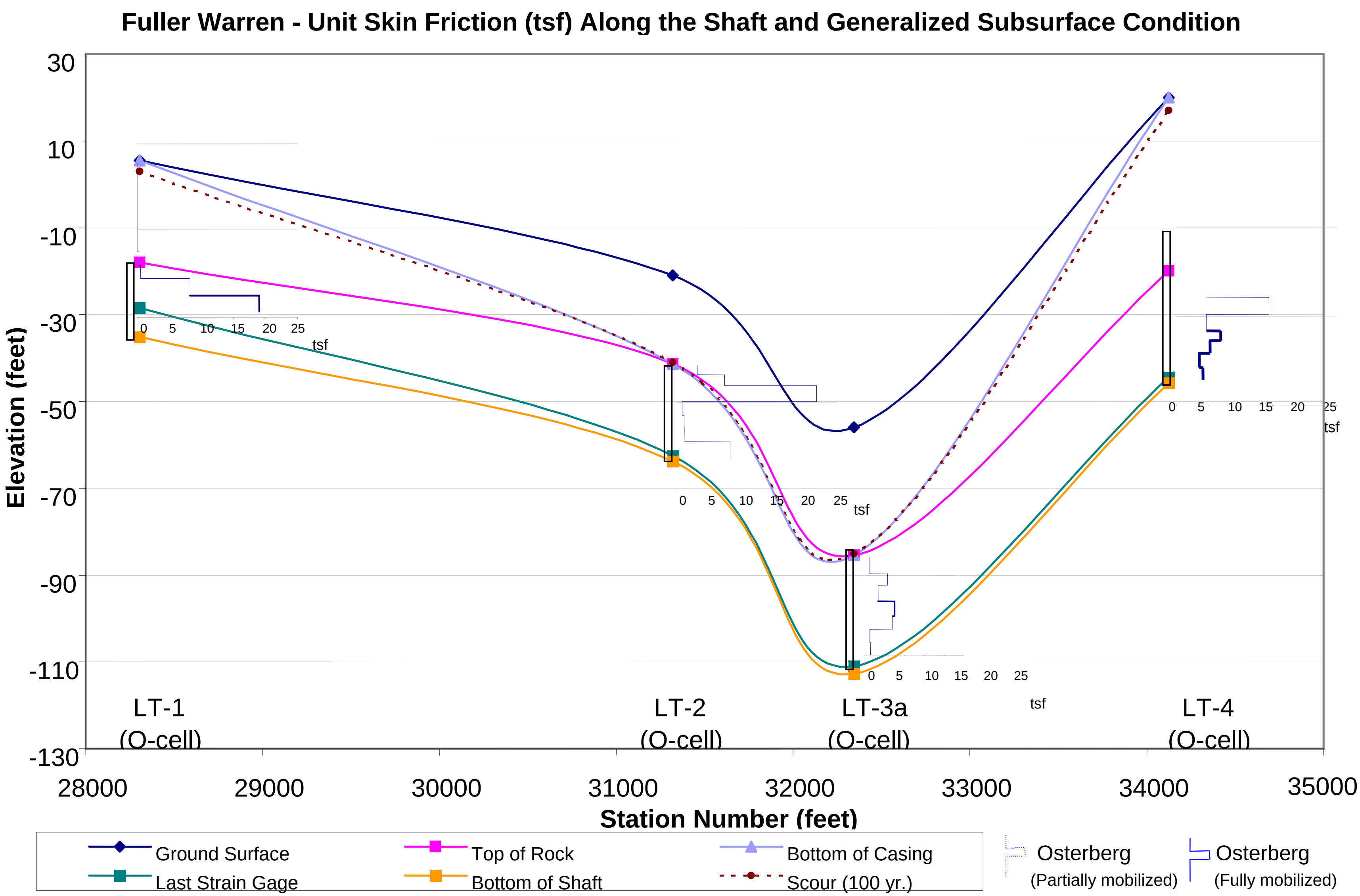


Figure 4.25 Unit Skin Friction Along Each Shaft from Osterberg and Statnamic Testing, Fuller Warren Bridge

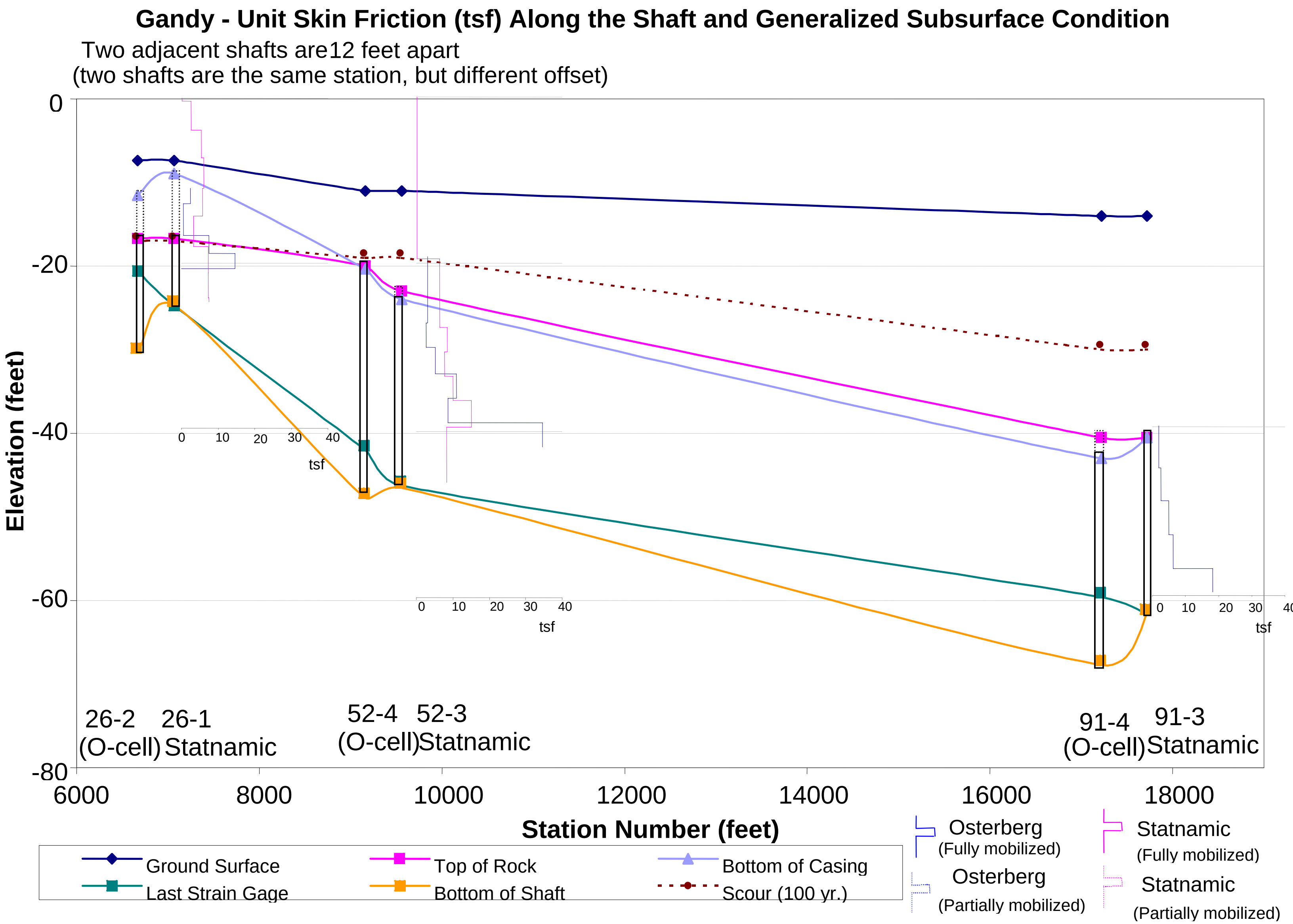


Figure 4.26 Unit Skin Friction Along Each Shaft from Osterberg and Statnamic Testing, Gandy Bridge

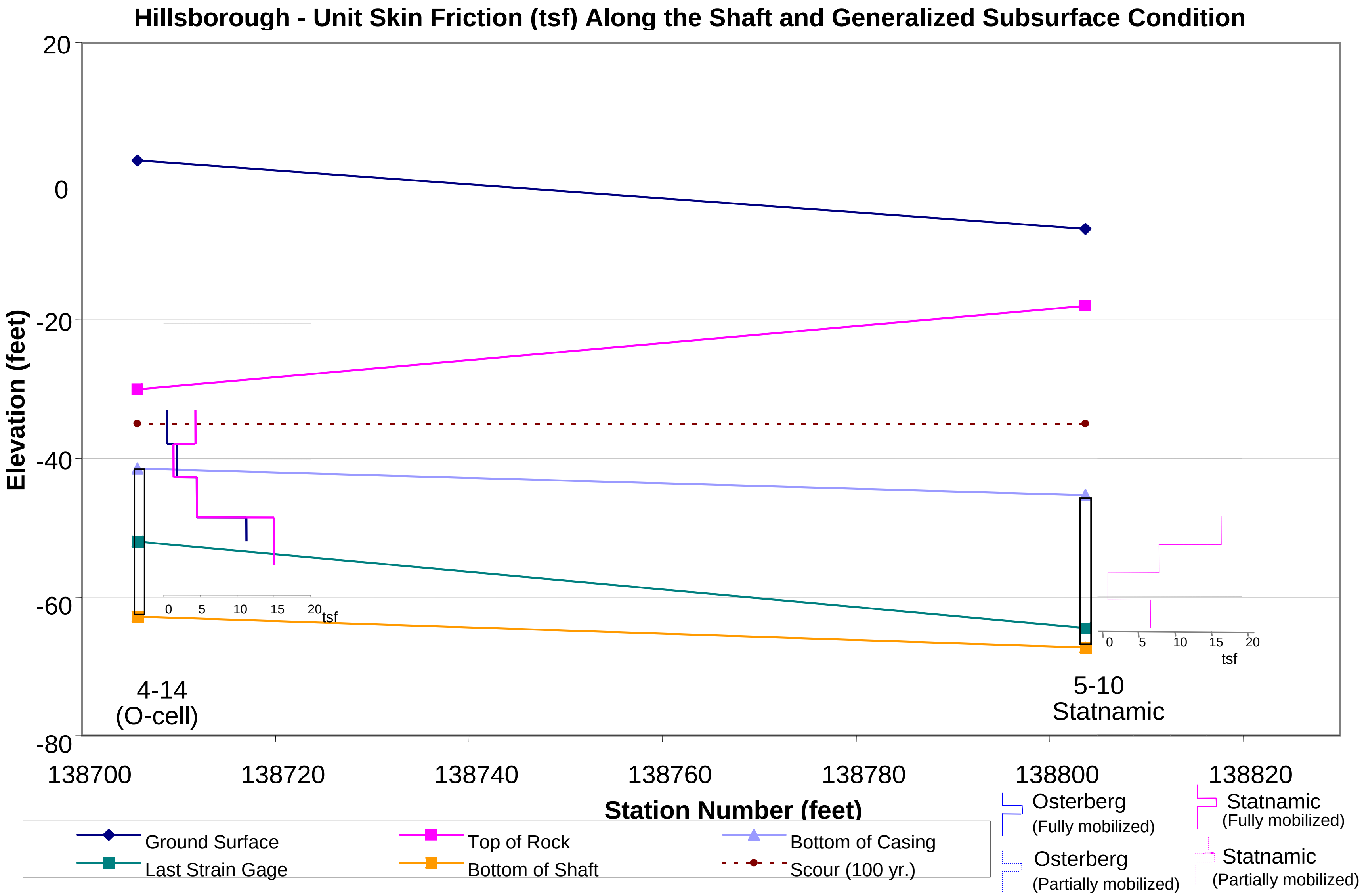


Figure 4.27 Unit Skin Friction Along Each Shaft from Osterberg and Statnamic Testing, Hillsborough Bridge

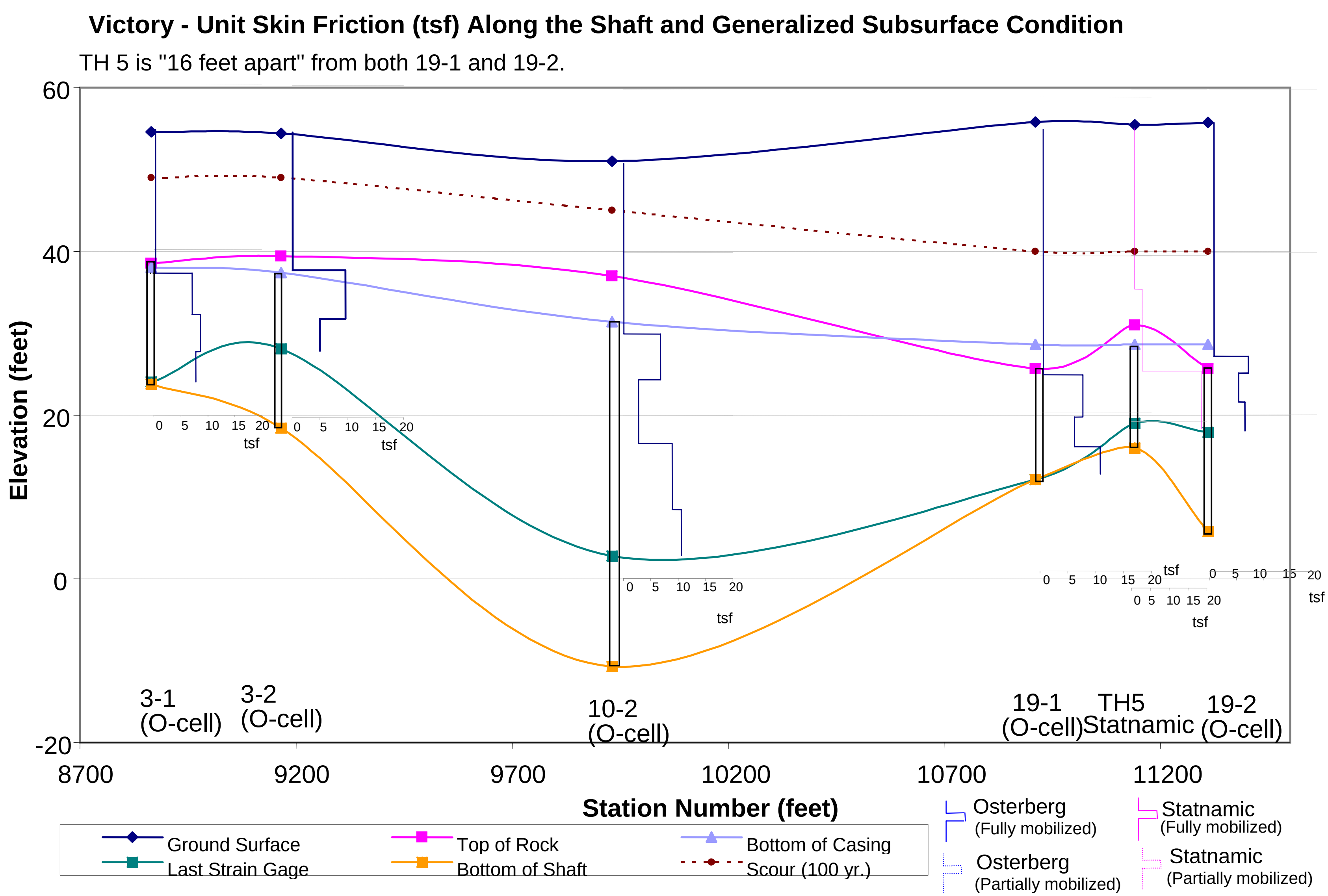


Figure 4.28 Unit Skin Friction Along Each Shaft from Osterberg and Statnamic Testing, Victory Bridge

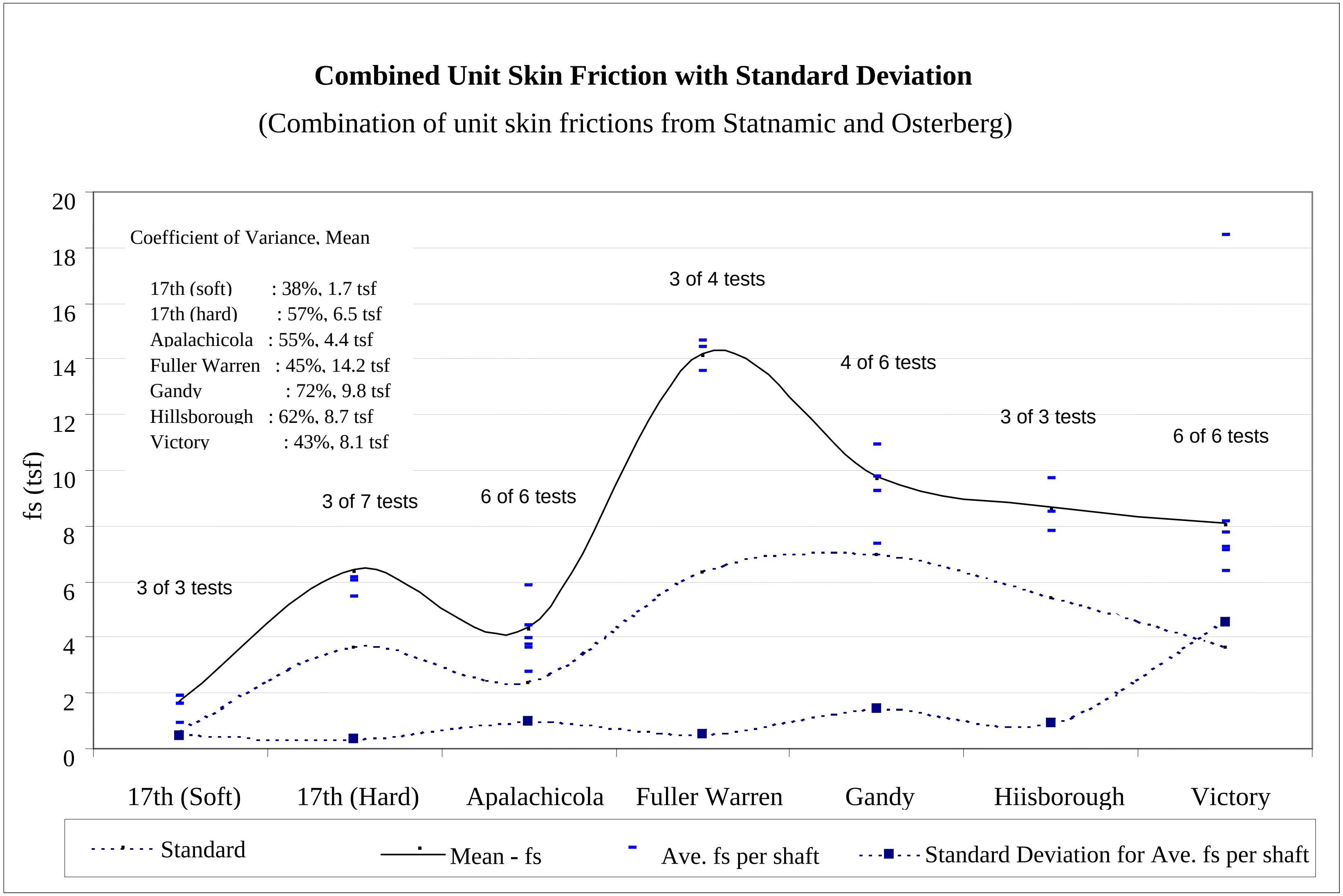


Figure 4.29 Combined Osterberg and Statnamic Unit Skin Friction with Standard Deviation

An average unit skin friction for each shaft is calculated. These average skin frictions are shown in Figure 4.29 as dashed points. It should be noted that the dashed points represent only average skin friction for each test shaft that is considered to be close to ultimate skin failure. Figure 4.13 was used to quantify how close each test shaft was to ultimate failure (skin friction). The test shafts, which are equal to or higher than 80% of ultimate skin friction in Figure 4.13 (y-axis, $fs/fs_{ultimate}$), was the cutoff consideration for ultimate failure. The number of shafts that reached more than 80% of ultimate failure is shown above the dashed points as a number out of total number of load tests performed. The dashed points represent the variability of the site in terms of an average skin friction for each shaft and its prediction is of great interest for design. The dashed points will be used in assessment of LRFD resistance factors, ϕ, for current FDOT design of unit skin friction in Florida Limestone in chapter eight. The variability of the dashed points will also be predicted in chapter eight based on the laboratory data.

4.4.2 End Bearing Analysis and Summary

Since most of the Statnamic tests did not reach the FDOT failure criterion, a direct comparison of Osterberg and Statnamic unit end bearings was not possible. In general, the comparison of Tables 4.1 and 4.2 shows that the unit end bearing from the Statnamic test are smaller than the Osterberg due to the smaller measured tip displacements.

CHAPTER 5

COMPARISON OF ULTIMATE SKIN FRICTION FROM OSTERBERG AND STATNAMIC LOAD TESTS

5.1 GENERAL

As part of this research, a comparison of the capacity predictions based on Osterberg and Statnamic were sought. From the collected data, a total of six shafts were found in which a comparison of Osterberg and Statnamic results could be obtained: LTSO1 at 17th Street Bridge; LTSO3 at 17th Street Bridge; 26-1 at Gandy Bridge; 54-3 at Gandy Bridge; 4-14 at Hillsborough Bridge; and TH5 at Victory Bridge. For shafts LTSO1, LTSO3, and 4-14, both Osterberg and Statnamic tests were performed on the same shafts. The others involved comparison shafts, i.e., 26-1 vs. 26-2; 54-3 vs. 54-4; 91-3 vs. 91-4; and shafts TH5 vs. 19-1&2. The distance separating each of the comparison shafts were no more than 20 ft (configurations given in Figures 5.1 and 5.2).

For the comparison, i.e., Osterberg vs. Statnamic, two different approaches were employed: 1) point values of unit skin frictions along the shafts were compared, and 2) average unit skin frictions for the whole shaft were compared. For the average unit skin friction comparison (2), the total capacities in tons along the same lengths of shaft were investigated. The following sections describe each site with the comparison following.

5.2 17TH STREET BRIDGE

Even though Osterberg and Statnamic tests were each performed on LTSO1, LTSO2, LTSO3, and LTSO4, only LTSO1 and LTSO3 were compared. In the case of the LTSO2 and LTSO4 Osterberg tests, very little vertical shaft movement was recorded.

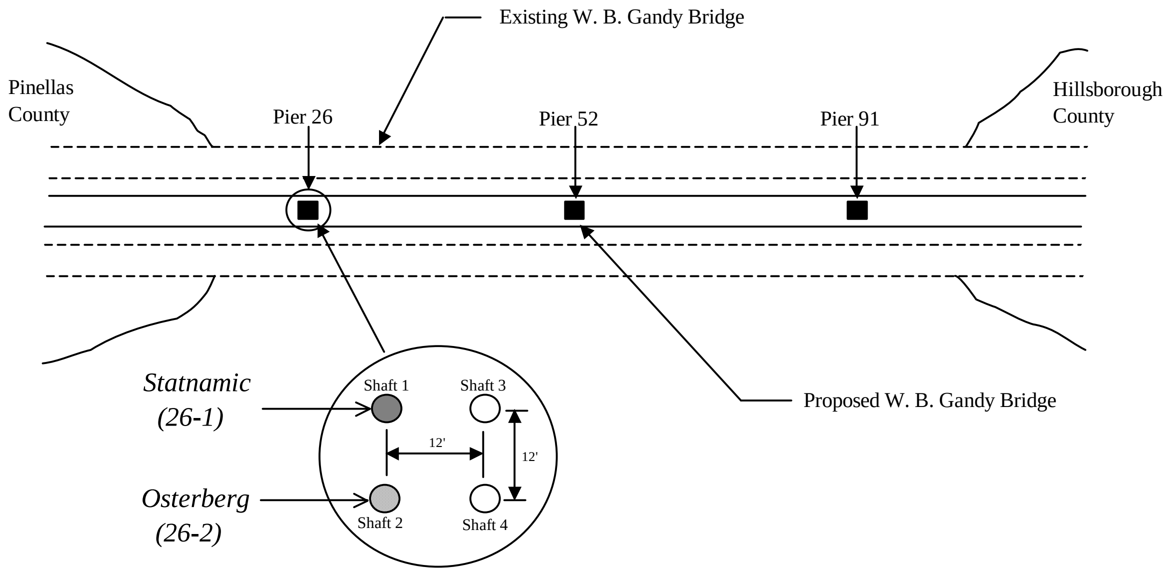


Figure 5.1 Gandy Load Test Location Plan

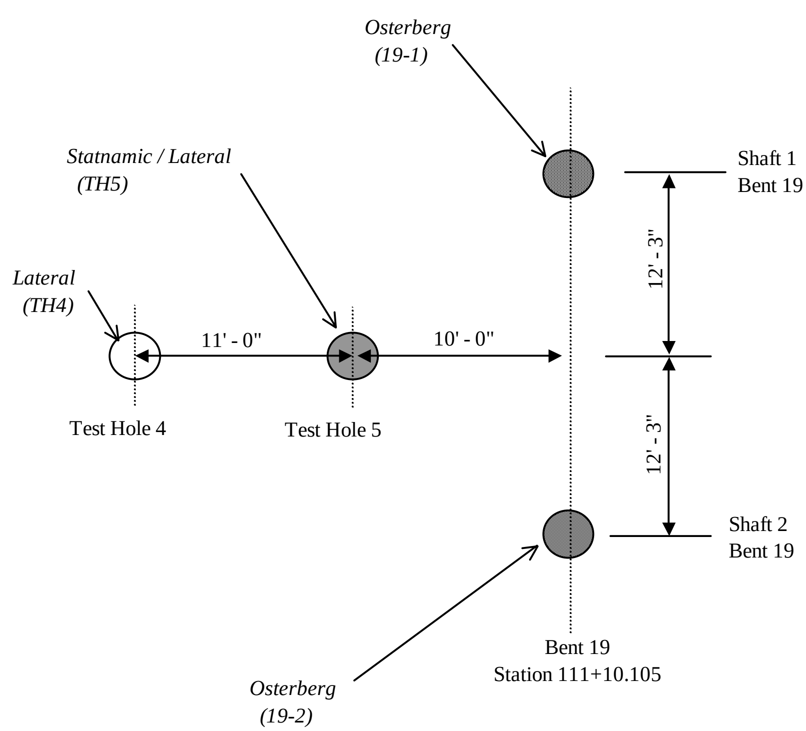


Figure 5.2 Victory Load Test Location Plan

Consequently the ultimate skin frictions did not develop for the latter shafts, and could not be compared to Statnamic, which had different equivalent static load. The unit skin frictions along shafts LTSO1 and LTSO3, which did fail, are presented in Figure 5.3 for the Osterberg and Statnamic tests. Evident from the figure, Statnamic and Osterberg unit skin frictions are similar along the length of LTSO3 but they are different for LTSO1 (see 5.61 discussion).

5.3 GANDY BRIDGE

The Osterberg and Statnamic tests were conducted at Pier 26, 52, and 91, and illustrated in Figure 5.1. Three 3000-ton Statnamic tests were performed on adjacent shafts (12 feet north) near the Osterberg shafts.

The unit skin frictions along the shafts from the Osterberg and Statnamic tests are shown in Figure 5.4. From the plot, the unit skin frictions at the same elevation agree in some areas but not in others. The Statnamic skin frictions at 91-3 are not drawn since the Statnamic data for this shaft was not provided in the load test report. Evident from Figure 5.4, the Osterberg tests (52-4, 26-2) transferred larger percentage of skin friction to the rock just above the Osterberg cell vs. the Statnamic test. The latter may be attributed to the Poisson Effect.

5.4 HILLSBOROUGH BRIDGE

Both Osterberg and Statnamic tests were performed on Pier 4, shaft 14. The Statnamic test had a maximum applied load of 3,287 tons with a maximum top displacement of 1.63 inches and a permanent displacement was 1.06 inches. With the latter movements, the ultimate unit skin frictions were obtained and are shown in Figure 5.5. Evident from the figure, the point unit skin friction from Osterberg and Statnamic compare favorably with one another.

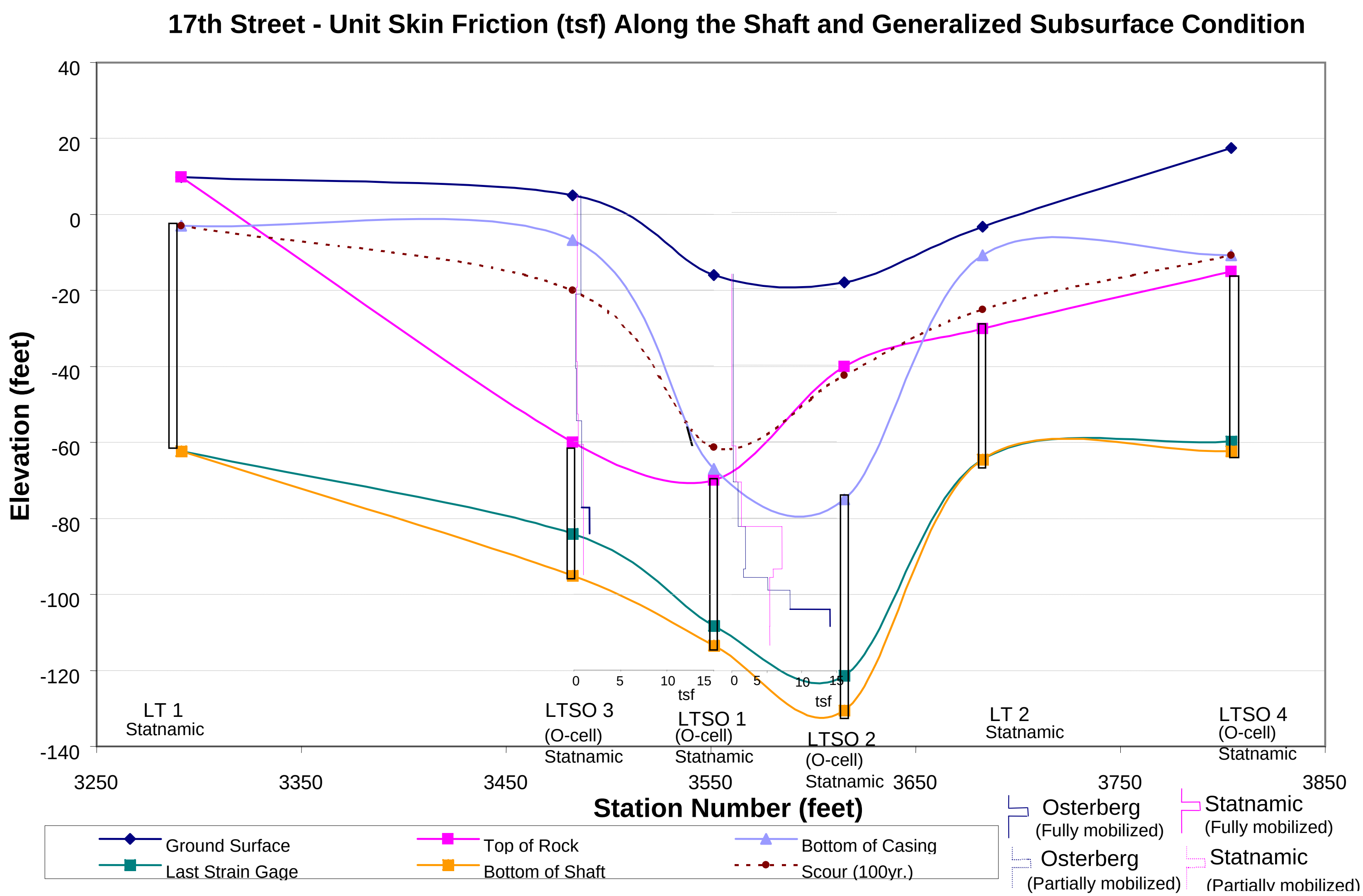


Figure 5.3 Comparisons of Osterberg and Statnamic Unit Skin Friction on 17th Bridge

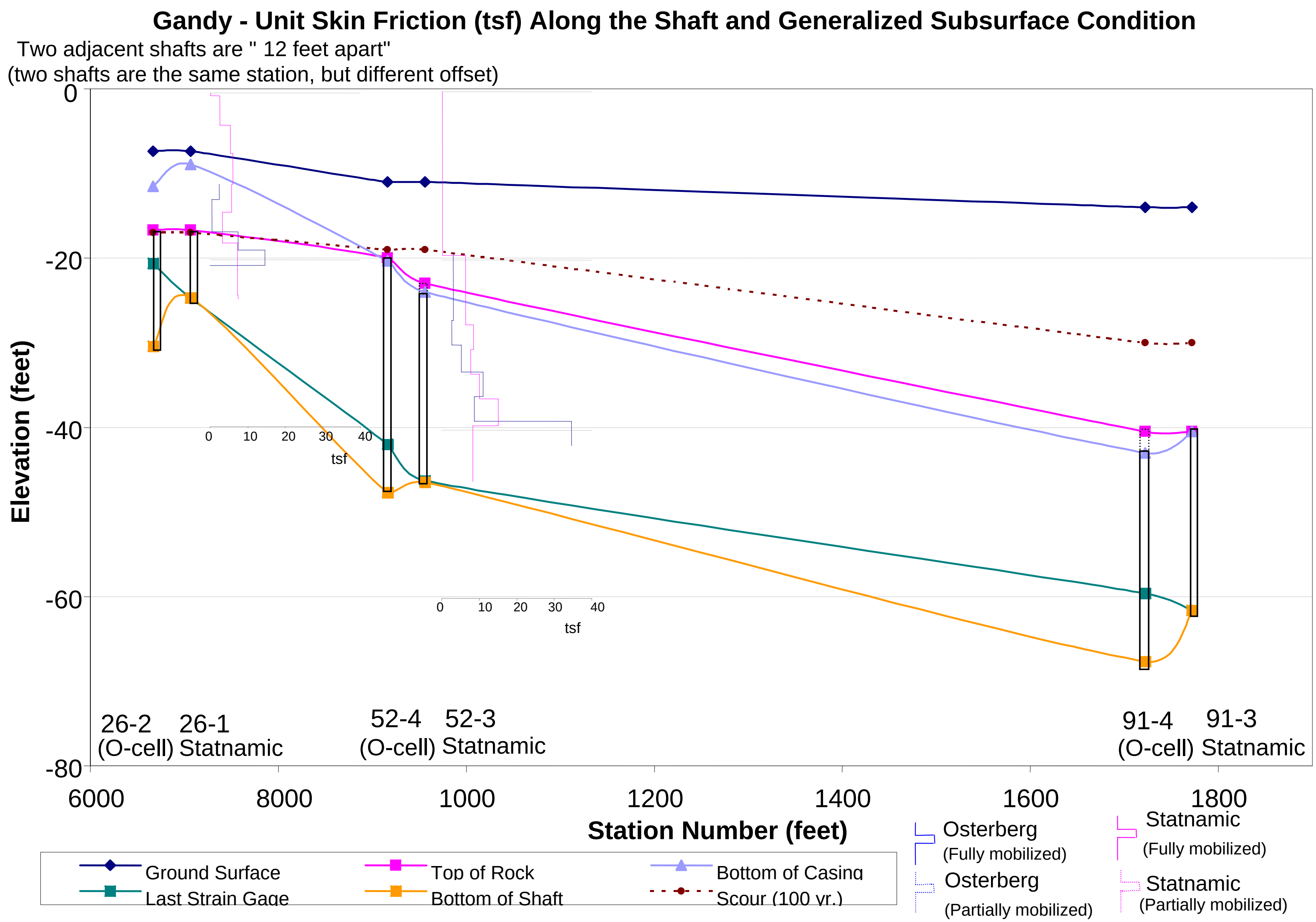


Figure 5.4 Comparisons of Osterberg and Statnamic Unit Skin Friction on Gandy Bridge

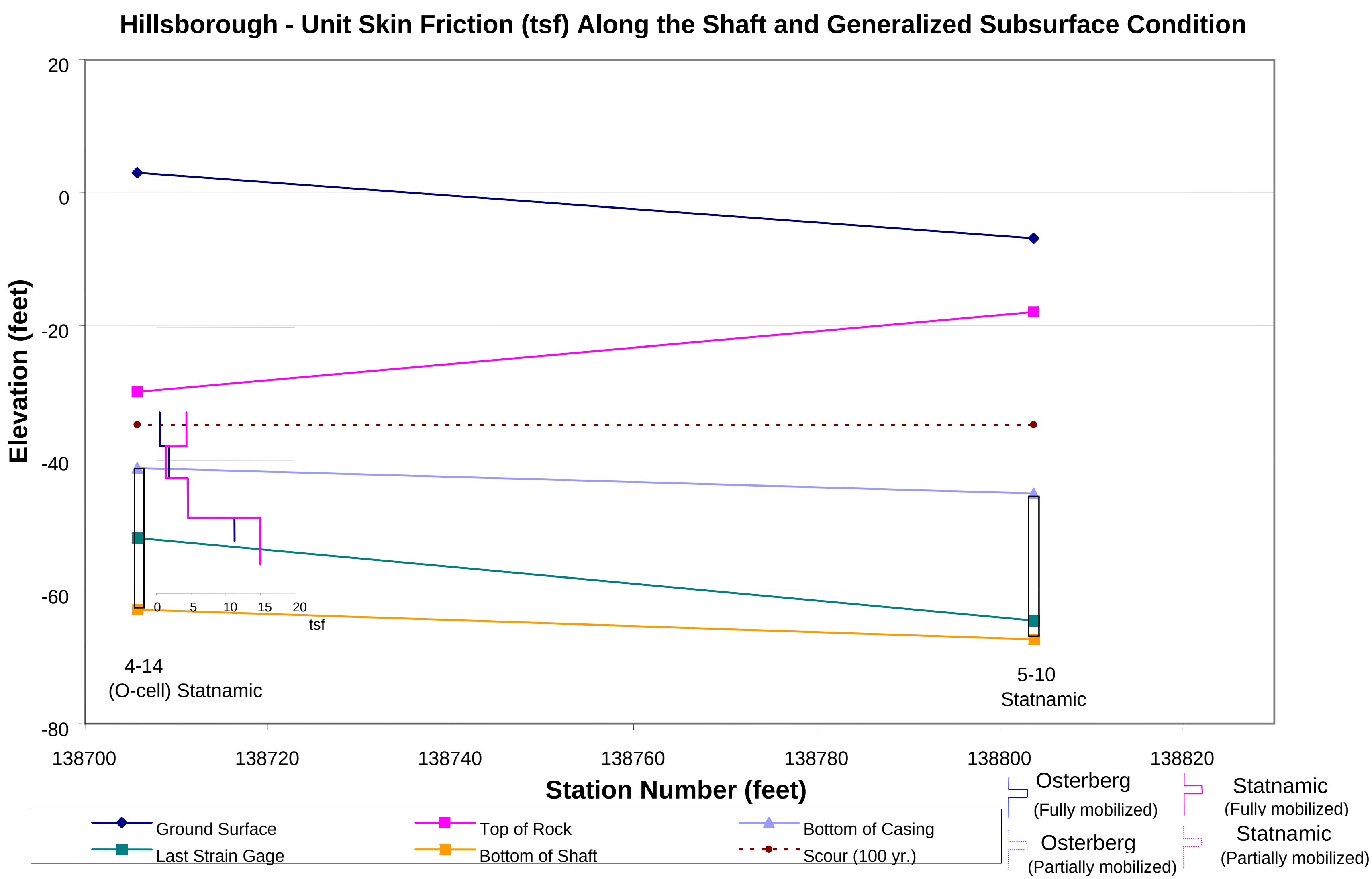


Figure 5.5 Comparisons of Osterberg and Statnamic Unit Skin Friction on the Hillsborough Bridge

5.5 VICTORY BRIDGE

For this site, two Osterberg tests were performed at piers 19-1 and 19-2, and one Statnamic test was performed at test location TH5, as illustrated in Figure 5.2. The distance separating the Osterberg and Statnamic tests was approximately 16 feet. The Osterberg and Statnamic unit skin frictions on each shaft are given in Figure 5.6 with the Statnamic superimposed on the Osterberg results. It is evident that the Osterberg unit skin friction for shafts 19-1 and 19-2 are smaller than the Statnamic values for TH5. According to the Statnamic load test report, the discrepancy between the Osterberg and Statnamic was due to the different soil properties near the test shafts.

5.6 ANALYSIS AND SUMMARY OF COMPARISON

5.6.1 Comparison Unit Skin Frictions Along the Shafts

In this comparison, the Osterberg and Statnamic loads were converted to average unit skin frictions from the strain gauge locations along the shafts. The unit skin friction was obtained by dividing the load change by the effective surface area between any two adjacent gauges. Only the unit skin frictions from the limestone (rock socket) were compared. The results for Pier 26, Shaft 2 in Gandy Bridge is shown in Figure 5.7. A total of 19 pairs of unit skin friction were generated from four different sites, 17th Street, Gandy, Hillsborough, and Victory Bridges. The 19 generated pairs (Osterberg vs. Statnamic) are plotted in Figure 5.8. The data points above the 45-degree line (10 points) identify that the Statnamic unit skin frictions are higher than Osterberg unit skin frictions at the same elevations. The points falling below the 45 degree line (6 points) represent the opposite (Osterberg unit skin friction > Statnamic unit skin friction). Four points fall on the line indicating that both tests developed the same unit skin frictions at the same elevation. As shown in Figure 5.8, the 19 points are scattered resulting in a coefficient of correlation of 8.8 %.

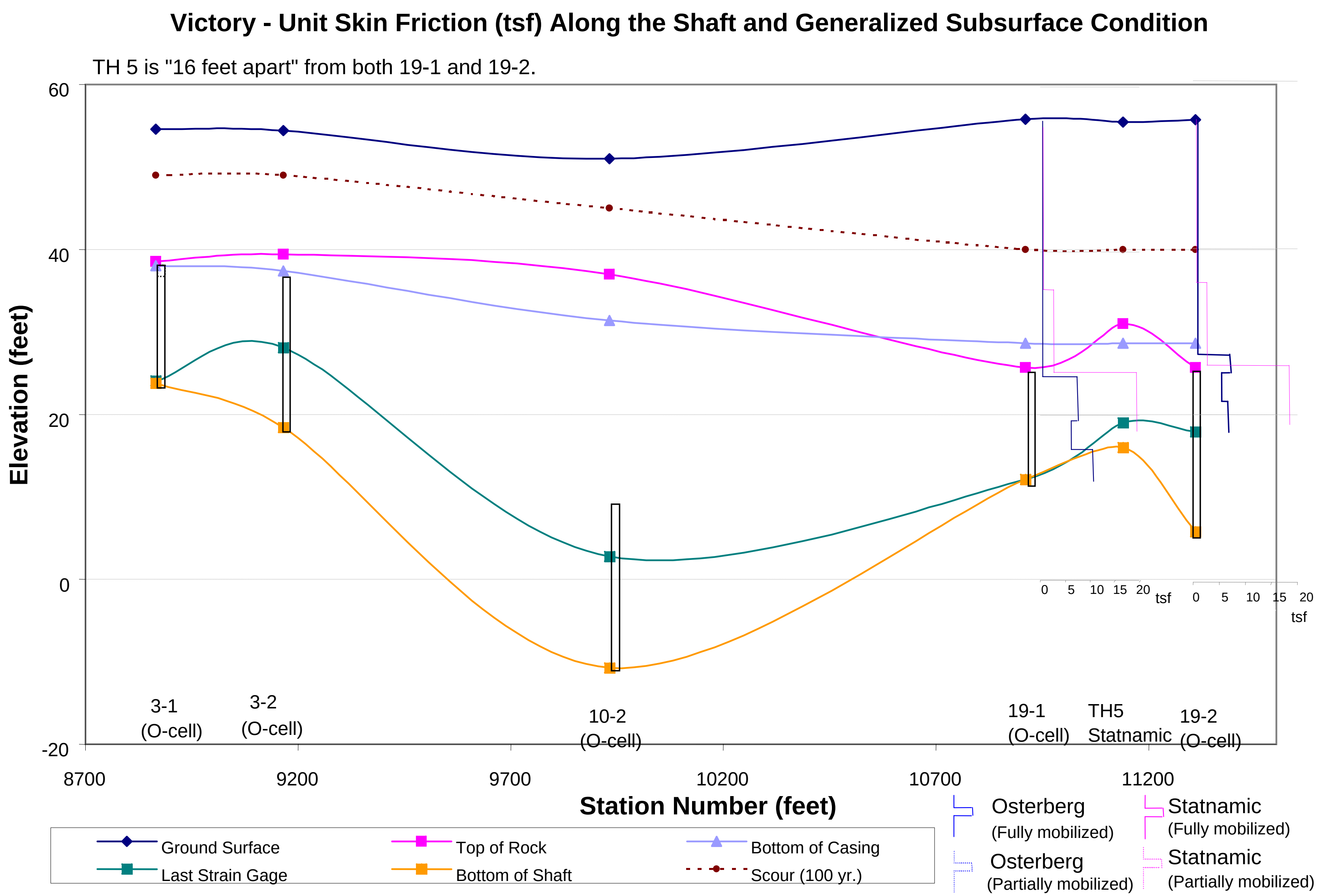


Figure 5.6 Comparisons of Osterberg and Statnamic Unit Skin Friction on the Victory Bridge

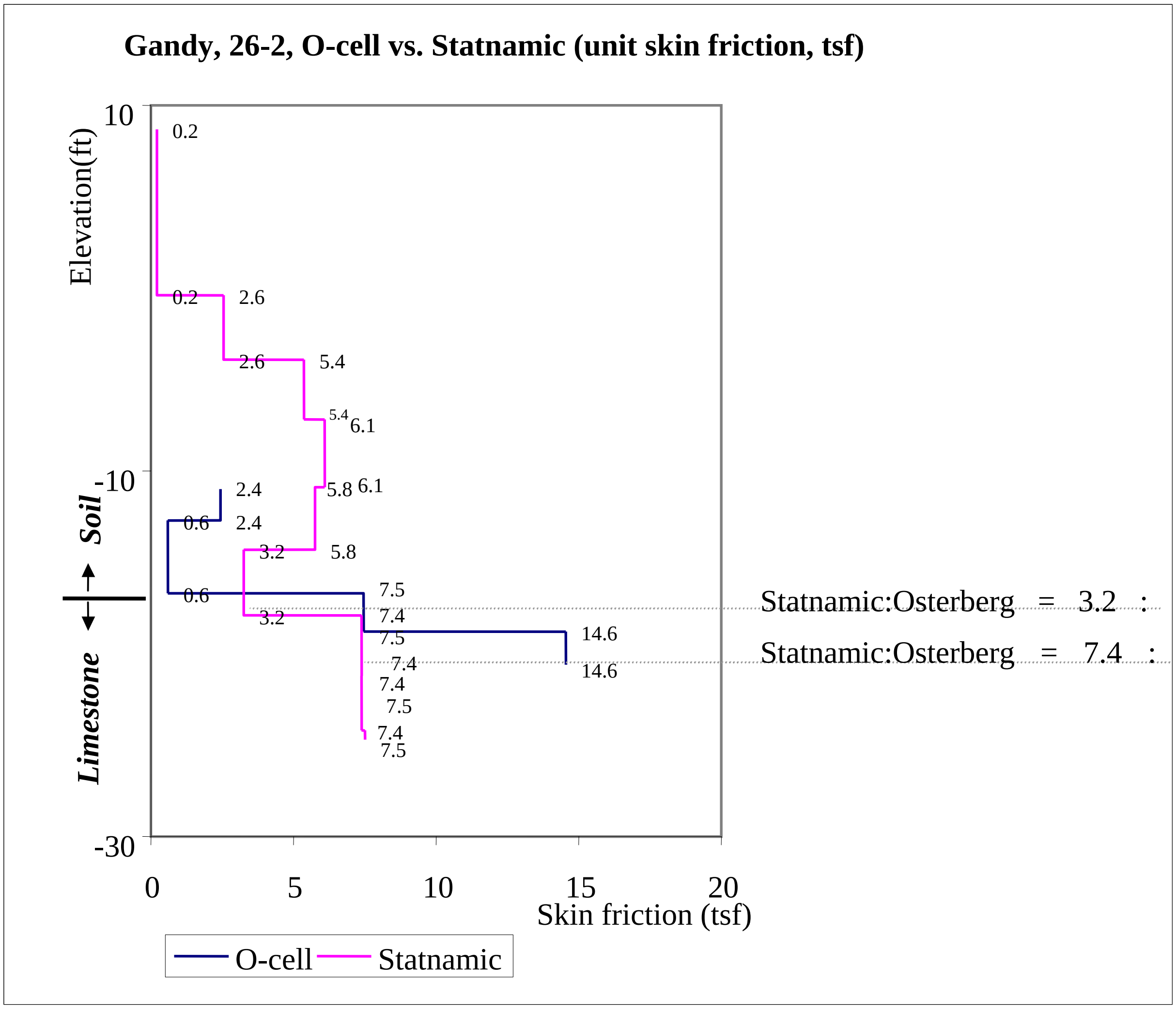


Figure 5.7 Osterberg vs. Statnamic Unit Skin Friction Along Shaft 2, Gandy Bridge

It is concluded that the Osterberg and Statnamic load tests exhibit little if any point to point correlation along the shafts. This could be due to two possible reasons.

First, there exists spatial variability in the limestone, which physically separates the shafts. For instance, in this comparison, both the Osterberg and the Statnamic tests from the Gandy and Victory were spaced apart as shown in Figure 5.1 and 5.2, respectively. The second difference could be a result of the testing mechanisms themselves. Specifically, the Osterberg device generates an up-lifting force, whereas, the Statnamic test generates a top-down force. For

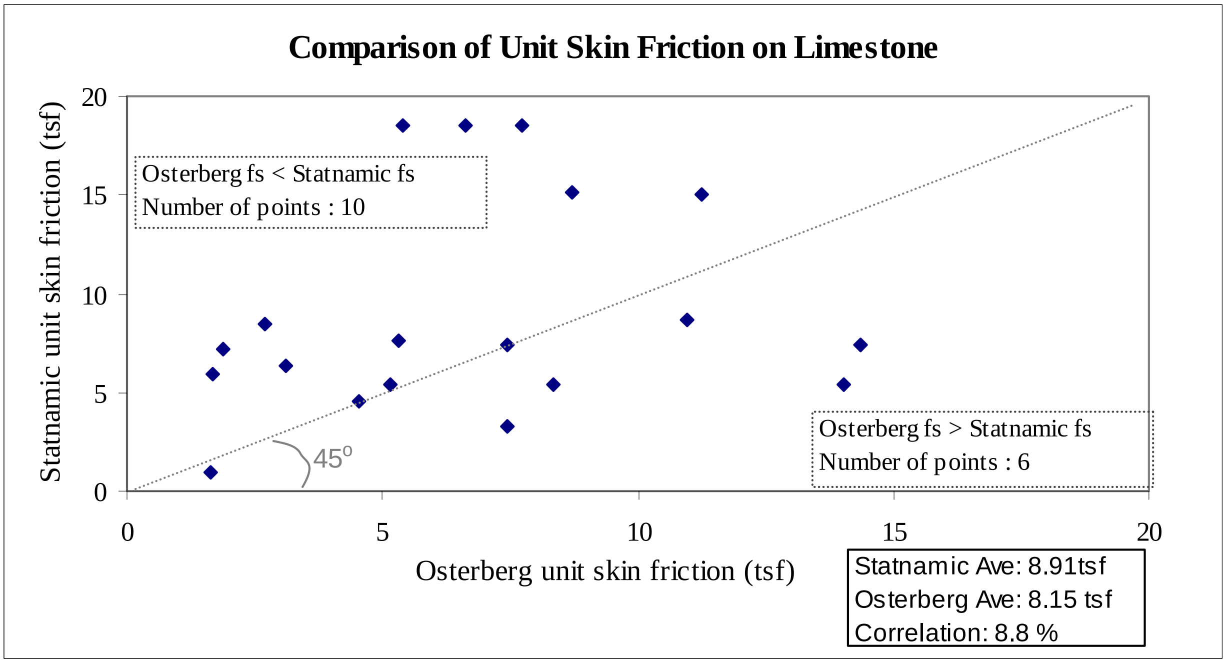


Figure 5.8 Comparison of Unit Skin Friction in Limestone

a number of the tests, i.e., Figures 5.4 and 5.6, it was observed that the Osterberg tests developed higher skin friction than the Statnamic tests near the Osterberg cell, whereas, the Statnamic test generated higher skin friction near top of rock (Fig. 5.4). The latter may be explained by the pressure sensitive nature of the limestone, i.e., the higher the vertical load in the shaft, the higher the horizontal stress on the rock (Poisson effect), the higher the unit skin friction. Interestingly, if the 19 unit skin frictions values from the Statnamic and Osterberg tests are averaged, bottom of Figure 5.8, the average Statnamic unit skin friction would be 8.91 tsf, whereas the average Osterberg unit skin frictions would be 8.15 tsf. The latter led to the second phase of the study, i.e., the average unit skin friction (load) for individual shafts.

5.6.2 Comparison of Average Skin Capacity (tons)

For this approach, the total skin capacities (tons) of individual shafts were compared. Since the unit skin frictions along the test shafts were known, they were converted to forces by multiplying them by surface area and summing to obtain total capacities (skin) in tons. It should be noted that the capacities were computed using the same lengths of the shafts for both the Statnamic and Osterberg tests. Figure 5.9 shows a graphical comparison and Figure 5.10 shows a histogram comparison between Osterberg and Statnamic estimated skin capacities.

Evident from Figure 5.9 the correlation of Statnamic and Osterberg skin capacities are much closer than the unit point values (Fig. 5.8). The latter approach reduces the spatial variability, as well as Poisson effects (discussed earlier), with its averaging approach. Also evident from Figures 5.9 and 5.10 are that the capacities from the Statnamic tests are generally either similar or higher than the capacities of the Osterberg tests. However, a strong linear relationship is observed in Figure 5.9, with a correlation coefficient of 83 %. The correlation would be much higher if the Victory site was disallowed. As shown in chapter eight, the latter difference is higher than the spatial differences reported over all the sites.

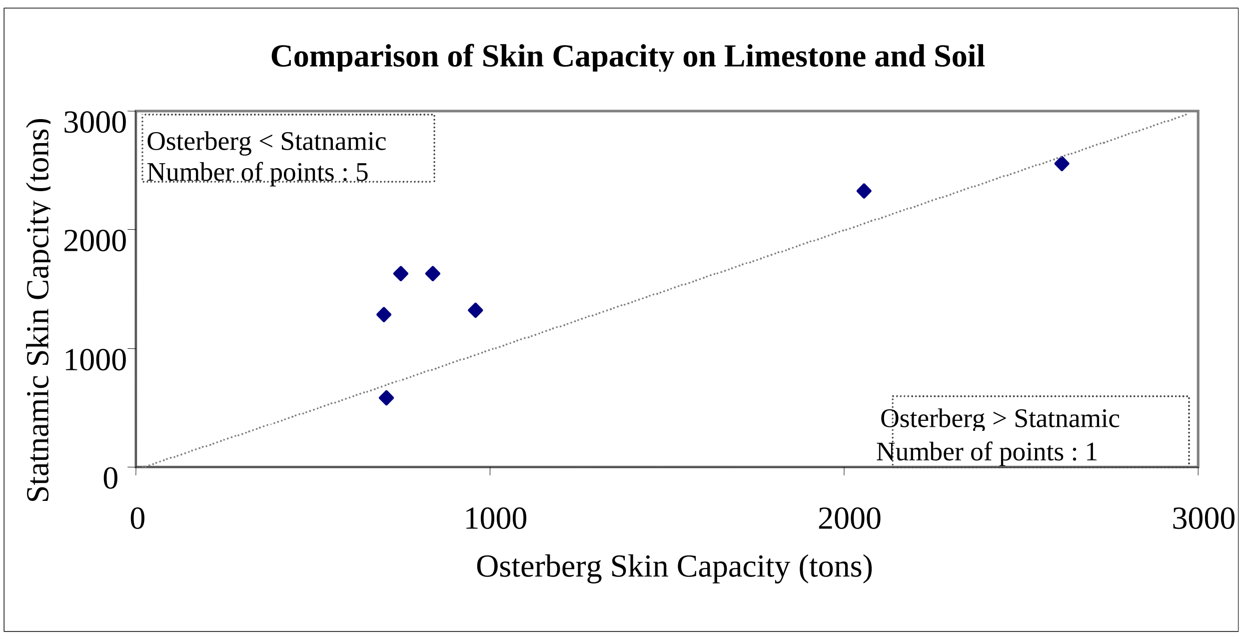


Figure 5.9 Comparisons of Statnamic and Osterberg Skin Capacities in Limestone

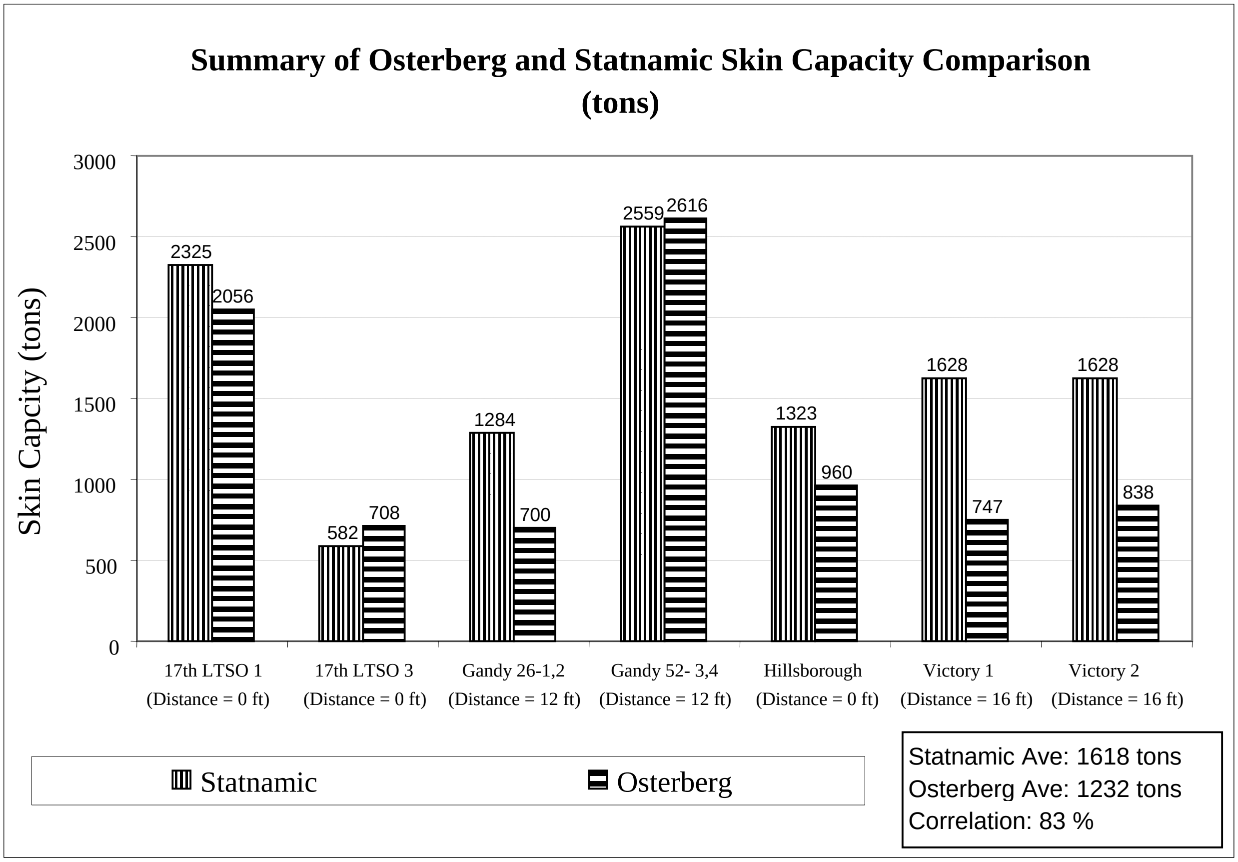


Figure 5.10 Summary Comparisons of Statnamic and Osterberg Skin Capacities

CHAPTER 6
LATERAL LOAD TEST ANALYSES

6.1 GENERAL

A total of 13 lateral load tests were performed at five sites: 17th Street Bridge (2 tests); Apalachicola Bridge (1 test); Fuller Warren Bridge (2 tests); Gandy Bridge (6 tests); and Victory Bridge (2 tests). For all the tests, hydraulic jacks were used to apply the lateral load (Figures 6.1 and 3.7). The tests were generally conducted at sites where significant lateral loads were expected (i.e., ship impact).

From the load test reports, the testing was performed to validate design, establish tip cut-off elevation, as well as ensure good construction practices. For instance, the tip cut-off elevation was generally established from the shaft's displacement (curvature) response in the load test. However, since the foundation are constructed with groups of shafts, their displacement shape will be different than the measured lateral field-test response (i.e., fixed vs. free head). To model the latter, the CEI typically validated the predicted free-headed response, i.e., modifying assumed or measured strength parameters, such as friction angle, soil unit weight, subgrade modulus, cohesion, and ε_{50}, until the free head shaft response is matched, and then predict (i.e., software) the designed fixed head response. The obtained parameters may also be used to further evaluate the bridge foundation design under different load, scour conditions, shaft sizes, and load conditions in the similar soil conditions. This work focused on the quality of match between the measured and predicted response.

FB-Pier (Hoit and McVay, 1996) was developed to design/analyze single or groups of shafts subject to lateral loads. It has the capability of modeling complicated material cross-sections, as well as nonlinear material response, which impacts stiffness and lateral deformation response. The soil and rock properties used in FB-Pier were based on insitu data, recommended

Victory Bridge Load Test on TH-4 and TH-5

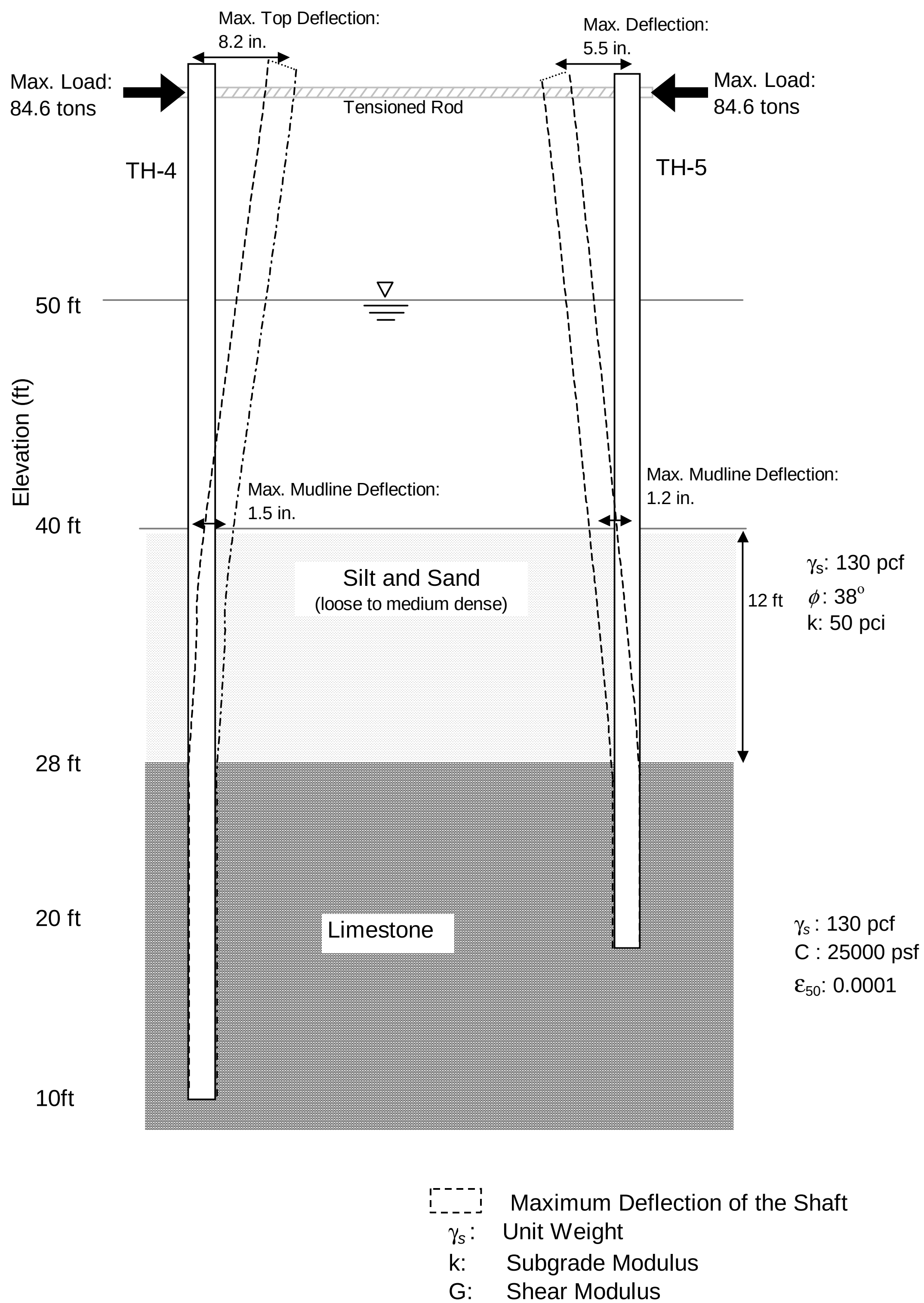


Figure 6.1 Lateral Test Setup, Condition, and Maximum Deflection

correlations (FB-Pier Help), as well as back fitting (free head response). The group response (i.e., fixed head) was simulated using a rotational spring at the top of each shaft. Subsequently, the shear and bending moment along the shaft were compared for single and group response.

Presented in Figures 6.2 and 6.3 are the measured and predicted lateral deflections with and without fixed head conditions. A discussion of each follows.

6.2 17TH STREET BRIDGE

A total of two lateral load tests were performed on May 21, 1998: LLT-1 at Pier 7, Shaft (TH-2); and LLT-2 at Pier 7, Shaft (TH-3). These lateral tests were used to evaluate whether the drilled shafts' minimum tip elevations could be raised. Unfortunately, the load test report did not identify the deflected shape of the shafts, and a FB-Pier analysis could not be performed.

6.3 APALACHICOLA BRIDGE

A single lateral load test was performed at Pier 59, Shaft 8. The load was applied near the top of the test shaft using horizontal jacks pushing against each shaft at Pier 59. The test shaft was a 9-foot diameter shaft constructed near the main pier. Its tip elevation was at –78.5 feet and its top elevation was at 55.5 feet resulting in an overall shaft length of 134 feet. The shaft's unsupported length above the mudline was 37.5 feet. In an effort to partially simulate the anticipated scour condition, an excavation was made around the test shaft at elevation –5 feet (about 22 feet below the mudline). The maximum movement of the test shaft was 12 inches at the maximum load of 325 tons, Figure 6.2.

6.4 FULLER WARREN BRIDGE

A total of two lateral load tests were performed at shafts LLT-1 (station 323+72 75'LT) and LLT-2 (station 323+72, 99'LT). The test shafts were each 6-foot diameter with lengths of

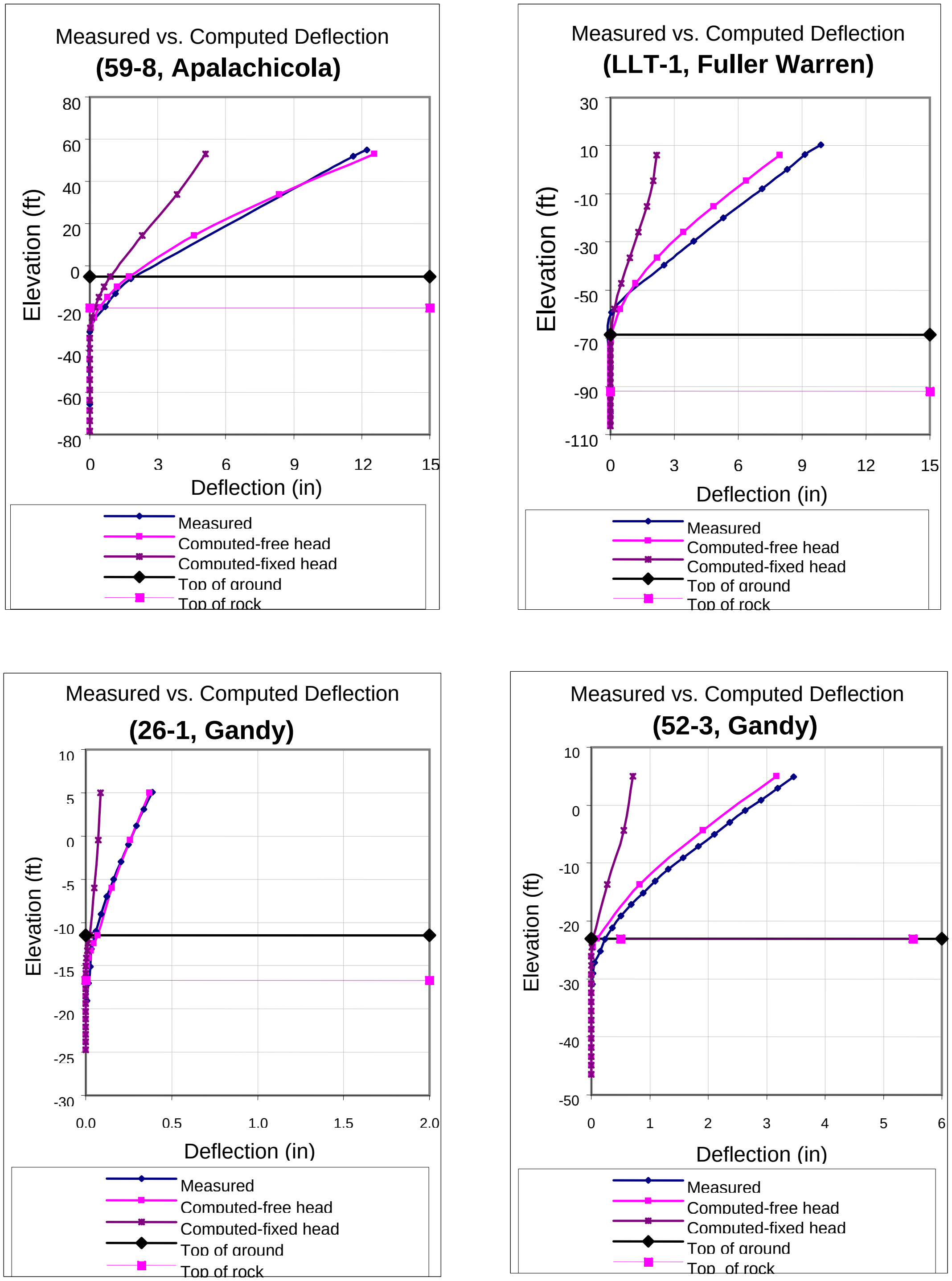
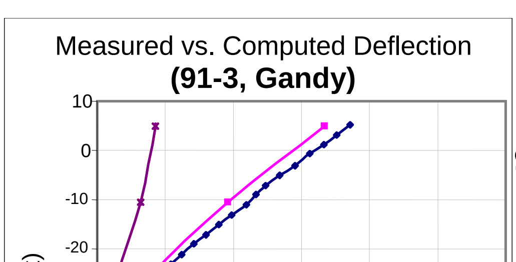


Figure 6.2 FB-Pier: Measured vs. Predicted Lateral Deflection



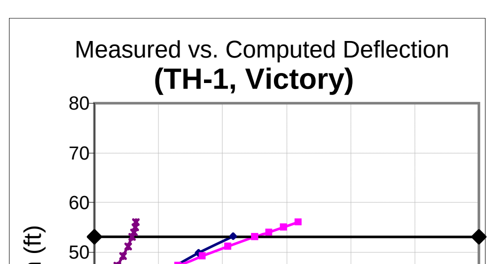


Figure 6.3 FB-Pier: Measured vs. Computed Lateral Response

113.2 feet (LLT-1) and 114.8 feet (LLT-2), respectively. LLT-2 was spaced approximately 24 feet center-to-center from the test shaft LLT-1. A hydraulic jack acting between the two test shafts applied the lateral load. Lateral shaft movements were measured by dial gauges mounted on an independently supported reference beam structure. The maximum test load applied was 36.5 tons resulting in the maximum shaft top movement of 8.5 inches for LLT-1 and 9.3 inches for LLT-2. According to Figure 6.2, most of the deflection occurred above the ground surface since both shafts had 74 feet of unsupported length from the ground surface. Since the deflections of LLT-1 and LLT-2 are relatively close, the deflections of LLT-1 and LLT-2 are averaged and shown as LLT-1 in Figure 6.2.

6.5 GANDY BRIDGE

A total of six lateral load tests were performed at the test site: Pier 26, Shaft 1 against Shaft 2 (26-1 against 26-2); Pier 52, Shaft 3 against Shaft 4 (52-3 against 52-4); and Pier 91, Shaft 3 against Shaft 4 (91-3 against 91-4). As shown in Figure 5.1, the two adjacent shafts were used as reaction shafts (12 feet apart). The test shafts were 4-foot diameter with casings extending down to the scour elevation for each shaft. The designed lateral load was 20 tons. The following describes the maximum load and deflection for each shaft:

Shaft #	Max. load (tons)	Max. deflection (inches)
26-1	23.9	0.2
26-2	23.9	0.7
52-3	60.6	3.7
52-4	60.6	3.3
91-3	40.4	4.0
91-4	40.4	3.6

As shown in Figure 6.2, the deflections for each pair of shafts at a pier are averaged and shown as 26-1, 52-3, and 91-3.

6.6 VICTORY BRIDGE

A total of four lateral load tests were performed at Test Sites 1 (TH-1 against TH-2) and Site 3 (TH-4 against TH-5). The lateral load tests were performed on two adjacent 4-foot diameter shafts (11 feet apart). The location of Test Site 3 (TH-4 & TH-5) is illustrated in Figure 5.2.

The maximum applied loads were 141.5 tons for Test Site 1 and 84.6 tons for Site 3. The corresponding average defection at the top was 3.2 inches for Test Site 1 (TH-1 is an average of TH-1 and 2) and 6.9 inches for Test Site 2 (TH-4 is an average of TH-4 and 5).

The two test shafts were loaded toward each other using a center hole jack and a single tension rod (see Figure 6.1). It is concluded from the load test report that TH-5 was somewhat stiffer than TH-4, even though TH-4 extended to a greater depth into the limestone. The deflected shafts (dashed line) in Figures 6.1 and 6.3 suggest that the displacements within the hard limestone are exceedingly small and thus most of the load transfer occurred within the overlying sands. Therefore, the additional length of the rock socket for TH-4 did not contribute to additional stiffness; very little of the lateral load goes this deep. The different behaviors of the two test shafts were due to stronger soil and rock conditions at TH-5. Figure 6.3 also suggests that bending stresses are the highest just above the hard limestone.

6.7 BACK-ANALYSIS OF LATERAL LOAD TEST DATA AND SUMMARY

The back-analysis of the load tests was carried out using FB-Pier (Hoit and McVay, 1996). The soil and rock parameters, such as friction angle, soil unit weight, subgrade modulus, cohesion, and $\varepsilon_{50,}$ used to generate *p-y* curves were varied to obtain the best-match between the actual and computed displacement at the maximum load. The computed deflections were also considerably dependent on the stiffness of test shafts: thickness of casing, modulus of steel and concrete, and yield stress of steel and concrete. A large number of analyses, changing soil properties and shaft stiffness, were performed, and relatively good agreements were obtained between computed (computed-nonfixed condition) and measured displacements as shown in Figures 6.2 and 6.3. It should be noted that all the analyses were performed with the non-linear shaft option. The suggested soil/rock and shaft parameters are tabulated in Appendix D.

In FB-Pier, Florida limestone is modeled as soft clay with rock strength parameters since no p-y models have been developed to date to characterize Florida Limestone. Appendix D shows the parameters used in FB-Pier. While computing displacements with FB-Pier, it was observed that the response (lateral displacements) of the shafts were mainly dependent on undrained shear strength (C), strain at 50 (ε_{50}: strain at half of the maximum compressive stress) of the limestone. For the best matches, the following ranges of limestone parameters are suggested:

Undrained shear strength (Cu): 18000 ~ 35000 psf
Strain at 50% failure stress, ε_{50} (ε_{50}): 0.0001 ~ 0.0009

The latter parameters are slightly different than Reese (1999), LPILE, suggested parameters, i.e., ε_{50} for weak rock in range of 0.00005 to 0.0005, and undrained shear strength for hard clay approximately 4000 to 8000 psf.

It was also observed that because of relatively small displacements in the competent limestone, the computed deflections were relatively insensitive to outlying strength and strain values (Cu $\geq$ 35000 psf, $\varepsilon_{50} \leq 0.0001$). Also of interest for all of the tests, little if any lateral deflections were observed for any shafts embedded 7 feet or more into the rock formation. However, for this study only one test, Gandy, 91-3, was performed on a shaft, which had limestone at the ground surface.

The shaft tip cut-off elevation was taken as the elevation where the shear and moment become zero in the shaft. The shear and moment were analyzed with and without fixed head condition. In the case of the former, a rotational spring of 5,000,000 kips/in. was attached at the top of each test shaft. The results with and without the spring are shown in Table 6.1. In the table, the maximum shear, maximum bending moment, zero shear elevation, and zero moment elevation were compared between two head conditions (i.e., fixed and free). Evident from the table, the maximum shear and the maximum moment decreased with the fixed head condition (i.e., spring). Moreover, the elevations of zero shear and moment decreased in depth, typically 5 ft. The latter may be explained from the decreased shaft head displacements and rotations for fixed vs. free head model. Since the model without the spring is more conservative (i.e., free head), the latter may be used to set cut-off elevations conservatively.

Table 6.1 Summary of Shear and Moment for Lateral Load Test

	59-8 (Apalachicola)		**LLT-1 (Fuller Warren)**	
	Non-spring	Rotational spring (5,000,000 kips)	Non-spring	Rotational spring (5,000,000 kips)
Max. shear (kips)	3152	2127	773	405
Max. shear ele. (ft)	-30	-30	-74	-71
Zero shear ele. (ft)	-59	-54	-86	-83
Max. moment (kips)	44853	26140	5439	2681

Max. moment ele. (ft)	-25	-25	-69	-69
Zero moment ele. (ft)	-59	-54	-86	-76
Y rotation (rad)	-0.0185	-0.0042	-0.0124	-0.0006

	26-1 (Gandy)		**52-3 (Gandy)**	
	Non-spring	Rotational spring (5,000,000 kips)	Non-spring	Rotational spring (5,000,000 kips)
Max. shear (kips)	114	53	759	405
Min. shear ele. (ft)	-16	-16	-26	-25
Zero shear ele. (ft)	-25	-22	-35	-33
Max. moment (kips)	686	249	3394	1428
Max. moment ele. (ft)	-10	-11	-23	-23
Zero moment ele. (ft)	-23	-18	-34	-28
Y rotation (rad)	-0.0022	-0.0001	-0.0115	-0.0004

	91-3 (Gandy)	
	Non-spring	Rotational spring (5,000,000 kips)
Max. shear (kips)	817	467
Max. shear ele. (ft)	-43	-43
Zero shear ele. (ft)	-53	-51
Max. moment (kips)	3640	1717
Max. moment ele. (ft)	-41	-41
Zero moment ele. (ft)	-51	-46
Y rotation (rad)	-0.0080	-0.0004

	TH 1 (Victory)		**TH 3 (Victory)**	
	Non-spring	Rotational spring (5,000,000 kips)	Non-spring	Rotational spring (5,000,000 kips)
Max. shear (kips)	706	327	782	449
Max. shear ele. (ft)	36	36	26	26
Zero shear ele. (ft)	25	26	16	19
Max. moment (kips)	4186	283	3733	1464
Max. moment ele. (ft)	40	40	32	30
Zero moment ele. (ft)	25	33	17	23
Y rotation (rad)	-0.0190	-0.0005	-0.0262	-0.0005

CHAPTER 7
LABORATORY RESULTS

7.1 SAMPLING SIZE AND VARIABILITY

Sedimentary rocks form at or near the earth's surface at relatively low temperatures and pressures through one of the following processes: 1) deposition by water, wind or ice, 2) precipitation from solution (may be biologically mediated); and/or 3) growth in position by organic processes (e.g., carbonate reefs). In the case of Florida Limestone, process (2) or (3) are the primary formation mechanisms. Due to the latter formation, sedimentary rocks generally are non-homogeneous with hundreds of sub-classifications. In Florida, there are at least fourteen major classifications and a multitude of sub-classifications with strength (unconfined) varying from 10 tsf to beyond 500 tsf.

Of interest is the variability (mean and standard deviation) of the rock properties (i.e., strength, modulus, etc.) over a specific site where a deep foundation is to be installed. The latter is extremely important for design, i.e., skin friction, end bearing, and lateral resistance. Designers have to weigh the cost of collecting laboratory specimens as well as testing vs. spatial variability and representative sample population sizes for each site. The means to quantify the spatial variability and uncertainty (sample size) is found in the mathematical disciplines of statistics and probability. The application of statistics and probability to material variability has grown rapidly in the last 20 years, and now forms one of the important branches of geotechnical engineering.

Technically, a sample is a subset of the population. That is, a small set of values, i.e., the sample, is taken from the larger set of values that compose the population. This process is called sampling. The validity of the inferences concerning the population is dependent on how well the sample represents the population. Samples that systematically differ from the population are said to be biased. More technically, a biased sample is a sample selected in such a way that all values in the population do not have an equal chance of being selected. Samples that are not biased are **random** samples.

Error in the value of rock properties can arise in several ways. First, the sample population may be small and not representative of the whole site (rock variability); second, the sample properties have been altered or disturbed in the process of sampling and/or transportation to the laboratory (i.e., sampling errors); or finally, the laboratory tests were not scrupulously performed according to prescribed standard methods (i.e., testing errors). Each of these error sources may contain both bias and random error. Lumb (1974) has discussed the identification and the contribution of each of these error types to the total error.

Generally, it is too expensive to take hundreds of Geotechnical samples from a site (i.e., over length of bridge foundation, etc.). However, sample size must be "large enough" that its scientific/engineering significance is also statistically significant. In general, an under-sized study can be a waste of resources for not having the capability to produce useful results, while an over-sized one uses more resources than are necessary. To quantify the latter, Equation 7.1 is used to estimate the sample size, N, needed to obtain a specified accuracy and precision based on variability (Moore, 1995).

$$N = \left(\frac{\sigma}{\varepsilon}\right) * (Z_{\alpha} + Z_{\beta})^2 \qquad \text{Equation 7.1}$$

N = Number of Sample Needed
σ = Standard Deviation
ε = Error Distance (True Mean – Sample Mean)
α = Type I Error (Confidence Level)
β = Type II Error (Statistical Power)

Accuracy refers to the closeness of the measurements to the “actual” or “real” value of the physical quantity, based on a confidence level (i.e., 95%). The statistical power is used to indicate the closeness with which the measurement agrees with another, independently of any systematic error involved. Typically, the precision is set to be 20%, which results in an “accurate” estimation with a small bias. A “precise” estimation has both small bias and variance. Quality is proportionate to the inverse of variance. There is nothing special about a 95% confidence level and a 20% precision level, however, statisticians agree that this combination is “very significant” for the total population.

It has been found in this research that variability, specifically the standard deviation, for the Florida limestone becomes constant below 120 tsf for qu (unconfined strength) and 20 tsf for qt (split tensile strength), which are the upper limits for rock strength, that fail the rock instead of the interface for drilled shafts (discussed in Chapter 8). The result shows a constant standard deviation of about 30 tsf for qu and 6 tsf for qt. Using equation 7.1, for example, 17th St bridge has s = 29, $Z\alpha = 1.96$, $Z\beta = .84$, a total of 23 samples are needed, if an error, ε, of 10 tsf is acceptable. However, the values of qu <120 tsf, qt < 20 tsf, represent about 60% of the sample data population. Due to the significance of the latter data, the sample size should be adjusted for the data that is not in the range of 120 tsf for qu and 20 tsf for qt. A simple solution is to adjust the number obtained for equation 7.1 by multiplying by the reciprocal of 1/0.6, i.e.,

$$N_{\text{required}} = N \text{ (from Equation 7.1) } *1.6 \qquad \text{Equation 7.2}$$

From Eq. 7.2, the average minimum number of samples to be taken for this project is approximately 40 each (i.e., qu and qt). Moreover, since all the sites have similar standard deviations, the latter number (i.e., 40 samples), should result in representative field populations. However, this is assuming that the field is homogeneous. The latter is not the case when different formations are present on a site (e.g., 17th Street both sides of causeway) or layering is present. For such scenarios, representative samples should be recovered from each formation or layer. A simple way to identify layering or different formations is from the frequency strength (qu and qt) distribution plots and the presence of multiple strong peaks over the site.

7.2 SITE SPECIFIC LABORATORY DATA

7.2.1 17th Street Causeway

A total of 56 unconfined compressive strength (ASTM D-2938) tests and 34 split tensile strength (ASTM D-39) tests were performed on the north and south sides of the existing bridge. Both core Recovery (%) and RQD (%) tests for a total of 119 Recoveries and RQD were performed. The tests were performed on rock cores collected at an approximate interval ranging from -29 to -142 ft (NGVD). A total of 16 tests for Young's modulus values were also performed near the site of the load tests.

The averages of qu and qt values for this site were 161.0 and 58.3 tsf. The Standard Deviations of qu and qt values were 138.6 and 44.3 tsf. From the rock core samples of this site, the average of Recovery was 24 % and the average of RQD values was 6 %. The average (mean) of Young's modulus values was 422,170 psi and the Standard Deviations of the modulus values was 356,360 psi.

Although the mean is typically used in Geotechnical Engineering, for skewed populations, the mode and median values will probably represent a better estimate of the distribution of the data. Figures 7.1, 7.2 and 7.3, which depicts the limestone strength and stiffness properties (i.e., qu, qt, Young's Modulus), present the mode and median value as well as mean, and standard deviation of the data over the site.

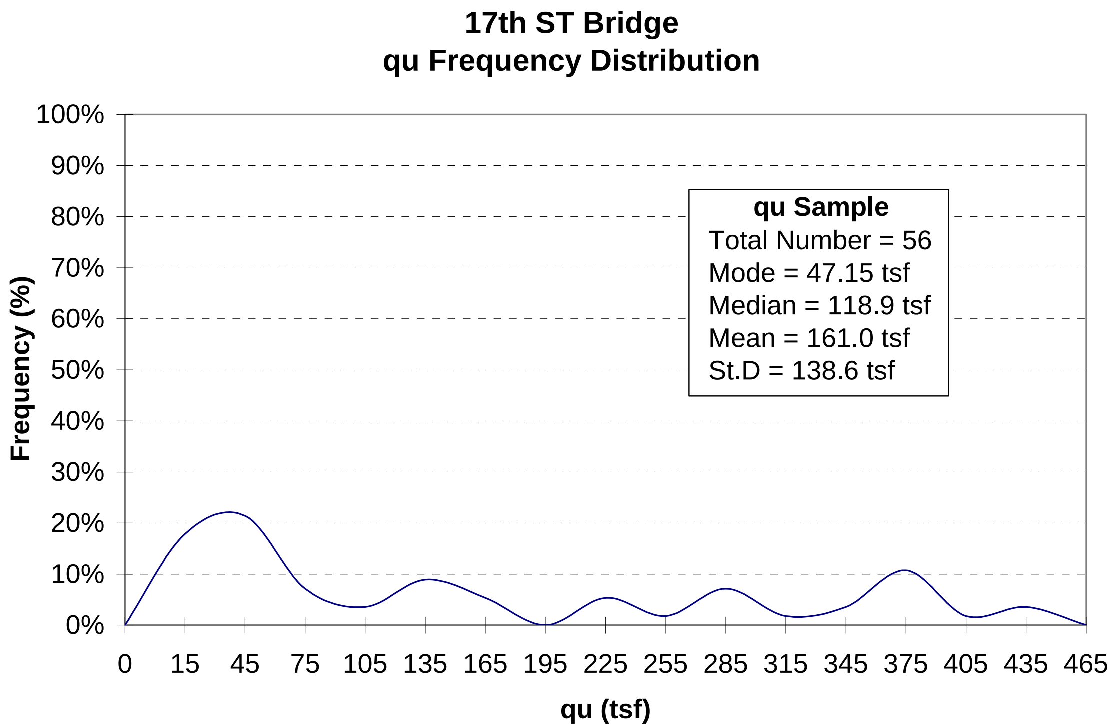


Figure 7.1 17th Street Causeway Site – qu Frequency Distribution

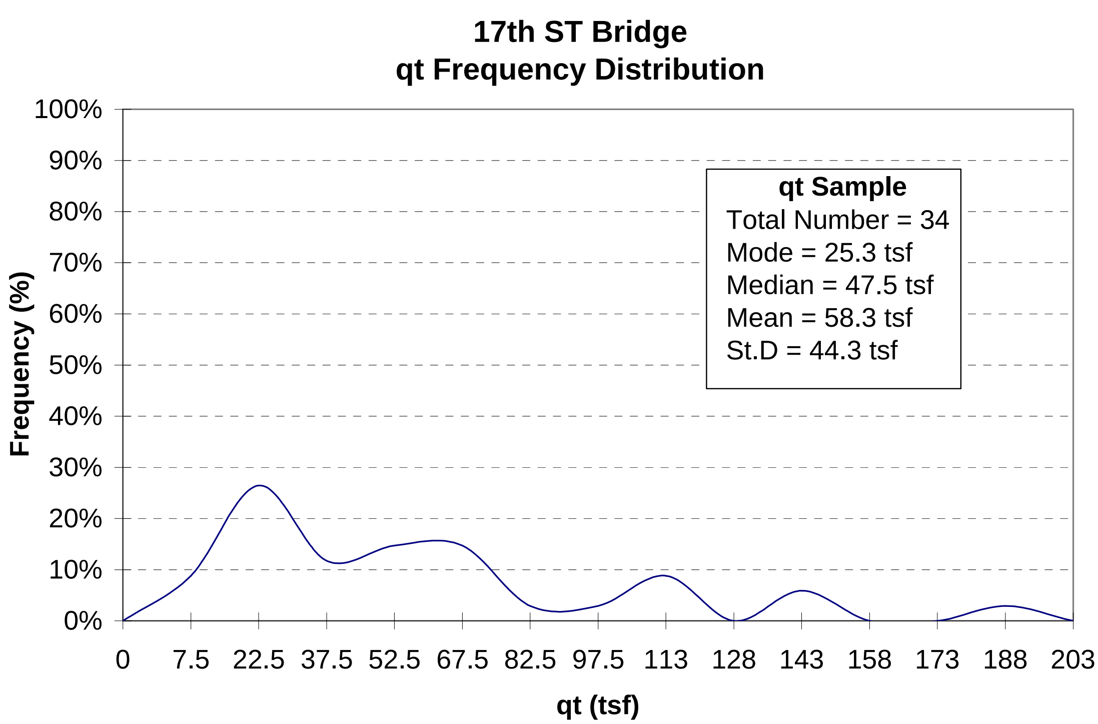


Figure 7.2 17th Street Causeway Site – qt Frequency Distribution

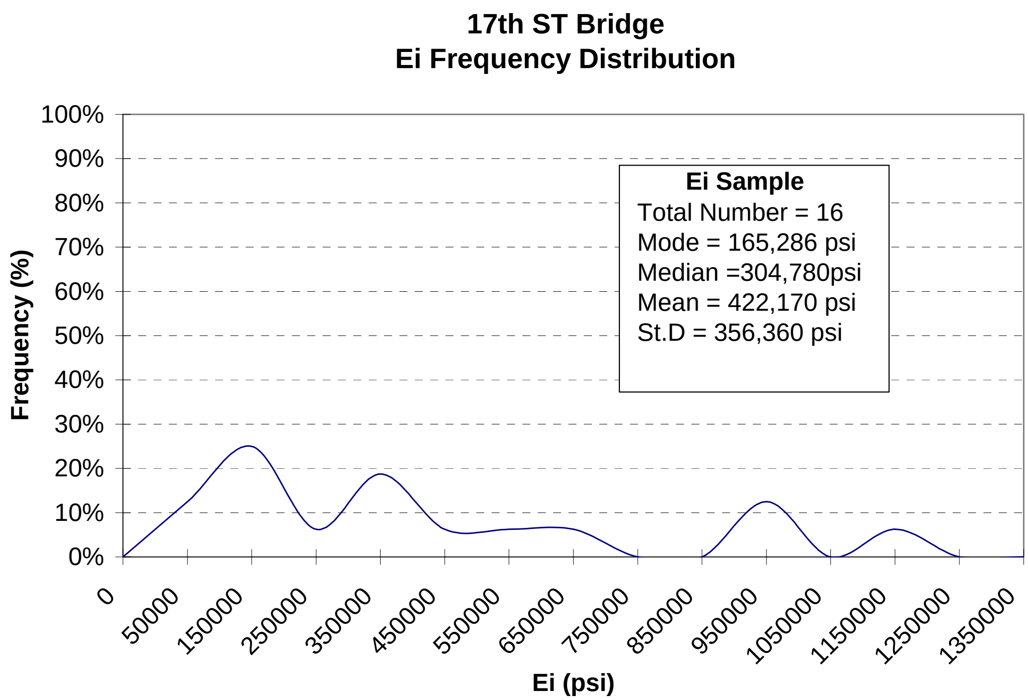


Figure 7.3 17th Street Causeway Site – Ei Frequency Distribution

It is quite common in Geotechnical Engineering to measure data in pairs, (e.g., qu, qt, and Ei). Generally, both of the paired items will have uncertainty associated with them. Of course, the graphical and numerical summaries are applicable to each of the items, but it is often of interest to determine if a relationship exists between the two sets of data.

A plot of the observed pairs, one against the other, is called a scattergram. The correlation coefficient R_{xy} (Moore, 1995) is:

$$R_{xy} = \sum\left(x_i - \bar{x}\right) * \left(y_i - \bar{y}\right) / \sqrt{\sum\left(x_i - \bar{x}\right)^2 * \left(y_i - \bar{y}\right)^2} \qquad \text{Equation 7.3}$$

X is linearly related to Y in a perfect linear relationship with no scatter when Rxy = ±1; whereas if no linear relationship exists, then Rxy = 0. Note that this is a measure of how well the data fits a straight line, data which has a nonlinear relationship will also give a lower correlation coefficient. Figure 7.4 shows the results for the correlation between qu and Ei. Evident from the figure, there is a high correlation coefficient (Rxy = 0.89) between qu and Ei, as calculated by equation 7.3.

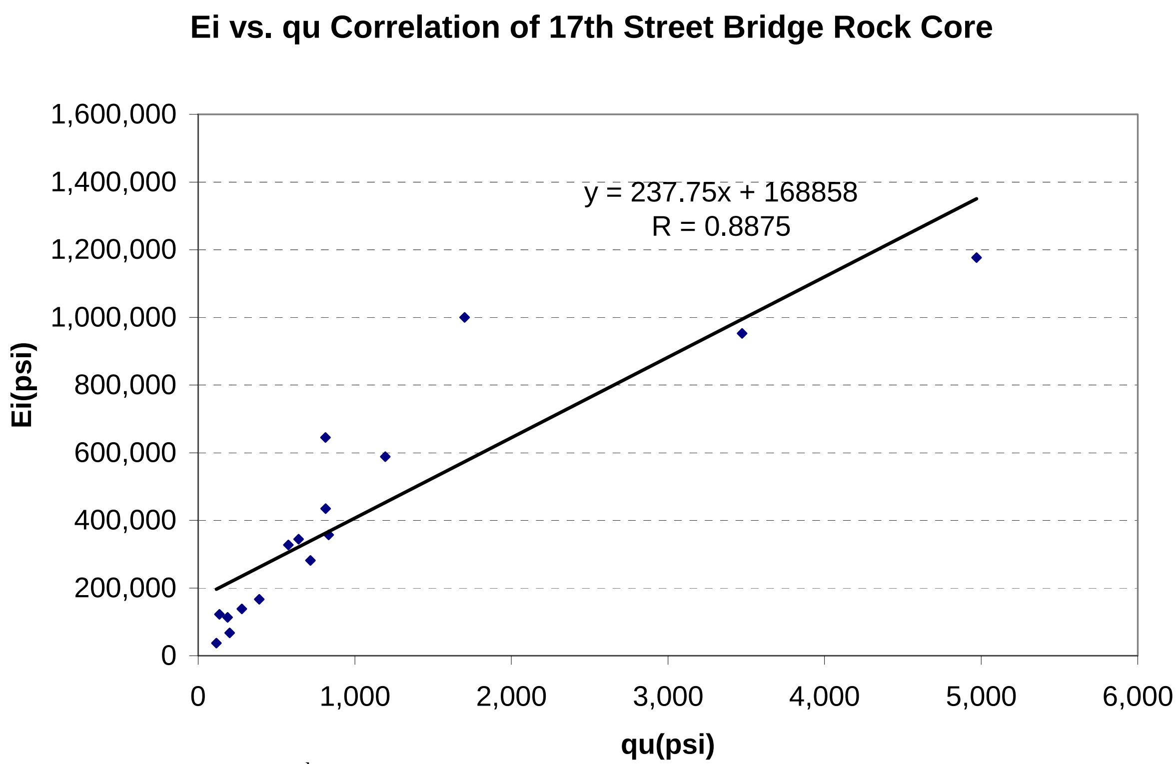


Figure 7.4 17th Street Causeway Site – Scattergram Between the qu and Ei

7.2.2 Acosta Bridge

A total of 21 unconfined compressive strength (ASTM D-2938) tests were performed on limestone samples collected in the vicinity of the bridge. No split tensile tests were conducted for this project. Both core Recovery (%) and RQD (%) tests for a total of 14 Recoveries and RQD were performed and a total of 11 tests for Young's modulus values were performed near the site of the load tests.

The average of the qu values for this site was 74.9 tsf. The Standard Deviation of qu values was 82.7tsf. From the rock core samples obtained from the site, the average of Recovery was 64.4 % and the average of RQD values was 37.1 %. The average of Young's modulus values was 480,911 psi and the Standard Deviation of the modulus values was 841,720 psi. Figures 7.5 and 7.6 show the variability of the limestone properties (i.e., qt, Young's Modulus) along with their mode, median, mean, and standard deviation over the site.

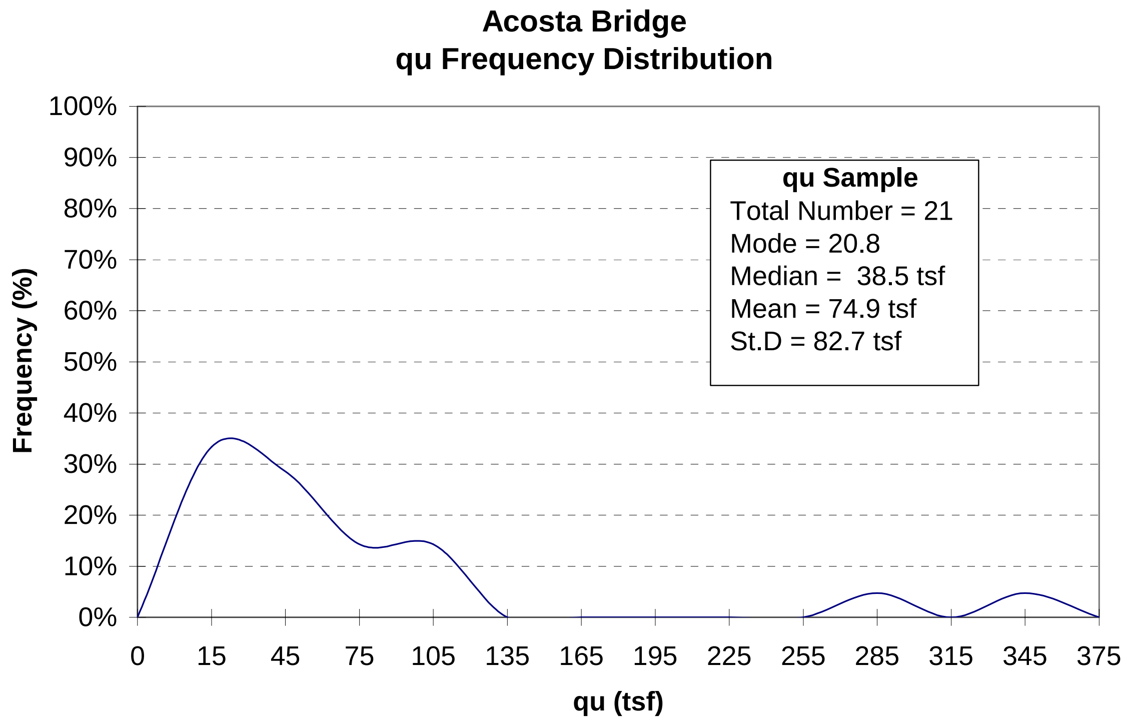


Figure 7.5 Acosta Bridge Site – qu Frequency Distribution

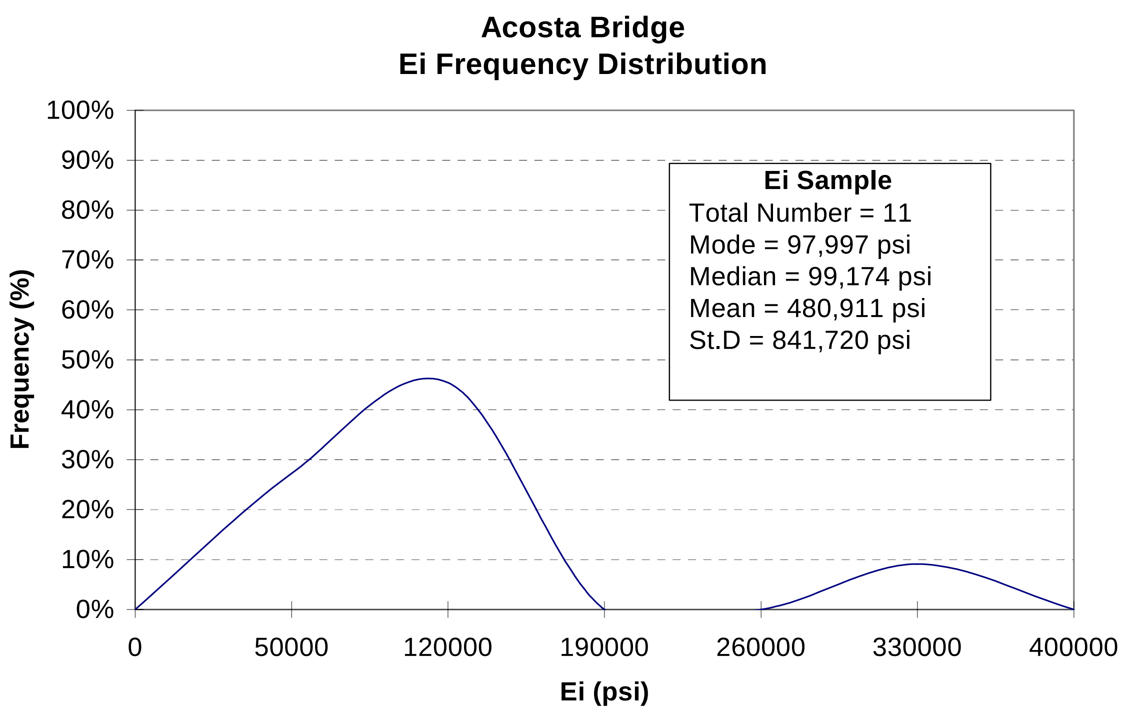


Figure 7.6 Acosta Bridge Site – Ei Frequency Distribution

Figure 7.7 shows the correlation between qu and Ei. From the Figure 7.8, it appears that there is high correlation coefficient (Rxy = 0.99) between qu and Ei calculated by equation 7.3.

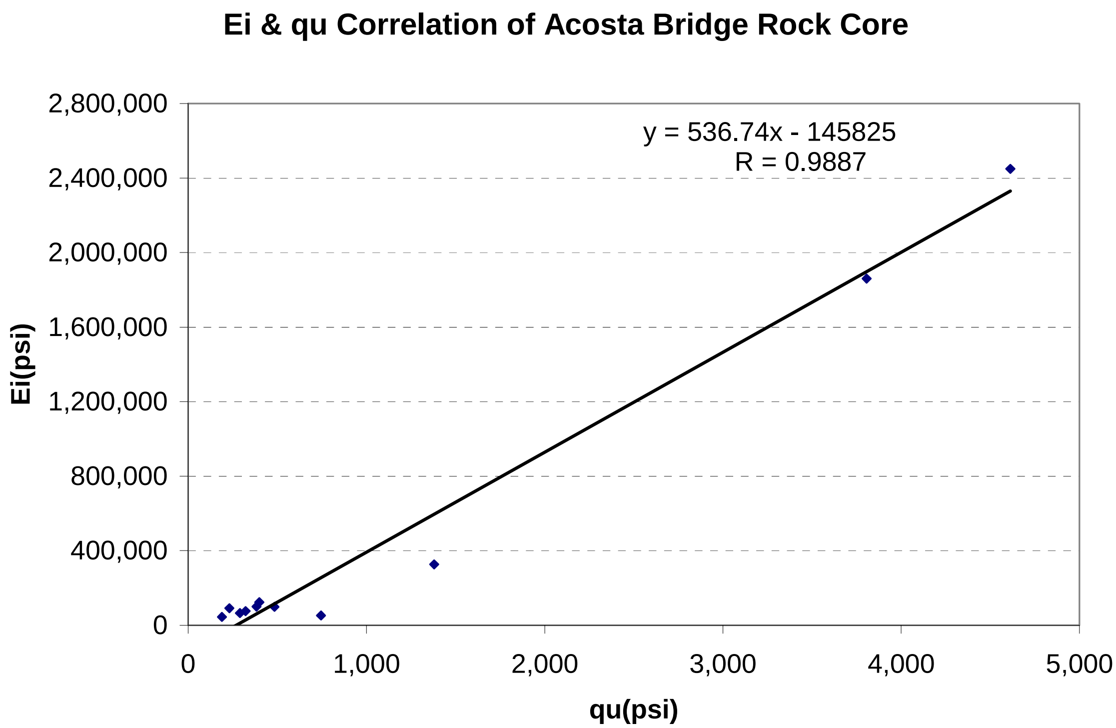


Figure 7.7 Acosta Bridge Site – Scattergram Between the qu and Ei

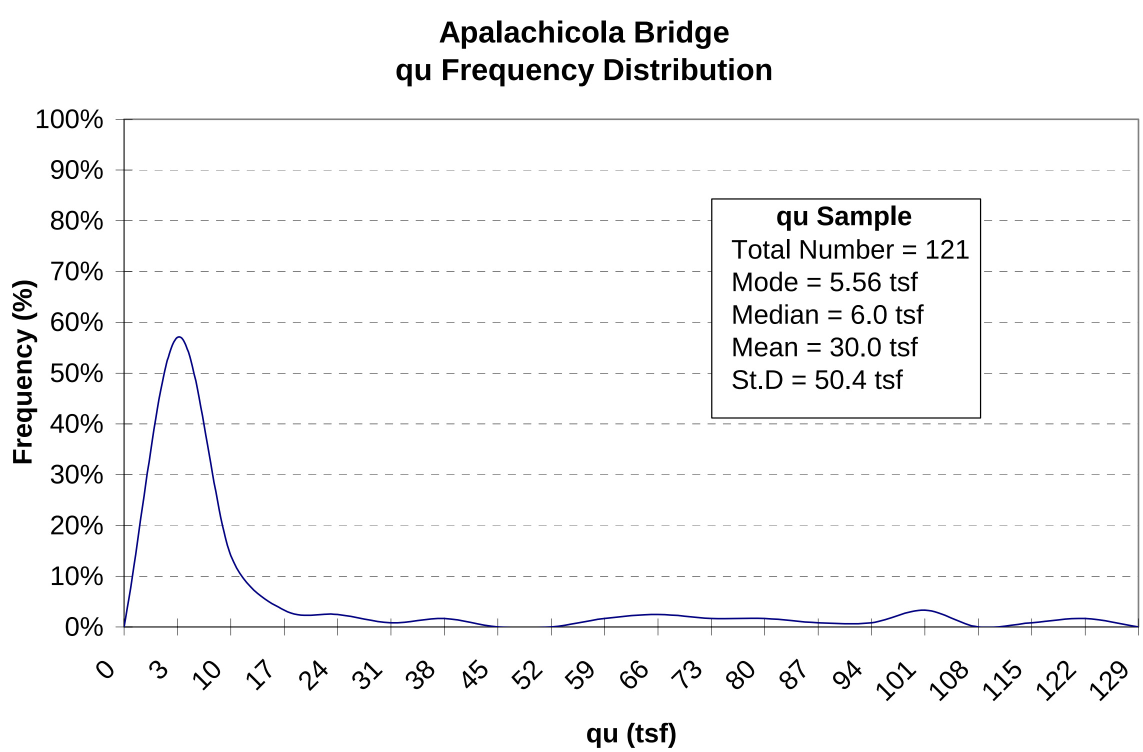


Figure 7.8 Apalachicola Bridge Site – qu Frequency Distribution

7.2.3 Apalachicola River State Road (SR) 20

A total of 121 unconfined compressive strength (ASTM D-2938) tests and 76 split tensile strength (ASTM D-39) tests were performed in the vicinity of the bridge. Both core Recovery (%) and RQD (%) were obtained; 384 Recoveries and RQD were recorded. Laboratory strength tests were performed on rock cores collected at over the interval ranging from +30 to –40 feet (NGVD). A total of 23 tests for Young's modulus values were also performed.

The averages of qu and qt values for this site were 29.7 and 3.2 tsf. The Standard Deviations of qu and qt values were 50.4 and 5.9 tsf, respectively. From the rock core samples, the average Recovery was 56 % and the average of RQD values was 30 %. From 23 rock core sample tests, the stiffness (Young Modulus) was determined. The average Young's Modulus and associated Standard Deviation was 170,735 and 177,847 psi respectively. Figures 7.8, 7.9 and 7.10 plot the frequency distribution of the limestone strength and stiffness properties (i.e., qu, qt, Young's Modulus) along with their mode, median, mean, and standard deviation for the site.

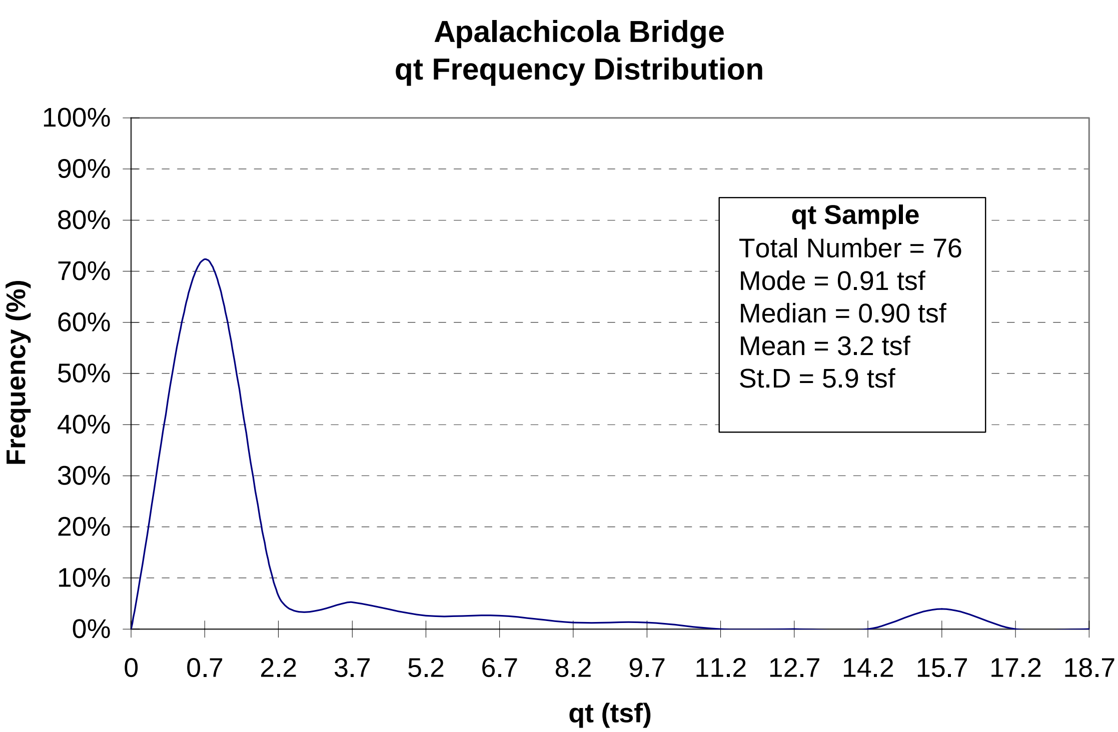


Figure 7.9 Apalachicola Bridge Site – qt Frequency Distribution

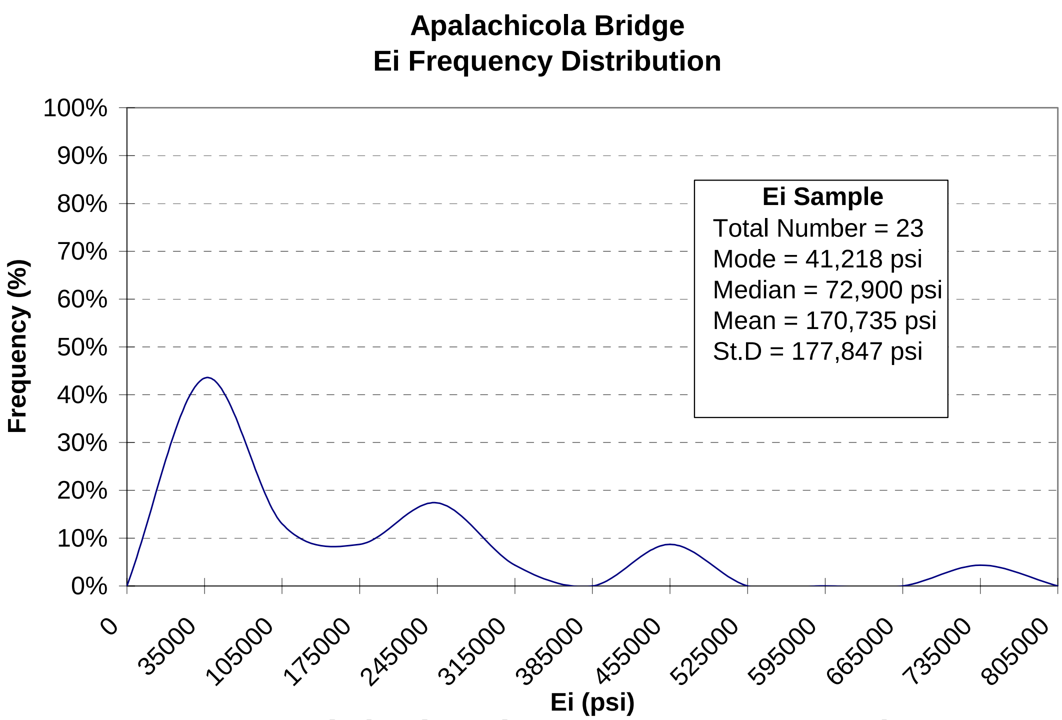


Figure 7.10 Apalachicola Bridge Site – Ei Frequency Distribution

Figure 7.11 shows the results for the correlation between qu and Ei. Evident from the figure there is correlation (Rxy = 0.82) between qu and Ei, based on equation 7.3.

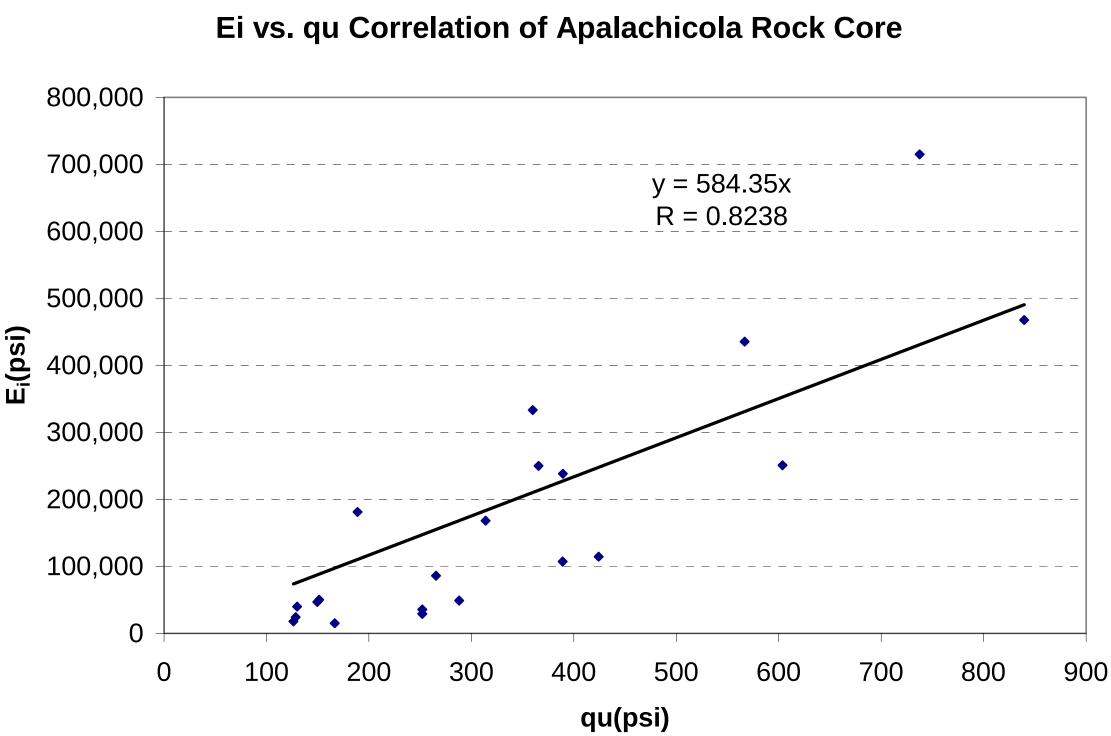


Figure 7.11 Apalachicola Bridge Site – Scattergram Between the qu and Ei

7.2.4 Fuller Warren Bridge

Testing was performed on intact core samples from both the limestone and marl formations. In the case of the marl, both cemented and non-cemented specimens were tested. A total of 53 Recovery (%) and RQD (%) were recorded over the site. Laboratory testing included 51 unconfined compression strength, qu, and 22 split tension tests, qt. A total of 33 Young's Modulus values were recorded from the unconfined strength tests.

The averages of qu and qt values were 74.0 and 21.0 tsf, respectively. The Standard Deviation of qu and qt values was 74.6 and 12.8 tsf. From the rock core samples over the site, the average of Recovery was 58 % and the average of RQD values was 37 %. The average and standard deviation of Young's Modulus was 362,167 and 475,836 psi from 33 rock core samples from the site. Figures 7.12, 7.13 and 7.14 display the frequency distribution of the limestone properties (i.e., qu, qt, Young's Modulus) along with the mode, median, mean, and standard deviation for each property on the site.

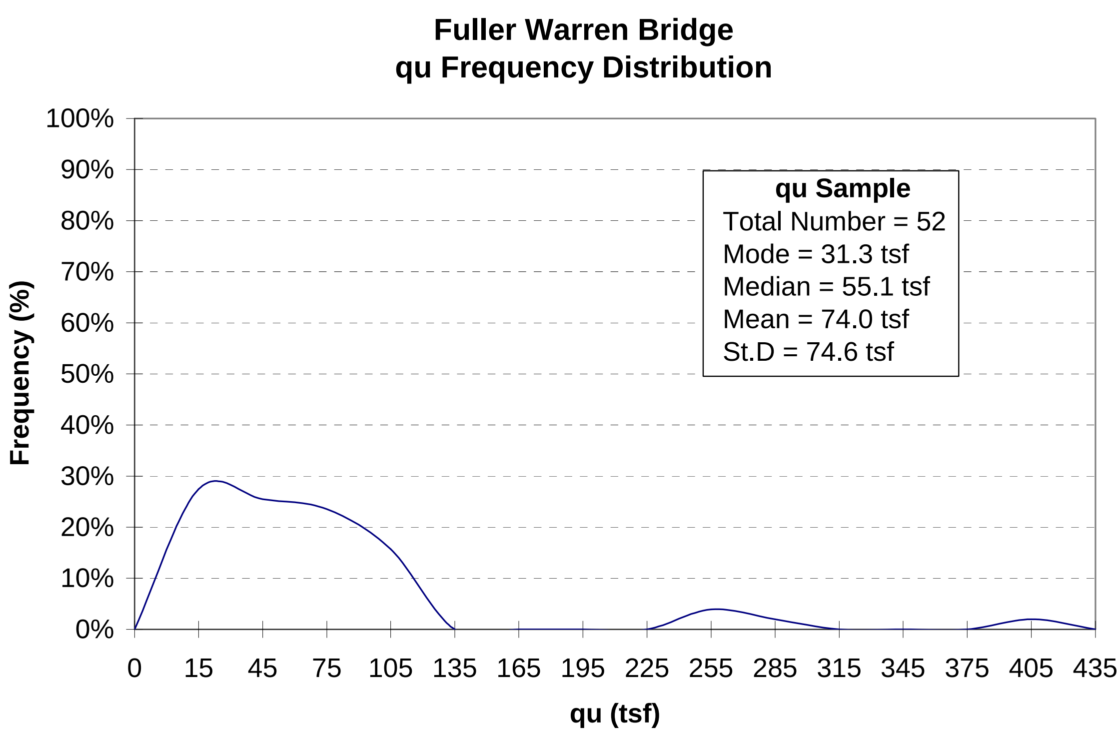


Figure 7.12 Fuller Warren Bridge Site – qu Frequency Distribution

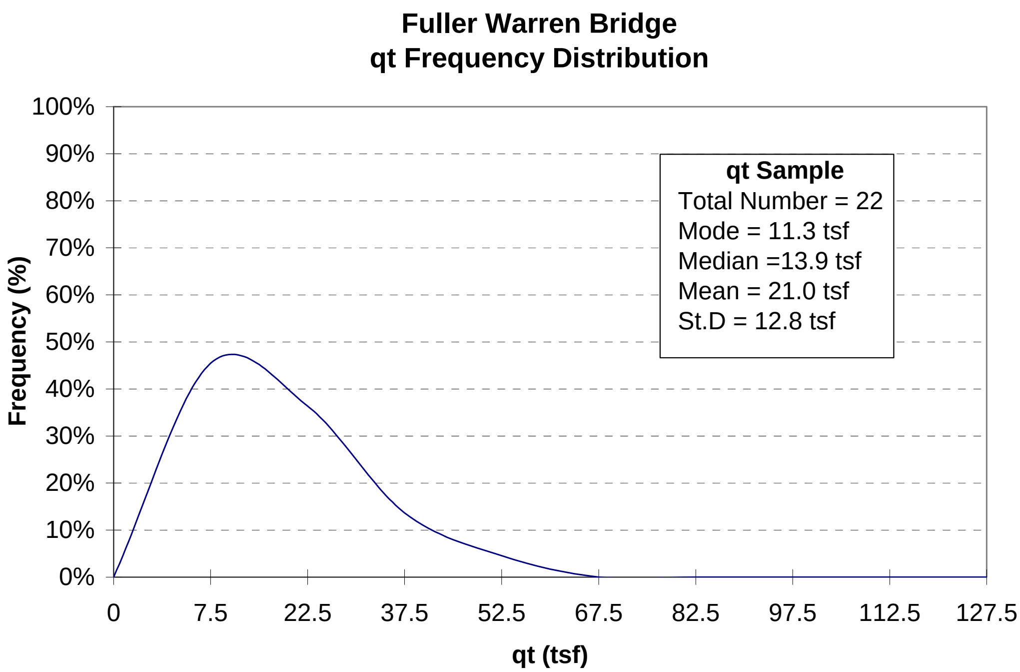


Figure 7.13 Fuller Warren Bridge Site – qt Frequency Distribution

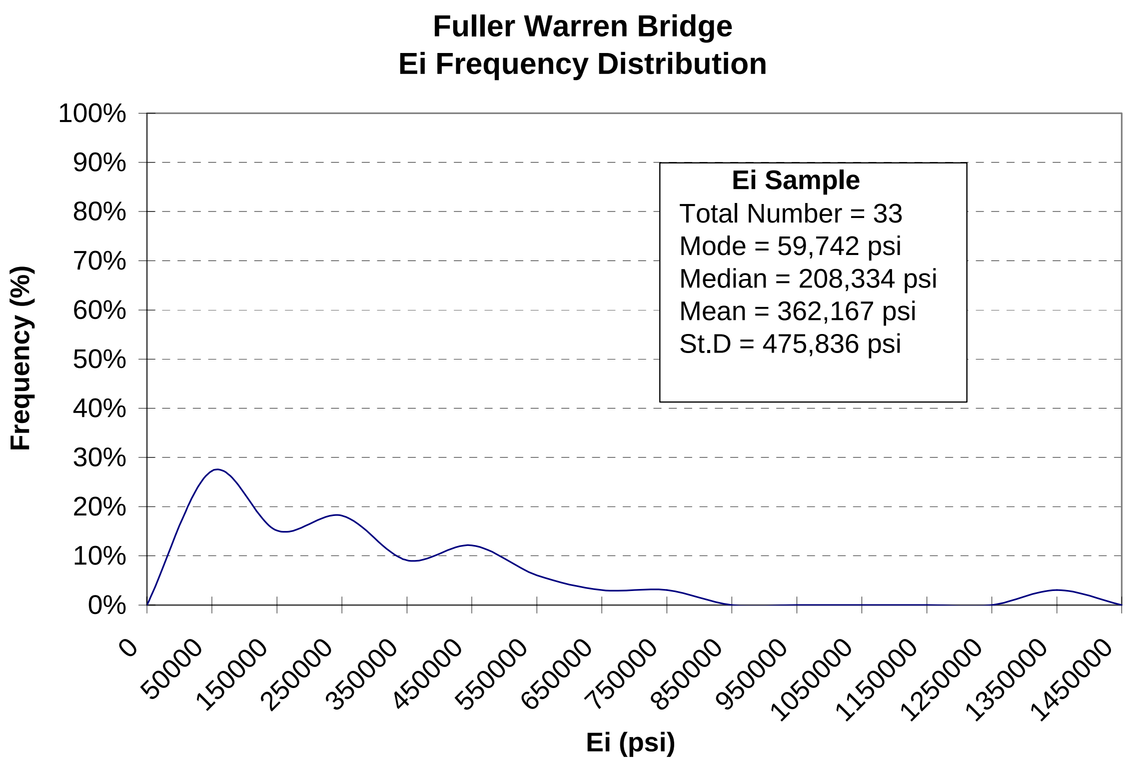


Figure 7.14 Fuller Warren Bridge Site – Ei Frequency Distribution

Figure 7.15 shows the results for the correlation between qu and Ei. From the Figure, it appears that there is high correlation coefficient (Rxy = 0.98) between qu and Ei.

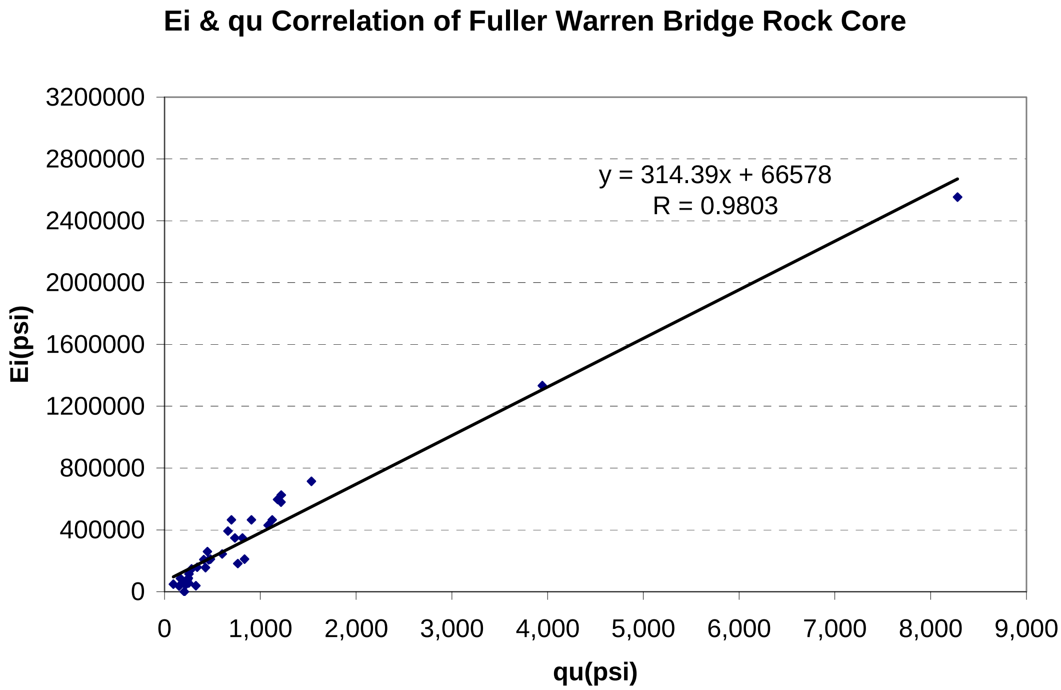


Figure 7.15 Fuller Warren Bridge Site – Scattergram Between the qu and Ei

7.2.5 Gandy Bridge

A total of 38 unconfined compressive strength (ASTM D-2938) and 28 split tensile strength (ASTM D-39) tests were conducted in the laboratory on the recovered cores. In addition, the 38 unconfined compression tests had the Young's Modulus measured (i.e., cross-head movement was recorded). A total of 39 Recoveries (%) and RQD (%) were recorded from the core runs.

The averages of qu and qt values for this site were 115.0 and 17.8 tsf, respectively. Standard Deviations of 148.4 and 17.7 tsf were found for qu and qt on the site. From all the rock core samples, the average of Recovery was 83.2 % and the average of RQD values was 56.3 %. The mean and standard deviation of Young's modulus for this site were 547,642 and 428,656 psi, respectively. Figures 7.16 through 7.18 plot the frequency distribution of the limestone properties (i.e., qu, qt, Young's Modulus) along with their mode, median, mean, and standard deviation over the site.

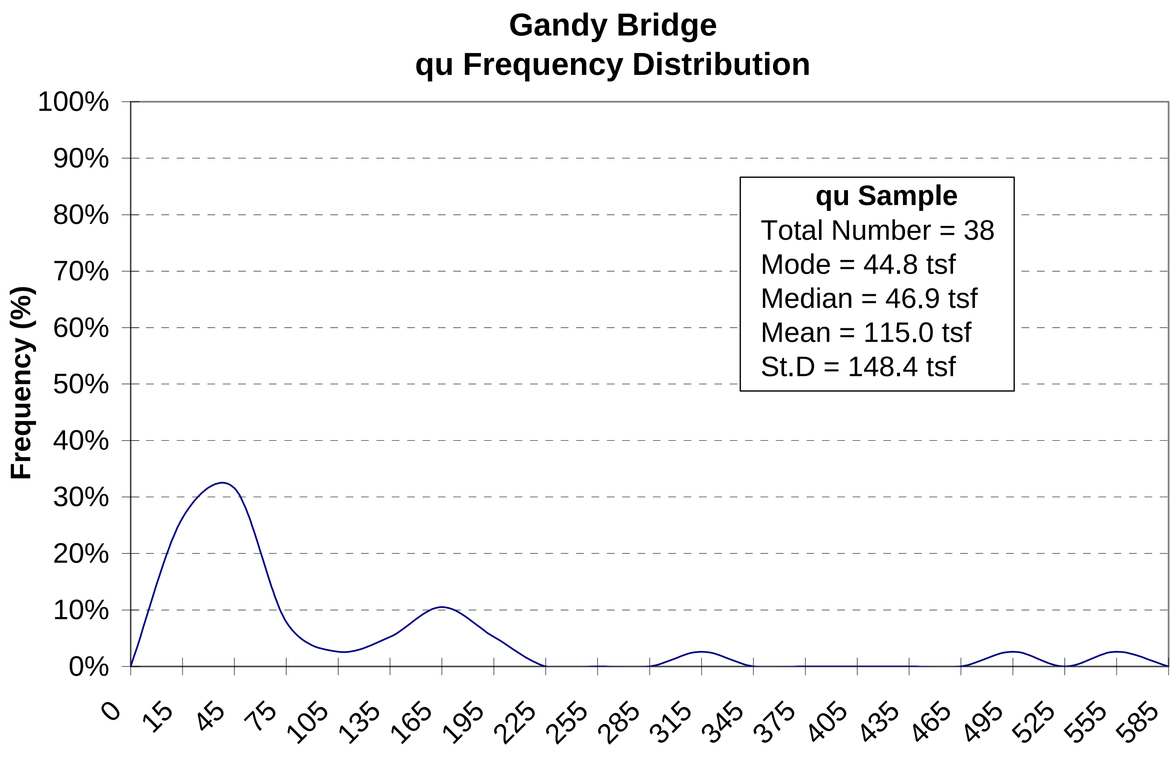


Figure 7.16 Gandy Bridge Site – qu Frequency Distribution

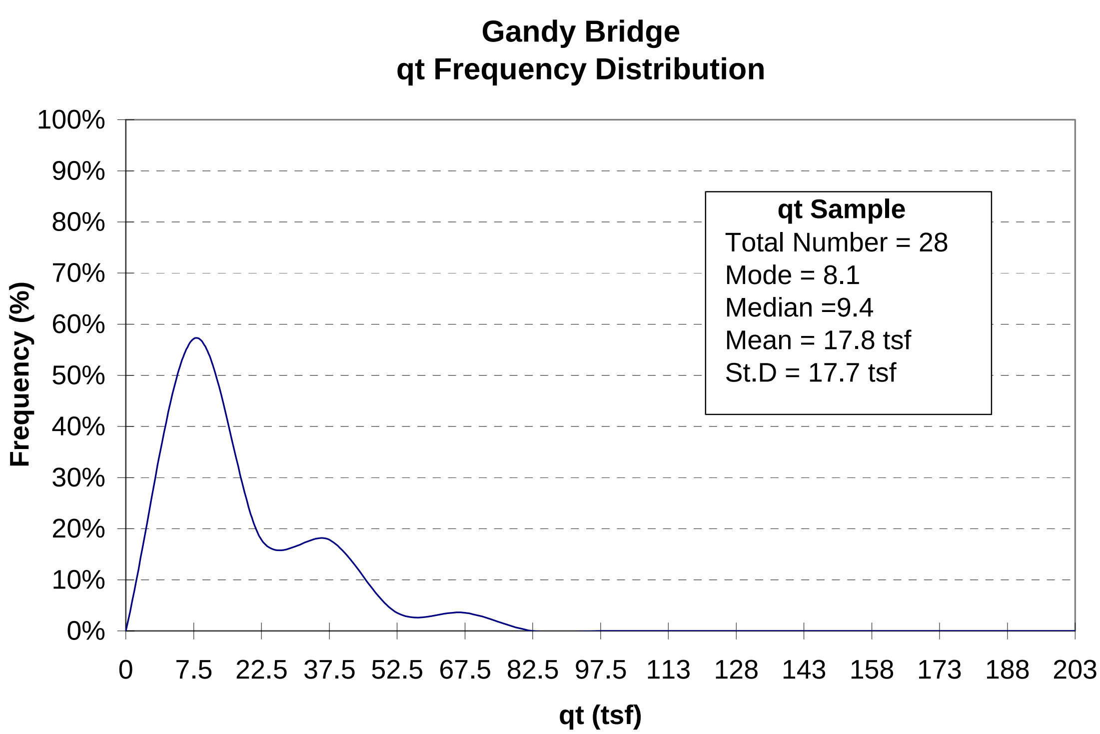


Figure 7.17 Gandy Bridge Site – qt Frequency Distribution

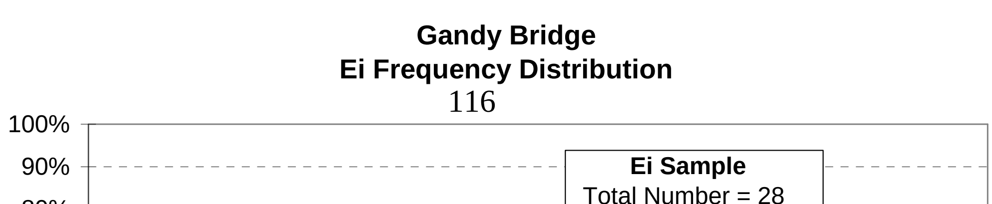


Figure 7.18 Gandy Bridge Site – Ei Frequency Distribution

Figure 7.19 shows the correlation between qu and Ei for all the data. Evident from the figure there is correlation (Rxy = 0.84) between qu and Ei as calculated by Eq. 7.3.

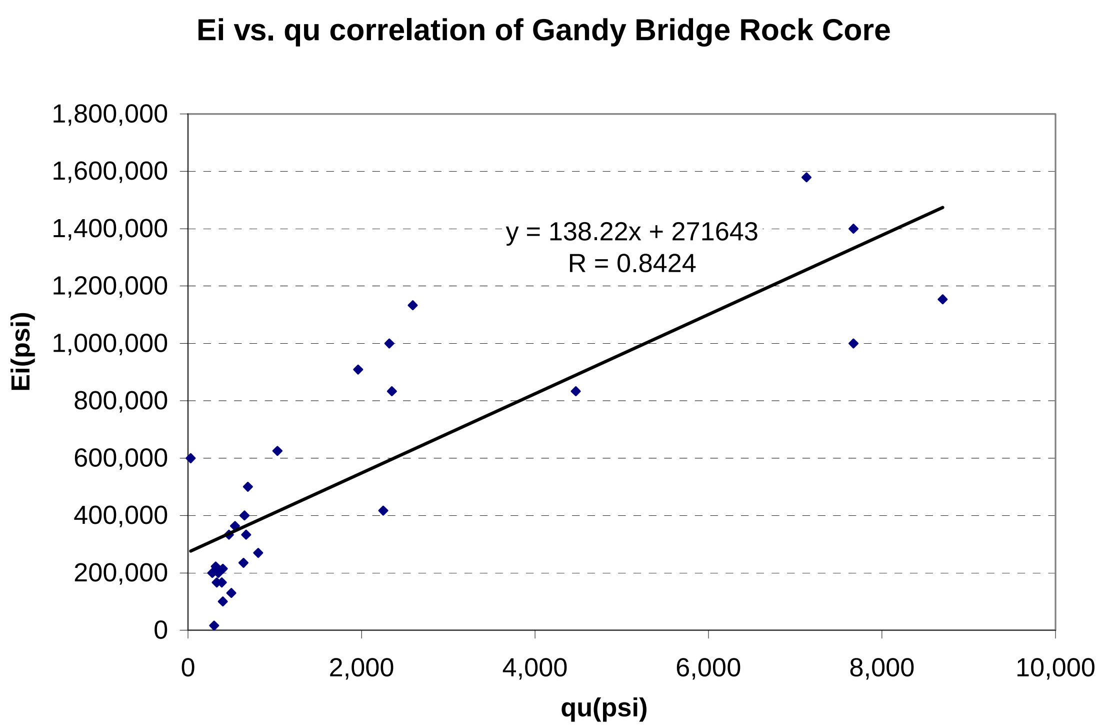


Figure 7.19 Scattergram Between the qu and Ei

7.2.6 Hillsborough

A total of 18 unconfined compression (ASTM D-2938) and 17 split tensile (ASTM D-39) tests were performed on cores recovered at the site. A total of 104 core Recoveries (%) and RQD (%) were measured. Strength measurements were performed on specimens obtained from elevation ranging from -15 to -40 feet (NGVD).

The averages of qu and qt values for this site were 89.5 and 12.0 tsf, respectively. Standard Deviations of 130.5 and 16.0 tsf were computed for the qu and qt values. From the rock core samples, the average Recovery was 70 % and the average RQD value was 27 %. Figures 7.20 and 7.21 plot the variability (i.e., frequency distribution) of the limestone strength properties (i.e., qu, qt), as well as identifying their mode, median, mean, and standard deviation for the whole site.

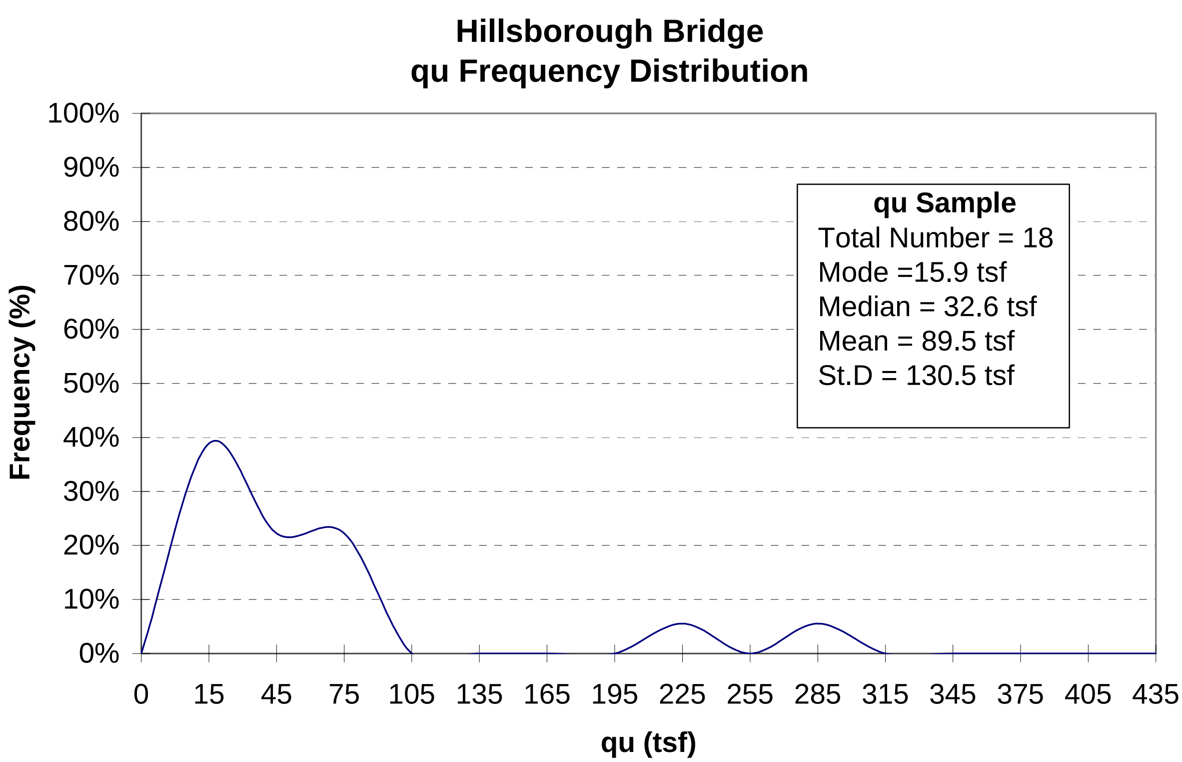


Figure 7.20 Hillsborough Bridge Site – qu Frequency Distribution

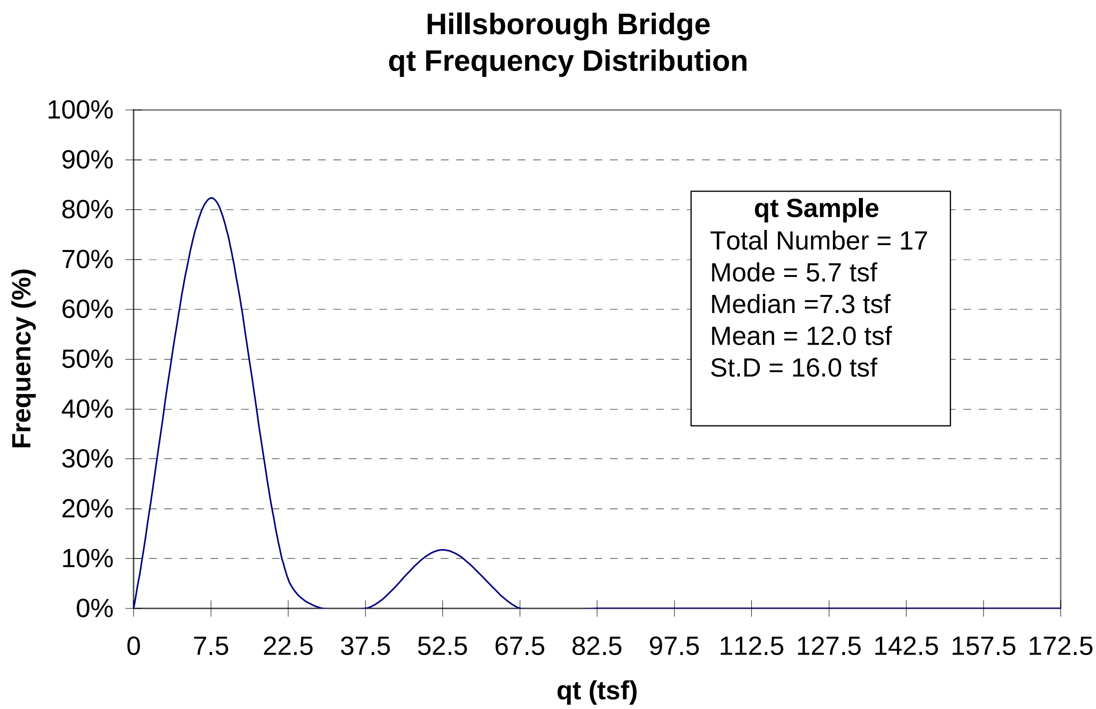


Figure 7.21 Hillsborough Bridge Site – qt Frequency Distribution

7.2.7 Victory Bridge

A total of 56 unconfined compression (ASTM D-2938) and 28 split tensile (ASTM D-39) tests were performed on cores recovered from the site. A total of 277 Recoveries and RQD values were recorded. The strength tests were performed on specimens collected from elevation + 40 to – 20 feet (NGVD). From the unconfined strength tests, 24 Young Modulus were determined.

The averages of qu and qt values for this site were 90.3 and 31.0 tsf, respectively. Standard Deviations of 63.2 and 21.7 tsf were found for the qu and qt values. From the recovered rock core samples, an average Recovery of 67 % and an average RQD value of 42 % was found. The mean and Standard Deviation of the Young's Modulus was 2,676,038 and 1,112,087 psi, respectively. Figures 7.22, 7.23 and 7.24 display the variability of the limestone properties (i.e., qu, qt, Young's Modulus) along with their mode, median, mean, and standard deviation over the site. Figure 7.25 shows the correlation between qu and Ei. From the figure, it appears that there is correlation coefficient (Rxy = 0.75) between qu and Ei as calculated by equation 7.3.

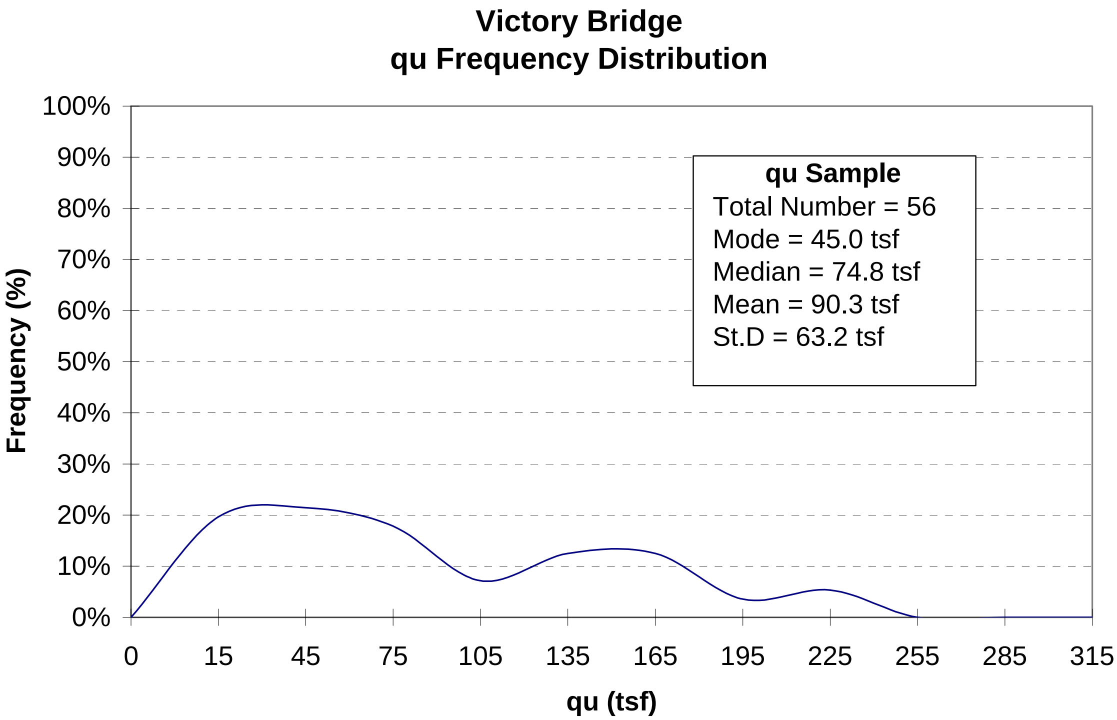


Figure 7.22 Victory Bridge Site – qu Frequency Distribution

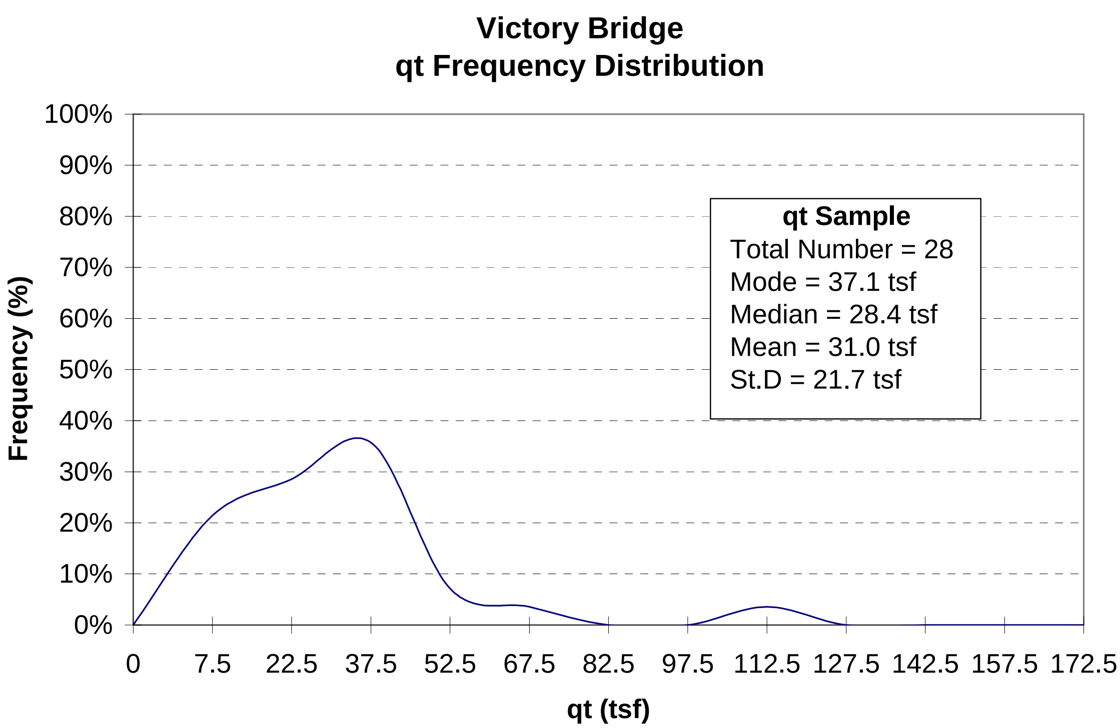


Figure 7.23 Victory Bridge Site – qt Frequency Distribution

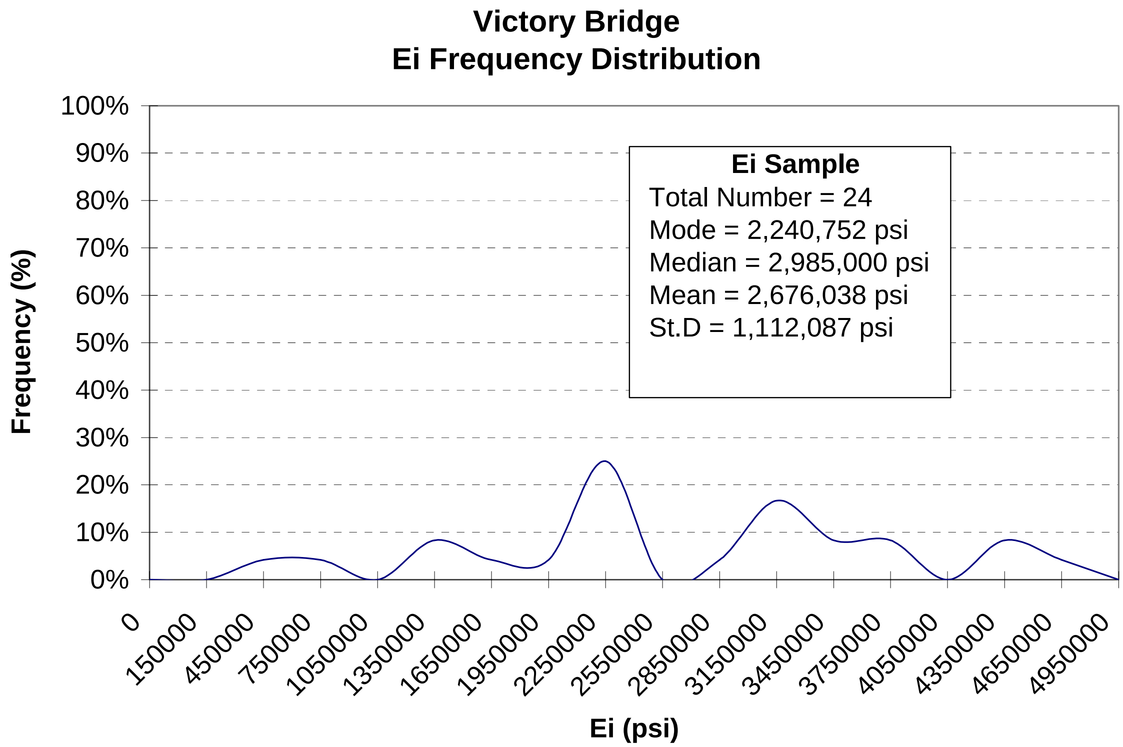


Figure 7.24 Victory Bridge Site – Ei Frequency Distribution

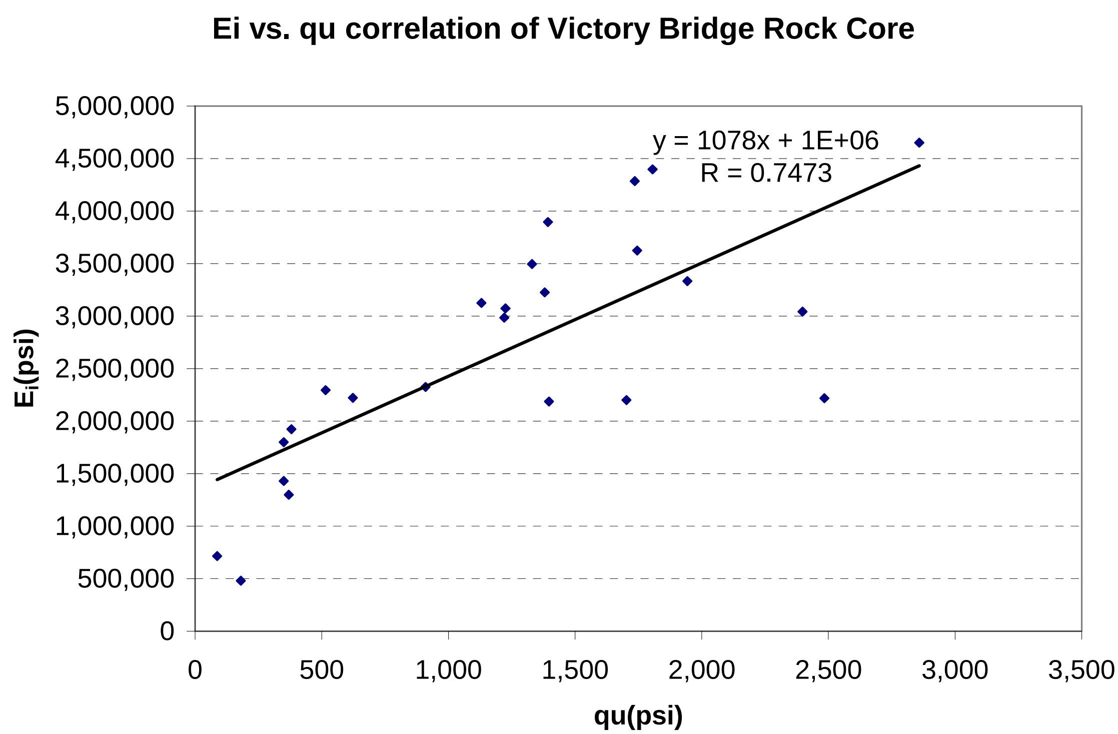


Figure 7.25 Victory Bridge Site – Scattergram Between the qu and Ei

7.3 RELATIONSHIP of COEFFICIENT of VARIATION (COV) for LIMESTONE STRENGTH TESTS

Table 7.1 and 7.2 summarize the mean, standard deviation, and coefficient of variation of unconfined compressive strength (qu) and tensile strength (qt) for all the sites. Table 7.3 shows the correlation between coefficient of variations for qu and qt. From Table 7.3, it appears that there is a high correlation between coefficient of variations of qu and qt (Rxy = 0.92) from equation 7.3. Figure 7.26 and 7.27 also show graphically the high correlation between coefficient of variation of qu and coefficient of variation of qt. The latter suggest a strong relationship between qu and qt for each formation.

7.4 RELATIONSHIP BETWEEN PERCENTAGE RECOVERY and ROCK QUALITY DESIGNATION (RQD)

Of interest was the correlation of % Recovery, with Rock Quality Designation (i.e., RQD), since both are determined from the recovered cores and are reported for all the projects (i.e., Geotechnical reports). Consequently, a correlation analysis was performed on all the RQD and % REC from boreholes, as shown in Table 7.4.

Table 7.1 Mean, Standard Deviation, and Coefficient of Variation of qu

Bridge	Mean	Standard Deviation	COV
17^{th} ST	161.0	138.6	86%
Aparachicola	30.0	50.4	168%
Fuller Warren	74.0	74.6	101%
Gandy	115.0	148.4	129%
Hillsborough	89.5	130.5	146%
Victory	90.3	21.3	70%

Table 7.2 Mean, Standard Deviation, and Coefficient of Variation of qt

Bridge	Mean	Standard Deviation	COV
17th ST	58.3	44.3	71%
Aparachicola	3.2	5.9	184%
Fuller Warren	21.0	12.8	61%
Gandy	17.8	17.7	99%
Hillsborough	12.0	16.0	133%
Victory	31.0	21.7	70%

Table 7.3 Statistical Analysis of qu vs. qt

		qu	qt
qu	Pearson Correlation	1	.915(**)
	Sig. (2-tailed)		0.01
	N	6	6
qt	Pearson Correlation	.915(**)	1
	Sig. (2-tailed)	0.01	
	N	6	6

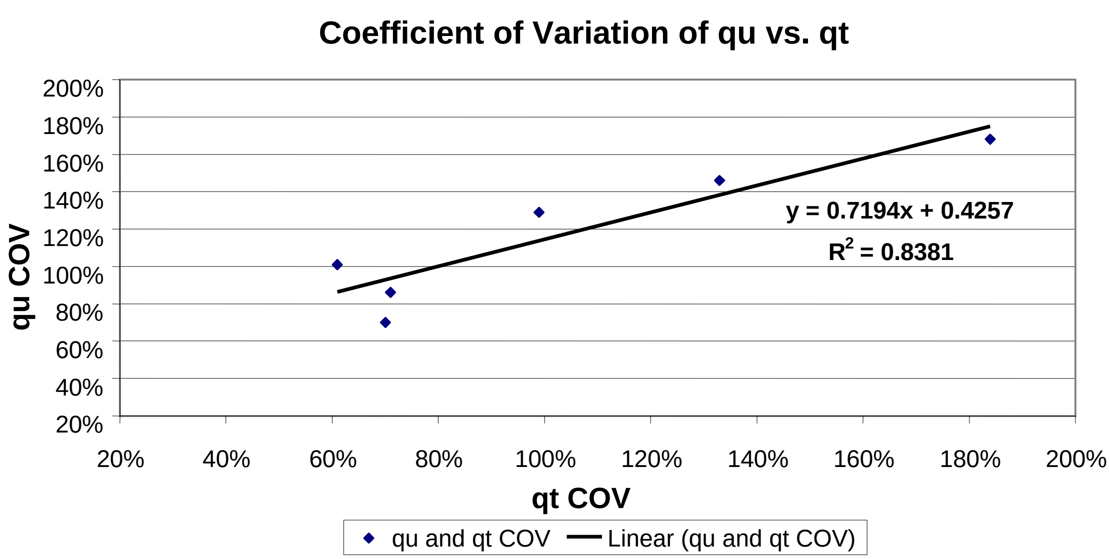


Figure 7.26 Scattergram Plot of the Coefficient of Variation qu and qt

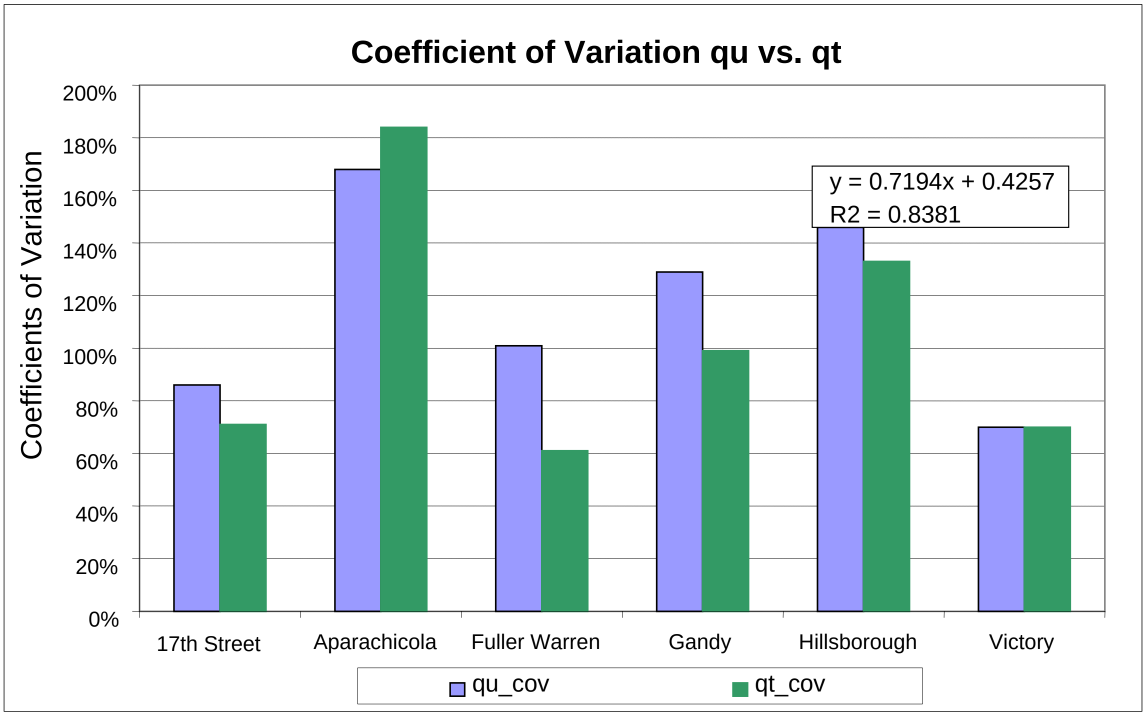


Figure 7.27 Histogram Between the Coefficient of Variation qu and qt

Table 7.4 Statistical Analyses of Percentage Recovery vs. RQD

		Recovery	RQD
Recovery	Pearson Correlation	1	.810(**)
	Sig. (2-tailed)		0.01
	N	976	976
RQD	Pearson Correlation	.810(**)	1
	Sig. (2-tailed)	0.01	
	N	976	976

Based on the Pearson Correlation and Sig. (2-tailed) values, RQD and % Recovery are correlated. Specifically, the analysis shows a progressive increase in RQD with corresponding increase in % Recover. However there are exceptions, i.e., when the rock is interleaved with clay instead of sand (Victory), then the % Recovery jumps relative to RQD. The latter is important, since the FDOT recommends in their "Soil Handbook" to adjust the predicted unit skin friction by the average % Recovery to account for voids in the rock (i.e., reduced skin friction). But in the case of Victory Bridge, which has high recoveries, i.e., clay filling the rock voids, the RQD would be a better indication of rock quality and unit skin friction.

The correlation of RQD or % Recovery with rock strength, i.e., qu, was also attempted. However, both showed no statistical correlation. The latter may be expected, since both measure different quantities, i.e., strength, and presence of rock, and must be treated separately.

7.5 CORRELATION BETWEEN ROCK STRENGTH (qu and qt) WITH STANDARD PENETRATION N VALUES

In an attempt to obtain more information on rock strength and variability in each site, an attempt was made to correlate SPT N value with qu and qt values. Subsequently, the N value at each spatial point where either a laboratory qu or qt test had been conducted was gathered. Figure 7.28 and 7.29 show the correlation between N and qu and N and qt, respectively. Evident from the figures, it appears that there is little if any correlation between SPT N values and rock strength. Also, from the variability, it was doubtful if SPT N values could be correlated directly to unit skin friction on the drilled shafts.

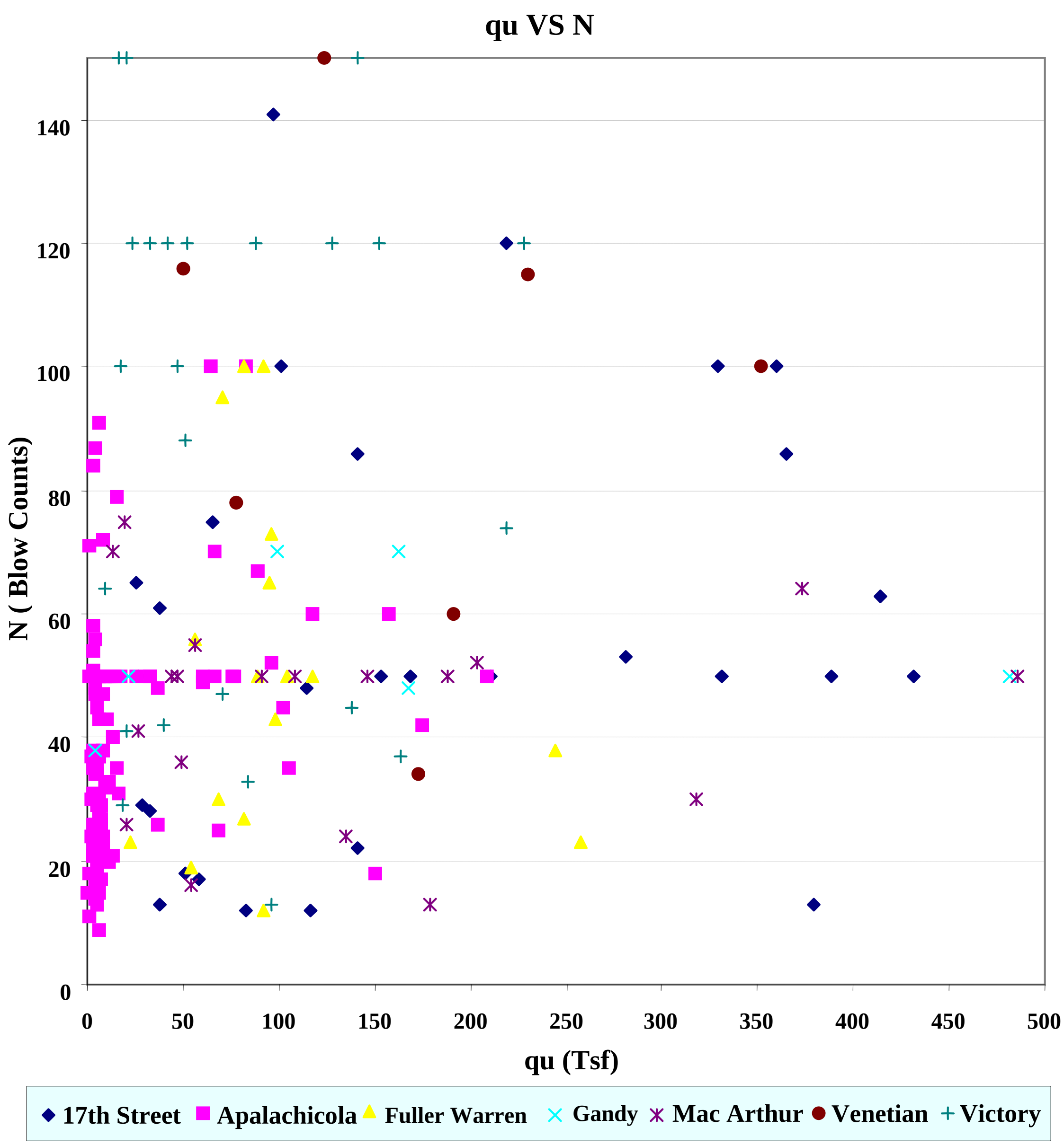


Figure 7.28 Correlation Between N vs. qu (Tsf)

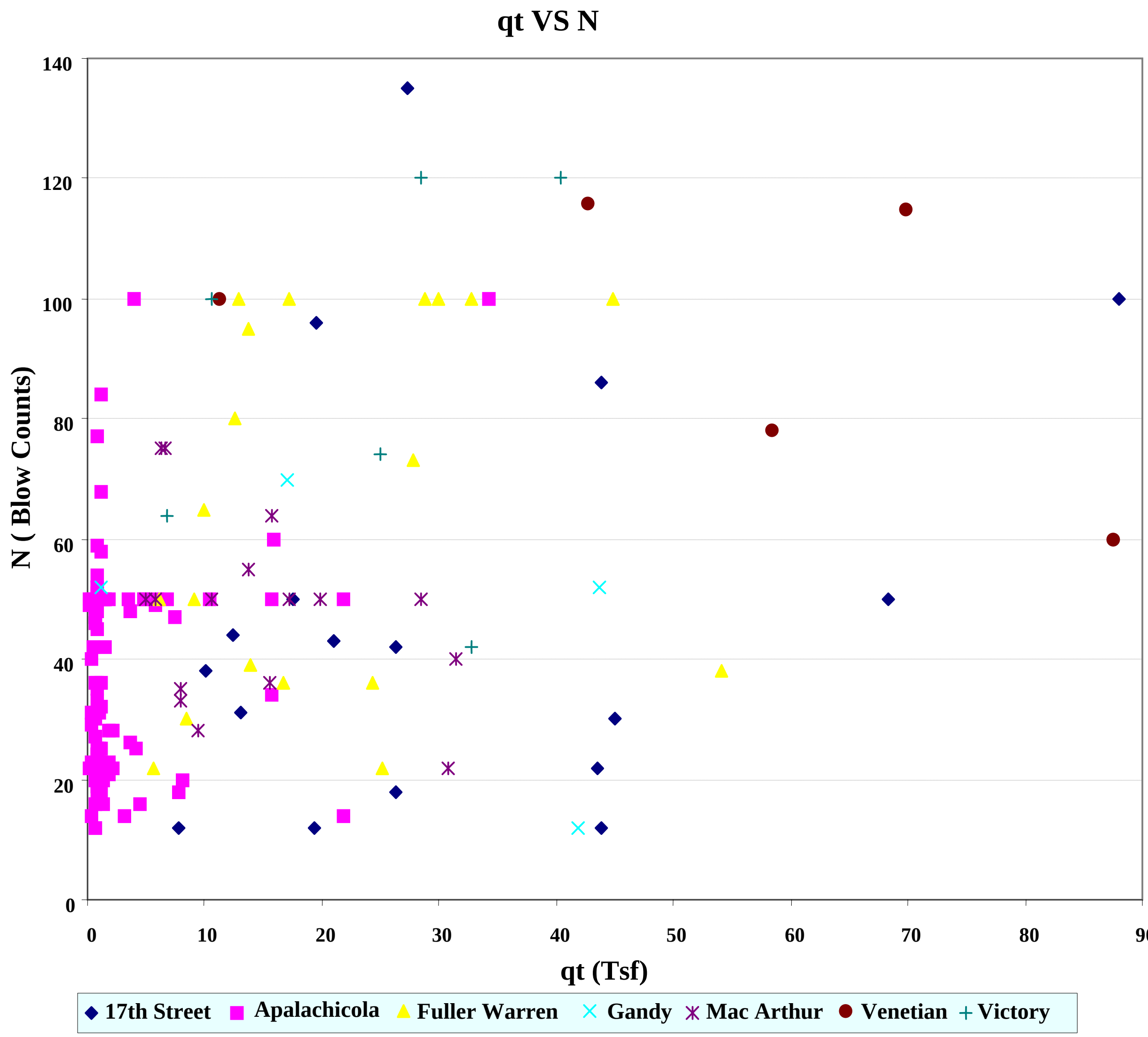


Figure 7.29 Correlation Between N vs. qt (Tsf)

CHAPTER 8
PREDICTED UNIT SKIN FRICTION and LRFD RESISTANCE FACTORS

8.1 STRENGTH DATA ANALYSIS

As discussed earlier (Chapter 7), to predict unit skin friction on a shaft, differences in layering must be considered. The latter may be determined from visual observation of the recovered cores or from the strength frequency distribution plots. In the case of the latter, multiple equal size peaks are strong indications of either layering or presence or multiple formations. Figures 8.1 and 8.2 display the frequency distributions of rock strength (qu and qt) for all the sites. For all the projects studied here, only 17th Street exhibited multiple layers or formations, i.e., different on each side of bridge (see Figs. 7.1 and 7.2) and will be treated separately.

Once the layer formations have been identified, current FDOT practice is to compute the mean of both the unconfined compression, qu, and split tension, qt, sample data to estimate the unit skin friction. Evident from Figures 8.1 and 8.2, the sample distributions are not normal (i.e., equally distributed about the mean). In addition, the outliers (Figs. 8.1 and 8.2) are skewed to the high side of the mode or median of the data, increasing the standard deviation of the data (Fig. 8.3) as well as resulting in a un-conservative, i.e., over estimate skin friction compared to majority of the data (i.e., mode).

Also, in the original development of the FDOT unit skin friction assessment, it was assumed that the rock at the interface of the shaft failed (McVay et al., 1991), i.e., rock strength has to be less than the concrete strength. Evident from Figures 8.1 and 8.2 the median and mode for all the data is well below typical concrete strength. However, some of the outliers, i.e., above the mean, exceed typical concrete strengths. Based on the maximum unit skin friction

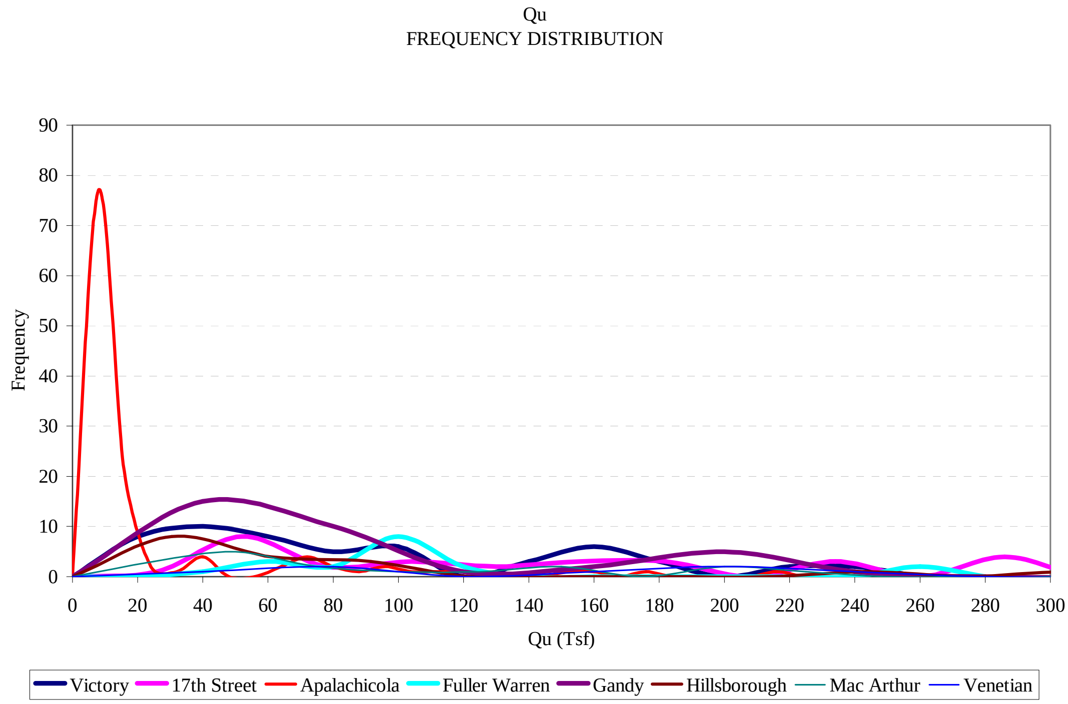


Figure 8.1 qu Frequency Distribution (100 % of Population)

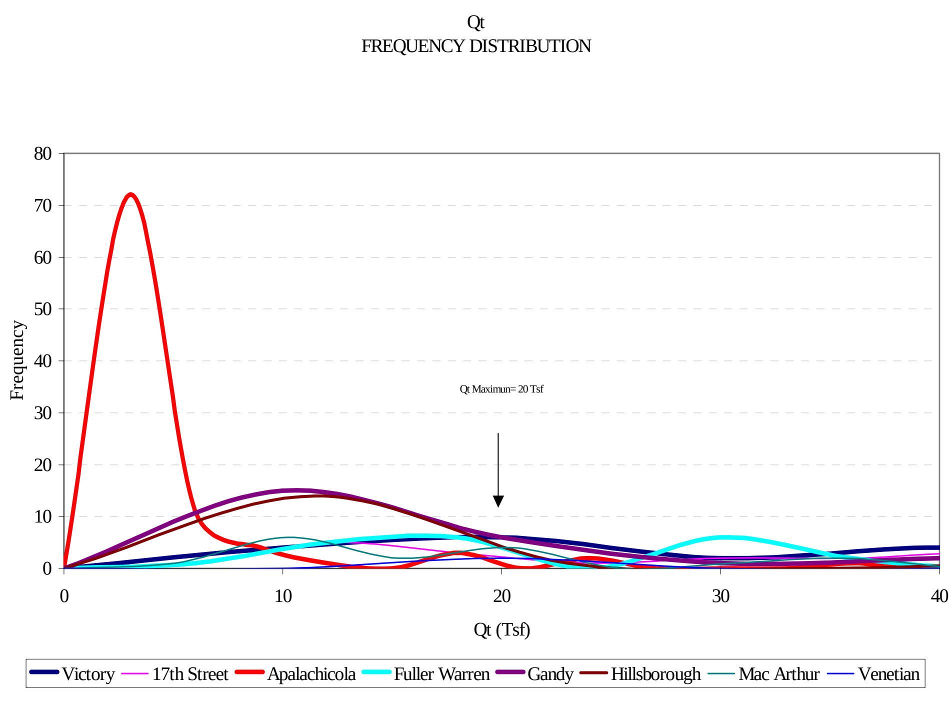


Figure 8.2 qt Frequency Distribution (100 % of Population)

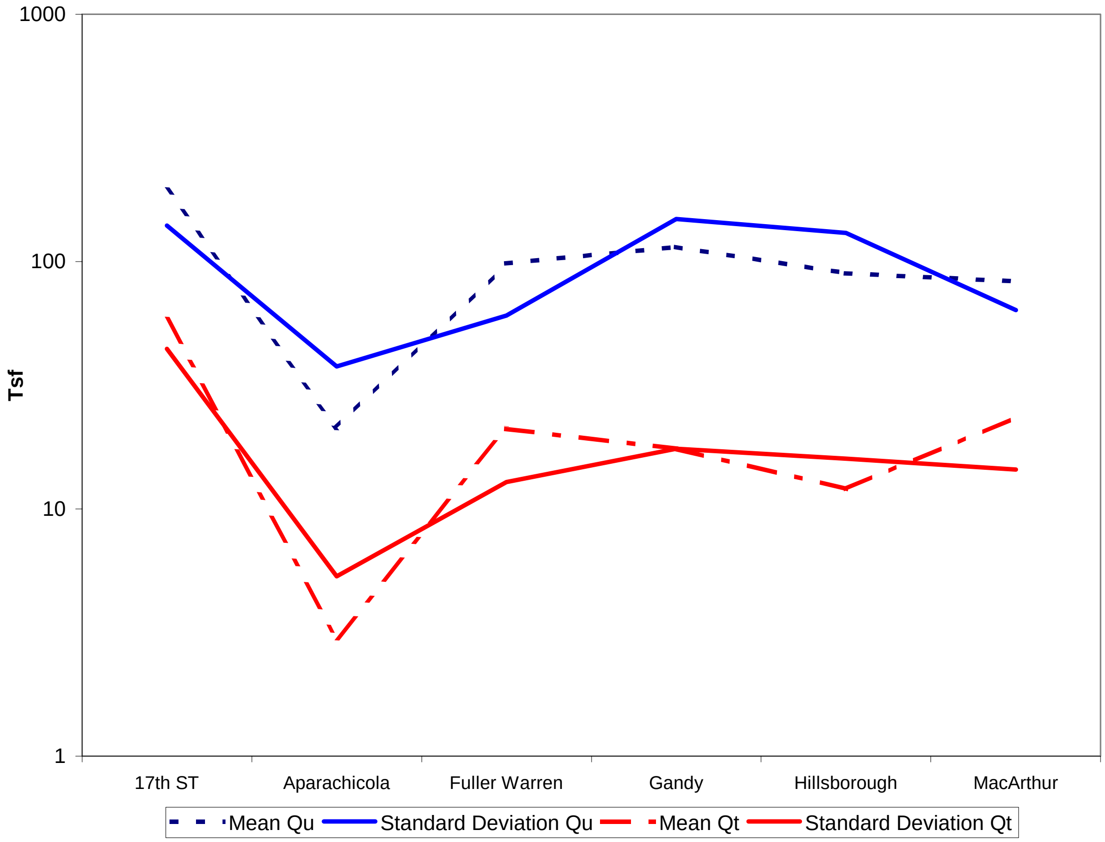


Figure 8.3 Mean and Standard Deviation of Data

measured at discrete points on the shafts (see chapter 4), it was decided to limit rock strengths to qu < 120 tsf (unconfined compression), and qt < 20 tsf (split tension) for unit skin friction calculations (i.e., assure rock failure). It should be noted that the two separate approaches to estimating unit skin friction, section 8.3 (estimating point variability of unit skin friction on all shafts) and section 8.5 (estimating shaft unit skin friction variability over the sites) handles the rock limiting strength (i.e., qu < 120 tsf, and qt < 20 tsf) differently. In the case of the point variability of shaft unit skin frictions (section 8.3), all the outliers greater than qu > 120 tsf, and qt > 20 tsf are neglected because of their limiting nature. For site variability of unit skin friction (section 8.5), based on probabilistic approach, all strength values are considered but limited qu of 120 tsf and qt of 20 tsf.

8.2 UNIT SKIN FRICTION ADJUSTED for % RECOVERY and ROCK QUALITY (RQD)

Either RQD or Percentage Recovery may be used to adjust the value of limestone unit skin friction due to voids, infill, etc. with similar results (see section 7.4) with the except for clay infill. FDOT recommends in their "Soil Handbook" an adjustment of the predicted unit skin friction by the average Percentage Recovery calculated for the site. It has been found in this research that such method would slightly under predict unit skin frictions at low to mid % Recoveries (i.e., 30% to 60%) and slightly over predict high % Recoveries. Consequently, an adjustment has been developed based on the typical recoveries and RQD for Florida Limestone and statistical analysis as follow:

A regular statistical analysis with 95% confidence level is shown in Table 8.1 on % Recovery for all the sites. For instance from the analysis (table), there is a 95% confidence level that all the sites will have 43% recovery. From the result of the statistical levels and a comparison of measured to predicted unit skin frictions, an empirical curve of recovery adjustment was created (Figure 8.4). The latter adjustment is recommended for all limestone formations, which have sand infill or voids. In the case clay infill, e.g., Victory Bridge, % Recovery is not an

Table 8.1 Confidence Level for Recoveries

			Percentage	Std. Error
RECOVERY	Mean		58.16	6.52
	95% Confidence	Lower Bound	42.73	
		Upper Bound	73.58	
	5% Trimmed Mean		59.17	
	Median		63.98	
	Variance		340.29	
	Std. Deviation		18.45	
	Minimum		20.62	
	Maximum		77.36	
	Range		56.74	

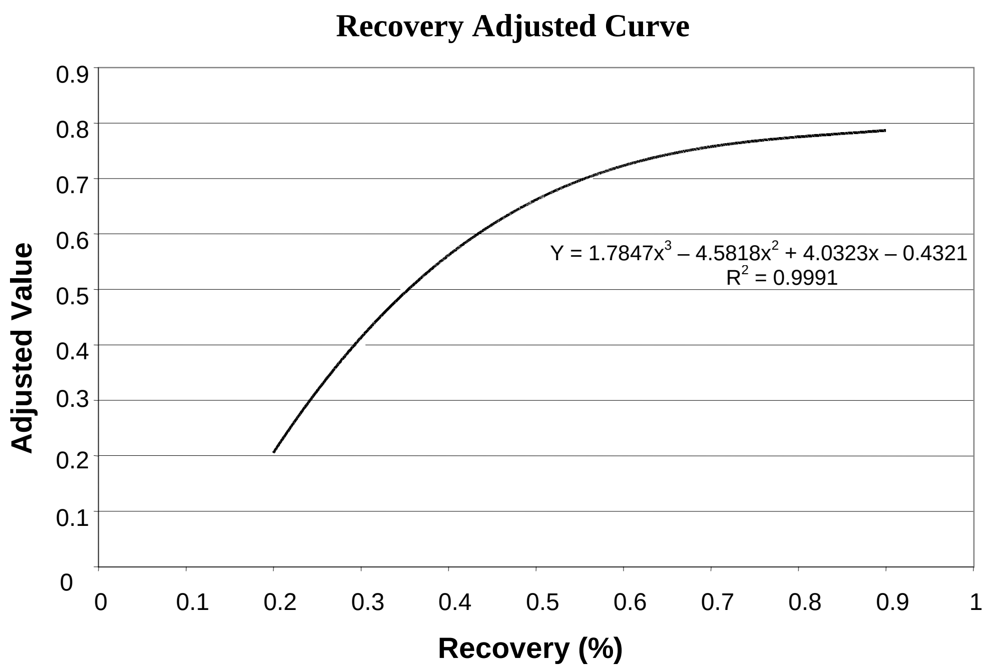


Figure 8.4 Recovery Adjustment Chart

accurate assessment of quantity of limestone at the site. For instance, core recoveries of clay may be obtained which is not limestone. For the latter cases (e.g., Victory Bridge) the Rock Quality Designation (RQD) value is recommended since clays will usually occur in lengths smaller than 4 inches.

8.3 PREDICTED AVERAGE UNIT SKIN FRICTIONS AND POINT VARIABILITY

As pointed earlier, the open literature identifies a number of different methods (FHWA, Kulhway, etc.) to estimate unit skin friction. For this study, the FDOT general method, $fs = \frac{1}{2}\sqrt{qu}\sqrt{qt}$, which attempts to characterize the rock's cohesive strength based on its unconfined compressive strength (qu) and tensile strength (qt) was employed. The latter differs from the single parameter models (i.e., qu: Kulhway, Carter, etc.), because it estimates fs from two independent parameters (i.e., qu, and qt).

With the FDOT design procedure, the use of recovery ratios to account for the voids and infill in the rock (section 8.2) was employed. Specifically, the unit skin friction was computed from the FDOT equation as follows:

$$fs = \frac{1}{2}\sqrt{qu}\,\sqrt{qt} * \text{ (Adjusted \% REC or RQD)} \qquad \text{Equation 8.1}$$

where Adjusted % REC was obtained from Figure 8.4. The following process was used to calculate the skin friction:

- qu and qt values were obtained from the Geotechnical Reports. (Data Collection)
- Statistical Analysis was performed (i.e., Frequency Distribution)
- Mean, Standard Deviation, as well as Minimum and maximum values of strength data was determined from the data of qu < 120 tsf and qt < 20 tsf.
- Using the strength data, the mean unit skin friction was found for each project from Eq. 8.1 using the mean strength data, as well as one standard deviation above and below using one standard deviation in strength data.
- The adjusted recover/RQD (Fig 8.4) was used in estimating (Eq. 8.1) the unit skin friction (Table 8.2).

Shown in Table 8.2 is the predicted mean, as well as point variability of unit skin friction for all the sites. The latter referred to as **Min** and **Max** in Table 8.2 was obtained from Eq. 8.1 by adding or subtracting one standard deviation of the strength data (qu and qt) in the fs calculation. It represents the predicted variability of unit skin friction on any shaft (i.e., point) over the entire site. Because the FDOT equation for skin friction uses square root of both q_u and q_t the variability of transformed fs is lower than the strength variability (qu and qt).

8.4 MEASURED vs. PREDICTED UNIT SKIN FRICTION POINT VARIABILITY

Presented in Table 8.3 and Figure 8.5 is a comparison between measured and predicted unit skin friction for each shaft and every point on the shaft (i.e., Chapter 4). The solid lines represent mean values (measured and predicted) and the dashed lines are one standard deviation

Table 8.2 Predicted Unit Skin Frictions

Bridge	fs (tsf; max value qu=120, qt=20)						
	Min	Mean	Max	St. D	Actual Recovery	Actual RQD	Adjusted Recovery/RQD
17th Street (hard)	4.27	6.31	8.35	2.04	24%	6%	0.30
Apalachicola	0.10	2.78	7.01	4.23	56%	30%	0.70
Fuller Warren	8.52	12.74	16.97	4.22	58%	37%	0.71
Gandy	4.00	10.64	17.27	6.64	83%	56%	0.78
Hillsborough	1.94	7.84	13.74	5.90	70%	27%	0.76
Victory	4.54	7.50	10.46	2.96	67%	42%	0.42

above and below the mean values (measured and predicted). Evident from Table 8.3 or Figure 8.5 the comparisons are quite good with a high correlation (R_{xy} = .96, p = .002) between measured and predicted. The study suggests that the FDOT unit skin friction approach is quite acceptable with small corrections (Figure 8.4; qu < 120 tsf; and qt < 20 tsf). In terms of design however, the variability of the average unit shaft skin friction is of interest instead of point variability on any shaft. It should be noted that the soft rock 17th Street results (Chap. 4) were neglected due to insufficient strength data.

Table 8.3 Predicted Unit Skin Friction vs. Measured Unit Skin Friction

Bridge	Measured fs (tsf) (Load Test With Standard)				Predicted fs (tsf) (Max value qu = 120, qt = 20)			
	Min	Mean	Max	St. D	Min	Mean	Max	St. D
17th Street (hard)	2.25	5.95	9.65	3.70	4.27	6.31	8.35	2.04
Apalachicola	1.90	4.30	6.70	2.40	0.10	2.78	7.01	4.23
Fuller Warren	7.85	14.20	20.55	6.35	8.08	12.09	16.09	4.01
Gandy	2.66	9.76	16.86	7.10	3.96	10.53	17.10	6.57
Hillsborough	3.28	8.71	14.14	5.43	1.94	7.84	13.74	5.90
Victory	4.64	8.10	11.56	3.46	4.54	7.50	10.46	2.96

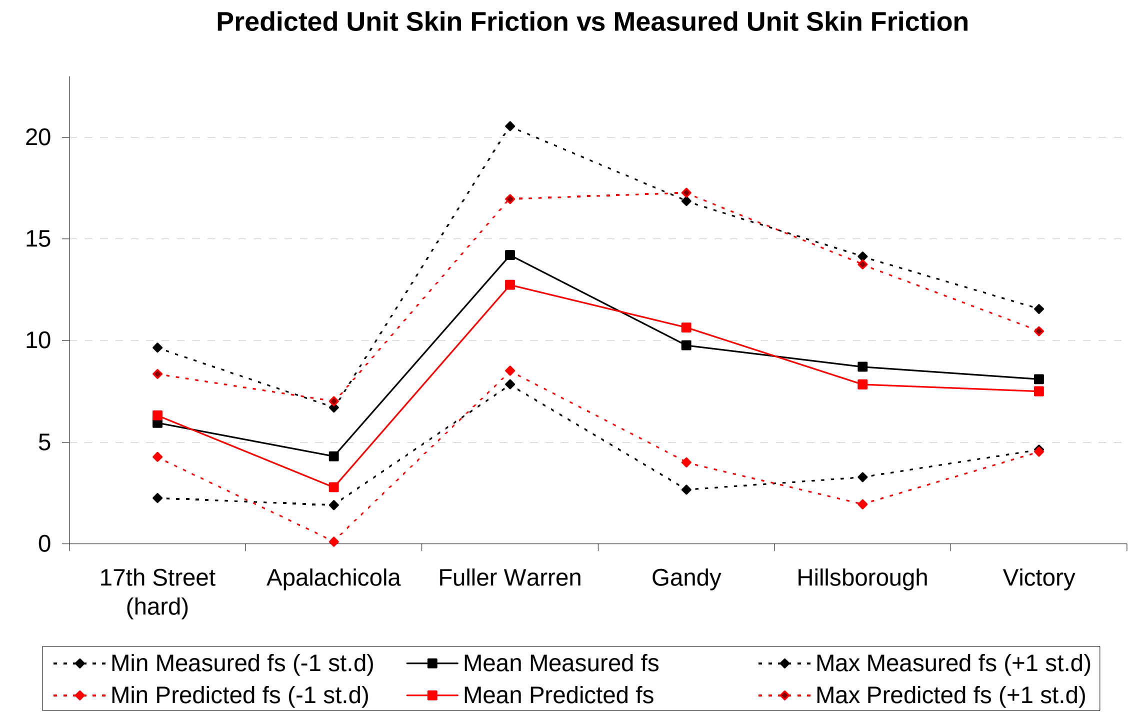


Figure 8.5 Predicted Unit Skin Friction vs. Measured Unit Skin Friction

8.5 VARIABILITY of SHAFTS' AVERAGE UNIT SKIN FRICTION OVER a SITE

For each site, all of the strength data (qu and qt) were analyzed for correlation with depth, RQD, % Recovery and Spatial Dependency. With the exception of 17th Street (each side of bridge treated separately) all of the data showed no correlation and were subsequently considered as homogeneous random populations. Consequently, if a drilled shaft was installed in any of the rock formations, the chances of getting a particular set of strength characteristic (i.e., qu and qt) were considered random. Moreover, based on the typical unit skin friction variability reported on the shafts (see chapter 4), 6 to 10 different values (i.e., strengths) were representative of any given shaft. For instance, Figure 8.6 shows the unit skin friction variability for the Gandy shafts.

Of interest in design is the average unit skin friction (Fig. 8.6) for each shaft and its variability over the site. This may be addressed through random sampling of the strength data (qu

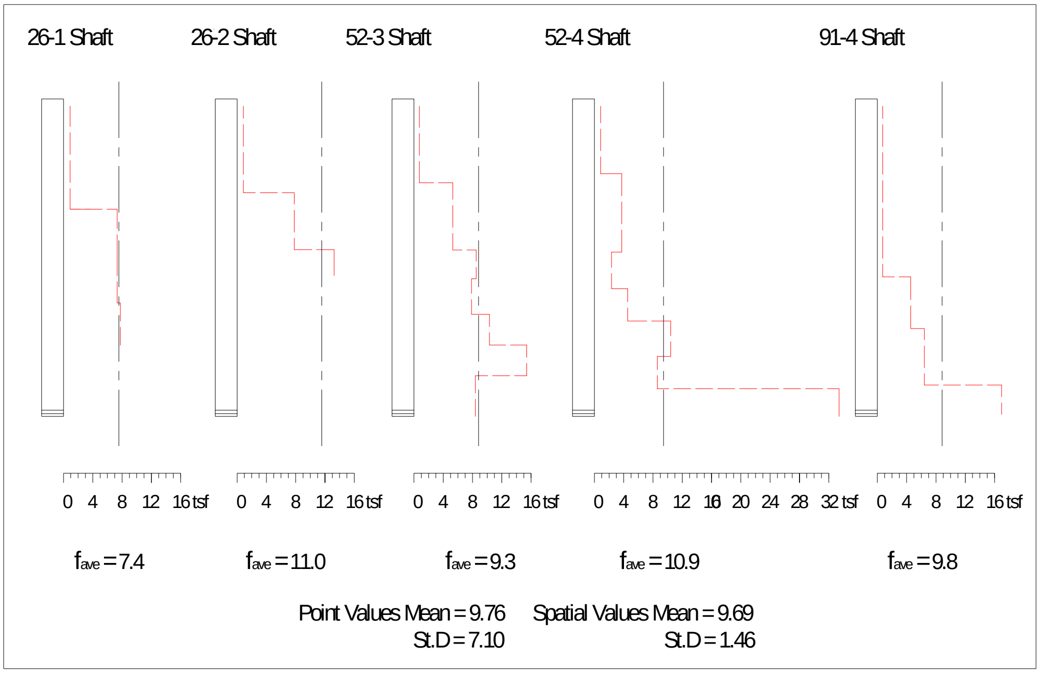


Figure 8.6 Unit Skin Friction Variability for Gandy Load Tests

and qt) and the use of Eq. 8.1. Specifically, it was decided to randomly sample qu and qt six times (average point variability/shaft, Fig 5.4) using a simple random selection (SRS) approach. Subsequently, the means of the six qu and six qt values were averaged to obtain qu_{avg} and qt_{avg}, and were substituted into Eq. 8.1 along with % Recover (Fig 8.4) to obtain an average fs value (Fig. 8.6). Next, another random six samples of qu and qt values were selected and the average fs was computed. The process was repeated (50 to 100 times) until a distribution of average fs was obtained (Figure 8.7). It should be noted that the complete frequency distribution plots for qu and qt were used. In the case of qu values greater and 120 tsf and qt values greater and 20 tsf, they set at cutoff values (i.e., 120 tsf, and 20 tsf: limit of measured unit skin friction). The distribution (Fig. 8.7) has the following characteristics:

(1) The overall shape of the distribution is symmetric and approximately normal
(2) There are no outliers or other important deviations from the overall pattern
(3) Because the distribution is close to normal, we can use the standard deviation to describe its spread.

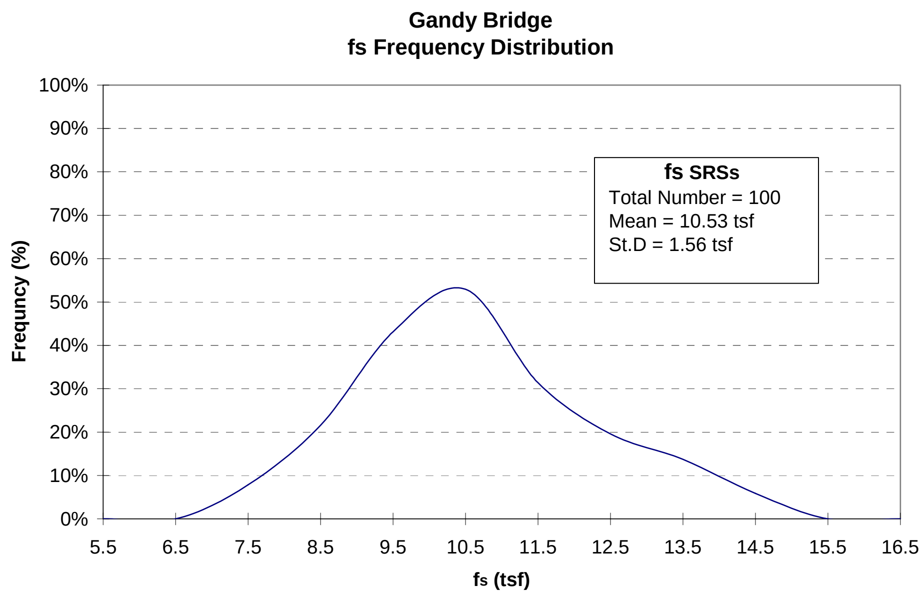


Figure 8.7 Distribution of Average Unit Skin Friction at Gandy Bridge

Of great interest is the standard deviation of the average unit skin friction, Figure 8.7 (1.56 tsf), since it represents the variability of the whole site. This process was repeated for all of the sites with results presented in Table 8.4 and Figure 8.8 along with the measured field variability.

Table 8.4 Measured and Predicted Average Unit Skin Friction and Variability for Sites

f_s	Predicted Mean	Measured Mean	Predicted St. D	Measured St. D
17th Street (hard)	6.13	5.93	0.40	0.38
Apalachicola	2.76	4.12	1.16	1.02
Fuller Warren	12.52	14.27	1.00	0.59
Gandy	10.53	9.69	1.56	1.46
Hillsborough	7.65	8.75	1.35	0.96
Victory	7.60	7.40	0.96	0.69

Evident from Tables 8.3 and 8.4, the predicted mean unit skin frictions are almost identical as expected (random population). However, the standard deviation of the of the average unit skin friction (Table 8.4), is significantly less than the point variability (Table 8.3) and it matches the predicted site variability (Table 8.4) quite well.

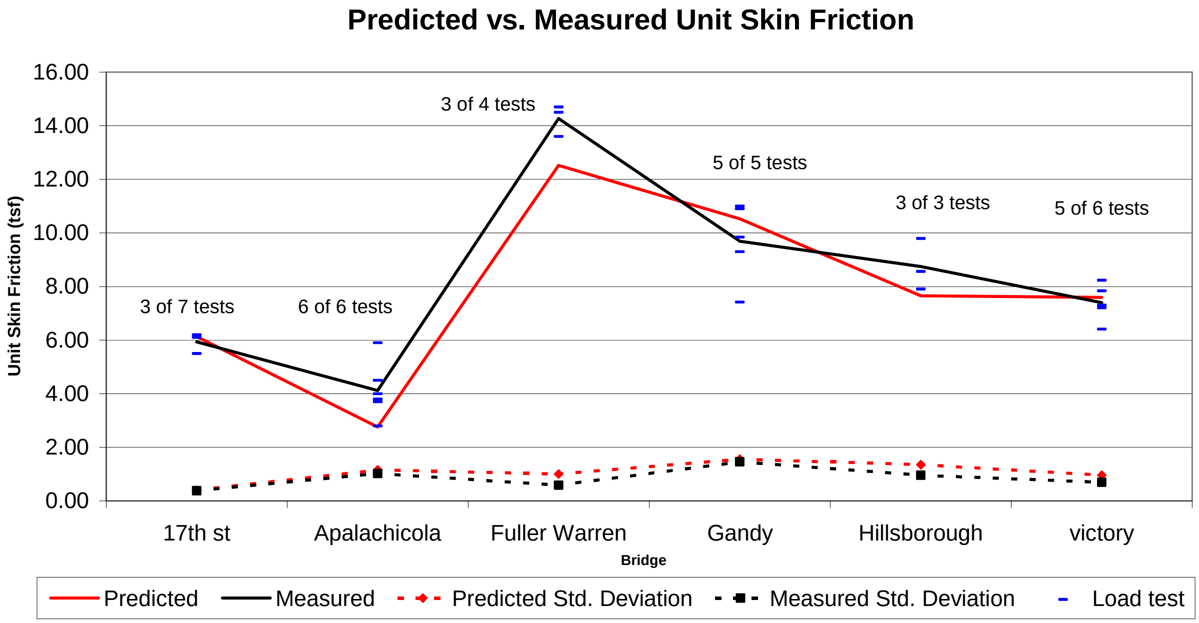


Figure 8.8 Measured vs. Predicted Average Unit Skin Friction with Variability

Shown in Figure 8.8 is a comparison between predicted (Table 8.4) average unit skin friction and the measured average unit skin on each shaft for each site. The number (i.e., 5 of 6, Victory) identifies the shafts, which reached failure (see Chapter 4) on a site. The solid lines represent the mean values and the dashed lines represent the standard deviation, identified as site variability for the predicted and measured unit skin friction. According to Figure 8.8, the repeated simple random samples approach does an excellent job of predicting both the mean and site variability of the average unit skin friction. Statistically the correlation between the predicted and the measured unit skin friction and its variability was significant (r=.79, $\underline{p}$<.01). Also, it should be noted that variability between Osterberg and Statnamic results was within the site variability.

8.6 LRFD PHI FACTORS (or F.S.) BASED on RELIABILITY (RISK)

The LRFD specifications as approved by AASHTO recommend the use of load factors to account for uncertainty in the loads, and resistance factors to account for the uncertainty in the materials. The design criterion may be expressed as:

$$\phi R_n \geq \eta \sum \gamma_i Q_i \qquad \text{Equation 8.2}$$

where: R_n = Nominal resistance (i.e. predicted average unit skin friction),

η = Load modifier to account for effects of ductility, redundancy and operational importance. The value of η usually ranges from 0.95 to 1.00. Herein, η = 1.00,

Q_i = Load,

γ_i = Load factor. Based on current AASHTO recommendation, the following factors are used: γ_D = 1.25 for dead load, γ_L = 1.75 for live load,

ϕ = Resistance factor (Typically ranges from 0.3 to 0.8).

The ϕ factor can be viewed as the reciprocal of factor of safety (FS) in ASD. There are three levels of probabilistic design that can be used to estimate resistance factors, ϕ for deep foundations. The one recommended by AASHTO and FHWA, is referred to as the first order second moment method (FOSM). According to Barker, et al., 1991 and Withiam, et al., 1997, using an assumption of lognormal distribution function for resistance (R_n), and bias factors (λ_R, λ_{QD}, λ_{QL}), the resistance factor, ϕ, can be obtained from:

$$\phi = \frac{\lambda_R \left(\gamma_D \dfrac{Q_D}{Q_L} + \gamma_L\right) \sqrt{\dfrac{1 + COV_{QD}^2 + COV_{QL}^2}{1 + COV_R^2}}}{\left(\lambda_{QD} \dfrac{Q_D}{Q_L} + \lambda_{QL}\right) \exp\left(\beta_T \sqrt{\ln\left[(1 + COV_R^2)(1 + COV_{QD}^2 + COV_{QL}^2)\right]}\right)} \qquad \text{Equation 8.3}$$

where: γ_D = Dead load factor (1.25, recommended by AAHSTO (*1*)),

γ_L = Live load factor (1.75, recommended by AAHSTO 1996/2000),

Q_D/Q_L = Dead to live load ratio, varies from 1.0 to 3.0 (spans, L = 57-170 ft, ϕ is not very sensitive, a value of 2.0 used herein)

λ_R = Resistance bias factor,

COV_R = Resistance coefficient of variation ,

λ_{QD}, λ_{QL} = Dead load and live load bias factors,

λ_{QD} = 1.08, λ_{QL} = 1.15 (recommended by AASHTO 1996/2000),

COV_{QD}, COV_{QL} = Dead load and live load coefficients of variation,

COV_{QD} = 0.128, COV_{QL} = 0.18 (recommended by AASHTO)

β_T = Target reliability index, AASHTO and FHWA recommend values from 2.0 to 3.

Of major importance in estimating, ϕ, are the resistance's bias factor (λ_R), and the resistance's coefficient of variation (COV_R). Both are computed from: 1) the nominal predicted resistance, R_{ni} (predicted average unit skin friction), and 2) the measured average unit skin friction, R_{mi}, for the whole database as follows:

Mean: $$\lambda_R = \frac{\sum \lambda_{Ri}}{N}$$ Equation 8.4

Standard deviation: $$\sigma_R = \sqrt{\frac{\sum (\lambda_{Ri} - \lambda_R)^2}{N-1}}$$ Equation 8.5

Coefficient of variation: $$COV_R = \frac{\sigma_R}{\lambda_R}$$ Equation 8.6

where: $$\lambda_{Ri} = \frac{R_{mi}}{R_{ni}}$$ Equation 8.7

R_{mi} = Measured resistance, average unit skin friction from a load test.

R_{ni} = Predicted resistance, or average unit skin friction from laboratory data.

From the measured and predicted average unit skin frictions from all the sites (Figure 8.8), the mean, λ_R (Eq. 8.4), standard deviation, σ_R (Eq. 8.5), and coefficient of variation, COV_R (Eq. 8.6), were found for the FDOT design approach. Using the computed mean, λ_R (1.06) and coefficient of variation, COV_R (0.28), the LRFD resistance factors, ϕ, were determined for different values of reliability index, β from Eq. 8.3 and are presented in Table 8.5.

Also shown in Table 8.5 is the probability of failure, P_f (%), that the resistance (left side of Eq. 8.2) is smaller than the demand (right side of Eq. 8.2). The latter, P_f, was determined from Rosenblueth and Esteva (1972) equation relating P_f with β:

$$P_f = 460\, e^{-4.3\,\beta} \qquad (2 < \beta < 6) \qquad \text{Equation 8.8}$$

Table 8.5 LRFD Phi Factors, Probability of Failure (P_f) and FS Based on Reliability, β

Reliability, β	LRFD ϕ	P_f (%)	Factor of Safety
2.0	0.81	8.5	1.75
2.5	0.69	1.0	2.0
3.0	0.59	0.1	2.4
3.5	0.51	0.01	2.85
4.0	0.42	0.002	3.35
4.5	0.36	0.0002	3.95

Evident is the significant change in the probability of failure (Risk) for a beta value of 2 (P_f = 8.5%: 8 out of hundred) and a beta value of 3.5 (P_f = 0.01%: 1 out of ten thousand). Note AASHTO typically recommends reliabilities between 2.5 and 3.0 for multiple foundation elements in a group (redundancy, importance, etc.).

Knowing the LRFD phi factors versus probability of failure (risk), its association with Factor of Safety (FS) may also be determined. Allowable Stress Design (ASD), relates the resistance to the estimated loads (or stresses) $\sum Q_i$ as:

$$\frac{R_n}{Fs} \geq \sum Q \qquad \text{Equation 8.9}$$

where: R_n = Nominal resistance,

Fs = Factor of safety—usually from 2.0 to 4.0, and

ΣQ_i = Load effect (dead, live and environmental loads).

Next relating the nominal resistance, Rn, from Eq. 8.9 (ASD) to Eq. 8.2 (LRFD), the following relationship between Fs and phi was obtained:

$$Fs = \frac{\gamma_D \dfrac{Q_D}{Q_L} + \gamma_L}{\phi \left(\dfrac{Q_D}{Q_L} + 1 \right)} \qquad \text{Equation 8.10}$$

Using a dead load factor, γ_D, of 1.25, a live load factor, γ_L, of 1.75 and a dead to live load ratio, Q_D/Q_L, of 2 (typically varies between 1 and 3), the Factor of Safety, Fs, in Table 8.5 were obtained from Eq. 8.10. Also note that the same probability of failure or risk associated with the LRFD phi values apply to the ASD Factors of Safety.

8.7 CURRENT PRACTICE vs. PROPOSED FS, and RESISTANCE FACTORS, ϕ

Of interest are the Factors of Safety, Fs, (ASD) and Resistance Factors, ϕ, (LRFD) given in Table 8.5 vs. current FDOT practice. Also impacting the study is AASHTO's (1999) practice of allowing the Fs (ASD) to vary between 1.9 and 3.5 depending on the amount of construction control, including load testing as shown in Table 8.6.

Table 8.6 AASHTO (1999) Recommended Fs Based on Specified Construction Control

Increasing Construction Control--------->				
Subsurface Exploration (e.g., SPT, cores, etc.)	[1]X	X	X	X
Static Calculations (e.g., FDOT design method)	X	X	X	X
Dynamic Field Measurements			X	
Static Load Testing			X	X
Factor of Safety, Fs	3.5	2.5	2.0	1.9

[1] X = Construction Control Specified on Contract Plans

Using the loads identified in the plans for the piers, as well as the given shaft sizes, the shaft lengths for all the foundation elements were recalculated based on Eq. 8.11.

$$L = \frac{\text{Factored Shaft Loads (Live \& Dead} \quad - \text{Soil \& Scour)}}{\phi \ fs \ \pi \ D} \qquad \text{Equation 8.11}$$

The average unit skin friction, fs, was obtained for Figure 8.8 for each site, and the resistance factor, ϕ, was selected from Table 8.5 for multiple reliabilities. Shown in Figure 8.9 are predicted shaft socket lengths in the limestone for each site for two different reliabilities (β), 2.0 and 3.0 (see Table 8.5 for associated P_f). Also shown in the figure are the actual rock socket lengths for each site. From the actual lengths, both the reliability, β, and Fs were back computed (Eq. 8.11, and Table 8.5) for each site and shaft diameter.

Evident from Figure 8.9, the as constructed shafts had Factors of Safeties, which varied from 2.2 to 4.8 with an average of 3.2. The reliabilities varied from 2.6 to 4.9 with an average of 3.6. The latter may be slightly conservative (AASHTO $2.5 < \beta < 3.0$) when load testing is performed (see Table 8.6: $1.9 > Fs > 3.5$ depending on construction control) and when end bearing is neglected all together. Note the influence of overlying soil and the removal (i.e., disregard) of all material above the scour depth (see Chapter 4) was considered.

8.8 COST vs. RISK (RELIABILITY) of DRILLED SHAFT FOUNDATIONS

Also of interest is the cost vs. the risk (Reliability) of all the drilled shaft foundations on a site. With the reliability (risk) vs. resistance factors, ϕ, values given in Table 8.5, all the shaft lengths for a site may be computed (Eq. 8.11) for different reliability values, β. Then using cost/ft of shaft diameters selected, Figure 8.10, the change in foundation cost vs. risk (or reliability) may be computed as shown in Figure 8.11. The x-axis in Figure 8.11 is the different reliability and associated LRFD phi factor, ASD factor safety, and probability of failure (risk) from Table 8.5; the y-axis is the change in cost of the drilled shaft foundation for the whole site compared to length/cost computed from a reliability, β, of 2.0. Note any β value or risk could be used for the comparison purposes. Also, in the latter calculations, the average cost/ft of a specific diameter of shaft (Fig. 8.10) was employed, with cost data obtained from FDOT records.

Evident from Figure 8.11, the total cost vs. reliability (or risk) varies significantly over some of the sites and relatively little for other sites. Obviously, the change in total cost vs. reliability is a function of the number of shafts on a site, size of shafts, cost of shafts/ft, as well as the average unit skin friction (shown to right of each curve in Fig 8.11) for each site. The effect of the latter is shown from a comparison of Apalachicola (62 shafts: fs = 2.89 tsf) versus Hillsborough (59 shafts: fs = 7.72 tsf).

Figure 8.11 is an excellent tool to aide the design engineer in qualitatively quantifying the number of load tests to perform on a site. For instance, for the sites with steep cost vs. risk plots (i.e., Fuller Warren, Gandy, etc.), the use of a few load tests to reduce ϕ, or Fs, as suggested by AASHTO (Table 8.6), is very cost effective. That is the number of load tests would offset savings in shortening shaft lengths.

Figure 8.11 also suggests that there are some sites were little if any cost savings occur with field-testing. An example of the latter is the Victory Bridge site, where the total bridge foundation cost only increases by a hundred thousand dollars if the reliability increases from 2 (probability of failure 8 in hundred) to a value of 4.5 (probability of failure 2 in a million). Consequently, the design engineer may decide to only perform a single field load test to validate design or ensure proper construction practice.

Whatever the decision, the cost of field load testing has to be known and is the focus of the next two chapters.

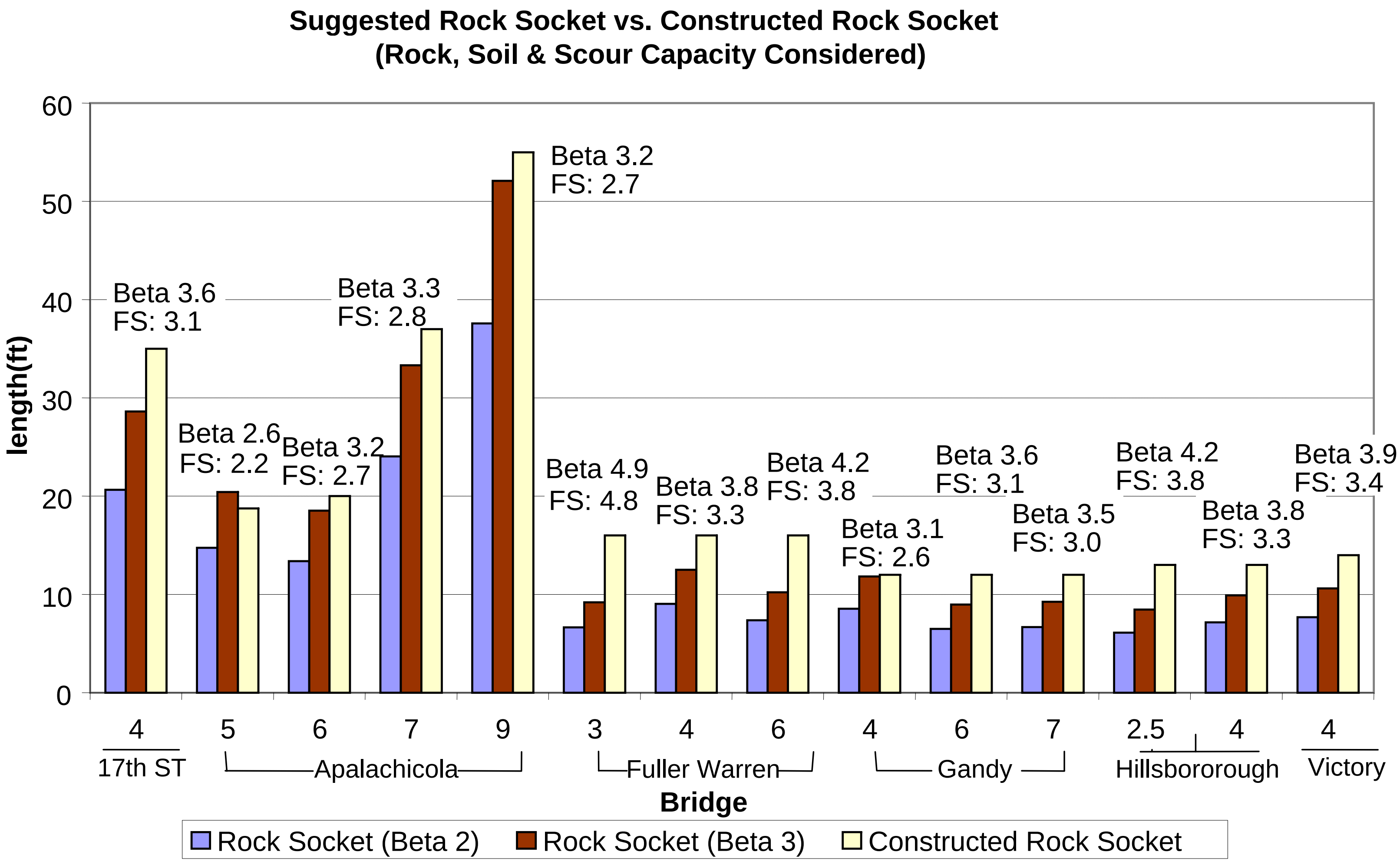


Figure 8.9 Average Rock Socket Lengths for Various Shaft Diameters with Associated LRFD Phi Values

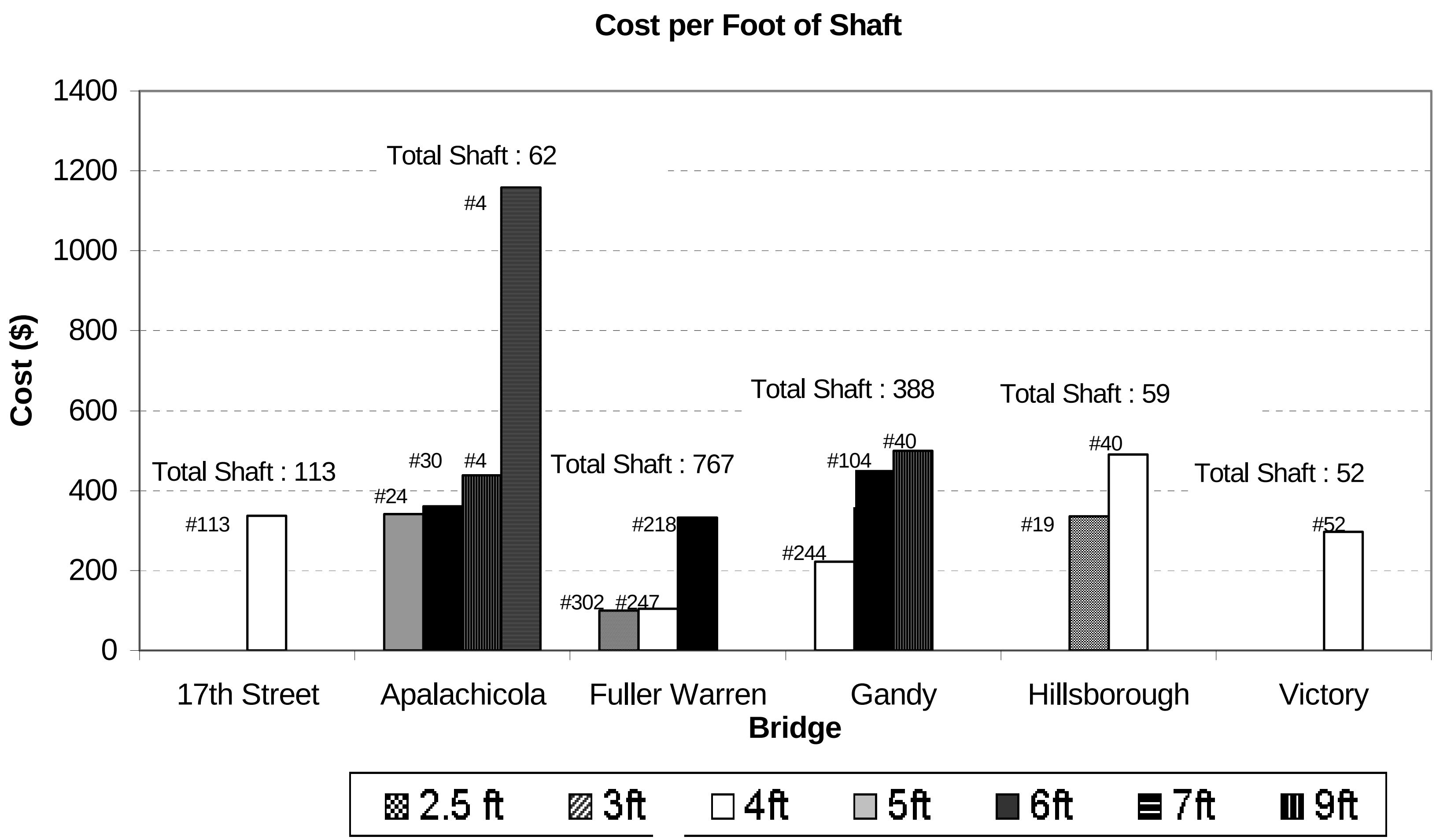


Figure 8.10 Cost Per Foot of Drilled Shafts Based on Diameter

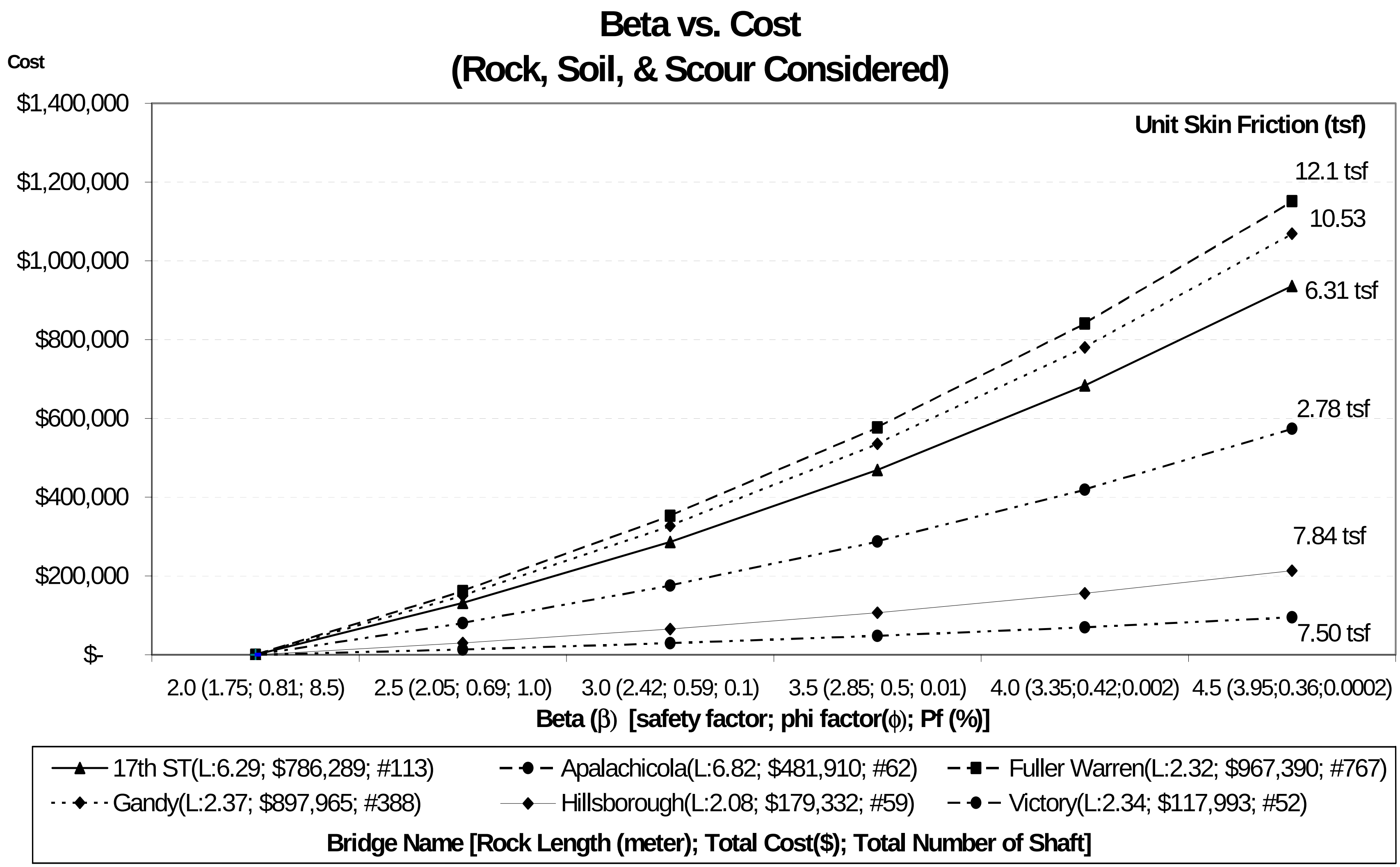


Figure 8.11 Change in Foundation Cost vs. Reliability [Factor of Safety (Fs), LRFD phi, Probability of Failure (Risk)]

CHAPTER 9

ANALYSIS OF THE EFFECTS OF LOAD TESTING ON PROJECT COST

9.1 INTRODUCTION

One key objective of this study was to develop an understanding of the effect of drilled shaft load testing on project cost at the Florida Department of Transportation (FDOT) major bridge projects. When considering a testing strategy for a specific project or when suggesting testing guidelines for a construction program, cost is an essential consideration. Although cost is clearly not the only factor, it remains an issue that should not be ignored.

Therefore, the intent of this chapter is to define the probable actual cost of performing drilled shaft testing. The source of this information is from factual cost data obtained from the FDOT's historical cost database and project records. Additionally, cost information has been obtained from consulting firms experienced in performing drilled shaft load testing.

9.2 DIRECT COST OF DRILLED SHAFT TESTING

9.2.1 Contractor Bid Prices

The Florida Department of Transportation (FDOT) uses a unit price bidding system on its major bridge projects. Contractors submit unit bid prices on multiple bid items for which the owner provides and estimated bid quantity. Computation of the awarded contract value and actual payments on the project are a function of multiplying unit bid prices by the work quantities. The FDOT maintains a system where each bid/pay item is given a unique pay item number and description. Drilled shaft testing involves the following pay items:

Pay Item Number	2455101 1	Osterberg Test (Large Shaft Diameter)
Pay Item Number	2455101 1	Osterberg Test (Small Shaft Diameter)
Pay Item Number	2455102	Statnamic Test
Pay Item Number	2455119	Lateral Test

Bid prices for the above shaft testing pay items for all bids received by the FDOT from 1993 to 1999 were obtained from the FDOT's State Estimates Office database of bid prices. It should be noted that this pricing data includes the bid prices of all bidders including those that were not successful in receiving the project award. In order to investigate the average load testing cost bids for different types of tests, this pricing data has been rearranged by categorizing different pay items respectively. Then, the individual prices were adjusted to the year 2001 using the ENR Building Cost Index because this pricing data spans a significant period of years. Additionally, to provide a better representation of the prices, the outlying values have been dropped. Specifically, values in the top 5 percent and those in the bottom 5% of the distribution have been eliminated. Therefore, the statistical values reported in summary part represent the center 90% of the distribution. Table 9.1, 9.2 and 9.3 show the adjusted bid prices for Osterberg load test that has different pay item numbers.

Bid prices for Statnamic load tests are displayed in Table 9.4, 9.5 and 9.6. Different pay items are laid down corresponding to the weights of load for tests through the FDOT bidding system.

Table 9.7 and 9.8 present unit costs for Lateral load tests. Different pay item numbers indicates that different amount of loads had been used for drilled shaft load testing.

Based on the above analysis for the bid cost for major different shaft test item, a summary of the descriptive statistics is presented as shown in Table 9.9.

Both the Osterberg and the Statnamic tests (30MN) bid costs averaged a little over $100 thousand per test. The Lateral test (901-5300 KN) bid costs averaged a little under $100 thousand per test. The variability was remarkably high for the Osterberg and Statnamic test costs. These variances are attributed to project specific factors.

Table 9.1 Bid Prices for Pay Item Number 2455101 1 on FDOT Projects 1994-1999

Osterberg Load Test (Less Than Five Cells)									
Contract No.	20592	20129	19871	19783	19521	19426	19197	19125	18741
Letting Date	1999	1998	1997	1997	1996	1996	1995	1995	1994
Adj. Year	2001	2001	2001	2001	2001	2001	2001	2001	2001
Unit	Each	Each	Each	Each	Each	Each	Each	Each	Each
Quantity	2	2	4	1	1	2	3	1	4
Bid 02	$128,117.77	$104,667.78	$105,297.27	$84,237.81	$62,377.93	$138,237.59	$198,318.68	$155,421.85	$79,702.35
Bid 03	$65,596.30	$193,248.89	$73,907.71	$139,467.28	$165,885.11	$165,885.11	$170,790.74	$79,702.35	$79,702.35
Bid 04	$102,494.21	$88,790.03	$70,549.17	$100,032.40	$165,885.11	$110,590.07	$136,632.59	$111,362.93	$79,702.35
Bid 05	$90,066.79	$110,957.65	$42,118.91	$68,443.22	$105,060.57	$138,237.59	$125,246.54	$136,632.59	$96,781.42
Bid 06	$153,741.32	$88,790.03	$63,178.36	$368,540.43	$202,950.48	$323,586.55	$113,860.50	$113,860.50	$58,068.85
Bid 07	$106,715.37	$417,835.45		$73,708.09	$442,360.29	$121,649.08	$199,255.87		$75,147.93
Bid 08	$102,494.21	$88,790.03			$199,062.13	$138,237.59	$170,790.74		$74,009.32
Bid 09	$112,743.63				$330,606.81	$197,531.40	$194,701.45		$58,649.54
Bid 10							$85,395.37		$85,395.37
Bid 11							$104,388.47		$68,316.30
Bid 12							$341,581.49		$83,810.43
Bid 13							$199,255.87		$79,702.35
Bid 14							$348,413.11		$91,088.40
Bid 15									$79,702.35
Bid 16									$142,325.62
Summary	Low =	$63,178							
	Average =	$127,941							
	High =	$341,581							
	STD =	$60,379							
	N =	67							

Table 9.2 Bid Prices for Pay Item Number 2455101 1 AA on FDOT Projects 1994-1998

Osterberg Load Test (Less Than Five Cells)			
Contract No.	20214	19426	18734
Letting Date	1998	1996	1994
Adj. Year	2001	2001	2001
Unit	Each	Each	Each
Quantity	1	2	3
Bid 02	$104,459	$248,828	$94,990
Bid 03	$104,459	$165,885	$13,663
Bid 04		$193,533	$56,930
Bid 05	$78,344	$199,062	$97,465
Bid 06	$105,579	$263,656	$102,474
Bid 07	$365,606	$121,649	$85,395
Bid 08		$165,885	
Bid 09		$274,864	
Summary		Low =	$56,930

		Average =	$144,909
		High =	$274,864
		STD =	$68,730
		N =	17

Table 9.3 Bid Prices for Pay Item Number 2455101 1 AB on FDOT Projects 1995-1998

Osterberg Load Test (Less Than Five Cells)			
Contract No.	20214	19197	
Letting Date	1998	1995	
Adj. Year	2001	2001	
Unit	Each	Each	
Quantity	1	2	
Bid 02	$152,999	$333,597	
Bid 03		$341,581	
Bid 04		$136,633	
Bid 05	$104,459	$144,034	
Bid 06		$113,860	
Bid 07		$182,177	
Bid 08		$199,256	
Bid 09		$398,512	
Bid 10		$85,395	
Bid 11		$154,530	
Bid 12		$318,809	
Bid 13		$171,929	
Summary		Low =	$104,459
		Average =	$196,155
		High =	$341,581
		STD =	$85,764
		N =	12

Table 9.4 Bid Prices for Pay Item Number 2455102 30 on FDOT Projects 1994-1999

Statnamic Load Test (30 MN)								
Contract No.	20757	20595	20592	20248	19871	19197	19125	18741
Letting Date	1999	1999	1999	1998	1997	1995	1995	1994
Adj. Year	2001	2001	2001	2001	2001	2001	2001	2001
Unit	Each	Each	Each	Each	Each	Each	Each	Each
Quantity	2	3	2	2	6	2	2	3
Bid 02	$170,223	$97,370	$128,118	$208,918	$63,178	$121,077	$92,946	$88,811
Bid 03	$256,236	$204,988	$102,494	$83,567	$77,919	$113,860	$62,623	$91,088
Bid 04	$153,741	$108,059	$153,741	$208,918	$91,609	$136,633	$145,938	$85,395
Bid 05	$145,542	$101,096	$94,807	$156,688	$31,589	$79,702	$85,395	$142,326
Bid 06	$153,741	$97,370	$102,494	$62,675	$73,708	$68,316	$113,860	$58,069
Bid 07		$128,118	$96,489	$104,459		$199,256		$85,395
Bid 08		$107,619	$107,619			$170,791		$96,781
Bid 09			$89,170			$170,791		$58,650
Bid 10						$113,860		$91,088
Bid 11						$118,302		$68,316
Bid 12						$284,651		$83,301
Bid 13						$159,405		$79,702
Bid 14						$166,749		$100,197
Bid 15								$85,395
Bid 16								$170,791
Summary		Low =	$62,675					
		Average =	$113,966					
		High =	$204,988					
		STD =	$35,418					
		N =	56					

Table 9.5 Bid Prices for Pay Item Number 2455102 16 on FDOT Projects 1999

Statnamic Load Test (16 MN)			
Contract No.	20505		
Letting Date	1999		
Adj. Year	2001		
Unit	Each		
Quantity	4		
Bid 02	$153,741		
Bid 03	$92,245		
Bid 04	$102,494		
Bid 05	$69,696		
Bid 06	$92,245		
Bid 07	$129,143		
Bid 08	$122,993		
Bid 09	$87,120		
Summary		Low =	$87,120
		Average =	$104,373
		High =	$129,143
		STD =	$17,639
		N =	6

Table 9.6 Bid Prices for Pay Item Number 2455102 10 on FDOT Projects 1996

Statnamic Load Test (10 MN)			
Contract No.	19540		
Letting Date	1996		
Adj. Year	2001		
Unit	Each		
Quantity	3		
Bid 02	$22,118		
Bid 03	$79,625		
Bid 04	$55,295		
Bid 05	$64,791		
Bid 06	$71,884		
Bid 07	$82,943		
Bid 08	$119,253		
Bid 09	$82,943		
Summary		Low =	$55,295
		Average =	$72,913
		High =	$82,943
		STD =	$11,168
		N =	6

Table 9.7 Bid Prices for Pay Item Number 2455119 303 on FDOT Projects 1997

Lateral Load Test (901-5300 KN)			
Contract No.	19871		
Letting Date	1997		
Adj. Year	2001		
Unit	Each		
Quantity	1		
Bid 02	$105,297		
Bid 03	$147,230		
Bid 04	$81,079		
Bid 05	$63,178		
Bid 06	$73,708		
Summary		Low =	$73,708
		Average =	$86,695
		High =	$105,297
		STD =	$16,526
		N =	3

Table 9.8 Bid Prices for Pay Item Number 2455119 302 on FDOT Projects 1994

Lateral Load Test (51-100 Tons)			
Contract No.	18741		
Letting Date	1994		
Adj. Year	2001		
Unit	Each		
Quantity	2		
Bid 02	$22,772		
Bid 03	$22,772		
Bid 04	$22,772		
Bid 05	$28,465		
Bid 06	$34,158		
Bid 07	$22,772		
Bid 08	$22,772		
Bid 09	$34,500		
Bid 10	$22,772		
Bid 11	$39,851		
Bid 12	$18,598		
Bid 13	$34,158		
Bid 14	$42,698		
Bid 15	$17,079		
Bid 16	$34,158		
Summary		Low =	$18,598
		Average =	$27,732
		High =	$39,851
		STD =	$6,759
		N =	13

Table 9.9 Bid Item Cost of Drilled Shaft Testing on FDOT Projects 1993-1999
(Adjusted to 2001 dollars)

Osterberg Load-Test Cost	Statnamic Load-Test (30MN) Cost	Lateral Load-Test (901-5300 KN) Cost
Mean = $ 127,941	**Mean = $ 113,966**	**Mean = $ 86,695**
High = $ 341,581	High = $ 204,988	High = $ 105,297
Low = $ 63,178	Low = $ 62,675	Low = $ 73,708
STD = $ 60,379	STD = $ 35,418	STD = $ 16,526
N = 67	N = 56	N = 3

More specifically, cost data was analyzed on seven recent FDOT bridge projects for which the actual project cost and as-built data was available. In this analysis only the successful contractor's prices were used. Table 9.10 presents the drilled shaft test pricing for the seven case study projects.

Table 9.10 Bid Item Cost of Drilled Shaft Testing on FDOT Case Study Projects

Projects	Contract Number	Osterberg	Statnamic Load Test		Lateral Load Test	
		2455101 1	2455102 30	2455102 4 -	2455119 302 -	2455119 303 -
		(Less Than Five Cells)	(30 MN)	(4 MN)	(51-100 Tons)	(101-600 Tons)
17th Street	19871	$100,000	$60,000			$100,000
Apalachicola	19197	$174,177	$106,338			$12,912
Fuller Warren	19426	$125,000				$50,000
Gandy	18734			$71,230	$13,787	$24,261
Hillsborough	19125	$136,502	$81,632			
Victory	18741	$70,000	$78,000		$20,000	
Mean Cost		$121,136	$81,492	$71,230	$16,894	$46,793

Analysis of the cost data indicates that the Osterberg test had an average price of $121,136. Both Statnamic test costs were somewhat below the Osterberg, at an average of $81,492 and $71,230 each. The lateral test for 51 to 100 Tons had an average cost of $16,894 and the cost for a lateral test for 101 to 600 Tons averaged $46,793.

Table 9.11 presents a comparison of the total shaft testing bid cost to the total bid cost of shaft installation for the case study projects. The total bid cost for installing shaft was calculated by adding the costs for constructing shaft itself and excavation. Shaft testing bid costs averaged 31.2 % of the shaft installation bid costs.

A comparison of the total shaft testing bid cost to the total shaft installing cost for the FDOT projects from 1993 to 1999 has also been made as shown in Table 9.12. The average bid price for load testing was $547,802, and the average bid price for installing shafts was $2,305,234. The testing bid cost is 28.9% of the total average shaft installation bid cost.

Table 9.11 Total Direct Bid Cost of Drilled Shaft Testing on Reviewed Projects
Compared to Total Shaft Installation Bid Cost

Projects	Total Direct Cost ($)		
	Shaft Installation Cost	Total Test Cost	Test Cost as a Percentage of Shaft Installation Cost
17th Street	$2,755,995	$960,000	34.8%
Apalachicola	$2,025,713	$1,334,092	65.9%
Fuller Warren	$6,092,410	$800,000	13.1%
Gandy	$4,662,126	$505,334	10.8%
Hillsborough	$950,434	$299,766	31.5%
Mean Cost	$3,297,336	$779,838	31.2%

In Figure 9.1, a regression analysis was performed to investigate any correlation between the amounts of shaft installing costs and direct testing costs. The result indicated that there was no strong correlation between those two costs with R^2 = 0.1581. The indication is that the relative size of the foundation work has not been a consistent factor in determining the amount of shaft load testing.

Table 9.12 Total Bid Cost of Drilled Shaft Testing on FDOT Projects Compared to Total Shaft Installation Bid Cost 1993-1999

Contract No.	Total Direct Cost of Low Bidder ($)*			
	Letting Date	Shaft Installation Cost	Total Test Cost	Test Cost as a Percentage of Shaft Installation Cost
18734	1994	$5,308,320	$575,376	10.8%
18741	1994	$993,712	$695,688	70.0%
19125	1995	$1,082,168	$341,315	31.5%
19197	1995	$2,306,487	$1,519,004	65.9%
19426	1996	$6,737,601	$884,721	13.1%
19521	1996	$825,483	$62,378	7.6%
19783	1997	$451,515	$84,238	18.7%
19871	1997	$2,901,987	$1,010,854	34.8%
20129	1998	$2,805,487	$209,336	7.5%
20214	1998	$3,351,664	$656,293	19.6%
20248	1998	$1,164,478	$417,835	35.9%
20505	1999	$1,333,566	$614,965	46.1%
20592	1999	$1,271,707	$512,471	40.3%
20595	1999	$1,642,034	$292,109	17.8%
20757	1999	$2,402,296	$340,446	14.2%
Mean Cost		$2,305,234	$547,802	28.9%

* Bid costs were adjusted to year 2001.

Table 9.13 presents a comparison of the total shaft testing bid cost to the total project bid cost for the case study projects. Shaft testing bid cost averaged 2.9 % of the total project cost. Table 9.14 presents a comparison of the total shaft testing bid cost to the total project cost for the FDOT projects from 1993 - 1999. Shaft testing bid cost averaged 2.3 % of the total project cost. These historical bid prices provide a baseline indication of future drilled shaft testing bid costs. However, a more precise estimate of drilled shaft testing cost must be made for each specific future project taking in to account those project specific factors unique to the project in question.

9.2.2 Accuracy of Bid Price Data Base

Since the drilled shaft testing generally occurs during the first trimester of the project construction schedule, there may exist a motivation for the contractor to "front load" the bid price for testing and inflate the pricing. Front loading is certainly a possibility, however the

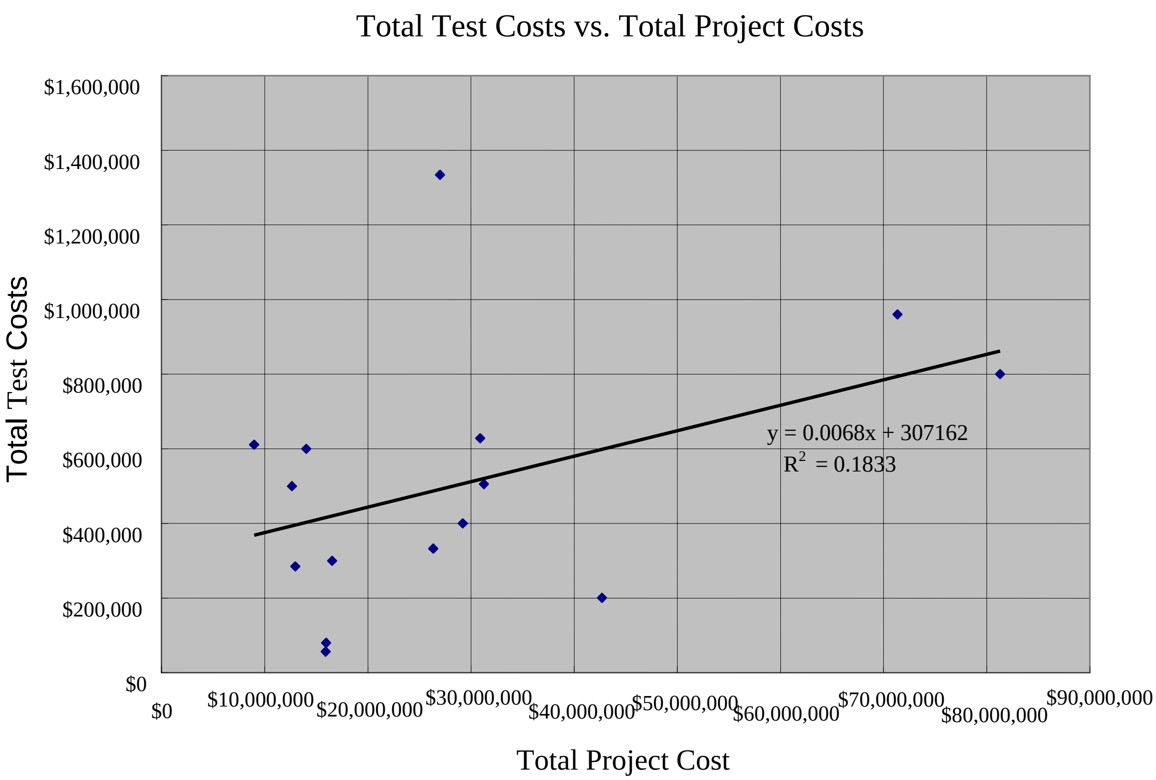


Figure 9.1 Total Shaft Installation Cost vs. Shaft Test Cost

Table 9.13 Total Direct Bid Cost of Drilled Shaft Testing on Reviewed Projects Compared to Total Project Cost

Projects	Total Direct Cost ($)		
	Total Project Cost	**Total Test Cost**	**Test Cost as a Percentage of Total Project Cost**
17th Street	$71,348,263	$960,000	1.3%
Apalachicola	$27,000,000	$1,334,092	4.9%
Fuller Warren	$81,289,714	$800,000	1.0%
Gandy	$31,260,654	$505,334	1.6%
Hillsborough	$16,547,833	$299,766	1.8%
Victory	$8,982,529	$611,000	6.8%
Mean Cost	$39,404,832	$751,699	2.9%

Table 9.14 Total Direct Bid Cost of Drilled Shaft Testing on All FDOT Projects 1993 – 1999

Contract No.	Total Direct Cost of Low Bidder ($)		
	Total Project Cost	Total Test Cost	Test Cost as a Percentage of Total Project Cost
18734	$31,260,654	$505,334	1.6%
18741	$8,982,529	$611,000	6.8%
19125	$16,547,833	$299,766	1.8%
19197	$27,000,000	$1,334,092	4.9%
19426	$81,289,714	$800,000	1.0%
19521	$15,912,842	$56,405	0.4%
19783	$15,962,000	$80,000	0.5%
19871	$71,348,263	$960,000	1.3%
20129	$42,695,327	$200,400	0.5%
20214	$30,896,404	$628,279	2.0%
20248	$29,205,002	$400,000	1.4%
20505	$14,043,510	$600,000	4.3%
20592	$12,649,684	$500,000	4.0%
20595	$12,968,156	$285,000	2.2%
20757	$26,345,531	$332,161	1.3%
Mean Cost	$29,140,497	$506,162	2.3%

research team was unable to determine the extent, if at all, this has occurred. The bid costs for the case study projects compare closely to the historical bid price seven-year period. Additionally, the FDOT State Estimates Office performs statistical reviews of all bids. It is the research team's understanding that the State Estimates Office is unaware of any information to suggest bid unbalancing.

9.3 ESTIMATED REPRESENTATIVE SHAFT TESTING COSTS

The accuracy of the previously reported bid cost for shaft testing may be questionable because of the possibility of front loading. Therefore, to provide additional supporting cost information, a component cost estimate has been prepared as shown in Table 9.15. The cost of testing may include the following cost components:

Table 9.15 Estimated Representative Unit Cost Components for Field Load Testing of Drilled Shafts

Item	Consultant Price to Perform Test ($)	Contractor Support Cost ($)	Contractor Daily Project Overhead Cost at $10,800. per day($)*	Owner Daily Project Overhead Cost at $8,000 per day ($) *	Total Estimated Cost ($)
Osterberg Test of 42 inch diameter shafts (Ea.)	$54,000	$16,000	$21,600	$16,000	$107,600
Osterberg Test of 72 inch diameter shafts (Ea.)	$65,500	$16,000	$21,600	$16,000	$119,100

* Two days estimated time impact

- Load test cost
  - This is the direct cost to the contractor from the consultant for performing and reporting the test
- Contractor support
  - This is the direct cost to the contractor for plant, equipment and personnel required to support the testing operation.
- Contractor Project Overhead
  - This is the direct cost to the contractor for its daily administration, management and staffing support at the project site.
- Owner Project Overhead
  - This is the direct cost to the owner for its daily administration, management and staffing support at the project site.

Project overhead cost for both the contractor and the owner are time related. They are directly related to whether or not the testing extends the duration of the project.

It should be noted that we would expect the general contractor to a markup to the above contractor cost items. The estimated true contractor bid cost assuming a Markup of 15%, is compared to the historical bid prices in Table 9.16. This comparison suggests the possibility of an inflation of approximately 15% to the historical bid prices.

Table 9.16 Comparison of Estimated Contractor Bid Price to Historical Average Bid Price Received on Case Study Projects for Osterberg Testing

Estimated Contractor Direct Cost	Estimated Contractor Markup at 15%	Estimated Contractor Bid Unit Price	Historical Average Bid Unit Price on Case Study Projects
$91,600	$13,740	$105,340	$121,126

9.4 EFFECTS OF DRILLED SHAFT TESTING ON PROJECT COST

9.4.1 Changes in Shaft Lengths

One possible outcome from the drilled shaft testing is a modification of the design shaft tip elevation. Selecting a higher tip elevation will result in a reduction in total shaft lengths and a lower installation cost. As-built installation lengths of drilled shafts were compared to design lengths on five projects. The installation of drilled shafts is also bid as a unit price item on FDOT projects. The total shaft installation cost can be obtained by multiplying the total length by the bid unit prices. A comparison was made of the actual as-built and design lengths. Shaft installation cost savings or excess were compared to the direct cost of drilled shaft testing. Table 9.17 presents a comparison of drilled shaft testing bid cost with corresponding savings in drilled shaft installation bid cost.

Four of the five projects had an as-built drilled shaft bid cost slightly lower than the theoretical as-designed bid cost. However, the average as-built bid cost exceeded the as-design bid cost. The average bid cost of drilled shaft testing was $779,838. The average drilled shaft bid cost savings was - $28,139.

Figures 9.2, 9.3, 9.4 and 9.5 show the correspondence between equivalent shaft lengths and the average load test bid costs. The average load test bid costs divided by the average bid costs for installing different sizes of shafts per unit gave equivalent shaft lengths.

Table 9.17 Comparison of Drilled Shaft Testing Bid Cost and Savings In Drilled Shaft Installation Bid Cost

Project Name	As Built Cost – Planned Cost ($)	Testing Cost ($)
17th St. Causeway	-$56,537.00	$960,000.00
Gandy Bridge	$169,961.28	$505,334.00
Apalachicola	-$59,678.54	$1,334,092.00
Fuller Warren	-$78,556.95	$800,000.00
Hillsborough	-$115,883.59	$299,766.00

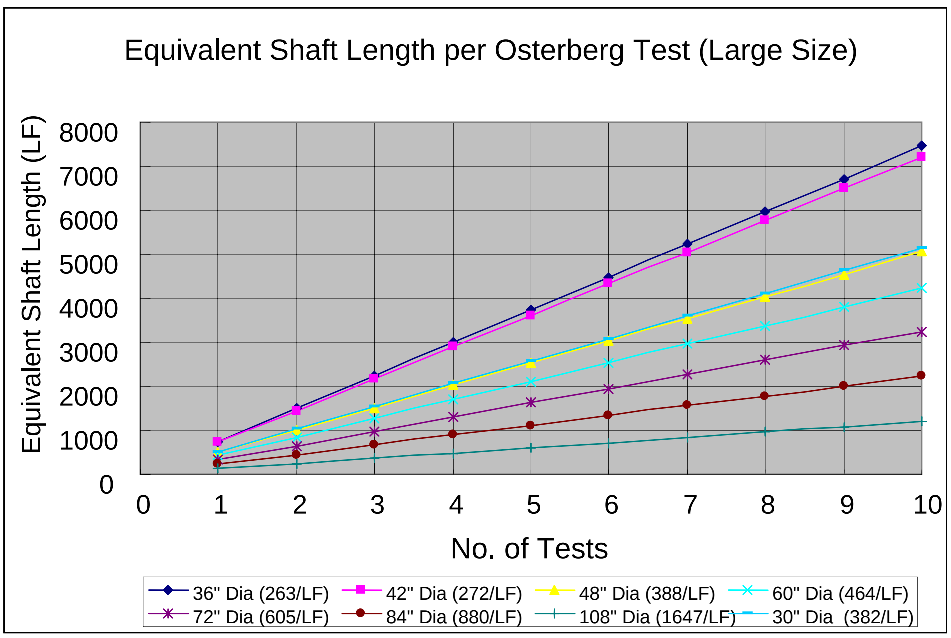


Figure 9.2 Equivalent Shaft Length to the Average Osterberg (Large Size) Test Bid Cost

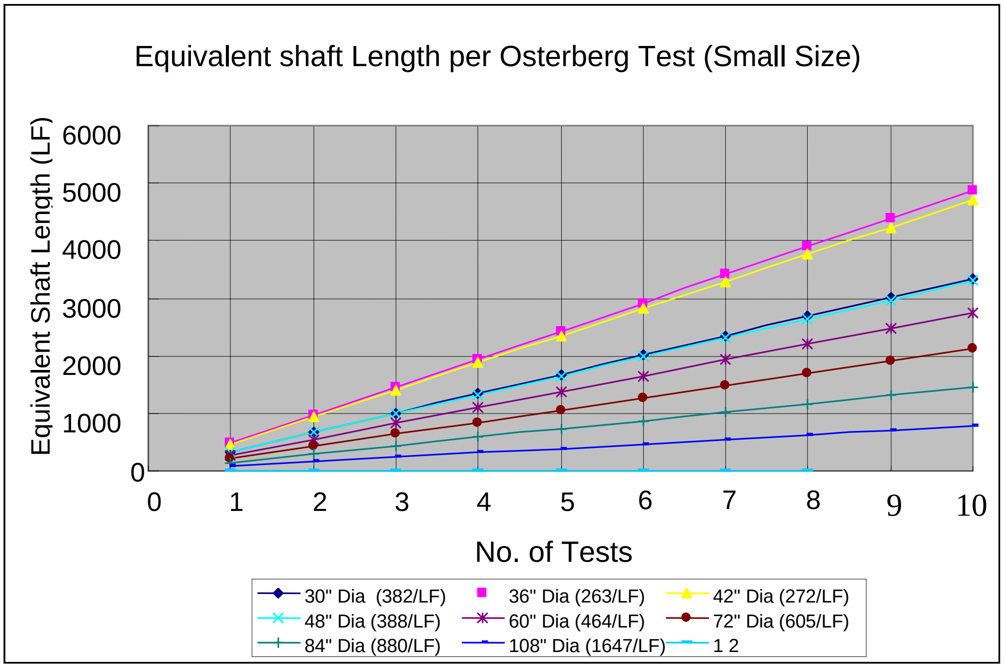


Figure 9.3 Equivalent Shaft Length to the Average Osterberg (Small Size) Test Bid Cost

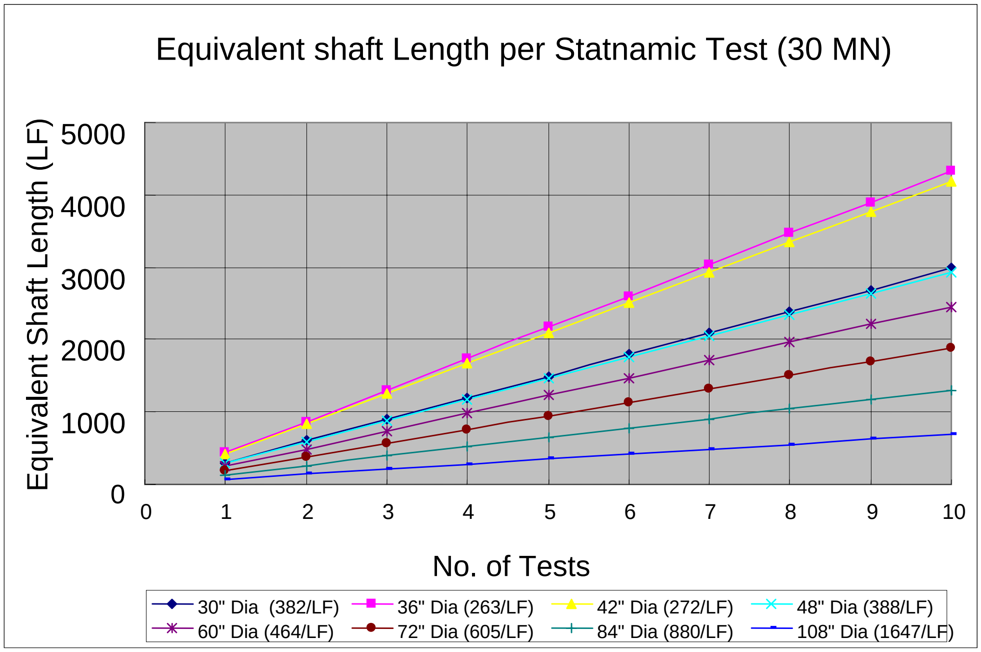


Figure 9.4 Equivalent Shaft Lengths to the Average Statnamic (30 MN) Test Bid Cost

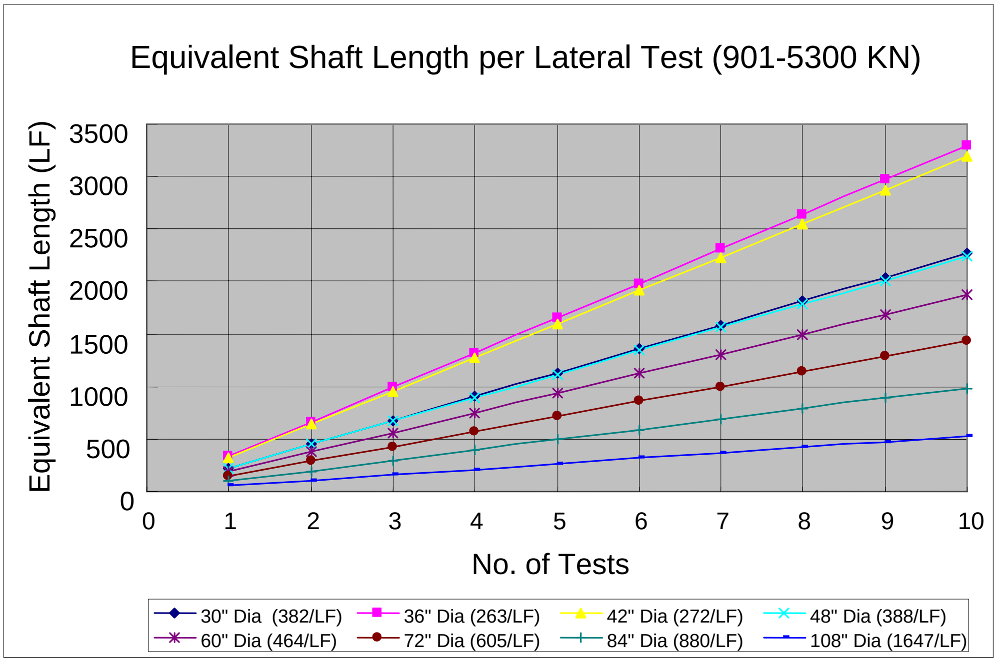


Figure 9.5 Equivalent Shaft Length to the Average Lateral (901-5300 KN) Test Bid Cost

9.4.2 Other Possible Causes for Changes in Tip Elevation

A review of the as-built tip elevation data indicates that different elevation changes occur from shaft to shaft. Appendix A, B and C show the examples of the review studies for FDOT case projects. The random nature of the changes suggests that the elevation changes may be the direct result of the drilling situation at a particular shaft. It appears that much of the change may be to accommodate the geotechnical condition encountered at a particular shaft rather than a global change resulting from the engineering review of a shaft test.

9.5 DISCUSSION OF RESULTS

First, it is appropriate to say that project cost should not be considered the only measure of the benefit of field drilled shaft testing. Drilled shaft testing provides many benefits to the FDOT. As with most professional engineering decisions, benefits must be weighed against cost and risk. However, this discussion is limited to the metrics of project cost.

The information reviewed suggest the following preliminary conclusions:

- Historically load testing of drilled shafts has represented a significant share of the project foundation cost. (Testing bid cost was 45% of the shaft installation bid cost.)
- Historically we see considerable variability in bid unit prices for shaft load testing, which is attributed to market and project specific factors. (For the case study projects the Osterberg test bid price varied from $70,000. to $174,177.)
- Project planners and designers should consider the estimated cost of drilled shaft testing. (Designers are encouraged to consult with the FDOT Estimates Office when developing estimated costs.)

CHAPTER 10
EFFECTS of DRILLED SHAFT TESTING on PROJECT SCHEDULE

10.1 INTRODUCTION

The main purpose of this chapter is to present the effect of drilled shaft load testing on project time for Florida Department of Transportation (FDOT) case study projects. Time is one of the important factors when developing a load testing strategy because it may cause increased job overhead cost, specifically the Construction Engineering and Inspection (CEI) cost, corresponding to the period of load testing. Although some activities for drilled shaft load testing were not on critical path on the schedule, they still required a certain period of time to perform them. Therefore, both direct cost and potential overhead cost should be considered with other factors together when planning drilled shaft load testing program for FDOT major bridge projects.

In this chapter, project as-built CPM schedules have been analyzed to determine the effect of drilled shaft testing on the project duration. The following section describes the step-by-step analysis process.

10.2 THE ANALYSIS OF PROJECT SCHEDULES

A forensic analysis of the five as-built CPM project schedules for the case study projects was performed to determine the effect of drilled shaft testing on the total project schedule. The schedules were basically analyzed according to the following three steps as shown in Figure 10.1.

Figure 10.1 The Analysis Process of As-Built Project Schedules

The first step was to analyze the five as-built CPM project schedules, which were provided from the FDOT. The schedule for Gandy Bridge was looked over on the Microsoft Project Planner, and other four schedules were investigated thoroughly on the P3 (Primavera Project Planner) in order to figure out the load testing activities of drilled shafts. As a result, it was found that the activity descriptions relating to the load testing process were somewhat different for each project schedule. However, it was also found that the work process typically consisted of three phases: construction of the test shaft, performing the test and the review of the test results. Some project schedules actually included all these three activity descriptions, and others showed the combined activity descriptions or one or two descriptions among them.

The second step was performed to develop the schematic schedules of each project graphically based on the analysis results of as-built CPM schedules. Since the original as-built schedules were so complex, the simplified schedules were needed to figure out the effects of load testing on the total project schedules. Focusing on the three typical load-testing activities for drilled shafts made them. However, some simplified schedules illustrated only one or two typical activities because their project schedules did not classify clearly the testing activities into three phases. Figure 10.2, 10.3, 10.4, 10.5 and 10.6 illustrates the simplified load testing schedules of each project.

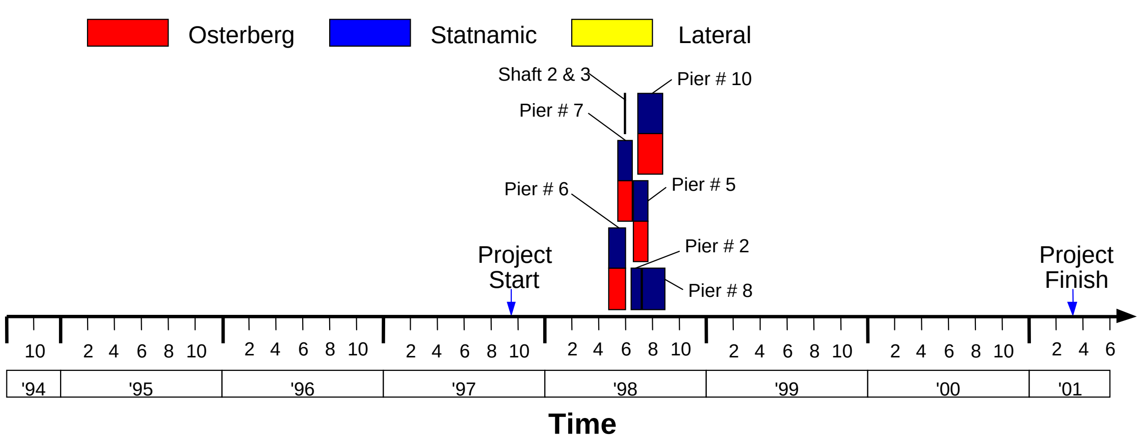


Figure 10.2 The Simplified Schedule of 17th St. Causeway

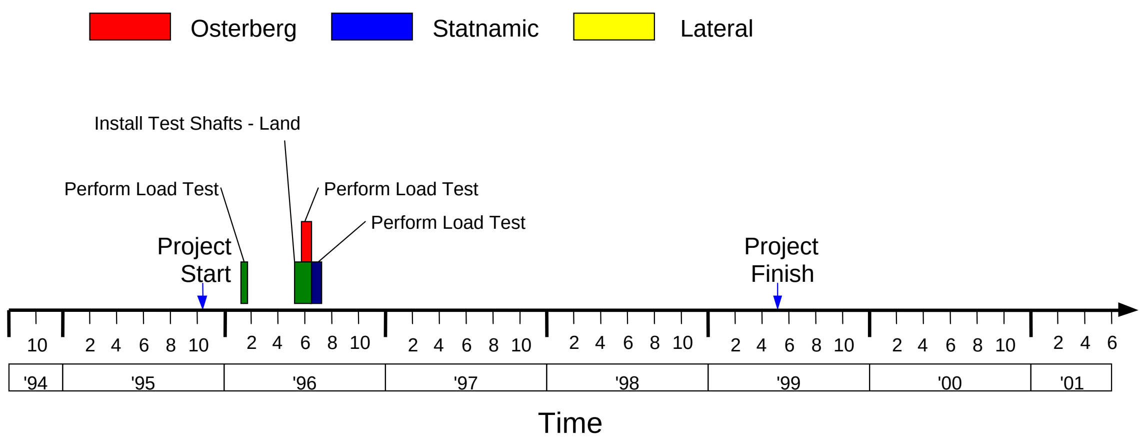


Figure 10.3 The Simplified Schedule of Apalachicola

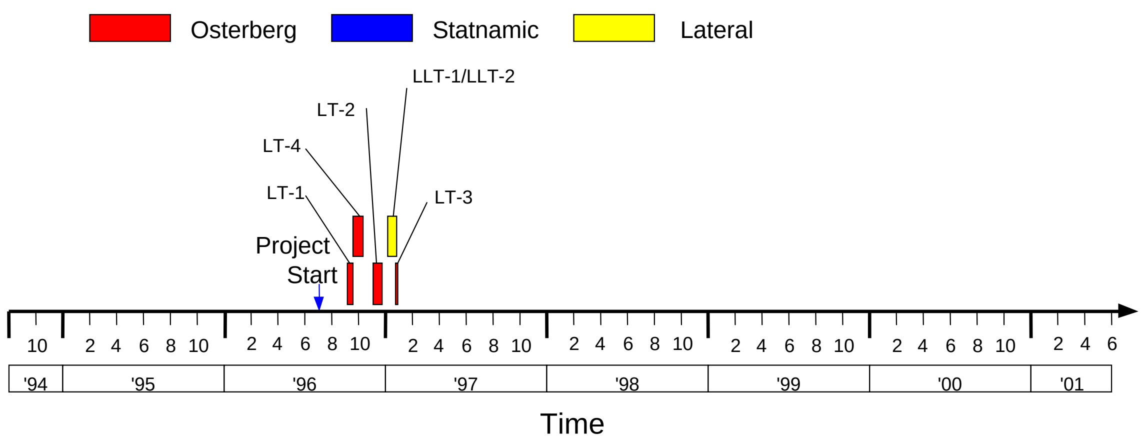


Figure 10.4 The Simplified Schedule of Fuller Warren

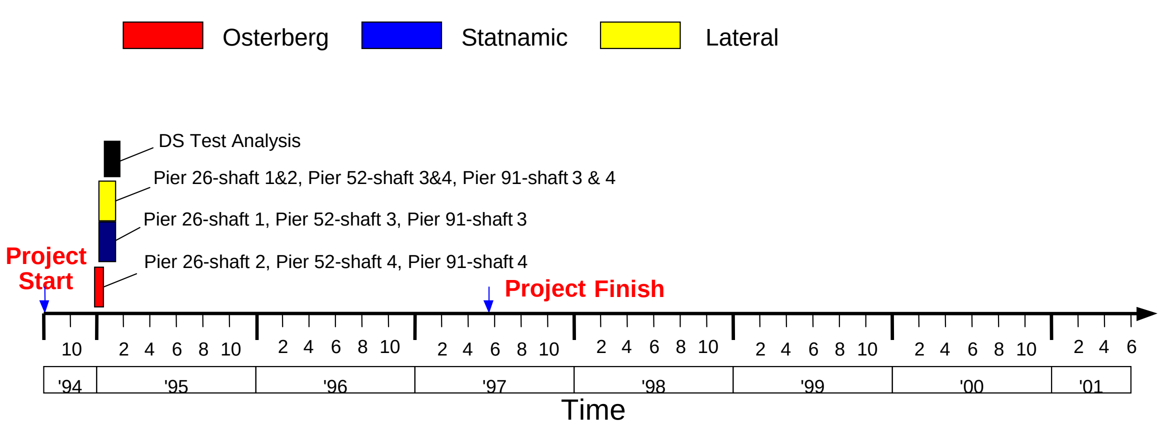


Figure 10.5 The Simplified Schedule of Gandy

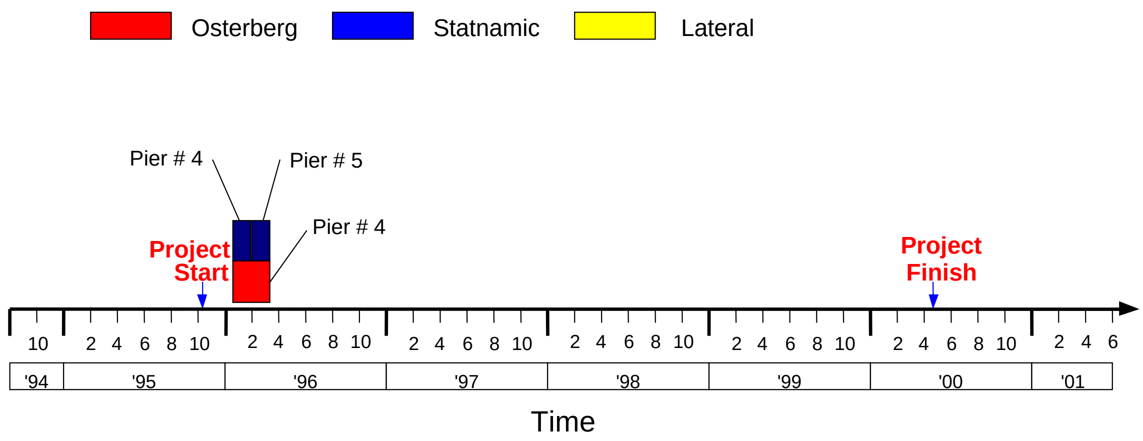


Figure 10.6 The Simplified Schedule of Hillsborough

The third step of the project schedule analysis was to calculate the net durations of drilled shaft testing activities. Since each project used different activity descriptions for the drilled shaft testing as mentioned above, it was not easy to determine what activity durations should be involved in the net durations of load testing. Thus, the schematic schedules developed at the second step were used to calculate them. The calculation of net durations was begun to investigate the early start and the early finish of testing activities. The early starts and early finishes of testing activities were combined together, and then the overlapped times were eliminated. Finally, the adjusted net durations were obtained by considering working days from the times what we calculated above.

Table 10.1 presents the shaft testing contribution to total project time for the reviewed projects. The estimated average time contribution from the drilled shaft testing was 80 days, which represents approximately 8.8 percent of the total project construction time.

Table 10.1 Time Contribution from Drilled Shaft Testing on Reviewed Projects

Projects	Total Project Duration (days)	Time Utilized for Drilled Shaft Testing (days)	Time Utilized for Drilled Shaft Testing as Percentage of Total Project Time
17th Street	914	145	15.9%
Apalachicola	939	55	5.9%
Fuller Warren	1517	85	5.6%
Gandy	458	52	11.4%
Hillsborough	765	65	8.5%
Mean	919	80	8.8%

10.3 THE EFFECT OF TESTING TIME ON PROJECT COST

Clearly, field-testing of drilled shafts can add time to the project duration. The installation of drilled shafts is a critical work activity. Testing adds time to the shaft installation process. Each project day represents a project overhead cost to the contractor and to the FDOT.

It is logical to assume that the contractor may have included an allocation of overhead cost in the bid unit prices for drilled shaft testing. Therefore the bid cost for drilled shaft testing is assumed to include the contractor's overhead cost. However, the overhead cost for the FDOT remains to be determined. FDOT project daily overhead cost includes its own internal cost for administering the project and the Construction Engineering and Inspection (CEI) cost for the project consultant. The CEI cost is the major FDOT project overhead cost item.

Table 10.2 presents an analysis of the additional CEI cost resulting from the drilled shaft testing on the case study projects. The average CEI cost was $5,848. per day. Obviously, there are other indirect costs associated with drilled shaft testing which are not included in this calculation. FDOT internal administrative costs have not been included. However, these are believed to be much less than the CEI consultant cost on projects where the contact administration has been out sourced.

Table 10.2 Additional Project CEI Cost Resulting from Drilled Shaft Testing on the FDOT Case Study Projects

Projects	Contract No.	Additional Time for Testing (Day)	CEI Daily Cost ($)	Additional CEI Cost ($)
Gandy	18734	52	$4,099	$213,148
17th St. Causeway	19871	145	$9,384	$1,360,680
Apalachicola	19197	55	$3,683	$202,565
Fuller Warren Bridge	19426	85	$9,211	$782,935
Hillsborough	19125	65	$2,863	$186,095
Mean		80.4	$5,848	$549,085

A review of the actual work process developed from discussions with project personnel suggest the typical work sequences for one test and two tests that are presented in Figure 10.7 and Figure 10.8. These work sequences are believed to be representative and indicate critical path times of 16 and 23 days respectively. However, individual contractors and project specifics may result in different approaches.

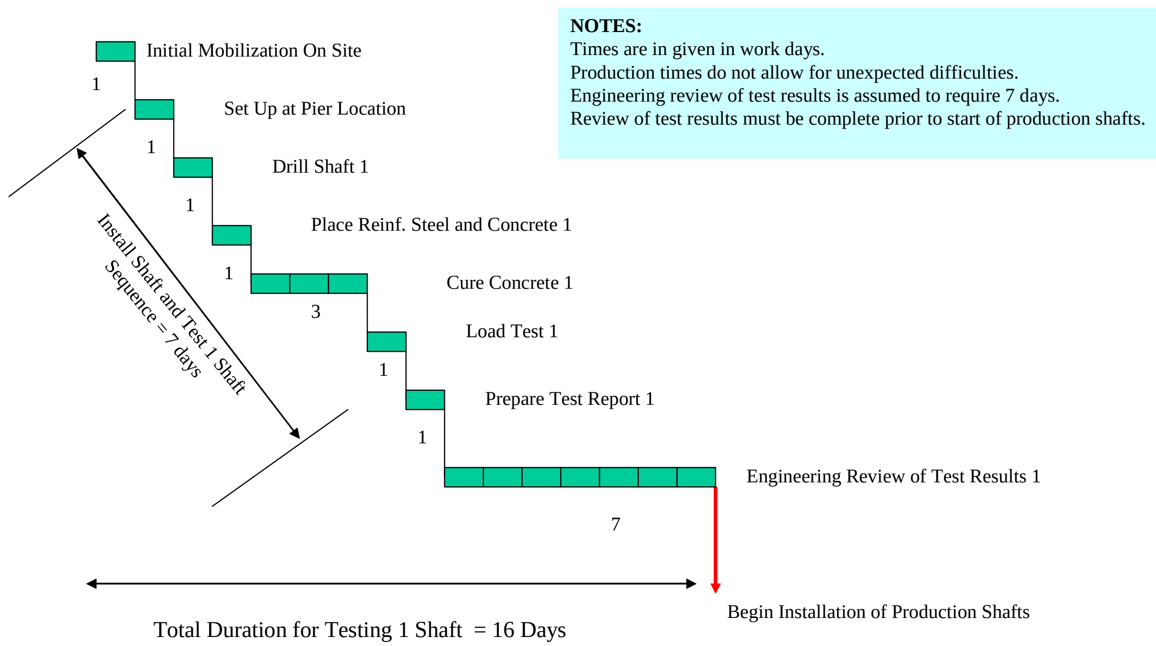


Figure 10.7 Typical Work Sequence for Drilled Shaft Testing with One Test

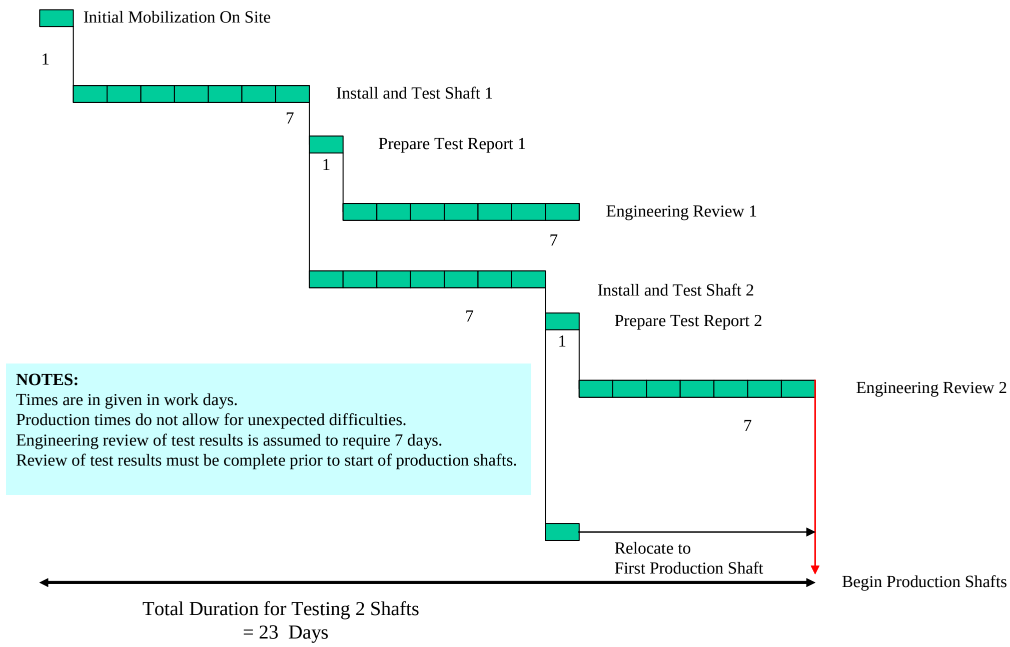


Figure 10.8 Typical Work Sequence for Drilled Shaft Testing with Two Tests

10.4 DISCUSSION OF RESULTS

An average of 6 shaft load tests were performed on each of the case study projects. The average estimated additional time required by the shaft testing for the case study projects was 80 days per project as indicated by a review of the projects' as-built CPM schedules. The precision of this estimate is subject to several factors. The primary issue is how accurately the CPM schedule reflects the actual work process. Even though the schedules are reviewed and approved by the FDOT, the best schedules only approximate the work process. Secondly, there is the possibility that if the testing time was removed, the contractor may use the time for other activities resulting in no reduction in total project duration. However, a review of the case study projects clearly indicates that drilled shaft load testing is likely to be on the project critical path and should be considered a key factor in the time required to complete the foundation work.

CHAPTER 11

DRILLED SHAFT UNIT END BEARING IN FLORIDA LIMESTONE WITH SUGGESTED LRFD RESISTANCE FACTORS

11.1 MEASURED UNIT END BEARING

A drilled shaft develops its load capacity from side shear (i.e., skin friction) and end bearing between the concrete and soil/rock materials. End bearing contributes to the total load capacity of a shaft as a function of tip displacement. The amount of top displacement required to develop significant tip displacements is generally large compared to side shear. For instance, amount of top movement, required to mobilize the ultimate side shear transfer is approximately ¼" to ½" (see Figure 4.13). However, the end bearing reaches its ultimate load capacity with tip movements on the order of 5 percent of the shaft diameter (i.e., FHWA Failure Criterion). Since the shaft butt movements for failure capacities may exceed tolerable (service) limits of the structure, a nonlinear tip resistance vs. movement is needed.

Also of concern is the spatial variability of Florida Limestone properties (qu, qu and Young's Modulus, E) and its impact on measured/predicted end bearing. Because of the latter, many designers either neglect end bearing or use a small nominal value. However, evident from the Osterberg tip results (Chapter 4), significant end bearing (> 145 tsf) has been generated on drilled shafts founded in Florida Limestone and should be considered in design.

Shown in Table 11.1 are the measured Osterberg unit end bearings as well the FDOT failure tip resistance. Note, failure is usually defined with a settlement criterion (i.e., FDOT defines failure as settlement equal to the elastic compression plus 1/30 of the shaft diameter), which may occur prior to plunging (very large settlement). For those shafts, which did not reach FDOT failure, mobilized bearing, is reported (i.e., less than FDOT), and for tip resistance above FDOT failure, Maximum Failure stress is given.

Table 11.1 Measured Unit End Bearing From Osterberg Load Tests in Florida

Bridge	Shaft Name	Shaft Length (ft)	*Unknown Friction (ft)	Tip Movement (in)	Failure Status	Mobilized Bearing (tsf)	FDOT Failure (tsf)	Maximum Failure (tsf)
17th Street Bridge	LTSO1	119.4	5.2	0.624	Both	x	x	x
	LTSO2	142.0	9.1	1.95	Tip Failure	x	x	x
	LTSO3	100.1	11.1	1.89	Both	41.5	x	x
	LTSO4	77.5	2.6	3.53	Tip Failure	x	x	66.4
Acosta Bridge	Test 1	64.2	0	4.41	Tip Failure	x	61.7	90.3
	Test 2	101.2	0	2.97	Tip Failure	x	28	39
	Test 4	113.9	0	3.2	Tip Failure	x	22.4	30.2
	Test 5A	87.8	0	5.577	Tip Failure	x	18.5	29.4
Apalachicola Bridge	46-11A	85.0	0	5.977	Both	x	62	92
	53-2	72.0	0	2.1	Both	62.5	x	x
	57-10	84.0	0	1.7	Both	56**	x	x
	59-8	134.0	9	1.3	Both	57	x	x
	62.5	89.2	0	2.69	Both	x	33.2	40
	69-7	99.1	0	4.46	Both	x	30.5	44
Fuller Warren Bridge	LT-1	41.0	0	0.23	Skin Failure	87	x	x
	LT-2	27.9	0	2.56	Both	x	80.8	89.5
	LT-3a	120.7	0	2.94	Both	x	34	34
	LT-4	66.8	0	3.12	Both	x	54	70
Gandy Bridge	26-2	39.4	9.8	0.4	Skin Failure	x	x	x
	52-4	54.5	5.33	2.9	Both	x	139.2	x
	91-4	74.7	6.7	2.5	Both	X	42.9	x
Victory Bridge	3-1	33.2	0	0.5	Both	109	x	x
	3-2	38.6	9.66	0.4	Skin Failure	X	x	x
	10-2	46.6	7.7	2.367	Both	X	145	x
	19-1	45.0	0	0.528	Both	124.4	x	x
	19-2	50.7	12.14	0.4	Skin Failure	X	x	x
Hillsborough Bridge	4-14	70.8	7.33	1.74	Both	X	x	x

Note:

1) *Unknown Friction: Distance from tip to the lowest strain gage. (side skin friction is unknown)

2) **: Ultimate unit end bearing due to plunging

3) x: Not determined

4) Shaft Length: Distance from the top to the tip of the shaft

5) Failure Status: Both means that the shaft fails in both side and end resistance

6) Mobilized: The mobilized unit end bearing when the bottom movement is 1/30 of the shaft diameter

7) FDOT Failure: The unit end bearing when the bottom movement is 1/30 of the shaft diameter

8) Maximum Failure: The unit end bearing when bottom movement is larger than 1/30 of the shaft diameter.

A number of methods have been suggested for shafts' ultimate or failure tip capacity (i.e., Kulhawy, Tomlinson, etc.), however very few identify tip resistance vs. tip displacement. One method, which considers both, is the FHWA (O'Neill) approach employed in FB-Pier. Developed for intermediate geomaterials (soft rock), the approach is dependent on the rocks' compressibility (i.e., Young's Modulus, E) and strength (qu) characteristics. A discussion of the method, prediction of eighteen Osterberg test, as well development of LRFD resistance factors, ϕ, and ASD factor of safeties, Fs, follows.

11.2 O'NEILL (FB-PIER) TIP RESISTANCE MODEL

Drilled shaft foundations are particularly attractive for use in "intermediate geomaterials," since boreholes in such geomaterials are relatively stable, are not difficult to excavate, and provide excellent skin and tip resistance. In 1995, O'Neill published an approach to predict tip resistance vs. displacement of drilled shafts in intermediate geomaterials based on an extensive finite element studies. The latter investigation reported that the tip response was a function of the following rock properties:

- q_u, Unconfined compressive strength of the rock;
- q_t, Split tensile strength of the rock;
- E_i, Young's modulus of the intact rock cores;
- E_m, Young's modulus of the rock mass;
- Initial interface pressure between concrete and limestone;
- Depth and diameter of the socket;
- RQD suggested relationship between E_m and E_i.

According to FHWA (O'Neill), the total shaft resistance (force), Q_t, is found from the smaller of:

$$Q_t = \pi D L_2 f_{s,soil} + \pi D L_1 \Theta_f f_{s,rock} + \frac{\pi D^2}{4} q_b , \ \Theta_f \le n \qquad \text{Equation 11.1}$$

$$Q_t = \pi \ D \ L_2 \ f_{s,soil} + \pi DLK_f f_{s,rock} + \frac{\pi \ D^2}{4} q_b \ , \ \Theta_f \ > \ n \qquad \text{Equation 11.2}$$

where $f_{s,soil}$ = unit skin friction of soil

$f_{s,rock}$ = unit skin friction of the rock

Θ_f(Theta) = Elastic compressibility parameter

K_f = Elastic compressibility parameter

q_b = Unit end bearing function of shaft displacement, W_t

The shaft's tip resistance, q_b is calculated from the shaft's top displacement, W_t as:

$$q_b = \Lambda W_t^{0.67} \qquad \text{Equation 11.3}$$

where Λ(Lambda) = Elastic compressibility parameter

W_t = Displacement at top of shaft (value assumed)

The settlement of at the bottom of the shaft, W_b, may computed from the elastic shortening of the shaft as follows:

$$W_b = W_t - \left[\frac{2(Q_t + Q_b)L}{\pi E_c D^2} \right] \qquad \text{Equation 11.4}$$

where E_c = Young's Modulus of the concrete shaft

Q_t = Force at Top of Shaft

Q_b = Force at Bottom of Shaft (q_b A_{shaft}) (Fig. 11.1)

In the calculation of shaft resistance, Q_t , elastic compressibility parameters, Θ_f (Theta), Λ(Lambda), etc. must first be determined. The latter are function of the Young's Modulus of the rock mass, E_m , and the horizontal stress, σ_n , acting between the shaft and the rock when the concrete is fluid. If no other information is available, general guidance on the selection of σ_n can be obtained from Figure 11.2, which is based on measurements of Bernal and Reese.

For Florida conditions, i.e., rough rock socket,

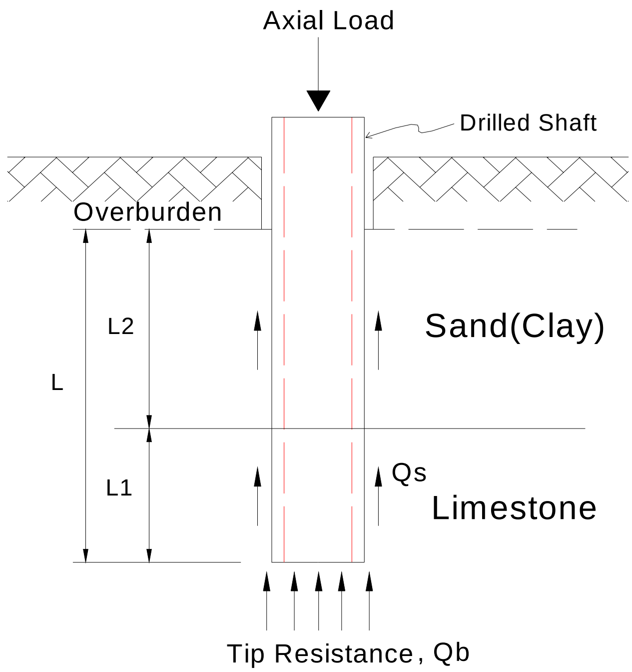


Figure 11.1 Schematic of a Typical Drilled Shaft Foundation

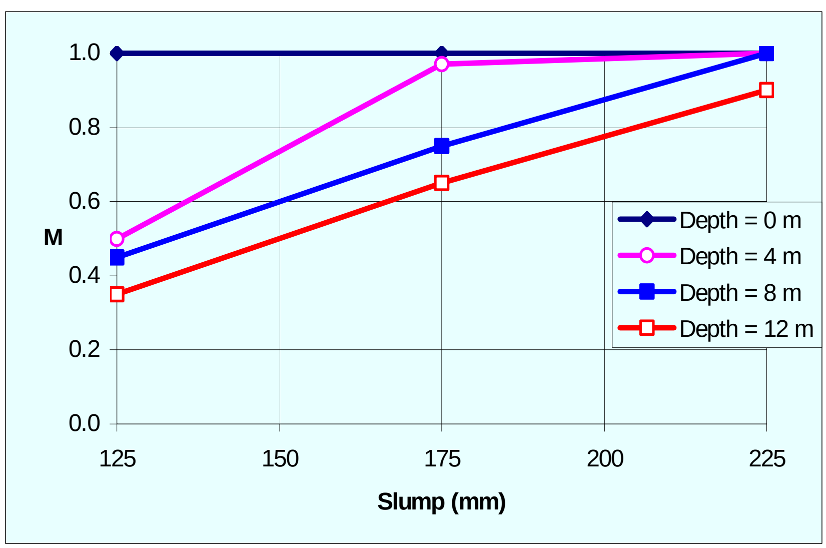


Figure 11.2 Coefficient of Lateral Resistance, M vs. Concrete Slump

$$\sigma_n = M.\gamma_c.Z_c \qquad \text{Equation 11.5}$$

where γ_c = unit weight of the concrete

Z_c = distance from top of shaft to middle of rock socket

M = Coefficient of lateral resistance, from Figure 11.2

In O'Neill tip resistance vs. tip displacement approach, the Young's Modulus of the rock mass, E_m , is of significant importance. The latter is different from the Young's modulus of intact rock samples, E_i , measured in laboratory (ASTM D3148) for limestone samples. The rock mass Young's Modulus, E_m , represents the whole mass including fissures, voids, slip planes, etc. O'Neill suggests a correlation (Table 11.2) between the E_i , and E_m based on Rock Quality Description (RQD). If RQD values are less than 20 percent, the 20 percent's RQD correlation was used (Load Transfer for Drilled Shaft in Intermediate Geomaterials, 1996).

Table 11.2 Estimation of Em/Ei Based on RQD
(Load Transfer for Drilled Shafts in Intermediate Geomaterials, 1996)

RQD (%)	E_m/E_i (closed joint)
100	1.00
70	0.70
50	0.15
20	0.05

Note: Values fo E_m/E_i for RQD values between those shown can be estimated by linear interpolation on RQD.

Once the Young's Modulus of the rock mass, E_m, is calculated, the elastic compressibility parameters, Λ(Lambda), Γ(Gamma), and Ω(Omega), may be computed as:

$$\Gamma = 0.37\left(\frac{L}{D}\right)^{0.5} - 0.15\left[\left(\frac{L}{D}\right)^{0.5} - 1\right]\log_{10}\left(\frac{E_c}{E_m}\right) + 0.13 \qquad \text{Equation 11.6}$$

$$\Omega = 1.14\left(\frac{L}{D}\right)^{0.5} - 0.05\left[\left(\frac{L}{D}\right)^{0.5} - 1\right]\log_{10}\left(\frac{E_c}{E_m}\right) - 0.44 \qquad \text{Equation 11.7}$$

$$\Lambda = 0.0134E_m \frac{\left(\frac{L}{D}\right)}{\left(\frac{L}{D}+1\right)} \left\{ \frac{200\left[\left(\frac{L}{D}\right)^{0.5} - \Omega\right]\left[\frac{L}{D}+1\right]}{\pi L \Gamma} \right\}^{0.67} \qquad \text{Equation 11.8}$$

Next, the parameter, n, is found:

$$n = \frac{\sigma_n}{q_u} \qquad \text{Equation 11.9}$$

Then the lateral coefficient parameters Θ_f(Theta), K_f may be found from:

$$\Theta_f = \frac{E_m \Omega}{\pi L \Gamma f_s} W_t \qquad \text{Equation 11.10}$$

$$K_f = n + \frac{(\Theta_f - n)(1-n)}{\Theta_f - 2n + 1} \leq 1 \qquad \text{Equation 11.11}$$

Based on the lateral coefficient parameters (Eqs. 11.9 –11.11), the total shaft resistance, Q_t, may be computed from the tip resistance, q_b (Eqs. 11.3 and 11.8), and the assumed shaft top displacement, W_t. The unit skin friction in the rock is based on the FDOT equation (Eq. 11.12) discussed in Chapter 8.

$$f_s = 0.5\sqrt{q_u}\sqrt{q_t} \quad \% \quad \text{Recovery / RQD} \qquad \text{Equation 11.12}$$

The tip resistance (Eq. 11.3) vs. tip displacement (Eq. 11.4) will be computed for all the Osterberg tests reported in Table 11.1. Two different approaches will be employed: 1) Nearest boring (within 100 ft); and 2) Random Selection based on all the site data.

11.3 PREDICTION OF THE UNIT END BEARING USING THE NEAREST BORING

11.3.1 17th Street Causeway

Based on both the Geotechnical report and boring logs, the elevation of the top of the limestone formation varied considerably within the site (Chapter 4). Also, the strength of the limestone varied significantly on each side of the channel. The soft limestone areas include LT1 shaft (ST. 35+46) and LTSO3 shaft (ST. 34+82), and the hard limestone area includes LT2 and LTSO4.

The nearest borings for the Osterberg Tests LTSO 3 (ST. 34+82) and LTSO 4 (38+04) were BB4 (ST. 34+81) and BB6 (ST.38+07). The qu values from boring BB9, at elevations -118 feet to -130 feet were also considered for tip strength of LTSO 3. The latter was used because no qu and qt values were available in BB4 (end at elevation –95 ft) in for the deeper depths which underlain by hard limestone where the tip of LTSO 3 resided.

For boring BB6 near shaft test LTSO 4, limestone was soft and sparse from elevations -43 feet to -105 feet, but from elevation -105 feet to -190 feet the limestone had high strength.

RQD (%) for boring BB9 used in predicting tip resistance of LTSO3 was 43.0 %. In the case of LTSO4, nearest boring BB6 had an average of RQD value of 41.9 %. The average qu and qt values for BB4 were 30.7 tsf and 9.6 tsf, whereas the average qu and qt values for BB6 were 138.6 tsf and 14.5 tsf, respectively

In order to determine the Young Modulus values, E_i, for LTSO3 (ST 34+82) an additional boring was placed near boring BB 4 by District 2 personnel. The new boring is located 10 feet on the north side of boring BB 4. Limestone was found at a depth of –90 feet to –121 feet. A total of 16 rock core samples were tested for Young Modulus, $E_{i,}$ for were tested at the State Materials Office. The latter was used to establish an E_i vs. qu relationship (chapter 7) for all the rock. From the E_i vs. qu correlation, the Young's modulus value, E_i for borings BB6 and BB9 were established as 466,377 psi and 626,529 psi. Next, the mass Young's modulus, Em, of the rock mass were estimated based on the RQD (FHWA-RD-95-172, 1996) values in Table 11.2. Using RQD value of 43 (%) and 41.9 (%), adjustment (i.e., factors) of 0.127 and 0.123 were found.

Based on the shaft dimensions, Young's Mass Modulus, and rock strength, the predicted tip resistances vs. displacement were computed for shafts LTSO3 and LTSO4 with O'Neill's approach (11.2). Shown in Table 11.3 and 11.4 are predicted tip resistance vs. tip displacements. Predicted FDOT capacities of 50.0 tsf and 64 tsf are given in Table in 11.5 along with measured vs. predicted response in Figures 11.3 and 11.4.

Table 11.3 Unit End Bearing vs. Tip Settlement for Shaft LTSO 3

W_t(in)	W_b(in)	q_b(psi)
0.2	0.1657	164.9
0.4	0.3564	262.4
0.8	0.7487	417.4
1.2	1.1439	547.7
1.6	1.5400	664.1
2	1.9367	771.2
2.4	2.3336	871.5
2.8	2.7307	966.3

Table 11.4 Unit End Bearing vs. Tip Settlement for Shaft LTSO 4

W_t(in)	W_b(in)	q_b(psi)
0.2	0.1588	206.0
0.4	0.3347	327.8
0.8	0.7022	521.5
1.2	1.0807	684.3
1.6	1.4650	829.8
2	1.8529	963.6
2.4	2.2429	1088.8
2.8	2.6346	1207.3

Table 11.5 Prediction of Unit End Bearing from the Nearest Boring (17th Bridge)

		Mobilized Bearing	FDOT Failure	Maximum Failure
LTSO 3	Measured	41.5	x	x
	Predicted	x	50.0	x
LTSO 4	Measured	x	x	66.4
	Predicted	x	64.0	x

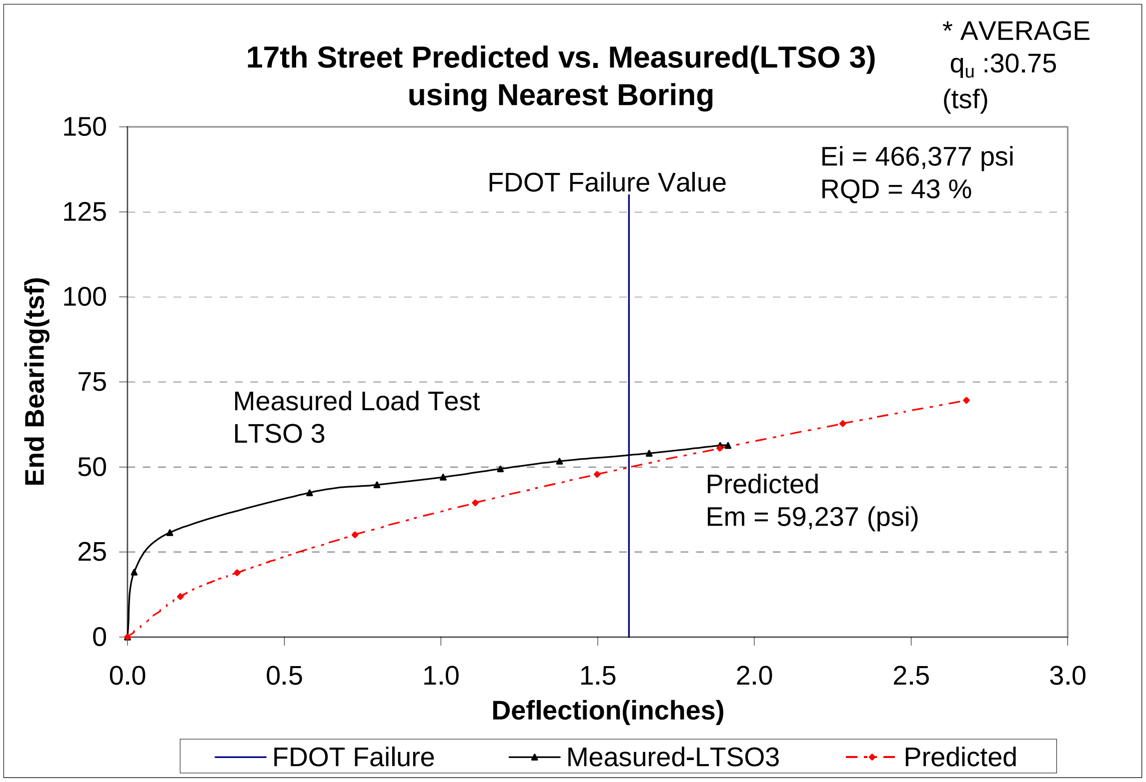


Figure 11.3 Predicted vs. Measured End Bearing (LTSO 3 in 17th Street Causeway)

Of interest is the shape of the end bearing vs. tip displacement curves, which are strongly impacted by the mass modulus, E_m, of the rock. Also note the tip resistance is still increasing at the FDOT failure criterion (i.e., elastic shortening plus settlements equal to 1/30 diameter of the shaft).

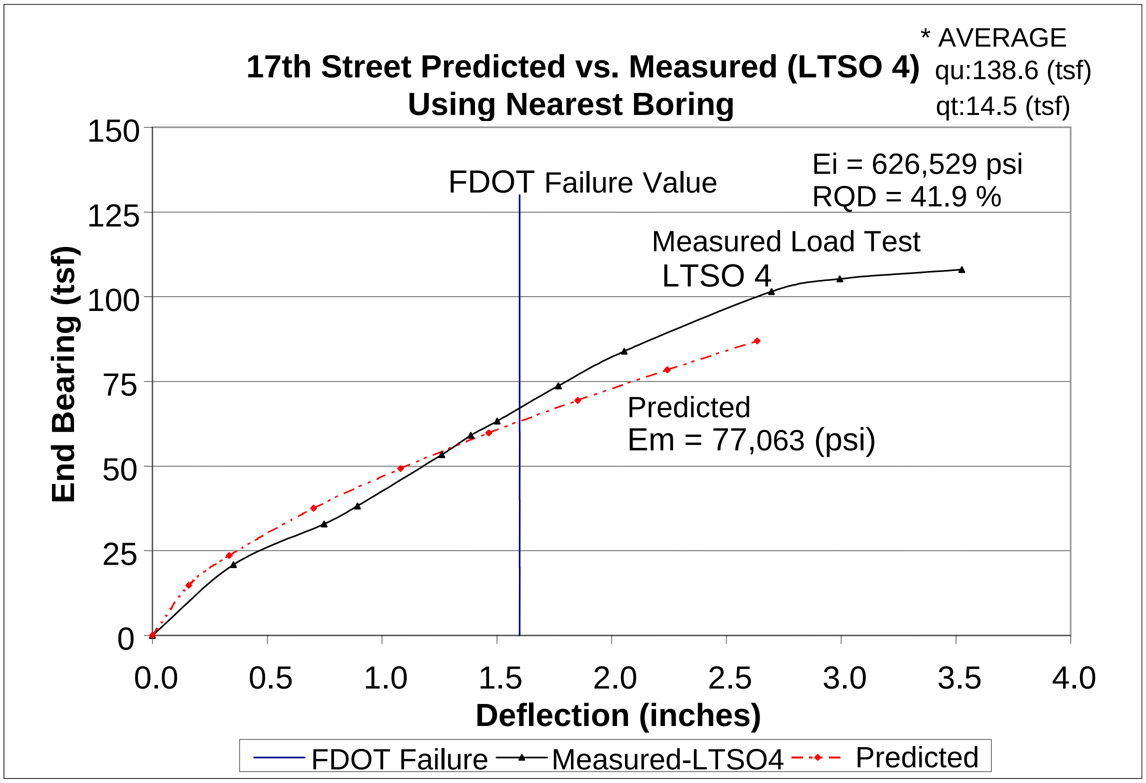


Figure 11.4 Predicted vs. Measured End Bearing (LTSO 4 in 17th Street Causeway)

11.3.2 Acosta Bridge

A total of 4 Osterberg tests (Test 1, Test 2, Test 4, Test 5A) were performed at this site with limestone found only from an elevation of –20 feet to –47 at the bottom of Test 1 shaft. The sites for Test 2, Test 4, and Test 5A were composed of medium sand, clay, and silt ranging from an elevation of –40 feet to –100 feet and were not considered (i.e., not rock). The nearest borings for Test 1 (ST. 134+39) shaft was boring WA-2 (ST. 134+40) which had limestone from an elevation -22 feet to -41 feet.

From the rock core samples, the average RQD was 34.2 % with a Recovery of 74 %. The average qu and qt for WA-2 were 85.3 tsf and 18.4 tsf, respectively.

To determine the Young Modulus, 11 rock core samples were recovered from two new borings near Test 1. The first boring was located 21 ft to the right of the boring SB 7 (St. 130 +

34) and the second boring was located 25 ft to the right of the boring NB 1 (St. 150 + 35). Soft limestone was found from a depth of –42 feet to –47 feet in the first boring and from a depth of –27 feet to –32 feet in the second boring. The 11 core samples gave an average Young Modulus of 480,911 psi with a standard deviation of 841,720 psi. Also, a correlation between qu and E_i was established (Chapter 7). Based on the nearest boring, and qu values, an E_i value of 579,042 psi was obtained. Using the RQD value of 34%, the mass Young's Modulus, E_m , (FHWA-RD-95-172, 1996) of 57,904 psi using Table 11.2.

Based on the rock's mass modulus and strengths (qu and qt), soil properties, a tip resistance vs. displacement response (Table 11.6) was determined from O'Neill's approach (section 11.2) for Test 1. Shown in Figure 11.5 is the both the predicted and measured tip resistance vs. tip displacement. Table 11.7 present both the predicted and measured FDOT failure tip resistance, as well as ultimate measured tip resistance. As with 17th Street, the tip resistance does not reach an ultimate value, but increases with tip displacements.

Table 11.6 Unit End Bearing vs. Tip Settlement for Test 1 at Acosta Bridge

W_t(in)	W_b(in)	q_b(psi)
0.2	0.1067	153.7
0.4	0.2377	244.6
0.8	0.5373	389.1
1.2	0.8665	510.6
1.6	1.2130	619.1
2	1.5707	719.0
2.4	1.9360	812.4
2.8	2.3068	900.8

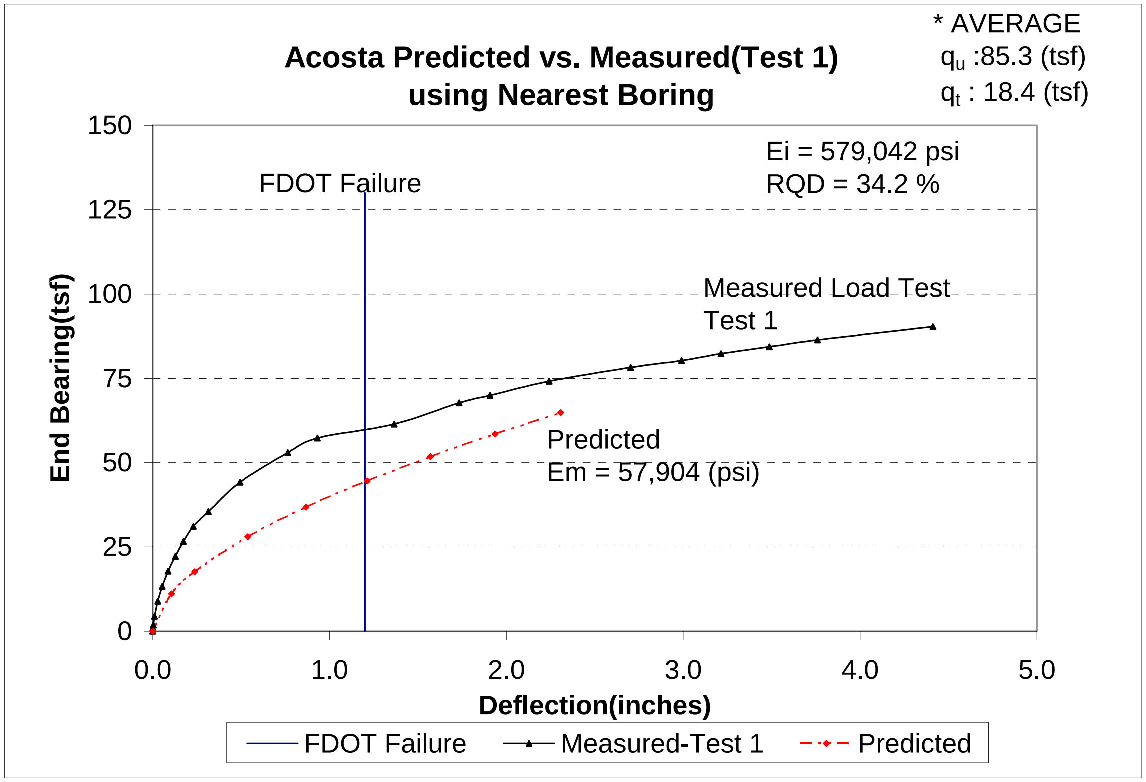


Figure 11.5 Predicted vs. Measured End Bearing (Test 1, Acosta Bridge)

Table 11.7 Prediction of Unit End Bearing from the Nearest Boring (Acosta)

		Mobilized Bearing	FDOT Failure	Maximum Failure
Test 1	Measured	x	61.7	90.3
	Predicted	x	40.0	X

11.3.3 Apalachicola Bridge, SR 20

A total of 6 Osterberg tests were performed at this site with shafts 46-11A, 62-5, and 69-7 reaching the FDOT failure states. The latter three shafts were subsequently modeled with O'Neill's method. The nearest borings for the Osterberg Tests of 46-11A (ST. 624+03) were TH-46A and TH-46B and for shaft 62-5 (ST. 645+97) were borings TH-62A and TH-62B, and for shaft 69-7 (ST. 653+41) it was borings TH-69A and TH-69B. The weathered Limestone included calcareous and cemented clay in boring TH-46A and TH-46B from elevation -13 feet to -42 feet. Boring TH-62A and TH-62B had 27 ft of limestone, starting at elevation -23 feet; boring TH-69A and TH-69B had limestone from elevation -23 feet to -35 feet.

The boring cores for TH-46A and TH-46B showed average RQD values of 64.0 % and a Recovery of 85 %. Borings TH-62A and TH-62B had average RQD values of 46.7 % with a Recovery of 80 %. Borings TH-69A and TH-69B had average RQD values of 47.0 % and a Recovery of 70 %. The average qu and qt values for borings TH-46A and TH-46B were 12.2 and 1.0 tsf, whereas, the average qu and qt values for borings TH-62A and TH-62B were 5.3 and 0.75 tsf. The average qu and qt values for TH-69A and TH-69B were 5.1 and 0.4 tsf.

From the developed correlation equation, i.e., qu (unconfined compressive strength,) with Young's modulus (23 rock core samples of this site, see Chap. 7, Fig. 7.11), initial Young's Modulus values, E_i, of 98,608 psi, 43,014 psi, and 41,391 psi were obtained for borings TH-46A and 46B; TH-62A and 62B; and TH-69A and 69B. However, since the back computed initial Young's modulus, E_i, from qu and qt values were on the very low end of the recorded E_i values, it was decided to use the % Recovery values instead of RQD (FHWA-RD-95-172, 1996) for estimating the mass modulus estimates, E_m. The adjustment values for the nearest borings (TH-46A and 46B, TH-62A and 62B, TH-69A and 69B) based on % Recovery values of 78.0 (%), 58.2 (%), and 54 (%) were 0.54, 0.37, and 0.26. E_m, rock mass values of 52,755 psi, 15915 psi, and 10,762 psi were subsequently computed for the limestone based on nearest borings.

Presented in Table 11.8, 11.9 and 11.10 are predicted tip resistance vs. tip displacements for shafts 46-11A, 62-5, and 69-7 based on O'Neill's method (11.2). Given in Table 11.11 are both the predicted and measured failure tip stresses based on the FDOT failure criterion. Note, the predicted and measured tip resistances were small, i.e., 45.5 tsf, 15.2 tsf, and 12.0, due to the low mass modulus of the rock, E_m. A graphical comparison the predicted and measured tip resistance vs. displacements is shown in Figures 11.6, 11.7, and 11.8.

Table 11.8 Unit End Bearing vs. Tip Settlement for Shaft 46-11A at Apalachicola Bridge

W_t(in)	W_b(in)	q_b(psi)
0.2	0.1800	130.0
0.4	0.3733	206.8
0.8	0.7642	329.1
1.2	1.1570	431.8
1.6	1.5507	523.6
2	1.9449	608.0
2.4	2.3396	687.0
2.8	2.7345	761.7
4	3.9206	967.4

Table 11.9 Unit End Bearing vs. Tip Settlement for Shaft 62-5 at Apalachicola Bridge

W_t(in)	W_b(in)	q_b(psi)
0.2	0.1935	41.3
0.4	0.3919	65.8
0.8	0.7897	104.6
1.2	1.1879	137.3
1.6	1.5863	166.5
2	1.9849	193.3
2.4	2.3835	218.5
2.8	2.7822	242.2
4	3.9786	307.6

Table 11.10 Unit End Bearing vs. Tip Settlement for Shaft 69-7 at Apalachicola Bridge

W_t(in)	W_b(in)	q_b(psi)
0.2	0.1905	33.9
0.4	0.3885	54.0
0.8	0.7861	85.9
1.2	1.1840	112.7
1.6	1.5822	136.6
2	1.9806	158.7
2.4	2.3790	179.3
2.8	2.7775	198.8
4	3.9735	252.5

Table 11.11 Predicted Unit End Bearing from Nearest Boring (Apalachicola)

		Mobilized Bearing	FDOT Failure	Maximum Failure
46-11A	Measured	x	62	92
	Predicted	x	45.5	X
62-5	Measured	x	33.2	40
	Predicted	x	15.2	X
69-7	Measured	x	30.5	40
	Predicted	x	12.0	X

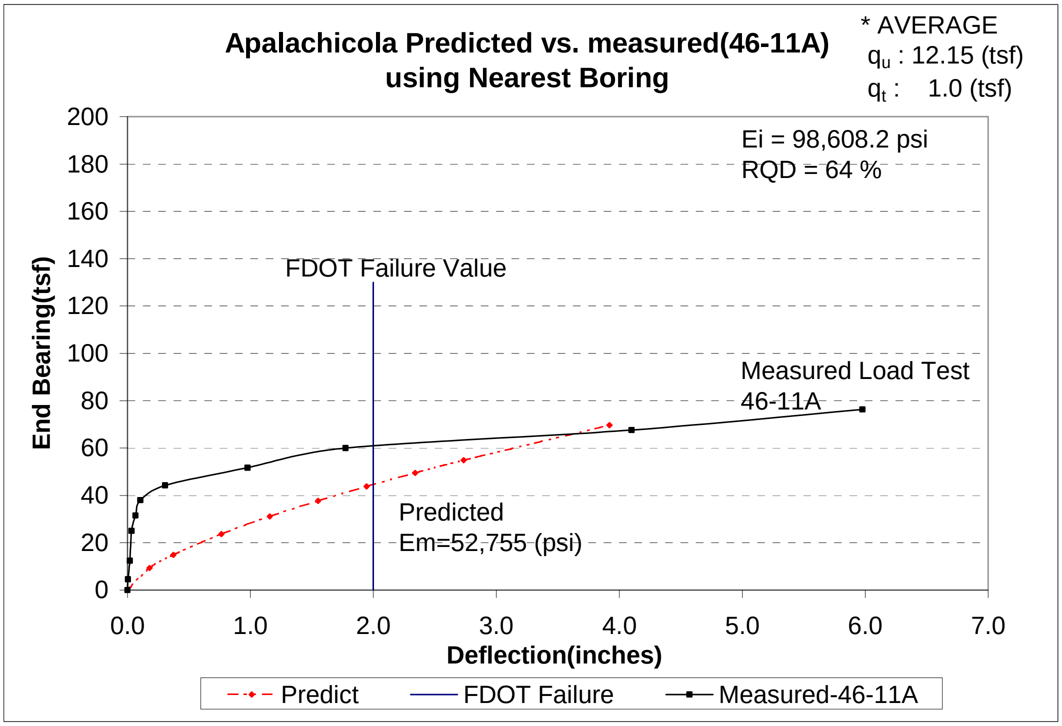


Figure 11.6 Predicted vs. Measured End Bearing (46-11A in Apalachicola Bridge)

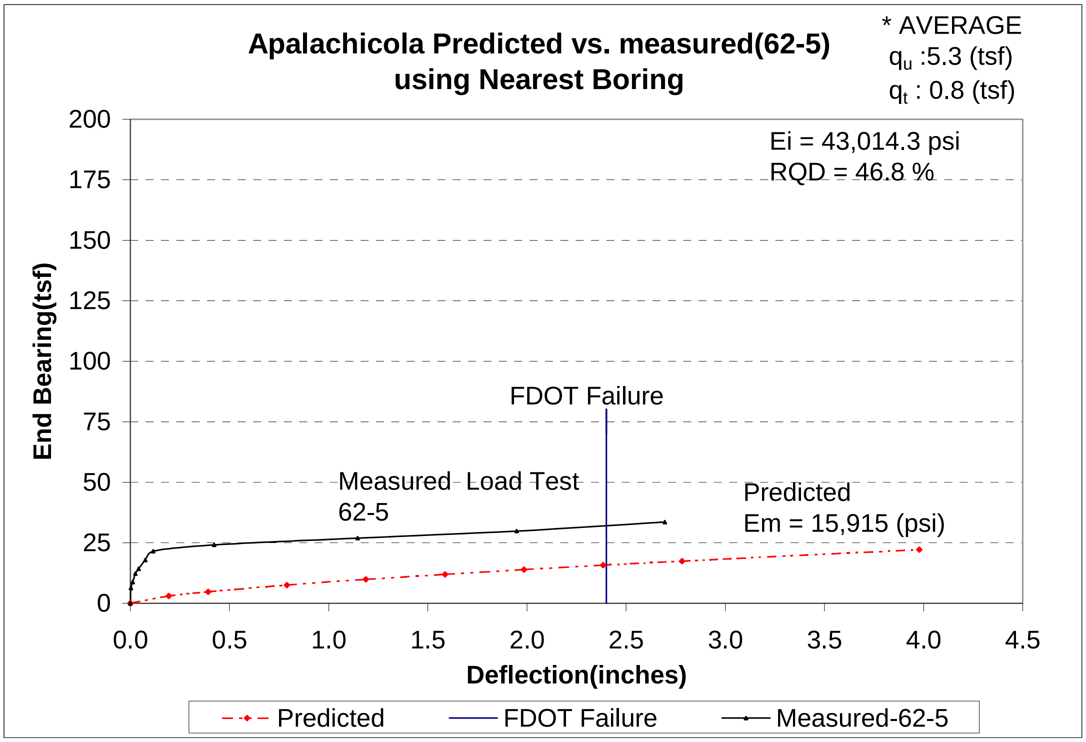


Figure 11.7 Predicted vs. Measured End Bearing (62-5 in Apalachicola Bridge)

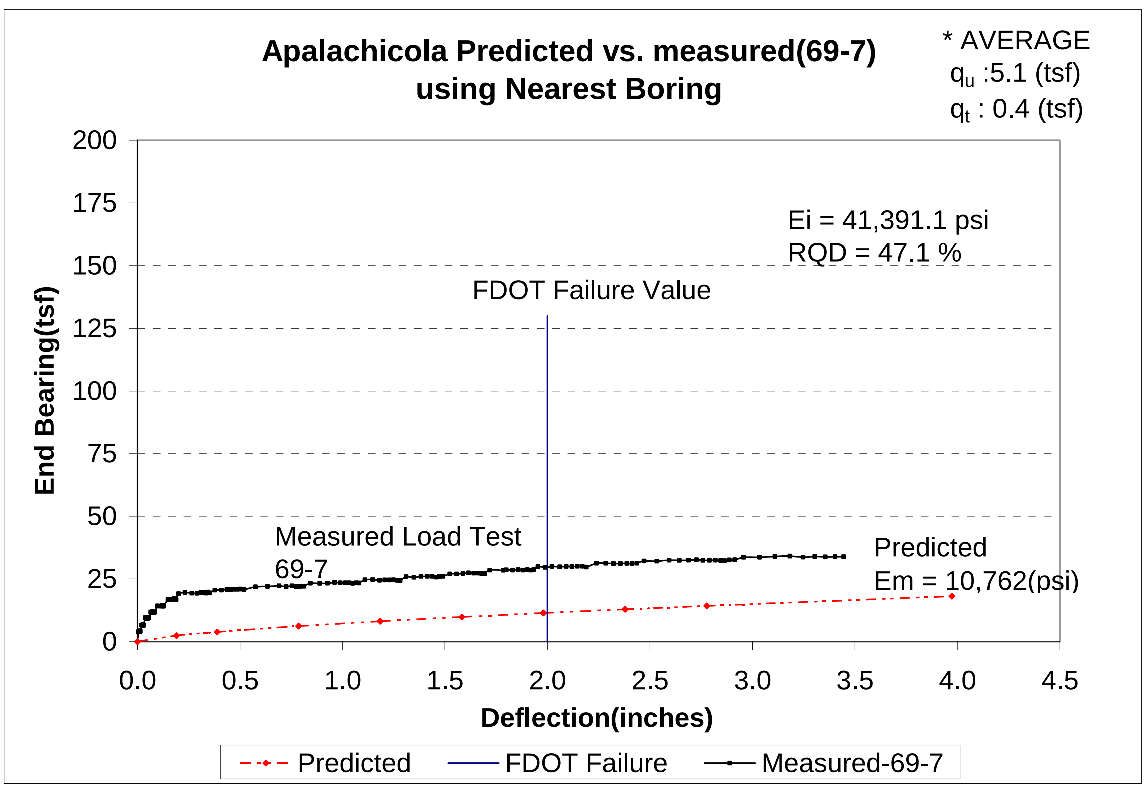


Figure 11.8 Predicted vs. Measured End Bearing (69-7 in Apalachicola Bridge)

11.3.4 Fuller Warren Bridge

A total of 3 Osterberg tests were performed at this site, which reached FDOT failure, LT-2, LT-3a, and LT-4 shafts. However shaft LT-2 was tipped in marl (calcareous sand & clay), and there was no strength data from existing borings within one hundred feet; consequently the test was discarded. The nearest boring for LT-3a (ST. 323+81) was boring BW-14 and for LT-4 (ST. 341+25) it was boring BL-23.

The rock cores for BW-14 and BL-23 had average RQD values of 31.0 % and 33.0 % with % Recoveries of 51 % and 49 %. The average strength, i.e., qu and qt, values for the boring BW-14 were 43.0 and 9.6 tsf and for BL-23 the average qu and qt values were 88.5 and 11.4 tsf.

To assess the Young's Modulus for the formation, District 2 Geotechnical personnel located two new borings at BL 1 (St. 284+04) and LT 1 (St. 285+30). Rock cores were recovered with limestone varying from a depth of –25 feet to –40 feet. From the latter cores, 33 samples were obtained and tested at the State Materials Office. Figure 7.15 shows the Initial Young's Modulus, E_i, vs. qu for all the rock cores. In the vicinity of the shaft tips, E_i values of 258,780 psi (LT-3a) and 456,618 psi (LT-4) were obtained. Based on boring BW-14 with RQD values 31.0 (%) an adjustment value of 0.093 (Table 11.2) was obtained to obtain E_m. For boring BL-23 an RQD value 33.0 (%) and an adjustment value of 0.096 was found. The mass Young's modulii, E_m, of the rock value based on the nearest borings were 23,290 psi and 43,835 psi.

Based on the strength data, and mass modulus of the rock, the tip resistance vs. tip displacement was predicted with O'Neill's methods for shafts LT-3a and Lt-4. Presented in Table 11.12 is the response of shaft LT-3a, and Table 11.13 presents the prediction for LT-4. Table 11.14 presents the measured and predicted FDOT failure resistance for each shaft. The predicted values (22.5 tsf and 37.0 tsf) compare favorably with the measured values and are slightly conservative. Figures 11.9 and 11.10 are plots of the measured and predicted tip resistance vs. tip displacements for shafts LT-3a and LT-4.

Table 11.12 Unit End Bearing vs. Tip Settlement for Shaft LT-3a, Fuller Warren Bridge

W_t(in)	W_b(in)	q_b(psi)
0.2	0.1844	57.3
0.4	0.3728	91.2
0.8	0.7549	145.0
1.2	1.1413	190.3
1.6	1.5305	230.8
2	1.9216	268.0
2.4	2.3140	302.8
2.8	2.7074	335.7

Table 11.13 Unit End Bearing vs. Tip Settlement for Shaft LT 4a, Fuller Warren Bridge

W_t(in)	W_b(in)	q_b(psi)
0.2	0.1411	112.5
0.4	0.2968	179.0
0.8	0.6308	284.8
1.2	0.9832	373.7
1.6	1.3466	453.2
2	1.7172	526.3
2.4	2.0928	594.6
2.8	2.4720	659.3

Table 11.14 Predicted vs. Measured Unit End Bearing at Fuller Warren

		Mobilized Bearing	FDOT Failure	Maximum Failure
LT-3a	Measured	x	34	34
	Predicted	x	22.5	x
LT-4	Measured	x	54	70
	Predicted	x	37	x

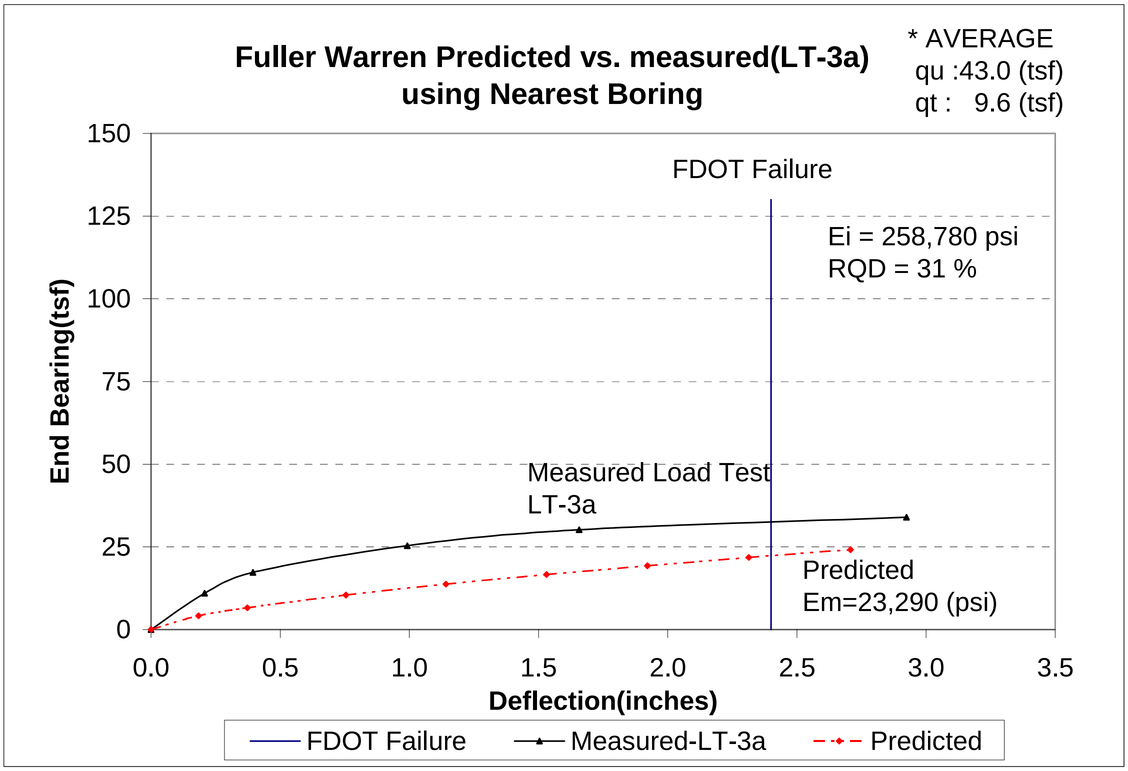


Figure 11.9 Predicted vs. Measured End Bearing (LT-3a, Fuller Warren Bridge)

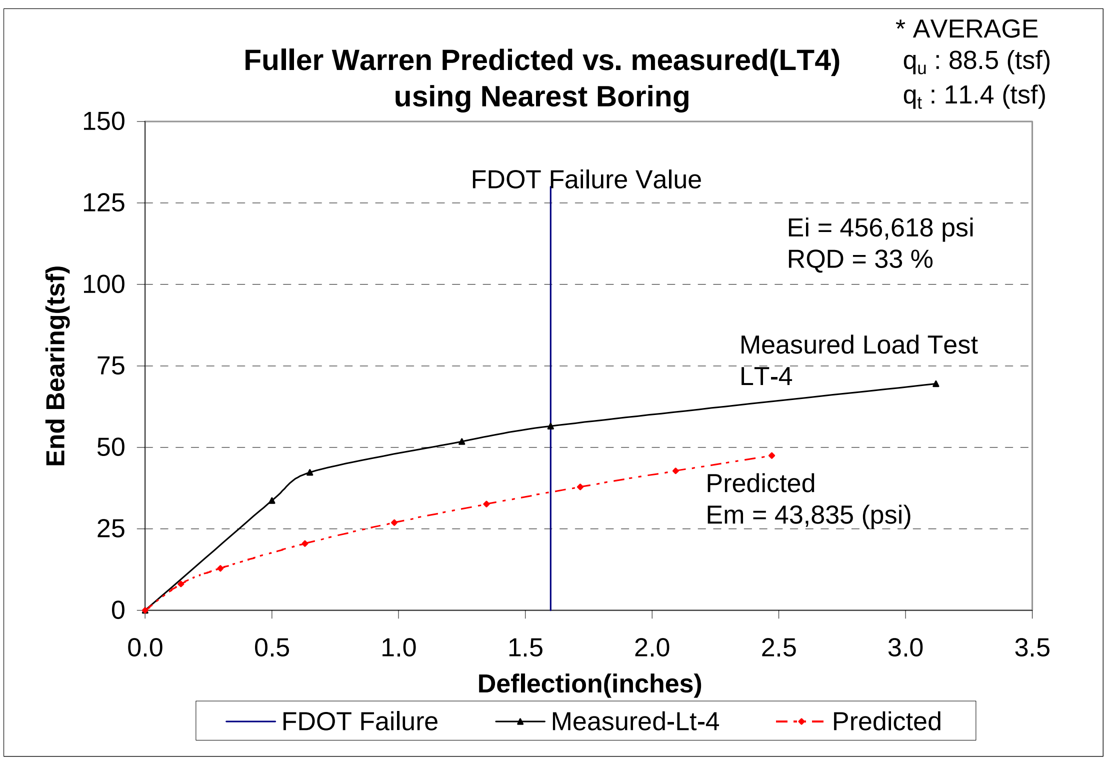


Figure 11.10 Predicted vs. Measured End Bearing (LT 4, Fuller Warren Bridge)

11.3.5 Gandy Bridge

Three Osterberg tests were performed at the Gandy site, with two reaching the FDOT failure state, i.e., tests 52-4 and 91-4. The nearest boring for test 52-4 (ST. 93+63) was boring SB-36 and for test 91-4 (ST. 174+21) was boring SB-91.

From the recovered cores at SB-36 and SB-91, the average RQD values were 78.0 % and 52.0 % and the % Recoveries were 88 % and 70 %. The average qu and qt values for the boring SB-36 were 36.6 and 13.9 tsf and for SB-91, the average qu and qt values were 4.3 and 1.1 tsf, respectively.

Twenty-eight Young's Modulus tests were available for this site and were correlated with the rock's unconfined compressive strength, qu (see Fig. 7.19). Based on nearest borings for tests 52-4 and 91-4, the intact Young's Modulus, E_i, of 322,857 psi and 274,136 psi was found. Using RQD value of 78%, an estimated adjustment of 0.78 (Table 11.2) was obtained to give a mass modulus of 251,828 psi for the rock. For boring SB-91 with and RQD of 52%, an estimated adjustment of 0.21 (Table 11.2) was found to give a mass modulus of the rock of 57,569 psi.

Based on the strength data, and mass modulus of the rock, the tip resistance vs. tip displacement was predicted with O'Neill's methods for shafts 52-4 and 91-4. Presented in Table 11.15 is the response of shaft 52-4, and Table 11.16 presents the prediction for 91-4. Table 11.17 presents the measured and predicted FDOT failure resistance for each shaft. The predicted values (154 tsf and 45 tsf) compare very favorably with the measured values. Figures 11.11 and 11.12 are plots of the measured and predicted tip resistance vs. tip displacements for shafts LT-3a and LT-4.

Table 11.15 Unit End Bearing vs. Tip Settlement for Shaft 52-4 at Gandy Bridge

W_t(in)	W_b(in)	q_b(psi)
0.2	0.069	469.94
0.4	0.215	747.72
0.8	0.555	1189.67
1.2	0.914	1561.02
1.6	1.280	1892.85
2	1.651	2198.10
2.4	2.025	2483.69
2.8	2.401	2753.93

Table 11.16 Unit End Bearing vs. Tip Settlement for Shaft 91-4 at Gandy Bridge

W_t(in)	W_b(in)	q_b(psi)
0.2	0.1738	147.9
0.4	0.3655	235.3
0.8	0.7535	374.3
1.2	1.1437	491.2
1.6	1.5351	595.6
2	1.9272	691.6
2.4	2.3198	781.5
2.8	2.7128	866.5

Table 11.17 Predicted vs. Measured Unit End Bearing at Gandy

		Mobilized Bearing	FDOT Failure	Maximum Failure
52-4	Measured	x	139.2	x
	Predicted	x	154	x
91-4	Measured	x	42.9	x
	Predicted	x	45	x

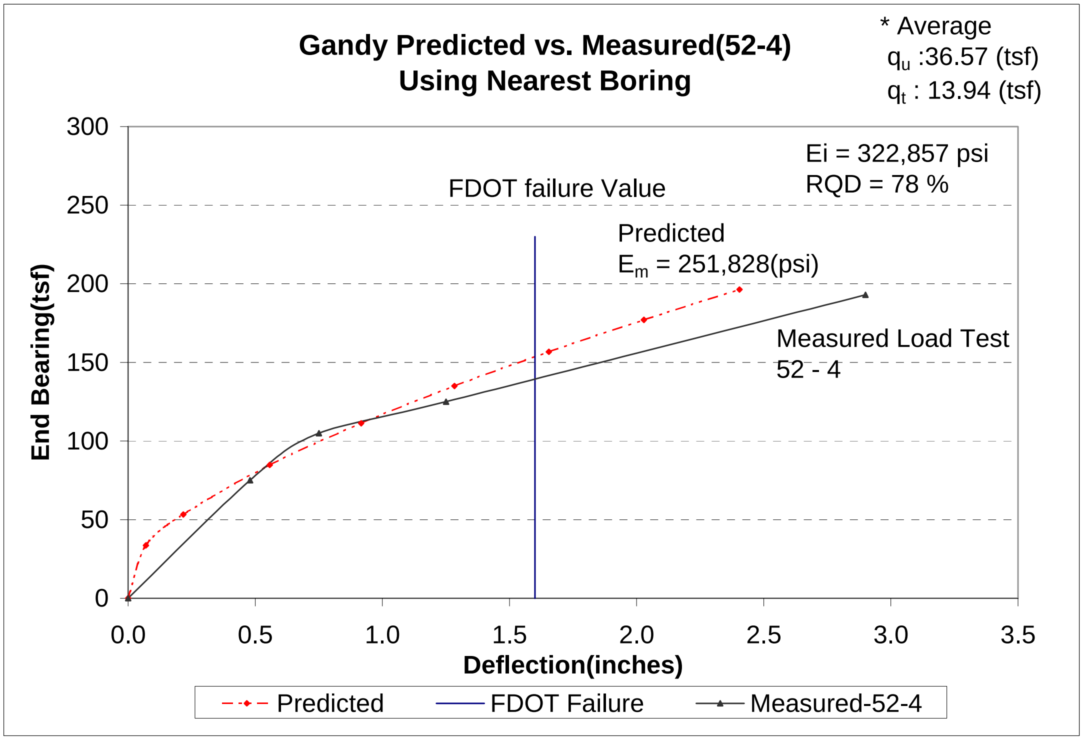


Figure 11.11 Predicted vs. Measured End Bearing (52-4 in Gandy Bridge)

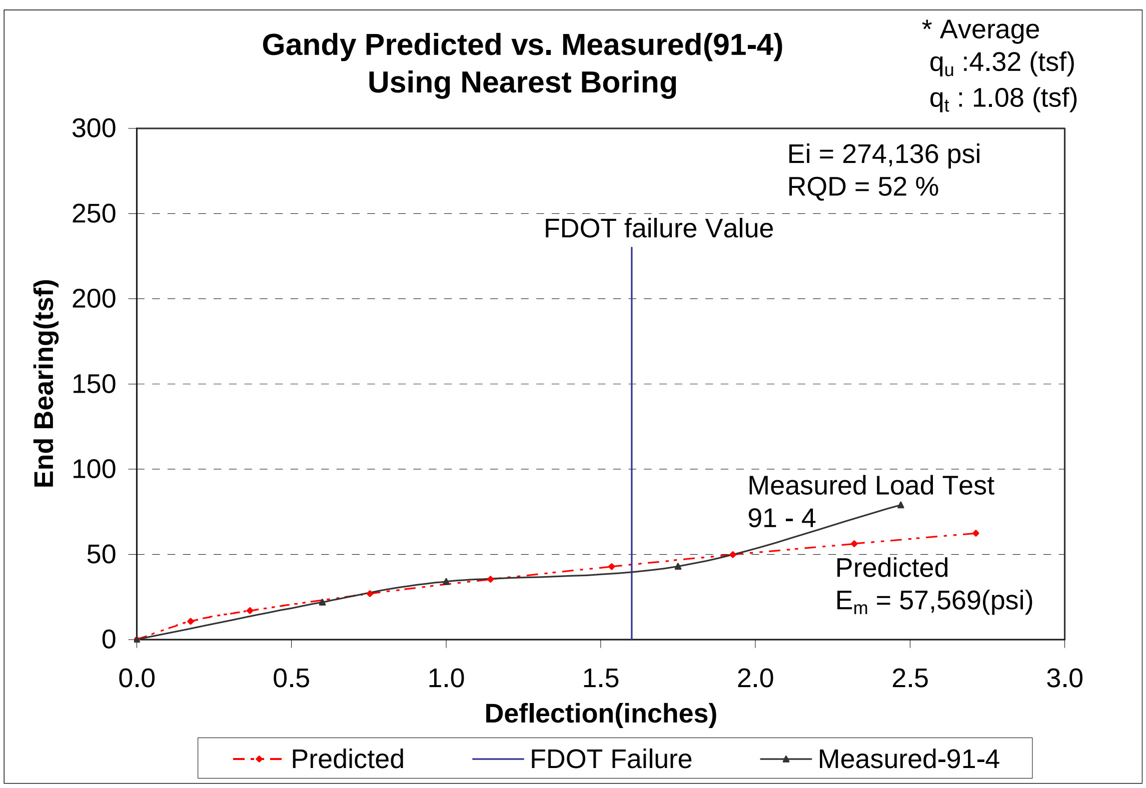


Figure 11.12 Predicted vs. Measured End Bearing (91-4 in Gandy Bridge)

11.3.6 Victory Bridge

A total of Five Osterberg tests (3-1, 3-2, 10-2, 10-2, 19-1, 19-2) were performed and the site with only shaft 10-2 reaching the FDOT failure criterion which will be predicted. The nearest boring for shaft 10-2 (ST. 99+31) shaft was boring TB-9 (ST. 99+70), which had limestone from elevations +37 feet to -15 feet. The rock cores in boring TB-9 had average RQD values of 29.8 % and % Recoveries of 100 %. The average strength (qu and qt) for the rock in boring TB-9 was 157 tsf and 25.1 tsf.

Twenty-four rock core samples at the site were tested to determine their Young's Modulus. From the latter a correlation between qu and E_i was established (Fig. 7.25). Using Figure 7.25 and the unconfined compressive strength, qu, from boring TB-9, the Young's Modulus, E_i for test shaft 10-2 was estimated to be 2,880,476 psi. The mass modulus of the rock, E_m was determined from the RQD value (29.8 %) and Table 11.2 (0.083) as 216,036 psi.

Based on the strength data, and mass modulus of the rock, the tip resistance vs. tip displacement was predicted with O'Neill's methods for shaft 10-2. Presented in Table 11.18 is the top and tip displacement along with tip resistance for shaft 10-2. Table 11.19 presents the measured and predicted FDOT failure resistance for shaft 10-2. The predicted value (159 tsf) compares very favorably with the measured value, 145 tsf. Figure 11.13 plots the measured and predicted tip resistance vs. tip displacements for shaft 10-2.

Table 11.18 Unit End Bearing vs. Tip Settlement for Shaft 10-2 at Victory Bridge

W_t(in)	W_b(in)	q_b(psi)
0.2	0.0942	475.6
0.4	0.2331	756.7
0.8	0.5573	1204.0
1.2	0.9082	1579.8
1.6	1.2712	1915.6
2	1.6410	2224.5
2.4	2.0150	2513.5
2.8	2.3919	2787.0

Table 11.19 Predicted vs. Measured Unit End Bearing at Victory

		Mobilized Bearing	FDOT Failure	Maximum Failure
10-2	Measured	x	145	x
	Predicted	x	159	x

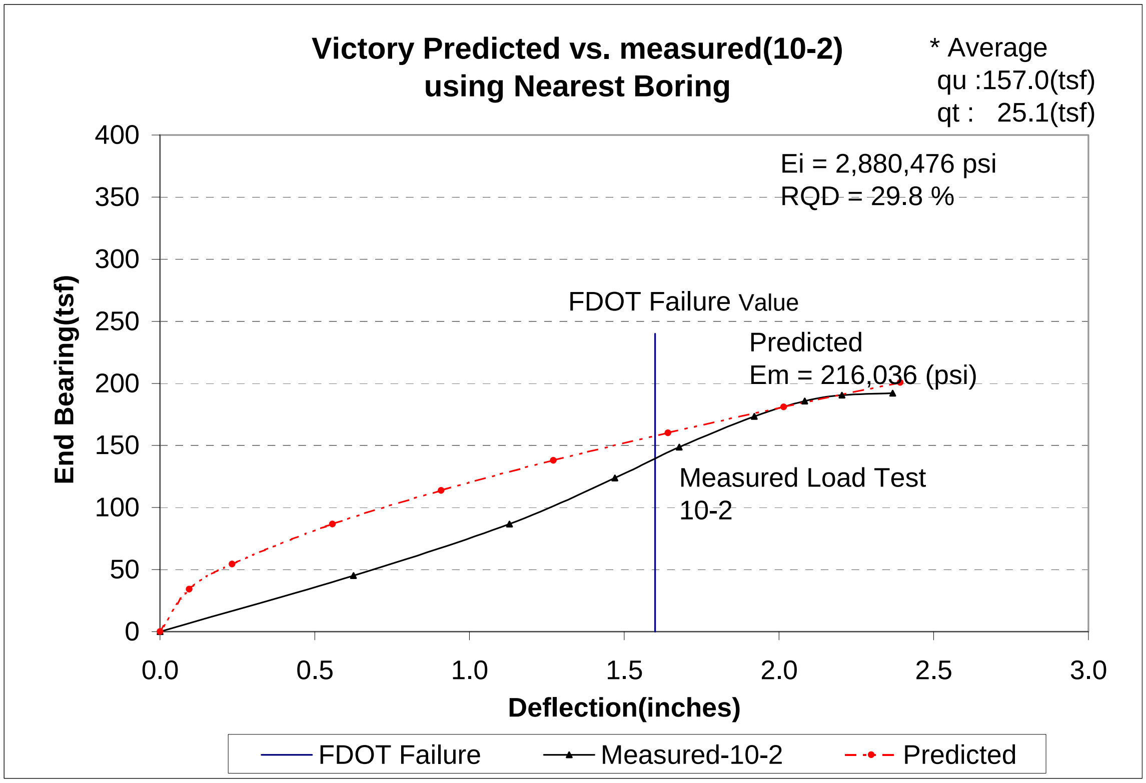


Figure 11.13 Predicted vs. Measured End Bearing (10-2 in Victory Bridge)

11.4 COMPARISON of MEASURED and PREDICTED END BEARING from NEAREST BORING

Based on the nearest boring data to the load tests, a total of 11 end bearing resistances at FDOT failure were predicted as shown as Table 11.20. The mean of predicted end-bearing values was 58.6 tsf and the mean of the measured values was 65.7 tsf. The standard deviation of measured end bearing values was 39.8 tsf and the standard deviation of predicted end bearing values was 50.8 tsf.

Table 11.20 Measured and Predicted Unit End Bearing Using the Nearest Boring

Bridge Name	Shaft Name	Unit End Bearing	
		Measured	Predicted
17th Street	LTSO 3	54	50
	LTSO 4	66.4	64
Apalachicola	46-11A	62	45.5
	62-5	33.2	15.2
	69-7	30.5	12
Fuller Warren	LT-3a	34	22.5
	LT4	54	37
Gandy	52-4	139.2	154
	91-4	42.9	45
Acosta	Test 1	61.7	40
Victory	10-2	145	159
Mean		65.7	58.6
Standard Deviation		39.8	50.8

Shown in Figure 11.14 is a graphical comparison of measured and predicted end bearing based for all shafts, which reached FDOT failure. The solid lines represent predicted end bearing and the dashed line is the measured end from the load tests at FDOT failure.

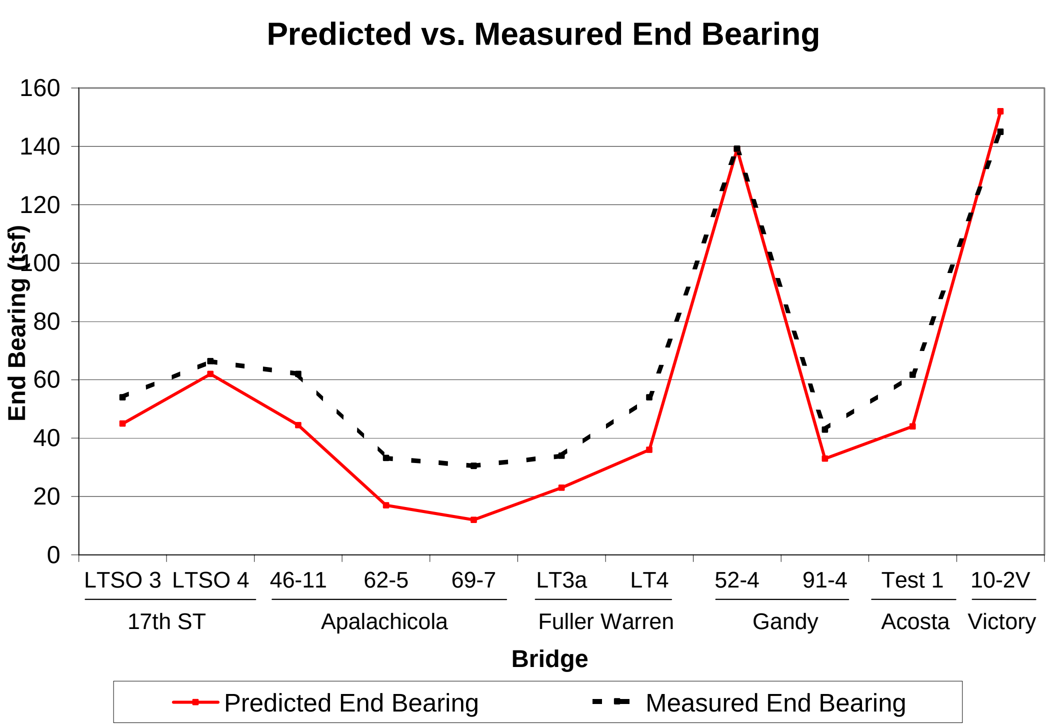


Figure 11.14 Comparison of Measured and Predicted End Bearing

11.5 UNIT END BEARING PREDICTION USING RANDOM SELECTION

The prior estimate of unit end bearing was based on the existence of boring information, i.e., rock strength, RQD, and modulus (E_i) within 100 ft of a foundation. For those scenarios where the foundation locations are unknown or if the rock information is lacking at a particular locale, an estimate of **mean or average** unit bearing over a site may be obtained. It will be shown that even though the mean unit end bearing will be predicted accurately over the site, the variability (i.e., standard deviation) will higher than the nearest boring approach and will result in lower ASD Fs and LRFD resistance factors (see section 11.6).

The approach starts by employing Monte Carlo simulation to generate representative strength (qu, and qt), and RQD data for a shaft embedded in a particular bearing rock layer from all the sample site information. For instance, consider the strength data (qu and qt) for Victory Bridge as shown in Figures 11.15 and 11.16. Since the sample size has only 30 to 100 samples, a Monte Carlo simulation will generate a large field population (> 10,000 points) using the statistic of the sample population (i.e., mean and standard deviation) with an assumed frequency distribution function (i.e., log normal, normal, etc.). For this example a lognormal distribution was assumed for qu and qt and a normal distribution was assumed for RQD (Fig. 11-18). Also, since Ei is correlated to qu (Chapter 7), the Ei field population (Figure 11.17) may be generated from the qu distribution (Figure 11.15).

After the field population of rock properties is known (Figs. 11.15-11.18), a representative sample (six to ten) of the strength (qu and qt), RQD and Ei are **randomly** selected to characterize the tip of a drilled shaft. The six to ten representative values characterize the zone of influence around the tip of shaft. It is expected that within 100 ft from the shaft (i.e., nearest boring), the rock properties would not be random; however outside this zone (i.e., no boring information), the latter assumption is valid. Subsequently, the six to ten random values were averaged to

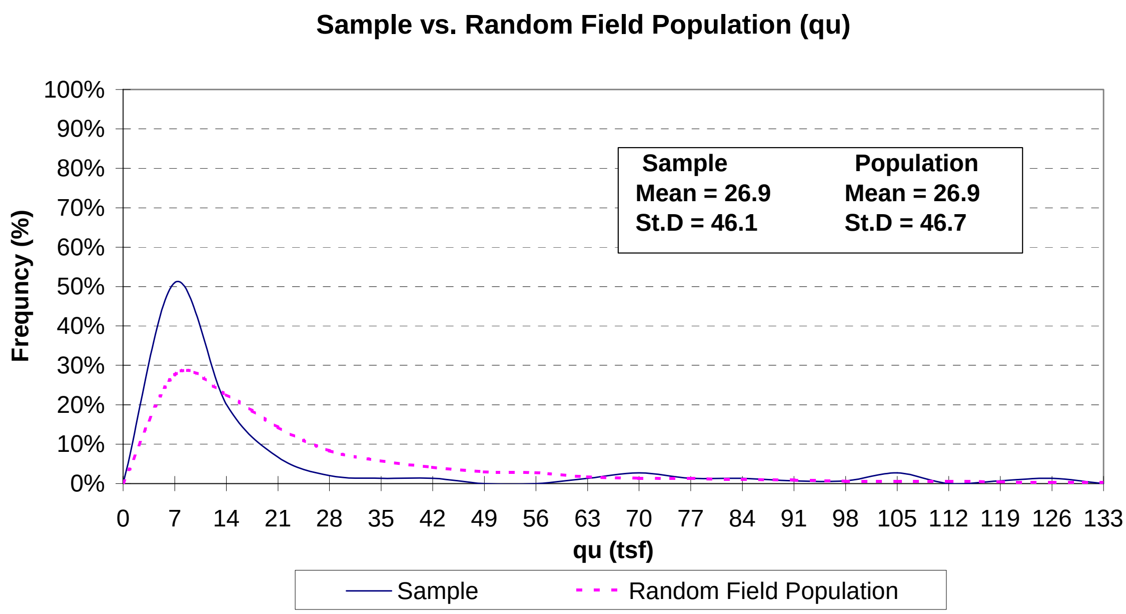


Figure 11.15 Sample and Random Field Population for qu at Victory.

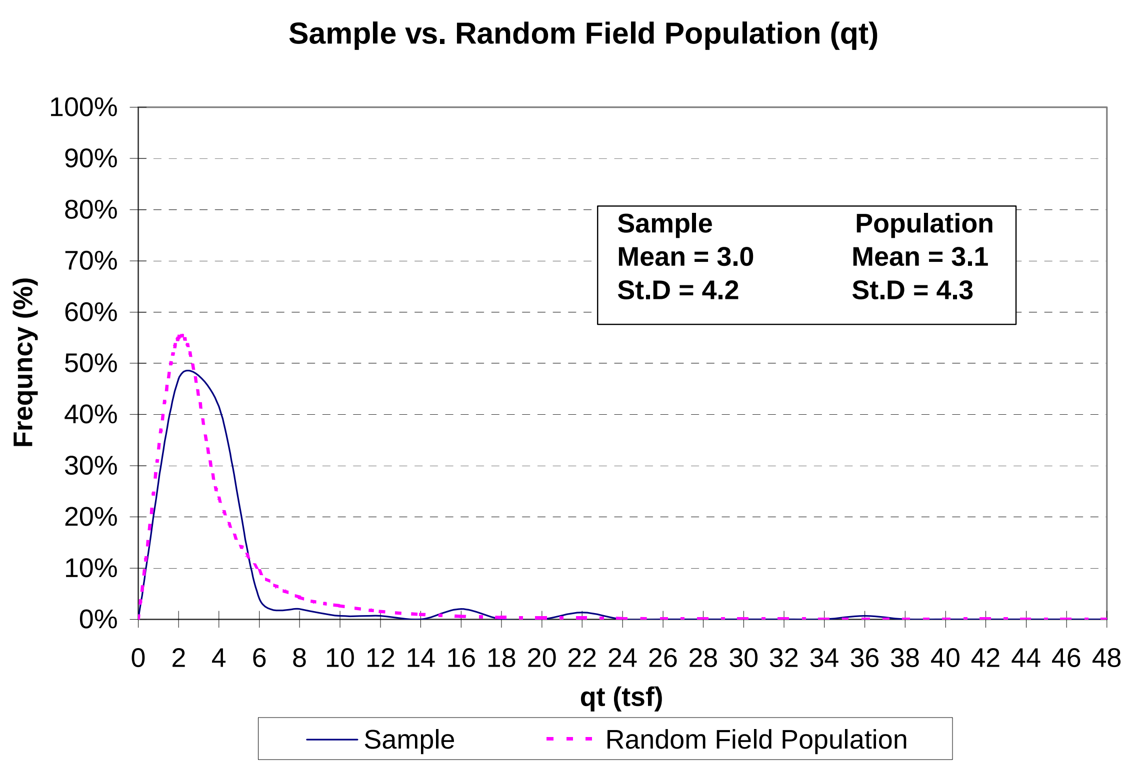


Figure 11.16 Sample and Random Field Population for qt at Victory

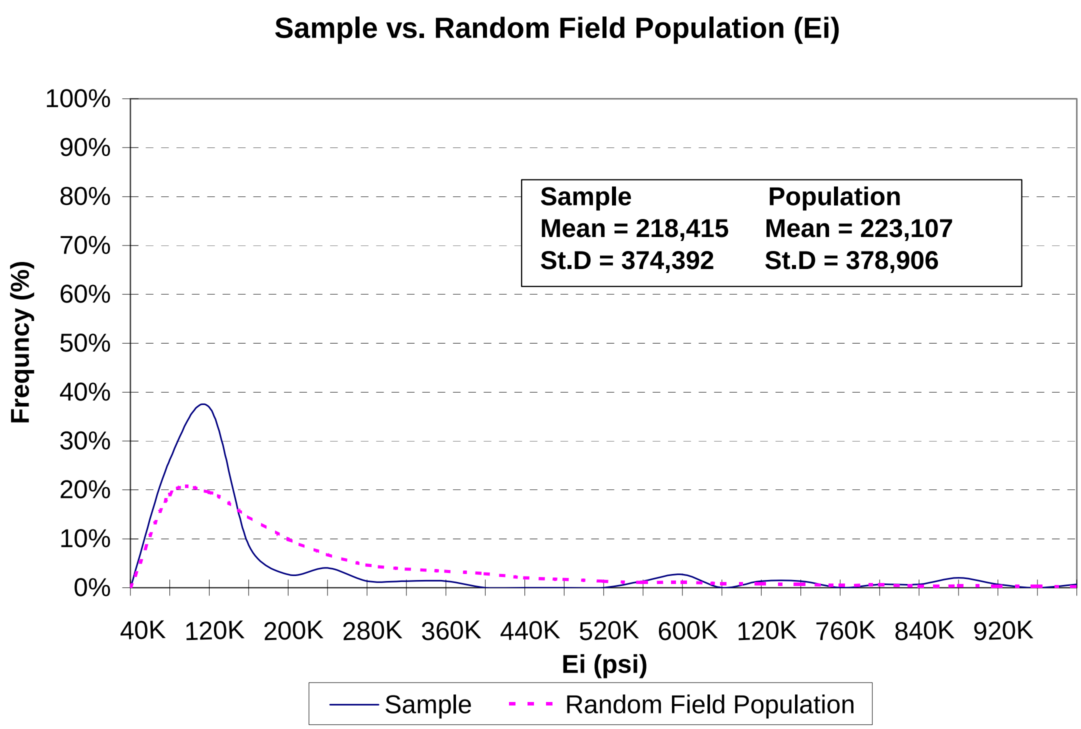


Figure 11.17 Sample and Random Field Population of Ei at Victory

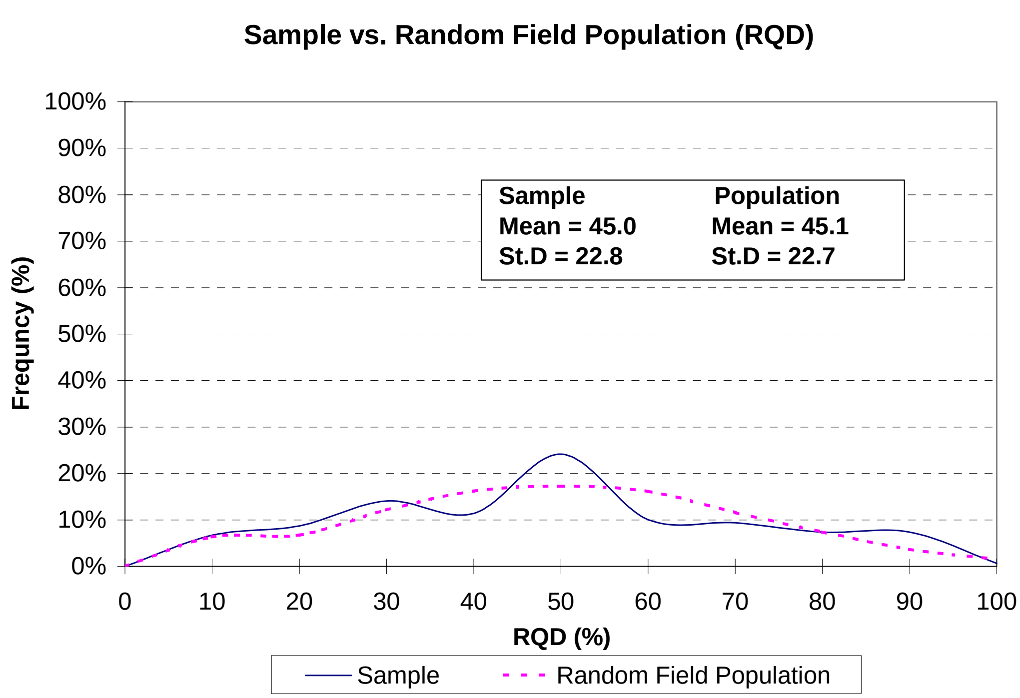


Figure 11.18 Sample and Random Field Population of RQD at Victory

obtain: strengths: qu_{avg}, qt_{avg}, Ei_{avg}, and RQD_{avg}, for a shaft. Then another random six to ten samples were selected and another set of average qu_{avg}, qt_{avg}, Ei_{avg}, RQD_{avg} values were found. The process was repeated (500 to 1000 times) until an average qu_{avg}, qt_{avg}, Ei_{avg}, RQD_{avg} distributions and associated standard deviations were determined for each property for the site. Using the estimated average qu_{avg}, qt_{avg}, Ei_{avg}, and RQD_{avg},, the average end bearing value at FDOT failure and its associated standard deviation were found from O'Neill's approach (section 10.2) for each site. Figure 11.19 shows both the predicted mean unit end bearing and its standard deviation for each site. Also shown in Figure 11.19 is the measured mean unit end bearing at FDOT failure for each site. The point markers in the figure represent individual load test results for each site. Based on the individual load test results, the standard deviation of unit end bearing for each site was computed and shown in Figure 11.19. The comparison between both mean and standard deviation for measured and predicted end bearing is quite good with the exception of the Gandy site's measured standard deviation. The high standard deviation may be may explained by the limited number of load tests (2).

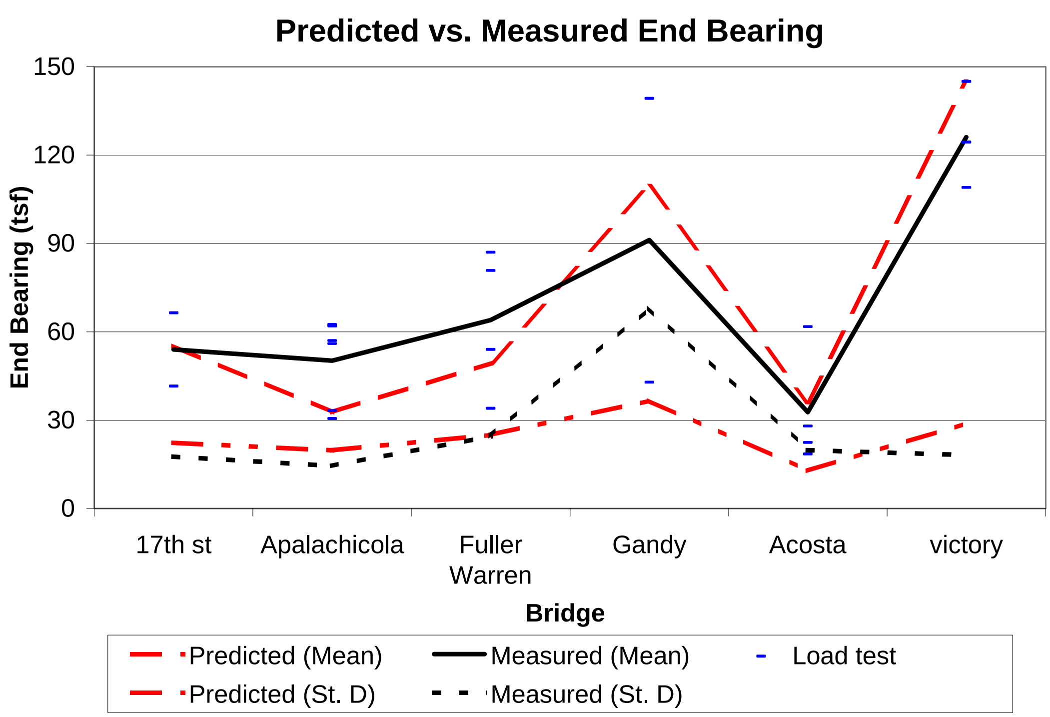


Figure 11.19 Monte Carlo Simulated End Bearing

11. 6 LRFD PHI FACTORS and ASD FS FOR O'NEILL'S END BEARING

The LRFD specifications as approved by AASHTO recommend the use of load factors to account for uncertainty in the loads, and resistance factors to account for uncertainty in the materials. AASHTO recommends that a probabilistic approach for estimating resistance factors be used based on a database of measured and predicted values.

In the case of the nearest boring data for load test, a total of 11 End bearing values was predicted as shown as Table 11.20. From the 11 measured and predicted unit end bearings at all the sites (Figure 11.14), the mean, λ_R (1.4), standard deviation, σ_R, and coefficient of variation, COV_R (0.29), were found for the FDOT/FHWA design approach. Using the computed mean, λ_R (1.40) and coefficient of variation, COV_R (0.29), the LRFD resistance factors, ϕ, were determined for different values of reliability index, β, (or probabilities of failure) from Eq. 8.3 and are presented in Table 11.21.

Using a dead load factor, γ_D, of 1.25, a live load factor, γ_L, of 1.75 and a dead to live load ratio, Q_D/Q_L, of 2 (typically varies between 1 and 3), the Factor of Safety, Fs, was determined, Eq. 8.10, and shown in Table 11.21. The same probability of failure or risk associated with the LRFD phi values apply to the ASD Factors of Safety. Note, the LRFD resistance values may be considered high (e.g., 0.71 for β = 2.5), but that is due to the slightly conservative nature of the prediction (i.e., λ_R =1.40).

Table 11.21 LRFD ϕ Factors, Probability of Failure (P_f) and Fs Based on Reliability, β for Nearest Boring Approach

Reliability, β	LRFD ϕ	P_f (%)	Factor of Safety
2.0	0.86	8.5	1.65
2.5	0.71	1.0	1.98
3.0	0.60	0.1	2.37
3.5	0.50	0.01	2.84
4.0	0.42	0.002	3.40
4.5	0.35	0.0002	4.07

For the case of no nearest boring, and the random selection approach (employing Monte Carlo simulation), section 11.5 (Fig. 11.19), the mean, λ_R, standard deviation, σ_R, and coefficient of variation, COV_R, were found. Using the computed mean, λ_R (1.21) and coefficient of variation, COV_R (0.46), the LRFD resistance factors, ϕ, were determined for different values of reliability index, β, and are presented in Table 11.22. Using a dead load factor, γ_D, of 1.25, a live load factor, γ_L, of 1.75 and a dead to live load ratio, Q_D/Q_L, of 2 (typically varies between 1 and 3), the Factor of Safety, Fs, was determined, Eq. 8.10, and shown in Table 11.22. The same probability of failure or risk associated with the LRFD phi values apply to the ASD Factors of Safety. Evident from a comparison of LRFD ϕ factors from nearest boring or random selection approach, the nearest boring has higher resistance factors for the same design approach. This difference may be attributed to the COV_R for each approach (0.29: nearest boring and 0.46: random selection). Evidently, the use of the nearest boring greatly diminishes the variability of rock properties and the subsequent mobilized unit end bearing. However, when no nearest boring information exists (i.e., within 100 ft), Table 11.22 has recommended Resistance Factors or Factors of Safety if the whole formations rock properties are used in conjunction with random selection.

Table 11.22 LRFD Phi Factors, Probability of Failure (P_f) and Fs for Random Selection (Monte Carlo Approach)

Reliability, β	LRFD ϕ	P_f (%)	Factor of Safety
2.0	0.56	8.5	2.60
2.5	0.43	1.0	3.32
3.0	0.33	0.1	4.24
3.5	0.26	0.01	5.42
4.0	0.21	0.002	6.92
4.5	0.16	0.0002	8.84

CHAPTER 12
CONCLUSIONS AND RECOMMENDATIONS

12.1 BACKGROUND

The Geotechnical, Construction, and Design information from eleven major FDOT bridges involving drilled shafts was collected for this research: 17th Causeway Bridge (State Job #86180-3522), Acosta Bridge (State Job #72160-3555), Apalachicola River Bridge, SR20 (State job #47010-3519/56010-3520), Christa Bridge (State job #70140-3514), Fuller Warren Bridge Replacement Project (State Job #72020-3485/2142478), Gandy Bridge (State Job #10130-3544/7113370), Hillsborough Bridge (State Job #10150-3543/3546), McArthur Bridge (State job 87060-3549), Venetian Causeway (State job #87000-3601), Victory Bridge (State job #53020-3540), and West 47th over Biscayne Water Way (State job #87000-3516). From the Geotechnical Reports, a total of thirty-eight load tests (27 Osterberg and 11 Statnamic) and associated information (skin friction, end-bearing, load displacement, etc.) was collected. The construction information included: bid documents, CPM schedules, as well as the built drilled shaft information (size, lengths, etc.). The design information included bridge plans, which detailed shaft sizes, shaft lengths and shaft loads, as well as soil boring information.

Of interest in this study was the current practice of design, as well as load testing of drilled shaft foundations in Florida. In the case of design, the following was investigated and reported on:

- Evaluation of FDOT's design skin friction approach for Florida limestone with recommended LRFD resistance factors;
- Evaluation of FDOT tip resistance vs. tip displacement model (O'Neill) for shafts founded in Florida limestone with recommended LRFD resistance factors;
- Rock properties for a number of formations (Tampa, Fort Thompson, etc.) and associated correlations (i.e., qu with Ei)

For load testing, the following was investigated and reported on:

- Comparison of Skin Friction and End bearing from Osterberg and Statnamic Load testing.
- Direct Cost to FDOT for performing field load testing (Osterberg or Statnamic) as function of shaft size
- Impact of Load Testing on foundation construction process

Based on both the design and load testing information, the report concludes with suggested guidelines on the use of drilled shaft field-testing for FDOT structures (i.e., bridges).

12.2 DRILLED SHAFT DESIGN IN FLORIDA LIMESTONE

In the case of predicting both the mean and variability of drilled shaft skin friction in limestone, the FDOT procedure does an excellent job as shown Figure 12.1. The variability (i.e., 1-standard deviation) represents the range in unit skin friction for a shaft on any site. The latter was also predicted from the laboratory strength data.

For design, the mean and standard deviation of the average unit skin friction is required for a site. The latter may be predicted successfully with simple random sampling and is shown with measured shaft variability in Figure 12.2. Table 12.1 shows the computed LRFD ϕ factors, and ASD safety factors, Fs, for various reliability or probabilities of failure.

In the case of end bearing, the O'Neill method as described in Chapter 11.2 and implemented in FB-Pier was evaluated. Shown in Figure 12.3 is both the measured and predicted unit end bearing at FDOT shaft failure (elastic compression + settlement equal to shaft diameter/30) from boring information within 100 ft of each load test. Table 12.2 shows the LRFD ϕ, and ASD Fs for given reliability values. For the predictions, the mass Young's Modulus of the rock, E_m, or Young's Modulus of the intact rock samples, E_i, and average RQD for the rock has to be known. In the case of only having strength data of the rock, i.e., qu, individual correlations of qu vs. E_i are given for different formations (Tampa, Jacksonville, etc.).

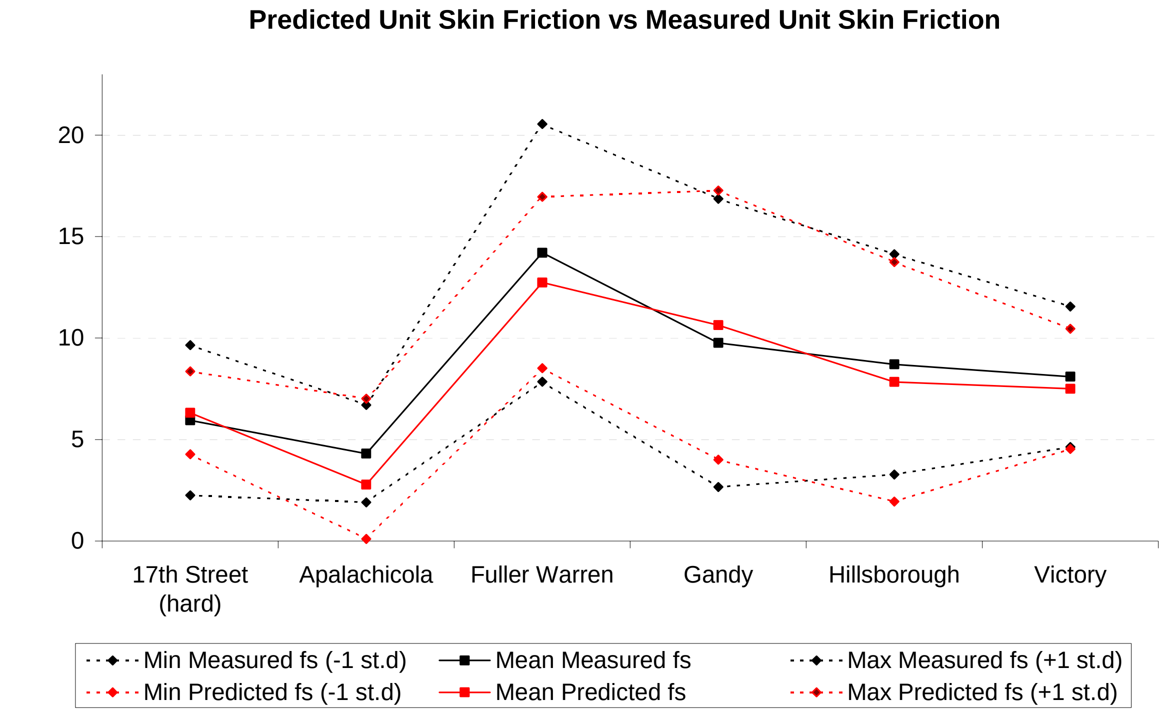


Figure 12.1 (Fig. 8.5) Predicted Unit Skin Friction vs. Measured Unit Skin Friction

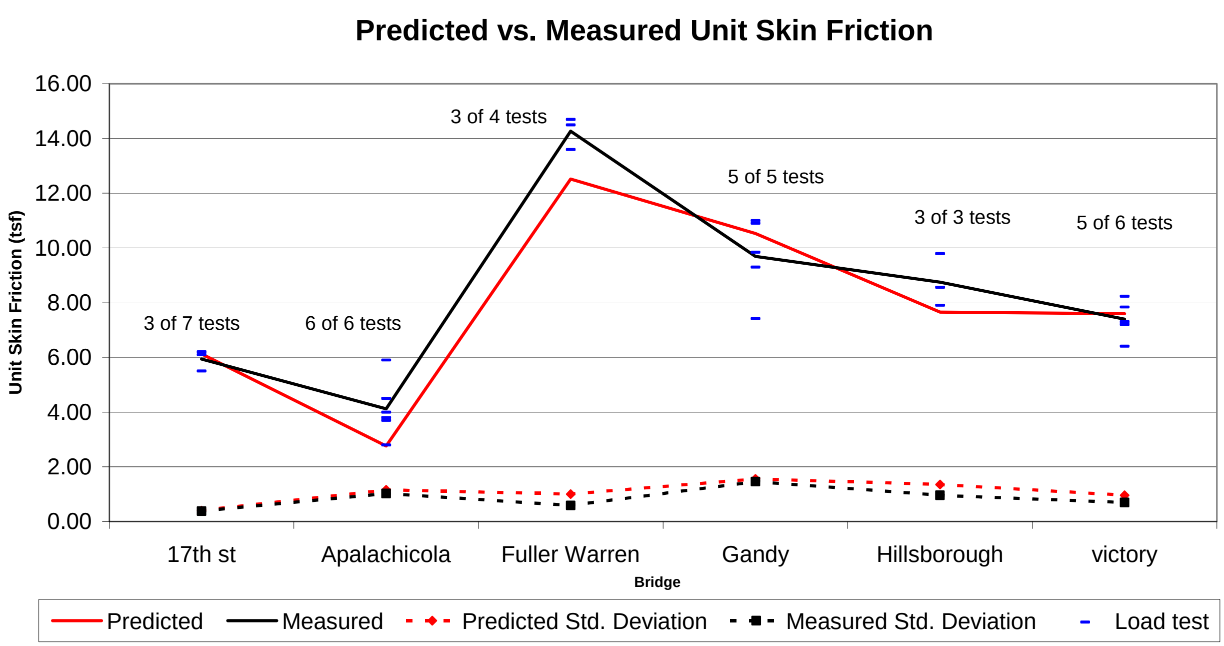


Figure 12.2 (Fig. 8.8) Measured vs. Predicted Average Unit Skin Friction with Variability

Table 12.1 LRFD ϕ Factors, Probability of Failure (P_f) and Fs for Skin Friction

Reliability, β	LRFD ϕ	P_f (%)	Factor of Safety, Fs
2.0	0.81	8.5	1.75
2.5	0.69	1.0	2.0
3.0	0.59	0.1	2.4
3.5	0.51	0.01	2.85
4.0	0.42	0.002	3.35
4.5	0.36	0.0002	3.95

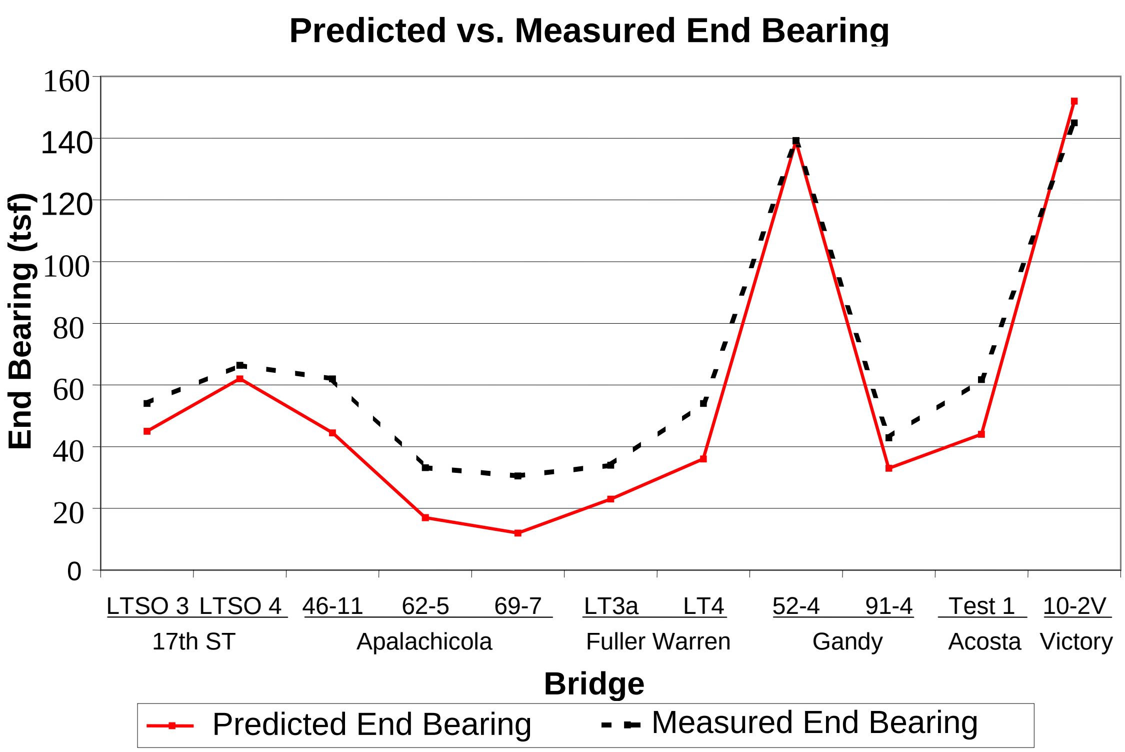


Figure 12.3 (Fig. 11.15) Comparison of Measured and Predicted End Bearing

Table 12.2 LRFD ϕ Factors, Probability of Failure (P_f) and ASD Fs for End Bearing Based on Nearest Boring

Reliability, β	LRFD ϕ	P_f (%)	Factor of Safety
2.0	0.86	8.5	1.65
2.5	0.71	1.0	1.98
3.0	0.60	0.1	2.37
3.5	0.50	0.01	2.84
4.0	0.42	0.002	3.40
4.5	0.35	0.0002	4.07

In the case of no rock properties (qu, Ei, RQD) within 100 ft of a planned drilled foundation, the use of mean rock properties of a formation are recommended. However, due to the increased variability, the design LRFD ϕ, and ASD Fs diminish as shown in Table 12.3.

Table 12.3 LRFD Phi Factors, and ASD Fs for End Bearing Using All Site Data

Reliability, β	LRFD ϕ	P_f (%)	Factor of Safety
2.0	0.56	8.5	2.60
2.5	0.43	1.0	3.32
3.0	0.33	0.1	4.24
3.5	0.26	0.01	5.42
4.0	0.21	0.002	6.92
4.5	0.16	0.0002	8.84

12.3 FIELD LOAD TESTING DRILLED SHAFT IN FLORIDA

The study initiated with a comparison of drilled shaft capacities (skin friction and end bearing) measured with Osterberg and Statnamic equipment (see Chapter 5). For instance, Figure 12.4 shows average unit skin friction from five Statnamic and Osterberg tests within 25 ft of each other. The recorded variability (Statnamic vs. Osterberg), Figure 12.4, was well within the site variability of all the shafts.

Next, the study determined the cost of performing load testing and the time impact the testing has on the overall construction. For instance, shown in Table 12.4 is the estimated cost of performing variable size Osterberg Tests. Generally, load testing 42” shafts and smaller cost approximately $108,000, whereas 72” size shafts cost $120,000 per test. The latter includes direct costs, i.e., materials, CEI, etc. But it does not consider in direct cost, such as impact on construction schedule (see Chapter 10: approximately 2 days for each test).

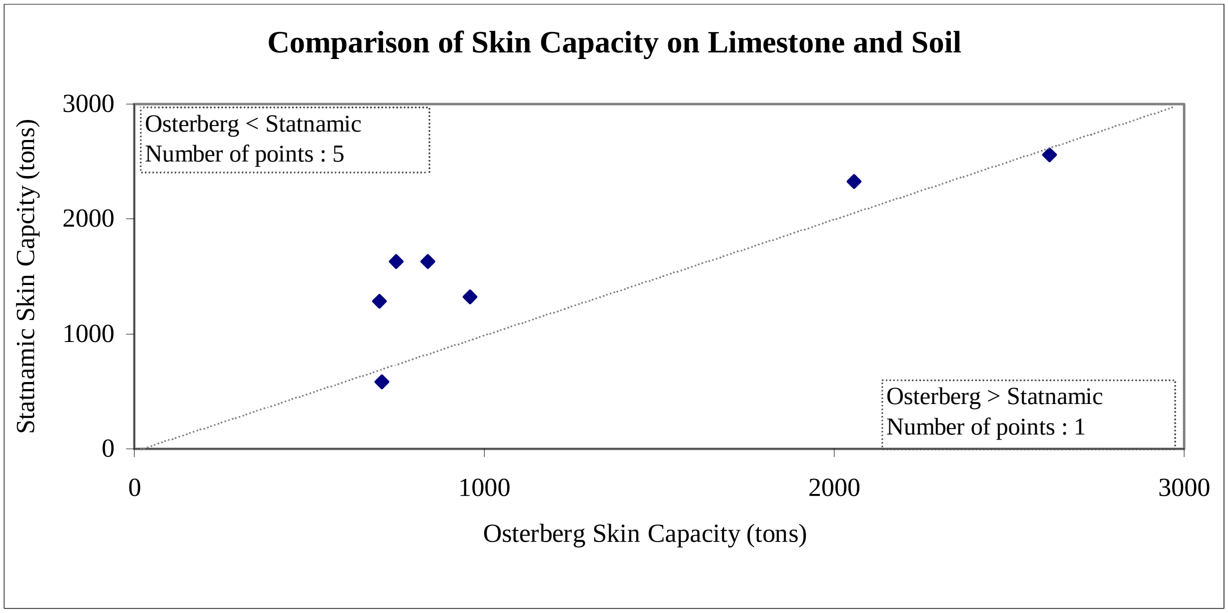


Figure 12.4 Unit Skin Friction from Statnamic vs. Osterberg Results

Table 12.4 Estimated Unit Cost for Field Load Testing of Drilled Shafts

Item	Consultant Price to Perform Test ($)	Contractor Support Cost ($)	Contractor Daily Project Overhead Cost at $10,800. per day ($)*	Owner Daily Project Overhead Cost at $8,000 per day ($) *	Total Estimated Cost ($)
Osterberg Test of 42 inch diameter shafts (Ea.)	$54,000	$16,000	$21,600	$16,000	$107,600
Osterberg Test of 72 inch diameter shafts (Ea.)	$65,500	$16,000	$21,600	$16,000	$119,100

* Two days estimated time impact

The study also looked at the influence of load testing on the as built shaft lengths vs. the designed lengths (see Chapter 9.4). Four of the five projects studied had an as-built drilled shaft bid cost slightly lower than the as-designed bid cost, but the average as-built bid cost exceeded the as-design bid cost. For instance, the average bid cost of drilled shaft testing on the five projects studied was $779,838 with an average drilled shaft bid cost savings of (-) $28,139 (i.e., they ended up costing more).

The study concludes with the recommendations or suggested guidelines on field load testing drilled shafts in Florida.

12.4 SUGGESTED GUIDELINES ON SELECTING THE NUMBER OF FIELD LOAD TESTS IN DESIGN

It is readily recognized that loading testing drilled shafts to validate construction practices (i.e., wet/cased hole construction, etc.) is a necessity; however, the number of load tests, which should be considered in design and their impact, is a question.

As identified in chapter eight, both AASHTO and FDOT allows the designer to change the ASD Fs or LRFD resistance factors ϕ, based on construction control (i.e., load testing) as shown in Table 12.5. Of interest is the total change in foundation cost on a site if different ASD Fs or LRFD resistance factors, ϕ, are considered. For instance, shown in Figure 12.5 (Fig. 8.11) is the change in foundation cost if different ASD Fs or LRFD ϕ are used with increasing reliabilities (2.0 and above) (see Chapter 8.8).

Table 12.5 AASHTO (1999) Recommended Fs Based on Specified Construction Control

Increasing Construction Control --------->				
Subsurface Exploration (e.g., SPT, cores, etc.)	[(1)]X	X	X	X
Static Calculations (e.g., FDOT design Method)	X	X	X	X
Dynamic Field Measurements			X	
Static Load Testing			X	X
Factor of Safety, Fs	3.5	2.5	2.0	1.9

[(1)] X = Construction Control Specified on Contract Plans

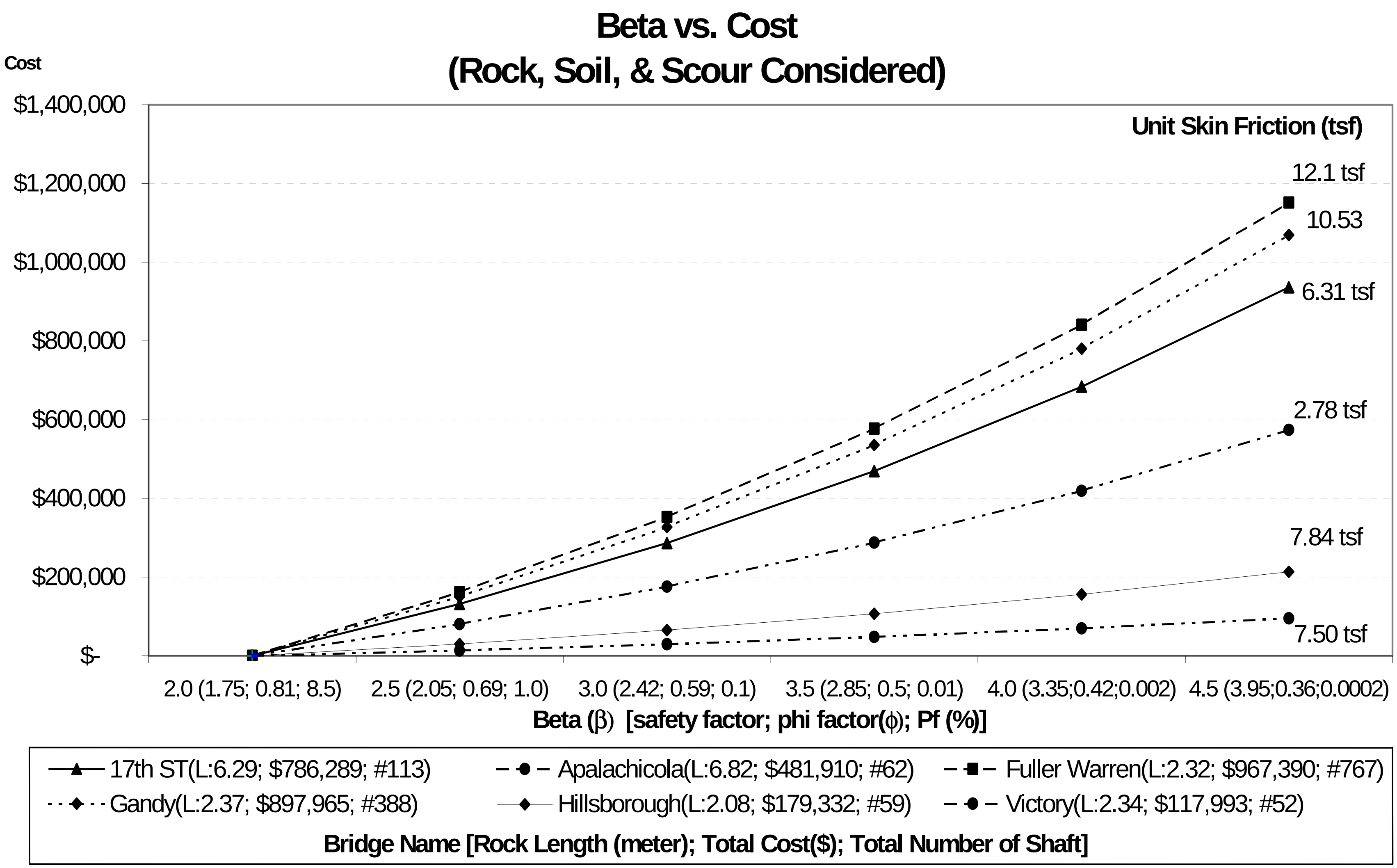


Figure 12.5 (Fig. 8.11) Change in Foundation Cost vs. Reliability [Factor of Safety (Fs), LRFD phi, Probability of Failure (Risk)]

Evident from Figure 12.5 there may be significant differences in foundation costs if different ASD Fs or LRFD ϕ are used in the design. Consequently as suggested by AAHSTO, significant savings may occur if load testing on a site (Table 12.5) is done. For instance, consider the Fuller Warren site in Figure 12.5; the design engineers may have originally designed the foundation system with Fs of 3.5. However, the latter design has a probability of failure of one in a half-million. Consequently, the engineers may have decided to reduce the Fs to 2.5 (probability of failure five in ten thousand, Table 12.1) with the use of three-field load tests cost at an additional cost of three hundred and forty six thousand dollars, (2x $119K + 108K, Table 12.4) to verify design. From Figure 12.5, the savings from reducing the Fs from 3.5 to 2.5 would be approximately five hundred thousand dollars, ($900K -$400K). The net savings (shaft length savings – cost of load testing) would be approximately one hundred fifty-four thousand dollars ($500K - $346K) with very acceptable risk.

However, Figure 12.5 (foundation cost vs. reliability), and Table 12.4 (field testing cost), suggests that there are some sites were extensive cost savings would not occur with field-testing. An example of the latter is the Victory Bridge site, where the total bridge foundation cost only increases by a hundred thousand dollars if the reliability increases from 2 (probability of failure 8 in hundred) to a value of 4.5 (probability of failure 2 in a million). Consequently, the design engineer may decide to only perform one or two field load test to validate design or ensure proper construction practice.

With the above approach, the design engineer has a rapid means of comparing the complete foundation costs (i.e., shaft cost as well as load testing) of alternate systems (i.e., drilled shafts, driven piles, etc.). Also, if the FDOT adopts the end bearing analysis with associated LRFD resistance factors, ϕ, developed in Table 12.2, it is suggested that the benefit/cost plot (Figure 12.5) be modified to include both skin friction and end bearing analyses.

12.5 ACKNOWLEDGEMENTS

The authors would like to acknowledge the help of Districts 1,2, and 7 Geotechnical personnel in obtaining rock cores for Fuller Warren, Acosta, and Gandy Bridges. Also, the testing of the cores by the State Materials Office (Dave Horhota, Ben Watson, and Summer White) is much appreciated.

APPENDIX A

SHAFT DIMENSIONS, SOIL DESCRIPTION and ROCK ELEVATIONS
for AXIAL LOAD TESTS

The following is a description of the Nomenclature used in Appendix A Tables:

Length: total length of the shaft.

Date of Test: the date in which load test was performed.

Ground Level: the elevation of the ground surface.

Bottom of Casing: the elevation of the bottom of casing in the test shaft.

Last Strain Gauge Elevation: the elevation of the lowest stain gauge that measures the load transfer. For Osterberg, commonly the Osterberg cell is located at the lowest location.

Top of Rock Elevation: the top elevation of rock socket.

Total Rock Socket: the length of the shaft from the top of rock elevation to tip elevation.

Top Elevation: the elevation of the top of the shaft.

Tip Elevation: the elevation of the tip of the shaft.

Embedded Length: the length of the shaft from the ground elevation to the tip of the shaft.

Soil type: general soil profile along the shaft.

Casing length: the length of casing.

Method Used: method of constructions – wet (slurry) or casing.

17^{th} Street Bridge

Name	LTSO1, 17^{th}
Type	Osterberg & Statnamic
Station No	35+46.49 13.73(m) LT
Nearest boring	BB-7
Length (ft)	119.392
Diameter (in)	48
Date of Test	4/28/98
Water Level (ft)	1.64
Ground Level (ft)	-16
Bottom of casing	-67
Last strain gauge Elevation	-108.3
Top of Rock Elevation	-90
Test Rock Socket (ft)	18.3
Total Rock Socket	23.5
Top Elevation	6.6
Tip Elevation	-113.5
Embedded Length	97.48
Soil type	Sand/lime
Casing length	11.8
Method Used	Wet (Sea water)

Name	LTSO3, 17^{th}
Type	Osterberg & Statnamic
Station No	34+82.495 11.865(m) LT
Nearest boring	BB-4
Length (ft)	100.1
Diameter (in)	48
Date of Test	6/22/98
Water Level (ft)	1.64
Ground Level (ft)	5
Bottom of casing	-6.8
Last strain gauge Elevation	-84
Top of Rock Elevation	-76.19
Test Rock Socket (ft)	7.81
Total Rock Socket	18.91
Top Elevation (ft)	5
Tip Elevation (ft)	-95.1
Embedded Length (ft)	100.1
Soil type	Sand/lime
Casing length (ft)	11.8
Method used	Wet (Sea water)

Name	LTSO2, 17^{th}
Type	Osterberg & Statnamic
Station No	36+15.15 13.73(m) LT
Nearest boring	BB-10
Length (ft)	142
Diameter (in)	48
Date of Test	5/12/98
Water Level (ft)	1.64
Ground Level (ft)	-17.94
Bottom of casing	-75
Last strain gauge Elevation	-121.4
Top of Rock Elevation	-40
Test Rock Socket (ft)	46.4
Total Rock Socket	55.5
Top Elevation (ft)	11.5
Tip Elevation (ft)	-130.5
Embedded Length (ft)	112.6
Soil type	Sand/lime
Casing length (ft)	85.87
Method used (ft)	Wet (Sea water)

Name	LTSO 4, 17^{th}
Type	Osterberg & Statnamic
Station No	38+04.145 11.865(m) LT
Nearest boring	No info
Length (ft)	77.47
Diameter (in)	48
Date of Test	6/26/98
Ground Level (ft)	17.48
Bottom of casing	-10.7
Last strain gauge Elevation	-59.7
Top of Rock Elevation	No info
Test Rock Socket (ft)	No info
Total Rock Socket	No info
Top Elevation	15.1
Tip Elevation	-62.3
Embedded Length	79.78
Soil type	Sand/lime
Casing length	32.47
Method used	Wet (Sea water)

17^{th} Street Bridge

Name	LT 1, 17^{th}
Type	Statnamic Test
Station No	32+91.395, 10.11LT
Nearest boring	BB-1, N-6
Length (ft)	72.12
Diameter (in)	48
Date of Test	7/10/98
Water Level (ft)	No info
Ground Level (ft)	9.8
Bottom of casing	-3
Last strain gauge Elevation	-62.32
Top of Rock Elevation	9.8
Test Rock Socket (ft)	59.32
Total Rock Socket	59.32
Top Elevation	9.8
Tip Elevation	-62.32
Embedded Length	72.12
Soil type	Sand/lime
Casing length	No info
Method used	Wet (Sea water)

Name	LT 2, 17^{th}
Type	Statnamic Test
Station No	36+82.745, 10.11LT
Nearest boring	N-19, BB-3, BB-5
Length (ft)	61.27
Diameter (in)	48
Date of Test	7/11/98
Water Level (ft)	No info
Ground Level (ft)	-3.23
Bottom of casing	No info
Last strain gauge Elevation	No info
Top of Rock Elevation	No info
Test Rock Socket (ft)	No info
Total Rock Socket	No info
Top Elevation	-3.23
Tip Elevation	-64.5
Embedded Length	61.27
Soil type	Sand/lime
Casing length	No info
Method used	Wet (Sea water)

Acosta Bridge

Name	Test 1, Acosta
Type	Osterberg Test
Station No	136+39.86
Nearest boring	No info
Length (ft)	64.19
Diameter (in)	36
Date of Test	4/13/90
Water Level (ft)	0
Ground surface (ft)	-22.3
Bottom of casing	-30.17
Last strain gauge Elevation	-53.86
Top of Rock Elevation	-22.3
Test Rock Socket (ft)	7.87
Total Rock Socket	32.89
Top Elevation (ft)	9
Tip Elevation (ft)	-55.19
Embedded Length (ft)	32.89
Soil type	Sand/clay/rock
Casing length (ft)	39.17
Method used (ft)	Wet (Slurry)

Name	Test 2, Acosta
Type	Osterberg & Conventional
Station No	138+27
Nearest boring	No info
Length (ft)	101.2
Diameter (in)	36
Date of Test	4/26/90
Water Level (ft)	0
Ground surface (ft)	-24
Bottom of casing	-31.38
Last strain gauge Elevation	-90.86
Top of Rock Elevation	-95
Test Rock Socket (ft)	0
Total Rock Socket	0
Top Elevation (ft)	9
Tip Elevation (ft)	-92.2
Embedded Length (ft)	68.2
Soil type	Rock/salty sand
Casing length (ft)	40.38
Method used (ft)	Wet (Slurry)

Acosta Bridge

Name	Test 4, Acosta
Type	Osterberg & Conventional
Station No	145+35.75
Nearest boring	No info
Length (ft)	113.92
Diameter (in)	36
Date of Test	5/12/90
Water Level (ft)	0
Ground surface (ft)	-28.4
Bottom of casing	-32.72
Last strain gauge Elevation	-103.57
Top of Rock Elevation	-105
Test Rock Socket (ft)	0
Total Rock Socket	0
Top Elevation (ft)	9
Tip Elevation (ft)	-104.92
Embedded Length (ft)	76.52
Soil type	Rock/salty sand
Casing length (ft)	41.72
Method used (ft)	Wet (Slurry)

Name	Test 5A, Acosta
Type	Osterberg Test
Station No	147+90, 8'RT
Nearest boring	No info
Length (ft)	87.83
Diameter (in)	36
Date of Test	5/16/90
Water Level (ft)	0
Ground surface (ft)	-25.5
Bottom of casing	-28.57
Last strain gauge Elevation	-77.49
Top of Rock Elevation	-79
Test Rock Socket (ft)	0
Total Rock Socket	0
Top Elevation (ft)	9
Tip Elevation (ft)	-78.83
Embedded Length (ft)	53.33
Soil type	Rock/salty sand
Casing length (ft)	37.57
Method used (ft)	Wet (Slurry)

Apalachicola Bridge

Name	46 -11A, Apalachicola
Type	Osterberg Test
Station No	624+03, 2.5'RT
Nearest boring	TH-46A, 46B
Length (ft)	85
Diameter (in)	60
Date of Test	8/26/96
Water Level (ft)	37
Ground Level (ft)	45
Bottom of casing	45
Last strain gauge Elevation	-37
Top of Rock Elevation	-13
Test Rock Socket (ft)	24
Total Rock Socket	24
Top Elevation (ft)	48
Tip Elevation (ft)	-37
Embedded Length (ft)	82
Soil type	Sand/Soft Li/Hard Li
Casing length (ft)	50
Method used (ft)	Wet

Name	53-2, Apalachicola
Type	Osterberg Test
Station No	631+79, 17.5'RT
Nearest boring	TH-53A, 53B
Length (ft)	89.5
Diameter (in)	72
Date of Test	7/17/96
Water Level (ft)	34
Ground Level (ft)	46.4
Bottom of casing	46.4
Last strain gauge Elevation	-40.2
Top of Rock Elevation	-14.87
Test Rock Socket (ft)	25.33
Total Rock Socket	25.33
Top Elevation (ft)	47.8
Tip Elevation (ft)	-40.2
Embedded Length (ft)	88.1
Soil type	Sand/Soft Li/Hard Li
Casing length (ft)	50
Method used (ft)	Wet

Apalachicola Bridge

Name	57-10, Apalachicola
Type	Osterberg Test
Station No	636+12, 2.5'RT
Nearest boring	P57-1, 2,3,4
Length (ft)	103.7
Diameter (in)	84
Date of Test	8/19/96
Water Level (ft)	37
Ground Level (ft)	47.5
Bottom of casing	-21
Last strain gauge Elevation	-52
Top of Rock Elevation	-20
Test Rock Socket (ft)	32
Total Rock Socket	35.2
Top Elevation (ft)	48.5
Tip Elevation (ft)	-55.2
Embedded Length (ft)	102.7
Soil type	Sand/Soft Li/Hard Li
Casing length (ft)	69.5
Method used (ft)	Wet

Name	59-8, Apalachicola
Type	Osterberg Test
Station No	641+38, 62.5'RT
Nearest boring	P59-3, 4
Length (ft)	134
Diameter (in)	108
Date of Test	2/18/97
Water Level (ft)	46
Ground Level (ft)	17
Bottom of casing	-28
Last strain gauge Elevation	-69.3
Top of Rock Elevation	-20
Test Rock Socket (ft)	49.3
Total Rock Socket	58.5
Top Elevation (ft)	55.5
Tip Elevation (ft)	-78.5
Embedded Length (ft)	95.5
Soil type	Sand/Soft Li/Hard Li
Casing length (ft)	47
Method used (ft)	Wet

Apalachicola Bridge

Name	62-5, Apalachicola
Type	Osterberg Test
Station No	645+97, 17.5'RT
Nearest boring	TH-62A, 62B
Length (ft)	89.2
Diameter (in)	72
Date of Test	12/6/96
Water Level (ft)	32
Ground Level (ft)	45.9
Bottom of casing	47
Last strain gauge Elevation	-42.2
Top of Rock Elevation	-24
Test Rock Socket (ft)	18.2
Total Rock Socket	18.2
Top Elevation (ft)	47
Tip Elevation (ft)	-42.2
Embedded Length (ft)	88.1
Soil type	Sand/Soft Li/Hard Li
Casing length (ft)	50
Method used (ft)	Wet

Name	69-7, Apalachicola
Type	Osterberg Test
Station No	653+41+17.8'RT
Nearest boring	TH-69A, 69B
Length (ft)	99.1
Diameter (in)	60
Date of Test	12/4/96
Water Level (ft)	35
Ground Level (ft)	45.3
Bottom of casing	45.3
Last strain gauge Elevation	-49
Top of Rock Elevation	-27
Test Rock Socket (ft)	22
Total Rock Socket	25.1
Top Elevation (ft)	47
Tip Elevation (ft)	-52.1
Embedded Length (ft)	97.4
Soil type	Sand/Soft Li/Hard Li
Casing length (ft)	50
Method used (ft)	Wet

Fullern Warren Bridge

Name	LT-1, Fuller Warren
Type	Osterberg Test
Station No	283+05 50'LT
Nearest boring	LT-1
Length (ft)	41.01
Diameter (in)	36
Date of Test	9/11/96
Water Level (ft)	3
Ground Level (ft)	5.44
Bottom of casing	5.44
Last strain gauge Elevation	-28.5
Top of Rock Elevation	-18
Test Rock Socket (ft)	10.5
Total Rock Socket	17.1
Top Elevation (ft)	5.91
Tip Elevation (ft)	-35.1
Embedded Length (ft)	40.54
Soil type	Sand/lime
Casing length (ft)	36
Method used (ft)	Wet

Name	LT-2, Fuller Warren
Type	Osterberg Test
Station No	313+21 66'RT
Nearest boring	LT-2
Length (ft)	27.85
Diameter (in)	72
Date of Test	10/30/96
Water Level (ft)	0.5
Mud line (ft)	-21
Bottom of casing	(-41.38) Double casing
Last strain gauge Elevation	-62.55
Top of Rock Elevation	-41.38
Test Rock Socket (ft)	21.17
Total Rock Socket	22.47
Top Elevation (ft)	-36
Tip Elevation (ft)	-63.85
Embedded Length (ft)	22.62
Soil type	Sandy silt/fine sand
Casing length (ft)	47.89
Method used (ft)	Wet

Fuller Warren Bridge

Name	LT-3a, Fuller Warren
Type	Osterberg Test
Station No	323+45 19'RT
Nearest boring	LT-3a
Length (ft)	120.73
Diameter (in)	72
Date of Test	12/9/96
Water Level (ft)	0.5
Mud line (ft)	-56
Bottom of casing	-85.4
Last strain gauge Elevation	-111
Top of Rock Elevation	-85.4
Test Rock Socket (ft)	25.6
Total Rock Socket	27.33
Top Elevation (ft)	8
Tip Elevation (ft)	-112.73
Embedded Length (ft)	27.58
Soil type	Salty fine sand
Casing length (ft)	99
Method used (ft)	Wet

Name	LT-4, Fuller Warren
Type	Osterberg Test
Station No	341+25 78'RT
Nearest boring	LT-4
Length (ft)	66.75
Diameter (in)	48
Date of Test	9/24/96
Water Level (ft)	8
Ground surface (ft)	20
Bottom of casing	20(Temporary)
Last strain gauge Elevation	-44.5
Top of Rock Elevation	-11.2
Test Rock Socket (ft)	33.3
Total Rock Socket	34.55
Top Elevation (ft)	21
Tip Elevation (ft)	-45.75
Embedded Length (ft)	65.75
Soil type	Sand/lime/silt
Casing length (ft)	46.5
Method used (ft)	Wet

Gandy Bridge

Name	26-1, Gandy
Type	Statnamic Test
Station No	68+66.75
Nearest boring	No info
Length (ft)	33.4
Diameter (in)	48
Date of Test	12/17/94
Water Level (ft)	0
Ground Level (ft)	-7.4
Bottom of casing	-8.94
Last strain gauge Elevation	-24.7
Top of Rock Elevation	-16.7
Test Rock Socket (ft)	8
Total Rock Socket	8
Top Elevation (ft)	8.7
Tip Elevation (ft)	-24.7
Embedded Length (ft)	8
Soil type	Sand/lime
Casing length (ft)	1.54
Method used (ft)	Wet

Name	26-2, Gandy
Type	Osterberg Test
Station No	68+66.75 RT 6'
Nearest boring	SB-21
Length (ft)	38.4
Diameter (in)	48
Date of Test	About 11/28/94
Water Level (ft)	0
Ground Level (ft)	-7.4
Bottom of casing	-11.5
Last strain gauge Elevation	-20.6
Top of Rock Elevation	-16.7
Test Rock Socket (ft)	3.9
Total Rock Socket	13.7
Top Elevation (ft)	8
Tip Elevation (ft)	-30.4
Embedded Length (ft)	23
Soil type	Sand/lime
Casing length (ft)	19.5
Method used (ft)	Wet

Gandy Bridge

Name	52-3, Gandy
Type	Statnamic Test
Station No	93+62.75
Nearest boring	No info
Length (ft)	55.6
Diameter (in)	48
Date of Test	12/13/94
Water Level (ft)	0
Ground Level (ft)	-11
Bottom of casing	-24
Last strain gauge Elevation	-46.3
Top of Rock Elevation	-23
Test Rock Socket (ft)	22.3
Total Rock Socket	23.5
Top Elevation (ft)	9.1
Tip Elevation (ft)	-46.5
Embedded Length (ft)	23.5
Soil type	Sand/lime
Casing length (ft)	13
Method used (ft)	Wet

Name	52-4, Gandy
Type	Osterberg Test
Station No	93+62.75
Nearest boring	SB-36
Length (ft)	54.5
Diameter (in)	48
Date of Test	About 11/21/94
Water Level (ft)	0
Ground Level (ft)	-11
Bottom of casing	-20.33
Last strain gauge Elevation	-42
Top of Rock Elevation	-20
Test Rock Socket (ft)	21.67
Total Rock Socket	27.7
Top Elevation (ft)	6.8
Tip Elevation (ft)	-47.7
Embedded Length (ft)	36.7
Soil type	Sand/lime
Casing length (ft)	27.1
Method used (ft)	Wet

Gandy Bridge

Name	91-3, Gandy
Type	Statnamic Test
Station No	174+21.25
Nearest boring	No info
Length (ft)	70.68
Diameter (in)	48
Date of Test	12/8/94
Water Level (ft)	0
Ground Level (ft)	-14
Bottom of casing	-40.5
Last strain gauge Elevation	-61.6
Top of Rock Elevation	-40.5
Rock Socket (ft)	21.1
Total Rock Socket	21.1
Top Elevation (ft)	9.08
Tip Elevation (ft)	-61.6
Embedded Length (ft)	-21.1
Soil type	Sand/lime
Casing length (ft)	26.5
Method used (ft)	Wet

Name	91-4, Gandy
Type	Osterberg Test
Station No	174+21.25
Nearest boring	SB-91
Length (ft)	74.7
Diameter (in)	48
Date of Test	About 11/11/94
Water Level (ft)	0
Ground Level (ft)	-14
Bottom of casing	-43
Last strain gauge Elevation	-59.6
Top of Rock Elevation	-40.5
Test Rock Socket (ft)	19.1
Total Rock Socket	27.2
Top Elevation (ft)	7
Tip Elevation (ft)	-67.7
Embedded Length (ft)	53.7
Soil type	Sand/lime
Casing length (ft)	50
Method used (ft)	Wet

Hillsborough Bridge

Name	4-14, Hillsborough
Type	Osterberg & Statnamic
Station No	1387+05.73, 9.1LT
Nearest boring	P4-S14
Length (ft)	70.83
Diameter (in)	48
Date of Test	6/26/96
Water Level (ft)	No info
Ground Level (ft)	3
Bottom of casing	-41.5
Last strain gauge Elevation	-52
Top of Rock Elevation	-34
Test Rock Socket (ft)	18
Total Rock Socket	28.83
Top Elevation (ft)	8
Tip Elevation (ft)	-62.83
Embedded Length (ft)	65.83
Soil type	Sand/lime
Casing length (ft)	40
Method used	Wet

Name	5-10, Hillsborough
Type	Statnamic Test
Station No	1388+03.73, 2'-3"LT
Nearest boring	P5-5, P5-15
Length (ft)	75.33
Diameter (in)	48
Date of Test	6/30/96
Water Level (ft)	No info
Ground Level (ft)	-6.86
Bottom of casing	-45.33
Last strain gauge Elevation	-64.5
Top of Rock Elevation	-35
Test Rock Socket (ft)	19.17
Total Rock Socket	22
Top Elevation (ft)	8
Tip Elevation (ft)	-67.33
Embedded Length (ft)	60.47
Soil type	Sand/lime
Casing length (ft)	53.33
Method used	Wet

Macarthur Bridge

Name	Macarthur
Type	Conventional test
Station No	1069+06, 82.24' RT
Nearest boring	B-34, B-35
Length (ft)	93
Diameter (in)	30
Date of Test	10/13/93
Water Level (ft)	0
Ground Level (ft)	5.69
Bottom of casing	-35
Last strain gauge Elevation	-81.83
Top of Rock Elevation	-69
Test Rock Socket (ft)	12.83
Total Rock Socket	16
Top Elevation (ft)	8
Tip Elevation (ft)	-85
Embedded Length (ft)	90.69
Soil type	Sand/lime
Casing length (ft)	43
Method used (ft)	Wet slurry

Name	3-1, Victory
Type	Osterberg Test
Station No	90+15.605, 12.25LT
Nearest boring	TB-3
Length (ft)	33.2
Diameter (in)	48
Date of Test	1/5/95
Water Level (ft)	56
Ground Level (ft)	54.56
Bottom of casing	38
Last strain gauge Elevation	24.05
Top of Rock Elevation	38.56
Test Rock Socket (ft)	14.51
Total Rock Socket	14.76
Top Elevation (ft)	57
Tip Elevation (ft)	24.1
Embedded Length (ft)	30.76
Soil type	Sand/rock
Casing length (ft)	21
Method used (ft)	Wet (Clean water)

Victory Bridge

Name	3-2, Victory
Type	Osterberg Test
Station No	90+15.605, 12.25RT
Nearest boring	TB-3
Length (ft)	38.56
Diameter (in)	48
Date of Test	12/15/94
Water Level (ft)	47
Ground Level (ft)	54.4
Bottom of casing	37.37
Last strain gauge Elevation	28.1
Top of Rock Elevation	39.4
Test Rock Socket (ft)	9.27
Total Rock Socket	18.93
Top Elevation (ft)	57
Tip Elevation (ft)	18.44
Embedded Length (ft)	35.96
Soil type	Sandy clay/rock
Casing length (ft)	22.23(permanent casing)
Method used (ft)	Wet (Clean water)

Name	10-2, Victory
Type	Osterberg Test
Station No	99+31.949, 12.25RT
Nearest boring	TB-8, 9
Length (ft)	46.64
Diameter (in)	48
Date of Test	8/9/95
Water Level (ft)	51
Ground Level (ft)	51
Bottom of casing	31.42
Last strain gauge Elevation	2.8
Top of Rock Elevation	9.98
Test Rock Socket (ft)	7.18
Total Rock Socket	20.68
Top Elevation (ft)	57.34
Tip Elevation (ft)	-10.7
Embedded Length (ft)	61.7
Soil type	Sandy clay/rock
Casing length (ft)	25.92

Victory Bridge

Name	19-1, Victory
Type	Osterberg Test
Station No	111+10.105, 12.25LT
Nearest boring	TB-18
Length (ft)	45.03
Diameter (in)	48
Date of Test	1/3/95
Water Level (ft)	50
Ground Level (ft)	55.8
Bottom of casing	28.64
Last strain gauge Elevation	12.1
Top of Rock Elevation	25.71
Test Rock Socket (ft)	13.61
Total Rock Socket	13.57
Top Elevation (ft)	57
Tip Elevation (ft)	12.14
Embedded Length (ft)	43.66
Soil type	Sand/rock
Casing length (ft)	29
Method used (ft)	Wet (Clean water)

Name	19-2, Victory
Type	Osterberg Test
Station No	111+10.105, 12.25RT
Nearest boring	TB-18
Length (ft)	50.73
Diameter (in)	48
Date of Test	12/6/94
Water Level (ft)	47
Ground Level (ft)	55.76
Bottom of casing	28.64
Last strain gauge Elevation	17.9
Top of Rock Elevation	25.71
Test Rock Socket (ft)	7.81
Total Rock Socket	19.95
Top Elevation (ft)	55.76
Tip Elevation (ft)	5.76
Embedded Length (ft)	50
Soil type	Sandy clay / rock
Casing length (ft)	28.42
Method used (ft)	Wet (Clean water)

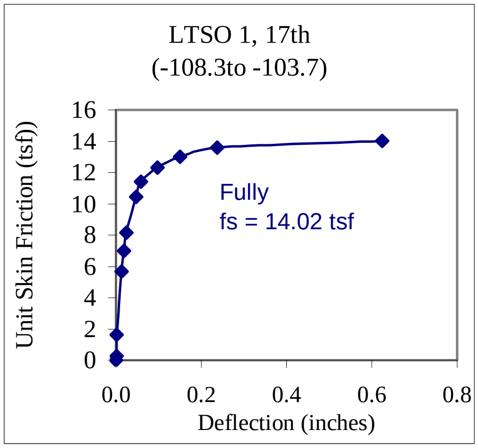
Victory Bridge

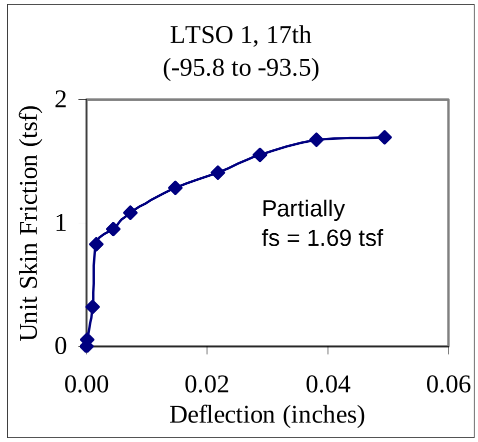
Name	Shaft 5, Victory
Type	Statnamic test
Station No	111+00.105
Nearest boring	TB-18
Length (ft)	43.12
Diameter (in)	48
Date of Test	11/23/94
Water Level (ft)	50
Mud Level (ft)	55.46
Bottom of casing	No info
Last strain gauge Elevation	19
Top of Rock Elevation	31
Test Rock Socket (ft)	12
Total Rock Socket	15.01
Top Elevation (ft)	59.11
Tip Elevation (ft)	15.99
Embedded Length (ft)	39.47
Soil type	Sandy clay/rock
Casing length (ft)	22.23
Method used (ft)	Wet (Clean water)

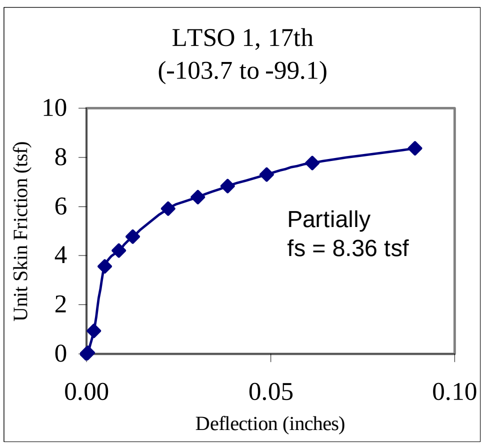
APPENDIX B

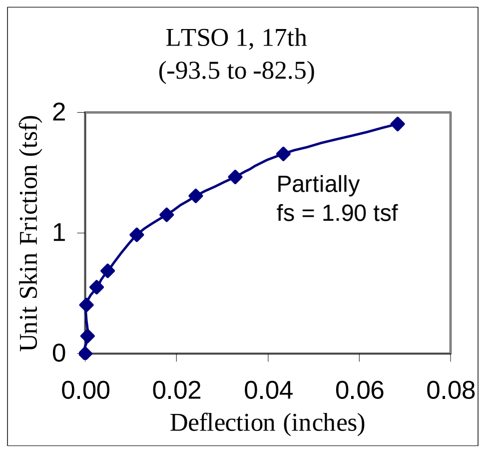
COMPUTED T-Z (AXIAL) CURVES in FLORIDA LIMESTONE

LTSO 1 (Osterberg), 17th Street Bridge

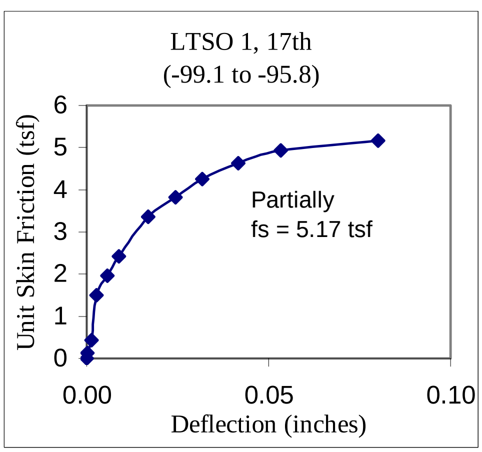


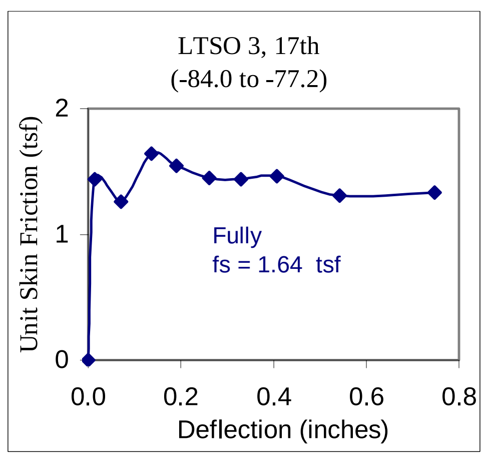




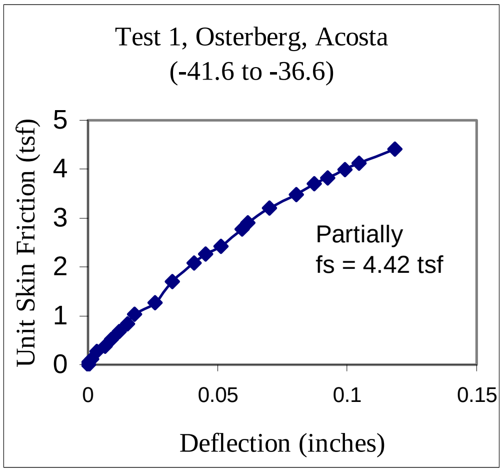


LTSO 3 (Osterberg), 17th Street Bridge

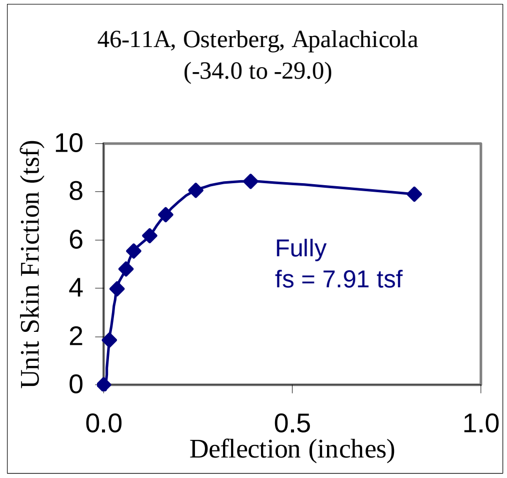




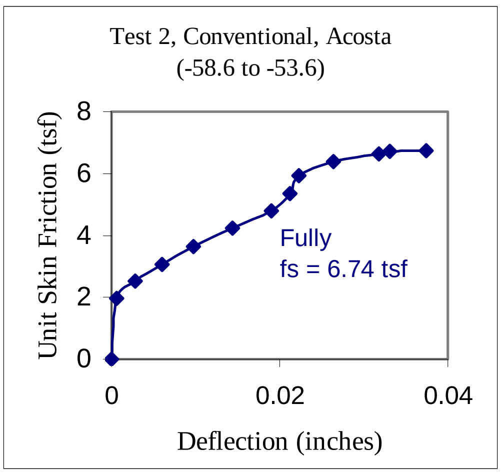
Test 1 (Osterberg), Acosta Bridge

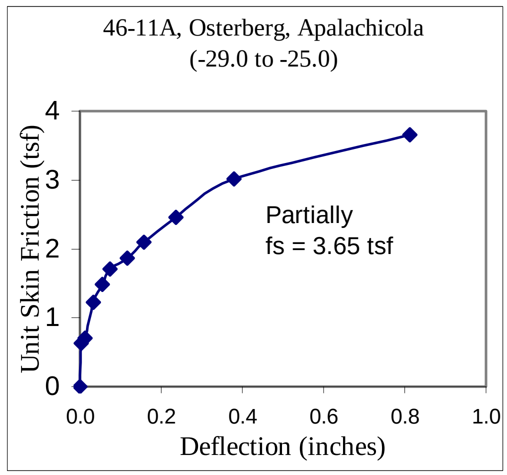


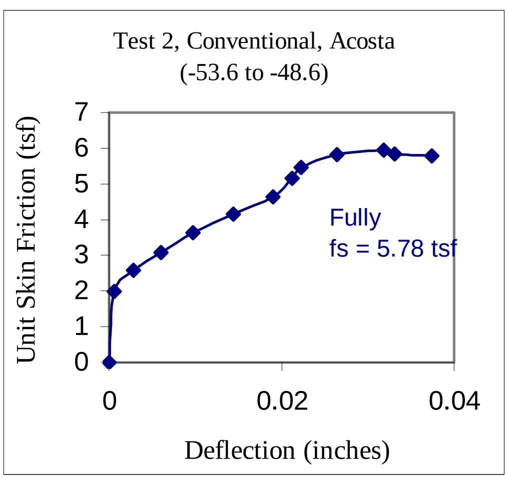
46-11A (Osterberg), Apalachicola Bridge

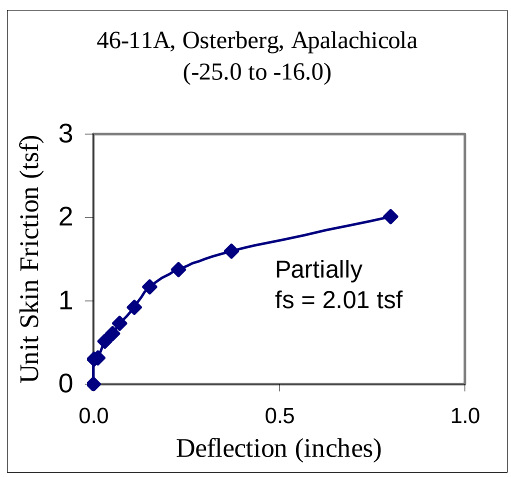


Test 2 (Conventional), Acosta Bridge

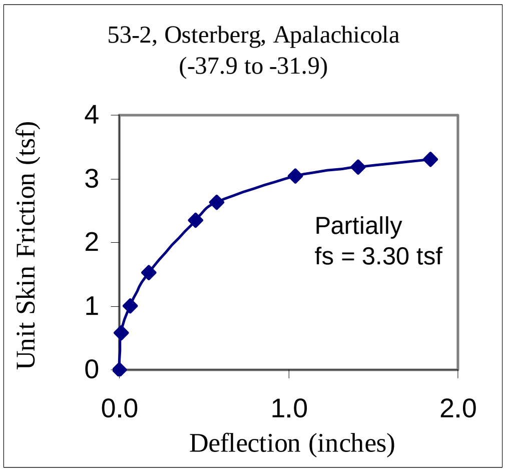




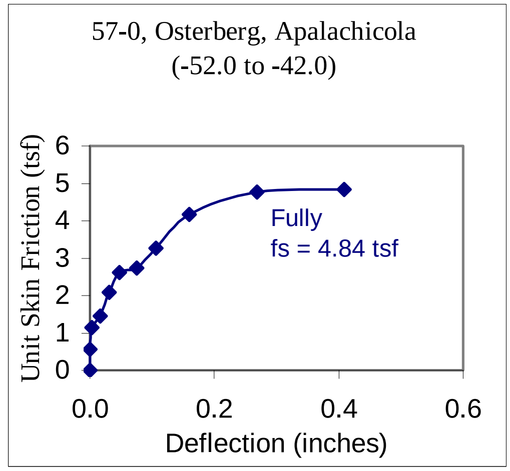


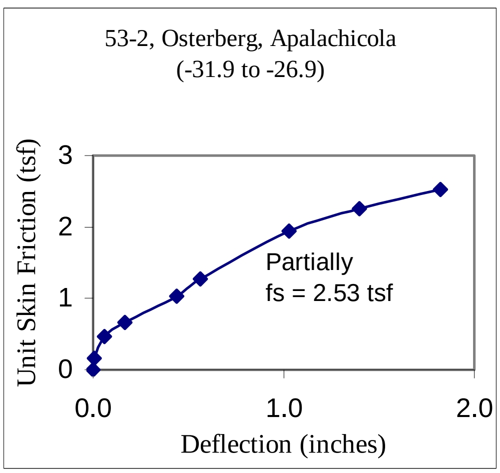


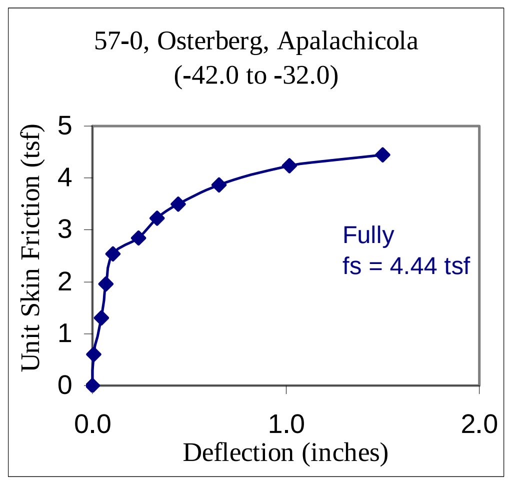
53-2 (Osterberg), Apalachicola Bridge

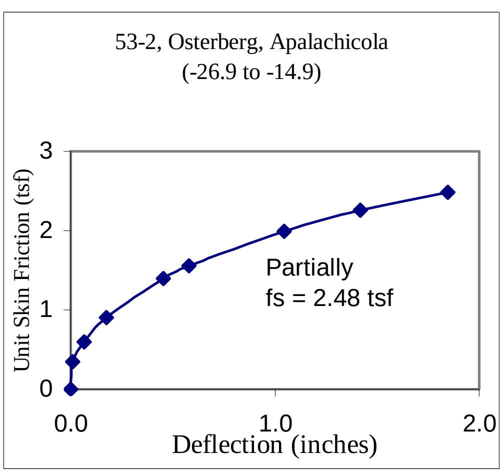


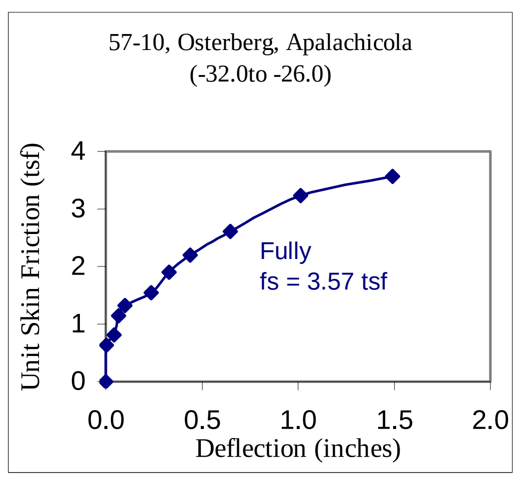
57-10 (Osterberg), Apalachicola Bridge

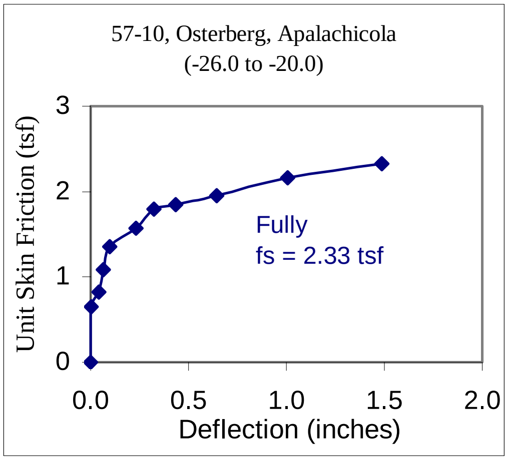


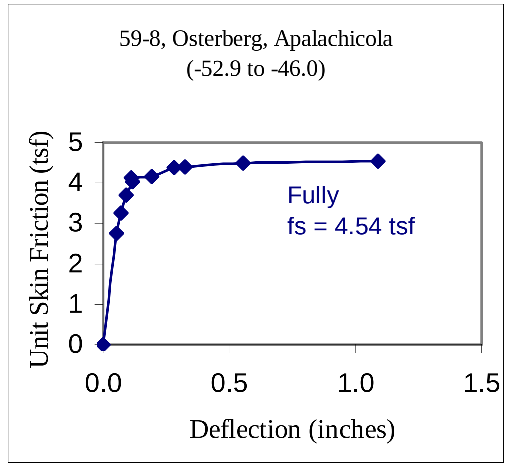




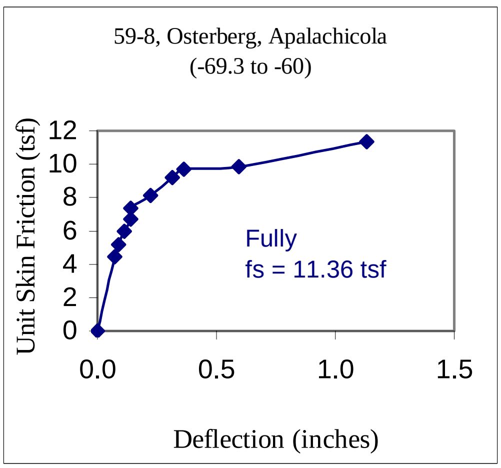


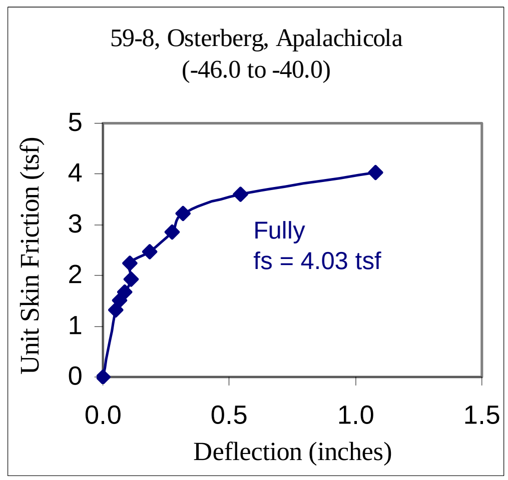


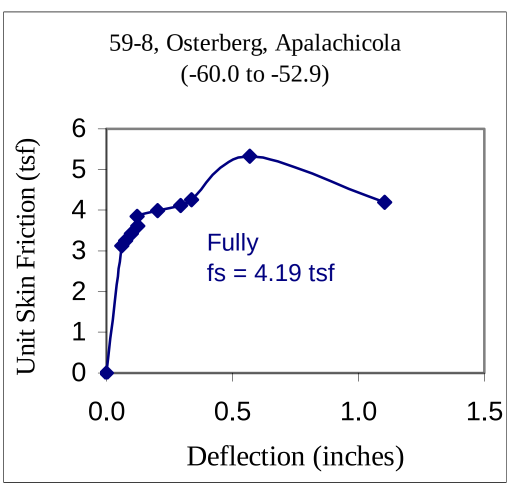


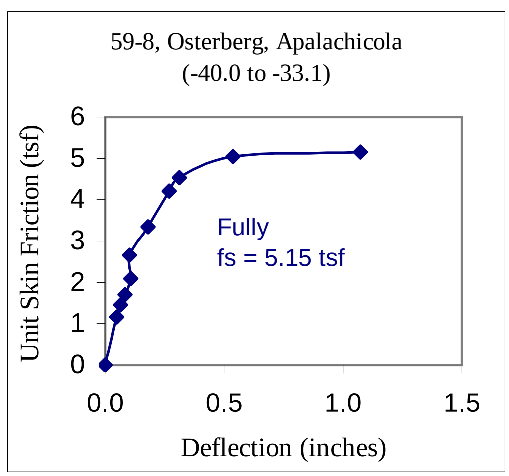


59-8 (Osterberg), Apalachicola Bridge

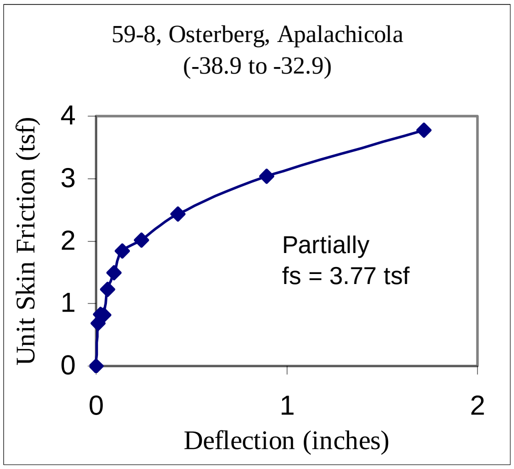


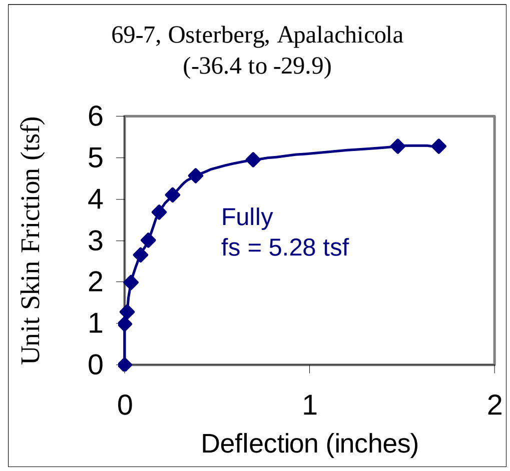




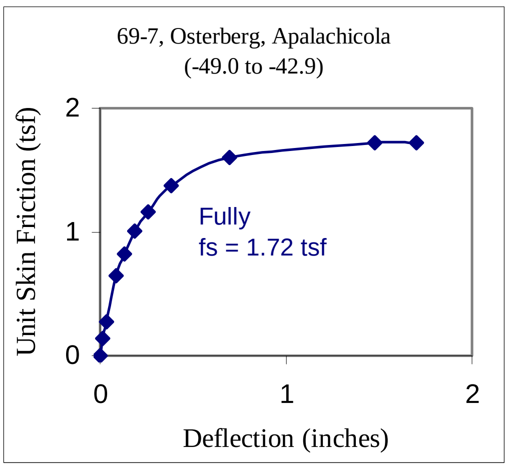


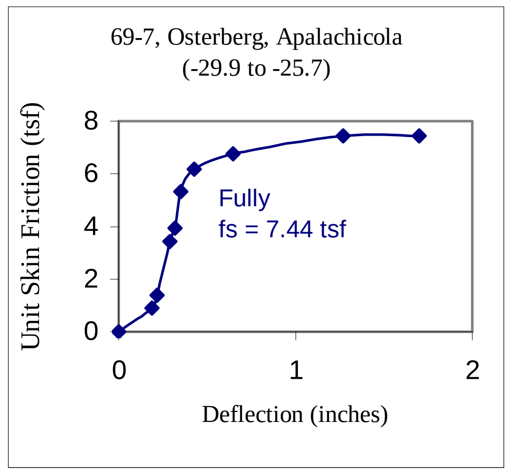
62-5 (Osterberg), Apalachicola Bridge

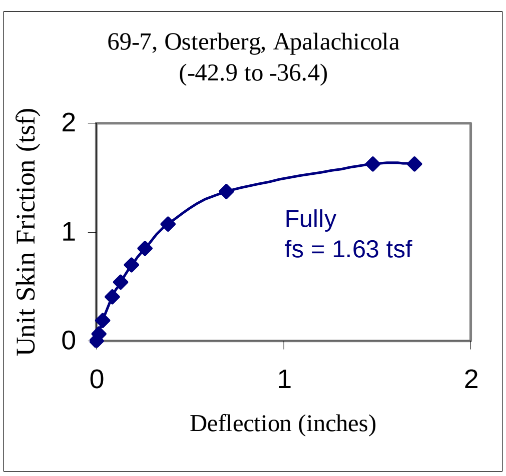




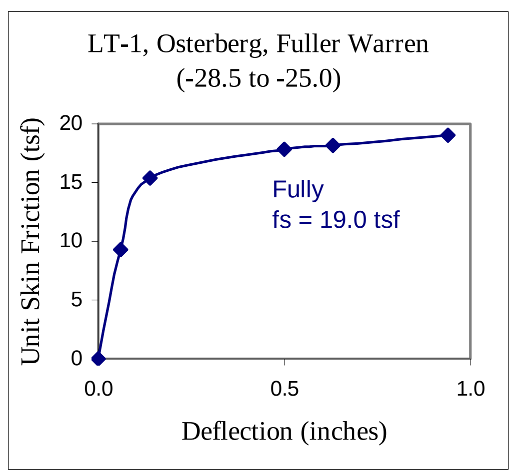
69-7 (Osterberg), Apalachicola Bridge



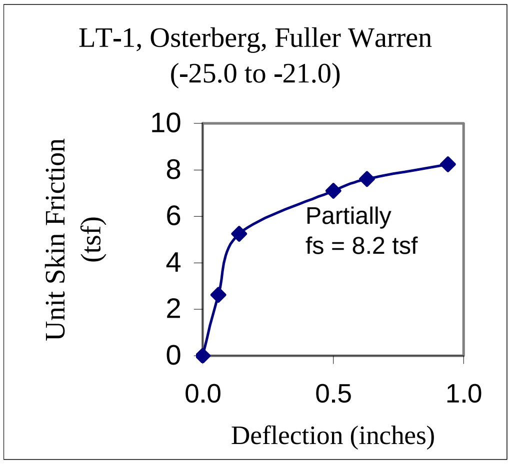


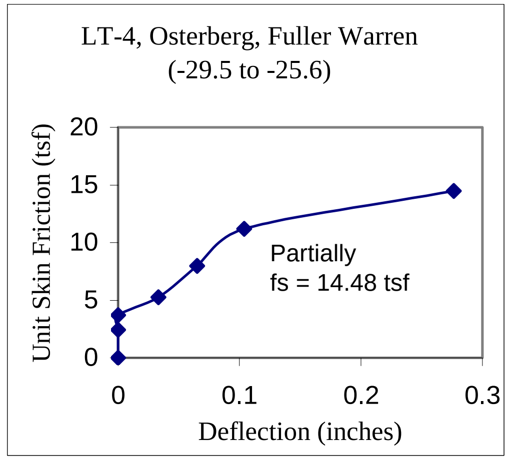


LT-1 (Osterberg), Fuller Warren Bridge

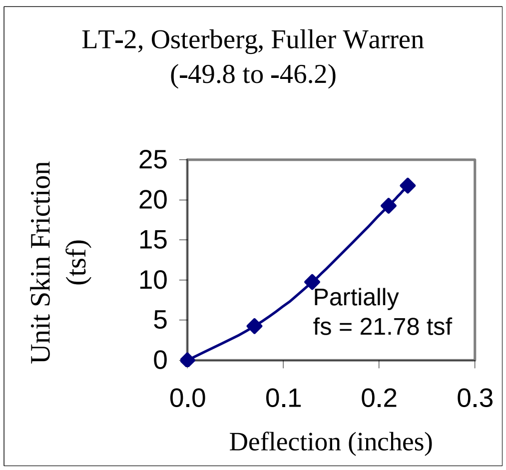


LT-4 (Osterberg), Fuller Warren Bridge

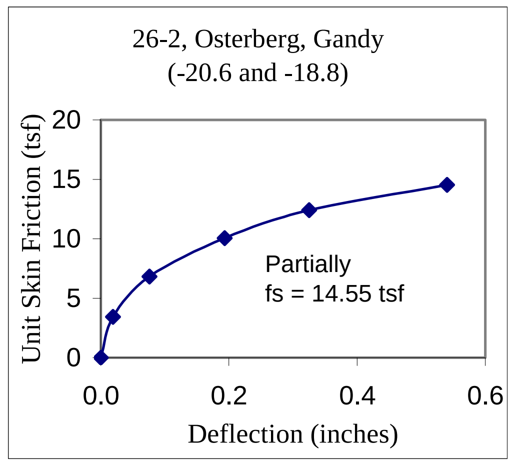


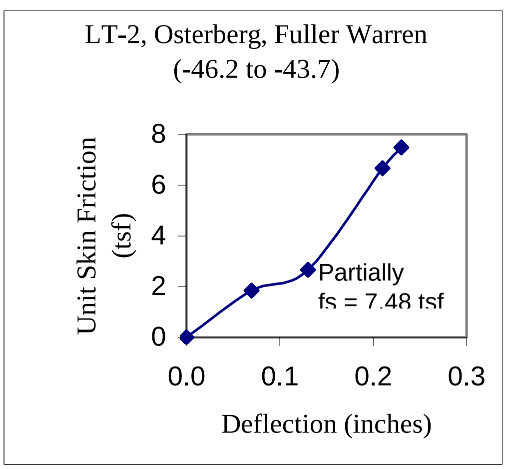


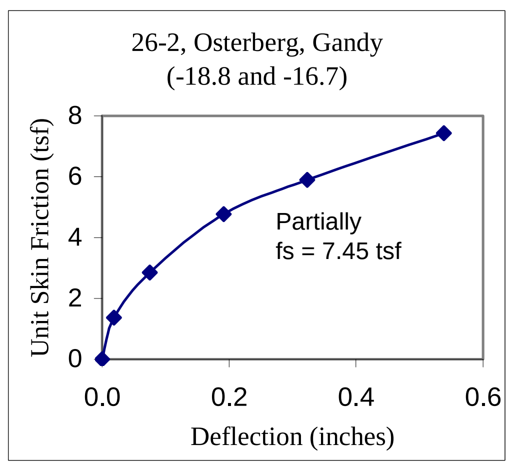
LT-2 (Osterberg), Fuller Warren Bridge



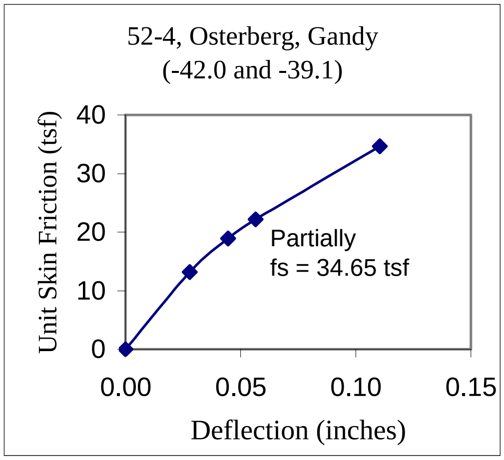
26-2 (Osterberg), Gandy Bridge

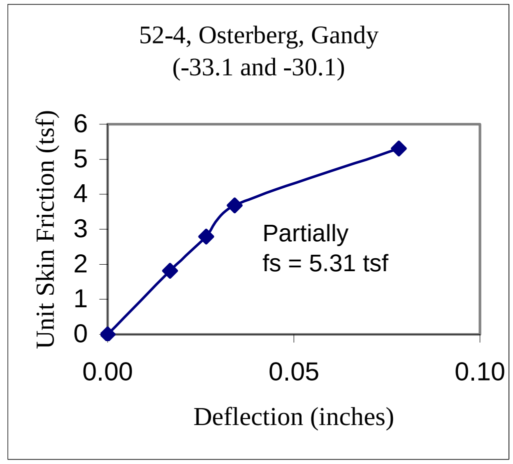


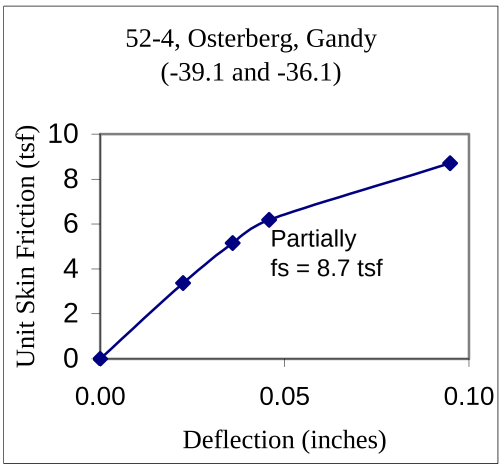


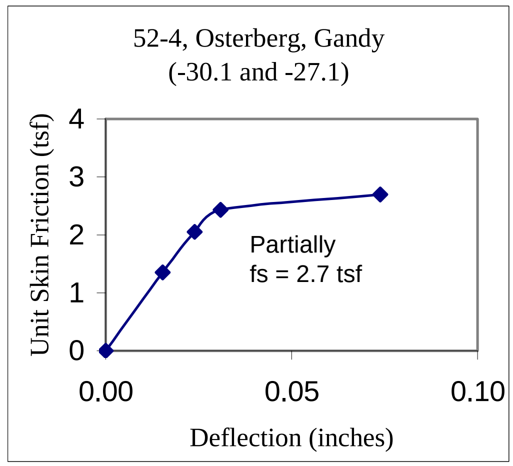


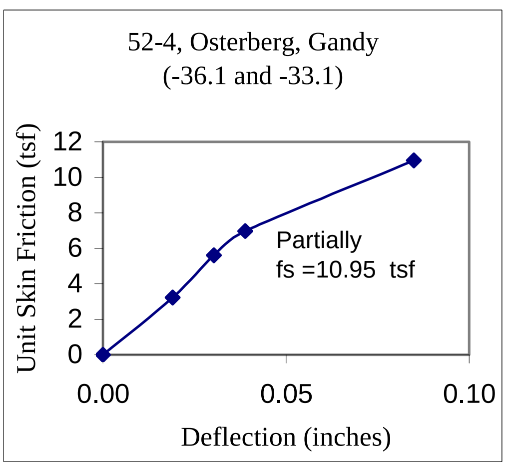
52-4 (Osterberg), Gandy Bridge

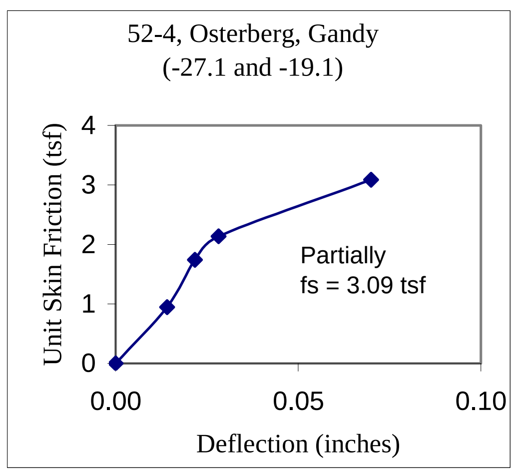




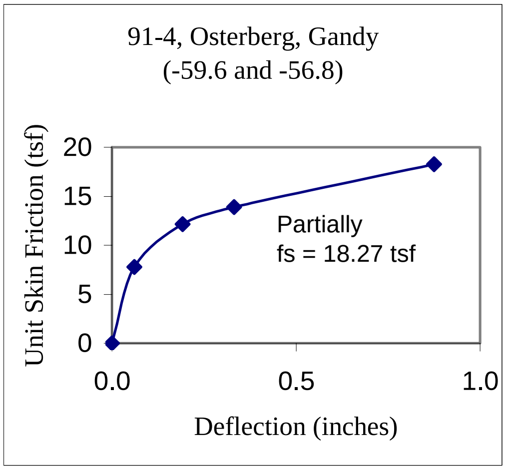


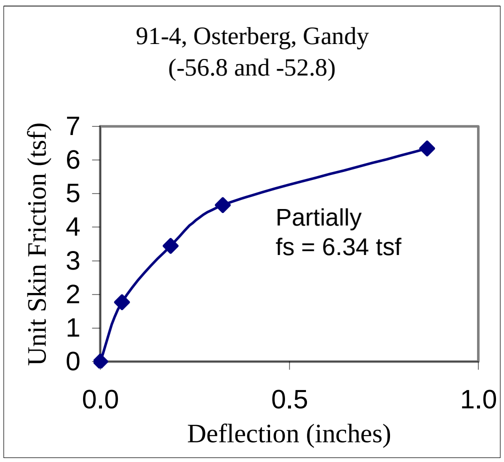


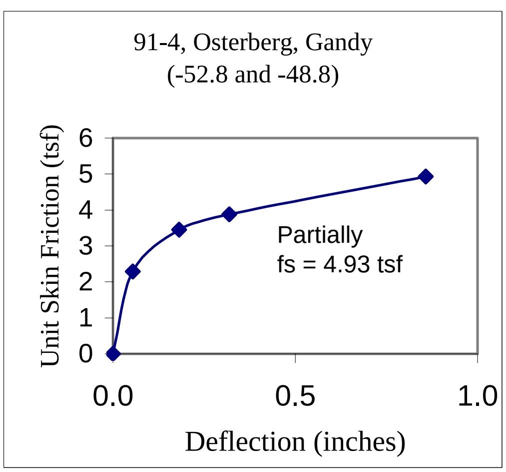




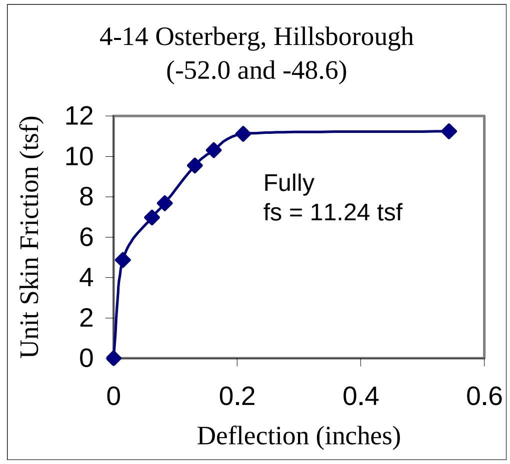
91-4 (Osterberg), Gandy Bridge

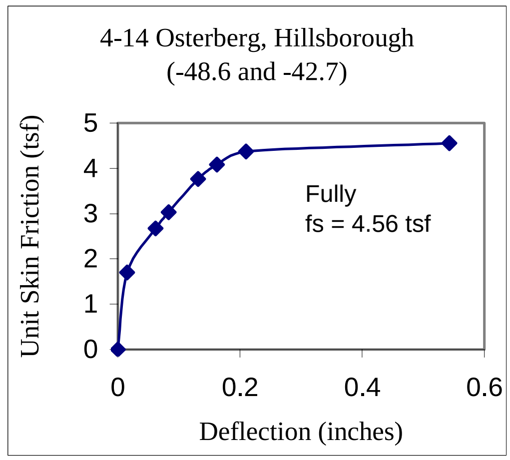




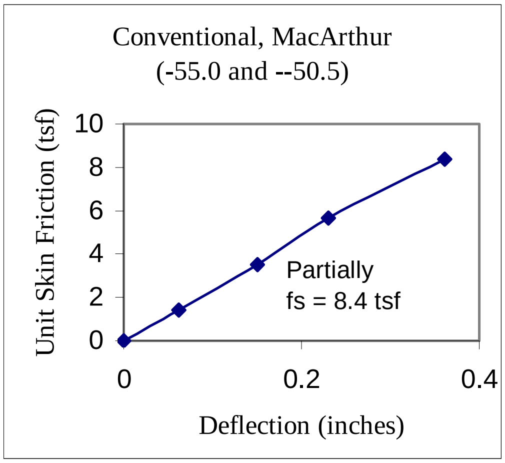


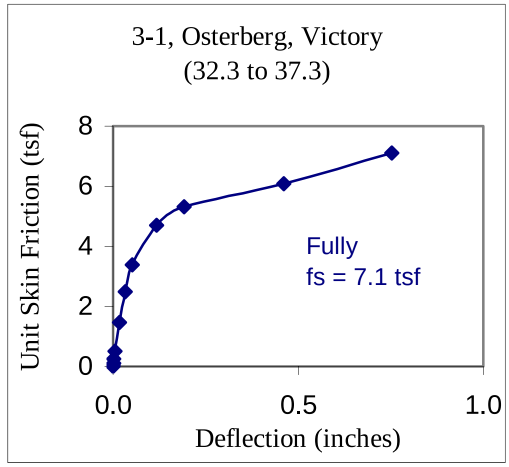
4-14 (Osterberg), Hillsborough Bridge



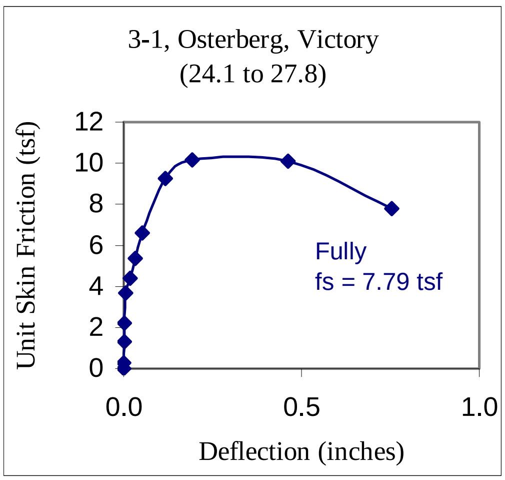


Conventional, MacArthur Bridge

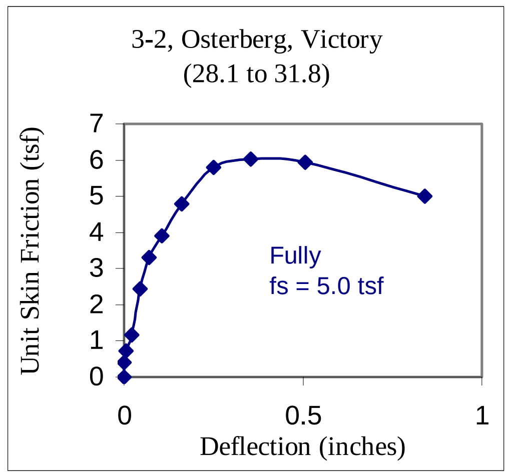


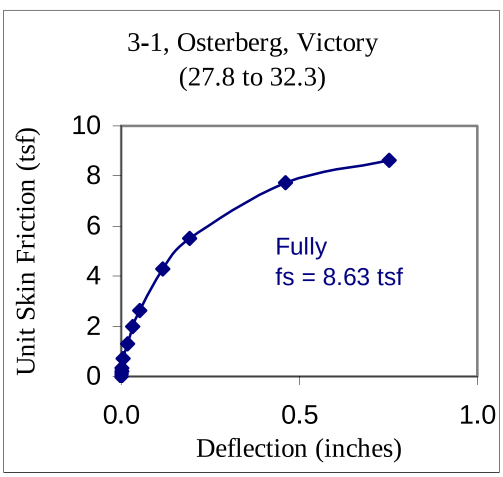


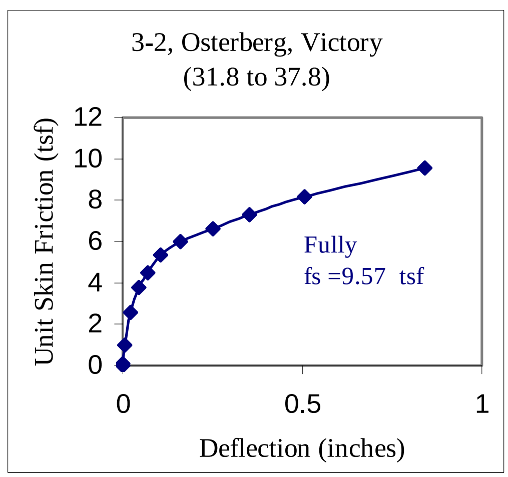
3-1 (Osterberg), Victory Bridge



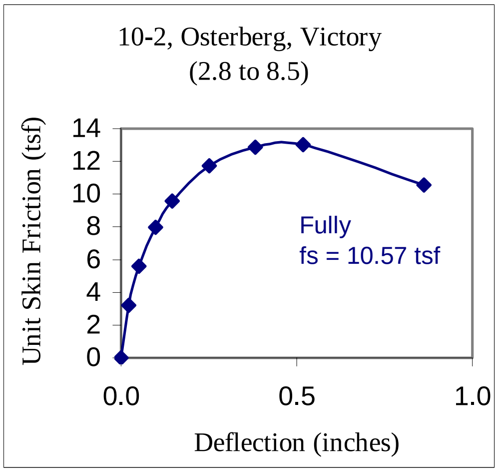
3-2 (Osterberg), Victory Bridge

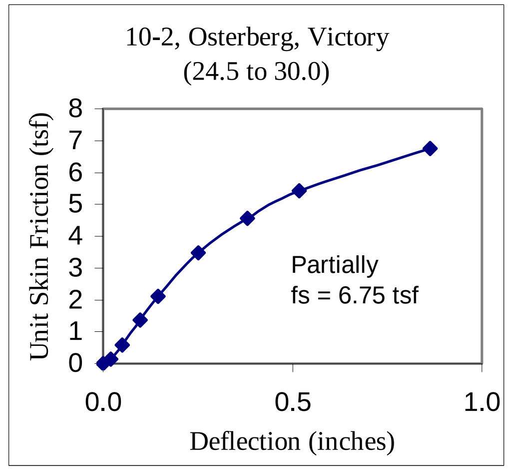


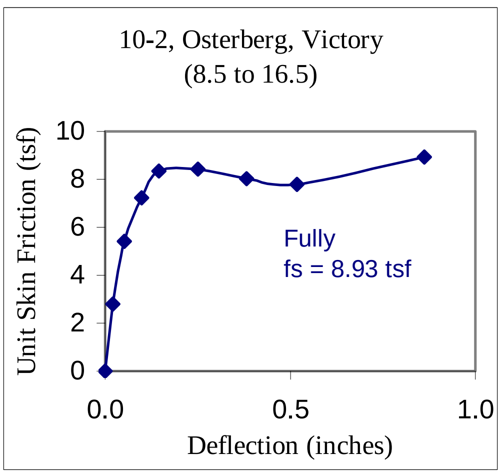




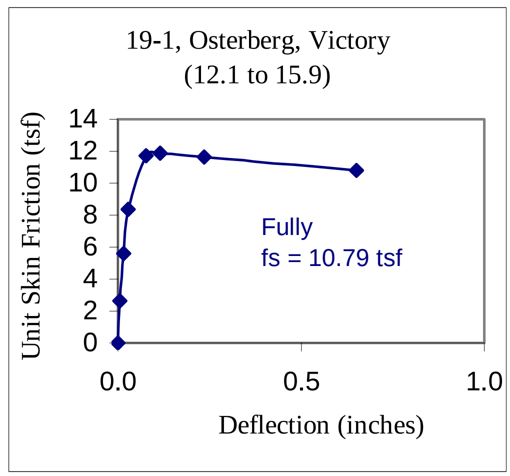
10-2 (Osterberg), Victory Bridge

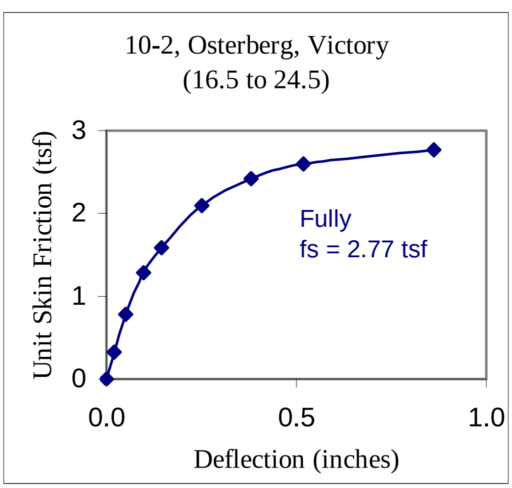


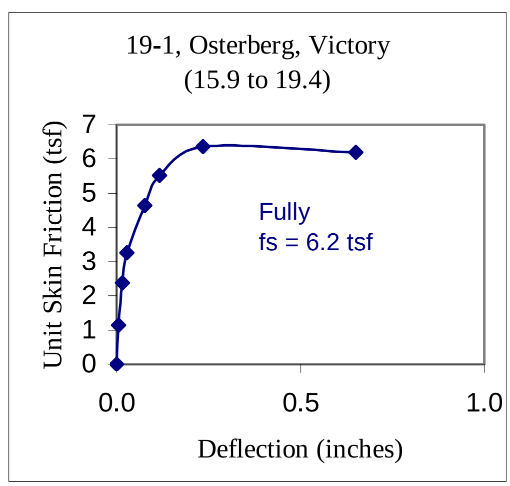


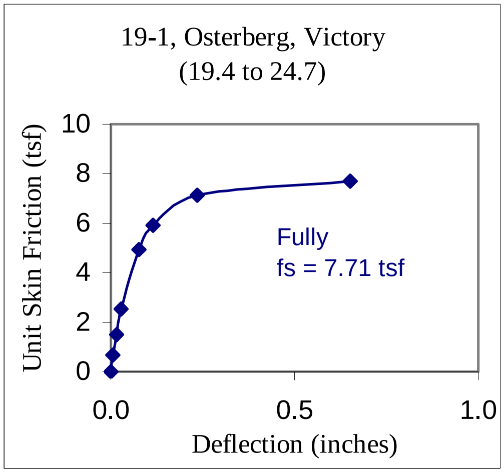


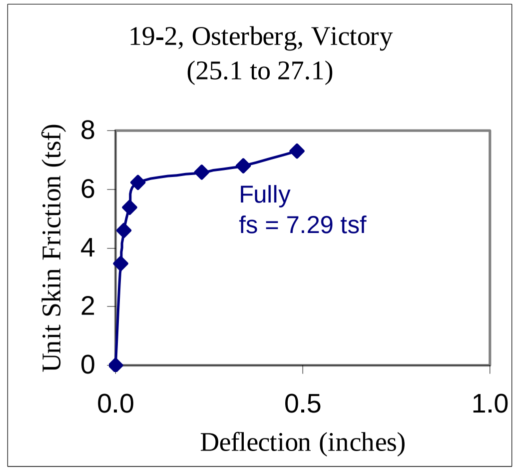
19-1 (Osterberg), Victory Bridge



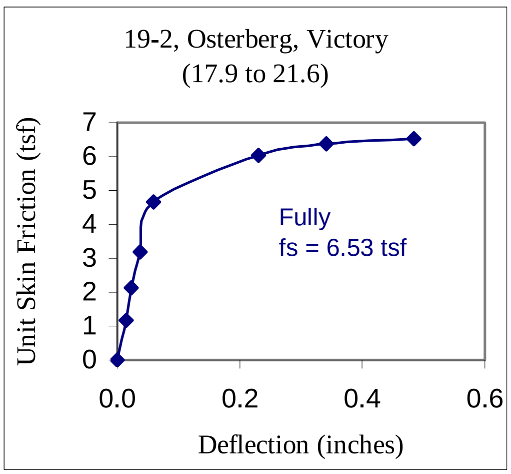


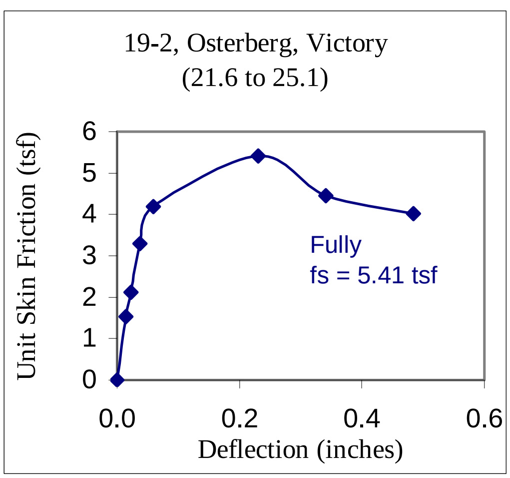






19-2 (Osterberg), Victory Bridge





ULTIMATE or MOBILIZED UNIT SKIN FRICTION
in ROCK SOCKETS with DEPTH

Unit Skin Friction along the Rock Socket, 17th Street Bridge

LTSO 1 (Hard), 17th Street Bridge				
Osterberg (Rock layers)				
Elevation			Skin friction	
From(ft)	To(ft)	Length(ft)	fs(tsf)	Status
-108.3	-103.7	4.6	14.02	fully
-103.7	-99.1	4.6	8.36	partially
-99.1	-95.8	3.3	5.17	partially
-95.8	-93.5	2.3	1.69	partially
-93.5	-82.0	11.5	1.90	partially

LTSO 1 (Hard), 17th Street Bridge				
Statnamic (Rock layers)				
Elevation			Skin friction	
From(ft)	To(ft)	Length(ft)	fs(tsf)	Status
-108.2	-95.8	12.5	5.43	partially
-95.8	-93.5	2.3	5.90	partially
-93.5	-82.0	11.5	7.19	partially

LTSO 2 (Hard), 17th Street Bridge				
Statnamic (Rock layers)				
Elevation			Skin friction	
From(ft)	To(ft)	Length(ft)	fs(tsf)	Status
-121.4	-113.5	7.9	1.24	partially
-113.5	-106.6	6.9	4.43	partially
-106.6	-98.4	8.2	11.21	partially
-98.4	-90.2	8.2	6.81	partially
-90.2	-77.1	13.1	4.02	partially

LTSO 3 (Soft), 17th Street Bridge				
Osterberg(Rock layers)				
Elevation			Skin friction	
From(ft)	To(ft)	Length(ft)	fs(tsf)	Status
-84.0	-77.2	6.8	1.64	fully

LTSO 3 (Soft), 17th Street Bridge				
Statnamic (Rock layers)				
Elevation			Skin friction	
From(ft)	To(ft)	Length(ft)	fs(tsf)	Status
-95.1	-83.9	11.2	0.99	partially

LT 1 (Soft), 17th Street Bridge				
Statnamic (Rock layers)				
Elevation			Skin friction	
From(ft)	To(ft)	Length(ft)	fs(tsf)	Status
-60.0	-55.8	4.3	1.29	partially
-55.8	-13.6	42.1	2.22	partially
-13.6	-5.9	7.7	2.73	partially
-5.9	9.8	15.7	1.43	partially

LT 2 (Hard), 17th Street Bridge				
Statnamic (Rock layers)				
Elevation			Skin friction	
From(ft)	To(ft)	Length(ft)	fs(tsf)	Status
-53.7	-46.7	6.9	10.37	partially
-46.7	-35.9	10.8	9.25	partially

Unit Skin Friction along the Rock Socket, Acosta Bridge

Test 1, Acosta Bridge				
Osterberg (Rock layers)				
Elevation			Skin friction	
From (ft)	To (ft)	Length (ft)	fs (tsf)	Status
-41.6	-36.6	5.0	4.42	partially

Test 2, Acosta Bridge				
Conventional (Rock layers)				
Elevation			Skin friction	
From (ft)	To (ft)	Length (ft)	fs (tsf)	Status
-58.6	-53.6	5.0	6.74	fully
-53.6	-48.6	5.0	5.78	fully

Unit Skin Friction along the Rock Socket, Apalachicola Bridge

46-11A, Apalachicola Bridge				
Osterberg (Rock layers)				
Elevation			Skin friction	
From (ft)	To (ft)	Length (ft)	fs (tsf)	Status
-34.0	-29.0	5.0	7.91	fully
-29.0	-25.0	4.0	3.65	partially
-25.0	-16.0	9.0	2.01	partially

53-2, Apalachicola Bridge				
Osterberg (Rock layers)				
Elevation			Skin friction	
From (ft)	To (ft)	Length (ft)	fs (tsf)	Status
-37.9	-31.9	6.0	3.30	partially
-31.9	-26.9	5.0	2.53	partially
-26.9	-14.9	12.0	2.48	partially

57-10, Apalachicola Bridge				
Osterberg (Rock layers)				
Elevation			Skin friction	
From (ft)	To (ft)	Length (ft)	fs (tsf)	Status
-52.0	-42.0	10.0	4.84	fully
-42.0	-32.0	10.0	4.44	fully
-32.0	-26.0	6.0	3.57	fully
-26.0	-20.0	6.0	2.33	fully

59-8, Apalachicola Bridge				
Osterberg (Rock layers)				
Elevation			Skin friction	
From (ft)	To (ft)	Length (ft)	fs (tsf)	Status
-69.3	-60.0	9.3	11.36	fully
-60.0	-52.9	7.1	4.19	fully
-52.9	-46.0	6.9	4.54	fully
-46.0	-40.0	6.0	4.03	fully
-40.0	-33.1	6.9	5.15	fully

62-5, Apalachicola Bridge				
Osterberg (Rock layers)				
Elevation			Skin friction	
From (ft)	To (ft)	Length (ft)	fs (tsf)	Status
-38.9	-32.9	6.0	3.77	partially

69-7, Apalachicola Bridge				
Osterberg (Rock layers)				
Elevation			Skin friction	
From (ft)	To (ft)	Length (ft)	fs (tsf)	Status
-49.0	-42.9	6.1	1.72	fully
-42.9	-36.4	6.5	1.63	fully
-36.4	-29.9	6.5	5.28	fully
-29.9	-25.7	4.2	7.44	fully

Unit Skin Friction along the Rock Socket, Fuller Warren Bridge

LT -1, Fuller Warren Bridge				
Osterberg (Rock layers)				
Elevation			Skin friction	
From (ft)	To (ft)	Length (ft)	fs (tsf)	Status
-28.5	-25.0	3.5	19.00	fully
-25.0	-21.0	4.0	8.20	partially

LT-2, Fuller Warren Bridge				
Osterberg (Rock layers)				
Elevation			Skin friction	
From (ft)	To (ft)	Length (ft)	fs (tsf)	Status
-49.8	-46.2	3.6	21.78	partially
-46.2	-43.7	2.5	7.48	partially

LT -4, Fuller Warren Bridge				
Osterberg (Rock layers)				
Elevation			Skin friction	
From (ft)	To (ft)	Length (ft)	fs (tsf)	Status
-29.5	-25.6	3.9	14.48	partially

Unit Skin Friction along the Rock Socket, Gandy

26-1, Gandy Bridge				
Statnamic (Rock layers)				
Elevation			Skin friction	
From (ft)	To (ft)	Length (ft)	fs (tsf)	Status
-24.7	-24.2	0.5	7.50	partially
-24.2	-21.2	3.0	7.38	partially
-21.2	-17.9	3.3	7.38	partially

26-2, Gandy Bridge				
Osterberg (Rock layers)				
Elevation			Skin friction	
From (ft)	To (ft)	Length (ft)	fs (tsf)	Status
-20.6	-18.8	1.8	14.55	partially
-18.8	-16.7	2.1	7.45	partially

52-3, Gandy Bridge				
Statnamic (Rock layers)				
Elevation			Skin friction	
From (ft)	To (ft)	Length (ft)	fs (tsf)	Status
-46.3	-39.5	6.8	8.40	partially
-39.5	-36.5	3.0	15.13	partially
-36.5	-33.5	3.0	10.03	partially
-33.5	-30.5	3.0	7.64	partially
-30.5	-27.5	3.0	8.51	partially
-27.5	-19.5	8.0	6.35	partially

52-4, Gandy Bridge				
Osterberg (Rock layers)				
Elevation			Skin friction	
From (ft)	To (ft)	Length (ft)	fs (tsf)	Status
-42.0	-39.1	2.9	34.65	partially
-39.1	-36.1	3.0	8.7	partially
-36.1	-33.1	3.0	10.95	partially
-33.1	-30.1	3.0	5.31	partially
-30.1	-27.1	3.0	2.7	partially
-27.1	-19.1	8.0	3.09	partially

91-4, Gandy Bridge				
Osterberg (Rock layers)				
Elevation			Skin friction	
From (ft)	To (ft)	Length (ft)	fs (tsf)	Status
-59.6	-56.8	2.8	18.27	partially
-56.8	-52.8	4.0	6.34	partially
-52.8	-48.8	4.0	4.93	partially

Unit Skin Friction along the Rock Socket, Hillsborough

Pier 4 Shaft 14, Hillsborough Bridge				
Osterberg (Rock layers)				
Elevation			Skin friction	
From (ft)	To (ft)	Length (ft)	fs (tsf)	Status
-52.0	-48.6	3.4	11.24	fully
-48.6	-42.7	5.9	4.56	fully

Pier 4 Shaft 14, Hillsborough Bridge				
Statnamic (Rock layers)				
Elevation			Skin friction	
From (ft)	To (ft)	Length (ft)	fs (tsf)	Status
-55.5	-48.6	6.9	15.01	fully
-48.6	-42.7	5.9	4.56	fully

Pier 5 Shaft 10, Hillsborough Bridge				
Statnamic (Rock layers)				
Elevation			Skin friction	
From (ft)	To (ft)	Length (ft)	fs (tsf)	Status
-64.5	-60.5	4.0	7.31	partially
-60.5	-56.5	4.0	1.38	partially
-56.5	-52.5	4.0	8.47	partially
-52.5	-48.5	4.0	17.11	partially

Unit Skin Friction along the Rock Socket, MacArthur

MacArthur Bridge				
Conventional (Rock layers)				
Elevation			Skin friction	
From (ft)	To (ft)	Length (ft)	fs (tsf)	Status
-55.0	-50.5	4.5	8.40	partially

Unit Skin Friction along the Rock Socket, Victory

Shaft 3-1, Victory Bridge			
Osterberg (Rock layers)			
Elevation		Skin friction	
To (ft)	Length (ft)	fs (tsf)	Status
27.8	3.7	7.79	fully
32.3	4.5	8.63	fully
37.3	5.0	7.10	fully

Shaft 3-2, Victory Bridge			
Osterberg (Rock layers)			
Elevation		Skin friction	
To (ft)	Length (ft)	fs (tsf)	Status
31.8	3.7	5.00	fully
37.8	6.0	9.57	fully

Shaft 10-2, Victory Bridge			
Osterberg (Rock layers)			
Elevation		Skin friction	
To (ft)	Length (ft)	fs (tsf)	Status
8.5	5.7	10.57	fully
16.5	8.0	8.93	fully
24.5	8.0	2.77	fully
30.0	5.5	6.75	partially

Shaft 19-1, Victory Bridge			
Osterberg (Rock layers)			
Elevation		Skin friction	
To (ft)	Length (ft)	fs (tsf)	Status
15.9	3.8	10.79	fully
19.4	3.5	6.20	fully
24.7	5.4	7.71	fully

Shaft 19-2, Victory Bridge				
Osterberg (Rock layers)				
Elevation			Skin friction	
From (ft)	To (ft)	Length (ft)	fs (tsf)	Status
17.9	21.6	3.7	6.53	fully
21.6	25.1	3.5	5.41	fully
25.1	27.1	2.0	7.29	fully

TH 5, Victory Bridge				
Statnamic (Rock layers)				
Elevation			Skin friction	
From(ft)	To(ft)	Length(ft)	fs(tsf)	Status
19.0	26.0	7.0	18.50	partially

APPENDIX C

SHAFT, SOIL, and ROCK INFORMATION for LATERAL LOAD PREDICTIONS

59-8 (Apalachicola)

Shaft Length	134.0 ft
Shaft Diameter	9 ft
Top of shaft	55.5 ft
Loading elevation	53 ft
Top of ground	-5 ft
Top of rock	-20 ft
Tip of shaft	-78.5 ft
Max load	325 tons
Max Movement	12.24 in

Cross section info
36 #18 Bars
#4 Spiral
6" Cover
#18 - Diameter (2.256")
#18 - Area ($4in^2$)
#4 - Diameter (0.5")
#4 - Area ($0.2in^2$)

Soil parameters

(Medium Dense Sand)

Layer elevation (ft)	-5 ~ -20
ϕ	34 ^{o}
γ_s	120 pcf
Subgrade Modulus	50 lb/in^3

Cased Shaft	
Elevation (ft)	53 ~ -58
Length (ft)	111
Unit Weight (pcf)	150
Mild Steel's Yield stress	60 ksi
Mild Steel's Modulus	29000 ksi
Concrete's f'c	5 ksi
Concrete's Modulus	4070 ksi
Shell Thickness (in)	0.5
Shell's Yield Stress	60 ksi
Shell's Modulus	30000 ksi

Rock parameters

(Limestone)

Layer elevation (ft)	-20 ~ continue
C_u	18000 psf
γ_s	130 psf
E_{50}	0.0005

Ave Qu 20 tsf

Ave Qt 2.3 tsf

Uncased Shaft	
Elevation (ft)	-58 ~ -78.5
Length (ft)	20.5
Unit Weight (pcf)	150
Mild Steel's Yield stress	60 ksi
Mild Steel's Modulus	29000 ksi
Concrete's f'c	5 ksi
Concrete's Modulus	4070 ksi

LLT-1 (Fuller Warren)

Shaft Length	114.5 ft
Shaft Diameter	6 ft
Top of shaft	8 ft
Loading elevation	6 ft
Top of ground	-68.5 ft
Top of rock	-92 ft
Tip of shaft	-106.5 ft
Max load	36.5 tons
Max Movement	8.5 in

Cross section info
20 #14 Bars
#4 Spiral
6" Cover
#14 - Diameter (1.693")
#14 - Area (2.25in^2)
#4 - Diameter (0.5")
#4 - Area (0.2in^2)

Soil parameters

(Dense to Very Dense Clay)

Layer elevation (ft)	-68.5 ~ -92
C	20000 psf
γ_s	130 pcf
E_{50}	0.0002

Uncased Shaft	
Elevation (ft)	6 ~ -106
Length (ft)	112
Unit Weight (pcf)	150
Mild Steel's Yield stress	60 ksi
Mild Steel's Modulus	29000 ksi
Concrete's f'c	5 ksi
Concrete's Modulus	4000 ksi

Rock parameters

(Medium dense to dense Marl)

Layer elevation (ft)	-92 ~ continue
C_u(=qu/2)	30000 psf
γ_s	130 psf
E_{50}	0.0001

Ave Qu	98 tsf
Ave Qt	11 tsf

26-1 (Gandy)

Shaft Length	33.4 ft
Shaft Diameter	4 ft
Top of shaft	8.7 ft
Loading elevation	5 ft
Top of ground	-11.5 ft
Top of rock	-16.7 ft
Tip of shaft	-24.7 ft
Max load	23.9 tons
Max Movement	0.39 in

Cross section info
20 #14 Bars
#4 Spiral
6" Cover
#14 - Diameter (1.693")
#14 - Area ($2.25in^2$)
#4 - Diameter (0.5")
#4 - Area ($0.2in^2$)

Soil parameters

(Stiff Clay)

Layer elevation(ft)	-8.5 ~ -16.7
C	7000 psf
γ_s	130 pcf
E_{50}	0.002

Cased Shaft	
Elevation (ft)	5 ~ -11.5
Length (ft)	16.5
Unit Weight (pcf)	150
Mild Steel's Yield stress	65 ksi
Mild Steel's Modulus	31000 ksi
Concrete's f'c	5 ksi
Concrete's Modulus	5000 ksi
Shell Thickness (in)	0.2
Shell's Yield Stress	60 ksi
Shell's Modulus	31000 ksi

Rock parameters

Layer elevation(ft)	-16.7 ~ continue
C_u	20000 psf
γ_s	110 psf
E_{50}	0.0009

Ave Qu	70 tsf
Ave Qt	8 tsf

Uncased Shaft	
Elevation (ft)	-11.5 ~ -30.45
Length (ft)	18.95
Unit Weight (pcf)	150
Mild Steel's Yield stress	65 ksi
Mild Steel's Modulus	31000 ksi
Concrete's f'c	5 ksi
Concrete's Modulus	5000 ksi

52-3 (Gandy)

Shaft Length	55.6 ft
Shaft Diameter	4 ft
Top of shaft	9.1 ft
Loading elevation	5 ft
Top of ground	-23 ft
Top of rock	-23 ft
Tip of shaft	-46.5 ft
Max load	60.6 tons
Max Movement	3.62 in

Cross section info
20 #14 Bars
#4 Spiral
6" Cover
#14 - Diameter (1.693")
#14 - Area (2.25in^2)
#4 - Diameter (0.5")
#4 - Area (0.2in^2)

Rock parameters

Layer elevation (ft)	-23 ~ continue
C_u	38000 psf
γ_s	110 psf
E_{50}	0.00015

Ave Qu	70 tsf
Ave Qt	8 tsf

Cased Shaft	
Elevation (ft)	5 ~ -20.33
Length (ft)	25.33
Unit Weight (pcf)	150
Mild Steel's Yield stress	60 ksi
Mild Steel's Modulus	29000 ksi
Concrete's f'c	5 ksi
Concrete's Modulus	4070 ksi
Shell Thickness (in)	0.4
Shell's Yield Stress	60 ksi
Shell's Modulus	29000 ksi

Uncased Shaft	
Elevation (ft)	-20.33 ~ -47.73
Length (ft)	27.4
Unit Weight (pcf)	150
Mild Steel's Yield stress	60 ksi
Mild Steel's Modulus	29000 ksi
Concrete's f'c	5 ksi
Concrete's Modulus	4070 ksi

91-3 (Gandy)

Shaft Length	70.7 ft
Shaft Diameter	4 ft
Top of shaft	9.08 ft
Loading elevation	5 ft
Top of ground	-41.5 ft
Top of rock	-41.5 ft
Tip of shaft	-61.6 ft
Max load	40 tons
Max Movement	3.72 in

Cross section info
20 #14 Bars
#4 Spiral
6" Cover
#14 - Diameter (1.693")
#14 - Area (2.25in^2)
#4 - Diameter (0.5")
#4 - Area (0.2in^2)

Rock parameters

Layer elevation (ft)	-40.5 ~ -80
C_u	40000 psf
γ_s	120 psf
E_{50}	0.00024

Ave Qu	70 tsf
Ave Qt	8 tsf

Cased Shaft	
Elevation (ft)	5 ~ -43
Length (ft)	48
Unit Weight (pcf)	150
Mild Steel's Yield stress	65 ksi
Mild Steel's Modulus	35000 ksi
Concrete's f'c	6 ksi
Concrete's Modulus	6000 ksi
Shell Thickness (in)	1.1
Shell's Yield Stress	60 ksi
Shell's Modulus	31000 ksi

Uncased Shaft	
Elevation (ft)	-43 ~ -61.6
Length (ft)	18.6
Unit Weight (pcf)	150
Mild Steel's Yield stress	65 ksi
Mild Steel's Modulus	35000 ksi
Concrete's f'c	6 ksi
Concrete's Modulus	6000 ksi

TH-1 (Victory)

Shaft Length	33.6 ft
Shaft Diameter	4 ft
Top of shaft	58 ft
Loading elevation	56 ft
Top of ground	53 ft
Top of rock	40 ft
Tip of shaft	24.4 ft
Max load	141.5 tons
Max Movement	2.16 in

Cross section info
20 #14 Bars
#4 Spiral
6" Cover
#14 - Diameter (1.693")
#14 - Area (2.25in^2)
#4 - Diameter (0.5")
#4 - Area (0.2in^2)

Soil parameters

(Medium Dense Sand)

Layer elevation (ft)	53 ~ 40
ϕ	23 ^{o}
γ_s	105 pcf
Subgrade Modulus	50 lb/in^3

Cased Shaft	
Elevation (ft)	56 ~ 38.3
Length (ft)	17.7
Unit Weight (pcf)	150
Mild Steel's Yield stress	60 ksi
Mild Steel's Modulus	29000 ksi
Concrete's f'c	5 ksi
Concrete's Modulus	4000 ksi
Shell Thickness (in)	0.05
Shell's Yield Stress	60 ksi
Shell's Modulus	29000 ksi

Rock parameters

(Limestone)

Layer elevation (ft)	40 ~ continue
C_u	20000 psf
γ_s	105 psf
E_{50}	0.0009

Ave Qu	83 tsf
Ave Qt	11 tsf

Uncased Shaft	
Elevation (ft)	38.3 ~ 24.4
Length (ft)	13.9
Unit Weight (pcf)	150
Mild Steel's Yield stress	60 ksi
Mild Steel's Modulus	29000 ksi
Concrete's f'c	5 ksi
Concrete's Modulus	4000 ksi

TH-3 (Victory)

Shaft Length	49.1 ft
Shaft Diameter	4 ft
Top of shaft	59.11 ft
Loading elevation	57 ft
Top of ground	40 ft
Top of rock	28 ft
Tip of shaft	10 ft
Max load	84.6 tons
Max Movement	6.9 in

Cross section info
20 #14 Bars
#4 Spiral
6" Cover
#14 - Diameter (1.693")
#14 - Area ($2.25in^2$)
#4 - Diameter (0.5")
#4 - Area ($0.2in^2$)

Soil parameters

(Medium Dense Sand)

Layer elevation(ft)	40 ~ 28
ϕ	38 ^{o}
γ_s	130 pcf
Subgrade Modulus	120 lb/in^3

Uncased Shaft	
Elevation (ft)	58 ~ 10
Length (ft)	48
Unit Weight (pcf)	150
Mild Steel's Yield stress	65 ksi
Mild Steel's Modulus	31000 ksi
Concrete's f'c	5 ksi
Concrete's Modulus	5000 ksi

Rock parameters

(Limestone)

Layer elevation(ft)	28 ~ continue
C_u	25000 psf
γ_s	110 psf
E_{50}	0.0001

Ave Qu 83 tsf

Ave Qt 11 tsf

APPENDIX D

17th Street (As Built vs. Planned Shaft Elevation)

SHAFTS INFORMATION																				
Pier No.	Center Station	Shaft	Test Shaft	Diameter		Planned Top Elevation		Actual Top Elevation		Planned Tip Elevation		Actual Tip Elevation		Planned Shaft Length		As Built Shaft Length		Average Length (As Built)	Difference (As Built - Planned)	
-	(100 ft.)	-	-	mm	feet	m	feet	m	feet	m	feet	m	feet	m	feet	m	feet	feet	m	feet
1	32.59	Shaft 1		1220	4	2.98	9.77	2.98	9.77	-26.40	-86.62	-22.57	-74.05	29.38	96.39	25.55	83.82		-3.83	-12.57
1	32.59	Shaft 2		1220	4	2.98	9.77	2.98	9.77	-26.40	-86.62	-21.12	-69.29	29.38	96.39	24.10	79.06		-5.28	-17.32
1	32.59	Shaft 3		1220	4	2.98	9.77	2.98	9.77	-26.40	-86.62	-22.57	-74.05	29.38	96.39	25.55	83.82		-3.83	-12.57
1	32.59	Shaft 4		1220	4	2.98	9.77	2.98	9.77	-26.40	-86.62	-21.12	-69.29	29.38	96.39	24.10	79.06	81.44	-5.28	-17.32
2	32.93	Shaft 1	LT-1	1220	4	-0.91	-2.99	-0.91	-2.99	-26.40	-86.62	-19.00	-62.34	25.49	83.63	18.09	59.35		-7.40	-24.28
2	32.93	Shaft 2		1220	4	-0.91	-2.99	-0.91	-2.99	-26.40	-86.62	-18.07	-59.29	25.49	83.63	17.16	56.30		-8.33	-27.33
2	32.93	Shaft 3		1220	4	-0.91	-2.99	-0.91	-2.99	-26.40	-86.62	-21.60	-70.87	25.49	83.63	20.69	67.88		-4.80	-15.75
2	32.93	Shaft 4		1220	4	-0.91	-2.99	-0.91	-2.99	-26.40	-86.62	-21.20	-69.56	25.49	83.63	20.29	66.57		-5.20	-17.06
2	32.93	Shaft 5		1220	4	-0.91	-2.99	-0.91	-2.99	-26.40	-86.62	-19.00	-62.34	25.49	83.63	18.09	59.35		-7.40	-24.28
2	32.93	Shaft 6		1220	4	-0.91	-2.99	-0.91	-2.99	-26.40	-86.62	-18.07	-59.29	25.49	83.63	17.16	56.30		-8.33	-27.33
2	32.93	Shaft 7		1220	4	-0.91	-2.99	-0.91	-2.99	-26.40	-86.62	-21.60	-70.87	25.49	83.63	20.69	67.88		-4.80	-15.75
2	32.93	Shaft 8		1220	4	-0.91	-2.99	-0.91	-2.99	-26.40	-86.62	-21.20	-69.56	25.49	83.63	20.29	66.57	62.53	-5.20	-17.06
3	33.56	Shaft 1		1220	4	-0.31	-1.02	-0.31	-1.02	-25.40	-83.34	-21.24	-69.69	25.09	82.32	20.93	68.67		-4.16	-13.65
3	33.56	Shaft 2		1220	4	-0.31	-1.02	-0.31	-1.02	-25.40	-83.34	-24.75	-81.20	25.09	82.32	24.44	80.19		-0.65	-2.13
3	33.56	Shaft 3		1220	4	-0.31	-1.02	-0.31	-1.02	-25.40	-83.34	-21.00	-68.90	25.09	82.32	20.69	67.88		-4.40	-14.44
3	33.56	Shaft 4		1220	4	-0.31	-1.02	-0.31	-1.02	-25.40	-83.34	-24.68	-80.98	25.09	82.32	24.37	79.96		-0.72	-2.36
3	33.56	Shaft 5		1220	4	-0.31	-1.02	-0.31	-1.02	-25.40	-83.34	-21.24	-69.69	25.09	82.32	20.93	68.67		-4.16	-13.65
3	33.56	Shaft 6		1220	4	-0.31	-1.02	-0.31	-1.02	-25.40	-83.34	-24.75	-81.20	25.09	82.32	24.44	80.19		-0.65	-2.13
3	33.56	Shaft 7		1220	4	-0.31	-1.02	-0.31	-1.02	-25.40	-83.34	-21.00	-68.90	25.09	82.32	20.69	67.88		-4.40	-14.44
3	33.56	Shaft 8		1220	4	-0.31	-1.02	-0.31	-1.02	-25.40	-83.34	-24.68	-80.98	25.09	82.32	24.37	79.96	74.18	-0.72	-2.36
4	34.18	Shaft 1		1220	4	-0.22	-0.73	-0.22	-0.73	-20.20	-66.28	-25.60	-83.99	19.98	65.54	25.38	83.26		5.40	17.72
4	34.18	Shaft 2		1220	4	-0.22	-0.73	-0.22	-0.73	-20.20	-66.28	-25.70	-84.32	19.98	65.54	25.48	83.59		5.50	18.05
4	34.18	Shaft 3		1220	4	-0.22	-0.73	-0.22	-0.73	-20.20	-66.28	-24.04	-78.88	19.98	65.54	23.82	78.14		3.84	12.60
4	34.18	Shaft 4		1220	4	-0.22	-0.73	-0.22	-0.73	-20.20	-66.28	-25.58	-83.93	19.98	65.54	25.36	83.20		5.38	17.65
4	34.18	Shaft 5		1220	4	-0.22	-0.73	-0.22	-0.73	-20.20	-66.28	-25.60	-83.99	19.98	65.54	25.38	83.26		5.40	17.72
4	34.18	Shaft 6		1220	4	-0.22	-0.73	-0.22	-0.73	-20.20	-66.28	-25.70	-84.32	19.98	65.54	25.48	83.59		5.50	18.05
4	34.18	Shaft 7		1220	4	-0.22	-0.73	-0.22	-0.73	-20.20	-66.28	-24.04	-78.88	19.98	65.54	23.82	78.14		3.84	12.60
4	34.18	Shaft 8		1220	4	-0.22	-0.73	-0.22	-0.73	-20.20	-66.28	-25.58	-83.93	19.98	65.54	25.36	83.20	82.05	5.38	17.65
5	34.81	Shaft 1		1220	4	-0.45	-1.48	-0.45	-1.48	-28.90	-94.82	-33.30	-109.26	28.45	93.34	32.85	107.78		4.40	14.44
5	34.81	Shaft 2		1220	4	-0.45	-1.48	-0.45	-1.48	-28.90	-94.82	-33.05	-108.44	28.45	93.34	32.60	106.96		4.15	13.62
5	34.81	Shaft 3	LTSO-3	1220	4	-0.45	-1.48	-0.45	-1.48	-28.90	-94.82	-29.00	-95.15	28.45	93.34	28.55	93.67		0.10	0.33
5	34.81	Shaft 4		1220	4	-0.45	-1.48	-0.45	-1.48	-28.90	-94.82	-33.20	-108.93	28.45	93.34	32.75	107.45		4.30	14.11
5	34.81	Shaft 5		1220	4	-0.45	-1.48	-0.45	-1.48	-28.90	-94.82	-33.30	-109.26	28.45	93.34	32.85	107.78		4.40	14.44
5	34.81	Shaft 6		1220	4	-0.45	-1.48	-0.45	-1.48	-28.90	-94.82	-33.05	-108.44	28.45	93.34	32.60	106.96		4.15	13.62
5	34.81	Shaft 7		1220	4	-0.45	-1.48	-0.45	-1.48	-28.90	-94.82	-29.00	-95.15	28.45	93.34	28.55	93.67		0.10	0.33
5	34.81	Shaft 8		1220	4	-0.45	-1.48	-0.45	-1.48	-28.90	-94.82	-33.20	-108.93	28.45	93.34	32.75	107.45	103.97	4.30	14.11

17th Street (As Built vs. Planned Shaft Elevation)

SHAFTS INFORMATION																				
Pier No.	Center Station	Shaft	Test Shaft	Diameter		Planned Top Elevation		Actual Top Elevation		Planned Tip Elevation		Actual Tip Elevation		Planned Shaft Length		As Built Shaft Length		Average Length (As Built)	Difference (As Built - Planned)	
-	(100 ft.)	-	-	mm	feet	m	feet	m	feet	m	feet	m	feet	m	feet	m	feet	feet	m	feet
6	35.43	Shaft 1		1220	4	-7.40	-24.28	-7.40	-24.28	-31.20	-102.37	-31.64	-103.81	23.80	78.09	24.24	79.53		0.44	1.44
6	35.43	Shaft 2		1220	4	-7.40	-24.28	-7.40	-24.28	-31.20	-102.37	-31.78	-104.27	23.80	78.09	24.38	79.99		0.58	1.90
6	35.43	Shaft 3		1220	4	-7.40	-24.28	-7.40	-24.28	-31.20	-102.37	-31.53	-103.45	23.80	78.09	24.13	79.17		0.33	1.08
6	35.43	Shaft 4		1220	4	-7.40	-24.28	-7.40	-24.28	-31.20	-102.37	-31.40	-103.02	23.80	78.09	24.00	78.74		0.20	0.66
6	35.43	Shaft 5		1220	4	-7.40	-24.28	-7.40	-24.28	-31.20	-102.37	-31.52	-103.42	23.80	78.09	24.12	79.14		0.32	1.05
6	35.43	Shaft 6		1220	4	-7.40	-24.28	-7.40	-24.28	-31.20	-102.37	-31.20	-102.37	23.80	78.09	23.80	78.09		0.00	0.00
6	35.43	Shaft 7		1220	4	-7.40	-24.28	-7.40	-24.28	-31.20	-102.37	-31.24	-102.50	23.80	78.09	23.84	78.22		0.04	0.13
6	35.43	Shaft 8		1220	4	-7.40	-24.28	-7.40	-24.28	-31.20	-102.37	-31.86	-104.53	23.80	78.09	24.46	80.25		0.66	2.17
6	35.43	Shaft 9		1220	4	-7.40	-24.28	-7.40	-24.28	-31.20	-102.37	-31.67	-103.91	23.80	78.09	24.27	79.63		0.47	1.54
6	35.43	Shaft 10	LTSO-1	1220	4	-7.40	-24.28	-7.40	-24.28	-31.20	-102.37	-34.61	-113.56	23.80	78.09	27.21	89.28		3.41	11.19
6	35.43	Shaft 11		1220	4	-7.40	-24.28	-7.40	-24.28	-31.20	-102.37	-31.70	-104.01	23.80	78.09	24.30	79.73		0.50	1.64
6	35.43	Shaft 12		1220	4	-7.40	-24.28	-7.40	-24.28	-31.20	-102.37	-31.25	-102.53	23.80	78.09	23.85	78.25		0.05	0.16
6	35.43	Shaft 13		1220	4	-7.40	-24.28	-7.40	-24.28	-31.20	-102.37	-31.30	-102.70	23.80	78.09	23.90	78.42		0.10	0.33
6	35.43	Shaft 14		1220	4	-7.40	-24.28	-7.40	-24.28	-31.20	-102.37	-31.64	-103.81	23.80	78.09	24.24	79.53		0.44	1.44
6	35.43	Shaft 15		1220	4	-7.40	-24.28	-7.40	-24.28	-31.20	-102.37	-31.78	-104.27	23.80	78.09	24.38	79.99		0.58	1.90
6	35.43	Shaft 16		1220	4	-7.40	-24.28	-7.40	-24.28	-31.20	-102.37	-31.53	-103.45	23.80	78.09	24.13	79.17		0.33	1.08
6	35.43	Shaft 17		1220	4	-7.40	-24.28	-7.40	-24.28	-31.20	-102.37	-31.40	-103.02	23.80	78.09	24.00	78.74		0.20	0.66
6	35.43	Shaft 18		1220	4	-7.40	-24.28	-7.40	-24.28	-31.20	-102.37	-31.52	-103.42	23.80	78.09	24.12	79.14		0.32	1.05
6	35.43	Shaft 19		1220	4	-7.40	-24.28	-7.40	-24.28	-31.20	-102.37	-31.20	-102.37	23.80	78.09	23.80	78.09		0.00	0.00
6	35.43	Shaft 20		1220	4	-7.40	-24.28	-7.40	-24.28	-31.20	-102.37	-31.24	-102.50	23.80	78.09	23.84	78.22		0.04	0.13
6	35.43	Shaft 21		1220	4	-7.40	-24.28	-7.40	-24.28	-31.20	-102.37	-31.86	-104.53	23.80	78.09	24.46	80.25		0.66	2.17
6	35.43	Shaft 22		1220	4	-7.40	-24.28	-7.40	-24.28	-31.20	-102.37	-31.67	-103.91	23.80	78.09	24.27	79.63		0.47	1.54
6	35.43	Shaft 23		1220	4	-7.40	-24.28	-7.40	-24.28	-31.20	-102.37	-34.61	-113.56	23.80	78.09	27.21	89.28		3.41	11.19
6	35.43	Shaft 24		1220	4	-7.40	-24.28	-7.40	-24.28	-31.20	-102.37	-31.70	-104.01	23.80	78.09	24.30	79.73		0.50	1.64
6	35.43	Shaft 25		1220	4	-7.40	-24.28	-7.40	-24.28	-31.20	-102.37	-31.25	-102.53	23.80	78.09	23.85	78.25		0.05	0.16
6	35.43	Shaft 26		1220	4	-7.40	-24.28	-7.40	-24.28	-31.20	-102.37	-31.30	-102.70	23.80	78.09	23.90	78.42	79.88	0.10	0.33

17th Street (As Built vs. Planned Shaft Elevation)

SHAFTS INFORMATION																				
Pier No.	Center Station	Shaft	Test Shaft	Diameter		Planned Top Elevation		Actual Top Elevation		Planned Tip Elevation		Actual Tip Elevation		Planned Shaft Length		As Built Shaft Length		Average Length (As Built)	Difference (As Built - Planned)	
-	(100 ft.)	-	-	mm	feet	m	feet	m	feet	m	feet	m	feet	m	feet	m	feet	feet	m	feet
7	36.18	Shaft1		1220	4	-7.40	-24.28	-7.40	-24.28	-35.50	-116.48	-32.15	-105.48	28.10	92.20	24.75	81.20		-3.35	-10.99
7	36.18	Shaft2		1220	4	-7.40	-24.28	-7.40	-24.28	-35.50	-116.48	-31.69	-103.97	28.10	92.20	24.29	79.70		-3.81	-12.50
7	36.18	Shaft3	LTSO-2	1220	4	-7.40	-24.28	-7.40	-24.28	-35.50	-116.48	-39.84	-130.72	28.10	92.20	32.44	106.44		4.34	14.24
7	36.18	Shaft4		1220	4	-7.40	-24.28	-7.40	-24.28	-35.50	-116.48	-31.50	-103.35	28.10	92.20	24.10	79.07		-4.00	-13.12
7	36.18	Shaft5		1220	4	-7.40	-24.28	-7.40	-24.28	-35.50	-116.48	-31.80	-104.34	28.10	92.20	24.40	80.06		-3.70	-12.14
7	36.18	Shaft6		1220	4	-7.40	-24.28	-7.40	-24.28	-35.50	-116.48	-32.50	-106.63	28.10	92.20	25.10	82.35		-3.00	-9.84
7	36.18	Shaft7		1220	4	-7.40	-24.28	-7.40	-24.28	-35.50	-116.48	-31.65	-103.84	28.10	92.20	24.25	79.56		-3.85	-12.63
7	36.18	Shaft8		1220	4	-7.40	-24.28	-7.40	-24.28	-35.50	-116.48	-31.50	-103.35	28.10	92.20	24.10	79.07		-4.00	-13.12
7	36.18	Shaft9		1220	4	-7.40	-24.28	-7.40	-24.28	-35.50	-116.48	-32.10	-105.32	28.10	92.20	24.70	81.04		-3.40	-11.16
7	36.18	Shaft10		1220	4	-7.40	-24.28	-7.40	-24.28	-35.50	-116.48	-31.56	-103.55	28.10	92.20	24.16	79.27		-3.94	-12.93
7	36.18	Shaft11		1220	4	-7.40	-24.28	-7.40	-24.28	-35.50	-116.48	-31.70	-104.01	28.10	92.20	24.30	79.73		-3.80	-12.47
7	36.18	Shaft12	Three	1220	4	-7.40	-24.28	-7.40	-24.28	-35.50	-116.48	-32.00	-104.99	28.10	92.20	24.60	80.71		-3.50	-11.48
7	36.18	Shaft13	Shafts	1220	4	-7.40	-24.28	-7.40	-24.28	-35.50	-116.48	-31.65	-103.84	28.10	92.20	24.25	79.56		-3.85	-12.63
7	36.18	Shaft14	from	1220	4	-7.40	-24.28	-7.40	-24.28	-35.50	-116.48	-32.15	-105.48	28.10	92.20	24.75	81.20		-3.35	-10.99
7	36.18	Shaft15	the	1220	4	-7.40	-24.28	-7.40	-24.28	-35.50	-116.48	-31.69	-103.97	28.10	92.20	24.29	79.70		-3.81	-12.50
7	36.18	Shaft16	temporary	1220	4	-7.40	-24.28	-7.40	-24.28	-35.50	-116.48	-39.84	-130.72	28.10	92.20	32.44	106.44		4.34	14.24
7	36.18	Shaft17	bridge	1220	4	-7.40	-24.28	-7.40	-24.28	-35.50	-116.48	-31.50	-103.35	28.10	92.20	24.10	79.07		-4.00	-13.12
7	36.18	Shaft18	are	1220	4	-7.40	-24.28	-7.40	-24.28	-35.50	-116.48	-31.80	-104.34	28.10	92.20	24.40	80.06		-3.70	-12.14
7	36.18	Shaft19	utilized.	1220	4	-7.40	-24.28	-7.40	-24.28	-35.50	-116.48	-32.50	-106.63	28.10	92.20	25.10	82.35		-3.00	-9.84
7	36.18	Shaft20		1220	4	-7.40	-24.28	-7.40	-24.28	-35.50	-116.48	-31.65	-103.84	28.10	92.20	24.25	79.56		-3.85	-12.63
7	36.18	Shaft21		1220	4	-7.40	-24.28	-7.40	-24.28	-35.50	-116.48	-31.50	-103.35	28.10	92.20	24.10	79.07		-4.00	-13.12
7	36.18	Shaft22		1220	4	-7.40	-24.28	-7.40	-24.28	-35.50	-116.48	-32.10	-105.32	28.10	92.20	24.70	81.04		-3.40	-11.16
7	36.18	Shaft23		1220	4	-7.40	-24.28	-7.40	-24.28	-35.50	-116.48	-31.56	-103.55	28.10	92.20	24.16	79.27		-3.94	-12.93
7	36.18	Shaft24		1220	4	-7.40	-24.28	-7.40	-24.28	-35.50	-116.48	-31.70	-104.01	28.10	92.20	24.30	79.73		-3.80	-12.47
7	36.18	Shaft25		1220	4	-7.40	-24.28	-7.40	-24.28	-35.50	-116.48	-32.00	-104.99	28.10	92.20	24.60	80.71		-3.50	-11.48
7	36.18	Shaft26		1220	4	-7.40	-24.28	-7.40	-24.28	-35.50	-116.48	-31.65	-103.84	28.10	92.20	24.25	79.56	82.14	-3.85	-12.63

17^{th} Street (As Built vs. Planned Shaft Elevation)

SHAFTS INFORMATION																				
Pier No.	Center Station	Shaft	Test Shaft	Diameter		Planned Top Elevation		Actual Top Elevation		Planned Tip Elevation		Actual Tip Elevation		Planned Shaft Length		As Built Shaft Length		Average Length (As Built)	Difference (As Built - Planned)	
-	(100 ft.)	-	-	mm	feet	m	feet	m	feet	m	feet	m	feet	m	feet	m	feet	feet	m	feet
8	36.81	Shaft 1		1220	4	-4.75	-15.58	-4.75	-15.58	-19.40	-63.65	-18.25	-59.88	14.65	48.07	13.50	44.29		-1.15	-3.77
8	36.81	Shaft 2		1220	4	-4.75	-15.58	-4.75	-15.58	-19.40	-63.65	-18.24	-59.85	14.65	48.07	13.49	44.26		-1.16	-3.81
8	36.81	Shaft 3	LT-2	1220	4	-4.75	-15.58	-4.75	-15.58	-19.40	-63.65	-19.67	-64.54	14.65	48.07	14.92	48.95		0.27	0.89
8	36.81	Shaft 4		1220	4	-4.75	-15.58	-4.75	-15.58	-19.40	-63.65	-18.96	-62.21	14.65	48.07	14.21	46.62		-0.44	-1.44
8	36.81	Shaft 5		1220	4	-4.75	-15.58	-4.75	-15.58	-19.40	-63.65	-18.25	-59.88	14.65	48.07	13.50	44.29		-1.15	-3.77
8	36.81	Shaft 6		1220	4	-4.75	-15.58	-4.75	-15.58	-19.40	-63.65	-18.24	-59.85	14.65	48.07	13.49	44.26		-1.16	-3.81
8	36.81	Shaft 7		1220	4	-4.75	-15.58	-4.75	-15.58	-19.40	-63.65	-19.67	-64.54	14.65	48.07	14.92	48.95		0.27	0.89
8	36.81	Shaft 8		1220	4	-4.75	-15.58	-4.75	-15.58	-19.40	-63.65	-18.96	-62.21	14.65	48.07	14.21	46.62	46.03	-0.44	-1.44
9	37.43	Shaft 1		1220	4	-0.35	-1.15	-0.35	-1.15	-17.20	-56.43	-17.50	-57.42	16.85	55.28	17.15	56.27		0.30	0.98
9	37.43	Shaft 2		1220	4	-0.35	-1.15	-0.35	-1.15	-17.20	-56.43	-17.25	-56.60	16.85	55.28	16.90	55.45		0.05	0.16
9	37.43	Shaft 3		1220	4	-0.35	-1.15	-0.35	-1.15	-17.20	-56.43	-17.50	-57.42	16.85	55.28	17.15	56.27		0.30	0.98
9	37.43	Shaft 4		1220	4	-0.35	-1.15	-0.35	-1.15	-17.20	-56.43	-17.30	-56.76	16.85	55.28	16.95	55.61		0.10	0.33
9	37.43	Shaft 5		1220	4	-0.35	-1.15	-0.35	-1.15	-17.20	-56.43	-17.50	-57.42	16.85	55.28	17.15	56.27		0.30	0.98
9	37.43	Shaft 6		1220	4	-0.35	-1.15	-0.35	-1.15	-17.20	-56.43	-17.25	-56.60	16.85	55.28	16.90	55.45		0.05	0.16
9	37.43	Shaft 7		1220	4	-0.35	-1.15	-0.35	-1.15	-17.20	-56.43	-17.50	-57.42	16.85	55.28	17.15	56.27		0.30	0.98
9	37.43	Shaft 8		1220	4	-0.35	-1.15	-0.35	-1.15	-17.20	-56.43	-17.30	-56.76	16.85	55.28	16.95	55.61	55.90	0.10	0.33
10	38.06	Shaft 1	LTSO-4	1220	4	-0.25	-0.82	-0.25	-0.82	-17.20	-56.43	-19.00	-62.34	16.95	55.61	18.75	61.52		1.80	5.91
10	38.06	Shaft 2		1220	4	-0.25	-0.82	-0.25	-0.82	-17.20	-56.43	-17.40	-57.09	16.95	55.61	17.15	56.27		0.20	0.66
10	38.06	Shaft 3		1220	4	-0.25	-0.82	-0.25	-0.82	-17.20	-56.43	-17.50	-57.42	16.95	55.61	17.25	56.60		0.30	0.98
10	38.06	Shaft 4		1220	4	-0.25	-0.82	-0.25	-0.82	-17.20	-56.43	-17.20	-56.43	16.95	55.61	16.95	55.61		0.00	0.00
10	38.06	Shaft 5		1220	4	-0.25	-0.82	-0.25	-0.82	-17.20	-56.43	-19.00	-62.34	16.95	55.61	18.75	61.52		1.80	5.91
10	38.06	Shaft 6		1220	4	-0.25	-0.82	-0.25	-0.82	-17.20	-56.43	-17.40	-57.09	16.95	55.61	17.15	56.27		0.20	0.66
10	38.06	Shaft 7		1220	4	-0.25	-0.82	-0.25	-0.82	-17.20	-56.43	-17.50	-57.42	16.95	55.61	17.25	56.60		0.30	0.98
10	38.06	Shaft 8		1220	4	-0.25	-0.82	-0.25	-0.82	-17.20	-56.43	-17.20	-56.43	16.95	55.61	16.95	55.61	57.50	0.00	0.00
11	38.40	Shaft 1		1220	4	6.73	22.07	6.73	22.07	-17.20	-56.43	-17.30	-56.76	23.93	78.50	24.03	78.83		0.10	0.33
11	38.40	Shaft 2		1220	4	6.73	22.07	6.73	22.07	-17.20	-56.43	-17.50	-57.42	23.93	78.50	24.23	79.49		0.30	0.98
11	38.40	Shaft 3		1220	4	6.73	22.07	6.73	22.07	-17.20	-56.43	-17.30	-56.76	23.93	78.50	24.03	78.83		0.10	0.33
11	38.40	Shaft 4		1220	4	6.73	22.07	6.73	22.07	-17.20	-56.43	-17.50	-57.42	23.93	78.50	24.23	79.49	79.16	0.30	0.98
	No. of Shafts	122	**Total**											**2742.27**	**8997.39**	**2655.29**	**8712.01**		**-86.98**	**-285.38**

Apalachicola (As Built vs. Planned Shaft Elevation)

SHAFTS INFORMATION																					
Pier No.	Center Station	Shaft	Test Shaft	Diameter	Planned Top Elevation	Actual Top Elevation	Planned Tip Elevation	Actual Tip Elevation	Planned Shaft Length (feet)				As Built Shaft Length (feet)				Ave. Length (As Built)	Difference (As Built - Planned) (feet)			
-	(100 ft.)	-	-	feet	feet	feet	feet	feet	5 feet	6 feet	7 feet	9 feet	5 feet	6 feet	7 feet	9 feet	feet	5 feet	6 feet	7 feet	9 feet
43	620.93	Shaft 1		5	47.0	47.0	-34.0	-34.6	81.0				81.6					0.6			
43	620.93	Shaft 2		5	47.0	47.0	-34.0	-35.1	81.0				82.1				81.9	1.1			
44	622.03	Shaft 1		5	47.0	47.0	-34.0	-36.1	81.0				83.1					2.1			
44	622.03	Shaft 2		5	47.0	47.0	-34.0	-35.0	81.0				82.0				82.6	1.0			
45	623.13	Shaft 1		5	47.0	47.0	-34.0	-33.7	81.0				80.7					-0.3			
45	623.13	Shaft 2		5	47.0	47.0	-34.0	-35.3	81.0				82.3				81.5	1.3			
46	624.23	Shaft 1		5	47.0	47.0	-34.0	-34.4	81.0				81.4					0.4			
46	624.23	Shaft 2		5	47.0	47.0	-34.0	-35.1	81.0				82.1				81.8	1.1			
47	625.33	Shaft 1		5	47.0	47.0	-34.0	-34.6	81.0				81.6					0.6			
47	625.33	Shaft 2		5	47.0	47.0	-34.0	-34.7	81.0				81.7				81.7	0.7			
48	626.43	Shaft 1		6	47.0	47.0	-29.0	-28.6		76.0				75.6					-0.4		
48	626.43	Shaft 2		6	47.0	47.0	-29.0	-28.5		76.0				75.5			75.6		-0.5		
49	627.53	Shaft 1		6	47.0	47.0	-29.0	-29.3		76.0				76.3					0.3		
49	627.53	Shaft 2		6	47.0	47.0	-29.0	-29.7		76.0				76.7			76.5		0.7		
50	628.63	Shaft 1		6	47.0	47.0	-29.0	-29.4		76.0				76.4					0.4		
50	628.63	Shaft 2		6	47.0	47.0	-29.0	-29.0		76.0				76.0			76.2		0.0		
51	629.73	Shaft 1		6	47.0	47.0	-33.0	-34.2		80.0				81.2					1.2		
51	629.73	Shaft 2		6	47.0	47.0	-33.0	-34.2		80.0				81.2			81.2		1.2		
52	630.83	Shaft 1		6	47.0	47.0	-33.0	-34.6		80.0				81.6					1.6		
52	630.83	Shaft 2		6	47.0	47.0	-33.0	-41.7		80.0				88.7			85.2		8.7		
53	631.93	Shaft 1		6	47.0	47.0	-33.0	-36.0		80.0				83.0					3.0		
53	631.93	Shaft 2		6	47.0	47.0	-33.0	-34.2		80.0				81.2			82.1		1.2		
			Test Hole No.1		47.0																
			Test Hole No.2		47.0																
54	633.03	Shaft 1		6	47.0	47.0	-33.0	-35.5		80.0				82.5					2.5		
54	633.03	Shaft 2		6	47.0	47.0	-33.0	-35.8		80.0				82.8			82.7		2.8		
55	634.13	Shaft 1		6	47.0	47.0	-33.0	-41.7		80.0				88.7					8.7		
55	634.13	Shaft 2		6	47.0	47.0	-33.0	-32.8		80.0				79.8			84.3		-0.2		
56	635.23	Shaft 1		6	47.0	47.0	-38.0	-38.4		85.0				85.4					0.4		
56	635.23	Shaft 2		6	47.0	47.0	-38.0	-41.3		85.0				88.3			86.9		3.3		
57	636.33	Shaft 1		7	55.5	55.5	-55.0	-55.2			110.5				110.7					0.2	
57	636.33	Shaft 2		7	55.5	55.5	-55.0	-58.8			110.5				114.3		112.5			3.8	
			Test Hole No.9																		
			Test Hole No.10																		

Apalachicola (As Built vs. Planned Shaft Elevation)

SHAFTS INFORMATION																					
Pier No.	Center Station	Shaft	Test Shaft	Diameter	Planned Top Elevation	Actual Top Elevation	Planned Tip Elevation	Actual Tip Elevation	Planned Shaft Length (feet)				As Built Shaft Length (feet)				Ave. Length (As Built)	Difference (As Built - Planned) (feet)			
-	(100 ft.)	-	-	feet	feet	feet	feet	feet	5 feet	6 feet	7 feet	9 feet	5 feet	6 feet	7 feet	9 feet	feet	5 feet	6 feet	7 feet	9 feet
58	638.58	Shaft 1		9	55.5	55.5	-72.0	-61.2				127.5				116.7					-10.8
58	638.58	Shaft 2		9	55.5	55.5	-72.0	-60.6				127.5				116.1	116.4				-11.4
59	641.38	Shaft 1		9	55.5	55.5	-75.0	-74.9				130.5				130.4					-0.1
59	641.38	Shaft 2		9	55.5	55.5	-75.0	-59.8				130.5				115.3	122.9				-15.2
			Test Hole No.8																		
60	643.63	Shaft 1		7	55.5	55.5	-65.0	-52.4			120.5				107.9					-12.6	
60	643.63	Shaft 2		7	55.5	55.5	-65.0	-62.5			120.5				118.0		113.0			-2.5	
61	644.73	Shaft 1		6	47.0	47.0	-35.0	-31.0		82.0				78.0					-4.0		
61	644.73	Shaft 2		6	47.0	47.0	-35.0	-31.0		82.0				78.0			78.0		-4.0		
62	645.83	Shaft 1		6	47.0	47.0	-39.0	-31.0		86.0				78.0					-8.0		
62	645.83	Shaft 2		6	47.0	47.0	-39.0	-31.8		86.0				78.8			78.4		-7.2		
			Test Hole No.5																		
			Test Hole No.6																		
63	646.93	Shaft 1		6	47.0	47.0	-39.0	-31.1		86.0				78.1					-7.9		
63	646.93	Shaft 2		6	47.0	47.0	-39.0	-31.9		86.0				78.9			78.5		-7.1		
64	648.03	Shaft 1		6	47.0	47.0	-39.0	-31.1		86.0				78.1					-7.9		
64	648.03	Shaft 2		6	47.0	47.0	-39.0	-31.4		86.0				78.4			78.3		-7.6		
65	649.13	Shaft 1		6	47.0	47.0	-39.0	-31.3		86.0				78.3					-7.7		
65	649.13	Shaft 2		6	47.0	47.0	-39.0	-31.4		86.0				78.4			78.4		-7.6		
66	650.23	Shaft 1		6	47.0	47.0	-37.0	-31.1		84.0				78.1					-5.9		
66	650.23	Shaft 2		6	47.0	47.0	-37.0	-31.0		84.0				78.0			78.1		-6.0		
67	651.33	Shaft 1		5	47.0	47.0	-49.0	-36.0	96.0				83.0					-13.0			
67	651.33	Shaft 2		5	47.0	47.0	-49.0	-35.9	96.0				82.9				83.0	-13.1			
68	652.43	Shaft 1		5	47.0	47.0	-49.0	-35.3	96.0				82.3					-13.7			
68	652.43	Shaft 2		5	47.0	47.0	-49.0	-35.2	96.0				82.2				82.3	-13.8			
69	653.53	Shaft 1		5	47.0	47.0	-49.0	-35.0	96.0				82.0					-14.0			
69	653.53	Shaft 2		5	47.0	47.0	-49.0	-35.7	96.0				82.7				82.4	-13.3			
			Test Hole No.7																		
70	654.63	Shaft 1		5	47.0	47.0	-39.0	-34.5	86.0				81.5					-4.5			
70	654.63	Shaft 2		5	47.0	47.0	-39.0	-33.8	86.0				80.8				81.2	-5.2			
71	655.73	Shaft 1		5	47.0	47.0	-39.0	-33.2	86.0				80.2					-5.8			
71	655.73	Shaft 2		5	47.0	47.0	-39.0	-33.4	86.0				80.4				80.3	-5.6			
72	656.83	Shaft 1		5	47.0	47.0	-39.0	-37.0	86.0				84.0					-2.0			
72	656.83	Shaft 2		5	47.0	47.0	-39.0	-35.7	86.0				82.7				83.3	-3.3			
73	657.82	Shaft 1		5	47.0	47.0	-39.0	-35.2	86.0				82.2					-3.8			
73	657.82	Shaft 2		5	47.0	47.0	-39.0	-35.5	86.0				82.5				82.4	-3.5			
	No. of	64	**Total**						**2074.0**	**2446.0**	**462.0**	**516.0**	**1968.0**	**2400.0**	**450.9**	**478.5**		**-106.0**	**-46.0**	**-11.1**	**-37.5**

Gandy (As Built vs. Planned Shaft Elevation)

SHAFTS INFORMATION																			
Pier No.	Center Station	Shaft	Test Shaft	Diameter		Planned Top Elevation	Actual Top Elevation	Planned Tip Elevation	Actual Tip Elevation	Planned Shaft Length (feet)			As Built Shaft Length (feet)			Ave. Length (As Built)	Difference (As Built - Planned) (feet)		
-	(100 ft.)	-	-	Type	feet	feet	feet	feet	feet	4 feet	6 feet	7 feet	4 feet	6 feet	7 feet	feet	4 feet	6 feet	7 feet
2W	45.63	Shaft 1		I	4	-1.5	-1.5	-61.0	-49.3	59.5			47.8				-11.7		
2W	45.63	Shaft 2		I	4	-1.5	-1.5	-61.0	-55.6	59.5			54.1				-5.4		
2W	45.63	Shaft 3		I	4	-1.5	-1.5	-61.0	-49.0	59.5			47.5				-12.0		
2W	45.63	Shaft 4		I	4	-1.5	-1.5	-61.0	-49.6	59.5			48.1			49.4	-11.4		
3W	46.59	Shaft 1		I	4	-1.5	-1.5	-52.0	-52.0	50.5			50.5				0.0		
3W	46.59	Shaft 2		I	4	-1.5	-1.5	-52.0	-52.5	50.5			51.0				0.5		
3W	46.59	Shaft 3		I	4	-1.5	-1.5	-52.0	-58.4	50.5			56.9				6.4		
3W	46.59	Shaft 4		I	4	-1.5	-1.5	-52.0	-52.0	50.5			50.5			52.2	0.0		
4W	48.03	Shaft 1		I	4	-1.5	-1.5	-47.0	-48.0	45.5			46.5				1.0		
4W	48.03	Shaft 2		I	4	-1.5	-1.5	-47.0	-47.3	45.5			45.8				0.3		
4W	48.03	Shaft 3		I	4	-1.5	-1.5	-47.0	-48.2	45.5			46.7				1.2		
4W	48.03	Shaft 4		I	4	-1.5	-1.5	-47.0	-47.6	45.5			46.1			46.3	0.6		
5W	49.47	Shaft 1		I	4	-1.5	-1.5	-48.0	-56.5	46.5			55.0				8.5		
5W	49.47	Shaft 2		I	4	-1.5	-1.5	-48.0	-48.4	46.5			46.9				0.4		
5W	49.47	Shaft 3		I	4	-1.5	-1.5	-48.0	-49.3	46.5			47.8				1.3		
5W	49.47	Shaft 4		I	4	-1.5	-1.5	-48.0	-49.2	46.5			47.7			49.4	1.2		
6W	50.91	Shaft 1		I	4	-1.5	-1.5	-55.0	-55.0	53.5			53.5				0.0		
6W	50.91	Shaft 2		I	4	-1.5	-1.5	-55.0	-55.4	53.5			53.9				0.4		
6W	50.91	Shaft 3		I	4	-1.5	-1.5	-55.0	-55.0	53.5			53.5				0.0		
6W	50.91	Shaft 4		I	4	-1.5	-1.5	-55.0	-55.3	53.5			53.8			53.7	0.3		
7W	52.35	Shaft 1		I	4	-1.5	-1.5	-52.0	-52.3	50.5			50.8				0.3		
7W	52.35	Shaft 2		I	4	-1.5	-1.5	-52.0	-52.3	50.5			50.8				0.3		
7W	52.35	Shaft 3		I	4	-1.5	-1.5	-52.0	-52.4	50.5			50.9				0.4		
7W	52.35	Shaft 4		I	4	-1.5	-1.5	-52.0	-51.8	50.5			50.3			50.7	-0.2		
8W	53.79	Shaft 1		I	4	-1.5	-1.5	-41.0	-41.4	39.5			39.9				0.4		
8w	53.79	Shaft 2		I	4	-1.5	-1.5	-41.0	-42.0	39.5			40.5				1.0		
8w	53.79	Shaft 3		I	4	-1.5	-1.5	-41.0	-41.5	39.5			40.0				0.5		
8w	53.79	Shaft 4		I	4	-1.5	-1.5	-41.0	-42.5	39.5			41.0			40.4	1.5		
9W	55.23	Shaft 1		I	4	-1.5	-1.5	-41.0	-41.5	39.5			40.0				0.5		
9W	55.23	Shaft 2		I	4	-1.5	-1.5	-41.0	-41.4	39.5			39.9				0.4		
9W	55.23	Shaft 3		I	4	-1.5	-1.5	-41.0	-41.1	39.5			39.6				0.1		
9W	55.23	Shaft 4		I	4	-1.5	-1.5	-41.0	-41.0	39.5			39.5			39.8	0.0		
10W	56.67	Shaft 1		I	4	-1.5	-1.5	-31.0	-33.1	29.5			31.6				2.1		
10W	56.67	Shaft 2		I	4	-1.5	-1.5	-31.0	-32.2	29.5			30.7				1.2		
10W	56.67	Shaft 3		I	4	-1.5	-1.5	-31.0	-33.1	29.5			31.6				2.1		
10W	56.67	Shaft 4		I	4	-1.5	-1.5	-31.0	-32.2	29.5			30.7			31.2	1.2		
11W	58.11	Shaft 1		I	4	-1.5	-1.5	-42.0	-42.3	40.5			40.8				0.3		
11W	58.11	Shaft 2		I	4	-1.5	-1.5	-42.0	-42.0	40.5			40.5				0.0		
11W	58.11	Shaft 3		I	4	-1.5	-1.5	-42.0	-42.0	40.5			40.5				0.0		
11W	58.11	Shaft 4		I	4	-1.5	-1.5	-42.0	-42.0	40.5			40.5			40.6	0.0		

Gandy (As Built vs. Planned Shaft Elevation)

SHAFTS INFORMATION																			
Pier No.	Center Station	Shaft	Test Shaft	Diameter		Planned Top Elevation	Actual Top Elevation	Planned Tip Elevation	Actual Tip Elevation	Planned Shaft Length (feet)			As Built Shaft Length (feet)			Ave. Length (As Built)	Difference (As Built - Planned) (feet)		
-	(100 ft.)	-	-	Type	feet	feet	feet	feet	feet	4 feet	6 feet	7 feet	4 feet	6 feet	7 feet	feet	4 feet	6 feet	7 feet
12W	59.55	Shaft 1		I	4	-1.5	-1.5	-38.0	-38.2	36.5			36.7				0.2		
12W	59.55	Shaft 2		I	4	-1.5	-1.5	-38.0	-38.1	36.5			36.6				0.1		
12W	59.55	Shaft 3		I	4	-1.5	-1.5	-38.0	-38.4	36.5			36.9				0.4		
12W	59.55	Shaft 4		I	4	-1.5	-1.5	-38.0	-38.3	36.5			36.8			36.8	0.3		
13W	60.99	Shaft 1		I	4	-1.5	-1.5	-52.0	-51.8	50.5			50.3				-0.2		
13W	60.99	Shaft 2		I	4	-1.5	-1.5	-52.0	-52.4	50.5			50.9				0.4		
13W	60.99	Shaft 3		I	4	-1.5	-1.5	-52.0	-52.4	50.5			50.9				0.4		
13W	60.99	Shaft 4		I	4	-1.5	-1.5	-52.0	-52.1	50.5			50.6			50.7	0.1		
14W	62.43	Shaft 1		I	4	-1.5	-1.5	-26.0	-27.1	24.5			25.6				1.1		
14W	62.43	Shaft 2		I	4	-1.5	-1.5	-26.0	-26.5	24.5			25.0				0.5		
14W	62.43	Shaft 3		I	4	-1.5	-1.5	-26.0	-26.0	24.5			24.5				0.0		
14W	62.43	Shaft 4		I	4	-1.5	-1.5	-26.0	-26.1	24.5			24.6			24.9	0.1		
15W	63.87	Shaft 1		I	4	-1.5	-1.5	-32.0	-32.1	30.5			30.6				0.1		
15W	63.87	Shaft 2		I	4	-1.5	-1.5	-32.0	-32.3	30.5			30.8				0.3		
15W	63.87	Shaft 3		I	4	-1.5	-1.5	-32.0	-33.0	30.5			31.5				1.0		
15W	63.87	Shaft 4		I	4	-1.5	-1.5	-32.0	-32.8	30.5			31.3			31.1	0.8		
16W	65.31	Shaft 1		I	4	-1.5	-1.5	-21.0	-25.5	19.5			24.0				4.5		
16W	65.31	Shaft 2		I	4	-1.5	-1.5	-21.0	-25.4	19.5			23.9				4.4		
16W	65.31	Shaft 3		I	4	-1.5	-1.5	-21.0	-25.7	19.5			24.2				4.7		
16W	65.31	Shaft 4		I	4	-1.5	-1.5	-21.0	-25.7	19.5			24.2			24.1	4.7		
17W	66.75	Shaft 1		I	4	-1.5	-1.5	-30.0	-30.5	28.5			29.0				0.5		
17W	66.75	Shaft 2		I	4	-1.5	-1.5	-30.0	-30.2	28.5			28.7				0.2		
17W	66.75	Shaft 3		I	4	-1.5	-1.5	-30.0	-30.1	28.5			28.6				0.1		
17W	66.75	Shaft 4		I	4	-1.5	-1.5	-30.0	-30.3	28.5			28.8			28.8	0.3		
18W	68.19	Shaft 1		I	4	-1.5	-1.5	-36.0	-36.3	34.5			34.8				0.3		
18W	68.19	Shaft 2		I	4	-1.5	-1.5	-36.0	-36.2	34.5			34.7				0.2		
18W	68.19	Shaft 3		I	4	-1.5	-1.5	-36.0	-36.2	34.5			34.7				0.2		
18W	68.19	Shaft 4		I	4	-1.5	-1.5	-36.0	-36.1	34.5			34.6			34.7	0.1		
19W	69.63	Shaft 1		I	4	-1.5	-1.5	-22.0	-22.4	20.5			20.9				0.4		
19W	69.63	Shaft 2		I	4	-1.5	-1.5	-22.0	-22.7	20.5			21.2				0.7		
19W	69.63	Shaft 3		I	4	-1.5	-1.5	-22.0	-22.6	20.5			21.1				0.6		
19W	69.63	Shaft 4		I	4	-1.5	-1.5	-22.0	-22.5	20.5			21.0			21.1	0.5		
20W	71.07	Shaft 1		I	4	-1.5	-1.5	-57.0	-30.4	55.5			28.9				-26.6		
20W	71.07	Shaft 2		I	4	-1.5	-1.5	-57.0	-30.7	55.5			29.2				-26.3		
20W	71.07	Shaft 3		I	4	-1.5	-1.5	-57.0	-30.1	55.5			28.6				-26.9		
20W	71.07	Shaft 4		I	4	-1.5	-1.5	-57.0	-30.4	55.5			28.9			28.9	-26.6		
21W	72.51	Shaft 1		I	4	-1.5	-1.5	-29.0	-29.6	27.5			28.1				0.6		
21W	72.51	Shaft 2		I	4	-1.5	-1.5	-29.0	-29.2	27.5			27.7				0.2		
21W	72.51	Shaft 3		I	4	-1.5	-1.5	-29.0	-29.8	27.5			28.3				0.8		
21W	72.51	Shaft 4		I	4	-1.5	-1.5	-29.0	-29.5	27.5			28.0			28.0	0.5		

Gandy (As Built vs. Planned Shaft Elevation)

SHAFTS INFORMATION																			
Pier No.	Center Station	Shaft	Test Shaft	Diameter		Planned Top Elevation	Actual Top Elevation	Planned Tip Elevation	Actual Tip Elevation	Planned Shaft Length (feet)			As Built Shaft Length (feet)			Ave. Length (As Built)	Difference (As Built - Planned) (feet)		
-	(100 ft.)	-	-	Type	feet	feet	feet	feet	feet	4 feet	6 feet	7 feet	4 feet	6 feet	7 feet	feet	4 feet	6 feet	7 feet
22W	73.95	Shaft 1		I	4	-1.5	-1.5	-30.0	-30.2	28.5			28.7				0.2		
22W	73.95	Shaft 2		I	4	-1.5	-1.5	-30.0	-30.6	28.5			29.1				0.6		
22W	73.95	Shaft 3		I	4	-1.5	-1.5	-30.0	-32.3	28.5			30.8				2.3		
22W	73.95	Shaft 4		I	4	-1.5	-1.5	-30.0	-30.1	28.5			28.6			29.3	0.1		
23W	75.39	Shaft 1		I	4	-1.5	-1.5	-41.0	-41.0	39.5			39.5				0.0		
23W	75.39	Shaft 2		I	4	-1.5	-1.5	-41.0	-41.2	39.5			39.7				0.2		
23W	75.39	Shaft 3		I	4	-1.5	-1.5	-41.0	-41.3	39.5			39.8				0.3		
23W	75.39	Shaft 4		I	4	-1.5	-1.5	-41.0	-41.8	39.5			40.3			39.8	0.8		
24 W	76.83	Shaft 1		I	4	-1.5	-1.5	-21.0	-22.9	19.5			21.4				1.9		
24W	76.83	Shaft 2		I	4	-1.5	-1.5	-21.0	-23.5	19.5			22.0				2.5		
24W	76.83	Shaft 3		I	4	-1.5	-1.5	-21.0	-23.1	19.5			21.6				2.1		
24W	76.83	Shaft 4		I	4	-1.5	-1.5	-21.0	-22.5	19.5			21.0			21.5	1.5		
25W	78.27	Shaft 1		I	4	-1.5	-1.5	-32.0	-32.6	30.5			31.1				0.6		
25W	78.27	Shaft 2		I	4	-1.5	-1.5	-32.0	-33.4	30.5			31.9				1.4		
25W	78.27	Shaft 3		I	4	-1.5	-1.5	-32.0	-33.0	30.5			31.5				1.0		
25W	78.27	Shaft 4		I	4	-1.5	-1.5	-32.0	-33.0	30.5			31.5			31.5	1.0		
26W	79.71	Shaft 1	Statnamic	I	4	-1.5	-1.5	-24.0	-24.7	22.5			23.2				0.7		
26W	79.71	Shaft 2	Osterberg	I	4	-1.5	-1.5	-24.0	-30.5	22.5			29.0				6.5		
26W	79.71	Shaft 3		I	4	-1.5	-1.5	-24.0	-24.4	22.5			22.9				0.4		
26W	79.71	Shaft 4		I	4	-1.5	-1.5	-24.0	-24.1	22.5			22.6			24.4	0.1		
27 W	81.15	Shaft 1		I	4	-1.5	-1.5	-42.0	-43.5	40.5			42.0				1.5		
27W	81.15	Shaft 2		I	4	-1.5	-1.5	-42.0	-41.5	40.5			40.0				-0.5		
27W	81.15	Shaft 3		I	4	-1.5	-1.5	-42.0	-41.9	40.5			40.4				-0.1		
27W	81.15	Shaft 4		I	4	-1.5	-1.5	-42.0	-41.7	40.5			40.2			40.7	-0.3		
28W	82.59	Shaft 1		I	4	-1.5	-1.5	-47.0	-47.2	45.5			45.7				0.2		
28W	82.59	Shaft 2		I	4	-1.5	-1.5	-47.0	-47.5	45.5			46.0				0.5		
28W	82.59	Shaft 3		I	4	-1.5	-1.5	-47.0	-47.4	45.5			45.9				0.4		
28W	82.59	Shaft 4		I	4	-1.5	-1.5	-47.0	-47.6	45.5			46.1			45.9	0.6		
29W	84.03	Shaft 1		I	4	-1.5	-1.5	-37.0	-42.1	35.5			40.6				5.1		
29W	84.03	Shaft 2		I	4	-1.5	-1.5	-37.0	-37.5	35.5			36.0				0.5		
29W	84.03	Shaft 3		I	4	-1.5	-1.5	-37.0	-37.4	35.5			35.9				0.4		
29W	84.03	Shaft 4		I	4	-1.5	-1.5	-37.0	-37.8	35.5			36.3			37.2	0.8		
30W	85.47	Shaft 1		I	4	-1.5	-1.5	-38.0	-38.3	36.5			36.8				0.3		
30W	85.47	Shaft 2		I	4	-1.5	-1.5	-38.0	-38.4	36.5			36.9				0.4		
30W	85.47	Shaft 3		I	4	-1.5	-1.5	-38.0	-38.4	36.5			36.9				0.4		
30W	85.47	Shaft 4		I	4	-1.5	-1.5	-38.0	-38.2	36.5			36.7			36.8	0.2		
31W	86.91	Shaft 1		I	4	-1.5	-1.5	-35.0	-34.9	33.5			33.4				-0.1		
31W	86.91	Shaft 2		I	4	-1.5	-1.5	-35.0	-35.4	33.5			33.9				0.4		
31W	86.91	Shaft 3		I	4	-1.5	-1.5	-35.0	-35.3	33.5			33.8				0.3		
31W	86.91	Shaft 4		I	4	-1.5	-1.5	-35.0	-35.1	33.5			33.6			33.7	0.1		
32W	88.35	Shaft 1		I	4	-1.5	-1.5	-32.0	-37.0	30.5			35.5				5.0		
32W	88.35	Shaft 2		I	4	-1.5	-1.5	-32.0	-37.6	30.5			36.1				5.6		
32W	88.35	Shaft 3		I	4	-1.5	-1.5	-32.0	-37.0	30.5			35.5				5.0		
32W	88.35	Shaft 4		I	4	-1.5	-1.5	-32.0	-32.4	30.5			30.9			34.5	0.4		

Gandy (As Built vs. Planned Shaft Elevation)

SHAFTS INFORMATION																			
Pier No.	Center Station	Shaft	Test Shaft	Diameter		Planned Top Elevation	Actual Top Elevation	Planned Tip Elevation	Actual Tip Elevation	Planned Shaft Length (feet)			As Built Shaft Length (feet)			Ave. Length (As Built)	Difference (As Built - Planned) (feet)		
-	(100 ft.)	-	-	Type	feet	feet	feet	feet	feet	4 feet	6 feet	7 feet	4 feet	6 feet	7 feet	feet	4 feet	6 feet	7 feet
33W	89.79	Shaft 1		I	4	-1.5	-1.5	-24.0	-23.9	22.5			22.4				-0.1		
33W	89.79	Shaft 2		I	4	-1.5	-1.5	-24.0	-24.0	22.5			22.5				0.0		
33W	89.79	Shaft 3		I	4	-1.5	-1.5	-24.0	-24.0	22.5			22.5				0.0		
33W	89.79	Shaft 4		I	4	-1.5	-1.5	-24.0	-24.0	22.5			22.5			22.5	0.0		
34W	91.23	Shaft 1		I	4	-1.5	-1.5	-22.0	-22.5	20.5			21.0				0.5		
34W	91.23	Shaft 2		I	4	-1.5	-1.5	-22.0	-22.8	20.5			21.3				0.8		
34W	91.23	Shaft 3		I	4	-1.5	-1.5	-22.0	-22.6	20.5			21.1				0.6		
34W	91.23	Shaft 4		I	4	-1.5	-1.5	-22.0	-22.6	20.5			21.1			21.1	0.6		
35W	92.67	Shaft 1		I	4	-1.5	-1.5	-25.0	-27.0	23.5			25.5				2.0		
35W	92.67	Shaft 2		I	4	-1.5	-1.5	-25.0	-25.1	23.5			23.6				0.1		
35W	92.67	Shaft 3		I	4	-1.5	-1.5	-25.0	-33.0	23.5			31.5				8.0		
35W	92.67	Shaft 4		I	4	-1.5	-1.5	-25.0	-25.8	23.5			24.3			26.2	0.8		
36W	94.11	Shaft 1		I	4	-1.5	-1.5	-33.0	-33.6	31.5			32.1				0.6		
36W	94.11	Shaft 2		I	4	-1.5	-1.5	-33.0	-33.4	31.5			31.9				0.4		
36W	94.11	Shaft 3		I	4	-1.5	-1.5	-33.0	-35.5	31.5			34.0				2.5		
36W	94.11	Shaft 4		I	4	-1.5	-1.5	-33.0	-33.8	31.5			32.3			32.6	0.8		
37W	95.55	Shaft 1		I	4	-1.5	-1.5	-35.0	-34.8	33.5			33.3				-0.2		
37W	95.55	Shaft 2		I	4	-1.5	-1.5	-35.0	-46.7	33.5			45.2				11.7		
37W	95.55	Shaft 3		I	4	-1.5	-1.5	-35.0	-35.5	33.5			34.0				0.5		
37W	95.55	Shaft 4		I	4	-1.5	-1.5	-35.0	-45.7	33.5			44.2			39.2	10.7		
38W	96.99	Shaft 1		I	4	-1.5	-1.5	-41.0	-41.6	39.5			40.1				0.6		
38W	96.99	Shaft 2		I	4	-1.5	-1.5	-41.0	-41.7	39.5			40.2				0.7		
38W	96.99	Shaft 3		I	4	-1.5	-1.5	-41.0	-41.3	39.5			39.8				0.3		
38W	96.99	Shaft 4		I	4	-1.5	-1.5	-41.0	-47.7	39.5			46.2			41.6	6.7		
39W	98.43	Shaft 1		I	4	-1.5	-1.5	-46.0	-46.9	44.5			45.4				0.9		
39W	98.43	Shaft 2		I	4	-1.5	-1.5	-46.0	-46.3	44.5			44.8				0.3		
39W	98.43	Shaft 3		I	4	-1.5	-1.5	-46.0	-45.5	44.5			44.0				-0.5		
39W	98.43	Shaft 4		I	4	-1.5	-1.5	-46.0	-46.0	44.5			44.5			44.7	0.0		
40W	99.87	Shaft 1		I	4	-1.5	-1.5	-45.0	-45.1	43.5			43.6				0.1		
40W	99.87	Shaft 2		I	4	-1.5	-1.5	-45.0	-45.2	43.5			43.7				0.2		
40W	99.87	Shaft 3		I	4	-1.5	-1.5	-45.0	-45.5	43.5			44.0				0.5		
40W	99.87	Shaft 4		I	4	-1.5	-1.5	-45.0	-45.2	43.5			43.7			43.8	0.2		
41W	101.31	Shaft 1		I	4	-1.5	-1.5	-57.0	-57.2	55.5			55.7				0.2		
41W	101.31	Shaft 2		I	4	-1.5	-1.5	-57.0	-57.2	55.5			55.7				0.2		
41W	101.31	Shaft 3		I	4	-1.5	-1.5	-57.0	-57.4	55.5			55.9				0.4		
41W	101.31	Shaft 4		I	4	-1.5	-1.5	-57.0	-57.4	55.5			55.9			55.8	0.4		
42W	102.75	Shaft 1		I	4	-1.5	-1.5	-50.0	-50.5	48.5			49.0				0.5		
42W	102.75	Shaft 2		I	4	-1.5	-1.5	-50.0	-50.0	48.5			48.5				0.0		
42W	102.75	Shaft 3		I	4	-1.5	-1.5	-50.0	-50.4	48.5			48.9				0.4		
42W	102.75	Shaft 4		I	4	-1.5	-1.5	-50.0	-50.1	48.5			48.6			48.8	0.1		

Gandy (As Built vs. Planned Shaft Elevation)

SHAFTS INFORMATION																			
Pier No.	Center Station	Shaft	Test Shaft	Diameter		Planned Top Elevation	Actual Top Elevation	Planned Tip Elevation	Actual Tip Elevation	Planned Shaft Length (feet)			As Built Shaft Length (feet)			Ave. Length (As Built)	Difference (As Built - Planned) (feet)		
-	(100 ft.)	-	-	Type	feet	feet	feet	feet	feet	4 feet	6 feet	7 feet	4 feet	6 feet	7 feet	feet	4 feet	6 feet	7 feet
43W	104.19	Shaft 1		I	4	-1.5	-1.5	-42.0	-41.8	40.5			40.3				-0.2		
43W	104.19	Shaft 2		I	4	-1.5	-1.5	-42.0	-42.2	40.5			40.7				0.2		
43W	104.19	Shaft 3		I	4	-1.5	-1.5	-42.0	-42.0	40.5			40.5				0.0		
43W	104.19	Shaft 4		I	4	-1.5	-1.5	-42.0	-42.0	40.5			40.5			40.5	0.0		
44W	105.63	Shaft 1		I	4	-1.5	-1.5	-38.0	-38.0	36.5			36.5				0.0		
44W	105.63	Shaft 2		I	4	-1.5	-1.5	-38.0	-38.0	36.5			36.5				0.0		
44W	105.63	Shaft 3		I	4	-1.5	-1.5	-38.0	-38.2	36.5			36.7				0.2		
44W	105.63	Shaft 4		I	4	-1.5	-1.5	-38.0	-38.0	36.5			36.5			36.6	0.0		
45W	107.07	Shaft 1		I	4	-1.5	-1.5	-44.0	-44.3	42.5			42.8				0.3		
45W	107.07	Shaft 2		I	4	-1.5	-1.5	-44.0	-44.5	42.5			43.0				0.5		
45W	107.07	Shaft 3		I	4	-1.5	-1.5	-44.0	-44.0	42.5			42.5				0.0		
45W	107.07	Shaft 4		I	4	-1.5	-1.5	-44.0	-44.2	42.5			42.7			42.8	0.2		
46W	108.51	Shaft 1		I	4	-1.5	-1.5	-41.0	-41.0	39.5			39.5				0.0		
46W	108.51	Shaft 2		I	4	-1.5	-1.5	-41.0	-41.1	39.5			39.6				0.1		
46W	108.51	Shaft 3		I	4	-1.5	-1.5	-41.0	-41.0	39.5			39.5				0.0		
46W	108.51	Shaft 4		I	4	-1.5	-1.5	-41.0	-41.2	39.5			39.7			39.6	0.2		
47W	109.95	Shaft 1		I	4	-1.5	-1.5	-40.0	-40.1	38.5			38.6				0.1		
47W	109.95	Shaft 2		I	4	-1.5	-1.5	-40.0	-40.1	38.5			38.6				0.1		
47W	109.95	Shaft 3		I	4	-1.5	-1.5	-40.0	-40.2	38.5			38.7				0.2		
47W	109.95	Shaft 4		I	4	-1.5	-1.5	-40.0	-40.2	38.5			38.7			38.7	0.2		
48W	111.39	Shaft 1		I	4	-1.5	-1.5	-44.0	-44.1	42.5			42.6				0.1		
48W	111.39	Shaft 2		I	4	-1.5	-1.5	-44.0	-44.1	42.5			42.6				0.1		
48W	111.39	Shaft 3		I	4	-1.5	-1.5	-44.0	-44.7	42.5			43.2				0.7		
48W	111.39	Shaft 4		I	4	-1.5	-1.5	-44.0	-44.0	42.5			42.5			42.7	0.0		
49W	112.83	Shaft 1		I	4	-1.5	-1.5	-56.0	-38.0	54.5			36.5				-18.0		
49W	112.83	Shaft 2		I	4	-1.5	-1.5	-56.0	-31.2	54.5			29.7				-24.8		
49W	112.83	Shaft 3		I	4	-1.5	-1.5	-56.0	-56.1	54.5			54.6				0.1		
49W	112.83	Shaft 4		I	4	-1.5	-1.5	-56.0	-56.7	54.5			55.2			44.0	0.7		
50W	114.27	Shaft 1		I	4	-1.5	-1.5	-61.0	-46.2	59.5			44.7				-14.8		
50W	114.27	Shaft 2		I	4	-1.5	-1.5	-61.0	-43.4	59.5			41.9				-17.6		
50W	114.27	Shaft 3		I	4	-1.5	-1.5	-61.0	-40.0	59.5			38.5				-21.0		
50W	114.27	Shaft 4		I	4	-1.5	-1.5	-61.0	-61.1	59.5			59.6			46.2	0.1		
51W	115.71	Shaft 1		I	4	-1.5	-1.5	-47.0	-47.0	45.5			45.5				0.0		
51W	115.71	Shaft 2		I	4	-1.5	-1.5	-47.0	-47.4	45.5			45.9				0.4		
51W	115.71	Shaft 3		I	4	-1.5	-1.5	-47.0	-47.2	45.5			45.7				0.2		
51W	115.71	Shaft 4		I	4	-1.5	-1.5	-47.0	-47.6	45.5			46.1			45.8	0.6		
52W	117.15	Shaft 1		I	4	-1.5	-1.5	-46.0	-46.3	44.5			44.8				0.3		
52W	117.15	Shaft 2		I	4	-1.5	-1.5	-46.0	-46.1	44.5			44.6				0.1		
52W	117.15	Shaft 3	Statnamic	I	4	-1.5	-1.5	-46.0	-46.3	44.5			44.8				0.3		
52W	117.15	Shaft 4	Osterberg	I	4	-1.5	-1.5	-46.0	-47.9	44.5			46.4			45.2	1.9		

Gandy (As Built vs. Planned Shaft Elevation)

SHAFTS INFORMATION																			
Pier No.	Center Station	Shaft	Test Shaft	Diameter		Planned Top Elevation	Actual Top Elevation	Planned Tip Elevation	Actual Tip Elevation	Planned Shaft Length (feet)			As Built Shaft Length (feet)			Ave. Length (As Built)	Difference (As Built - Planned) (feet)		
-	(100 ft.)	-	-	Type	feet	feet	feet	feet	feet	4 feet	6 feet	7 feet	4 feet	6 feet	7 feet	feet	4 feet	6 feet	7 feet
53W	118.59	Shaft 1		I	4	-1.5	-1.5	-47.0	-48.0	45.5			46.5				1.0		
53W	118.59	Shaft 2		I	4	-1.5	-1.5	-47.0	-48.2	45.5			46.7				1.2		
53W	118.59	Shaft 3		I	4	-1.5	-1.5	-47.0	-47.8	45.5			46.3				0.8		
53W	118.59	Shaft 4		I	4	-1.5	-1.5	-47.0	-47.8	45.5			46.3			46.5	0.8		
54W	120.03	Shaft 1		II	6	-1.5	-1.5	-42.0	-42.0		40.5			40.5				0.0	
54W	120.03	Shaft 2		II	6	-1.5	-1.5	-42.0	-42.0		40.5			40.5				0.0	
54W	120.03	Shaft 3		II	6	-1.5	-1.5	-42.0	-41.8		40.5			40.3				-0.2	
54W	120.03	Shaft 4		II	6	-1.5	-1.5	-42.0	-42.0		40.5			40.5		40.5		0.0	
55W	121.47	Shaft 1		II	6	-1.5	-1.5	-46.0	-58.9		44.5			57.4				12.9	
55W	121.47	Shaft 2		II	6	-1.5	-1.5	-46.0	-65.6		44.5			64.1				19.6	
55W	121.47	Shaft 3		II	6	-1.5	-1.5	-46.0	-46.0		44.5			44.5				0.0	
55W	121.47	Shaft 4		II	6	-1.5	-1.5	-46.0	-46.2		44.5			44.7		52.7		0.2	
56W	122.91	Shaft 1		II	6	-1.5	-1.5	-36.0	-50.8		34.5			49.3				14.8	
56W	122.91	Shaft 2		II	6	-1.5	-1.5	-36.0	-50.1		34.5			48.6				14.1	
56W	122.91	Shaft 3		II	6	-1.5	-1.5	-36.0	-41.3		34.5			39.8				5.3	
56W	122.91	Shaft 4		II	6	-1.5	-1.5	-36.0	-52.4		34.5			50.9		47.2		16.4	
57W	124.35	Shaft 1		II	6	-1.5	-1.5	-41.0	-41.0		39.5			39.5				0.0	
57W	124.35	Shaft 2		II	6	-1.5	-1.5	-41.0	-41.4		39.5			39.9				0.4	
57W	124.35	Shaft 3		II	6	-1.5	-1.5	-41.0	-41.1		39.5			39.6				0.1	
57W	124.35	Shaft 4		II	6	-1.5	-1.5	-41.0	-41.0		39.5			39.5		39.6		0.0	
58W	125.79	Shaft 1		II	6	-1.5	-1.5	-32.0	-32.1		30.5			30.6				0.1	
58W	125.79	Shaft 2		II	6	-1.5	-1.5	-32.0	-32.2		30.5			30.7				0.2	
58W	125.79	Shaft 3		II	6	-1.5	-1.5	-32.0	-33.8		30.5			32.3				1.8	
58W	125.79	Shaft 4		II	6	-1.5	-1.5	-32.0	-32.0		30.5			30.5		31.0		0.0	
59 W	127.23	Shaft 1		II	6	-1.5	-1.5	-32.0	-39.1		30.5			37.6				7.1	
59W	127.23	Shaft 2		II	6	-1.5	-1.5	-32.0	-42.0		30.5			40.5				10.0	
59W	127.23	Shaft 3		II	6	-1.5	-1.5	-32.0	-38.3		30.5			36.8				6.3	
59W	127.23	Shaft 4		II	6	-1.5	-1.5	-32.0	-37.7		30.5			36.2		37.8		5.7	
60W	128.67	Shaft 1		I	6	-3.5	-3.5	-33.0	-33.3		29.5			29.8				0.3	
60W	128.67	Shaft 2		I	6	-3.5	-3.5	-33.0	-33.0		29.5			29.5				0.0	
60W	128.67	Shaft 3		I	6	-3.5	-3.5	-33.0	-52.5		29.5			49.0				19.5	
60W	128.67	Shaft 4		I	6	-3.5	-3.5	-33.0	-33.3		29.5			29.8		34.5		0.3	
61W	130.11	Shaft 1		I	6	-3.5	-3.5	-35.0	-35.1		31.5			31.6				0.1	
61W	130.11	Shaft 2		I	6	-3.5	-3.5	-35.0	-35.2		31.5			31.7				0.2	
61W	130.11	Shaft 3		I	6	-3.5	-3.5	-35.0	-35.2		31.5			31.7				0.2	
61W	130.11	Shaft 4		I	6	-3.5	-3.5	-35.0	-35.0		31.5			31.5		31.6		0.0	
62W	131.55	Shaft 1		I	6	-3.5	-3.5	-46.0	-46.5		42.5			43.0				0.5	
62W	131.55	Shaft 2		I	6	-3.5	-3.5	-46.0	-45.5		42.5			42.0				-0.5	
62W	131.55	Shaft 3		I	6	-3.5	-3.5	-46.0	-44.4		42.5			40.9				-1.6	
62W	131.55	Shaft 4		I	6	-3.5	-3.5	-46.0	-46.0		42.5			42.5		42.1		0.0	

Gandy (As Built vs. Planned Shaft Elevation)

SHAFTS INFORMATION																			
Pier No.	Center Station	Shaft	Test Shaft	Diameter		Planned Top Elevation	Actual Top Elevation	Planned Tip Elevation	Actual Tip Elevation	Planned Shaft Length (feet)			As Built Shaft Length (feet)			Ave. Length (As Built)	Difference (As Built - Planned) (feet)		
-	(100 ft.)	-	-	Type	feet	feet	feet	feet	feet	4 feet	6 feet	7 feet	4 feet	6 feet	7 feet	feet	4 feet	6 feet	7 feet
63W	132.99	Shaft 1		I	6	-3.5	-3.5	-47.0	-50.7		43.5			47.2				3.7	
63W	132.99	Shaft 2		I	6	-3.5	-3.5	-47.0	-47.3		43.5			43.8				0.3	
63W	132.99	Shaft 3		I	6	-3.5	-3.5	-47.0	-47.3		43.5			43.8				0.3	
63W	132.99	Shaft 4		I	6	-3.5	-3.5	-47.0	-58.4		43.5			54.9		47.4		11.4	
64W	134.43	Shaft 1		I	6	-3.5	-3.5	-36.0	-44.7		32.5			41.2				8.7	
64W	134.43	Shaft 2		I	6	-3.5	-3.5	-36.0	-39.7		32.5			36.2				3.7	
64W	134.43	Shaft 3		I	6	-3.5	-3.5	-36.0	-46.1		32.5			42.6				10.1	
64W	134.43	Shaft 4		I	6	-3.5	-3.5	-36.0	-36.0		32.5			32.5		38.1		0.0	
65W	135.87	Shaft 1		I	6	-3.5	-3.5	-39.0	-39.1		35.5			35.6				0.1	
65W	135.87	Shaft 2		I	6	-3.5	-3.5	-39.0	-57.4		35.5			53.9				18.4	
65W	135.87	Shaft 3		I	6	-3.5	-3.5	-39.0	-39.0		35.5			35.5				0.0	
65W	135.87	Shaft 4		I	6	-3.5	-3.5	-39.0	-39.9		35.5			36.4		40.4		0.9	
66W	137.31	Shaft 1		I	6	-3.5	-3.5	-41.0	-41.6		37.5			38.1				0.6	
66W	137.31	Shaft 2		I	6	-3.5	-3.5	-41.0	-41.2		37.5			37.7				0.2	
66W	137.31	Shaft 3		I	6	-3.5	-3.5	-41.0	-41.5		37.5			38.0				0.5	
66W	137.31	Shaft 4		I	6	-3.5	-3.5	-41.0	-43.2		37.5			39.7		38.4		2.2	
67W	138.75	Shaft 1		I	6	-3.5	-3.5	-68.0	-52.8		64.5			49.3				-15.2	
67W	138.75	Shaft 2		I	6	-3.5	-3.5	-68.0	-63.8		64.5			60.3				-4.2	
67W	138.75	Shaft 3		I	6	-3.5	-3.5	-68.0	-43.0		64.5			39.5				-25.0	
67W	138.75	Shaft 4		I	6	-3.5	-3.5	-68.0	-64.8		64.5			61.3		52.6		-3.2	
68W	140.19	Shaft 1		I	6	-3.5	-3.5	-45.0	-45.0		41.5			41.5				0.0	
68W	140.19	Shaft 2		I	6	-3.5	-3.5	-45.0	-46.8		41.5			43.3				1.8	
68W	140.19	Shaft 3		I	6	-3.5	-3.5	-45.0	-68.3		41.5			64.8				23.3	
68W	140.19	Shaft 4		I	6	-3.5	-3.5	-45.0	-80.2		41.5			76.7		56.6		35.2	
69W	141.63	Shaft 1		I	6	-3.5	-3.5	-41.0	-84.7		37.5			81.2				43.7	
69W	141.63	Shaft 2		I	6	-3.5	-3.5	-41.0	-85.3		37.5			81.8				44.3	
69W	141.63	Shaft 3		I	6	-3.5	-3.5	-41.0	-86.7		37.5			83.2				45.7	
69W	141.63	Shaft 4		I	6	-3.5	-3.5	-41.0	-63.8		37.5			60.3		76.6		22.8	
70W	143.07	Shaft 1		I	7	-3.5	-3.5	-65.0	-66.2			61.5			62.7				1.2
70W	143.07	Shaft 2		I	7	-3.5	-3.5	-65.0	-65.3			61.5			61.8				0.3
70W	143.07	Shaft 3		I	7	-3.5	-3.5	-65.0	-65.7			61.5			62.2				0.7
70W	143.07	Shaft 4		I	7	-3.5	-3.5	-65.0	-65.8			61.5			62.3	62.3			0.8
71W	144.51	Shaft 1		I	7	-3.5	-3.5	-73.0	-73.1			69.5			69.6				0.1
71W	144.51	Shaft 2		I	7	-3.5	-3.5	-73.0	-73.8			69.5			70.3				0.8
71W	144.51	Shaft 3		I	7	-3.5	-3.5	-73.0	-73.7			69.5			70.2				0.7
71W	144.51	Shaft 4		I	7	-3.5	-3.5	-73.0	-73.5			69.5			70.0	70.0			0.5
72W	145.95	Shaft 1		I	7	-3.5	-3.5	-70.0	-71.0			66.5			67.5				1.0
72W	145.95	Shaft 2		I	7	-3.5	-3.5	-70.0	-69.8			66.5			66.3				-0.2
72W	145.95	Shaft 3		I	7	-3.5	-3.5	-70.0	-70.6			66.5			67.1				0.6
72W	145.95	Shaft 4		I	7	-3.5	-3.5	-70.0	-70.4			66.5			66.9	67.0			0.4

Gandy (As Built vs. Planned Shaft Elevation)

SHAFTS INFORMATION																				
Pier No.	Center Station	Shaft	Test Shaft	Diameter		Planned Top Elevation	Actual Top Elevation	Planned Tip Elevation	Actual Tip Elevation	Planned Shaft Length (feet)			As Built Shaft Length (feet)			Ave. Length (As Built)	Difference (As Built - Planned) (feet)		
-	(100 ft.)	-	-	Type	feet	feet	feet	feet	feet	4 feet	6 feet	7 feet	4 feet	6 feet	7 feet	feet	4 feet	6 feet	7 feet
73W	147.39	Shaft 1		I	7	-3.5	-3.5	-67.0	-67.0			63.5			63.5				0.0
73W	147.39	Shaft 2		I	7	-3.5	-3.5	-67.0	-67.4			63.5			63.9				0.4
73W	147.39	Shaft 3		I	7	-3.5	-3.5	-67.0	-67.5			63.5			64.0				0.5
73W	147.39	Shaft 4		I	7	-3.5	-3.5	-67.0	-67.2			63.5			63.7	63.8			0.2
74W	148.83	Shaft 1		I	7	-3.5	-3.5	-64.0	-64.0			60.5			60.5				0.0
74W	148.83	Shaft 2		I	7	-3.5	-3.5	-64.0	-64.1			60.5			60.6				0.1
74W	148.83	Shaft 3		I	7	-3.5	-3.5	-64.0	-64.0			60.5			60.5				0.0
74W	148.83	Shaft 4		I	7	-3.5	-3.5	-64.0	-64.0			60.5			60.5	60.5			0.0
75W	151.17	Shaft 1		I	7	-3.5	-3.5	-61.0	-63.6			57.5			60.1				2.6
75W	151.17	Shaft 2		I	7	-3.5	-3.5	-61.0	-62.7			57.5			59.2				1.7
75W	151.17	Shaft 3		I	7	-3.5	-3.5	-61.0	-61.7			57.5			58.2				0.7
75W	151.17	Shaft 4		I	7	-3.5	-3.5	-61.0	-61.4			57.5			57.9	58.9			0.4
76W	152.61	Shaft 1		I	7	-3.5	-3.5	-53.0	-54.9			49.5			51.4				1.9
76W	152.61	Shaft 2		I	7	-3.5	-3.5	-53.0	-58.3			49.5			54.8				5.3
76W	152.61	Shaft 3		I	7	-3.5	-3.5	-53.0	-58.3			49.5			54.8				5.3
76W	152.61	Shaft 4		I	7	-3.5	-3.5	-53.0	-54.7			49.5			51.2	53.1			1.7
77W	154.05	Shaft 1		I	7	-3.5	-3.5	-39.0	-43.5			35.5			40.0				4.5
77W	154.05	Shaft 2		I	7	-3.5	-3.5	-39.0	-44.7			35.5			41.2				5.7
77W	154.05	Shaft 3		I	7	-3.5	-3.5	-39.0	-40.1			35.5			36.6				1.1
77W	154.05	Shaft 4		I	7	-3.5	-3.5	-39.0	-43.2			35.5			39.7	39.4			4.2
78W	155.49	Shaft 1		I	7	-3.5	-3.5	-39.0	-44.4			35.5			40.9				5.4
78W	155.49	Shaft 2		I	7	-3.5	-3.5	-39.0	-41.4			35.5			37.9				2.4
78W	155.49	Shaft 3		I	7	-3.5	-3.5	-39.0	-45.7			35.5			42.2				6.7
78W	155.49	Shaft 4		I	7	-3.5	-3.5	-39.0	-40.8			35.5			37.3	39.6			1.8
79W	156.93	Shaft 1		I	7	-3.5	-3.5	-53.0	-53.7			49.5			50.2				0.7
79W	156.93	Shaft 2		I	7	-3.5	-3.5	-53.0	-53.3			49.5			49.8				0.3
79W	156.93	Shaft 3		I	7	-3.5	-3.5	-53.0	-54.0			49.5			50.5				1.0
79W	156.93	Shaft 4		I	7	-3.5	-3.5	-53.0	-54.8			49.5			51.3	50.5			1.8
80W	158.37	Shaft 1		I	6	-3.5	-3.5	-59.0	-70.0		55.5			66.5				11.0	
80W	158.37	Shaft 2		I	6	-3.5	-3.5	-59.0	-60.0		55.5			56.5				1.0	
80W	158.37	Shaft 3		I	6	-3.5	-3.5	-59.0	-58.8		55.5			55.3				-0.2	
80W	158.37	Shaft 4		I	6	-3.5	-3.5	-59.0	-59.0		55.5			55.5		58.5		0.0	
81W	159.81	Shaft 1		I	6	-3.5	-3.5	-51.0	-50.7		47.5			47.2				-0.3	
81W	159.81	Shaft 2		I	6	-3.5	-3.5	-51.0	-57.6		47.5			54.1				6.6	
81W	159.81	Shaft 3		I	6	-3.5	-3.5	-51.0	-51.0		47.5			47.5				0.0	
81W	159.81	Shaft 4		I	6	-3.5	-3.5	-51.0	-51.0		47.5			47.5		49.1		0.0	
82W	161.25	Shaft 1		I	6	-3.5	-3.5	-48.0	-48.1		44.5			44.6				0.1	
82W	161.25	Shaft 2		I	6	-3.5	-3.5	-48.0	-48.0		44.5			44.5				0.0	
82W	161.25	Shaft 3		I	6	-3.5	-3.5	-48.0	-48.2		44.5			44.7				0.2	
82W	161.25	Shaft 4		I	6	-3.5	-3.5	-48.0	-48.1		44.5			44.6		44.6		0.1	

Gandy (As Built vs. Planned Shaft Elevation)

SHAFTS INFORMATION																			
Pier No.	Center Station	Shaft	Test Shaft	Diameter		Planned Top Elevation	Actual Top Elevation	Planned Tip Elevation	Actual Tip Elevation	Planned Shaft Length (feet)			As Built Shaft Length (feet)			Ave. Length (As Built)	Difference (As Built - Planned) (feet)		
-	(100 ft.)	-	-	Type	feet	feet	feet	feet	feet	4 feet	6 feet	7 feet	4 feet	6 feet	7 feet	feet	4 feet	6 feet	7 feet
83W	162.69	Shaft 1		I	6	-3.5	-3.5	-50.0	-50.4		46.5			46.9				0.4	
83W	162.69	Shaft 2		I	6	-3.5	-3.5	-50.0	-50.7		46.5			47.2				0.7	
83W	162.69	Shaft 3		I	6	-3.5	-3.5	-50.0	-50.7		46.5			47.2				0.7	
83W	162.69	Shaft 4		I	6	-3.5	-3.5	-50.0	-50.2		46.5			46.7		47.0		0.2	
84W	164.13	Shaft 1		I	6	-3.5	-3.5	-67.0	-67.0		63.5			63.5				0.0	
84W	164.13	Shaft 2		I	6	-3.5	-3.5	-67.0	-67.1		63.5			63.6				0.1	
84W	164.13	Shaft 3		I	6	-3.5	-3.5	-67.0	-67.0		63.5			63.5				0.0	
84W	164.13	Shaft 4		I	6	-3.5	-3.5	-67.0	-67.7		63.5			64.2		63.7		0.7	
85W	165.57	Shaft 1		I	6	-3.5	-3.5	-53.0	-53.1		49.5			49.6				0.1	
85W	165.57	Shaft 2		I	6	-3.5	-3.5	-53.0	-53.6		49.5			50.1				0.6	
85W	165.57	Shaft 3		I	6	-3.5	-3.5	-53.0	-52.7		49.5			49.2				-0.3	
85W	165.57	Shaft 4		I	6	-3.5	-3.5	-53.0	-53.3		49.5			49.8		49.7		0.3	
86W	167.01	Shaft 1		I	6	-3.5	-3.5	-56.0	-56.0		52.5			52.5				0.0	
86W	167.01	Shaft 2		I	6	-3.5	-3.5	-56.0	-56.8		52.5			53.3				0.8	
86W	167.01	Shaft 3		I	6	-3.5	-3.5	-56.0	-56.3		52.5			52.8				0.3	
86W	167.01	Shaft 4		I	6	-3.5	-3.5	-56.0	-56.0		52.5			52.5		52.8		0.0	
87W	168.45	Shaft 1		I	6	-3.5	-3.5	-70.0	-70.3		66.3			66.8				0.5	
87W	168.45	Shaft 2		I	6	-3.5	-3.5	-70.0	-70.0		66.3			66.5				0.3	
87W	168.45	Shaft 3		I	6	-3.5	-3.5	-70.0	-70.0		66.3			66.5				0.3	
87W	168.45	Shaft 4		I	6	-3.5	-3.5	-70.0	-74.6		66.3			71.1		67.7		4.8	
88W	169.89	Shaft 1		I	6	-3.5	-3.5	-61.0	-61.1		57.5			57.6				0.1	
88W	169.89	Shaft 2		I	6	-3.5	-3.5	-61.0	-61.2		57.5			57.7				0.2	
88W	169.89	Shaft 3		I	6	-3.5	-3.5	-61.0	-60.9		57.5			57.4				-0.1	
88W	169.89	Shaft 4		I	6	-3.5	-3.5	-61.0	-72.1		57.5			68.6		60.3		11.1	
89W	171.33	Shaft 1		I	6	-3.5	-3.5	-59.0	-59.2		55.5			55.7				0.2	
89W	171.33	Shaft 2		I	6	-3.5	-3.5	-59.0	-59.8		55.5			56.3				0.8	
89W	171.33	Shaft 3		I	6	-3.5	-3.5	-59.0	-58.9		55.5			55.4				-0.1	
89W	171.33	Shaft 4		I	6	-3.5	-3.5	-59.0	-59.2		55.5			55.7		55.8		0.2	
90W	172.77	Shaft 1		I	4	-1.5	-1.5	-57.0	-58.6	55.5			57.1				1.6		
90W	172.77	Shaft 2		I	4	-1.5	-1.5	-57.0	-58.1	55.5			56.6				1.1		
90W	172.77	Shaft 3		I	4	-1.5	-1.5	-57.0	-59.4	55.5			57.9				2.4		
90W	172.77	Shaft 4		I	4	-1.5	-1.5	-57.0	-58.9	55.5			57.4			57.3	1.9		
91W	174.21	Shaft 1		I	4	-1.5	-1.5	-61.0	-62.1	59.5			60.6				1.1		
91W	174.21	Shaft 2		I	4	-1.5	-1.5	-61.0	-61.6	59.5			60.1				0.6		
91W	174.21	Shaft 3		I	4	-1.5	-1.5	-61.0	-61.6	59.5			60.1				0.6		
91W	174.21	Shaft 4	Osterberg	I	4	-1.5	-1.5	-61.0	-67.7	59.5			66.2			61.8	6.7		
92W	175.65	Shaft 1		I	4	-1.5	-1.5	-61.0	-61.0	59.5			59.5				0.0		
92W	175.65	Shaft 2		I	4	-1.5	-1.5	-61.0	-62.0	59.5			60.5				1.0		
92W	175.65	Shaft 3		I	4	-1.5	-1.5	-61.0	-62.3	59.5			60.8				1.3		
92W	175.65	Shaft 4		I	4	-1.5	-1.5	-61.0	-68.9	59.5			67.4			62.1	7.9		

Gandy (As Built vs. Planned Shaft Elevation)

SHAFTS INFORMATION																			
Pier No.	Center Station	Shaft	Test Shaft	Diameter		Planned Top Elevation	Actual Top Elevation	Planned Tip Elevation	Actual Tip Elevation	Planned Shaft Length (feet)			As Built Shaft Length (feet)			Ave. Length (As Built)	Difference (As Built - Planned) (feet)		
-	(100 ft.)	-	-	Type	feet	feet	feet	feet	feet	4 feet	6 feet	7 feet	4 feet	6 feet	7 feet	feet	4 feet	6 feet	7 feet
93W	177.09	Shaft 1		I	4	-1.5	-1.5	-61.0	-61.8	59.5			60.3				0.8		
93W	177.09	Shaft 2		I	4	-1.5	-1.5	-61.0	-61.9	59.5			60.4				0.9		
93W	177.09	Shaft 3		I	4	-1.5	-1.5	-61.0	-61.6	59.5			60.1				0.6		
93W	177.09	Shaft 4		I	4	-1.5	-1.5	-61.0	-61.3	59.5			59.8			60.2	0.3		
94W	178.53	Shaft 1		I	4	-1.5	-1.5	-68.0	-68.4	66.3			66.9				0.7		
94W	178.53	Shaft 2		I	4	-1.5	-1.5	-68.0	-68.3	66.3			66.8				0.5		
94W	178.53	Shaft 3		I	4	-1.5	-1.5	-68.0	-68.4	66.3			66.9				0.7		
94W	178.53	Shaft 4		I	4	-1.5	-1.5	-68.0	-68.2	66.3			66.7			66.8	0.5		
95W	179.97	Shaft 1		I	4	-1.5	-1.5	-64.0	-65.2	62.5			63.7				1.2		
95W	179.97	Shaft 2		I	4	-1.5	-1.5	-64.0	-64.7	62.5			63.2				0.7		
95W	179.97	Shaft 3		I	4	-1.5	-1.5	-64.0	-65.0	62.5			63.5				1.0		
95W	179.97	Shaft 4		I	4	-1.5	-1.5	-64.0	-72.8	62.5			71.3			65.4	8.8		
96W	181.41	Shaft 1		I	4	-1.5	-1.5	-76.0	-76.1	74.5			74.6				0.1		
96W	181.41	Shaft 2		I	4	-1.5	-1.5	-76.0	-76.2	74.5			74.7				0.2		
96W	181.41	Shaft 3		I	4	-1.5	-1.5	-76.0	-76.4	74.5			74.9				0.4		
96W	181.41	Shaft 4		I	4	-1.5	-1.5	-76.0	-75.8	74.5			74.3			74.6	-0.2		
97W	182.37	Shaft 1		I	4	-1.5	-1.5	-76.0	-76.4	74.5			74.9				0.4		
97W	182.37	Shaft 2		I	4	-1.5	-1.5	-76.0	-76.2	74.5			74.7				0.2		
97W	182.37	Shaft 3		I	4	-1.5	-1.5	-76.0	-77.1	74.5			75.6				1.1		
97W	182.37	Shaft 4		I	4	-1.5	-1.5	-76.0	-76.6	74.5			75.1			75.1	0.6		
			Total							**10039.0**	**4619.0**	**2196.0**	**10021.1**	**5024.5**	**2259.3**		**-17.9**	**405.5**	**63.3**

APPENDIX E

- Summary of the Unconfined Compressive Strength (qu) and Tensile Strength (qt) Data

  - EL. = Elevation in feet
  - Dep. = Depth in feet
  - REC = Percentage of Recovery (%)
  - RQD = Rock Quality Ratio (%)
  - qu = Unconfined Compressive Strength (tsf)
  - qt = Tensile Strength (tsf)
  - Ei = Initial Young’s Modulus

1. 17^{th} Street

17^{th} Street

State Project No. 86180-1522

BB-1 (32+93)							S-4 (33+00)						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-65				32.2	30	22	-32	211.2	68.3				
-72				27.34	67	28	-36	116.9	19.4				
-85	114.2	13	7										

BB-4 (34+81)							BB-9 (35+06)						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-69	32.74	22	5				-115				26.5	5	
-88	28.8	35	10				-131				24.63	43	
							-131				32.9	43	

S-12 (35+29)							BB-7 (35+44)						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-32	211						-49				43.5	18	4
-32				68.34			-65	414	20	7			
-49	117						-72	37.8					
-49				19.4			-82	120.8	98	38			
-72				19.6			-82				26.3	98	38
							-92	82.98	98	38			
							-102				117.47	66	7
							-108	82.44	35	8			
							-131	140.6	35	10			
							-131				64.6	35	10

BB-11 (35+53)							BB-8 (36+10)						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-36	379.4	38	35	143.97	38	35	-39	361.3	6	5			
-36				189.01	38	38	-46	158.7	48	19	55	48	19
-36				112.6	38	35	-46	272.7	48	19	53.99	48	19
-75				26.3	33		-46				76.78	48	19
-98	27.04	60	23	68.7	60	23	-56	285.02	50	12	40.4	50	12
-98	140.01	60	23				-95				14.04	17	5

BB-10 (36+07)							N-17 (36+19)						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-36	283.4	45.8	13.5	148.76	45.8	13.5	-66	432	33	28			
-36				52.64	45.8	13.5	-66				45.24	33	28
-49	444.3	69	36	49.84	69	36							
-49	212.84	69	36	60.5	69	36							
-49				65.7	69	36							
-56	377.6	84	59										
-56	381.6	84	59										
-56	320.6	84	59										
-105				219.04	31	7							

S-15 (37+10)							N-14 (35+55)						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-49	51.4						-39	389	63	31			
-49				17.5									
-72	38.34												
-72				7.7									

BB-6 (38+07)							N-25 (38+38)						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-49	64.95	42	17				-56	26.54	85	59			
-66	152.5	45	24				-72	281.3	80				
-82	58.2	45	24				-79	331.4	47				
-82				12.4	51	13							
-92	365.4	57	10										
-92				101.4	57	10							
-125	19.2	45	31										
-125				20.95	45	31							
-125	31.04	45	31										

Added Boring (34+81, 10' North+)													
Dep.(ft)	qu	REC	RQD	qt	REC	RQD	Dep.(ft)	qu	REC	RQD	qt	REC	RQD
-53	51.7	50.5	50.5										
	58.5												
	59.9												
	86.1												
-58	58.6												
-58	20.1	47	40.5										
-64	46.1	53	53										
-64	250.1	56.5	53										
	28.2												
	13.5												
	8.4												
-69	9.8												
-77	122.5	33	17										
-77	357.9												
-90	14.5	23.5	13										
-109	41.5	22	13										

2. Acosta

Acosta
State Project No. 87060-1549

WA-1							WA-2						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-35	109	47					-36	50.5	74				
							-37	120	74				

WA-4L							WA-5R						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-36	68	88					-34	52.5	100				
							-37	15	100				

WA-6							WA-7						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-30	82	52					-16	30.5	30				
-33	82	52											

WA-8													
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-40	38.5	60											

Added Boring (130+34, 21' R+)							Added Boring (150+35, 25' R+)						
Dep.(ft)	qu	REC	RQD	qt	REC	RQD	Dep.(ft)	qu	REC	RQD	qt	REC	RQD
-37	99.4	25	13				-22	53.6	100				
	332.1						-27	27.6					
-42	274						-27	16.7	50				
-42	28.8	49	38				-32	13.7					
	34.9												
	23.2												
-47	20.9												

3. Fuller Warren

Fuller Warren (Replacement)
State Project No. 72020-1485

BL-2							BL-4						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-35	96.5	62	32				-15	54.5	80	47			
-35				27.8	62	32	-15				12.5	80	47
-40	70.5	41	8				-20	98.5	43	20			
-40				13.7	41	8	-20				6.1	43	20
							-25	89	100				
							-25				9.1	100	

BL-11							BL-13						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-35	92	25	25				-45	82	87	51			
-35				13.9	25	25	-45				17.2	87	51
-45	68.5	30	17										
-45				24.35	30	17							

BL-20							BL-23						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-35	56.5	25	17				-20	95	50	47			
-35				8.4	25	17	-20				9.9	50	47
							-25	82	48	19			
							-25				12.95	48	19

BL-36							BL-37						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-20	244.5	70	49				-15	258	100				
-20				54.05	70	49	-15				25.2	100	
-25	104	100					-20	22.5	87	47			
							-20				5.6	87	47
							-22	117.5	77	25			

BW-1							BW-3						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-30	92.5	32	17				-45				28.75		
-30				16.75	32	17							

BW-5							BW-12						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-25				44.8			-35				31.85		27
-30				32.75			-40				27.4		
-35				29.95									

BW-14													
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-30	43	20	42										
-30				9.55	20	42							

Added Boring (Hole 1: Near Boring BL-1)							Added Boring (Hole 2: Near Boring BL-1)						
Dep.(ft)	qu	REC	RQD	qt	REC	RQD	Dep.(ft)	qu	REC	RQD	qt	REC	RQD
-20	596.2	38	38				-23	65.4	15	10			
	284						-28	30.8					
	401.6						-28	34.6	52	33			
	17.4							24.6					
	60.2							33.3					
	80.9							14					
-25	110.5							11					
-30	43.4	63	26				-33	58.5					
	32.2												
	52.9												
	87.7												
	15.8												
	6.6												
-35	50.4												
-35	87.7	73	66										
	85.1												
	77.9												
	20.4												
	29.4												
	12												
	23.6												
	17.9												
	18.3												
	18.2												
	47.8												
-40	55.1												

4. Apalachicola

Apalachicola
State Project No. 47010-3519

121+00							122+11						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-40	82.7	90	78				-20	2.8	95	58			
-40				4	90	78	-20				0.8	95	
-50	8.5	100	93				-30	64.2	83				
-50				4.8	100	93	-30				34.2	83	
							-58	5.9	100	50			

123+21							124+30						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-20	5.1	95	72				-25	5.8	60	30			
-20				1	95	72	-25				1.1	60	30
-25	3.4	95	72				-30	6.2	80	40			
-25				1.1	95	72	-30				0.8	80	40
-40	76.9	95	72				-40	10.8	100	50			
							-40				0.9	100	50

124+31							125+41						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-25	6.8	100	75				-15	5.5	80	26			
-25				0.6	100	75	-25	4.4	100	75			
-30	13.45	100	57				-25				0.7	100	75
-30				1.8	100	57	-35	4.5	80	25			
							-35				1.3	80	25
							-50	7.4	100	60			

125+42							126+52						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-15	6.7	100	64				-15	3.6	100	50			
-25	11.4	100	67				-15				1.3	100	50
-25				1.9	100	67	-25	5.2	80	41			
-45	7.9	100	70				-25				0.85	80	41
-45				1.3	100	70							

127+60							127+64						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-15	4.1	80	21				-15	2.9	85	45			
-15				0.7	80	21	-15				0.9	85	45
-30	36.4	70	45				-25	4.4	95	45			
-30				3.7	70	45	-25				0.8	95	45
							-35	208.7	35	25			
							-35				4.1	35	25

128+75							128+73						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-25	10.5	95	36				-25	1.2	100	67			
-30	25.8	95	70				-25				1.8	100	67
-30				6.8	95	36	-30	16.1	100	60			
							-30				2.2	100	60
							-35	4.9	100				

129+84							129+88						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-25	8.5	90	40				-25	5.9	100	90			
-25				0.8	90	40	-25				0.6	100	90
-45	10.5	100	60				-35	8	90	50			
-45				3.5	100	60	-35				0.6	90	50
							-50	66.2	65	65			

132+05							133+16						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-25	5.9	80	50				-25	5.6	100	50			
-25				1	80	50	-30	15.4	80	50			
-35	7.4	95	73				-30				6	80	50
-35				0.6	95	73							

134+26							135+36						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-20	6	100	66				-25	4.3	100	50			
-20				1.2	100	66	-25				0.9	100	50
-45	102.6	80	80										
-45				10.4	80	80							

135+33							136+36						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-20	5.5	95	66				-30	3.55	30	0			
-20				0.7	95	66	-30				1	30	0
-25	4.1	100	25				-40	66.2	30	22			
-25				1.4	100	25	-40				21.9	30	22
-35	89.2	100	25				-50	95.8	67	47			
							-50				8.1	67	47

136+53							136+48						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-25	3.1	62	37				-35	7	45	7			
-25				0.8	62	37	-45	69	50	18			
-50	6.1	85	81				-55	11.4	93	73			
-50				7.8	85	81	-55				1.1	93	73
-60	32.75	92	82				-60	13.7	78	46			
-60				1.4	92	82	-60				1.5	78	46

136+34							138+64						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-30	3.95	80	42				-45				15.7	38	20
-30				1.45	80	42	-55	5.7	65	55			
-35	15.2	80	22				-55				4.4	65	55
-40	9.65	80					-65	7.1	65	23			
-40				1.2	80		-65				0.8	65	23
-50	117.5	45	28										

138+84							138+82						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-30	2	63	47				-25	3.1	75	68			
-30				0.2	63	47	-25				0.6	75	68
-55	150.55	45	42				-40	104.85	27	10			
-55				21.8	45	42	-55				0.8	100	63
-65	5.6	83	72										
-70	12.35	83											
-70				1.3	83								

138+60							141+40						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-45	37.05	37	25				-25	1	65	37			
-45				7.5	37	25	-25				0.2	65	37
-60	0.2	100	82				-55	12.85	40	23			
							-55				4.85	40	23
							-60	5.9	100	50			
							-60				1.2	100	50
							-85	75.35	100	88			

141+67							141+39						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-30	4.2	85	67				-30	3.2	50	45			
-30				0.3	85	67	-30				0.4	50	45
-65	9.95	100	80				-45	60.65	27	13			
-65				1.5	100	80	-60	9.8	100	77			
							-60				3.65	100	77
							-70	14.3	100	85			
							-70				1.9	100	85

143+60							143+88						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-40	16.95	35	8				-52	157.4	50	0			
-40				5.75	35	8	-52				15.9	50	0
-45				15.7	23		-65	6.2	93	82			
-50	60.4	37	7				-65				0.9	93	82
-55	32.8	100	83				-70	4.1	100	82			
-55				0.9	100	83							
-60	8.7	100											
-60				0.85	100								
-65	9	92											

144+81							144+86						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-25	5.3	100	59				-25	5.7	100				
-25				0.8	100	59	-25				0.9	100	
-30	3.5	100	88				-30	4.5	100				
-30				0.4	100	88	-30				0.8	100	
							-40	5.9	90				
							-40				1.2	90	

145+96							147+06						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-30	4.8	68	57				-30	4	75				
-30				1	68	57	-30				1.1	75	
-45	2.8	87	50				-40	2.5	75				
-45				0.5	87	50	-45	174.4	30				

148+15							149+21						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-25	5.7	68	41				-25	2.6	82	50			
-25				0.8	68	41	-25				0.3	82	50
-35	7.7	60	41										
-40	6	45	33										
-40				0.2	45	33							

149+30							150+34						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-45	5.9	73	50				-30	5.2	100	56			
-45				0.7	73	50	-30				0.8	100	56
							-40	5.4	64	34			

151+44							152+54						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-27	6.5	62	35				-25	3.2	30	24			
-27				1.1	62	35	-30	2.4	56	15			
							-40	9.7	53	37			

153+63							153+63						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-25	3.8	82	68				-30	3.9	100				
-25				0.9	82	68	-30				0.4	100	
-35	6.3	58	26										
-35				2.2	58	26							

154+73							154+73						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-30	1.2	100	89				-30	2.6	50	32			
-30				0.4	100	89	-35	8.2	57	13			
							-40				3.2	35	11

155+83							155+83						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-28	3.6	50	40				-32	5.1	60	32			
-42	6.7	100	61				-32				1.2	60	32
-42				0.9	100	61	-40	1.3	28	14			

156+92							157+91						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-30	5.5	80	67				-35	3.4	90	42			
-30				0.6	80	67	-35				0.9	90	42
-35	5.8	40	40										
-45				0.9	35	40							

5. Gandy

Gandy
State Project No. 10130-1544

B-4							B-9						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-40				4.329	92	64	-29				8.2973	84	
-40	48.341	92	64				-30				0.3608	84	
-42	186.87	92	64				-32	2.2	84				
-42				17.677	92	64							
-45	46.176	92											

B-15							B-21						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-30	20.202	100					-20				20.202	100	
-30				3.2468	100		-25	322.51	100	50			
							-25				13.709	100	50

B-26							B-31						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-15	141.41	100	56				-29	36.075	100	28			
-15				5.772	100	56							

B-36							B-42						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-22	49.784	88					-39	38.961	82				
-22				6.8543	88		-39				10.101	82	
-23				9.7	88		-42	28.139	82	72			
-23				5.7	88		-42				4.329	82	72
-24	33.911	88					-46	28.86	100	42			
-25	23.81	88	86				-46				7.215	100	42
-26	28.86	88	42										
-27	46.898	88											
-27				15.873	88								
-28				40.404	88								

B-47							B-52						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-40	25.253	100	90				-40	75.758	90				
-40				5.0505	100	90							
-41	23.088	100											
-41				15.152	100								
-43	74.315	100											
-43				9.0188	100								

B-58							B-63						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-21	58.442	96					-45	169.55	64	29			
-25				3.6075	96	58	-50	514.43	40				
							-50				394.3	40	

B-68							B-74						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-37	46.176	40					-64	627.71	70				
							-64				67.1	70	

B-75							B-86						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-80	18.759	32	14				-50	21.645	96	70			
-80				5.4113	32	14	-51	98.846	96	70			
							-51				16.96	96	70
							-52	167.39	96				
							-56	162.34	90				

B-91							B-96						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-72	4.329	70	52				-71	481.96	68				
-72				1.0823	70	52	-73				41.847	68	
							-74				43.651	68	
							-75	553.39	68				

6. Hillsborough

Hillsborough
State Project No. 10150-3543

BB-1							BB-2						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-49	68.95	70	24				-78	273.3	70				
-58		68		7.6	68		-91	22.6	100				
-59	60.4	100	35										
-62				6.05	100	35							
-83				45.35	84	66							
-86	518.9	84	66										
-89	46.2	84	66										
-123	237.95	100											
-124				59.6	100								

EB1-3							B3-1						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-41	34.9	95					-51	28.65	90	16			
-41				2.23	95		-51				7.27	90	16

P4-5							P4-8						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-50	60	30					-44	1.75	100	74			
-50				4.03	30								
-55				7.27	80								

P4-20							P5-1						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-31	3.3	100	80				-30	30.3	100	80			
-31				2.16	100	80							

EB7-3							B-10						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-35	2.75	100	100				-48				9.65	65	
-35				2.375	100	100	-50	82.4	60				
-37	23.2						-55	86.95	60				
-39				0.935	100	100	-55				8.6	60	

B-11													
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-26.5				7.7	80								
-27				6.85	85								
-27	28.7	60											
-30				7.55	65								
-30				19.15	85								

7. Victory

Victory
State Project No. 53020-3540

TB-3 (90+38)							TB-4 (92+60)						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
35	47.4	50	11				40	52.38	21	8			
30				10.67	40	8	24	18.47	74	48			
30	17.6	40	8										

TB-5 (93+75)							TB-6 (95+50)						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
38	20.71	75	41				37	218.3	67	36			
35	95.67	45	13				37				25	67	36
							25	51.44	91	88			
							-10	138	80	45			

TB-7 (97+68)							TB-8 (98+64)						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
12	36.5	89					33	152.8	100	96			
12				11.8	89			228.4	67	44			
5	15.95												

TB-9 (99+70)							TB-10 (100+75)						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
0	157	100	83				20	239.3	33	12			
							20				15.51	33	12
							10	74.96	57	37			
							-13	164.1	78	32			

TB-11 (101+75)							TB-12 (102+75)						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
27				28.43	100	100	25	83.62	50				
19	127.6	80					5	70.8	60				
10	41.5	100	50										
-5	86.6	90	87										

TB-13 (103+85)							TB-14 (104+80)						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
26	74.96	97	88				30	32.7	100	77			
26				40.4	97	88	30	23.7	100	77			
							20	56.8	87	63			
							18	169.7	62	36			

TB-15 (105+80)							TB-16 (106+80)						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
24	95.6	100	100				15				38.02	69	45
24				17.17	100	100	0	142.6	65	50			
15				42.3	89	16	-10	15.95	74	29			
12	85.9	92	45										
10	29.22	92	45										

TB-17 (108+89)							TB-18 (110+80)						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
20				30.5	88	71	30	30.23	100	83			
-15	75.97	100	20				30				15.73	100	83
							25	20.5	82	42			
							25				2.89	82	42
							20	58.2	82	42			
							15	159.5	84	61			

TB-19 (112+90)							TB-20 (113+80)						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
22	141.2	97	73				30	9.6	75				
22				19.84	97	73	30				6.8	75	
18	15.95	97	73				25	14.7	70				
0	33.48				78	17	-20	146.8	71				

TB-21 (116+04)							TB-23 (118+93)						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
8	45.5	90					8				8.7	100	
-10	16.45	54					5				7.7	97	
							-8	23.2	92				

TB-24 (127+30)							TB-26 (130+00)						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
12	39.9	94	94				0	45.7	94	30			
8				32.7	94	94	0				39.7	94	30
2	73.4	97	70										

TB-27 (131+65)						
EL.(ft)	qu	REC	RQD	qt	REC	RQD
20	127.6	25	17			
20				52.1	25	17
9	167.5	55				
2	207.5	55	55			

8. Mac Arthur

Mac Arthur
State Project No. 87060-1549

WB-24							WB-14						
El	Qu	%	Rqd	Qt	%	Rqd	El	Qu	%	Rqd	Qt	%	Rqd
-40	485.5	97	83				-33	91	92	79			
-40				17.15	97	83	-33				31.39	92	79
-45	188	97	83				-43	54.5	63	25			
-45				4.92	97	83	-43					63	25
-50	56.5	100					-60	134.5	47	11			
-50				13.65	100		-60					47	11
-51	20	100					-70	179	32	30			
-51				9.45	100		-70					32	30
-55	26	77	63				-90	203	45	18			
-55				7.95	77	63	-90					45	18

WB-18							B-10						
El	Qu	%	Rqd	Qt	%	Rqd	El	Qu	%	Rqd	Qt	%	Rqd
-33	108.5	100	100					49	60	60			
-33				28.51	100	100					15.6	60	60
-43	317.5	49	43					47	95	77			
-43				30.78	49	43					19.92	95	77
-95	146.5	68	43					26.5	95	77			
-95				10.55	68	43					7.93	95	77

B-32						
El	Qu	%	Rqd	Qt	%	Rqd
	373.5	70				
				15.78	70	
	13	86				
				6.585	86	
	44	86				
				5.765	86	
	19.5	86				
				6.26	86	

9. Venetian

Venetian
State Project No. 87000-3601

B-11							B-13						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-40	191.05	71	48				-50	351.66	45	45			
-40				87.59	71	48	-50				11.328	45	45
-55	32.468	58	38				-55	172.8	53	13			
-55				11.255	58	38							

B-15							B-17						
EL.(ft)	qu	REC	RQD	qt	REC	RQD	EL.(ft)	qu	REC	RQD	qt	REC	RQD
-45	230.3	93	50				-40	50.144	76	45			
-45				69.841	93	50	-40				42.713	76	45
-55	123.67	75	63				-50	78.139	88	76			
							-50				58.369	88	76

A.1.1 17th Street Causeway and Acosta qu & Ei

17th Street Causeway			Acosta		
Core Sample	qu(psi)	Ei(psi)	Core Sample	qu(psi)	Ei(psi)
1U	201	67,568	1U	4,613	2,450,000
2U	576	327,869	2U	3,806	1,860,465
3U	718	281,690	3U	400	123,711
4U	813	645,161	4U	322	76,190
5U	833	357,142	7U	485	99,174
6U	1,196	588,235	8U	291	65,574
7U	813	434,782	9U	1,380	326,923
8U	280	138,889	5U	745	52,632
9U	641	344,827	6U	383	100,000
10U	3,474	952,381	11U	231	90,909
11U	391	166,667	10U	190	44,444
12U	188	113,636			
13U	117	37,453			
14U	136	121,951			
15U	1,701	1,000,000			
16U	4,971	1,176,470			

A.1.2 Apalachicola and Fuller Warren qu & Ei

Apalachicola			Fuller Warren		
Core Sample	qu(psi)	Ei(psi)	Core Sample	qu(psi)	Ei(psi)
Blt11	424	114,300	1AU	8,281	2,553,191
Blt21	604	251,000	2U	3,944	1,333,333
Blt210	864	212,100	4U	242	76,923
Blt212	738	715,000	5U	836	210,562
Blt213	567	435,421	6U	1,124	465,116
Blt215	366	250,000	7U	1,534	714,285
Blt216	389	107,120	1U	603	243,902
Blt217	150	46,800	2U	447	259,740
Blt218	129	24,300	3U	734	347,826
Blt219	151	50,000	4U	1,218	625,000
Blt22	265	86,000	5U	220	43,010
Blt222	840	467,500	6U	92	49,020
Blt271	252	29,000	7U	700	465,116
Blt57	547	59,800	1U	1,217	579,710
Blt570	288	48,900	2U	1,182	597,014
Blt571	252	35,600	3U	1,082	430,107
Blt572	167	15,200	4U	283	147,058
Blt573	314	168,300	5U	409	208,334
Blt574	389	238,267	6U	166	85,470
Blt575	360	333,300	7U	327	39,216
Blt577	189	181,200	8U	249	86,957
Blt578	126	17,800	9U	254	114,942
Blt579	130	40,000	10U	253	56,180
			11U	663	392,157
			12U	765	181,818
			1U	908	465,116
			2U	427	156,250
			1U	481	210,526
			2U	342	158,730
			3U	463	206,185
			4U	195	64,935
			5U	153	35,971
			7U	812	347,826

A.1.3 Gandy and Victory qu & Ei

Gandy			Victory		
Core Sample	qu(psi)	Ei(psi)	Core Sample	qu(psi)	Ei(psi)
S1	670	333,333	V1	1,131	3,125,000
S2	2,590	1,133,333	V2	2,398	3,041,900
S1	30	600,000	V3	2,859	4,651,200
S1	280	200,000	V4	370	1,299,000
S1	4,470	833,333	V5	623	2,222,000
S1	1,960	909,091	V6	1,745	3,625,000
S1	500	129,630	V7	1,943	3,333,300
S1	690	500,000	V8	1,735	4,286,000
S2	470	333,333	V9	1,397	2,186,000
S3	330	166,667	V16	380	1,923,000
S4	400	214,286	V17	350	1,800,000
S5	650	400,000	V18	87	715,500
S1	540	363,636	V20	1,330	3,496,000
S2	390	166,667	V21	1,703	2,200,000
S3	400	100,000	V22	2,484	2,216,912
S1	350	200,000	V28	1,225	3,073,508
S2	320	222,222	V29	181	479,000
S3	1,030	625,000	V30	350	1,429,000
S1	810	269,231	V31	1,380	3,226,000
S1	2,350	833,333	V32	1,393	3,896,000
S2	7,130	1,578,947	V33	1,220	2,985,000
S1	640	235,294	V34	910	2,325,600
S1	8,700	1,153,846	V35	515	2,294,000
S1	300	16,129	V36	1,806	4,396,000
S2	2,320	1,000,000			
S3	2,250	416,667			
S1	7,670	1,000,000			
S2	7,670	1,400,000			