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14. ABSTRACT

In the mid-1980s the Jamestown Verrazzano Bridge was built in Rhode Island spanning the west passage of Narragansett Bay. The bridge was to be founded primarily on pre-stressed concrete piles acting as friction piles. A test pile program was conducted at the beginning of construction and the measured capacity of the piles was significantly lower than predicted values. This led to significant delays in construction, cost overruns, and ultimately led to a change in the design of the foundations.

The overall objective of this thesis is to evaluate the results of the test pile program and attempt to understand why the measured capacities were so much lower than design values.

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A CASE STUDY ON PILE RELAXATION IN DILATIVE SILTS

BY

BRENT D. RICHARDSON

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE

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Abstract

In the mid-1980s the Jamestown Verrazzano Bridge was built in Rhode Island spanning the west passage of Narragansett Bay. The bridge was to be founded primarily on pre-stressed concrete piles acting as friction piles. A test pile program was conducted at the beginning of construction and the measured capacity of the piles was significantly lower than predicted values. This led to significant delays in construction, cost overruns, and ultimately led to a change in the design of the foundations.

The overall objective of this thesis is to evaluate the results of the test pile program and attempt to understand why the measured capacities were so much lower than design values.

The region in which the pre-stressed concrete test piles were driven is known to contain sands, non-plastic silts, and organic silts of varying densities. Pile relaxation is known to occur in dilative sands and silts, and it has been hypothesized that this occurred at this site. However, no one has been able to quantify how relaxation caused such a significant reduction in capacity. There are very few studies on the effects of cyclic pile driving in dilative silts, none of which provide correlations to observed pile relaxation and cyclic loading. Because of this and the fact that dilative sands and silts exist at other potential bridge sites in Rhode Island, this is an important case study to document.

Site characterization, geotechnical properties, and load test data was compiled from a large quantity of construction reports and correspondence from the project. Static capacity analysis was performed for each test pile at design depth and at the depth in which the static load tests were actually conducted. The analyses indicated the design depths should have been of sufficient depth to provide enough resistance for the design

capacities based upon provided boring logs and lab data; this was clearly not the case. The analyses also significantly over-predicted the ultimate capacities of the three test piles driven well beyond the design depths.

The disagreement between the static capacity analysis, CAPWAP and static load tests may have been a result of either one of the following reasons: arching, friction fatigue, post liquefaction behavior or dilation. It was sugggested that a combination of the effects leading to real or apparent pile relaxation may have caused the significant difference between measured and predicted ultimate capacities, however, none of these effects can fully explain the large differences between predicted capacities and the capacities measured in the test pile program.

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I would also like to thank Dr. Aaron S. Bradshaw for sharing his expertise in deep foundation design and providing me unique insight of which the conclusion of my research hinged upon. Special thanks to Alesandra C. Morales Vélez and Yaurel Guadalupe-Torres for their countless hours discussing soil mechanics.

I would like to thank my parents for their love and providing me the opportunity to accomplish my goals. Most of all, I would like to thank my wife, Jules, for her unwavering support, countless words of encouragement, patience while I burned the midnight oil and devotion to raising our wonderful and happy children.

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1. INTRODUCTION

1.1 Problem Statement

A thorough understanding of the geotechnical properties of soil is essential when determining the most cost effective type of deep foundation. Common types of in situ geotechnical investigations for cohesionless soils include Standard Penetration Tests (SPT) and Cone Penetration Tests. In the laboratory, tests such as Consolidated Drained (CD), Consolidated Undrained (CU), and Unconsolidated Undrained (UU) triaxial tests are performed to determine effective stress and total stress strength parameters. Both types of investigations provide shear strength properties necessary for deep foundation design, though lab tests typically provide more accurate results. Many correlations have been developed, however, for in situ tests like the SPT to account for sources of error inherent to the test providing the ability to predict the soil properties with good confidence.

One type of soil behavior these tests do not account for is dilation during cyclic loading and its long term effects. In the mid-1980s the Jamestown Verrazzano bridge was built in Rhode Island spanning the west passage of Narragansett Bay. The bridge was to be founded primarily on pre-stressed concrete piles acting as friction piles. The boring logs utilized in the design of the initial test pile program for this project indicated the existence of loose to very dense non-plastic silt layers (50 to 100 feet thick) with an underlying, very dense glacial till layer of 10 to 50 feet overlying bedrock. The significance of the thick till and silt layers is that there have been instances in which pile driving operations in these types of soils, specifically in Providence, Rhode Island, have led to decreased effective stresses, due to dilation, and significant movement of adjacent structures founded on the same soil types (Bradshaw et al., 2007). Decreased effective stresses can lead to pile relaxation, which is a measured decrease in ultimate pile capacity caused by decreased shaft resistance due to dissipation of excess pore pressures over time. This behavior has been shown to occur in dense non-plastic silts and glacial till with low permeability.

The Jamestown Verrazzano Bridge initial test pile program included the installation of four driven pre-stressed concrete test piles. The determination of the depth of penetration was based on static capacity analyses using SPT blow counts to characterize soil strength. Of the four test piles, three failed to reach the predicted ultimate capacity at the anticipated depth. These results are shown in Table 1.1. This failure to meet the design capacity ultimately resulted in the termination of the initial pile driving contract and associated costs, a redesign of the piles for two of the four sections of the bridge, and an 18 month construction delay. Although it has been speculated that a combination of problem soil conditions and insufficient site investigations led to the delays, no formal study of this important case study has ever been published.

Table 1-1 Predicted and Measured Ultimate Capacities of Jamestown Bridge Test Pile Program

Test Pile	Required Capacity (tons)	Static Load Test Results (tons)
West Approach Test Pile #1	340	83
West Approach Test Pile #2	340	240
Trestle Test Pile #1	330	180
Trestle Test Pile #4	330	520

1.2 Objectives and Methodology

The objective of this thesis is to investigate the reasons behind the $97 million cost overrun of the pile foundations associated with the Jamestown Verrazzano Bridge in the 1990's. In order to meet this objective, the following research was conducted:

- Review of the chronology of the test pile program
- Static capacity analyses of the four test pile sites
- Analysis and comparison of the static load tests and CAPWAP results
- Analysis of pile driving records

The objectives were met by reviewing the Jamestown Verrazzano Bridge project files only recently provided to the University of Rhode Island by the Rhode Island Department of Transportation (RIDOT). The project files contained correspondences between the prime contractor, geotechnical consultants and RIDOT as well as previous subsurface investigations, boring logs, lab data, static load tests and dynamic load tests. Through a review of the available correspondences, a succinct chronology was developed.

The lab data and boring logs provided the information required to conduct a static capacity analysis of each test pile site. Two different boring logs were used in the analysis as well as three different methods of determining the shaft resistance and three different methods of determining the toe resistance. The results were then compared to the ultimate design capacity for the four test piles, all of which were intended to be friction piles.

Because the boring logs indicated the existence of medium dense to very dense sand and silt, it was assumed the static load tests were conducted under drained conditions. Therefore, an analysis of the static load test results was conducted to determine if the tests were performed as such as a means of assessing how accurate the tests were in regards to the actual pile performance. The soil parameters used to model soil resistance in CAPWAP analysis can be altered based upon static load test results as an attempt to improve the predicted capacity of a pile. Therefore, the CAPWAP ultimate capacity values were compared to the static load test results to determine whether the soil models were an accurate depiction of the in situ conditions.

It is widely accepted that pile driving blow counts of less than ten blows per inch are required to fully mobilize the ultimate to capacity of a pile. In order to determine if the ultimate toe capacities were mobilized, an analysis of the pile driving logs was conducted and compared to the CAPWAP and static load test ultimate capacity values.

1.3 Organization of Thesis

The remainder of this thesis is organized as follows:

- Chapter 2 reviews methods of static capacity analysis, static and dynamic testing, and possible reasons for real and apparent pile relaxation
- Chapter 3 provides background information of the Jamestown Verrazzano Bridge test pile program.
- Chapter 4 provides a subsurface description of the Jamestown Verrazzano Bridge site.
- Chapter 5 presents the analysis of compiled ultimate capacity data.
- Chapter 6 summarizes the major findings and conclusions.

2. BACKGROUND AND LITERATURE REVIEW

The capacity of driven piles can be predicted using several different methods of static capacity analysis. During pile driving, the capacity of piles can be estimated using several different types of dynamic load testing. Upon completion of the pile driving process, the capacity of driven piles may be verified by conducting a static load test. This chapter will discuss in detail the methods used to determine the capacity of piles in addition to phenomena which affect the capacity of piles during and after pile driving.

2.1 Standard Penetration Test

Subsurface investigations are generally conducted prior to the design of a foundation in order to accurately determine the geotechnical properties of the underlying soil. One such investigation is the standard penetration test (SPT) and is the most common method used in the United States, initially developed in 1902 and standardized in the 1930's. SPT can be used to determine the resistance to penetration of a soil, the location of the water table, and to obtain a representative soil sample. The resistance of the soil is measured by dropping a hammer onto a drill rod which drives a split-spoon sampler that extracts a soil sample. The sample is used for classification, index tests and determination of changes in the strata. The number of drops of the hammer onto the drill rod are recorded as N-values. The height at which the hammer is dropped is 30 inches from the drill rod and the weight of the hammer is 140 lbs. Figures 2.1 and 2.2 show illustrations of typical samplers.

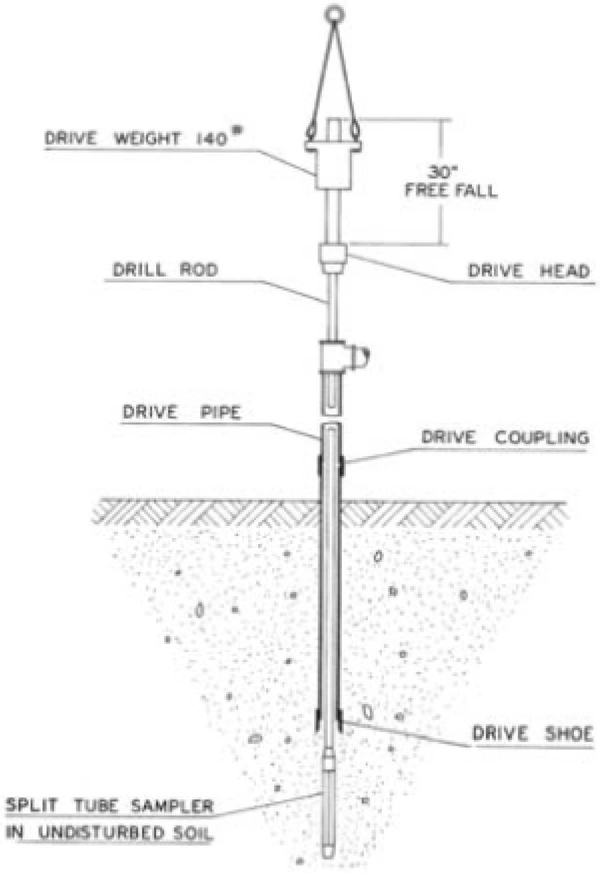


Figure 2-1 Split Spoon Sampler (Mohr, 1940)

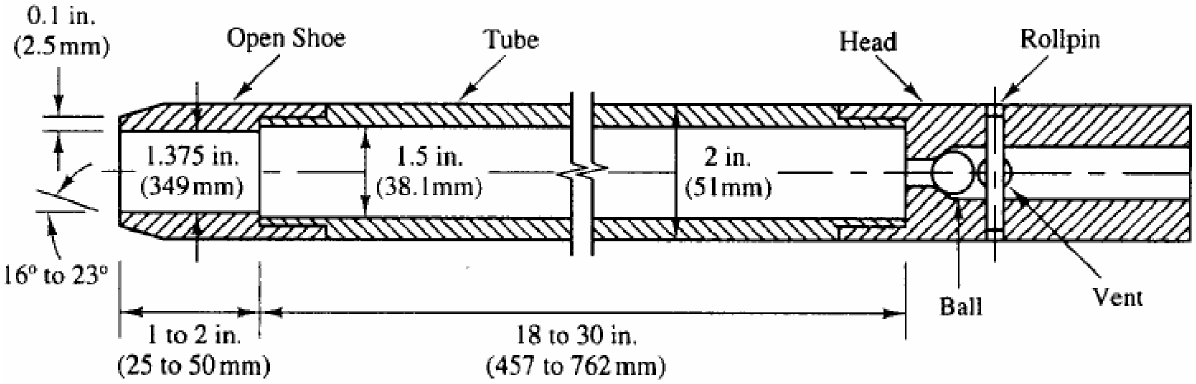


Figure 2-2 Split-Spoon Sampler Dimensions (taken from ASTM D 1586)

The size of the hammer, type of hammer and height at which the hammer is dropped can vary, but should be in line with standard practices outlined in ASTM D 1586 in order to ensure appropriate correlations of measured N-values. Numerous factors can affect the measured N-value and correlations have be developed to account for the affects. A simplified equation which normalizes the affects is listed below:

$$N_{1(60)} = N_m C_N C_E \tag{2.1}$$

$N_{1(60)}$ represents the SPT blow counts corrected for energy (C_E) and overburden stress (C_N), and may include other correction factors based upon the experience of the engineer (Baxter et al. 2005).

Vertical effective stress increases with depth and affect the N-values as depth increases due to increased confining pressure from overlying soil. There have been correlations developed between N-values and soil resistance which correct for this phenomenon in which case the N-values are normalized at any particular depth to a reference stress of 1 tsf. One of the most widely used equations to account for the overburden stress is (Peck et al., 1974):

$$C_N = 0.77\log\left(\frac{40}{\sigma'_v}\right) \tag{2.2}$$

The majority of correlations used to determine $N_{1(60)}$ were developed from tests in cohesionless soils under drained conditions. Therefore, the SPT method described above is not typically used for cohesive soils and presents some difficulty in accurately determining $N_{1(60)}$ in silts if the silts are found to have cohesive properties. Accordingly, Peck (1974) points out that N values obtained from saturated, fine or silty, dense or very dense sands may be abnormally greater due to the tendency of these types of soils to dilate during shear under undrained conditions and should be used conservatively.

Because the scope of this paper includes an analysis of provided SPT data which was correlated to the shear strength and capacity of cohesionless soils, it should be noted that the available literature recommends cone penetration tests (CPT) for cohesionless soils, which is not the standard practice in the United States.

2.2 Determination of Friction Angles and Soil Unit Weights

The angle of internal friction (φ'), commonly referred to as the effective stress friction angle, can be defined as a stress dependent component of shear strength of a soil similar to that of sliding friction in solids (Holtz and Kovacs, 1981). Because the unit weight of the soil determines the vertical stress at a given depth and because the stress at a given depth affects the behavior of the soil, unit weight of the soil must also be determined. The friction angle of a soil is a strength parameter in that the value can be correlated to the shear strength of a soil according to the Coulomb equation for a cohesionless soil (c' = 0):

$$\tau_f = \sigma' tan\varphi' \tag{2.3}$$

where:

τ_f = Shear strength of the soil.
φ' = Effective angle of internal friction.
σ' = Applied effective normal stress.

The friction angles of cohesionless soils can be derived from SPT data using a number of available correlations. One method used to correlate the two was the method put forth by Peck et al. (1974) as:

$$\varphi' = (0.3N)c_n + 27^o \qquad (2.4)$$

where:

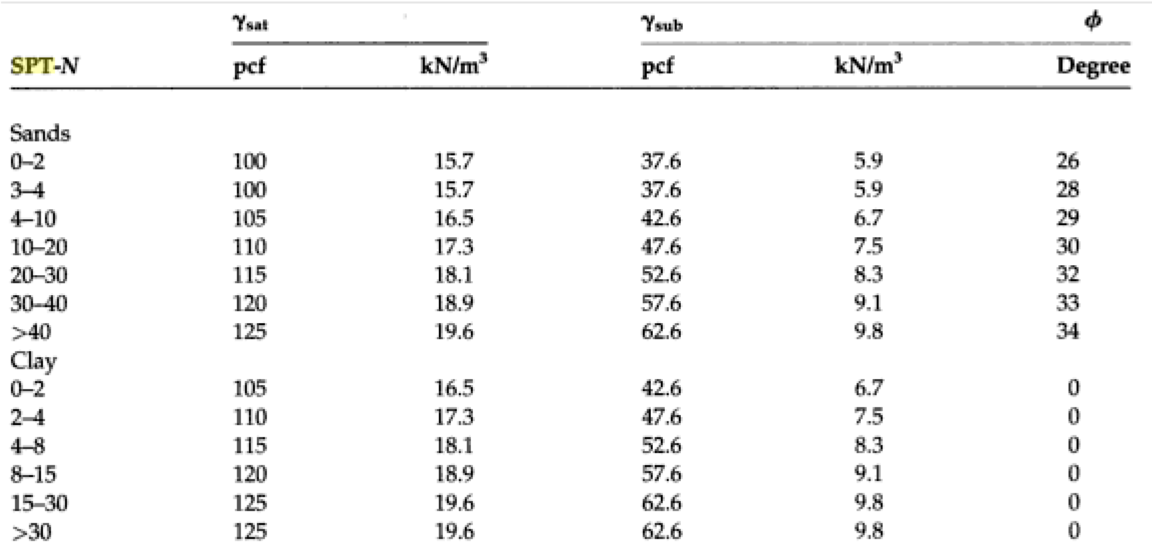
$$c_n = 0.77log\left(\frac{40}{\sigma'_v}\right) \qquad (2.5)$$

and:

φ' = Effective internal friction angle.
N = Uncorrected SPT blow count.
σ'_v = Effective vertical stress.
c_n = Overburden correction factor ($2.0 < c_n < 0.4$)

Friction angles were correlated to SPT data using the Peck et al. (1974) method as well as correlations presented inthe Bowles (1977) and Terzaghi, Peck & Mesri (1996). Each of these methods were developed from tests in cohesionless soils under drained conditions. If these correlations are to be used to determine friction angles for undrained conditions, the determined values would not be representative of the actual shear strength of the soil.

The unit weight of a soil can be determined from correlations to SPT N values, as shown in Table 2.1.



	γ_{sat}		γ_{sub}		ϕ
SPT-N	pcf	kN/m^3	pcf	kN/m^3	Degree
Sands					
0–2	100	15.7	37.6	5.9	26
3–4	100	15.7	37.6	5.9	28
4–10	105	16.5	42.6	6.7	29
10–20	110	17.3	47.6	7.5	30
20–30	115	18.1	52.6	8.3	32
30–40	120	18.9	57.6	9.1	33
>40	125	19.6	62.6	9.8	34
Clay					
0–2	105	16.5	42.6	6.7	0
2–4	110	17.3	47.6	7.5	0
4–8	115	18.1	52.6	8.3	0
8–15	120	18.9	57.6	9.1	0
15–30	125	19.6	62.6	9.8	0
>30	125	19.6	62.6	9.8	0

Table 2-1 Correlation of Uncorrected SPT N-Value with Total Unit Weight and Friction Angle (Kulhawy and Mayne, 1990)

2.3 Static Capacity Analysis

The purpose of a static capacity analysis is to determine the pile type, width, embedment depth and number of piles required to satisfy the calculated ultimate limit state in the axial direction. There exist numerous static capacity analysis methods, however, only the 3 most common methods to determine shaft resistance and 3 most common methods to determine toe resistance will be discussed.

2.3.1 Nordlund Method for Determining Shaft and Toe Resistance

The Nordlund Method was developed in 1963 (updated by Nordlund in 1979) and is the most widely used static capacity analysis method for calculating toe and shaft resistance in cohesionless soils by practicing engineers. The method was based upon the results of load test programs in cohesionless soils for numerous pile types and is considered semi-empirical. Some advantages of this method are that it includes the effects of the pile-soil friction coefficient when determining the shaft resistance and two limiting factors are included in the toe capacity method. In order to determine the ultimate capacity of a pile, Q_u, in a cohesionless soil the shaft resistance, R_s, and toe resistance, R_t are summed. The Nordlund Method equation is below:

$$Q_u = \sum_{d=0}^{d-D} K_\delta C_F p_d \frac{\sin(\delta + \omega)}{\cos(\omega)} C_d \Delta d + \alpha_t N'_q A_t p_t \tag{2.6}$$

Where the variables are (from Hannigan et al., 1998):

d=Depth.
D=Embedded pile depth.
K_δ=Coefficient of lateral earth pressure at depth d.
C_F=Correction factor for K_δ when $\omega = 0$.
p_d=Effective overburden pressure at the center of depth increment d.
δ=Friction angle between pile and soil.
ω=Angle of pile taper from vertical.
φ=Soil friction angle.
C_d=Pile perimeter at depth d.
Δd=Length of pile segment
α_t=Dimensionless factor (dependent on pile depth-width relationship).
N'_q=Bearing capacity factor
p_t=Effective overburden pressure at the pile toe.

(The K_δ, C_F, α_t, N'_q, values can be determined from figures taken from Hannigan et al., 1998.)

If the pile is not tapered it is considered to have a uniform cross section, therefore $\omega = 0$ as is the case for most precast pre-stressed concrete piles (PPC). If $\omega = 0$ and the soil layers vary in effective unit weight and friction angle, Equation 2.6 can be simplified in order to analyze each stratum:

$$Q_u = (K_\delta C_F p_d \sin \delta C_d D) + \left(\alpha_t N'_q A_t p_t\right) \tag{2.7}$$

$[Q_u = (R_s) + (R_t)]$

in which a limiting factor referred to as limiting toe resistance, q_L, is used when determining R_t. The toe resistance is equal to the lesser value of the following two equations:

$$R_t = q_L A_t \tag{2.8}$$

$$R_t = \alpha_t N'_q A_t p_t \tag{2.9}$$

The limiting factor, q_L, is determined from a Hannigan et al. figure as well and increases in an exponential manner as the soil friction angle increases.

The Nordlund method is also used to determine the capacity of piles in cohesionless soils by computer programs such as DRIVEN, which is recommended by the Federal Highway Administration (FHWA) as a means of verifying calculations.

The parameter which most influences the Nordlund method is the soil friction angle. If the soil friction angle is not determined through laboratory tests it is estimated using correlations of corrected SPT N_1 values. The different correlations between friction angle and SPT data will be discussed in section 2.5. Another disadvantage is the Nordlund method does not utilize a limiting factor for shaft resistance, though a limiting factor of 150kPa is used for the effective overburden pressure when calculating the toe resistance. Also, the values determined from the Hannigan et al. (1998) figures are subject to some interpretation which could produce some slight error.

2.3.2 β Method

The Effective Stress Method, commonly referred to as the β Method, is a widely used method of determining the shaft capacity of piles in cohesive and cohesionless soils. The method is best used when determining the static capacity of piles in drained soils because the method is based upon effective stresses at failure in an attempt to model the long term strength of the soil. The Effective Stress Method can be used to predict shaft and toe capacities, however, this section will only focus on calculating the shaft capacity in cohesionless soil. Like the Nordlund Method, the Effective Stress Method is also semi-empirical (Hannigan et al., 1998).

The unit shaft resistance is calculated using the following equation:

$$f_s = \beta \bar{p}_o \tag{2.10}$$

where:

f_s= Shaft resistance.
β= Bjerrum-Burland beta coefficient = K_s tan δ.
$\bar{p}_o$= Average effective overburden pressure along the pile shaft.
K_s= Earth pressure coefficient.
δ= Friction angle between pile and soil.

The ultimate shaft resistance, R_s, is then computed from the sum of f_s for each soil layer:

$$R_s = f_s A_s \tag{2.11}$$

where:

A_s= Surface area determined from pile perimeter and length of pile embedded in soil layer.

If the friction angle of the soil is known, beta values for Equation 2.10 can be taken from Figure 2.3.

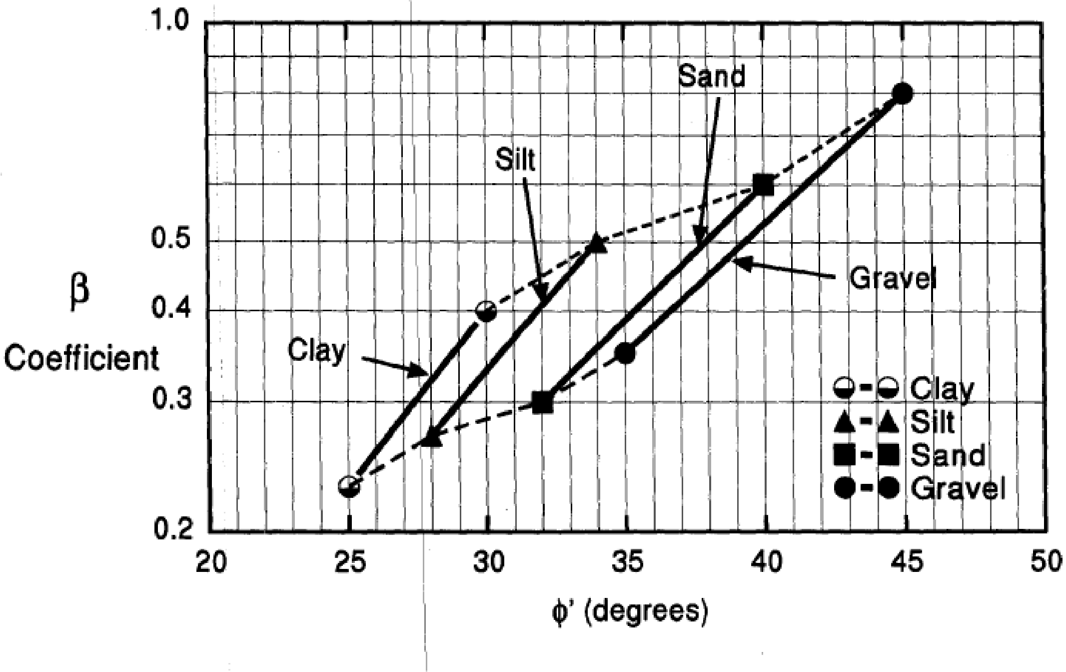


Figure 2-3 Chart for Estimating β Coefficients (after Fellenius, 1991)

The parameter which most influences the Beta Method is the friction angle as it used to determine the beta values. A major disadvantage to the Beta Method is that selecting the beta value is subject to interpretation based upon Figure 2.3 and it is recommended the engineer select a beta values based upon experience.

2.3.3 Bearing Capacity Theory

Bearing Capacity Theory has seen widespread use for over 70 years. The method is based upon developed bearing capacity factors derived from SPT data through an attempt to determine a shear model based upon friction angle, cohesion, and unit weight of the soil. The Bearing Capacity Theory equation used for determining the cohesionless soils under drained conditions is:

$$q_{max} = \gamma' B N_{\gamma}^{*} + q' N_{q}^{*} \tag{2.12}$$

where:

γ'= Effective unit weight of the soil.
B= Diameter of the pile.
N_{γ}^{*}= Bearing capacity factor based upon φ', γ' and I_r.
q'= Effective overburden stress at pile toe.
N_{q}^{*}= Bearing capacity factor based upon, φ', q' and I_r.

In order to determine the bearing capacity factors, the friction angle must be known and rigidity index must be calculated. A common equation for calculating the rigidity index is:

$$I_r = \frac{E_s}{2(1 + \mathrm{v}) q' tan\varphi'} \tag{2.13}$$

where:

I_r= Rigidity index.
E_s=Stress-strain modulus of the soil.
ν = Poisson's ratio (values for cohesionless soils range between 0.3 - 0.5)
q'=Effective overburden stress at pile toe.
φ'=Angle of internal friction of the soil layer.

2.3.4 Meyerhof Methods

The Meyerhof method was developed in 1976 and correlates SPT N_1 values directly to the static capacity of a pile in a homogenous and cohesionless soil. This method is based upon the analyses of many load tests in numerous types of cohesionless soils and is purely empirical (Meyerhof, 1976). Because the type of cohesionless soil does not directly influence the method and due to the method being based upon SPT data, it is commonly used as a quick means of determining capacity and is not recommended for design purposes (Hannigan et al., 1998).

The Meyerhof equation for the average shaft resistance of a driven displacement pile, such as a prestressed concrete pile, is:

$$f_s = 2\overline{N'} \leq 100\ kPa\ \ (N' = N_1) \qquad (2.14)$$

where $\overline{N}'$ is the average of the overburden stress corrected SPT values along the embedded length of the pile.

There are two Meyerhof equations for determining the unit toe resistance and both are just as straightforward as the shaft resistance equation. However, some discretion is required when determining whether the pile is embedded in a homogenous soil or if the embedment depth is near the interface of two strata in which the overlying strata is weaker. The two equations are:

$$q_t = 400\overline{N'}_O + \frac{(40\overline{N'}_B - 40\overline{N'}_O)D_B}{b} \leq 400\overline{N'}_B \qquad (2.15)$$

(Pile toe near interface of two strata)

$$q_t = \frac{40\overline{N'}_B D_B}{b} \leq 400 D_B \tag{2.16}$$

(Pile toe embedded in homogenous cohesionless soil)

where the variables are (Hannigan et al., 1998):

q_t=Unit toe resistance.
$\overline{N'}_O$= Average corrected SPT N' value for the stratum overlying the bearing stratum.
$\overline{N'}_B$=Average corrected SPT N' value of the bearing stratum.
D_B =Pile embedment depth into the bearing stratum.
b = Pile diameter

Limiting factors are placed upon q_t and they are:

$$q_L= 400\overline{N'}_B \text{(for sand and gravel)} \tag{2.17}$$

$$q_L=300\overline{N'}_B \text{(for silts)} \tag{2.18}$$

2.4 Static and Dynamic Pile Testing

The ultimate capacity of a pile determined using static capacity analysis is only as good as the subsurface investigations and the engineer's understanding of the local soil behavior. By performing dynamic pile testing and static load testing, the calculated capacity can be verified with greater certainty.

2.4.1 Static Load Testing

It is widely accepted that a properly performed static load test will yield the most accurate capacity of any given capacity analysis. Though the results of the test are the most accurate, the tests are generally only used if they are deemed cost-effective due to the relative high cost of performing the test compared to dynamic load tests.

A static load test provides the amount of elastic compression the pile endures while loaded, a measurement of pile displacement, and an accurate means of measuring the applied load. An example of a typical static load test set up is provided in Figure 2.4.

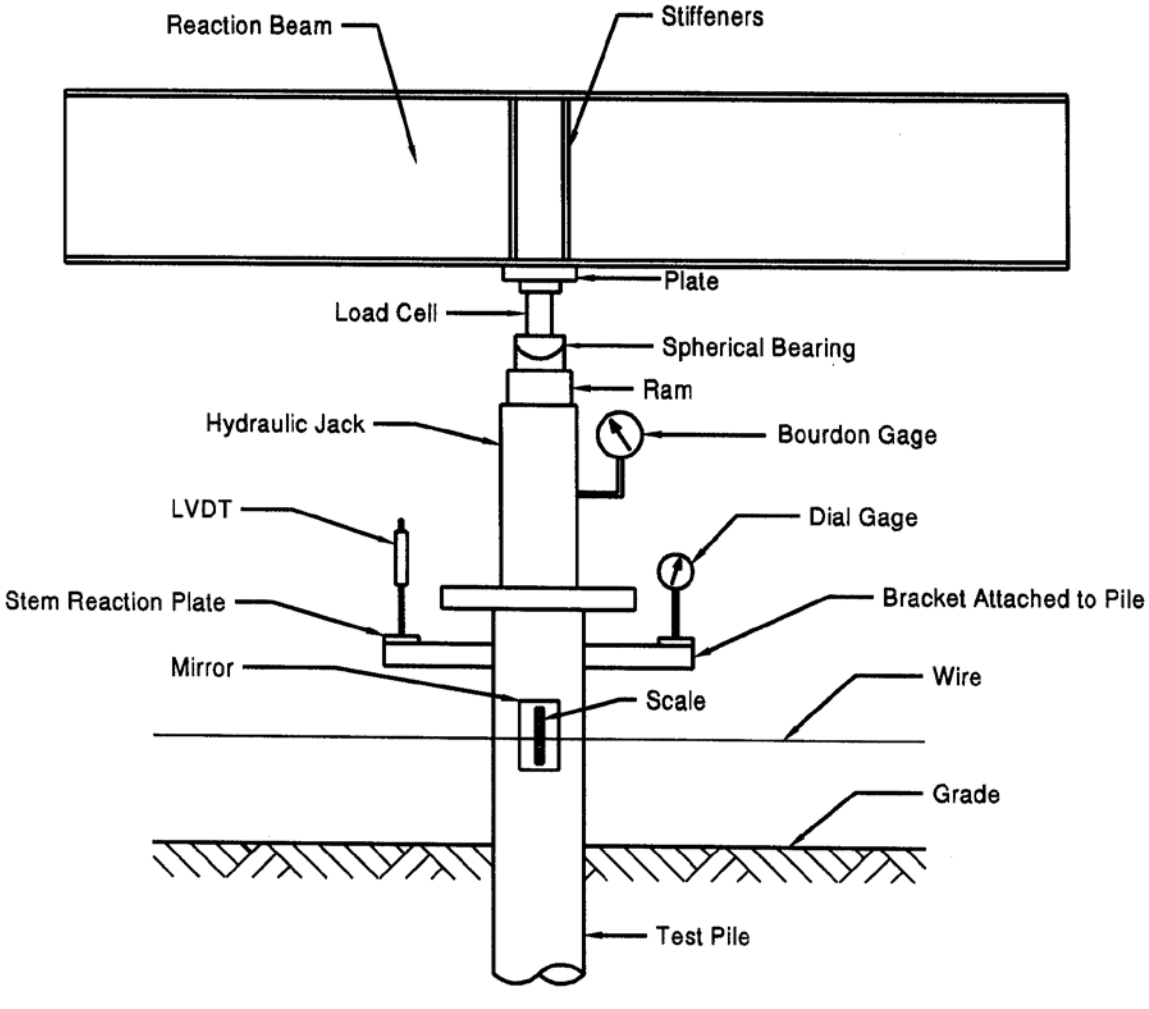


Figure 2-4 Typical arrangement for applying load in an axial compressive test (Kyfor et al. 1992)

As illustrated in Figure 2.4, the applied load is transferred from a reaction frame through a hydraulic ram. A measurement of the load being transferred is monitored by a dial gage and LVDT while the use of strain gages and tell-tales provides the quantity of the load which is transferred to the shaft and toe of the pile. The displacement (deflection) of the pile head is then plotted with a corresponding load and a failure criterion is applied based upon the elastic deformation of the pile. It is noted that the definition of failure used for all static load tests reviewed in this study was the Davisson failure criterion which is the recommended criteria by AASHTO (1992) and Kyfor et al. (1992).

The application of a failure criterion establishes a failure envelope based upon the elastic deformation of the pile. The failure criteria line is plotted at the determined distance from the elastic deformation line, Kyfor et al. (1992) recommends a value of D/30 where D is the pile diameter. The point at which the load-deflection curve crosses the failure criteria line indicates the ultimate capacity of the pile. A typical static load test result illustrating the use of an elastic deformation line, failure criteria and load-deflection curve is shown in Figure 2.5.

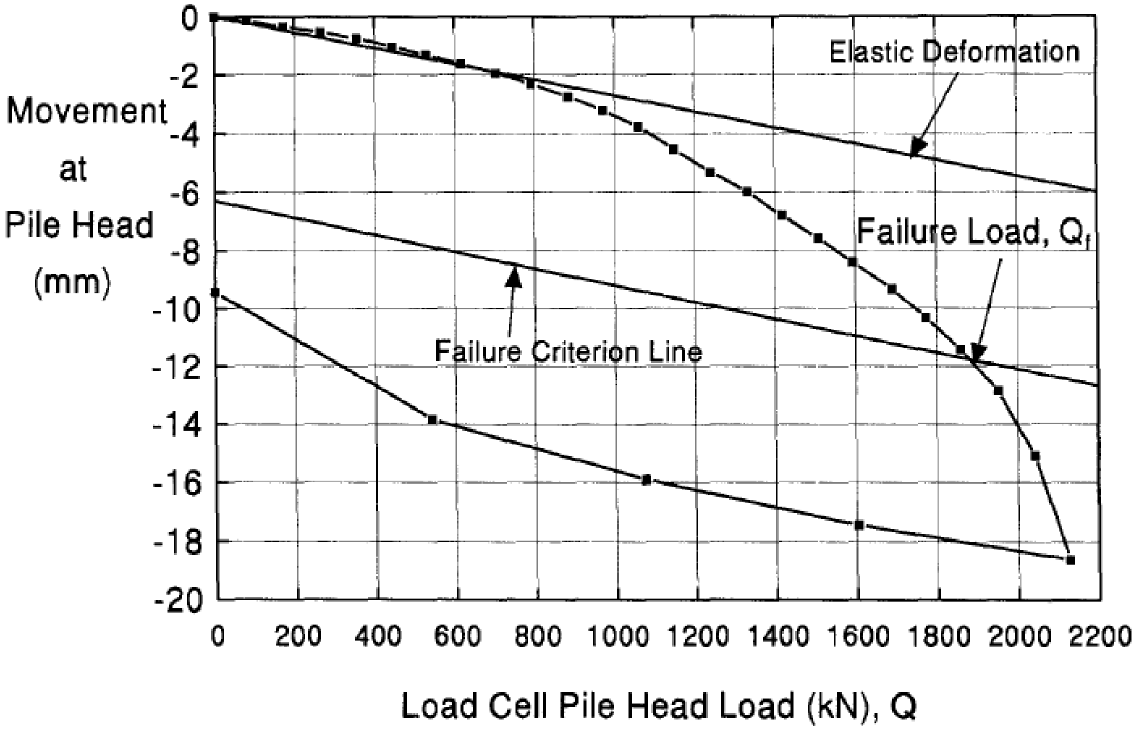


Figure 2-5 Typical Static Pile Load-Movement Results (Hannigan et al., 1998)

Static load tests can also provide a sense of the pile behavior in regards to if it is being supported by shaft resistance or toe resistance (Brüninghold, 2004).

2.4.2 Dynamic Pile Testing

Dynamic pile testing can be defined as a measurement of strain and acceleration at the pile head as a pile is driven by a pile driving hammer in which the measurements are used to evaluate the pile driving system, pile integrity, static pile capacity and soil resistance distribution along the pile (Hannigan et al., 1998).

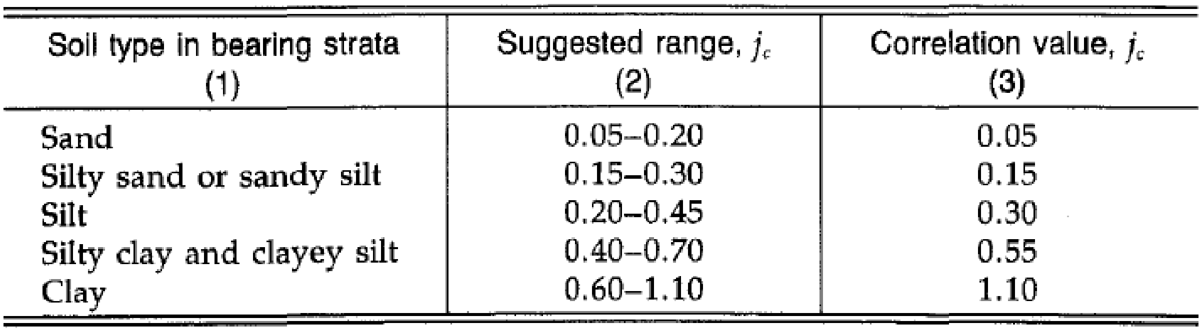
Force and acceleration data is typically measured by means two sets of strain gauges and accelerometers mounted at the pile head and correlated to pile capacity by means of a pile driving analyzer (PDA). The PDA consists of a software suite which allows the operator to input the pile hammer parameters, which in turn, analyzes the static capacity of the pile for each blow of the pile based upon the transferred energy from the hammer to the pile. The major advantage of this widely used method is that it provides real time estimates of static capacity to the operator. However, this method is not the most accurate means of predicting capacity as it does not take into account soil strength parameters.

It is widely accepted that the use of the Case Pile Wave Analysis Program (CAPWAP) is the most reliable means of determining the static capacity of a pile through dynamic testing. The major advantage of CAPWAP is that it provides the resistance distribution, providing an assessment of the toe and shaft capacities. CAPWAP utilizes the PDA force and velocity measurements from one blow of a pile driving hammer to perform an iterative curve fitting technique where the pile response determined in a wave equation model is matched to the measured response of the actual pile for a single hammer blow (Bradshaw et al., 2004). The wave equation analysis is modeled after a continuous pile segments and the soil resistance modeled by elasto-plastic springs and dashpots representing static and dynamic resistance, respectively. Once estimates of the soil resistance distribution are made, the program develops an equilibrium head force wave which is then compared to a PDA determined force wave. Because the waves will not agree initially, the soil model assumptions are continually adjusted until the two waves match. Possible errors in using the CAPWAP program include (Rausche et al., 1985):

1. Capacity is not fully mobilized at the time wave velocity is measured.
2. The impact energy is insufficient to mobilize all soil resistance.

3. The predicted capacity can change due to pile set up or relaxation.

As a way of correlating static and dynamic pile test data, damping factors, *J*, were proposed by Rausche et al. (1985). The notion behind developing a damping factor was to equate the velocity component of the dynamic load data set to static capacity. This was accomplished by determining the R_s of 69 piles and then correlating the values to the CAPWAP data through an empirical means illustrated in the study. Of the 69 piles, 49 piles were close end piles, 15 were prestressed concrete, 10 were timber and 3 were H-piles. A table of proposed *J* values is shown in Figure 2.6.



Soil type in bearing strata (1)	Suggested range, j_c (2)	Correlation value, j_c (3)
Sand	0.05–0.20	0.05
Silty sand or sandy silt	0.15–0.30	0.15
Silt	0.20–0.45	0.30
Silty clay and clayey silt	0.40–0.70	0.55
Clay	0.60–1.10	1.10

Figure 2-6 Proposed Values of Damping Factors Used in CAPWAP Analysis, (Rausche et al.,1985)

It was also determined by Rausche et al. (1985) that low damping values, 0.0-0.2, indicated very low toe velocity subsequently reducing the effects of the damping values on the equation governing the CAPWAP static capacity value. Conversely, as toe velocity increased, as did the sensitivity to the selected damping value. It is then implied that if there is a significant range of unit weights and friction angles in a particular strata, the accuracy of the static capacity values determined using CAPWAP possibly decreases.

2.5 Possible Mechanisms Leading to Reduced Capacity of Piles

In addition to the phenomenon of dilation in dense sands resulting in reduced shear strength, there exist other mechanisms that may affect shear strength to include arching, liquefaction and friction fatigue. The literature available concerning the mentioned methods of failure is not extensive. Furthermore, the literature available on these phenomena only provide recognition that the phenomena exists and do not directly provide correlations to the reduced pile capacity noted during the studies.

2.5.1 Liquefaction

Liquefaction can be defined as an undrained phenomenon which occurs during cyclic loading, typically earthquakes, and results in the transformation of a solid soil into a liquefied state. Referring specifically to saturated cohesionless soils, this phenomenon occurs when soils undergoes cyclic loading in such a manner to produce a tendency of densification, which, in turn, increases the pore water pressure. If the pore water pressure does not dissipate, which can be the case during rapid loading, the pressure builds up to a point the effective stress is equal to zero and the soil loses its strength (Seed & Idriss, 1982; Kramer, 1996). According to Terzaghi et al. (1996), the soils most susceptible to liquefaction are cohesionless soils that have a tendency towards contraction and have low permeability, or non-plastic silty sands containing less than 5% fines passing through the No. 200 sieve.

The cyclic resistance of soils can be quantified by field based methods or laboratory test results. An example of cyclic triaxial test results on Rhode Island silts is shown in Figure 2-7 and 2-8 (Taylor 2011).

The behavior displayed by the Rhode Island silt in regards to the hysteresis loop indicates a decrease in stiffness and therefore an increase of energy dissipated into the system. The results shown in Figure 2.7 indicate an accumulation of excess pore water pressures and an increase in strain as the number of cycles increase.

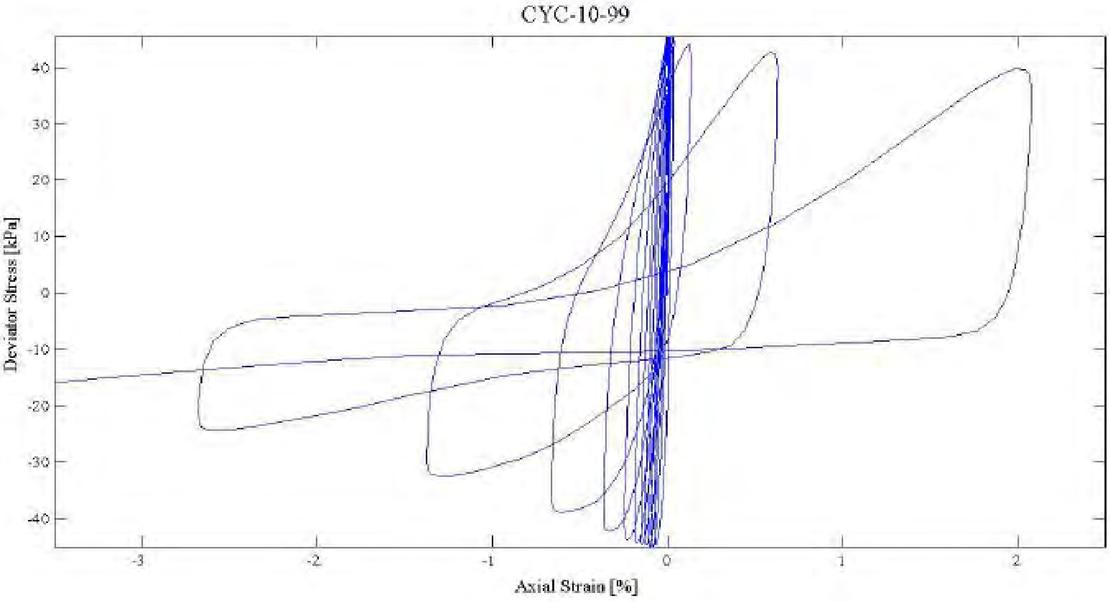


Figure 2-7 Deviator stress (kPa) with axial strain (%) hysteresis loop from a stress controlled cyclic triaxial test for a Rhode Island silt, Taylor (2011).

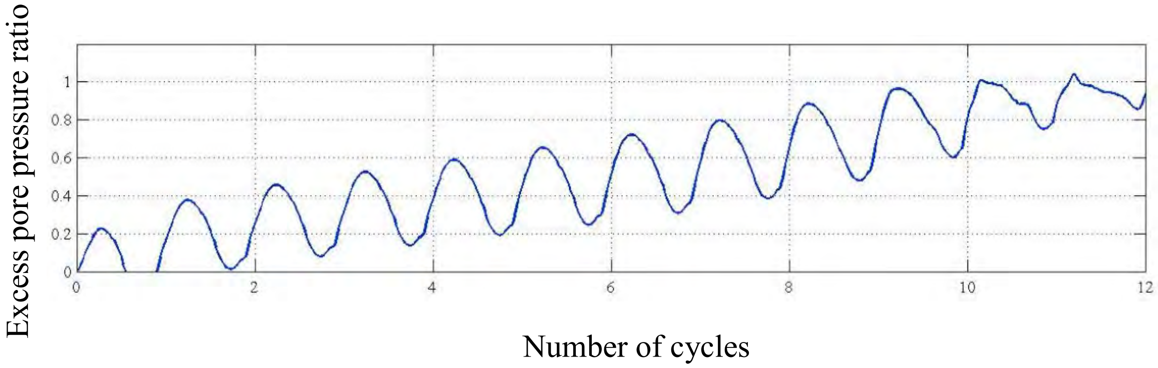


Figure 2-8 Pore pressure increase from a stress controlled cyclic triaxial test for a Rhode Island silt, Taylor (2011).

It can be derived from Figures 2.7 and 2.8 that as the material is cyclically loaded under undrained conditions in the field, there exists the potential for liquefaction due to the decreased stiffness and increased excess pore water pressures. Though it can be assumed the soil liquefies during pile driving, it is unclear whether or not the soil retains the same friction angle after it transitions from the liquefied state.

2.5.2 Arching

Arching can be defined as a circumferential mechanism which develops during pile driving in dense marine soils that limits the immediate amount of radial stress acting upon a pile shaft. This effect has been predicted to recede over time, allowing for significant increase not only in strength, but also in stiffness and dilation (Chow and Jardine, 1998).

The mechanism of arching has been determined to occur in cohesionless material as a result of the pile tip of a cyclic driven pile compressing the soil beneath it and at some lesser degree, the soil at some immediate distance from the tip (Vesic, 1963). As the pile continues to be driven, the soil at the pile-soil interface is pulled in a downward motion and can be considered loose, or less dense than the surrounding soil. As a result, the actual shaft capacity is likely to be lower than the predicted shaft capacity due to decreased horizontal effective stresses (effective radial stress) at the interface despite higher stress values (hoop stresses) in the surrounding soil. It can also be assumed that lower effective radial stresses at the interface are in a sense "locked in" by the higher stresses in the surrounding soil (Chow and Jardine, 1998). Figure 2.9 illustrates the arching effect.

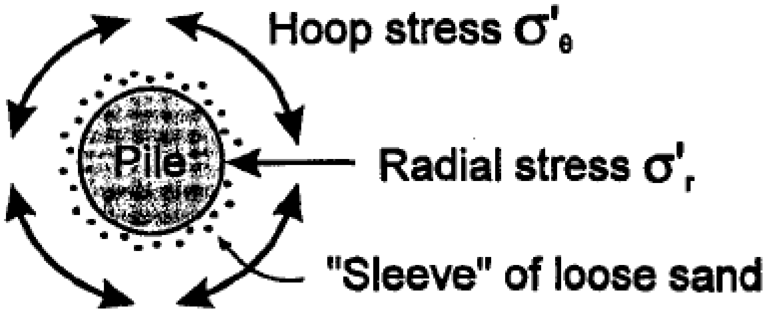


Figure 2-9 Arching Mechanisms around Pile Shaft (Chow and Jardine, 1998)

A study of this mechanism conducted by Chow and Jardine (1998) concluded that though shaft resistance is initially reduced, an 85% increase in shaft capacity is was noted to occur between 6 months to 5 years for open-ended piles driven into dense marine cohesionless soil (the piles used for this study met the plugging criteria, therefore it can be assumed the same increase in shaft resistance will occur for concrete piles under similar driving and soil conditions). Initially it was speculated that a change in the geologic conditions (increase in tidal pore pressure) in the area of the test piles was the cause of the increased pile capacity, however, upon further study it was deduced that sand creep was the likely component as the changes in tidal pressure was not significant enough to cause such a noted increase in capacity. Sand creep was concluded to be the cause as it could lead to a reduction in dilation caused during pile driving, thus reducing the arching mechanism and increasing the radial stress acting on the pile.

Though the study conducted by Chow and Jardine (1998) provides valuable insight into a potential cause of overestimating the initial capacity of piles cyclically driven into dense marine sands, a correlation to initial reduction in capacity was not provided. However, it is noted that the capacity of the piles studied was calculated by 5 groups of researchers using 5 different methods. The average value of calculated capacity, Q_c, to measured capacity, Q_m, was found to be 1.6, indicating that the initial capacity of the piles was 40% less than the predicted value.

2.5.3 Friction Fatigue

Friction fatigue can be defined as a reduction of mobilized horizontal effective stress due to cyclic loading during pile driving operations in sand which results in a reduction in shaft resistance (White & Lehane, 2004). This phenomenon has been produced and measured by several researchers through the testing of monotonic and cyclically installed model and instrumented piles.

The phenomenon of friction fatigue of piles driven in sand was first presented by Vesic (1970), though the term friction fatigue had not been coined yet. During his analysis of the phenomenon, Vesic (1970) states that bearing capacity is primarily dependent on three variables: the coefficient of lateral pressure dependent on friction angle and relative density (K_s^*), bearing capacity factor based upon friction angle and relative density (N_q^*) and the coefficient of friction between the soil and pile (δ). According to the author, these factors will affect the unit resistances in sand proportionally in a linear fashion. However, as he points out, anomalies affecting strength and unpredicted scale effects have occurred during testing of piles which could not be explained by the theories of the time.

Vesic's (1970) publication is focused on the results of two piles tests conducted at the Georgia Institute of Technology and at the Chevreuse Test Station near Paris. The testing involved measuring the shaft and toe capacities of driven, jacked and buried large scale piles. The findings of both tests, completed between 1960 and 1965, indicated toe and shaft resistance of piles in sand increases linearly with depth, but only to a certain point which was initially determined to be a function of arching. Because the arching phenomenon could not be fully explained and due to the ultimate loads of the shafts of the piles varying somewhat significantly in tension and compression, Vesic (1970) believed there was the possibility of a scaling error leading him to doubt the conclusions of the report. As a result, Vesic (1970) attempted to reproduce the Georgia Tech study using larger instrumented piles at the site of the Ogeechee River Bridge located approximately eighteen miles west of Savannah, Georgia. Boring logs taken at the site indicated medium dense to dense sand, similar to the composition of sands of the Georgia Tech study.

Two test piles were used for the study were an 18 inch diameter closed pipe pile and a 16 inch square PPC. The pipe pile measured 51.5 feet in length, the PPC 55 feet in length and both were driven by a diesel hammer to a depth of approximately 50 feet. The pile displacements were measured for both piles during driving and the axial pile loads were measured for the pipe pile through 6 strain gages installed at near equal spacing along the pile. The pipe pile was driven and load tested in 5 stages at corresponding depths of 10, 20, 30, 40 and 50 feet. The results reasonably compared to early tests in that the linear relationship between resistance and depth ceased at a certain depth. The depth at which this occurred was determined to be between a depth of 10 and 20 pile diameters.

This relationship was developed Vesic (1970) by plotting shaft resistance with depth at each stage of testing. The shaft resistance distribution displayed parabolic behavior, as shown in Figure 2.10, and indicated shaft resistance was concentrated towards the upper portion of the pile for shorter piles and more towards the pile toe in longer piles.

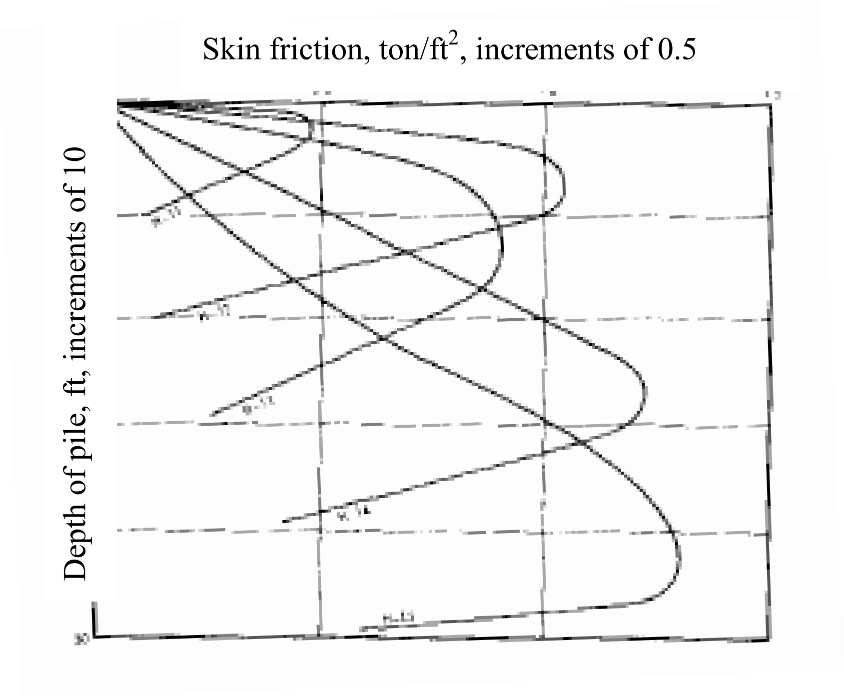


Figure 2-10 Distribution of Skin Friction Along Pile Shafts (Vesic, 1970)

Many other studies have since been conducted to determine the characteristics of friction fatigue, the most influential completed by Chow & Jardine (1996) in which the effective horizontal stress (σ'_{hs}) was measured for both monotonic and cyclic driven piles in sand outfitted with horizontal stress sensors. The focus of studying σ'_{hs} was not only to provide soil behavior characteristics, it was also required in order to interpret model pile test results. The results of the tests showed σ'_{hs} profiles corresponded to cone penetration test (CPT) end resistance profiles (q_c) indicating that σ'_{hs} was a function of the *in situ* sand state. For each pile, three of the previously mentioned horizontal stress sensors were installed at a distance (*h*) from the pile toe and was normalized by dividing *h* by the pile diameter (*D)*. This term *h/D* was then utilized to account for friction fatigue effects in the equation developed by Chow & Jardine (1996):

$$\sigma'_{hs} = 0.029 q_c \left(\frac{\sigma'_v}{P_{atm}}\right)^{0.12} \left(\frac{h}{R}\right)^{-0.38} \tag{2.19}$$

where:

σ'_{hs}=Effective horizontal stress.
q_c =CPT end resistance
σ'_v= Vertical effective stress.
P_{atm}= Atmospheric pressure.
R= Pile radius.
h=Distance from the pile toe.

A study conducted by White & Lehane (2004) focused on the Jardine & Chow (1996) results showed a reduction of available shaft resistance at a given soil layer during installation and during cyclic loading was more related to the number of cycles experienced at a specific point, indicating the Chow & Jardine non-dimensional h/D value was less reliable when determining σ'_{hs}. Due to this inference, White & Lehane conclude monotonically driven piles should provide more shaft resistance than cyclically driven piles.

Gavin & O'Kelley (2007) conducted similar pile tests on cyclically and monotonic driven model piles, however, the tests differed in that their tests included the application of working loads to the monotonic driven piles after the piles were placed at depth. The results of the test indicated that, indeed, the monotonic driven piles experienced greater levels of σ'_{hs} than cyclically driven piles at the same depths, however, the values between the two types of piles was "indistinguishable" after just a few working load cycles were applied. The findings of this study would allow for the removal of the *h/D* term from equation 2.15, which would indicate σ'_{hs} is a mostly a function of q_c as there is not a term correlating the number of cycles to reduced σ'_{hs}.

2.5.4 Pile Relaxation

It is commonly accepted the act of pile driving in sands causes the sand to displace and remold along the pile shaft. During this process it can be assumed the sands are disturbed resulting in the generation of positive pore pressures, especially at large strains. This increase in pore pressures will decrease the effective stress in the vicinity of the pile shaft, leading to a reduction in resistance to the pile. The rate at which the pore pressures dissipate (increasing effective stress) can be correlated to increased shaft resistance and is a function of the permeability of the sand. In rare instances, sands have displayed dilative behavior leading to negative pore pressure generation which has been shown to significantly decrease effective stresses resulting in decreased capacity over time. (York et al., 1994; Thompson et al., 1985). This behavior is referred to as pile relaxation and the conditions leading to this phenomenon are illustrated in Figure 2.11.

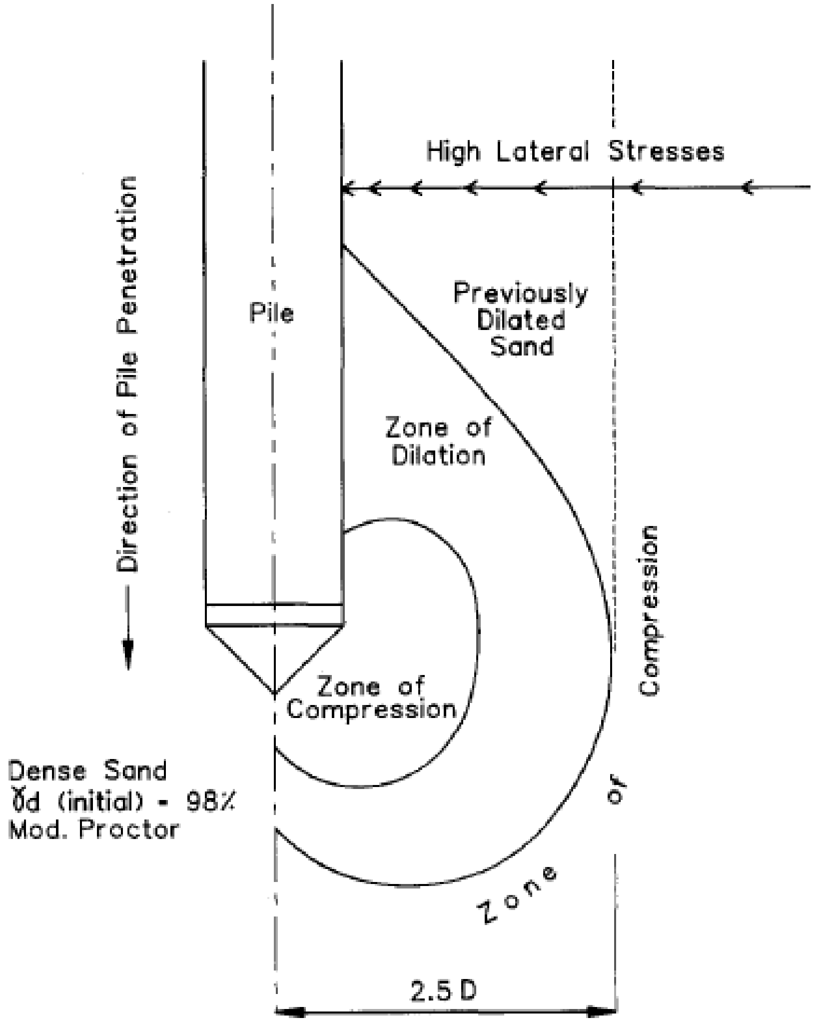


Figure 2-11Conditions Leading to Relaxation (York et al, 1994)

Pile relaxation due to dilation has been observed in dense, saturated, fine grained soils including non-cohesive silts, fine sands, and some types of shale. In these observed cases, it was assumed the pile driving process caused the dense soil in the vicinity of the pile toe to dilate, creating “suction” or, in other words, negative pore pressures. Consequently, the negative pore pressures increase the effective stresses on the pile providing a temporary increase in shaft resistance and driving resistance. As water is “sucked” into the dilated voids of the material, the effective stresses decrease, thus reducing the shaft capacity of the pile in the long term. (Hannigan et al., 1998)

A case study performed by Thompson et al. (1985) focused on pile relaxation in the glacial bearing deposits commonly encountered in southern Canada and north-eastern United States. The piles studied were close end pipe piles and H-piles driven in the Bayfront area of Toronto, Canada, where, as the authors point out, pile relaxation was not uncommon.

The subsurface conditions in the vicinity of the piles studied included a silty sand layer of 20-33 feet to shale bedrock which contained layers of limestone. Borings disclosed that weathered shale seams were present immediately below the limestone layers. Pile relaxation at two different sites occurred where a Penetration Resistance Equivalent (BOR/EOID) reduction ranged from 24% to 68% (H-piles) at one site and 30% to 80% (pipe piles) at the other.

H-piles have an inherently lower nominal contact stress than high displacement piles. This is pointed out because a common solution to overcome, or at least reduce, the observed pile relaxation in the Bayfront area was to specify H-piles in a contract which required deep foundation support. It was also noted that multiple restrikes over a period of time are common in the Bayfront area to ensure design capacity is met.

York et al. (1994) conducted a case study on observed relaxation of grouped monotube piles in glacial sands where a 30% reduction in capacity was noted. The driving record indicating this reduction is shown in Figure 2.12.

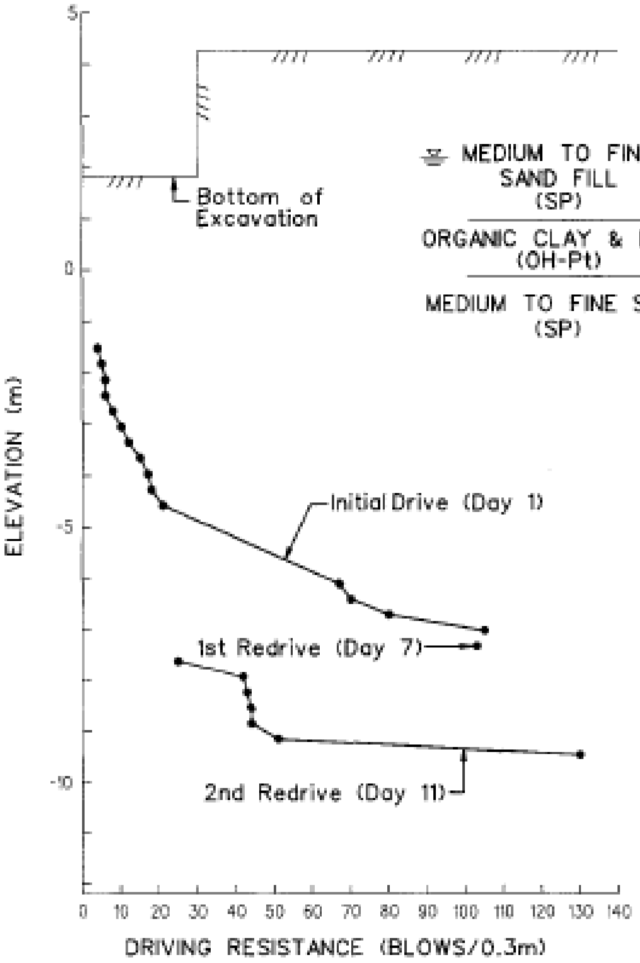


Figure 2-12 Driving Record from York et al. (1994)

The pile relaxation was attributed to the cumulative effects of driving piles in close proximity to each other resulting in the "progressive" densification of the surrounding soil. The densification together with the increase in lateral ground stresses resulting from soil displacement were determined to increase the resistance to pile penetration. Resistance to penetration was observed to subside as much as 80 blows per 0.3m. It is also interesting to note the amount of time required for the maximum value set-up capacity of the monotube piles driven in glacial sands was between 15 and 25 days.

As such, it can be concluded from the results of both case studies that pile relaxation should be accounted for when driving large displacement piles in dense glacial till and non-plastic silty sands.

3. PROJECT BACKGROUND

This section provides an overview of the construction of the Jamestown Verrazzano Bridge to include a synopsis of the contract requirements regarding the test pile program. The test pile performance will be discussed in detail in Chapter 5.

3.1 Location and Description

The Jamestown Verrazzano Bridge is a four lane segmental concrete box-girder bridge supported by piers founded on bedrock. The bridge spans 7400 feet of the West Passage of the Narragansett Bay, just a few miles north of the southern coast of Rhode Island. The bridge carries RI-138 from North Kingstown to Conanicut, allowing RI-138 to connect to the Pell Bridge which spans the East Passage of Narragansett Bay. The bridge was constructed approximately 400 feet to the north of the Jamestown Bridge and served as its replacement. The Jamestown Bridge was built in the 1940's and supported only two lanes of traffic. The location of the Jamestown Verrazzano Bridge is shown in Figure 3.1. (Anderson, 2006)

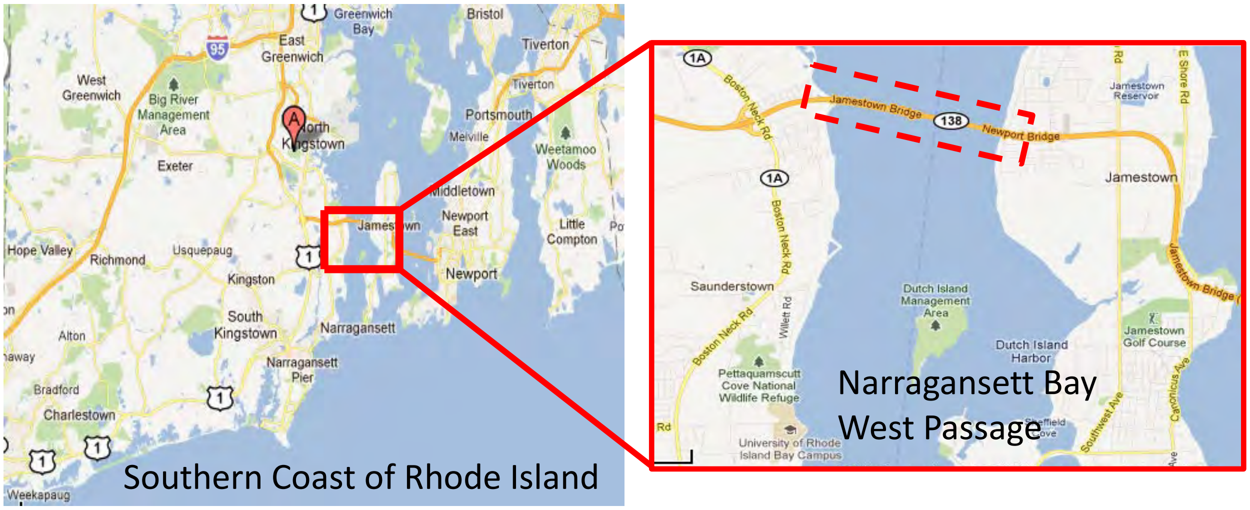


Figure 3-1Map of Narragansett Bay and Location of Jamestown Verrazzano Bridge (taken from Google maps)

Coming from the west, the bridge is supported by a 2400 foot Trestle Approach which transitions to the 2000 foot West Approach, shown in Figure 3.2, then a 1400 foot Main Span and lastly a 1600 foot East Approach. The initial contract called for the Trestle Approach and West Approach to be supported by pre-stressed concrete friction piles (Davisson, 1988). The depths to which the friction piles were to be driven ranged from 90 to 105 feet below Mean Sea Level (MSL). These depths corresponded to elevations of 50 to 100 feet above bedrock.

Figure 3-2 View of the Jamestown Verrazzano Bridge from the West (www.flickr.com)

3.2 Parties Involved

Construction of Jamestown Verrazzano Bridge was approved in May of 1981 and the Rhode Island Department of Transportation selected the design firms by December of the same year. Sverdrup & Parcel and Associates (SPA) were selected as the managing consultant, Gordon R. Archibald, Inc. (GRA) as the trestle designer and T.Y. Lin International (TYL) as the West Approach, Main Span and East Approach designer. TYL subsequently hired Lee and Praszker as geotechnical consultants.

The contract to construct the Jamestown Verrazzano Bridge was awarded to Clark Fitzpatrick, Inc. and Franki Foundation Co. (CFF) in June of 1985 who offered the lowest bid at $63.4 million. The firm Daniel, Mann, Johnson and Mendenhall (DMJM) was hired in 1986 as the construction manager. CFF hired Goldberg-Zoino and

Associates (GZA) as their geotechnical consultant engineer while RIDOT hired M.T. Davisson as their geotechnical consultant engineer. Construction of the bridge began in April of 1986 and was completed in October of 1992.

3.3 Overview of the Test Pile Program

The contract called for five test piles, four of which were pre-stressed concrete piles, the other a steel H-pile. The four pre-stressed piles were to be driven in the Trestle and West Approach Areas while the steel H-pile was to be driven in the Main Span Area. The lengths of the pre-stressed concrete piles along with the required capacities and size are shown in Table 3.1 and the locations of the test piles are shown in Figure 3.3.

Table 3-1 Required Test Pile Capacities and As-Built Dimensions

Test Pile	Required Capacity (tons)	Length (ft)	Width (in)
WATP-1	340	123	20 x 20
WATP-2	340	120	20 x 20
TTP-1	330	110	24 x 24
TTP-4	330	110	24 x 24

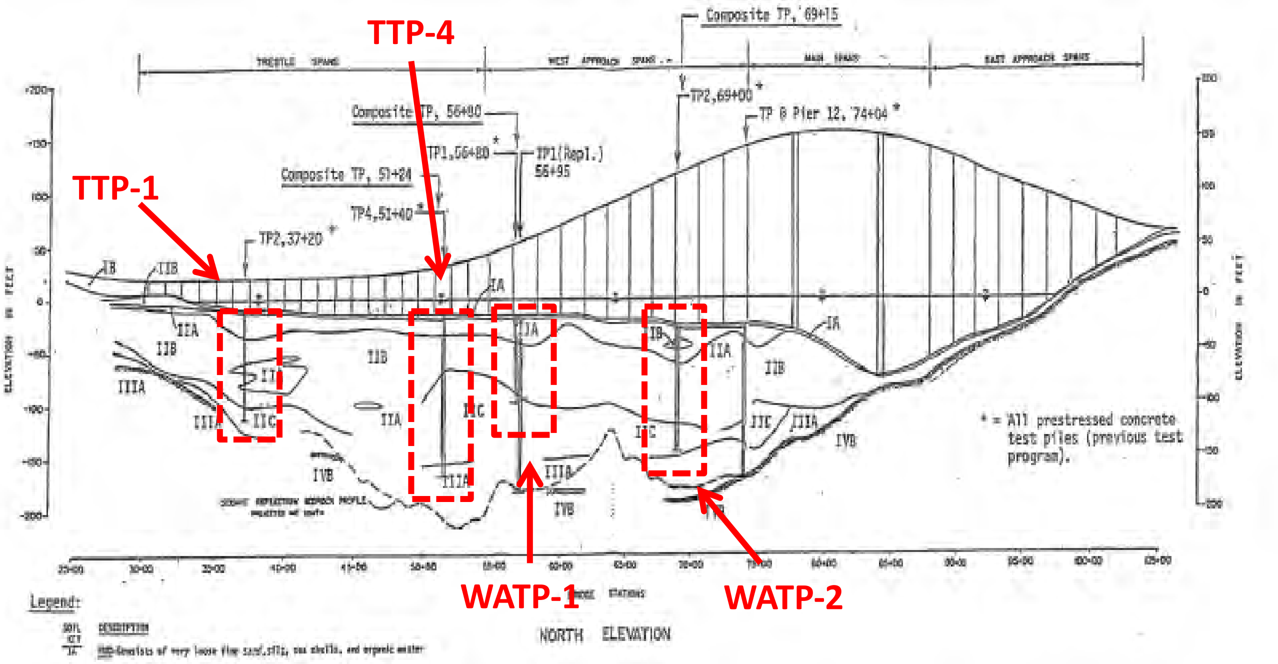


Figure 3-3 Location of Test Piles (After Lee and Praszker, 1983)

In correspondence dated February 29, 1984 (Lee and Praszker, 1984), approximately 15 months prior to the test pile program being initiated, Lee and Praszker noted that the pile tip elevations for the Trestle and West Approach area were all -100 feet in elevation and that the depth appeared to be selected due to poor soil conditions in the western portion of the Trestle Area. The consultant suggested to TYL that the pile tip elevation for the eastern part of the Trestle Area be reduced from -100 feet to -85 feet and the West Approach pile tip elevations be reduced to -90 feet on the western section and -95 feet in the eastern section due to better soil conditions. The consultant's recommendations were based upon the available test borings.

Conversely, in a correspondence dated December 11, 1985, GZA (GZA, 1985) determined that the West Approach Test Piles (WATP) 1 and 2 were to be driven into pockets of silt and predicted high blow counts (in excess of 10 blows per inch) in that material suggesting the test pile resistance would not be fully mobilized. As a result, GZA assumed there was a great possibility the test piles would need to be driven to depths greater than the contract drawings then required (-90 for WATP 1 and -95 for WATP 2). GZA's recommendation to bolster their prediction of insufficient pile capacity was to extend the length of the test piles to allow for an additional 20 feet of penetration. CFF agreed with the recommendation, forwarded the recommendation to RIDOT, who then accepted the change (RIDOT correspondence, 1986) and directed CFF to increase the test pile length by 20 feet. The dimension of the test piles, as shown in Table 3.1, reflect the final lengths of the test piles.

The initial test pile program began in April of 1986. It was observed during the test pile program that the pre-stressed concrete piles were not reaching their required ultimate capacities at the required design depths. Accordingly, CFF drove all but WATP 1, which was damaged prior to reaching the design depth, well past their design depths after adding splices ranging from 30 to 68 feet. WATP 1 was abandoned and later replaced with a composite test pile.

Dynamic load testing was performed on each pile using Pile Driving Analysis (PDA) which was further analyzed at the End of Driving (EOD) using CAPWAP. CAPWAP results indicated that each pre-stressed concrete test pile had not reached the required ultimate capacities. Static load tests (SLT) were also performed on each pre-stressed concrete test pile which showed that only Trestle Test Pile 4 (TTP 4) met the required ultimate capacity. A comparison of the CAPWAP and SLT results are shown in Table 3.2.

Table 3-2 Results of Static Load Tests and CAPWAP analysis

Test Pile	Required Capacity (tons)	Static Load Test Results (tons)	CAPWAP Capacity (tons)
WATP-1	340	83	135
WATP-2	340	240	211
TTP-1	330	180	308*
TTP-4	330	520	227**

* CAPWAP analysis performed two months prior to static load test

** Restrike performed 3 days after static load test

High blow counts were encountered during the driving of WATP 2, on the order of 26 to 30 blows per inch, near and up to elevation -141 feet. It was observed the pile hammer was operating at full throttle, which prompted GZA to inform CFF of a potential of pile damage as well as pile hammer damage. In the same correspondence dated May 22, 1986, it was stated by GZA that increased pile capacity may be realized (pile set-up) and recommended a re-strike prior to the static load test scheduled for June of 1986. Furthermore, GZA stated that if the pile did not obtain the required capacity they would recommend an alternate pile type be considered. CFF, taking into consideration GZA's recommendation, performed the re-strike and increased the pile hammer weight in order to increase drivability (the heavier hammer was subsequently used to drive the remaining test piles, TTP 1 and TTP 4).

Upon completion of WATP 1 and WATP 2 installation RIDOT, CFF, GZA, DMJM and Federal Highway Administration (FHWA) representatives met on sight in May of 1986 to discuss the preliminary results of the dynamic and static load tests. The meeting minutes indicated all parties discussed a need for additional exploratory procedures to determine the cause for the low ultimate capacities of the two test piles (RIDOT Correspondence, 1986). Exploratory procedures discussed included:

- Driving of test probes to find resistance.
- Perform 200 gradation tests on specific samples of test bore material.
- Perform cone penetrometer testing in the field at selected pier locations.
- Perform additional test borings at the test pile locations.
- Installation of additional test piles between WATP 1 and WATP 2.

Following the meeting, RIDOT directed CFF to provide an estimate to perform the 200 gradation tests in addition to performing 20 cone penetrometer tests. According to the documentation provided to the University of Rhode Island, none of the recommendations were carried out. It is noted additional borings were taken in the vicinity of the Trestle and West Approach Areas in October of 1987, however, the primary purpose of the borings was to determine the depth of bedrock.

In August of 1986, RIDOT determined that due to the failed pile load tests for WATP 1 and WATP2, it was apparent that the contract specifications requiring the piles be driven in one length could not be met. As such, alternatives to the initial design were developed by RIDOT in a memo dated August 8, 1986:

- Increase the weight of the driving hammer by 32,000 lbs in order to increase drivability.

- Substitute steel H-piles for the pre-cast concrete piles and have them driven to bedrock and have an ultimate capacity of 400 tons.

- Reduce the ultimate design load of the pre-stressed concrete piles to a maximum capacity determined by future load testing of un-spliced piles. If this alternative were selected, an estimated 100 additional piles would be required.

- Use composite steel and pre-stressed concrete piles (instead of precast concrete piles) to be driven to bedrock and have an ultimate capacity of 400 tons.

- Substitute un-spliced 24 inch pre-stressed concrete piles for the 20 inch piles if it was determined the larger piles could provide the required 340 ton ultimate capacity.

Installation and testing of TTP 1 and TTP 4 began in October of 1986, in accordance with the initial contract specifications and drawings, and completed in February of 1987. As previously mentioned, the piles required splicing and were driven to depths well beyond the design depths in order to attain the required ultimate capacity. Despite this, only TTP 4 mobilized the required ultimate capacity. The initial test pile program was considered satisfactorily completed by RIDOT in August of 1987.

3.4 Contract Termination

Due to the results of the test pile program producing a need for a pile re-design in addition to contracting issues beyond the scope of this case study, an agreement to terminate the contract between CFF and RIDOT occurred in February of 1988. In January of 1989 the contract was put out for rebid and was awarded in June of the same year at a cost of $101.5 million. Work resumed on the Jamestown Verrazzano Bridge in September of 1989. The total cost to construct the bridge was $161 million, which was significantly greater than the original contract cost of $63.4 million.

4. SITE DESCRIPTION

Five investigations were conducted as part of this thesis to determine the soil stratification, soil classification and depth of bedrock in the vicinity of the proposed site for the Jamestown Verrazzano Bridge. These included a review of the geology of the area, seismic reflection data conducted in the west passage of Narragansett Bay in the 1970s, geotechnical borings and laboratory testing conducted as part of the bridge contract, additional geotechnical borings conducted at the site following the failure of the test pile program, and laboratory testing performed at URI in 1987 on soil samples from the site. Each of these investigations are summarized below.

4.1 Narragansett Bay Geology

Narragansett Bay has experienced at least one period of glaciation, the last one known to be during the Wisconsin period which ended approximately 10,000 years ago. During this time, the entire state of Rhode Island was covered in glacial ice several thousand feet thick. As the ice sheet moved southward, the existing soil and rock were scraped down to the bedrock, with the ice sheet carrying the soil and rock until it melted. As the ice sheet melted, the deposits of soil and rock were re-deposited throughout the state overtop the exposed bedrock. (Baxter et al., 2005).

Approximately 20,000 years ago, melting ice formed a fresh water lake covering an area larger than the current size of Narragansett Bay. The soils found in Narragansett Bay today that were deposited during this time consist of sands and inorganic silts and are commonly referred to as outwash deposits. The outwash deposits throughout Rhode

Island can be made up of thick layers of silts overlain by gravelly sands. These layers of silts and sand have been observed to be as much 50 to 150 feet thick in the areas of Providence, and more importantly, North Kingstown, which is in the vicinity of the Jamestown Verrazzano Bridge (Baxter et al., 2005).

4.2 Continuous Seismic Reflection and Bathymetric Survey

An initial part of the site investigation for the project consisted of a continuous seismic reflection survey and a bathymetric survey conducted by the Graduate School of Oceanography (GSO) at the University of Rhode Island (URI) in conjunction with Guild Drilling Company. A technical report analyzing the seismic profiling was completed by URI in April of 1979 (McMaster, 1979) and furnished to Wilbur Smith and Associates. The purpose of the surveys was to determine the morphology of the bottom surface, the configuration and depth of the bedrock surface across the West Passage and thickness of the overburden covering the bedrock (McMaster, 1979).

Two tracklines were specified for the seismic reflection and bathymetric survey which were located 300 feet north and parallel to the existing Jamestown Bridge, and 300 feet south and parallel to the Jamestown Bridge. The trackline to the north was designated A-A’ and will be the only trackline referred to in this study due it being the trackline closest to the actual site of Jamestown Verrazzano Bridge (which was constructed approximately 450 feet north and parallel to the centerline of the Jamestown Bridge). The location of the trackline is shown in figure 4.1 which also indicates depth to bedrock. Also shown on Figure 4.1 are the estimated depths to bedrock along the trackline.

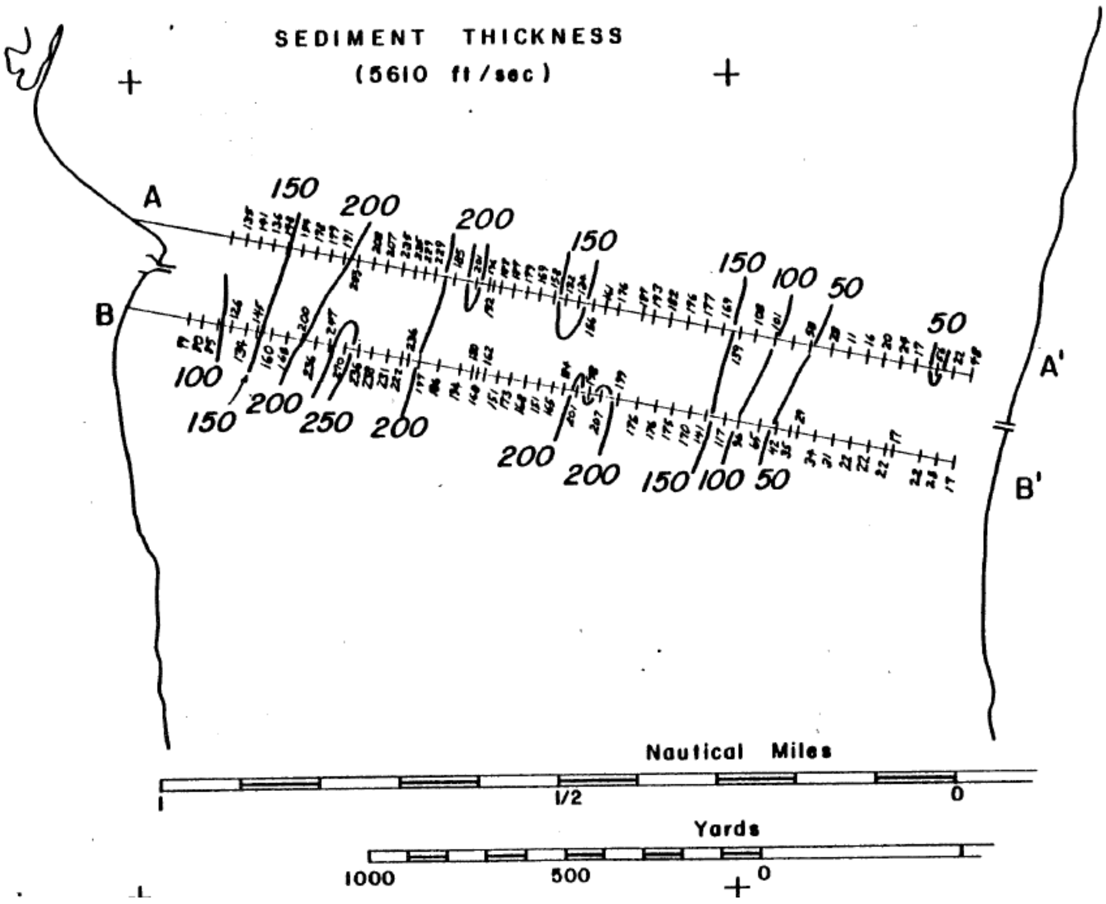


Figure 4-1 , Estimated Depth to Bedrock Based on Seismic Reflection Surveys (McMaster, 1979)

The purpose of the seismic survey was to determine the depth of bedrock and soil stratification, so the results of the two tracklines will not be compared. The bathymetric profile along section A-A’ indicated a range in water depths between 7 and 25 feet on the west approach and between 5 and 15 feet on the east approach. The profile also showed the slopes from the west and east converged on a “V” shaped valley with a depth of approximately 75 feet.

The sound signals produced from the seismic survey along trackline A-A’ were recorded and processed at URI. It was noted there was some difficulty in determining the sound velocities of the sediment deposits lying above the interpreted position of the bedrock due to numerous multiple reflections, reverberations, and bubble pulses. Therefore, the resolution was not sufficient enough to provide details of the stratification or match the velocities to the sediments obtained in the boring data provided by Guild Drilling Company. As a result, not a single sediment velocity could be determined.

Because of the multiple issues concerning the reliability of the seismic profile, the data were compared to other high resolution seismic profiles. The results of the comparison indicated the sediment overlying the bedrock most likely consisted of patches of glacial till overlain by regularly stratified sand-silt deposits, overlain by glacial outwash, which would be expected based upon the geology of the region. Based upon this assessment, a sediment velocity of 5,610 ft/sec was determined to best characterize the sediment pile in the Passage. The determined velocity was used to then determine the depth to bedrock in the Passage, which is also shown in Figure 4.1, and shows greater depths along the western approach, as deep as 235 feet.

4.3 Boring Logs and Lab Data from 1982-1984

The geotechnical consultants, Sverdrup, Parcel and Associates (SPA), provided two subsurface investigation reports, both of which included a boring program. The first report included thirty borings which were drilled by Warren George, Inc. of Jersey City, New Jersey, and assisted by Guild Drilling Co., Inc. of East Providence, Rhode Island during August and September of 1982. The second boring program included 30 borings as well which were drilled by Guild Drilling Co., Inc. as the subcontractor to SPA. Drilling occurred during September and October of 1984. (SPA, 1982)

4.3.1 Boring Log Data and Laboratory Results from 1982

The purpose of the 1982 report was to provide geotechnical data for the design of the alternatives for the bridge replacement. The borings for the proposed site of the main span and east approach were drilled to approximately 10 feet into bedrock. The borings for west approach and trestle area were drilled to predetermined depths, only some of which were drilled 10 feet into bedrock. Standard Penetration Test (SPT) data along with soils samples were generally obtained in 5 foot intervals.

Though soil samples were obtained, the state of the soil encountered during boring was noted to be insufficient for most laboratory tests (the contractor was only able to extract one undisturbed sample). As a result, the laboratory program only consisted of sieve analysis and Atterberg Limits tests which provided means of determining physical properties of the soils through an empirical evaluation. The laboratory tests were conducted by Goldberg Zoino and Associates, Inc..

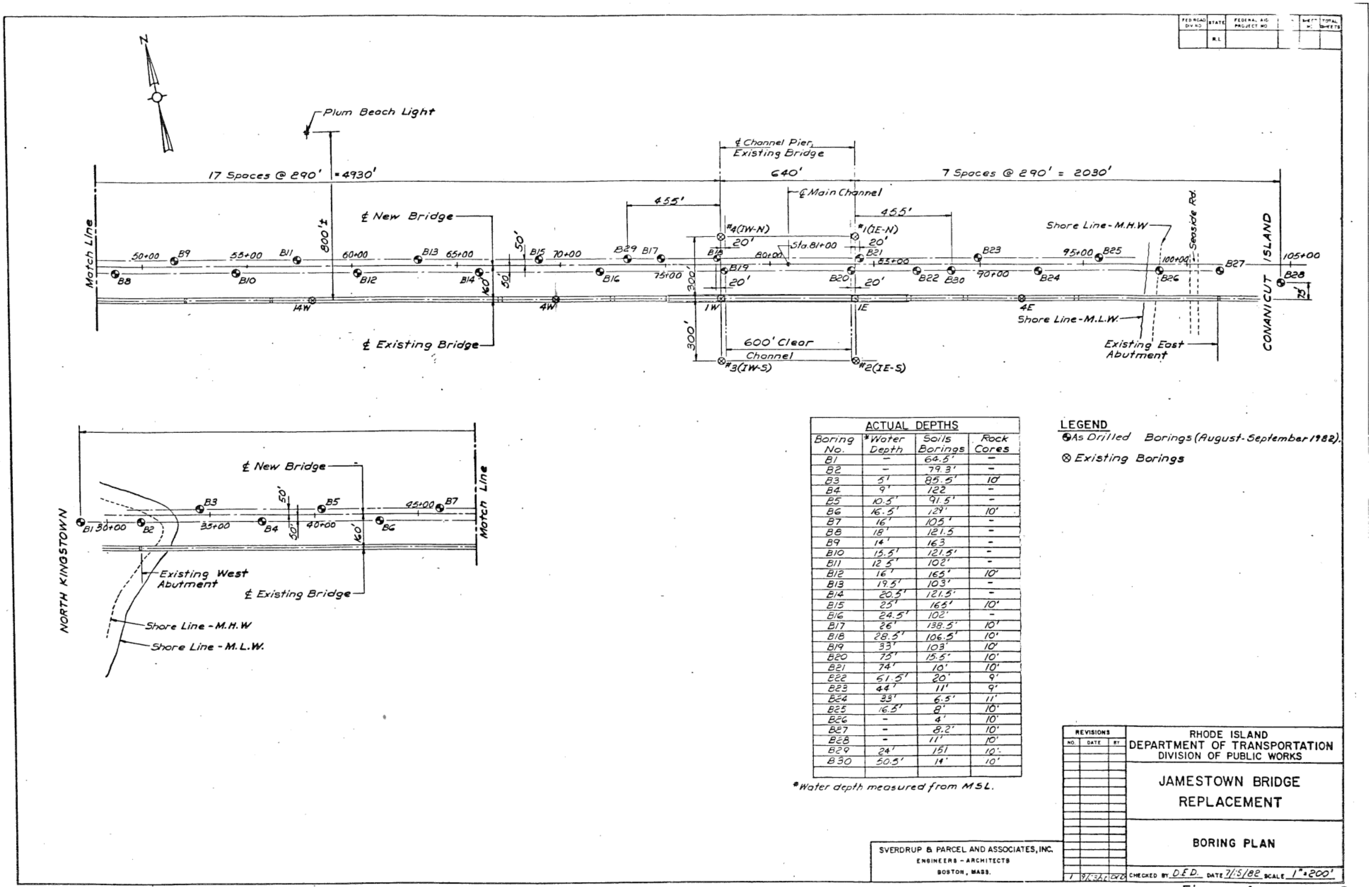
The results of the SPA (1982) report showed three general soil strata classified as strata I, II, and III. Stratum IA was determined to be a mixture of very loose fine sand, silt, sea shells and organic matter. Stratum II was determined to be a non-plastic soil containing mostly sand and silt size particles with ranging Unified Soil Classification System (USCS) classifications of SP, SM and ML. The density of the stratum was found to increase with depth from a state of very loose to very dense and was therefore subdivided into three categories; IIA (very loose to loose), IIB (medium dense to dense) and IIC (very dense). Stratum IIIA was classified as till and correlated to a ML USCS classification, with very dense to non-to-low plastic characteristics. The soil parameters, as determined by SPA, are shown in Table 4.1.

Table 4-1 Soil Parameters as Determined by SPA (1982)

Soil Stratum	φ' (deg)	γ (pcf)	c' (psf)	Average N Value (bpf)
IA	0	85	0	WOR
IIA	28	100	0	2
IIB	30	108	0	17
IIC	37	120	0	65
IIIA	40	130	0	113

The proposed span of the bridge was then divided into four areas for the purposes of providing generalized soil profiles. The four areas were: Trestle (Sta 28+50 to 54+00), North Kingstown Approach (Sta 54+00 to 73+25), Main Spans (Sta 73+25 to 88+75) and Jamestown Approach (Sta 88+75 to 104+50), also called the East Approach, areas. The test piles of interest in this study were located between Station 37+20 and Station 69+00, therefore only the Trestle and North Kingstown Approach (West Approach) areas will be discussed in detail. The SPT data taken from borings B-5, B-8, B-11 and B-15 were used by the designers to calculate the capacity and tip elevation of the piles. The locations of these borings are shown in Figure 4.2 and the boring logs can be found in Appendix A. Also, the borings were taken approximately every 250 feet along the proposed track of the bridge.

It is interesting to note that the geotechnical consultant, SPA, recommended precast prestressed concrete piles (to include pile load tests) for the Trestle area and pile foundations in the North Kingstown Approach area.



ACTUAL DEPTHS

Boring No.	*Water Depth	Soils Borings	Rock Cores
B1	–	64.5'	–
B2	–	79.3'	–
B3	5'	85.5'	10'
B4	9'	122	–
B5	10.5'	91.5'	–
B6	16.5'	129'	10'
B7	16'	105'	–
B8	18'	121.5	–
B9	14'	163	–
B10	15.5'	121.5'	–
B11	12 5'	102'	–
B12	16'	165'	10'
B13	19.5'	103'	–
B14	20.5'	121.5'	–
B15	25'	165'	10'
B16	24.5'	102'	–
B17	26'	138.5'	10'
B18	28.5'	106.5'	10'
B19	33'	103'	10'
B20	75'	15.5'	10'
B21	74'	10'	10'
B22	51.5'	20'	9'
B23	44'	11'	9'
B24	33'	6.5'	11'
B25	16.5'	8'	10'
B26	–	4'	10'
B27	–	8.2'	10'
B28	–	11'	10'
B29	24'	151	10'
B30	50.5'	14'	10'

*Water depth measured from MSL.

LEGEND

⊕ As Drilled Borings (August-September 1982).

⊗ Existing Borings

SVERDRUP & PARCEL AND ASSOCIATES, INC.
ENGINEERS - ARCHITECTS
BOSTON, MASS.

REVISIONS		
NO.	DATE	BY
1	9/23/82	DED

RHODE ISLAND
DEPARTMENT OF TRANSPORTATION
DIVISION OF PUBLIC WORKS

JAMESTOWN BRIDGE
REPLACEMENT

BORING PLAN

CHECKED BY D.E.D. DATE 7/15/82 SCALE 1" = 200'

A33215

Figure 4

6

Figure 4-2 Jamestown Verrazzano Bridge Boring Plan, SPA (1982)

4.3.1.1 Trestle Area Soil Profile

Borings B-1 through B-9 were taken from the Trestle area which was described as having a gently sloped bay bottom from the bank to a depth of 15 feet. The bay bottom surface material was observed to be approximately five feet of thick mud, overlaid by stratum IIA, which varied in thickness from 5 to 25 feet with an average blow count of 6 bpf. Stratum IIB was found to underlie IIA and varied in depth from 30 feet to greater than 100 feet. In boring B-5 it was observed that a five foot thick layer of stratum IIA was “sandwiched” by IIB and pockets of IIA. Throughout IIB were loose pockets of sand and silt. The average SPT N value for IIB was determined to be 30 bpf and was mostly fine sand. Underlying stratum IIB was stratum IIC which varied in thickness from approximately 5 feet at boring B-2 to 85 feet at boring B-9. It is interesting to note that borings B-7 and B-8 did not contain evidence of stratum IIC above an elevation of -140 feet, indicating quite a variance in stratification. The average SPT N value for stratum IIC was determined to be 66 bpf.

4.3.1.2 North Kingstown Approach Soil Profile

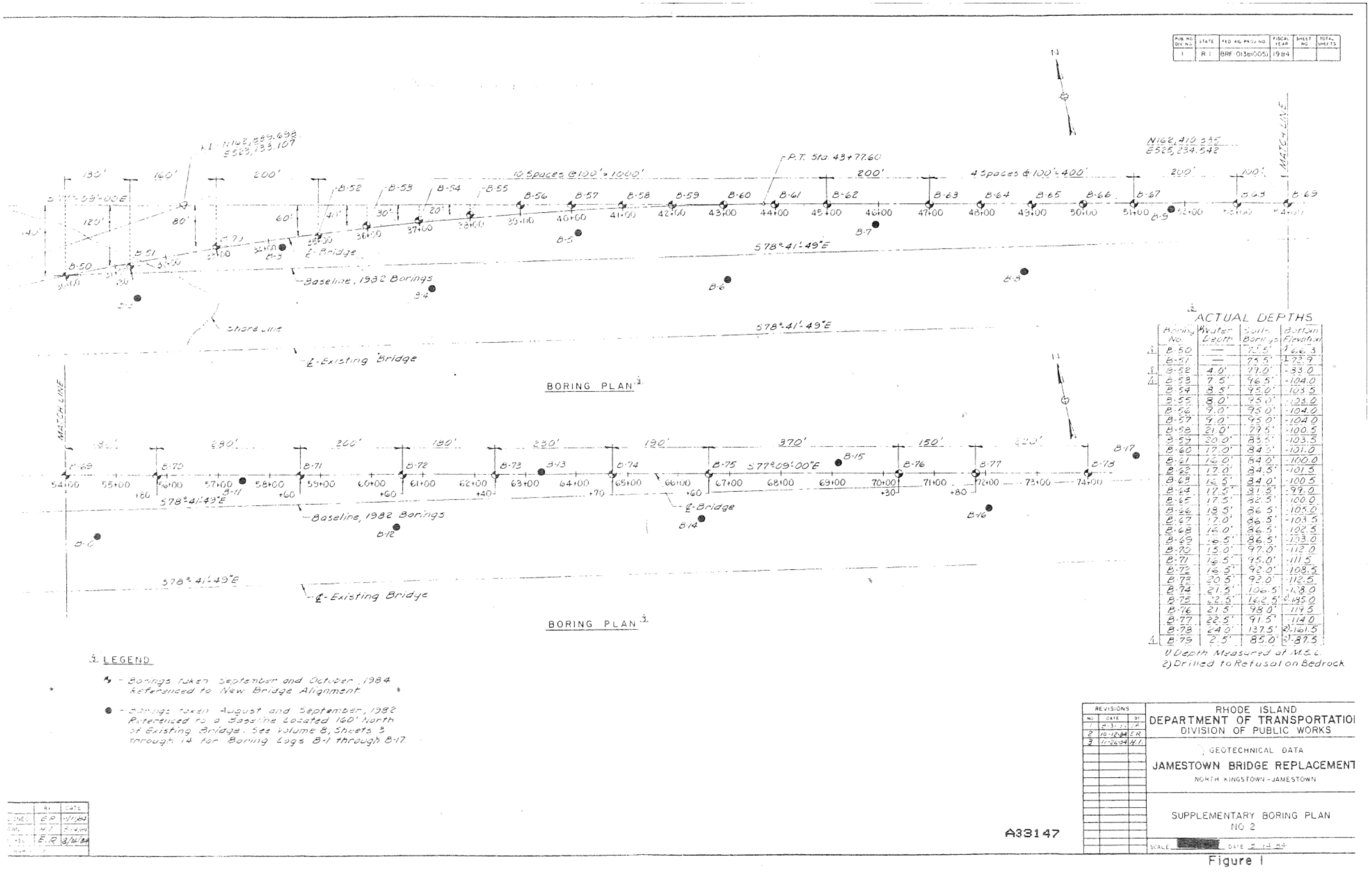
Borings B-10 through B-16 and B-29 were taken from the North Kingstown Approach area. It was determined that the soil stratification described in the Trestle area also applied to the North Kingstown Approach area with a few exceptions. The water depth varied from approximately 15 feet to 25 feet. Boring B-15 indicated very dense glacial till, stratum IIIA, overlying bedrock at elevation -190 feet. Boring B-15 also contained a 10 foot core of the bedrock which was determined to be of good quality with a Rock Quality Designation (RQD) of 80%. Coincidentally, boring B-15 was the only boring used for design purposes that contained evidence of bedrock.

4.3.2 Boring Log Data from 1984

The purpose of the 1984 subsurface investigation was to provide additional geotechnical data for the Trestle and North Kingstown Approach areas based upon the decision to move the proposed site of the bridge an additional 150 feet to the north of the original site (a report of the findings was not available to the author for analysis). Most borings were drilled to predetermined depths and core samples were not obtained. SPT data along with soils samples were generally obtained in 5 foot intervals. Lab tests were not performed on any of the samples by the contractor.

Borings B-50 through B-69 and B-79 were taken from the Trestle area and borings B-70 through B-78 were taken from the North Kingstown Approach area. The results of the borings indicated the same general soil strata classified as strata I, II, and III. On average, the borings were taken to elevations less than the borings taken in 1982 and not taken further than strata IIC or drilled to refusal in bedrock. In both cases, an indication of till, strata IIIA, was not recorded.

It was observed that the1984 borings show a more complex stratification in the Trestle area, particularly at depths between -40 and -80 feet. This may be a result of the borings being taken approximately every 100 feet along the defined baseline instead of every 250 feet, as was the case for the 1982 borings. The location of the 1984 borings are shown in relation to the 1982 borings in Figure 4.3. The boring logs can be found in Appendix B.



ACTUAL DEPTHS

Boring No.	Water Depth	Soils Borings	Bottom Elevation
B-50	—	72.5'	1) 66.3
B-51	—	75.5'	1) 72.9
B-52	4.0'	77.0'	-83.0
B-53	7.5'	96.5'	-104.0
B-54	8.5'	95.0'	-103.5
B-55	8.0'	95.0'	-103.0
B-56	9.0'	95.0'	-104.0
B-57	9.0'	95.0'	-104.0
B-58	21.0'	79.5'	-100.5
B-59	20.0'	83.5'	-103.5
B-60	17.0'	84.0'	-101.0
B-61	16.0'	84.0'	-100.0
B-62	17.0'	84.5'	-101.5
B-63	16.5'	84.0'	-100.5
B-64	17.5'	81.5'	-99.0
B-65	17.5'	82.5'	-100.0
B-66	18.5'	86.5'	-105.0
B-67	17.0'	86.5'	-103.5
B-68	16.0'	86.5'	-102.5
B-69	16.5'	86.5'	-103.0
B-70	15.0'	97.0'	-112.0
B-71	16.5'	95.0'	-111.5
B-72	16.5'	92.0'	-108.5
B-73	20.5'	92.0'	-112.5
B-74	21.5'	106.5'	-128.0
B-75	22.5'	162.5'	2) -185.0
B-76	21.5'	98.0'	-119.5
B-77	22.5'	91.5'	-114.0
B-78	24.0'	137.5'	2) -161.5
B-79	2.5'	85.0'	2) -87.5

1) Depth Measured at M.S.L.
2) Drilled to Refusal on Bedrock

LEGEND

- Borings taken September and October, 1984 Referenced to New Bridge Alignment
- ● - Borings taken August and September, 1982 Referenced to a Baseline Located 160' North of Existing Bridge. See Volume 8, Sheets 3 through 14 for Boring Logs B-1 through B-17.

REVISIONS		
NO	DATE	BY
1	8-31-84	J.F.
2	10-12-84	E.R.
3	11-26-84	H.I.

RHODE ISLAND
DEPARTMENT OF TRANSPORTATIO
DIVISION OF PUBLIC WORKS

GEOTECHNICAL DATA
JAMESTOWN BRIDGE REPLACEMENT
NORTH KINGSTOWN - JAMESTOWN

SUPPLEMENTARY BORING PLAN
NO 2

SCALE [redacted] DATE 8-14-84

Figure 1

Figure 4-3 Jamestown Verrazzano Bridge Boring Plan, SPA(1984)

4.4 Boring Logs from 1987

The purpose of the 1987 subsurface investigation was to determine the bedrock elevation. A total of 50 borings were taken, numbered D-100 through D-149. Unfortunately, a boring plan was not available, so the locations of the borings in relation to the proposed bridge site are unknown to the author. However, the stations of 18 borings were recorded on the boring logs and indicated depths which ranged from station 47+27 to 72+73 and indicated bedrock elevations ranging from -139 feet (MSL) to -218 feet (MSL). The mentioned borings span the area of the Trestle and North Kingstown Approach areas and, if it is assumed the borings were taken in the vicinity of the proposed bridge site, the bedrock elevation depths compare well to the bedrock elevation depths of the McMaster (1979) survey.

Had a boring plan been available, the contractor that performed the drilling, C. E. McGuire Inc., only collected SPT data near bedrock depth. Therefore, the subsurface investigation would not have been as relevant to the reduced pile capacity issue as the SPA(1982) and SPA(1984) subsurface investigations as they provided SPT data which could be correlated to shaft resistance. Furthermore, the test piles in the Trestle area and North Kingstown Approach are were initially to be friction piles so the depth to bedrock would only have been significant had the 1987 subsurface investigation indicated bedrock was at elevations much higher than was indicated in the McMaster (1979) survey.

4.5 Soil Testing and Analysis from 1988-1989

The University of Rhode Island provided soil testing and analysis in two phases for RIDOT and the Maguire Group, Inc., which provided physical and shear strength properties. The testing and analysis program was conducted on four soil borings from the supplementary boring log program conducted in 1987 and were provided by the Maguire Group, Inc. The testing program included USCS classification, direct shear tests, vane shear, consolidation tests, permeability tests, CD triaxial tests and CU triaxial tests. A summary of the soil classification and averaged strength parameters (determined from direct shear tests) are shown in Table 4.2. Of the four borings, 11 Shelby tubes were provided and the boring, station and depth corresponding to each Shelby tube is shown in Table 4.3.

Table 4-2 Summary of URI Soil Testing and Analysis Results (after Silva et al., 1988)

Material Category	φ'_p (deg)	φ'_r (deg)	$\varphi'_p - \varphi'_r$ (deg)	ρ (g/cm^3)	γ (pcf)	USCS Symbol
A	32.8	32.4	0.4	1.76	109.9	CL
B	43	28.6	14.4	2	124.8	ML
C_1	45.6	32	13.6	1.99	124.2	SM
C_2	36.4	36.4	0	1.95	121.7	SM
D	47.1	34	13.1	1.98	123.6	SP
E	43.4	39.3	4.1	1.75	109.2	SP

Table 4-3 Sample Tube Locations (after Silva et al., 1988)

Boring/Station	Depth (ft, below MSL)	Material Category
D-114 / 43+08	39.5-42.0	C_1
	54.0-56.5	B
	71.5-74.0	E
D-129 / 51+48	52.0-54.0	C_1
	62.0-64.0	D
D-136 / 57+30	71.0-73.0	D
D-145 / 68+46	32.5-34.0	C_1
	47.0-49.0	C_2
	57.0-59.0	A
	77.0-79.0	B
	102.0-103.6	B

The direct shear results from phase I (Silva et al., 1988) of the laboratory tests show that material category (soil stratum) A had the lowest φ'_p value of 32.8 degrees and category D the highest at 47.1 degrees. Category B had the lowest φ'_r value of 28.6 degrees and category E the highest at a value of 39.3 degrees. Categories A, C_2 and E showed little to no difference between φ'_p and φ'_r though categories B, C_1, and D showed a significant difference ranging between 13.1 and 14.4 degrees. The results of the direct shear tests for phase I indicated a wide range of volumetric behavior with categories B, C_1, D, and E showing an overall tendency towards dilation and categories C_2 and A showing a tendency towards contraction. The wet bulk densities (ρ) were determined and also shown in Table 4.2. Category B showed the highest γ value of 124.8 pcf whereas category E had the lowest value of 109.25 pcf.

Additional direct shear tests were conducted in phase II (Silva et al., 1989) of the laboratory tests with a focus on the effects of re-cycling after large strains. A series of five tests were conducted on category C_1 material in which the material was re-cycled just once. The results indicated peaking stress-strain behavior, indicative of dilative material, during the first cycle, and contractive behavior during the second cycle. Only one re-cylcing test was conducted on the category B, C_2, and E materials, in which similar behavior was observed. The author noted that the residual strength of the denser materials was the same for both cycles indicating little to no degradation of strength once the material was subjected to large deformations (Silva et al., 1989).

According to USCS classifications, soils identified as either ML or SM contain non-plastic to low plasticity fines indicating a greater tendency for deformation. Material category B was identified as ML and both C_1 and C_2 were identified as SM.

It is also noted that direct shear testing on non-plastic silts taken from Shelby tubes will lead to higher shear strength values. This is due to the tendency of the soil to display dilatant tendencies during shear, rather than contractive tendencies at an in situ state.

5. ANALYSIS OF THE JAMESTOWN VERRAZZANO BRIDGE TEST PILE PROGRAM

The intent of this chapter is to provide an analysis of the data used to determine the ultimate capacity of the piles and to assess the expected ultimate capacity based upon the static capacity results. It is noted that the original design calculations for the Trestle and West Approach foundations were not available to the University of Rhode Island.

5.1 PDA, CAPWAP and SLT Data

CAPWAP was used by CFF to determine the ultimate capacity of the test piles based upon the PDA results at the End Of Driving (EOD) and Beginning Of Restrike (BOR). The ultimate capacity values determined by PDA rarely indicated the pile had reached the design capacity. The CAPWAP results only once indicated the ultimate capacity had been reached, though the SLT proved otherwise. A summary and comparison of the PDA and associated CAPWAP and SLT values will be provided for each test pile in the following sections.

5.1.1 WATP-1

The design ultimate capacity for WATP-1 was estimated to be 340 tons at a design elevation of -90 feet. Pile driving of the test pile began on 9 April, 1986 and was stopped once the PDA indicated a capacity of 346 tons had been reached. A re-strike the following day indicated possible pile set-up and an ultimate capacity of 517 tons, though the CAPWAP analysis showed much lower capacities, on the order of 192 tons and 196 tons respectively. A SLT was conducted on 21 April, 1986 which provided an ultimate capacity of only 83 tons. The decision was made to splice the pile (the initial pile length was 110 feet and was shortened to 79 feet for the purpose of conducting the SLT) and continue to drive the pile to the design elevation of -90 feet. It is noted that after the SLT, the PDA damping value was adjusted from 0.2 to 0.6.

Pile driving continued after a 67 foot splice was added. Two instances of pile head damage occurred during driving, resulting in the removal of over 8 feet of pile. The pile was eventually driven to a depth of -96 feet at which point pile driving operation ceased due to the pile breaking (which was assumed to have broken in the area of the splice). The pile was subsequently abandoned and eventually replaced by a composite test pile. A comparison of the SLT, PDA and CAPWAP data is shown in Table 5.1 and indicates the CAPWAP values predicted 63% of the PDA capacity. Conversely, the CAPWAP value taken at BOR after the SLT was performed was 52 tons greater than the expected failure load.

Table 5-1 WATP-1 SLT, PDA and CAPWAP Results

WATP-1
(Design elevation of -90 ft, design capacity of 340 tons)

Date	PDA Capacity (tons)	CAPWAP (tons)	Tip Elevation (ft)
9-Apr-86	346	192	-71.4
10-Apr-86	517	196	-72.1
*21-Apr-86		135	-72.1
16-May-86	125	102	-72.4
**20-May-86		151	-78
27-Jun-86	180	140	-97.5

*SLT performed indicated an 83 ton ultimate capacity
**PDA data not available

At first glance the results in Table 5.1 show evidence of pile relaxation as the ultimate capacities determined with CAPWAP decrease with time as the pile remains at a relatively fixed depth. However, the pile was driven an additional 25 feet at which point the CAPWAP value is still less than half of the design capacity.

Lastly, the pile experienced two instances of pile head damage and failed at the splice. These occurrences suggests a drivability issue with the hammer used at the time, Delmag D36-23, was not driving the pile in a manner which allowed for the transfer of energy of the hammer in a such a way that mobilized skin and shaft capacity without causing damage to the pile.

It is widely accepted that if SPT blow counts exceed 10 blows per inch, the maximum toe capacity is not being mobilized. An analysis of the SPT blow counts, taken from the time the CAPWAP analysis was completed, and toe mobilization was performed. The results of the analysis are shown in Figure 5.1 along with the CAPWAP determined ultimate capacity at each BOR or EOD. Even if there was a drivability issue with the Delmag D36-23, Figure 5.1 suggests that the full pile toe capacity was developed as indicated by blow per inch values ranging from 3.3 to 8. Figure 5.2 shows the depth of the pile toe when each CAPWAP analysis was performed. The figure shows that despite the fact that the first four restrikes fully mobilized the pile toe capacity, decreased capacity was observed. Furthermore, when the pile is driven an additional 20 feet in June, the ultimate capacity decreases suggesting evidence of a loose soil, however the 1984 borings show the pile toe was located at a strata transition in which the deeper strata consisted of a very dense sand/silt. It is also noted that the 1984 borings indicated a mostly homogenous layer of medium dense sand from elevation -36 feet to -99 feet.

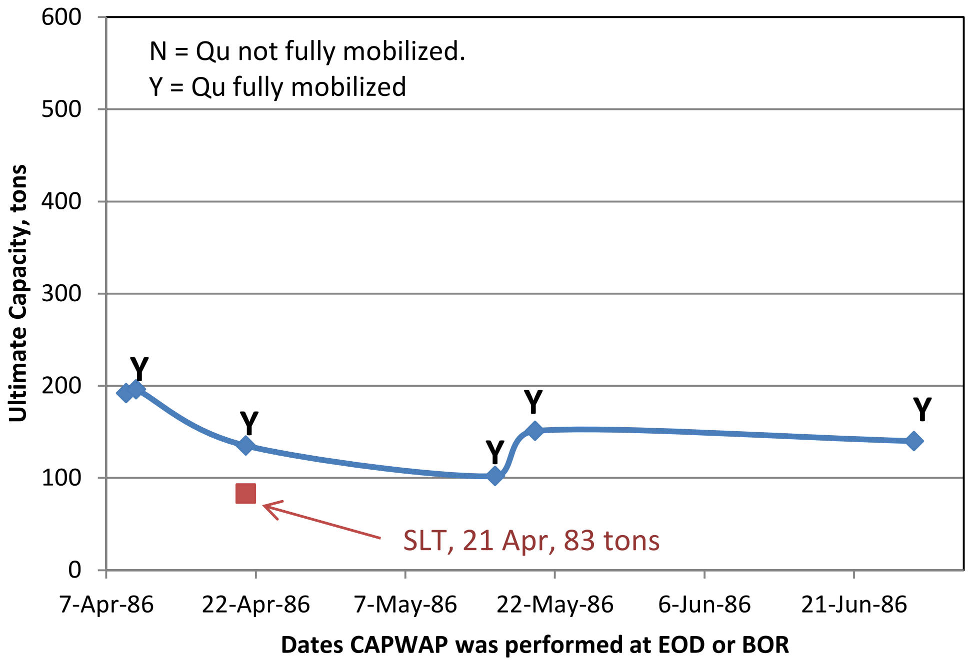


Figure 5-1 WATP-1 CAPWAP Determined Ultimate Capacity Values and Pile Toe Mobilization

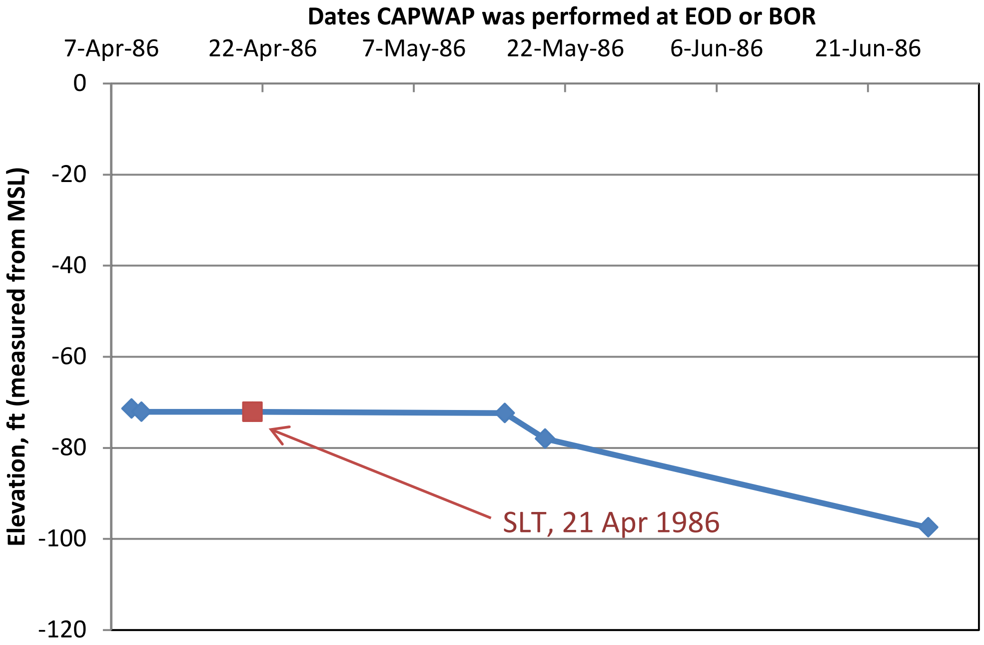


Figure 5-2 WATP-1 Pile Toe Elevation at Time CAPWAP Analysis was Performed

5.1.2 WATP-2

The design ultimate capacity for WATP-2 was estimated to be 340 tons at a design depth of -95 feet. The length of the initial pile was 120 feet. Pile driving of the test pile began on 1 May, 1986 and was initially stopped at an elevation of -109 feet, due to the need to splice the pile as a result of a CAPWAP determined capacity of 45 tons. Prior to installing the splice, the pile was re-struck on 13 May, 1986, and a slight increase in capacity was observed. A 20 foot splice was installed on 16 May, 1986 and pile driving operations continued. At an elevation of -128 feet, pile driving was ceased due to spalling of the pile head. The PDA damping value was then adjusted to 0.5 which corresponded to a CAPWAP ultimate capacity of 169 tons. After repairs to the pile head were completed, pile driving continued once again on 20 May, 1986 and was stopped at an elevation of -140 feet which corresponded to a CAPWAP capacity prediction of 205 tons in which an increased PDA damping value of 0.6 was used. A restrike was conducted over a month later on 30 June, 1986 and the CAPWAP determined capacity was 211 tons. On 24 July, 1986, a static load test was performed which yielded an ultimate capacity of 240 tons. Based on the results of the static load tests for both WATP-1 and WATP-2 it was agreed upon to increase the size of the hammer in August of 1986. This determination was made due to the predicted need to drive the remaining piles to depths well beyond the design depths requiring a more efficient hammer (CFF and RIDOT correspondences, 1986). The Delmag D46-23 was subsequently used by the contractor to conduct a restrike on 24 September, 1986. The results of the re-strike yielded a CAPWAP determined capacity of 149 tons.

A summary and comparison of the SLT, PDA and CAPWAP data is shown in Table 5-2. The CAPWAP values compared well to the PDA results up to the first restrike, however, afterwards the values varied as much as 48%. Because a restrike was not performed after the SLT, a direct comparison to the CAPWAP predicted capacity cannot be made. It is noted, however, that the CAPWAP capacity values determined on 30 June and 24 September indicate either pile relaxation or a lack of pile toe capacity mobilization.

Table 5-2 WATP-2 SLT, PDA and CAPWAP results

WATP-2
(Design elevation of -95 ft, design capacity of 340 tons)

Date	PDA Capacity (tons)	CAPWAP (tons)	Tip Elevation (ft)
1-May-86	35	45	-109
13-May-86	79	70	-110.2
16-May-86	278	169	-128
20-May-86	160	205	-140.5
30-Jun-86	280	211*	-140.6
24-Sep-86	284	149	-141.4

*SLT performed on 24 July 1986 indicated a 240 ton ultimate capacity

An analysis of ultimate pile toe mobilization was conducted and the results are shown in Figure 5.3 which indicates that after the second restrike further restrikes did not sufficiently mobilize the toe capacity. Decreasing CAPWAP capacity values over time indicate pile relaxation, however, the SLT value of 240 tons would indicate neither set up or relaxation considering the SLT capacity is only slightly greater than CAPWAP capacity values at nearly the same elevation as shown in Figure 5.4.

Furthermore, the 1984 boring logs indicated the soil stratum from elevation -107.5 feet to the end of boring, -119.5 feet, consisted of very dense to hard non-plastic silt with corresponding corrected average blow counts increasing from 22 to 46 at the mentioned depths (it was assumed the stratum underlying the end of boring was of the same soil type). The high blow counts encountered could have been a result of the build-up of excess pore water pressures due to undrained conditions in the dense non-plastic silts during SPT, which is also the same condition for CAPWAP. Upon dissipation of the excess pore water pressures, the shear strength of the stratum of silt would decrease, and could have effectively decreased the ultimate capacity of the pile.

Because the SLT capacity was greater than the CAPWAP capacity at the same depth, it could be assumed the pile tip was bearing in medium dense sand instead of dense non-plastic silt as a CAPWAP value obtained for a pile in dense non-plastic silt would likely yield a higher value than the measured value.

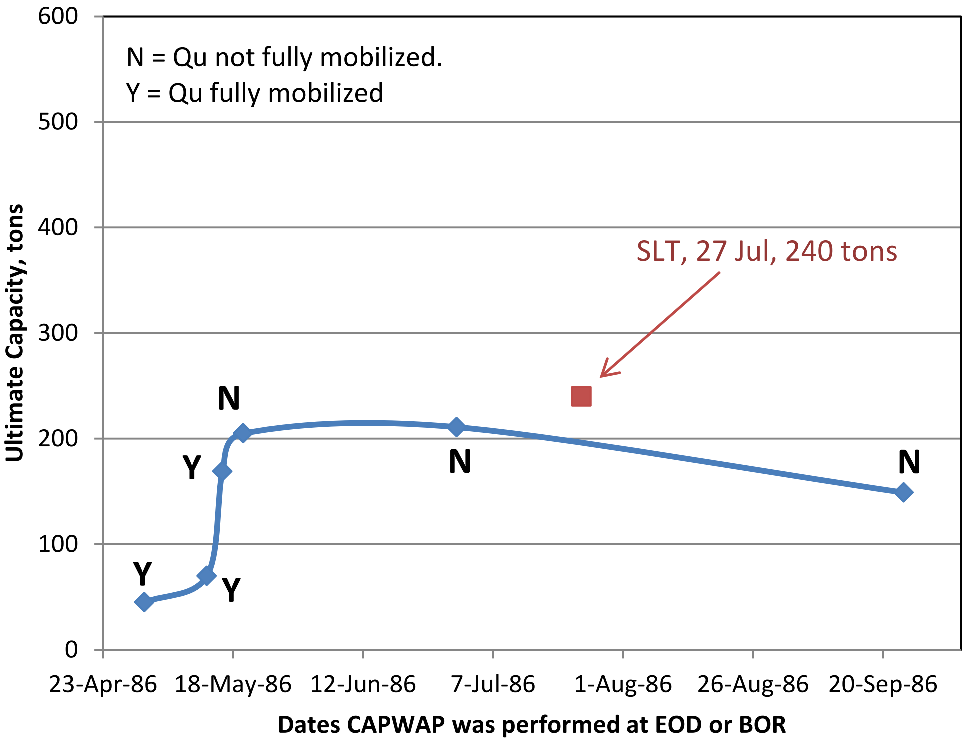


Figure 5-3 WATP-2, CAPWAP Determined Ultimate Capacity Values and Pile Toe Mobilization

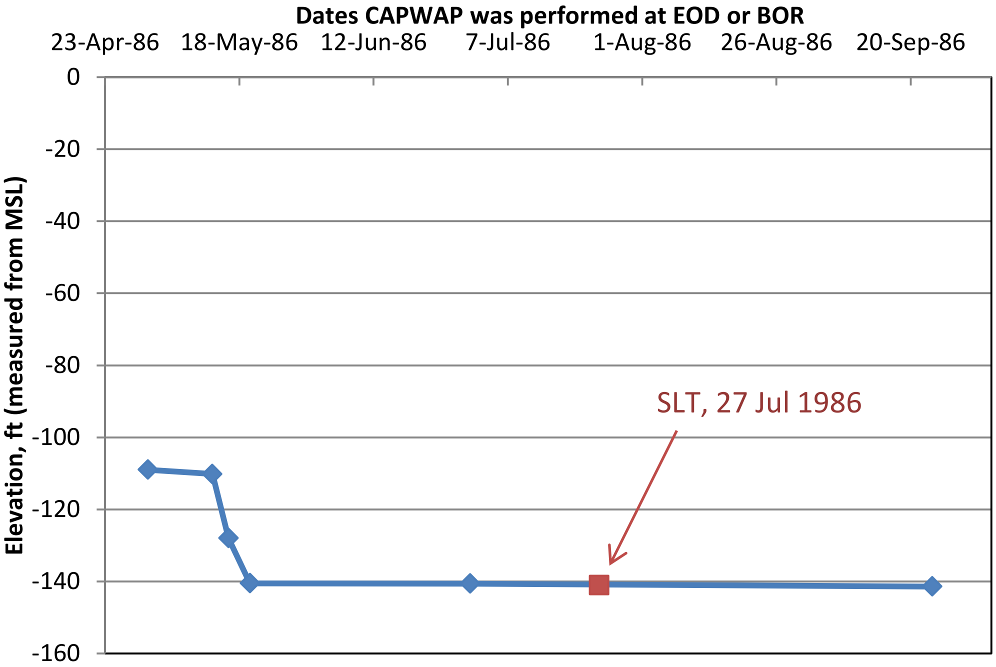


Figure 5-4 WATP-2 Pile Toe Elevation at Time CAPWAP Analysis was Performed

5.1.3 TTP-1

The design ultimate capacity for TTP-1 was estimated to be 330 tons at a design elevation of -77 feet. Pile driving of the test pile began on 16 October, 1986, using the Delmag D46-23 hammer, and stopped on 20 October, 1986 at an elevation of -100 feet. An adjusted PDA damping value of 0.5 at EOD corresponded to a CAPWAP ultimate capacity of 200 tons. A restrike was performed almost two months later on 11 December 1986 in which the CAPWAP capacity was determined to be 205 tons. The test pile was spliced and pile driving resumed on 17 December, 1986, and stopped when the PDA capacity value of 360 tons was achieved at an elevation of -110 feet. A subsequent CAPWAP analysis indicated a lower capacity of 237 tons. Pile driving operations continued the same day and after the pile was driven 6 inches, the PDA capacity value was 460 tons, using a damping value of 0.63, with a corresponding CAPWAP value of 242 tons. A restrike of the pile was conducted on 29 January, 1987 and after the pile was driven 1.5 inches the PDA capacity value was 340 tons, using a damping value of 0.65 and the CAPWAP determined value was 370 tons. A static load test was performed on the pile on 17 February, 1987, which yielded an ultimate capacity of 180 tons, 170 tons lower than the required 330 tons. A summary of the PDA, CAPWAP and SLT data is provided in Table 5.3.

Table 5-3 TTP-1 Soil Parameter Correlations

TTP-1
(Design elevation of -77 ft, design capacity of 330 tons)

Date	PDA Capacity (tons)	CAPWAP (tons)	Tip Elevation (ft)
20-Oct-86	200	200	-100
11-Dec-86	220	205	-100.1
17-Dec-86	330	237	-110
17-Dec-86	460	271	-110.5
29-Jan-87	340	370*	-110.6

*SLT performed 17-Feb-86 indicating an 180 ton ultimate capacity

An analysis of ultimate pile toe mobilization was conducted and the results are shown in Figure 5.5 which shows three of the five CAPWAP determined ultimate capacity values were obtained when the ultimate toe capacity was fully mobilized. The figure also shows that with each restrike the capacity of the pile increased whether or not the ultimate toe capacity was fully mobilized which could be expected as the pile elevation increased with each restrike, as shown in Figure 5.6. It is also noted that even though the ultimate toe capacity was not fully mobilized, the restrike performed on 29 January, 1987, indicates pile set-up as the previous restrike was performed more than month before and showed a significantly lower capacity. The SLT performed on 17 February, 1987, however, shows that either significant pile relaxation occurred following the last restrike, as indicated by a 190 ton difference in capacity, or that the CAPWAP soil model did not accurately represent the in situ conditions.

An analysis of boring log B-54 showed that the boring was taken to an elevation of -103.5 feet, which was 7 feet above the final elevation of the test pile. Although it may be assumed the soil layer was homogenous at this depth , the 100 ton increase in the

CAPWAP values (December 1986 and January 1987 values) resulting from less than 2 inches of additional penetration corresponding to a lack of ultimate pile toe mobilization suggests the pile toe bears in dense non-plastic silt. This assumption could explain the 190 difference in the final CAPWAP and SLT ultimate capacity values as the two values are obtained under different drainage conditions.

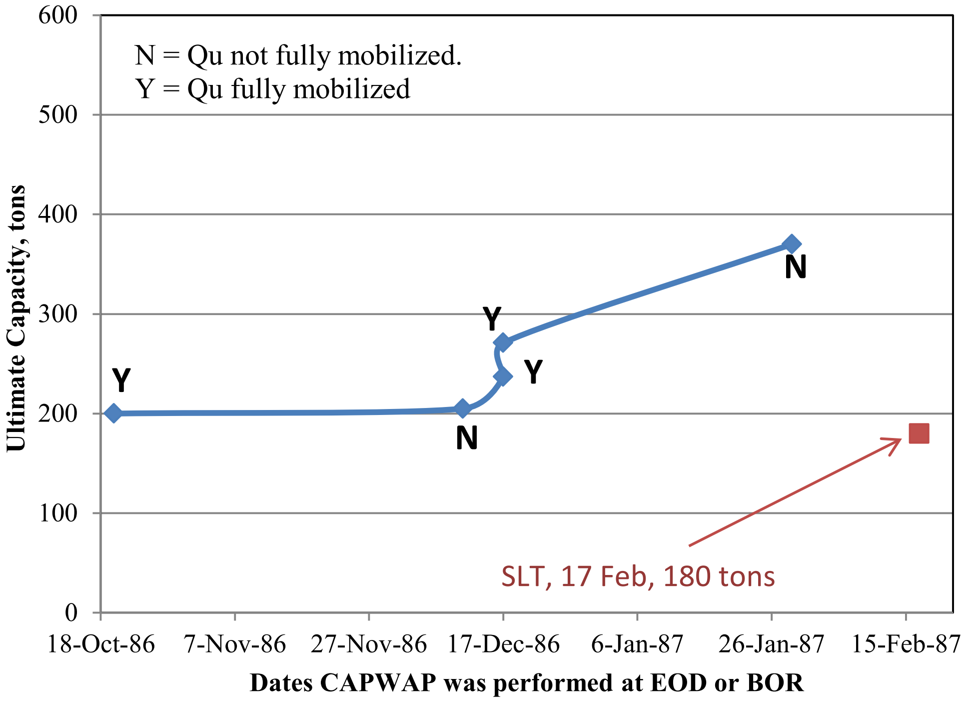


Figure 5-5 TTP-1 CAPWAP Determined Ultimate Capacity Values and Pile Toe Mobilization

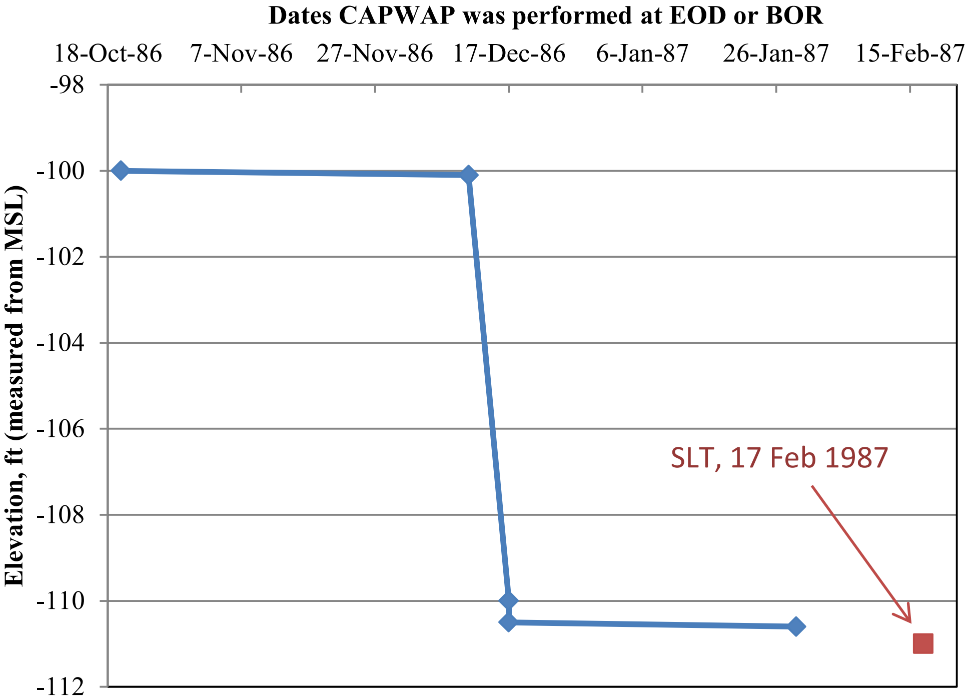


Figure 5-6 TTP-1 Pile Toe Elevation at Time CAPWAP Analysis was Performed

5.1.4 TTP-4

The design ultimate capacity for TTP-4 was estimated to be 330 tons at a design elevation of -57 feet. Pile driving of the test pile began on 21 October, 1986 using the Delmag D46-23 hammer, and stopped the same day at an elevation of -105 feet. A CAPWAP analysis performed at EOD indicated a capacity of 207 tons, corresponding to a PDA capacity of 230 tons using a J value of 0.5. The pile was spliced and pile driving operations continued on 13 November, 1986 until a PDA determined capacity of 556 tons was reached. A subsequent CAPWAP analysis indicated a much lower capacity of 252 tons. Pile driving operations continued the following day and stopped prematurely due to mechanical problems with the hammer, at which point a CAPWAP analysis was performed and indicated a capacity of 260 tons which corresponded to a PDA value of 212 tons using an increased J value of 0.9. Pile driving operations continued on 17 November, 1986, until the maximum pile length was driven which coincided to an elevation of -168 feet. A CAPWAP analysis was performed at EOD and indicated a capacity of 291 tons. A restrike was performed the following day and a CAPWAP capacity was determined to be slightly higher at 323 tons.

A static load test was performed on 1 December, 1986, and indicated an ultimate capacity of 520 tons which was 190 tons greater than the required capacity. A restrike and subsequent CAPWAP analysis was performed on 4 December, 1986, and indicated a lower capacity of 227 tons which corresponded to a PDA value of 210 tons using a damping value of 0.7. A summary of the PDA, CAPWAP and SLT data is provided in Table 5.4.

Table 5-4 TTP-4, Summary of PDA, CAPWAP and SLT data

TTP-4
(Design elevation of -57 ft, design capacity of 330 tons)

Date	PDA Capacity (tons)	CAPWAP (tons)	Tip Elevation (ft)
21-Oct-86	230	207	-105
13-Nov-86	-	188	-106.2
13-Nov-86	556	252	-111.5
14-Nov-86	212	260	-145
17-Nov-86	172	291	-168
18-Nov-86	180	291	-168.3
4-Dec-86	210	227	-168.4

*SLT performed 1-Dec-86 indicating a 520 ton ultimate capacity

An analysis of ultimate pile toe mobilization was conducted and the results are shown in Figure 5.7. The figure shows only three of the seven CAPWAP determined ultimate capacity values were obtained when the ultimate toe capacity was fully mobilized. The figure also shows that with each restrike the capacity of the pile increased whether or not the ultimate toe capacity was fully mobilized which could be expected as the pile elevation increased with each restrike, as shown in Figure 5.8. Even though it appears each restrike performed indicated pile set-up as capacities consistently increased, the CAPWAP values do not directly compare due to the varying J values used.

Based upon the SLT performed on 1 December, 1986, it is clear that the CAPWAP analysis does not correlate well to the actual soil conditions. This observation can also be confirmed by the varying PDA damping values used when CAPWAP analyses were performed as the values ranged from 0.5 to 0.9. Furthermore, the last two restrikes performed before and after the SLT indicate pile relaxation. Though it cannot be ruled out pile relaxation occurred, the fact that the SLT showed an ultimate capacity almost twice the value predicted using CAPWAP, again indicates poor correlation between the two methods.

An analysis of boring log B-67 showed that the boring was taken to an elevation of -103.5 feet, which was 65 feet above the final elevation of the test pile (-168.5 feet). The significant difference in CAPWAP and SLT values suggests the pile toe was bearing in medium dense sand/silt. This assumption is made due to the observed blow counts in excess of 20 blows per inch for the final three restrikes accomplished to obtain CAPWAP values which can be correlated to drained conditions and a dissipation of excess pore water pressures leading to pile set up (increased capacity).

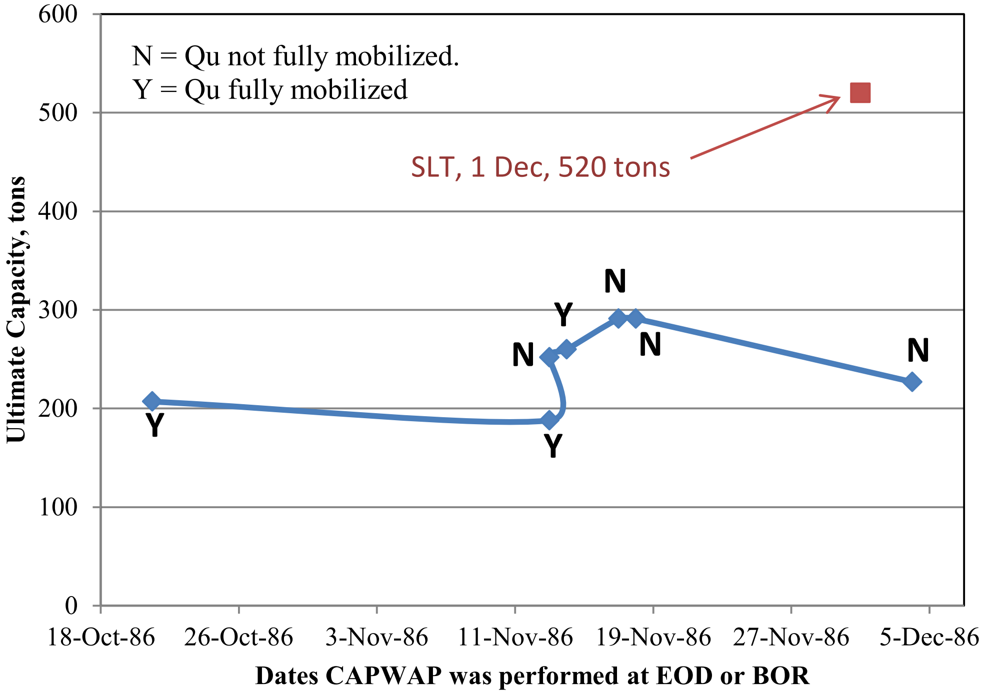


Figure 5-7 TTP-4 CAPWAP Determined Ultimate Capacity Values and Pile Toe Mobilization

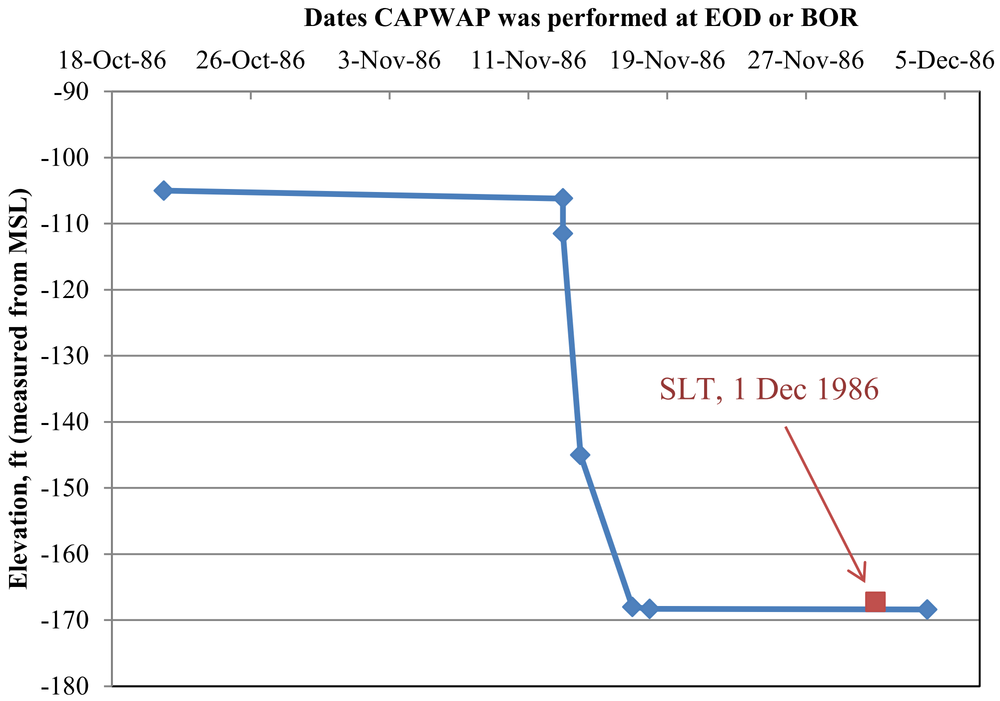


Figure 5-8 TTP-4 Pile Toe Elevation at Time CAPWAP Analysis was Performed

5.2 Static Capacity Analysis

Multiple boring logs were used in determining the static capacities of each pile for this thesis. This section will discuss the varying soil parameters determined from the boring log data and the ultimate capacities determined from each set of boring log data will also be discussed and compared to the static load test determined capacities.

5.2.1 Boring Logs Used for Analysis

Static capacity analysis was performed on each pile using SPA 1982 and 1984 boring log data corresponding to each test pile location. The same SPA 1982 borings the designers used to determine the test pile capacity (B-5, B-8, B-11, and B-15) were used in as a means to verify the ultimate capacities. Table 5.5 provides the boring numbers, depth to which the deepest boring was taken, depth at which the SLT was conducted and the distance from the test pile station. The reason the depth of the SLT is shown is that the static capacity analysis values were determined at the corresponding depth for each test pile in order to conduct a direct comparison.

Table 5-5 Borings Used for Static Capacity Analysis

Test Pile	Boring	Depth (MSL) (ft)	SLT Depth (ft)	Distance from pile (ft)
TTP-1	B-54	-103.5	-111	20
	B-5	-102		290
TTP-4	B-67	-103.5	-167.25	40
	B-8	-139.5		290
WATP-1	B-70	-112	-72.1	80
	B-11	-116		70
WATP-2	B-76	-119.5	-141	100
	B-15	-200		10

Table 5.5 shows that for TTP-1 and TTP-4 the static load test depth exceeded the depth to which the borings available for analysis were taken. The borings used in the analysis for WATP-1, however, were of sufficient depth and for WATP-2 only one of the borings was of sufficient depth. It is also noted that the boring distance from the test piles varied as little as 10 feet and as much as 290 feet.

5.2.2 Determination of Friction Angles

As mentioned in Chapter 2, the friction angles were correlated to SPT data from the boring logs using the Peck et al. (1974) method as well as the Bowles (1977) method and Terzaghi, Peck & Mesri (1996). The purpose of using three different methods was to determine the possible range of friction angle values. The Peck et al. (1974) method, as previously mentioned, is the most widely used method of determining friction angles based upon SPT data. Accordingly, the method was used to determine the friction angles. The values determined using this method on average fell between the high and low values for the three different methods used.

The friction angles determined for the layers comprised of mostly of sand compared well to the range of accepted values, 28-42 degrees. The friction angles of Rhode Island silt were obtained using samples from numerous sites during a study carried out by Page (2004) where friction angle values for normally consolidated inorganic silts where determined to range between 24 and 30 degrees. This range of values would have been expected for the layers corresponding to URI material categories B, C_1 and C_2 as the samples were designated as USCS categories ML or SM. Only material category B is within the expected range, however, material categories C_1 and C_2, as indicated by URI lab data, have on average higher void ratios, which would explain the higher friction angles.

5.2.2.1 WATP-1

Table 5.6 shows the soil characteristics by layer as determined by SPA data, URI lab analysis and SPT values from the 1984 boring log B-70. The SPA and URI correlations were developed by the depth and location of the borings relative to the test pile location. Because of the limited number of borings provided to URI for the purposes of lab testing, very few correlations were made. With the exception of layer 1, the friction angles determined from the 1984 borings compared well to the SPA values which were slightly higher on average. The URI determined peak friction angle values for layer 3 were an average of 13 degrees greater than the calculated values.

Table 5-6 WATP-1 Soil Parameter Correlations

		1984 Borings			URI Lab Analysis			1982 Borings	
Layer	Depth (ft) (from MSL)	φ' (deg)	N	$(N_1)_{60}$	Material Category	φ'$_p$ (deg)	φ'r (deg)	SPA Data Correlation	φ' (deg)
1	15-36	28	3	5				IA	0
2	36-51	32	16	17				IIB	30
3	51-99	33	25	22	D, C_1	47, 45	34,32	IIB	30
4	99-111	41	49	33				IIC	37

5.2.2.2 WATP-2

Table 5.6 shows the soil characteristics by layer as determined by correlations to SPA data, URI lab analysis and SPT values determined from the 1984 boring log B-76. With the exception of layers 5, the friction angle values determined from B-76 compare well with the SPA values. Regarding layer 5, the friction angle value determined from B-76 is 10-12 degrees higher than the SPA determined values. The URI determined peak friction angle values for layers 2 and 3 are significantly higher than the SPA and B-76 values.

Boring B-76 was taken to an elevation of -119.5 feet which was lower than the actual pile toe elevation of -141 feet, therefore, it was assumed the bottom layer was homogenous from the top of the layer to the pile toe elevation.

Table 5-7 WATP-2 Soil Parameter Correlations

		1984 Borings			URI Lab Analysis			1982 Borings	
Layer	Depth (ft) (from MSL)	φ' (deg)	N	$(N_1)_{60}$	Material Category	φ'_p (deg)	φ'r (deg)	SPA Data Correlation	φ' (deg)
1	21.5-37.5	38	21	37				IA.IIA	0, 28
2	37.5-62.5	31	12	14	C_1 , C_2 , A	45, 36 32	32,36,32	IIA,IIB	28, 30
3	62.5-92.5	29	6	6	B	43	28	IIA,IIB	28, 30
4	92.5-107.5	31	16	12				IIB	30
5	107.5-119	40	68	45				IIA,IIB	28, 30

5.2.2.3 TTP-1

Table 5.8 shows the soil characteristics by layer as determined by correlations to SPA data, URI lab analysis and SPT values determined from the 1984 boring log B-54. With the exception of layers 4, the friction angle values determined from B-54 compare well with the SPA values. Regarding layer 4, the friction angle value determined from B-54 is 4.5 degrees greater than the SPA determined values. The URI determined peak friction angle values for layers 1, 2 and 4 are significantly higher than the SPA and B-54 values.

Boring B-54 was taken to an elevation of -103.5 feet and the corresponding SPA boring, B-5, was taken to an elevation of -102 feet. Both elevations were lower than the actual pile toe elevation of -111 feet, therefore, it was assumed the bottom layer for both cases was homogenous from the top of the layer to the pile toe elevation.

Table 5-8 TTP-1 Soil Parameter Correlations

		1984 Borings			URI Lab Analysis			1982 Borings	
Layer	Depth (ft) (from MSL)	φ' (deg)	N	$(N_1)_{60}$	Material Category	φ'_p (deg)	φ'r (deg)	SPA Data Correlation	φ' (deg)
1	8.5-48.5	30	9	11	C_1	45	32	IIA, IIB	28, 30
2	48.5-57.5	31	14	14	B, D	43, 47	28,34	IIB	30
3	57.5-67.5	29	8	7				IIA	28
4	67.5-91.5	35	39	31	E	43	39	IIB	30
5	91.5-103.5	31	21	15				IIB	30

5.2.2.4 TTP-4

Table 5.9 shows the soil characteristics by layer as determined by correlations to SPA data, URI lab analysis and SPT values determined from the 1984 boring log B-67. Boring B-67 was taken to an elevation of -110 feet and the corresponding SPA boring, B-8, was taken to an elevation of -139 feet. Both elevations were lower than the actual pile toe elevation of -168.4 feet, therefore, it was assumed the bottom layer for both cases was homogenous from the top of the layer to the pile toe elevation.

The friction angle values determined from B-67 are slightly higher than the SPA values varying as much as 4.9 degrees. The URI determined peak friction angle values for layer 2 are significantly higher than the SPA and B-67 values, varying as much as 19 degrees.

Table 5-9 TTP-4 Soil Parameter Correlations

		1984 Borings			URI Lab Analysis			1982 Borings	
Layer	Depth (ft) (from MSL)	φ' (deg)	N	$(N_1)_{60}$	Material Category	φ'_p (deg)	$\varphi'r$ (deg)	SPA Data Correlation	φ' (deg)
1	17-38	32	12	17				IA, IIA	0, 28
2	38-83	35	26	25	D, C_1	47, 45	34,32	IIA, IIB	28,30
3	83-110	35	36	26				IIB	30

5.2.3 Pile Capacity Determined Using the Nordlund Method

Three static capacity methods were used to determine the pile toe capacity and three methods were used to determine shaft capacity. As mentioned in Chapter 2, the Nordlund Method is the most widely used static capacity method for cohesionless soils, therefore, the values determined using this method were compared to the design capacities, as shown in Table 5.10, and SLT capacities determined at the depth for which each SLT was conducted, as shown in Table 5.11. The soil description is included in Table 5.11 as a means of assessing the varying values between the calculated and measured capacities. Due to TTP-4 being driven to a depth well beyond the boring depths for boring B-67 and B-8 (1984 and 1982 borings, respectively), an end bearing soil description could not be ascertained. For each test pile, the ultimate capacities determined using the Nordlund Method fell between the high and low values determined using other static capacity methods mentioned in Chapter 2.

Table 5-10, Comparison of Design Capacity and Static Capacity Values

		1982 Boring Data	1984 Boring Data
Test Pile	Design Capacity (tons)	Nordlund Method at Design Depth (tons)	Nordlund Method at Design Depth (tons)
WATP-1	340	516	375.4
WATP-2	340	290.5	295.5
TTP-1	330	329.8	583
TTP-4	330	164	331.3

Table 5-11, Comparison of Design Capacity, SLT Values and Static Capacity Values

			1982 Boring Data	1984 Boring Data
Test Pile	End Bearing Soil Description	SLT Results (tons)	Nordlund Method at SLT Depth (tons)	Nordlund Method at SLT Depth (tons)
WATP-1	Sand	83	303	398
WATP-2	Sand	240	2332	1972
TTP-1	Dense Silt	180	2375	713
TTP-4	Inconclusive	520	1536	2435

Static capacities were also determined at design depth and were compared to the ultimate capacities determined using DRIVEN program, which utilized the Nordlund Method as well. The values determined by the DRIVEN program were based upon the 1982 boring log data. A summary of the DRIVEN results and comparison to predicted capacity values is shown in Table 5.12.The difference between the DRIVEN and calculated values was determined to be the result of:

- A δ/φ value of 0.85 was assumed. The value is normally taken from Hannigan et al. (1998) Figure 9.10, based upon displaced volume and pile type. However, a value was not obtainable due to the displaced volume value exceeding a point of intercept with the pile type curve.
- The C_F values were determined from Hannigan et al. (1998), Figure 9.15, which is subject to interpretation.
- The limiting unit toe resistance was determined from Hannigan et al (1998) Figure 9.17, which involved extracting values from an exponential curve corresponding to limiting values ranging from 0 - 40,000 kPa which was also subject to interpretation.

Table 5-12, Comparison of DRIVEN, Design Capacity, Measured Capacity and Static Capacity Values

			1982 Boring Data	1984 Boring Data	
Test Pile	End Bearing Soil Description	SLT Results (tons)	Nordlund Method at SLT Depth (tons)	Nordlund Method at SLT Depth (tons)	DRIVEN Results (tons)
WATP-1	Sand	83	303	397	353
WATP-2	Sand	240	2332	1972	2340
TTP-1	Dense Silt	180	2375	713	2192
TTP-4	Inconclusive	520	1536	2435	1977

It is shown through static capacity analysis that the design capacities were within the range of predicted ultimate capacities at the design depth. However, the static capacity values determined at the corresponding SLT depths were on the range of 3 to 13 times higher.

5.2.4 Effects of Drainage Conditions on SPT, CAPWAP and SLT

It is widely accepted that saturated cohesionless (sand) soils tend to dissipate pore pressures very rapidly when stresses are applied. As such, strength values are obtained in the laboratory under drained conditions. Though silt is considered cohesionless, it is composed of fine grained material and loading can occur under drained or undrained conditions. It was observed in the Silva et al. (1989) report that material categories B, C_1, C_2, and D displayed dilative behavior during shear under undrained conditions. This observation indicates that both SPT testing and pile driving likely occurred under undrained conditions, resulting in a decrease in pore pressures that lead to higher initial strengths. These higher strengths would be reflected in both the SPT blow counts and the pile capacity predicted by PDA and CAPWAP. However, this phenomenon is unlikely to occur during a SLT in silt as the loads applied during an SLT are small in comparison to the hammer blows imparted on the pile and are applied over a greater increment of time (the Quick Load Test was used for each SLT in which 10-15% of the design load was applied approximately every 5 minutes). To illustrate this point, Table 5.13 summarizes the assumed drainage conditions for sands and silts for SPT, CAPWAP analysis, and SLT.

Table 5-13 Drainage Conditions for Sands and Silts During CAPWAP, SPT and SLT

	Sand	Silt
SPT	Drained	Undrained
CAPWAP	Drained	Undrained
SLT	Drained	Drained

This highlights an important fact that in silts, the SPT and dynamic pile capacity methods may occur under undrained conditions, while a SLT is performed under drained conditions. For this case study, there was clear evidence of dilative silts in the vicinity of the Jamestown Verrazzano Bridge. SPT and CAPWAP was performed without mention of the different drainage conditions for both silts and sands and it is therefore implied that both methods were carried out under the assumption drained conditions occurred regardless of soil type.

As shown in Figure 5.14, the end bearing soil for both WATP-1 and WATP-2 consisted of sand, therefore SLT and CAPWAP values were obtained under drained conditions. The CAPWAP capacity value for WATP-1 is 39% greater than the SLT value. The CAPWAP capacity value for WATP-2 is less than 12% less than the SLT value. Though the variance is high, it is much less than the variance of 58% for TTP-1, for which the CAPWAP capacity value was most likely obtained under undrained conditions due to the end bearing soil consisting of dense silt. Because the end bearing soil was determined to be dense silt, it can be assumed the CAPWAP capacity value would have been greater than the actual capacity, which is shown to be the case. Though boring logs were not available to determine the end bearing soil for TTP-4, the large variance in SLT and CAPWAP capacity values could be attributed to the dilative tendencies of silt.

Table 5-14 Comparison of End Bearing Soil, SLT and CAPWAP at SLT Depth

Test Pile	End Bearing Soil Description	Static Load Test Results (tons)	CAPWAP Capacity (tons)
WATP-1	Sand	83	135
WATP-2	Sand	240	211
TTP-1	Dense Silt	180	308[1]
TTP-4	Inconclusive	520	227[2]

[1] CAPWAP analysis performed two months prior to static load test

[2] The restrike performed 3 days after load test

6. SUMMARY AND CONCLUSIONS

The Jamestown Verrazzano test pile program and associated correspondence and lab data was investigated for this thesis. The objective was to identify possible causes for the significant difference in predicted and measured pile capacities which led to a $97 million cost overrun and a pile redesign for the Trestle and West Approach sections of the bridge. The tasks accomplished included the following:

- An in depth literature review was conducted which focused on static capacity analysis methods, static and dynamic load testing, and phenomena which lead to real and apparent pile relaxation such as dilation, arching and friction fatigue.
- An analysis of all subsurface investigations was conducted.
- A summary of the test pile program was developed based upon project correspondence and reports.
- Static capacities for each test pile were estimated based upon the results of the subsurface investigations. The results were compared to the static load test and dynamic load test values of the test piles.
- Pile toe mobilization was analyzed based upon available blow count data for each pile.

6.1 Summary of Subsurface Investigations

The depth to bedrock as determined from the seismic reflection results compared well to the subsurface investigation of 1987. This indicates the seismic reflection data could have been sufficient to determine depth to bedrock for the purposes of pile design. An analysis of the URI lab tests showed that the difference between the peak and residual friction angles was not significant enough to cause the observed reduced capacities for each test pile alone. In fact, the residual friction angles were, on average, higher than the SPT determined friction angle values, which would have yielded greater capacity values if the residual friction angles were used for design purposes. These findings suggest that adequate subsurface investigations were conducted for the purpose of the test pile design.

6.2 Static Capacity Analysis

The results of the static capacity analysis for each test pile show that, given the wide range of determined densities based upon multiple boring logs, the test piles should have met the predicted ultimate capacities at the respective design depths. This was not the case for any of the test piles. Furthermore, for the three piles driven well beyond the design depth, only one reached the design capacity following driving. The results of static capacity analyses (both from hand calculations and using a FHWA software package DRIVEN) conducted at the SLT depth for each test pile indicated each test pile should have exceeded the design capacity, however, the measured capacities indicated significantly lower values for each test pile.

6.3 Summary of Conclusions

Based on a thorough analysis of the available data, there does not appear to be one overarching cause for the reduced capacity observed with each test pile. It is, therefore, hypothesized that a combination of factors lead to reduced capacities. These are summarized below.

Assessment of the Friction Angle - The friction angle and unit weight of a soil heavily influence the results of a static capacity analysis. If the URI determined peak friction angles were to be used to determine capacity, it is noted that the values were, on average, higher than both the friction angles determined by the managing consultant Sverdrup & Parcel and Associates (SPA) and the friction angles calculated from the 1984 boring logs. In only four instances were the URI determined residual friction angles lower than the SPA or 1984 boring log determined friction angles, the greatest variance being 2.6 degrees for layer 2 of the TTP-4 soil profile. When the lower value is used to determine the shaft capacity, the shaft resistance is reduced by 35 tons at the design depth and 106 tons at the SLT depth. It is noted the lower friction angle value reduces the calculated ultimate capacity of 295 tons, which is below the design capacity, however, this value is still greater than the CAPWAP determined value. The lower value at the SLT depth is insignificant as the value is still much greater than the SLT capacity.

Dilation - In a laboratory study of inorganic silts from on-land sites in Providence, RI, Page (2004) observed that the Skempton pore pressure parameter, $\bar{A}_f$, for contractive samples was approximately 1.3 and ranged from 0.40 (slight tendency for contractive behavior) to -0.26 for dilative samples. It was also shown that at as the Over Consolidation Ratio (OCR) increased from 2, the samples displayed dilative behavior. The URI lab data from the Jamestown Bridge project also indicated a tendency for dilation for four of the six soil materials studied. Soil materials B, C_1 and C_2 were found to have displayed dilative tendencies at high densities and contractive behavior at lower densities during direct shear tests. Although soil material D did not contain much evidence of silt, it was observed to be highly dilative.

Dilation in soils during pile driving has been linked to pile relaxation over time. During the onset of this research, it was preemptively assumed pile relaxation due to dilation was the cause for the significant difference between the predicted and measured ultimate capacities. Because the soil models used with the CAPWAP analysis were significantly different than the in situ conditions, as determined by comparison to the static load test results, a determination of whether actual or apparent pile relaxation occurred could not be ascertained as the values determined by each method did not correlate well. Also, CAPWAP capacities determined at varying time intervals could not be used with much certainty to determine the occurrence of real or apparent relaxation as different values of damping were used for the PDA data set utilized by the program at each interval or the ultimate toe capacity was not fully mobilized. However, if the negative pore pressures caused by dilation were generated rapidly and the soil material was of high permeability, it could be assumed a reduction in soil strength was realized following pile driving operations, which could account for some portion of reduced pile capacity.

Friction Fatigue - The method proposed by Vesic (1970) to determine the maximum shaft capacity due to the phenomenon of friction fatigue was to utilize the capacity at a depth of 10 to 20 times the pile diameter as the maximum value of shaft friction. This method was based upon a study conducted in cohesionless soil. Even if Vesic's (1970) method was reliable, the study was conducted in coarse-grained cohesionless soils, not including silt, and may not correlate well with the present study. The method also conflicts with the common methods used to determine shaft capacity in that limiting factors are not applied for such an effect. Also, the methods would have proved to be inaccurate beyond the limiting depth proposed by Vesic (1970).

Other studies have been conducted by Chow & Jardine (1996), White & Lehane (2004), Gavin & O'Kelley (2007), to correlate reduced shaft capacities to the effects of cyclically loading a pile in cohesionless soils, however, none of them provided a substantiated correlation. Though the concept of friction fatigue is generally accepted, there does not exist a means of predicting the effects of friction fatigue on the test piles at Jamestown.

If the method put forth by Vesic (1970) was utilized in a conservative fashion by determining the shaft capacity at a depth 20 times the pile diameter for TTP-4, the maximum shaft capacity, no matter what the depth the pile was to be driven to, would be 42 tons. This would imply that 478 of the 520 ton capacity determined from the SLT was developed by the pile toe capacity. The pile toe capacities for TTP-4 using the Nordlund Method from both the 1982 and 1984 borings were 73 and 248 tons, respectively, indicating shaft capacities of 447 and 272 tons. Therefore, it seems a shaft capacity value of 42 tons would not have been feasible.

Arching - According to Jardine and Chow (1998) it can be assumed arching occurred during pile driving. As indicated by the study, there was an average of a 40% reduction in predicted versus measured capacity at EOD. The reduction in predicted pile capacity due to the arching effect, however, occurs during pile driving and once pile driving operations cease, pile capacity begins to increase, as much as 85%, over a time span of six months to five years.

Because only one static load test was conducted per test pile and a lack of strong correlations between CAPWAP and the static load tests, the extent to which arching reduced the predicted capacities cannot be accounted for as there is not a credible capacity to compare the baseline capacity of the single static load test. Furthermore, an 85% increase in capacity would only account for the decreased capacities of WATP-1 and TTP-4, for which the SLT values were 27% and 34%, respectively, of the predicted capacities. As such, even though arching may have contributed to a reduction in predicted pile capacity, it does not account for the reduced pile capacity for each test pile alone.

Liquefaction - The behavior of Rhode Island silt during undrained cyclic loading was studied by Taylor (2011) and Bradshaw (2006), and the results of the study were discussed in Chapter 2. As mentioned in Chapter 5, a variance of *J* values, 0.2 - 0.9, was observed in the CAPWAP analysis. Liquefiable soils tend to dissipate more energy than non-liquefied soils, therefore, if it is assumed the increase in *J* values are attributed to increases in excess pore pressures, liquefaction may have occurred in strata associated with the higher *J* values. It is widely known that liquefaction must occur for the pile to be driven, however, it seems plausible that as the silt layers transitioned from a liquefied state, the friction angles were well below the residual friction angles determined in the lab (Silva, 1988) which could have led to reduced capacity values. This effect would also be exacerbated by the arching effect in the non-plastic silt pockets, further reducing the predicted capacity.

6.4 Recommendations for Future Research

According to Salgado (2008), there is a limited amount of literature discussing the effects of silt during pile driving in addition to a lack of correlations related to the static capacity analysis of displacement piles in silt. Perhaps the lack of literature is a result of the rare occurrence of experiencing large strata of silt in the vicinity of deep foundation installation. Nonetheless, the importance of understanding the behavior of silts during pile driving operations is critical to the design of deep foundations in the regions in which the encounters are not rare, such as the North Eastern United States and South Western Canada (Thompson, 1985). As such, the following recommendations for future research are offered:

- Perform drained and undrained cyclic load tests on Rhode Island silts commensurate to cyclic pile driving
    - Compare residual friction angles and post-liquefaction undrained strengths in order to determine the effects of cyclic loading on the shear strength of the soil
- Monitor pore pressure during cyclic loading

  - Pore pressure measurements taken over time after sample is cyclically loaded will lead to certainty regarding pile set up and relaxation
  - Pore pressure measurements will indicate the degree of contraction and dilation during and after cyclic loading
- Perform drained and undrained cyclic load tests on Rhode Island silts, commensurate to cyclic pile driving in submerged materials
  - Monitor the pile-soil interface for evidence of increased water content
  - Indications of increased water content will confirm water enters the pile-soil interface which could reduce the effective stress in the dilative strata

APPENDIX A: 1982 BORING LOGS

SVERDRUP & PARCEL
BORING LOG

BORING NO. B-5

Job No. 8089 Project Jamestown Bridge Replacement

Drilling Co. Warren George Inc.

Driller E. Peterson
P. Brescia Inspector H. Shaffer
E. Rundhovde

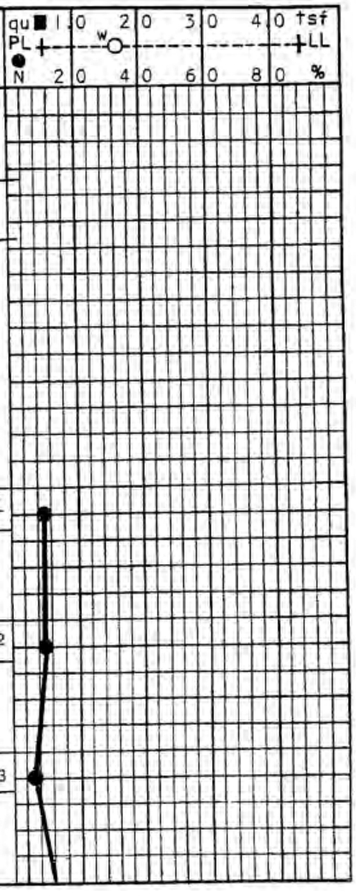
Date Started 8/30/82 Date Completed 8/31/82

Drilling Equipment Data Barge Mounted Mobile Drill B-40

Station	40+10
Offset	50' North ℄
Coordinates	
Ground Elev.	-10.5 (MSL)

Ground Water Table	
@ Complete	( ) Hrs. Later

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL	DEPTH (FT.)	SAMPLE
		WATER		
-10.5		MUD, dark-blue, fine sand, silt, sea shells, very soft, wet.	0	
-13.5		SAND, dark-grey, medium, trace of silt, loose, wet.	5 10 15	1 2 3
-26.5		SILT, dark-grey, some fine sand, loose, wet.		
-30			20	



Sheet 1 of 3

A33246

BORING NO. B-5

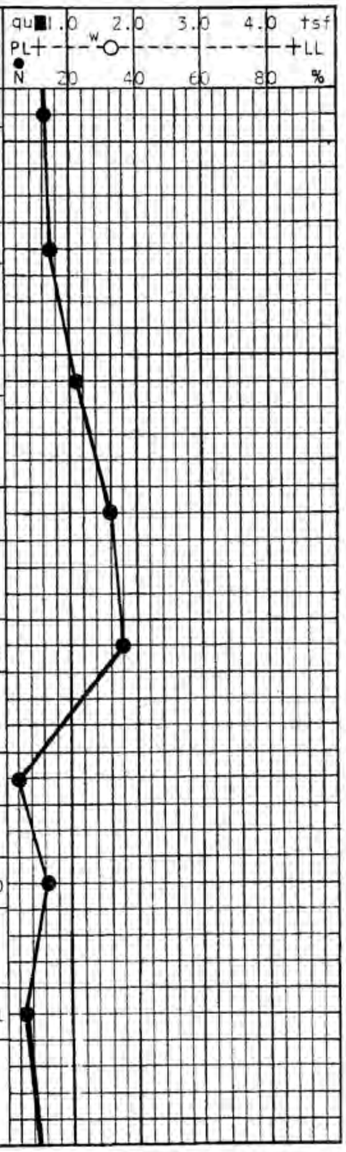
SVERDRUP & PARCEL

BORING LOG

BORING NO. B-5

Job No. 8089 Project Jamestown Bridge Replacement

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL	DEPTH (FT.)	SAMPLE
		SAND, dark-grey, medium, trace of silt, trace of coarse sand, medium dense, wet.	25 30	4 5 6
-44.5		SAND, brown-grey, fine to medium, trace of silt, dense, wet.	35 40	7 8
-54.5		SAND, dark-grey, fine, little silt, loose, wet.	45	9
-59.5		SAND, grey-brown, coarse, trace of silt, medium dense, wet.	50	10
-62.5		SAND, dark-grey, fine, little silt, loose, wet.	55 60	11



Sheet 2 of 3

A33247

BORING NO. B-5

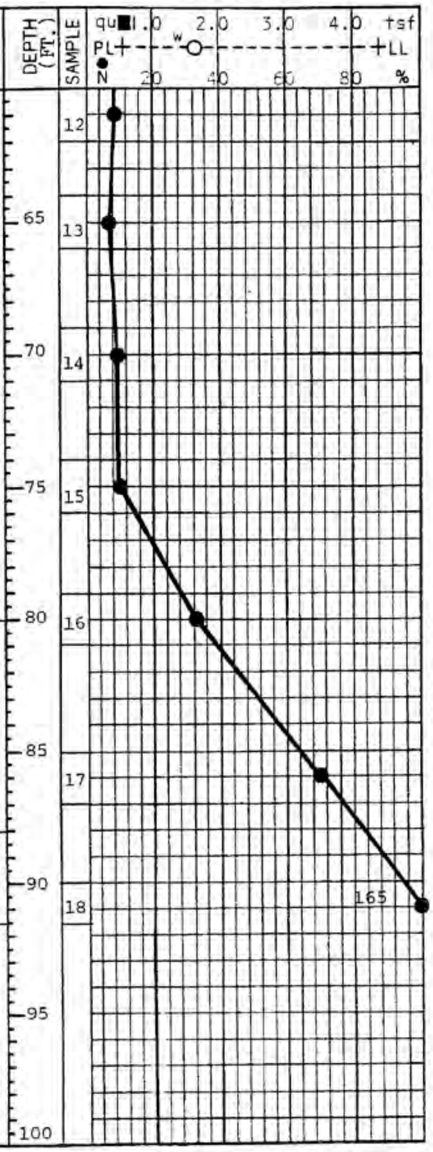
SVERDRUP & PARCEL

BORING LOG

BORING NO. B-5

Job No. 8089 Project Jamestown Bridge Replacement

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL
		SAND, dark-grey, fine, little silt, loose, wet.
-90.5		SILT, dark-grey, little clay, little fine sand, very stiff, varved, wet.
-98.5		SAND, dark-grey, fine, little silt, trace of coarse gravel, very dense, wet
-102		End Boring Notes: 1. Boring cased to El. 25 with SW casing. 2. Boring cased to El. -101.5 with HW casing.



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A33248

BORING NO. B-5

SVERDRUP & PARCEL
BORING LOG

BORING NO. B-8

Job No. 8089 Project Jamestown Bridge Replacement

Drilling Co. Warren George Inc.

Driller P. Brescia
E. Peterson Inspector H. Shaffer
E. Rundhovde

Date Started 9/9/82 Date Completed 9/10/82

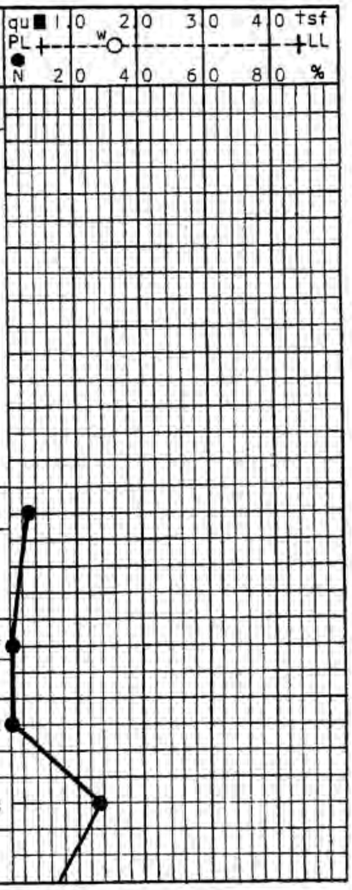
Drilling Equipment Data Barge Mounted

Mobile Drill B-40

Station	48+80
Offset	50' South ℄
Coordinates	
Ground Elev.	-18 (MSL)

Ground Water Table	
@ Complete	( ) Hrs. Later

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL	DEPTH (FT.)	SAMPLE
		WATER		
-18		MUD, black, fine sand, silt, sea shells, organic odor, very soft, wet.	0	
-23		SAND, grey, fine to medium, trace of silt, trace of sea shells, organic odor, very loose, wet. * no recovery	5 10	1 * 2
-33.5		SAND, grey, fine, little silt, medium dense, wet.	15	3
-38			20	



Sheet 1 of 4

A33258

BORING NO. B-8

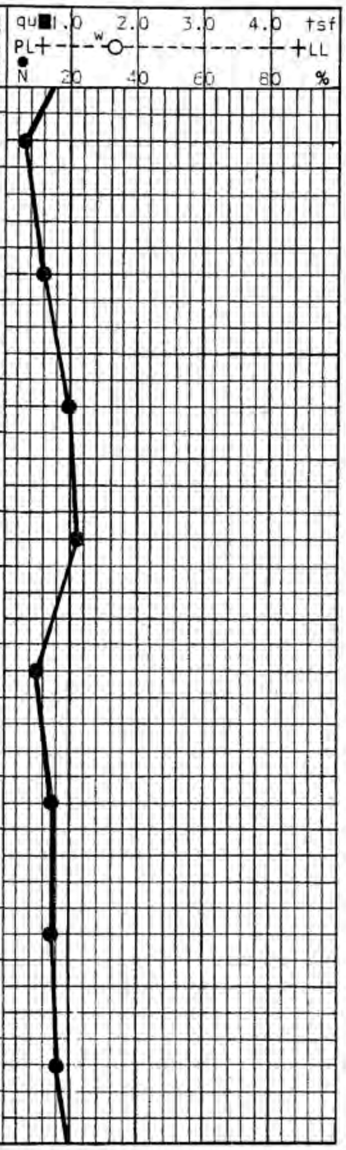
SVERDRUP & PARCEL

BORING LOG

BORING NO. B-8

Job No. 8089 Project Jamestown Bridge Replacement

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL	DEPTH (FT.)	SAMPLE
		SAND, grey, fine, little silt, loose, wet.	25	4 5
-48		SAND, brown-grey, fine to medium, little silt, trace of shell fragments, medium dense, wet.	30 35	6 7
-58		SAND, dark-grey, fine, some silt, loose, wet.	40	8
-62		SAND, dark-grey, fine, little silt, medium dense, wet.	45 50 55 60	9 10 11



Sheet 2 of 4 A33259

BORING NO. B-8

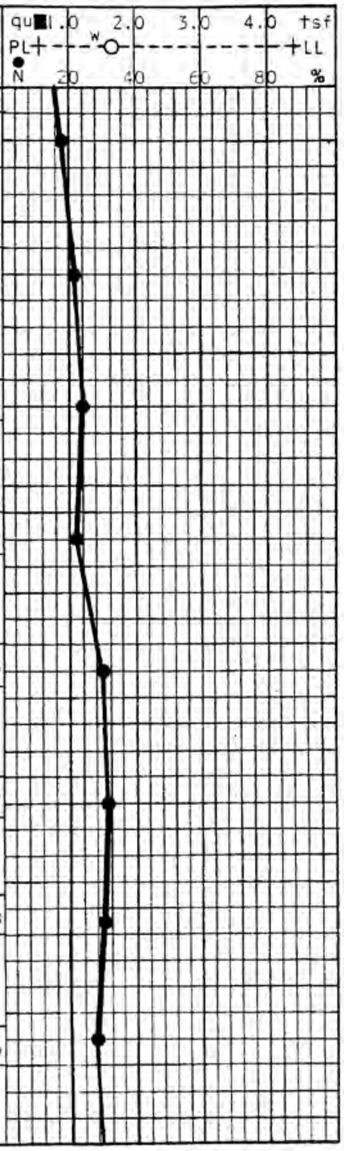
SVERDRUP & PARCEL

BORING LOG

BORING NO. B-8

Job No. 8089 Project Jamestown Bridge Replacement

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL	DEPTH (FT.)	SAMPLE
				12
		SAND, dark-grey, fine, little silt, medium dense, wet.	65	
				13
-88			70	
				14
			75	
				15
			80	
				16
		SAND, brown-grey, fine, some silt, medium dense to dense, wet.	85	
				17
			90	
				18
			95	
				19
			100	



Sheet 3 of 4

A33260

BORING NO. B-8

SVERDRUP & PARCEL
BORING LOG

BORING NO. B-8

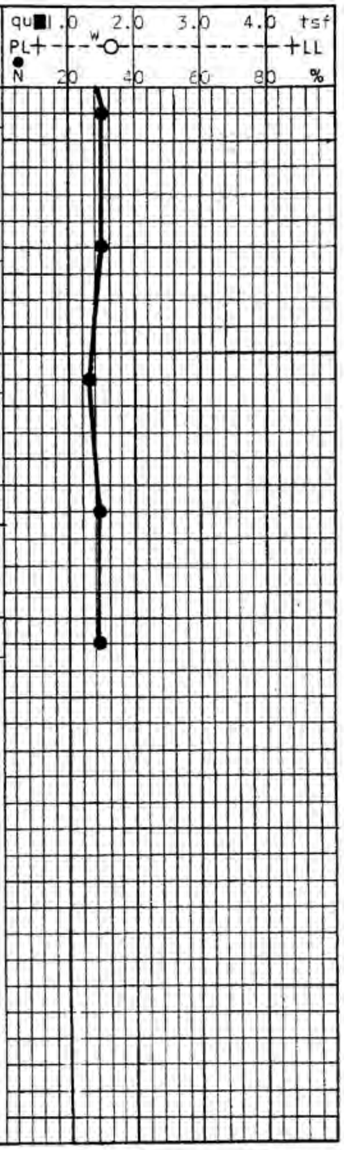
Job No. 8089 Project Jamestown Bridge Replacement

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL	DEPTH (FT.)	SAMPLE
				20
			105	21
		SAND, brown-grey, fine, some silt, medium dense to dense, wet.		
			110	22
			115	23
-136				
		SAND, blue-grey, fine, some silt, medium dense, wet.	120	24
-139.5				

End Boring

Notes:

1. Boring cased to EL-25 with SW casing.
2. Boring cased to EL-138 with HW casing.



Sheet 4 of 4

A33261

BORING NO. B-8

SVERDRUP & PARCEL

BORING LOG

BORING NO. B-11

Station	57+50
Offset	50' North ℄
Coordinates	
Ground Elev.	-15 (MSL)

Job No. 8089 Project Jamestown Bridge Replacement

Drilling Co. Warren George Inc.

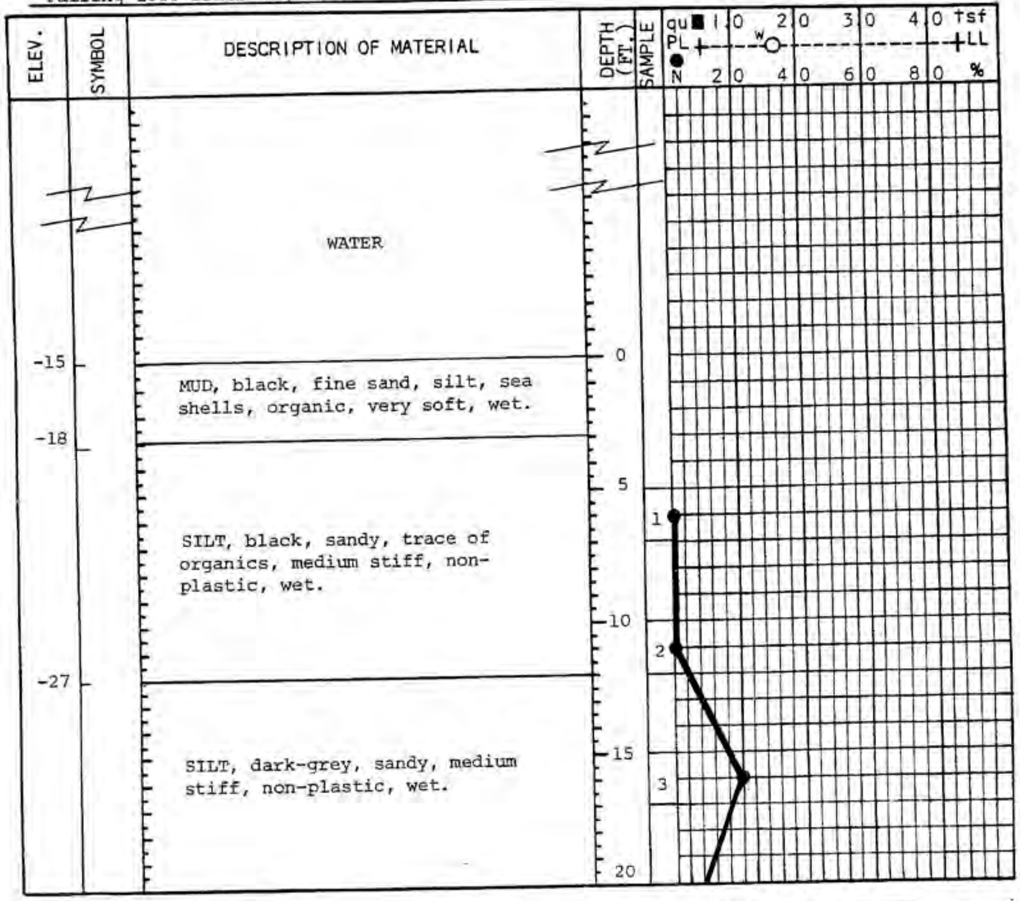
Driller J. Stevens
B. Muh Inspector M. Sonbolian
E. Rundhovde

Date Started 9/2/82 Date Completed 9/3/82

Drilling Equipment Data Barge Mounted

Failing 1500 Drill Rig

Ground Water Table	
@ Complete	( ) Hrs. Later



ELEV.	DESCRIPTION OF MATERIAL
	WATER
-15	MUD, black, fine sand, silt, sea shells, organic, very soft, wet.
-18	SILT, black, sandy, trace of organics, medium stiff, non-plastic, wet.
-27	SILT, dark-grey, sandy, medium stiff, non-plastic, wet.

Sheet 1 of 4

A33271

BORING NO. B-11

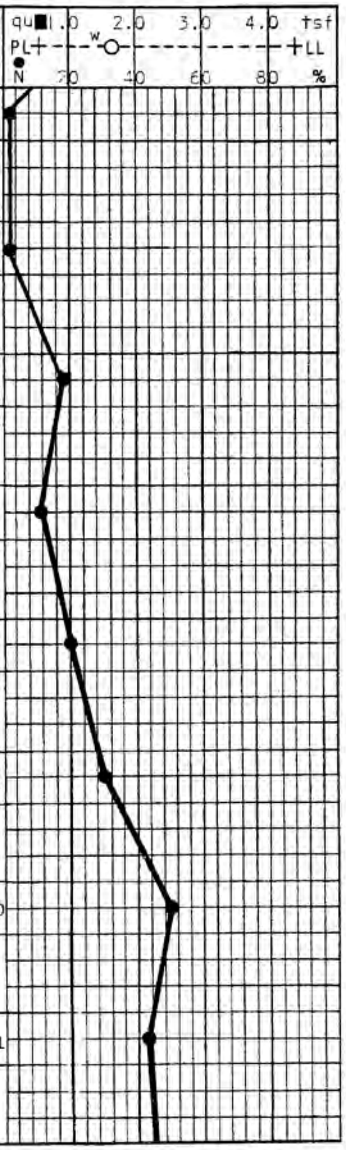
SVERDRUP & PARCEL

BORING LOG

BORING NO. B-11

Job No. 8089 Project Jamestown Bridge Replacement

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL	DEPTH (FT.)	SAMPLE
				4
			25	
				5
		SILT, dark-grey, sandy, trace of clay, trace of shells, medium stiff, non-plastic, wet.		
			30	
				6
			35	
				7
-54				
			40	
				8
			45	
				9
		SAND, dark-grey, fine to medium, trace of silt, medium dense to dense, wet.		
			50	
				10
			55	
				11
		(continue next page)	60	



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A33272

BORING NO. B-11

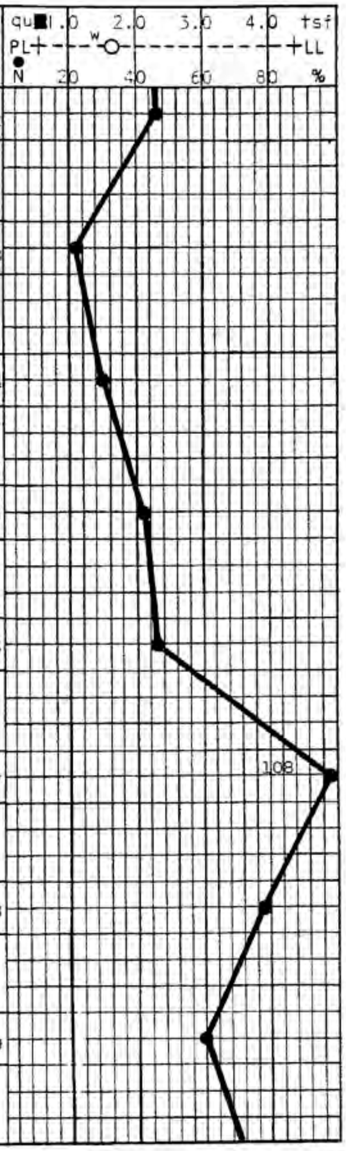
SVERDRUP & PARCEL

BORING LOG

BORING NO. B-11

Job No. 8089 Project Jamestown Bridge Replacement

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL	DEPTH (FT.)	SAMPLE
				12
-79		SILT, dark-grey, sandy, medium stiff, non-plastic, wet.	65 70	13 14
-88		SAND, dark-grey, fine, little silt, dense to very dense, wet.	75 80 85 90 95 100	15 16 17 18 19



Sheet 3 of 4

A33273

BORING NO. B-11

SVERDRUP & PARCEL

BORING LOG

BORING NO. B-11

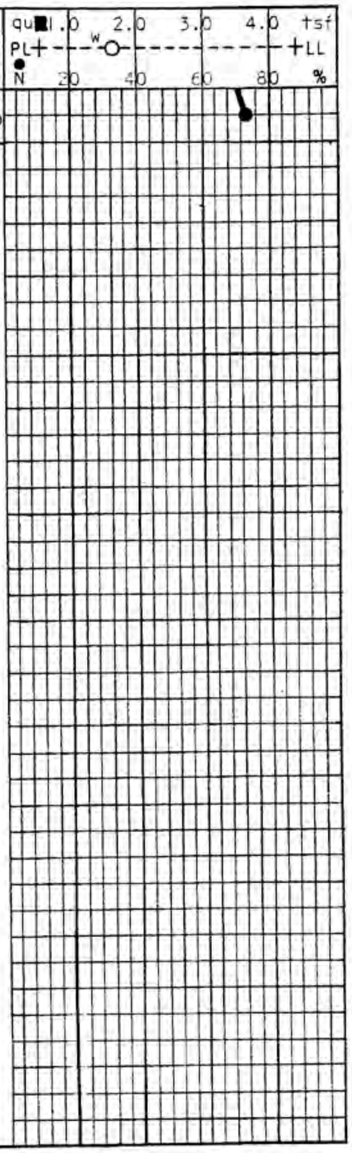
Job No. 8089 Project Jamestown Bridge Replacement

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL	DEPTH (FT.)	SAMPLE
				20
-117		End Boring	105	

End Boring

Notes:

1. Boring cased to EL-25 with PW casing.
2. Boring cased to EL-80 with HW casing.
3. Used Drill Mud from EL-80 to bottom of boring.



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A33274

BORING NO. B-11

SVERDRUP & PARCEL
BORING LOG

BORING NO. B-15

Job No. 8089 Project Jamestown Bridge Replacement

Drilling Co. Warren George, Inc.

Driller B. Muh
J. Stevens Inspector J. Franciose
M. Sonbolian

Date Started 8/30/82 Date Completed 9/1/82

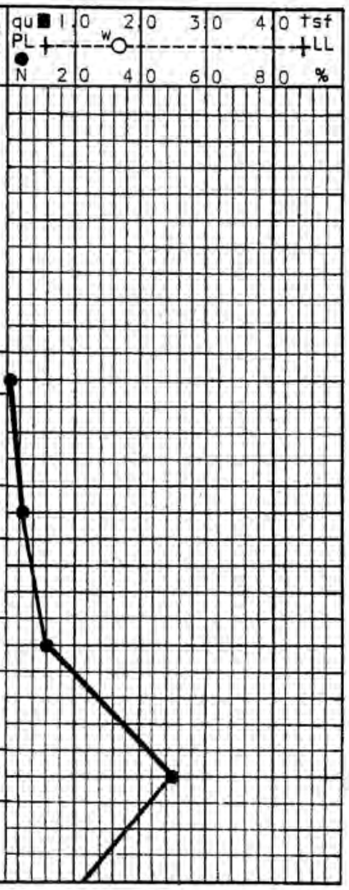
Drilling Equipment Data Barge Mounted

Failing 1500 Drill Rig

Station	69+10
Offset	50'North ℄
Coordinates	
Ground Elev.	-25(MSL)

Ground Water Table	
@ Complete	( ) Hrs. Later

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL	DEPTH (FT.)	SAMPLE
		WATER		
-25		MUD, black, sand, silt, sea shells, organic matter, very soft, wet.	0	1
-29		SAND, black, fine to coarse, some silt, trace of sea shells, loose to medium dense, wet.	5	2
			10	3
-39		SAND, dark-grey, fine, some silt, trace of sea shells, medium dense dense, wet.	15	4
		(Continue next page)	20	



Sheet 1 of 6

A33289

BORING NO. B-15

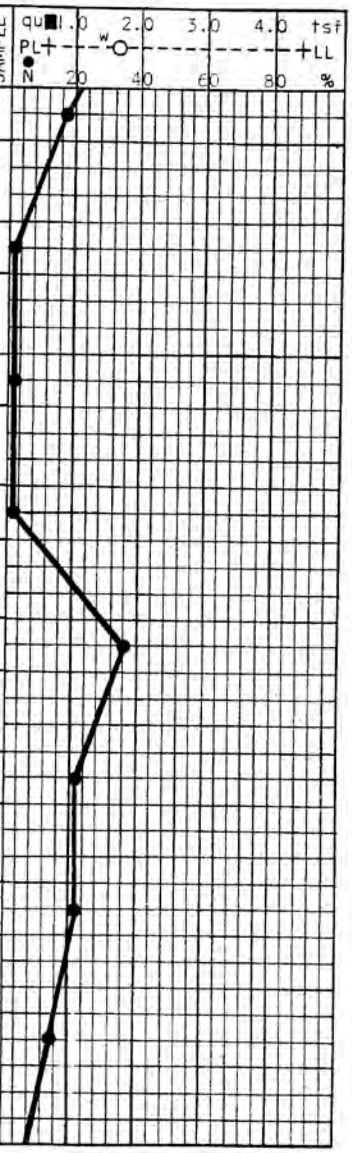
SVERDRUP & PARCEL

BORING LOG

BORING NO. B-15

Job No. 8089 Project Jamestown Bridge Replacement

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL	DEPTH (FT.)	SAMPLE
				5
-49		SAND, grey-brown, fine to medium, some silt, very loose, wet.	25	6
			30	7
			35	8
-63		SILT, dark-grey, trace of fine sand, medium dense to dense, non-plastic, wet.	40	9
			45	10
			50	11
			55	12
-85			60	



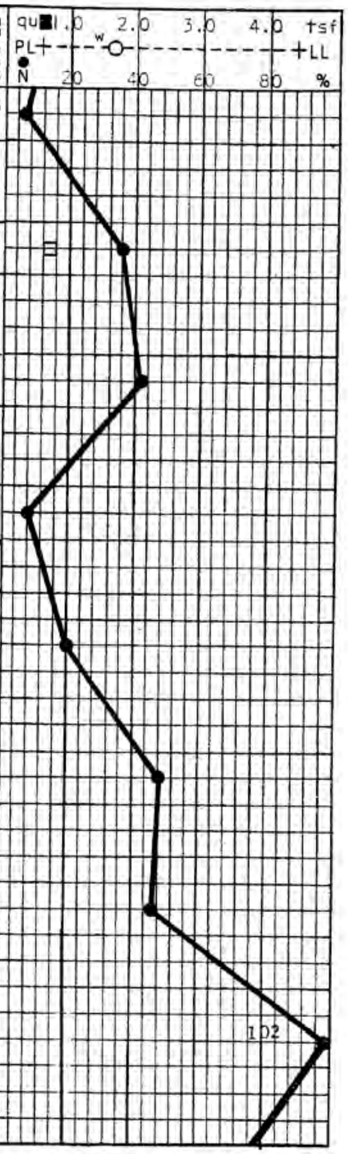
SVERDRUP & PARCEL

BORING LOG

BORING NO. B-15

Job No. 8089 Project Jamestown Bridge Replacement

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL	DEPTH (FT.)	SAMPLE
		SILT, dark-brown, trace of clay, trace of fine sand, soft, wet.		13
-90		SILT, dark-brown, little clay, trace of fine sand, medium stiff to stiff, wet.	65 70 75 80	14 15 16 17
-109		SILT, dark-grey, trace of clay, trace of fine sand, hard, moist, non-plastic.	85 90	18 19
-119		SAND, dark-grey, some silt, very dense, wet. (Continue next page)	95 100	20



Sheet 3 of 6

A33291

BORING NO. B-15

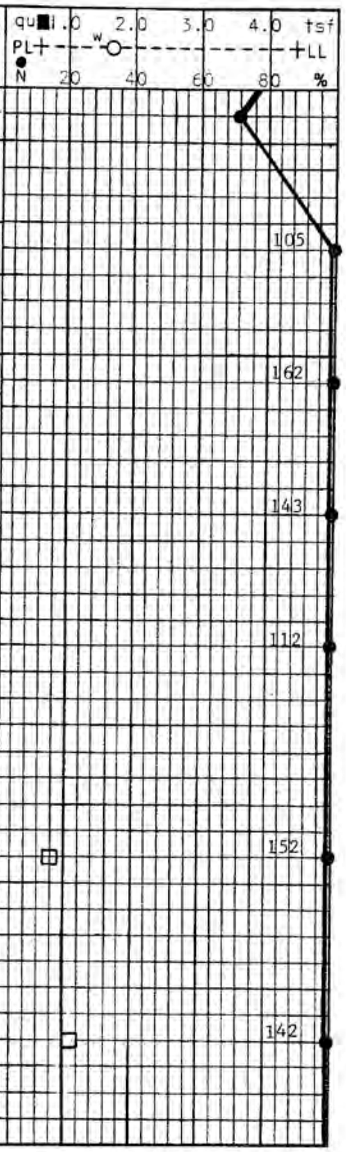
SVERDRUP & PARCEL

BORING LOG

BORING NO. B-15

Job No. 8089 Project Jamestown Bridge Replacement

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL	DEPTH (FT.)	SAMPLE
				21
			105	22
			110	23
			115	24
			120	25
-150		Boulder	125	
-151.5		Sand, dark-grey, fine, little silt, little gravel, trace of clay, very dense, wet (Till).		26
			130	
			135	27
		(Continue next page)	140	



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A33292

BORING NO. B-15

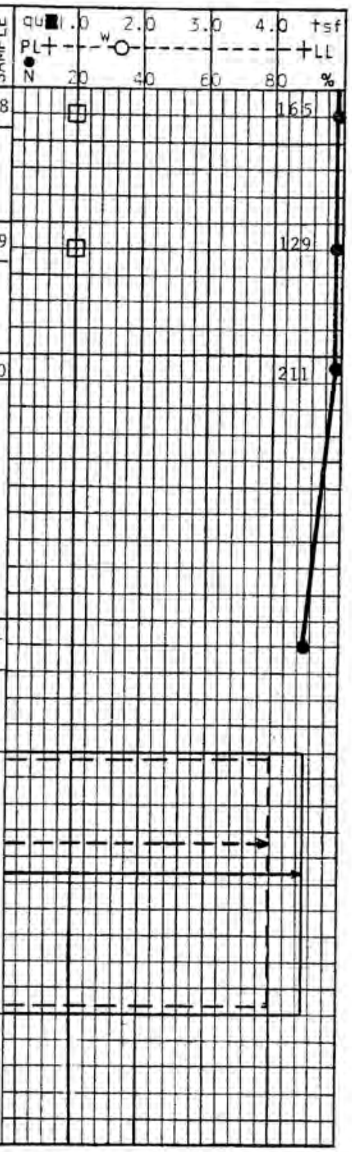
SVERDRUP & PARCEL

BORING LOG

BORING NO. B-15

Job No. 8089 Project Jamestown Bridge Replacement

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL	DEPTH (FT.)	SAMPLE
				28
			145	29
			150	30
			155	
			160	31
-190		SHALE, dark-grey, fractured, weathered	165	
		SANDSTONE, dark-grey, few tolialion planes, shaley, medium hard.	170	1
-200		End boring.	175	
			180	



SVERDRUP & PARCEL

BORING LOG

BORING NO. B-15

Job No. 8089 Project Jamestown Bridge Replacement

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL	DEPTH (FT.)	SAMPLE	qu ■ 1.0 2.0 3.0 4.0 tsf PL + - - - w ○ - - - - - - - - - + LL ● N 20 40 60 80 %
		Notes: 1. Full water return during coring. 2. Used NXD3 core barrel core diameter 2-1/2". 3. Boring cased to EL-40 with PW casing. 4. Boring cased to EL-90 with HW casing. 5. Boring cased to EL-190 with NW casing.			

Sheet 6 of 6

A33294

BORING NO. B-15

APPENDIX B: 1984 BORING LOGS

SVERDRUP & PARCEL

BORING LOG

BORING NO. B-54

Job No. 8089 Project Jamestown Bridge Replacement

Drilling Co. Guild Drilling Company

Driller A. Mason
K. Allen Inspector K. Bettger
R. Patterson

Date Started 9/20/84 Date Completed 9/21/84

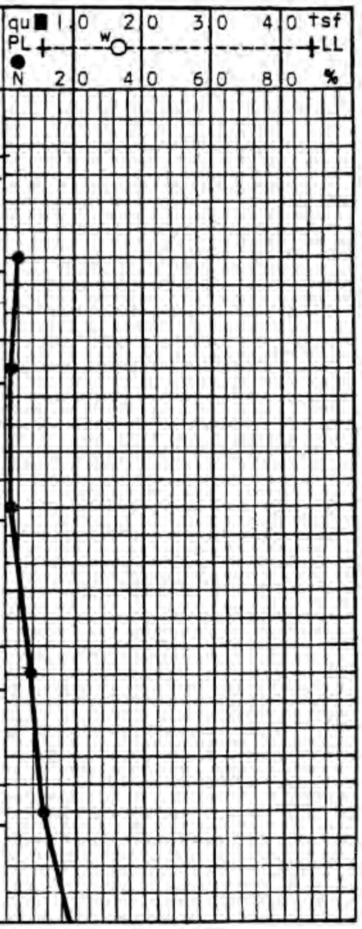
Drilling Equipment Data Barge Mounted

Acker Hillbilly Drill Rig

Station	37+00
Offset	0
Coordinates	
Ground Elev.	-8.5 (MSL)

Ground Water Table	
@ Complete	( ) Hrs. Later

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL	DEPTH (FT.)	SAMPLE
		WATER		
-8.5		SAND, dark grey, fine to coarse, some silt, trace of fine gravel, loose, wet.	0	1
-11.5		SAND, grey, fine, and silt, trace of roots, trace of shells, very loose, wet.	5	2
-17.5		SILT, grey, trace of fine sand, trace of clay, trace of shells, organic odor, non plastic, soft, set.	10 15	3 4
-27.5		SAND, grey brown, fine, little silt, medium dense, wet.	20	5
-30.5		See page 2 of 3.	25	



Sheet 1 of 3

BORING NO. B-54

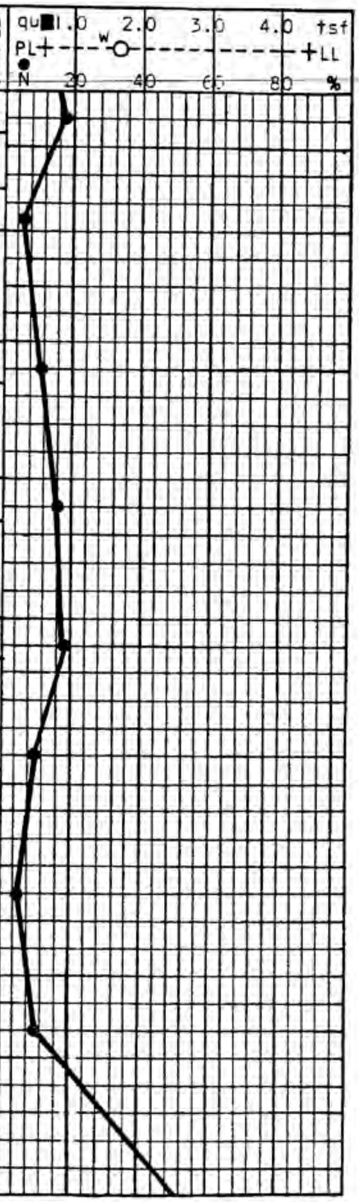
SVERDRUP & PARCEL

BORING LOG

BORING NO. B-54

Job No. 8089 Project Jamestown Bridge Replacement

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL	DEPTH (FT.)	SAMPLE
		SAND, brown, fine to coarse, little silt, trace of fine gravel, medium dense, wet.		6
-37.5		SAND, grey brown, fine to medium, trace of fine gravel, trace of silt, loose to medium dense, wet.	30 35	7 8
-46.5		SAND, grey brown, fine, little silt, medium dense, wet.	40	9
-52.5		SAND, grey, fine, some silt, trace of fine gravel, trace of clay, medium dense, wet.	45	10
-56.5		SILT, grey, and fine sand, medium stiff, non plastic, wet.	50 55	11 12
-66.5		SILT, grey, trace of fine sand, stiff to very stiff, non plastic, wet.	60	13
-73.5			65	



Sheet 2 of 3

BORING NO. B-54

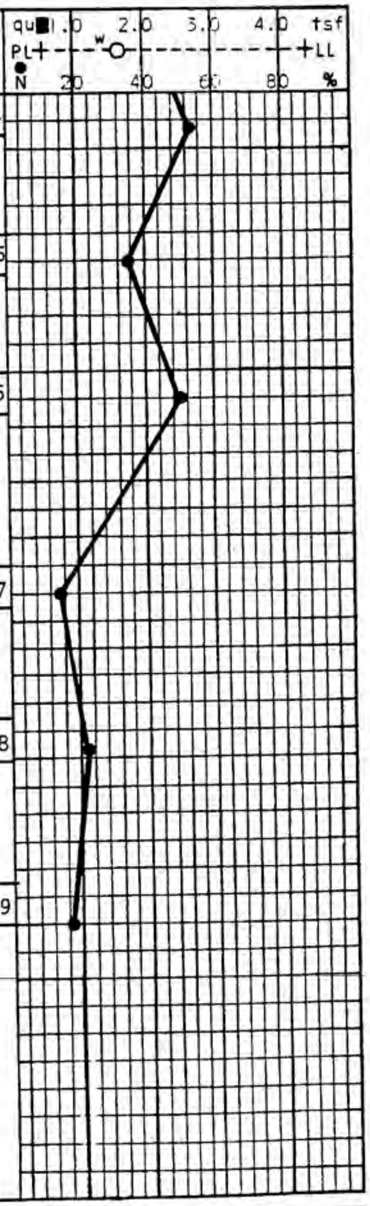
SVERDRUP & PARCEL

BORING LOG

BORING NO. B-54

Job No. 8089 Project Jamestown Bridge Replacement

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL	DEPTH (FT.)	SAMPLE
		SAND, grey, fine, some silt, dense to very dense, wet.		14
			70	15
-83.5		SAND, grey, fine to coarse, little silt, trace of fine gravel, dense, wet.	75	16
-88.5		SAND, grey, fine, little silt, trace of clay, medium dense, wet.	80	
				17
			85	
				18
			90	
				19
-103.5		END BORING	95	



NOTES:

1. Boring cased to El.-56.5 with HW casing.
2. Boring cased to El.-102 with NW casing.

SVERDRUP & PARCEL

BORING LOG

BORING NO. B-67

Job No 8089 Project Jamestown Bridge Replacement

Drilling Co. Guild Drilling Company

Driller D. Holley T. Paquette Inspector R. Patterson J. Franciose

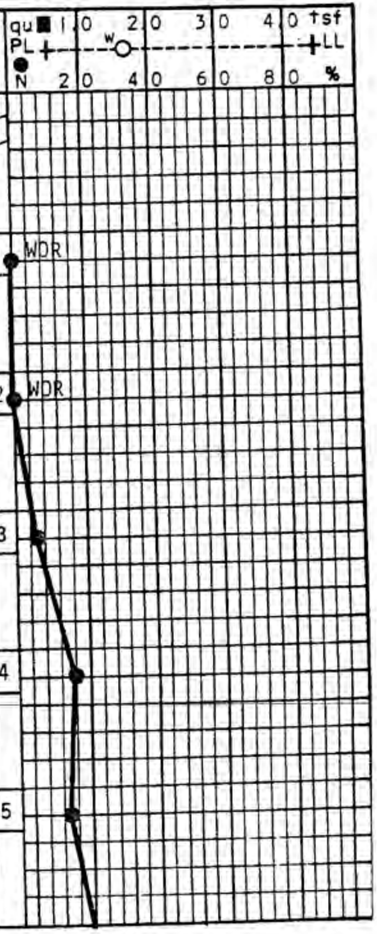
Date Started 10/8/84 Date Completed 10/9/84

Drilling Equipment Data Barge Mounted Mobil B-40 Drill Rig

Station	51+00
Offset	0
Coordinates	
Ground Elev.	-17 (MSL)

Ground Water Table	
@ Complete	( ) Hrs. Later

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL	DEPTH (FT.)	SAMPLE
		WATER		
-17		SAND, grey, fine to coarse, some silt, little shells, organic, very loose, wet.	0 5	1 2
-25		SILT, grey, some fine sand, non plastic, medium stiff, wet.	10	3
-30		SILT, grey brown, some fine sand, non plastic, stiff, wet.	15	4
-35		SAND, grey, fine, some silt, medium dense, wet.	20 25	5



BORING NO. B-67

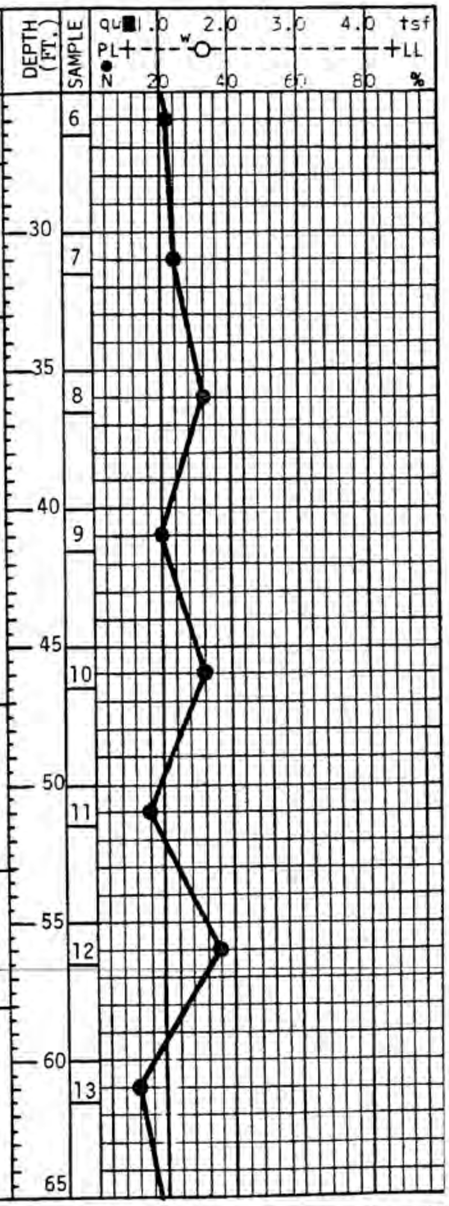
SVERDRUP & PARCEL

BORING LOG

BORING NO. B-67

Job No. 8089 Project Jamestown Bridge Replacement

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL
		See page 1 of 3.
-44.5		SAND, grey brown, fine, and silt, medium dense, wet.
-50		SAND, grey brown, fine to medium, trace of silt, trace of fine gravel, trace of shells, medium dense to dense, wet.
-64		SAND, grey, fine to medium, trace of silt, medium dense, wet.
-70		SAND, grey, fine, trace of silt, dense, wet.
-75		SAND, grey, fine, trace of silt, medium dense, wet.



BORING NO. B-67

SVERDRUP & PARCEL
BORING LOG

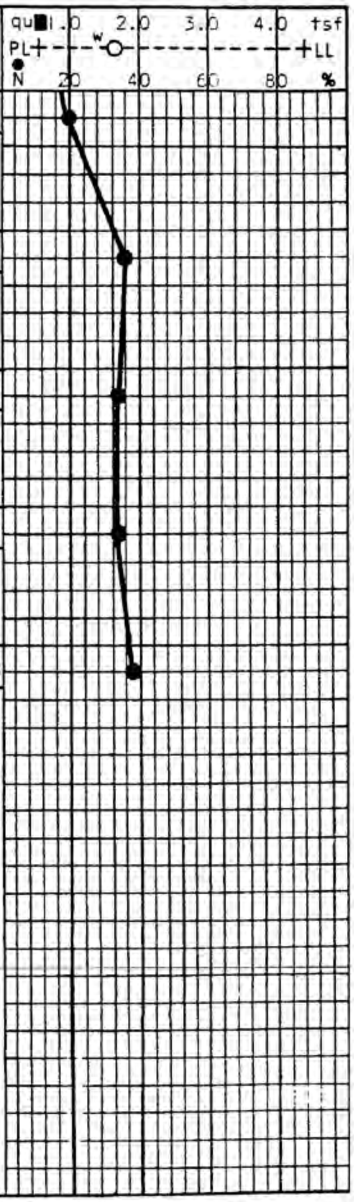
BORING NO. B-67

Job No. 8089 Project Jamestown Bridge Replacement

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL	DEPTH (FT.)	SAMPLE
		See page 2 of 3.		14
-84		SAND, grey, fine, and silt, dense, wet.	70	15
-89		SAND, grey, fine, little silt, dense, wet.	75 80 85	16 17 18
-103.5		END BORING		

NOTES:

1. Boring cased to El.-37 with SW casing.
2. Boring cased to El.-77 with HW casing.
3. Boring cased to El.-102 with NW casing.



SVERDRUP & PARCEL

BORING LOG

BORING NO. B-70

Job No. 8089 Project Jamestown Bridge Replacement

Drilling Co. Guild Drilling Company

Driller D. Holley
T. Paquette Inspector J. Farah
R. Patterson

Date Started 9/27/84 Date Completed 9/28/84

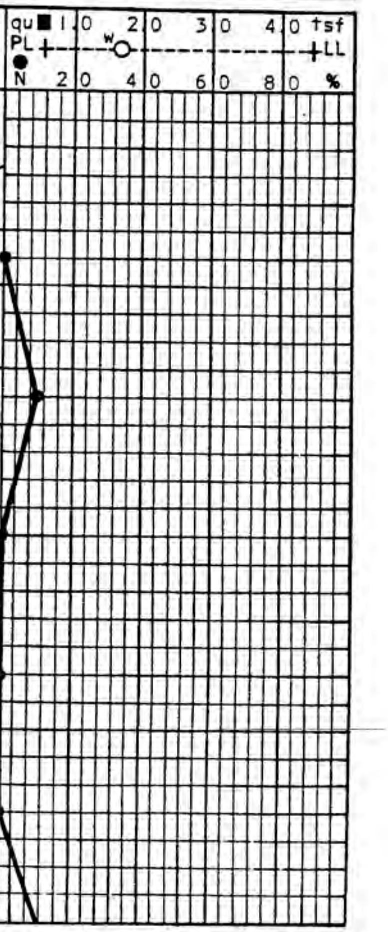
Drilling Equipment Data Barge Mounted

Mobil B-40 Drill Rig

Station	55+80
Offset	0
Coordinates	
Ground Elev.	-15 (MSL)

Ground Water Table	
@ Complete	( ) Hrs. Later

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL	DEPTH (FT.)	SAMPLE
		WATER		
-15		SILT, black, trace of sand, trace of shells, organic, non plastic, very soft, wet.	0	1
-19		SAND, grey, fine to medium, little silt, little shells, medium dense, wet.	5	2
-21.5		SILT, dark grey, some fine sand, some shells, organic, non plastic, very soft, wet.	10 15 20	3 4 5
-38		See page 2 of 3.	25	



Sheet 1 of 3

BORING NO. B-70

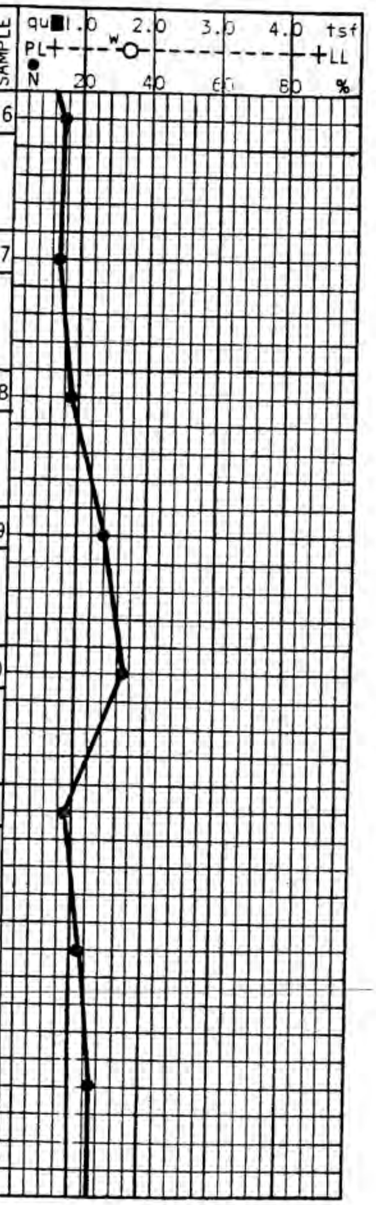
SVERDRUP & PARCEL

BORING LOG

BORING NO. B-70

Job No. 8089 Project Jamestown Bridge Replacement

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL	DEPTH (FT.)	SAMPLE
		SAND, grey, fine, trace of silt, medium, dense, wet.		6
			30	7
			35	8
			40	9
			45	10
-63		SAND, grey, fine to medium, trace silt, medium dense, wet.	50	11
			55	12
			60	13
-78		See page 3 of 3.	65	



Sheet 2 of 3

BORING NO. B-70

SVERDRUP & PARCEL

BORING LOG

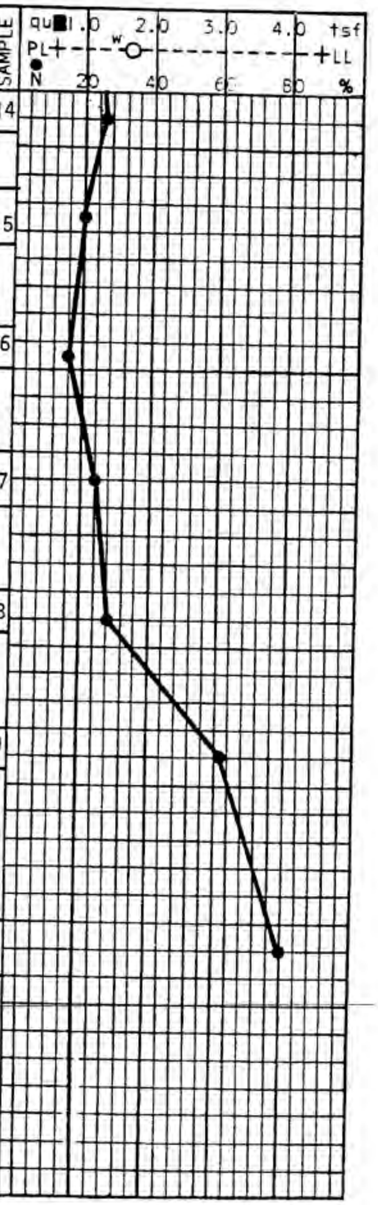
BORING NO. B-70

Job No. 8089 Project Jamestown Bridge Replacement

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL	DEPTH (FT.)	SAMPLE
		SAND, grey, fine, little silt, medium dense, wet.	70	14 15
-87		SAND, grey, very fine, some silt, medium dense, wet.	75 80	16 17 18
-100		SAND, grey, very fine, some silt, dense to very dense, wet.	85 90 95	19 20
-112		END BORING		

NOTES:

1. Boring cased to El.-38 with SW casing.
2. Boring cased to El.-110 with HW casing.



Sheet 3 of 3

BORING NO. B-70

SVERDRUP & PARCEL
BORING LOG

BORING NO. B-76

Job No. 8089 Project Jamestown Bridge Replacement

Drilling Co. Guild Drilling Company

Driller P. Brecia
T. Parquette Inspector R. Patterson, J. Franciose & J.Farah

Date Started 9/23/84 Date Completed

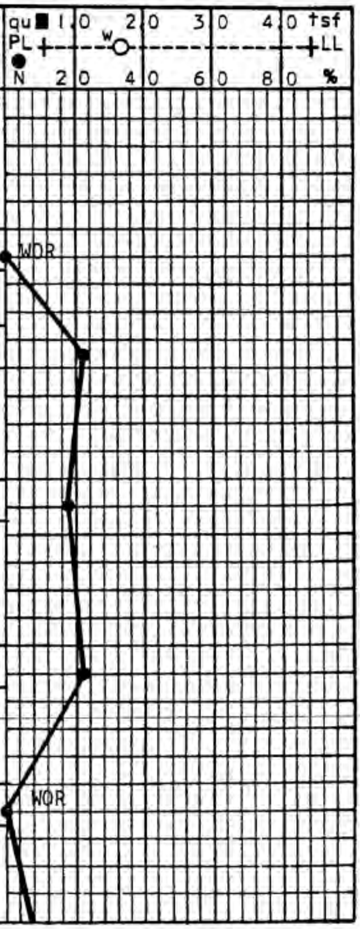
Drilling Equipment Data Barge Mounted

Mobil B-40 Drill Rig

Station	70+30
Offset	
Coordinates	
Ground Elev.	-21.5(MSL)

Ground Water Table	
@ Complete	( ) Hrs. Later

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL	DEPTH (FT.)	SAMPLE
		WATER		
-21.5		SILT, dark grey, little shells, little fine sand, organic, non plastic, very soft, wet.	0	1
-24.5		SAND, grey, medium to fine, some shells, trace of silt, medium dense, wet.	5	2
			10	3
			15	4
-39.5		SILT, grey, little fine sand, non plastic, very soft, wet.	20	5
-45.5		See page 2 of 3.	25	



Sheet 1 of 3

BORING NO. B-76

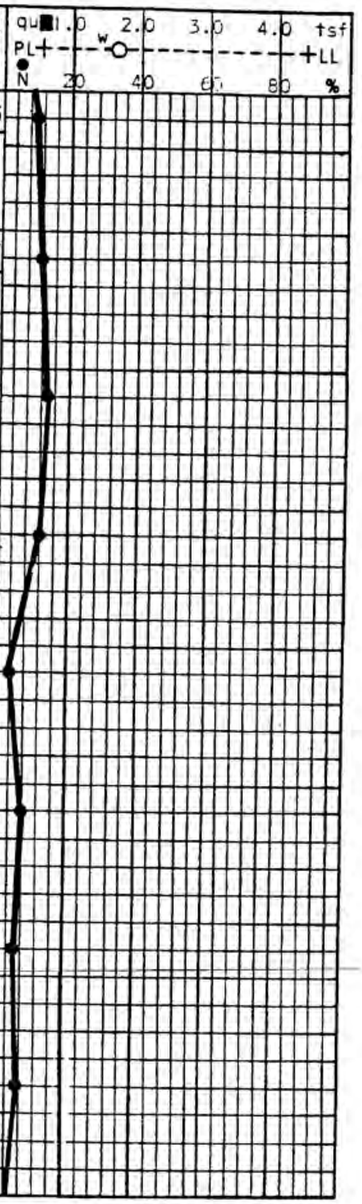
SVERDRUP & PARCEL

BORING LOG

BORING NO. B-76

Job No. 8089 Project Jamestown Bridge Replacement

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL	DEPTH (FT.)	SAMPLE
		SAND, brown, fine, some silt, loose to medium dense, wet.	30	6 7
-54.5		SAND, grey, fine, some silt, medium dense, wet.	35	8
-61.5		SILT, grey, little fine sand, non plastic, medium stiff to stiff, wet.	40 45	9 10
-71.5		SILT, dark grey, trace of fine sand, non plastic, medium stiff, wet.	50 55 60 65	11 12 13



Sheet 2 of 3

BORING NO. B-76

SVERDRUP & PARCEL

BORING LOG

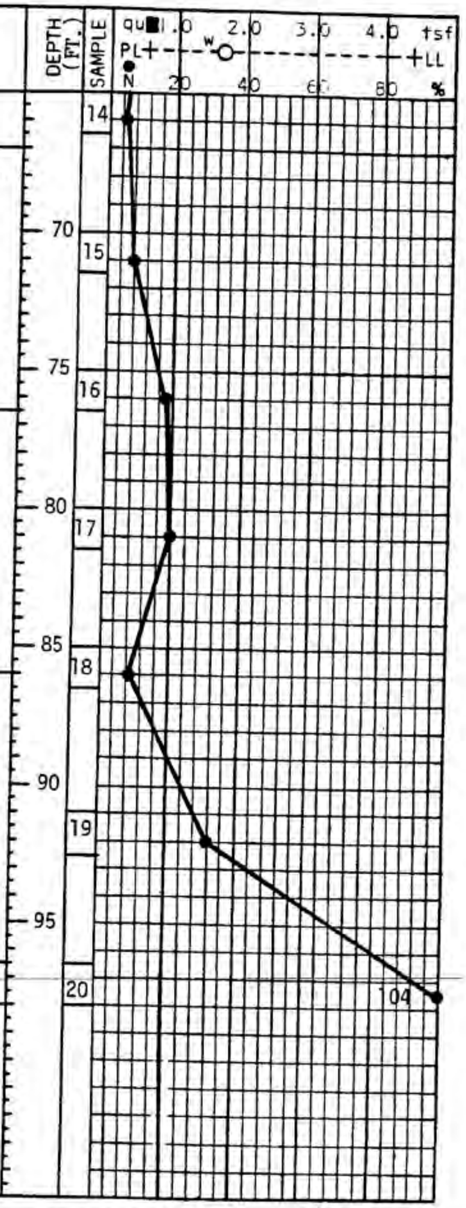
BORING NO. B-76

Job No. 8089 Project Jamestown Bridge Replacement

ELEV.	SYMBOL	DESCRIPTION OF MATERIAL
		See page 2 of 3.
-88.5		SAND, grey, fine, some silt, loose to medium dense, wet.
-98		SAND, grey, fine, trace of silt, medium dense, wet.
-107.5		SILT, grey, little fine sand, non plastic, very stiff to hard, wet.
-118		SAND, grey, fine, trace of silt, very dense, wet.
-119.5		END BORING

NOTES:

1. Boring cased to El.-17.5 with SW casing.
2. Boring cased to El.-74.5 with HW casing.
3. Boring cased to El.-118 with NW casing.



Sheet 3 of 3

BORING NO. B-76

APPENDIX C: SUMMARY OF THE JAMESTOWN VERRAZZANO BRIDGE PILE TEST PROGRAM

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R.I. Dept of Transportation
Office of Chief Legal Counsel

DRAFT

Draft - 23 Mar 88

SUMMARY OF PILE TEST PROGRAM

Prestressed Concrete Piles

Jamestown Bridge Replacement

March 1988

M.T. Davisson
D.M. Rempe

M.T. Davisson, Consulting Engineer

DRAFT

TABLE OF CONTENTS

DRAFT

SUMMARY OF PILE TEST PROGRAM

Prestressed Concrete Piles

Jamestown Bridge Replacement

1. INTRODUCTION

1.1. Summary.

This report summarizes the progress through February 1987 of the pile test program for the West Approach and Trestle sections of the Jamestown Bridge Replacement, and includes all testing of entirely prestressed-concrete piles at the site. Subsequent testing of composite prestressed-concrete and steel H-section piles at the site is not described herein.

The replacement bridge, to be known as the Jamestown-Verrazzano Bridge, crosses the West Passage of Narragansett Bay, Rhode Island, and connects North Kingstown on the west with Jamestown on the east. It is currently being constructed under contract with the Rhode Island Department of Transportation (RIDOT). The project has substantial Federal Aid.

Sverdrup & Parcel Associates, Inc. was selected in 1981 to be managing consultant for the project. Gordon R. Archibald, Inc., Pawtucket, Rhode Island, was designated as designers for the Trestle; T.Y. Lin International, San Francisco, was designated as designer for the West Approach, Main Span, and East Approach sections. A construction contract was awarded in 1985 to Clark-Fitzpatrick, Inc., and Franki Foundation Co., A Joint Venture, designated herein as the Contractor. In 1986, the firm DMJM, Los Angeles, was appointed construction manager. The Contractor has engaged Goldberg-Zoino and Associates (GZA), Newton Upper Falls, Massachusetts, as geotechnical consultant. During the design phase of the project, T.Y. Lin International engaged Lee and Praszker, San Francisco, as geotechnical consultant, and Gordon R. Archibald, Inc. engaged Kent A. Healy, Sc.D., Vineyard Haven, Massachusetts, as geotechnical consultant. In the Summer of 1986, Gordon R. Archibald, Inc. also engaged M.T. Davisson, Consulting Engineer, Savoy, Illinois, as piling consultant to Gordon R. Archibald, Inc. and to the Rhode Island Department of Transportation.

The replacement bridge will be approximately 7400 ft in length, including a 2400 ft long Trestle at the west end, 2000 ft West Approach, 1400 ft Main Spans, and 1600 ft East Approach (Figure 1). Foundations for the replacement bridge consist of piers founded directly on bedrock for bridge sections east of the main channel, and pile-supported piers for the bridge sections west of the main channel, including the west side of the Main Span as well as the Trestle and West Approach sections.

The two west-side Main Span piers, numbers 12 and 13, will be supported by steel H-piles driven to bedrock. Pile driving and testing has indicated that the H-piles can successfully be driven to bedrock, with ultimate bearing capacities in excess of the specified 400 tons.

The Trestle and West Support sections were designed to be supported by prestressed-concrete piles bearing in the alluvial soils, well above bedrock. However, results of initial pile testing indicated that piles driven to the depths listed on the contract drawings do not have adequate bearing capacity. Hence, decisions were made to lengthen the test piles, and to drive and test at deeper tip elevations. Subsequent tests indicated that it was necessary to penetrate to or near the dense till soils over bedrock or to bedrock, in order consistently to develop adequate capacity. In consideration of the time and equipment available for testing, and proximity of the tips of successful test piles to till or rock, the decision was made to test piles designed to be driven to or near till or bedrock.

Due to the length of piles extending to bedrock, and for the reasons discussed later herein, the decision was made to test a composite pile configuration consisting of a prestressed-concrete top section similar to the originally specified pile, and steel H-pile bottom section.

As this report is written, the testing of composite piles is complete, with two successful load tests indicating ultimate capacities in excess of 500 tons. The composite pile testing will be described in a separate report.

1.2. Object of Report.

This report will summarize the results of the testing of entirely prestressed-concrete piles at the Jamestown site, as well as the successful H-Pile test performed at Pier 12 (at west side of Main Span). Included herein are discussions of the soil conditions at the site, the original pile design, pile driving hammers, pile driving instrumentation, static pile load testing, and events prior to load testing; this is followed by a brief chronology of the test program including test results, decisions, and other related information. The H-pile testing at Pier 12 is described as it relates to the selection of the composite pile for subsequent testing. The proposed use of composite prestressed concrete piles with steel H-pile bottom extension sections is discussed and, finally, conclusions drawn on the basis of the data accumulated during the testing of the prestressed piles and H-piles are presented.

This report was requested by Mr. Graham Ray, RIDOT Resident Engineer for the project, in a letter dated 9 February 1987. The report is based on information supplied by DMJM and RIDOT, including geotechnical reports, drive records, pile instrumentation data, static load test reports, as well as selected correspondence and minutes of meetings.

1.3. Soil Conditions.

Soil conditions at the site, as indicated by soil borings, seismic surveys, etc., have been described in reports by Sverdrup & Parcel Associates (1982,1984), and Lee and Praszker (1983), and will be summarized briefly herein. In general, the reports indicate the following profile:

Designation	Description
IA	MUD - Very loose fine sand, silt, sea shells, and organic matter.
IIA	Very loose to loose SAND and SILT (SP,SM,ML) - Predominantly silt and sand with traces of clay and occasional organic material.
IIB	Medium dense to dense SAND and SILT (SP,SM,ML) - Predominantly silt and sand with traces of clay and fine gravel.
IIC	Dense to very dense SAND and SILT (SP,SM,ML) - Predominantly silt and sand with traces of clay and gravel.
IIIA	TILL (ML) - A mixture of silt and sand with clay and gravel containing cobbles and boulders.
IVA	BEDROCK - Grey to dark grey fractured and seamy shale.
IVB	BEDROCK - Grey to dark grey metamorphosed sandstone.

The approximate location of these soil/rock layers is shown on Figure 1, which is a profile sheet after Lee and Praszker (1983). Depths to bedrock based on available boring information, a seismic survey, and pile driving logs indicate a range of depths from 60 to 240 ft, approximately, below sea level.

Additional soil/rock boring information is contained in boring logs, and a forthcoming report by the Maguire Group Inc., Providence, Rhode Island, presenting the results of a Supplementary Boring Program performed in the Fall of 1987.

1.4. Original Pile Design.

Plans for the Trestle and West Approach piles, as originally issued for construction, called for prestressed-concrete piles generally bearing in the soils designated as IIA, IIB, and IIC, at estimated tip elevations (for bid purposes) ranging from -50 to -95 ft, MSL. The Trestle piles were to be 24 inch square in cross-section with a design compression service load of 165 tons, and minimum ultimate bearing capacity of 330 tons. For the West Approach, 20 inch square piles with a design compression service load of 170 tons, and minimum ultimate bearing capacity of 340 tons, were specified. For brevity, the term "prestressed concrete" is abbreviated herein as "PSC".

A pile test program was included in the construction contract, in order to estimate the required lengths of the service piles and to determine the driving resistance necessary to develop the specified capacities (State of Rhode Island, Department of Transportation, 1984).

1.5. Pile Driving Hammers.

Initial driving was performed with a Delmag open-top, variable-stroke diesel hammer, model D36-23, which has a 7900 lb ram and a rated energy (ram weight x stroke) of 88,600 ft-lbs. In September, 1986, a D46-23 hammer, which is identical in cylinder dimensions to the D36-23, but has a ram weighing 10,100 lbs and a rated energy of 107,100 ft-lbs, was mobilized by the Contractor. This hammer was used in all subsequent impact pile driving for the Trestle and West Approach test piles.

The fall-distance, or stroke, of the ram is a prime indicator of hammer performance. With the Delmag hammers, some measure of ram-stroke control is available through use of the fuel control. Four settings are available, ranging from #1, minimum fuel, to #4, maximum fuel. The ram stroke achieved at each setting varies according to a number of factors, including cushioning, pile stiffness, penetration resistance, mechanical condition of the hammer, and numerous other factors influencing the diesel combustion characteristics on the day of driving. Although several references to actual ram-stroke observations are included on the pile logs, in many cases only indirect indicators of stroke are available, namely the fuel setting and, in some instances, the cycle-frequency of the hammer (blows per minute). Cycle-frequency data is usually included with the dynamic pile instrumentation records; however, correlation of the readings with pile penetration is sometimes difficult.

Internal cushioning for the Delmag hammers consisted of a single, one-inch thick, 23 inch diameter disk of micarta plastic, combined with two, 1/2 inch thick aluminum disks of the same diameter. Pile cushioning consisted of 9 pieces of 3/4 inch thick plywood, for a total height of 6.75 inches; the plywood pieces were cut in 20 inch or 24 inch square shapes to match the pile-top dimensions.

1.6. Wave Equation Analysis.

Wave equation analyses were required to be performed and submitted by the Contractor for review prior to driving, to evaluate the ability of the Contractor's proposed hammer/cushion combination to drive the piles to the specified ultimate static capacities without damage. Wave equation analyses were not used as a primary means of estimating pile capacity during test driving; rather, pile instrumentation was employed as a means for control of test driving, as described below.

1.7. Pile Instrumentation.

The Pile Driving Analyzer (PDA), in combination with laboratory analysis of PDA data (CAPWAP), was used during all test pile driving. The PDA is a device represented by the developers to measure strain and acceleration at the pile head, and to estimate static pile capacity, dynamic energy transfer, pile force, and other quantities, at the time of driving ("real-time"). Data is recorded on FM tape for possible later analysis by CAPWAP, as discussed below. The capacity predictions are based on an algorithm (computer model) which is simpler than, and inferior to, wave equation analysis, but which is used in the PDA because results can be produced in a the short time interval between hammer blows.

Due to the high potential for error resulting from inadequacy of the PDA algorithm and other factors, the PDA pile capacity prediction must be correlated to actual capacity as measured in a static load test by means of a correction factor, J, dialed into the PDA prior to driving. The J factor, often referred to as a damping factor, is a dimensionless correlation factor used to correct for soil damping losses plus other pile and soil conditions. The PDA capacity predictions are very sensitive to the J factor used in the prediction. A correlation between J and predicted capacity can be estimated, offering an opportunity to modify the initial capacity prediction by assuming a different value for J. Hence various reports of a given drive may include different PDA capacity predictions, on the basis of identical measurements, if they assume different values of J. Further confusion in the reporting of PDA capacity predictions can arise because predictions can vary significantly from one hammer-blow to the next, leading to the necessity either to select the most representative value, or to report a range of values.

On the Jamestown Bridge project the J values used for PDA capacity predictions varied considerably over the span of the testing, ranging from 0.2 to 0.9, approximately, reflecting adjustments made to force the PDA predictions to match widely varying CAPWAP and static load test results.

CAPWAP analysis is performed on PDA data recorded on tape and transmitted or transported to the computer laboratory. The PDA data is analyzed in detail with the object of obtaining a more accurate estimate of static pile capacity than is provided directly by the PDA. Whereas PDA results are obtained at the time of driving, CAPWAP results are developed some time after driving. CAPWAP incorporates a pile/soil computer model similar to that used in wave equation analysis; however, input to the analysis consists of the pile-top measurements of strain and acceleration produced by the PDA. Because the CAPWAP pile/soil model is more realistic than that used in the PDA capacity prediction, CAPWAP has the potential for more accurate prediction of pile capacity. Nevertheless, CAPWAP predictions also are subject to significant error and must, therefore, be correlated with static load test results.

PDA and CAPWAP results were used throughout the test program as a means for preliminary estimation of pile capacity at the time of driving or shortly thereafter, in order to determine when adequate pile capacity has been achieved and, hence, to determine the depth of penetration at which driving should be stopped and a static load test performed.

1.8. Static Load Testing.

Static load testing was incorporated into the test program as the definitive measure of pile capacity. Tests were performed using quick, maintained-load test procedures as described in the project specifications. Loads were applied using a hydraulic jack, with reaction provided by temporary tension piles connected to a structural frame. Instrumentation and recording was provided by GZA. The Davisson method of interpretation of the load test results was required by the project specifications and has been employed throughout this report, using as a failure criterion a pile-top deflection equal to elastic compression plus 0.35 inches (PSC piles), or 0.27 inches (H-pile), as specified.

1.9. Events Prior to Pile Test Program.

In May of 1981 the final Environmental Impact Statement for the replacement bridge was approved. In December of the same year the final selection of design firms was announced. Final plans and specifications for construction were completed and the project was advertised for bid on 28 September 1984. Bids were opened on 19 December 1984 and on 12 June 1985 a contract was signed for construction of the bridge. Following a notice to proceed on 9 July 1985, construction began in the Fall of 1985 and test pile driving was initiated in April 1986.

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2. CHRONOLOGIES

Chronologies of the testing activities, including test results and decisions relative to the testing, are presented below. For clarity, a separate chronology has been prepared for each of the test piles. For brevity, details such as blowcounts, ram stroke observations, etc. are not included in the chronologies; rather, such information is summarized in supplementary tables and figures. For complete details, the reader is referred to correspondence, minutes, field logs and other records in the project files, which are too bulky for inclusion in this report.

Because the chronologies overlap in time, project decisions made with respect to a given test pile should be viewed in the context of all test results available at that point in time, including data from other test piles.

A total of five piles were tested in the Trestle and West Approach areas of the project, as indicated in Figure 1. Three test piles were located at two locations in the West Approach area, one at station 56+80, between Piers 2 and 3, a second at station 69+00, near between piers 9 and 10, and a third at station 56+95 to replace the test pile at station 56+80 which was broken during driving. Two test piles were located in the Trestle area, one near Bent 9 (station 37+20) and the other near Bent 26 (station 51+40). The station of the test piles noted herein is approximate.

A chart indicating the duration of driving and testing activities for each test pile is included as Figure 2.

2.1. Test Pile at Station 56+80 (West Approach, Pier 2/3, "TP-1").

Test pile driving began on 9 April 1986 at station 56+80, with the Contractor using the Delmag D36-23 hammer to drive a 20 inch square PSC section, 123 ft long. A drive log summary, including the results of the nearest soil boring, is included as Figure 3A, with notes on Figure 3G; further details are summarized in Table 1A.

At this station the estimated pile tip elevation as shown on the project drawings is -90 ft. However, driving was stopped at elevation -71 ft because at this point the PDA indicated a capacity of 346 tons, which approximates the required 340 tons minimum ultimate capacity for West Approach piles. After shortening the pile to 79 ft for purposes of load testing, a restrike on 10 April 1986 resulted in an even higher PDA capacity prediction of 517 tons. The PDA capacity predictions were based on an assumed J factor of 0.2. However, CAPWAP analyses of the PDA measurements indicated lower capacities: 192 tons at the end of initial driving, and 196 tons at the restrike.

On 21 April 1986 a static load test was performed, resulting in a failure load, according to the method of interpretation specified in the contract documents, of 83 tons (Figure 4A), or approximately 16 percent of the PDA restrike prediction, and well short of the 340 tons minimum ultimate capacity. A rerun of the CAPWAP analysis of the pretest PDA restrike data, performed after the load test, produced a reduced capacity prediction of 135 tons, 61 tons less than the pretest analysis of the same data but 52 tons greater than the actual failure load.

In a jobsite meeting on 28 April 1986 the dynamic and static test results were reviewed, and the decision was made to lengthen the test pile by splicing, then to drive to the plan tip elevation (-90 ft) and restrike after 12 hours or more, and finally to decide on the basis of CAPWAP results whether to cut off and perform a second static load test.

A restrike was performed on 16 May 1986, prior to lengthening the pile. The PDA capacity prediction, now based on an increased J factor of 0.6, was 110-125 tons; CAPWAP indicated a capacity of 102 tons. A 67-ft long section of 20 inch square PSC pile was then spliced to the test pile and, on 20 May 1986, the pile was redriven approximately 6 additional feet to elevation -78 ft, at which point driving was stopped due to pile-head damage. Redriving continued on 26 June 1986 after the damaged portion of the pile head was removed. Pile-head damage again caused driving to be stopped, this time at a pile tip elevation of -86 ft. After an 8 ft long damaged section was removed from the pile top, driving continued on 27 June 1986 to a tip elevation of -97 ft, at which point the pile broke, apparently in the vicinity of the splice. PDA capacity at the time of the break was approximately 180 tons (J = 0.6), whereas CAPWAP predicted 140 tons.

This test pile was then abandoned and replaced with a new test pile at station 56+95, as discussed below.

2.2. Test Pile at Station 69+00 (West Approach, Pier 9/10, "TP-2").

Test pile driving at Station 69+00 began on 1 May 1986, using the Delmag D36-23 hammer. A drive log summary and details are included in Figure 3B and Table 1B. The 128 ft long 20 inch PSC pile penetrated at a low blow count (final 12 blows per foot) to a tip elevation of -109 ft, or 14 ft below the estimated tip elevation, -95 ft, shown on the plans. Driving was stopped at this point, in position to be spliced, because the PDA sensors were approaching the template and would have been damaged by further driving. The PDA capacity prediction was 0 to 35 tons (J = 0.6); CAPWAP predicted 45 tons.

At a site meeting on 7 May 1986 the decision was made to restrike the test pile, lengthen by splicing, redrive, and load test. The restrike was performed on 13 May 1986, with some apparent increase in driving resistance. The PDA capacity prediction was 79 tons (J = 0.6); the CAPWAP capacity prediction was 70 tons. Subsequently, a section of 20 inch PSC pile was spliced to the top of the previously driven section, bringing the total length to 167 ft. On 16 May 1986, the lengthened pile was redriven to elevation -128 ft, where driving was stopped due to spalling of the pile top. The final PDA and CAPWAP capacity predictions were 278 tons (J = 0.5) and 169 tons, respectively. On 20 May 1986 the pile was redriven to elevation -140 ft, at which point the driving resistance exceeded 300 blows per ft, the PDA capacity prediction was only 160-180 tons (J = 0.6), and the CAPWAP capacity prediction was 205 tons.

The Contractor then was directed to restrike and load test the pile. A restrike on 30 June 1986 resulted in a PDA capacity of 260-280 tons (J = 0.6), and CAPWAP capacities of 211 and 204 tons for blows early and late in the restrike, respectively.

A static load test performed on 24 July 1986 resulted in a measured ultimate capacity of 240 tons, or 100 tons below the required minimum capacity (Figure 4B). At elevation -141 ft, the pile was approximately 20 ft into what is designated as layer IIC (dense to very dense sand and silt) and with Standard Penetration Test N-values ranging from 72 to more than 100 (Boring B15).

After review of the data developed in the testing at stations 69+00 and 56+80 (see above), RIDOT in correspondence dated 1 August 1986 noted that the Contractor's driving system might not be capable of developing the required pile capacity, and suggested a meeting to discuss their concern.

In a submittal dated 21 August 1986, the Contractor provided the results of wave equation analyses indicating the Delmag D46 hammer to be more efficient than the D36, and requested approval of the D46 hammer for driving test piles. On 25 August 1986 the Contractor was directed to restrike the test pile at station 69+00 using the larger hammer, as proposed by the Contractor.

The restrike was carried out on 24 September 1986, using a Delmag D46-23 hammer and resulting in a total 11 inches additional penetration. The PDA capacity prediction was 235 tons (J = 0.6) early in the restrike, with a corresponding CAPWAP capacity prediction of 196 tons. Late in the restrike, the PDA capacity prediction increased to 284 tons (J = 0.6), whereas the CAPWAP capacity prediction dropped to 149 tons.

No further testing was performed on this pile.

2.3. Test Pile at Station 56+95 (W. Approach, Replacement for "TP-1" at Station 56+80).

The replacement for the broken West Approach test pile TP-1, station 56+80, was driven at station 56+95 beginning on 7 October 1986, using the Delmag D46-23 hammer. A drive log summary and details are included in Figure 3C and Table 1C.

The 20 inch replacement test pile was initially 135 ft long. On 8 October 1986, the pile was driven to elevation -106 ft, at which depth the PDA indicated a capacity of 100 tons (J = 0.5), then to elevation -109 ft, where the PDA reading was 160 tons (J = 0.5); driving stopped at this time due to a damaged PDA sensor. Driving continued on 9 October 1986 to elevation -127 ft, where the PDA indicated 95 tons capacity (J = 0.5), and CAPWAP indicated 81 tons. Driving was stopped due to the pile length limitation and, also, because the PDA sensors were damaged.

In a jobsite meeting on 9 October 1986 the Contractor was directed to extend the test pile using a steel section. The extension was constructed using a W14x283 section, 55 ft long, and the extended pile was redriven on 23 December 1986. Driving continued to elevation -182 ft, where the PDA indicated a capacity of 340 tons (J = 0.5); however, the subsequent CAPWAP analysis indicated only 226 tons capacity. A restrike on 24 December 1986 resulted in a PDA value of 324 tons (J = 0.5), and a CAPWAP prediction of 264 tons.

A static load test performed 15 January 1987 indicated an ultimate capacity of 420 tons, or 80 tons more than the 340 tons ultimate capacity required for West Approach piling (Figure 4C).

Soil boring D136/D136a, performed at station 57+28 as a part of the Supplementary Boring Program in the Fall of 1987, encountered bedrock at elevation -179 ft, with the bedrock described as hard but highly fractured quartzite/schist. The test pile, at a final tip elevation of-182 ft, probably rests on till immediately over bedrock, or on bedrock.

No further testing was performed on this pile.

2.4. Test Pile at Station 37+20 (Trestle, Bent 9, "TP-2").

Initial driving of a 24 inch PSC pile, 110 ft long, began at Trestle station 37+20 on 16 October 1986, using the Delmag D46-23 hammer. A drive log summary and details are included in Figure 3D and Table 1D. Driving continued through 20 October 1986, with the pile at a final tip elevation of -100 ft, which is 33 ft below the estimated tip elevation indicated on the project drawings. Driving ceased at this point due to the pile length limitation. The final PDA capacity prediction was reported as 150-200 tons on the pile driving log and 200-230 tons in the GZA notes (J = 0.5); the CAPWAP capacity prediction was 200 tons.

Subsequent to the initial driving, the Contractor was directed to restrike, lengthen by splicing, redrive and load test. The restrike took place on 11 December 1986, with resulting PDA and CAPWAP capacity predictions of 220 tons (J = 0.5) and 205 tons, respectively.

Following the restrike, the pile was lengthened by splicing a 30 ft section of 24 inch PSC pile, bringing the total length to 140 ft. On 17 December 1986 the pile was redriven to tip elevation -110 ft, at which point the PDA capacity prediction was 330-360 tons (J = 0.5), indicating that the minimum specified capacity had been achieved; subsequently, however, CAPWAP indicated only 237 tons capacity.

Redriving continued the same day, for an additional 6 inches of penetration; the PDA capacity prediction at this point had increased to 430-460 tons (J = 0.5). A CAPWAP analysis was performed indicating a capacity of 271 tons. GZA then requested a second CAPWAP, on the same PDA records but using different soil modeling assumptions: this analysis predicted 242 tons capacity.

On 29 January 1987 another restrike was performed, resulting in a total 1.5 inches additional penetration. The PDA capacity prediction ranged from 294 at the beginning of the restrike to 340 at the end of the restrike, based on an increased J of .63 to .65. CAPWAP predictions varied from 308 tons at the beginning of the restrike to 370 tons at the end.

The pile at this point had a tip elevation of -111 ft, 44 ft below the original elevation indicated on the project drawings, but considerably above the rock elevation of -137 encountered in Boring D104, performed at station 37+21 as part of the Supplementary Boring Program in the Fall of 1987. A static load test, performed on 17 February 1987, indicated an ultimate pile capacity of 180 tons, considerably short of the 330 tons required for the Trestle piles (Figure 4D).

No further testing was performed on this pile.

2.5. Test Pile at Station 51+40 (Trestle, Bent 26, "TP-4").

A 24 inch PSC test pile, with an initial length of 110 ft, was driven at station 51+40 on 21 October 1986, using the Delmag D46-23 hammer. A drive log summary and details are included in Figure 3E and Table 1E. At a tip elevation of -105 ft, which is 48 ft below the elevation indicated on the contract drawings, the PDA capacity prediction was 200-230 tons (J = 0.5); the CAPWAP capacity prediction was 207 tons. Driving was stopped at this point due to the pile length limitation.

To permit deeper penetration, the pile was lengthened to 178 ft by splicing a section of 24 inch square PSC pile and was redriven on 13 November 1986 to a tip elevation of -111 ft, at which point the PDA capacity prediction was 556 tons (J = 0.5). CAPWAP, however, predicted only 252 tons capacity.

Due to the low CAPWAP capacity prediction, driving continued on 14 November 1986 to a tip elevation of -145 ft; driving was stopped at this point as a result of mechanical problems with the pile hammer. The PDA capacity prediction at final drive was 212 tons assuming a J factor of 0.9, which is much higher than the values previously used, and which brought the PDA capacity prediction closer to the CAPWAP capacity of 260 tons.

On 17 November 1986 driving resumed, and continued to the maximum penetration permitted by the available pile length, for a final tip elevation of -168 ft. Despite the high final driving resistance of 81 blows for 4 inches of penetration, the PDA capacity prediction at this point was only 172 tons (J = 0.9), whereas CAPWAP predicted 291 tons.

A restrike on 18 November 1986 resulted in similarly high driving resistance (30 blows for 2.25 inches) and a PDA capacity prediction of 180 tons (J = 0.9); CAPWAP predicted a capacity of 323 tons.

On 1 December 1986 a static load test indicated a capacity of 520 tons, considerably in excess of the required 330 tons (Figure 4E). A restrike on 4 December 1986 resulted in a somewhat increased PDA capacity prediction of 210 tons with a reduced J factor of 0.7; the CAPWAP prediction, however, decreased to 227 tons, or less than half the measured capacity.

Boring D129, performed at station 51+44 as part of the Supplementary Boring Program in the Fall of 1987, encountered bedrock (weathered slate) at elevation -232 ft, approximately 64 ft below the tip elevation of the test pile.

No further testing was carried out on this pile.

3. SUMMARY OF PRESTRESSED-CONCRETE TEST PILE CAPACITIES

The results of the testing on the 5 test piles as described above have been summarized in Table 2. Of the five test piles, one was broken and only two eventually developed adequate capacity, demonstrating that additional testing was required.

It is clear that at certain locations and depths, soil-supported piles have adequate capacity; in particular, some zones within the till layer (IIIA) are suitable for soil support. However, identification of these locations and depths apparently cannot be accomplished simply by blowcount criteria, as there is a poor correlation between blowcount and static load capacity. Nor can adequate pile capacity be assured by use of the PDA and CAPWAP, due to the failure of these methods to predict capacity within an acceptable margin of error.

A comparison of PDA and CAPWAP capacity predictions is presented in Table 3, for the five cases where the predictions were checked by static load test. In general PDA and CAPWAP predictions were recorded at the end of driving (final drive) and again during restrike prior to the load test. As noted on the table, in three of the cases additional estimates were made on the basis of restrikes performed after the load test.

In evaluating the PDA predictions it is important to recognize that, as discussed above, the predictions are strongly affected by the assumed "damping" factor, J. Widely varying J factors were used in an attempt to bring the PDA capacity predictions in line with the load test results; however, no single J factor was found to be usable for all test piles. As a result no consistent, reliable relationship between PDA predictions and load test capacity was developed; errors at final drive averaged 114 percent of the test capacity, ranging from -67 percent (underprediction of capacity) to +310 percent. On restrike, the errors averaged 101 percent, ranging from -65 to +523 percent.

Errors in CAPWAP capacity predictions were, on the average, less than in the PDA predictions. Nevertheless, the errors were unacceptably large, averaging 56 percent at final drive, with a range of -46 to +131 percent; at restrike, the errors averaged 48 percent, ranging from -56 to +99 percent. In contrast to the PDA capacity predictions, CAPWAP predictions do not require prior assumption of a correlation ("damping") factor. In the computer laboratory, CAPWAP analysis of the PDA data produces estimates of soil damping and other parameters, in addition to pile capacity. The results, therefore, should not be as dependent on site calibration as are the PDA predictions. Unfortunately, at the Jamestown site the CAPWAP analyses were not sufficiently accurate to provide dependable capacity predictions.

In the absence of a reliable method for assurance of adequate capacity of piles supported by the overburden soils above till or rock, the feasibility of piles supported on rock or on the competent tills immediately over the rock was considered, as discussed below.

4. H-PILE TESTING AT PIER 12.

In reviewing the results of the prestressed-concrete pile testing in the West Approach and Trestle sections, and evaluating the options for further testing, attention was directed to the H-pile driving and testing that had been carried out at Pier 12, station 74+04 (west side of Main Span). The records from this testing provided potentially useful on-site experience in developing pile bearing capacity with an H-pile at or near bedrock. At the Pier 12 location, the initial test pile failed structurally during static load testing, due to insufficient lateral support leading to a buckling failure, and not due to a bearing capacity failure. A replacement test pile, however, was installed and successfully tested; the details can be summarized as follows (Figure 3F and Table 4).

On 25 August 1986, a 181 ft long steel HP14x117 section was installed to tip elevation -136 ft, approximately, using an ICE 812 vibratory driver. On 26 August 1986, impact driving began using a Delmag D36-23 hammer; the total driven length was approximately 226 ft, consisting of the 181 ft long HP14x117 pile section and a 45 ft long HP14x117 follower. Driving resistance was generally 30 to 42 blows per foot for the first 31 ft, then increased sharply to 6, 8, and 12 blows per inch for the final three inches of driving, at a final tip elevation of -167.4 ft. The PDA capacity prediction at final drive was reported as 390 to 550 tons, with a J-factor of 0.5. Subsequently, a CAPWAP capacity prediction of 419 tons was obtained.

A restrike was performed on 28 August 1986, again using the D36-23 hammer. A total 20 blows were applied at #4 (maximum) fuel setting, with a penetration of approximately 1 inch. The PDA capacity prediction was 452 tons (J = 0.5); the CAPWAP prediction was 419 tons, as for the original drive.

On 18 September 1986 the pile was statically load tested to 400 tons without failure; hence the ultimate capacity is in excess of 400 tons (Figure 4F).

Soil borings near the Pier 12 test pile indicate that the pile tip is probably on bedrock, although the possibility remains that it rests on a boulder or in hard till above the bedrock surface. There are three borings, numbered B17 (station 74+90), B29 (73+25) and B78 (74+00), in the vicinity of the test pile, which indicate bedrock elevations ranging from 7 ft above to 8 ft below the tip elevation of the test pile. One of the borings, B17, encountered a boulder near the bedrock surface. Two borings, B17 and B29, indicate sand till immediately over the bedrock, at thicknesses of 9 and 41 ft, respectively. Boring B78 indicates 6 ft of dense sand, with a Standard Penetration Test N-value of 34, over bedrock. Top of bedrock is described as medium hard shaley sandstone in B17, weathered shale over medium hard fractured and seamy shale in B28, and weathered soft shale in B78.

The HP testing at Pier 12 is relevant to the selection of the composite pile for testing in the West Approach and Trestle sections, because it provides some indication of: 1) the relative ease of driving the HP section through the till soils overlying the bedrock, and 2) the ability to develop adequate bearing capacity by bearing on or near the bedrock.

5. COMPOSITE PILE

5.1. Configuration.

Various pile configurations were considered by RIDOT for the longer piles required in order to reach bedrock. The configuration proposed by RIDOT, and M.T. Davisson, Consulting Engineers, for the test program incorporates a composite design, with PSC concrete top section, and steel HP bottom extension section.

The selection of the composite design was based on a number of factors, including the following:

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Corrosion Resistance. Use of the PSC design for the pile top section preserves the corrosion resistance, stability with respect to buckling, and appearance of the original design.

Column Strength. It was necessary to provide buckling strength and lateral load resistance equivalent to the original design.

Appearance. In the case of the Trestle piles, which extend well above water level, the composite piles preserve the appearance of the original PSC piles.

Drivability. Based on jobsite experience at Pier 12, the HP bottom section affords the probability of easier driving through the soils overlying the bedrock, as compared to the much-larger PSC displacement pile sections. With reference to the high total blowcounts noted on the drive log summaries (Figures 3A through 3E), driving time was considered to be an important factor in pile selection.

Contractor's Equipment. The limitations of the Contractor's pile-handling equipment were an important factor in the selection of the HP bottom section.

Schedule. Time required for structural design of the pile, as well as supply and fabrication of pile material for testing and production driving, was a critical factor in the pile selection.

Economy. Estimated costs of the composite pile as compared to acceptable alternates were given strong consideration in the selection process.

The composite pile was determined to be the best option, considering all the factors listed above, and subject to verification by testing.

5.2. Details.

The proposed PSC concrete top sections are identical in section dimensions to the original pile design: 20 inch square for the W. Approach and 24 inch square for the Trestle; however, the level of prestress will be increased in the case of the 20 inch square piles in order to provide greater resistance to tensile cracking during driving. The length of the top sections were specified as approximately 100 ft for purposes of the test program. The steel HP bottom extension sections were specified as HP14 x 117, to be furnished in lengths sufficient to reach bedrock.

To provide a field connection of the top and bottom sections, the top sections were specified to be furnished with a steel end plate, and with an HP stub section welded to the end plate. The end plate was specified to be anchored to the PSC section using 9 to 11 ft long steel dowel bars welded to the plate and cast into the pile section.

5.3. Drivability.

Wave equation analyses of the proposed composite pile (M.T. Davisson, Consulting Engineer, 1987), indicated that the drivability of the composite pile with an HP14 x 117 bottom section is marginal at the longer lengths (over 150 ft total pile length). That is, the analyses indicated that the axial stiffness of the HP14 x 117 section is marginally adequate for development of the required pile capacities, for the longer piles on the project. Testing would be required to verify the adequacy of the H14 x 117 section.

6. CONCLUSIONS

6.1. Pile Tip Elevation.

The primary conclusion drawn from the testing is that in order reliably to develop the required pile capacities, and with due consideration for the constraints of schedule and other factors, it will be necessary to found the piles on bedrock or the till immediately over bedrock. Further testing should be directed at verification of the adequacy of pile details and installation methods.

6.2. Pile Configuration.

The composite pile potentially provides the ability to penetrate the stiff overburden soils and to develop adequate bearing capacity, and is recommended for further testing.

6.3. Instrumentation.

Neither the PDA nor CAPWAP have yielded dependable predictions of pile capacity at this site. Driving criteria should be developed on the basis of the results of the composite pile test program, without further reliance on pile capacity predictions from the PDA and CAPWAP.

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REFERENCES

M.T. Davisson, Foundation Engineer (1987), "Drivability Study - Composite Piles, Jamestown Bridge," memorandum to Gordon R. Archibald, Inc., and RIDOT, dated 5 January.

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Maguire Group Inc. (1987), Boring Logs, Supplemental Boring Program, performed in Fall of 1987.

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Sverdrup & Parcel and Associates, Inc. (1982), "Jamestown Bridge Replacement, North Kingstown and Jamestown, Rhode Island, Geotechnical Report", October, with revisions dated May 1983.

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TABLES

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TABLE 1A - DATA SUMMARY: W. APPROACH, TP1, STA 56+80 [20" PSC PILE]

Date	Ldrv	Hammer	Stroke	BPM	Fuel	TipEl	Blowcnt	........Ru-PDA, tons..........				Ru-CWP	Ru-Tst	Notes
	ft		ft		Set	ft		PDA-Sum	GZA-Notes	DrivLog	J	tons	tons	
09Apr86	123	D36-23												
Near end of drive:			-	40	4	-68.4	5/1"	-	380-400	-	0.2	-	-	Stop,adjust fuel.
End of drive:			-	45-47	1	-71.4	8/1"	346	340	346	0.2	192	-	Stop due to high PDA capacity.
10Apr86	79	D36-23	-	39-41	1	-72.1	48/9" (7/1")	517	520	523	0.2	196 135 Note->	-	Restrike after cutoff. Reran CAPWAP after load test (see 21Apr86)
21Apr86	STATIC LOAD TEST ==>												83	<340 tons req'd.
16May86	79	D36-23	-	-	4	-72.4	10/3"	110-125	-	110-120	0.6	102	-	Restrike before splice. Add PSC section.
20May86	146	D36-23	-	40	4	-78.0	93/ft	140-160	140-160	140-160	0.6	151	-	Redrive. Stop due to pile damage.
26Jun86	146	D36-23	-	39	4	-86.5	22/6"	-	200 Note->	-	0.6	-	-	Redrive. Stop due to pile damage. Remove top 8'. PDA:Corrected for sensor damage.
27Jun86	138	D36-23	-	39	4	-97.5	30/6"	-	180	170	0.6	140	-	Pile broke near splice.

Definitions:

Ldrv =	Length of pile at time of driving.
Stroke =	Ram stroke.
Fuel Setting =	Hammer throttle setting, at final drive unless otherwise noted; #1 is lowest, #4 is highest.
TipEl =	Estimated tip elevation at completion of driving.
Blowcnt =	Hammer blowcount: at completion of driving, unless otherwise noted.
Ru-PDA =	Pile capacity prediction by pile driving analyzer.
PDA-Sum =	PDA prediction as listed on the PDA summary sheet.
GZA-Notes =	PDA prediction as listed in Goldberg-Zoino & Associates notes or correspondence.
DriveLog =	PDA prediction as listed on pile driving logs.
J =	PDA correction factor.
Ru-CWP =	Pile capacity prediction by CAPWAP analysis of PDA data.
Ru-Test =	Pile capacity, as measured by static load test.

Note: Refer to original drive logs, PDA tapes and reports, and CAPWAP reports for additional details.

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TABLE 1B - DATA SUMMARY: W. APPROACH, TP2, STA 69+00 [20" PSC PILE]

Date	Ldrv	Hammer	Stroke	BPM	Fuel	TipEl	Blowcnt	........Ru-PDA, tons...........				Ru-CWP	Ru-Tst	Notes
	ft		ft		Set	ft		PDA-Sum	GZA-Notes	DriveLog	J	tons	tons	
01May86	128	D36-23	-	47-51	1	-109.0	12/ft	0	35	-	0.6	45	-	Stop due to pile length limitation.
13May86	128	D36-23	-	38-40	4	-110.2	26/14"	79	79	-	0.6	70	-	Restrike prior to splice. Add PSC section.
16May86	167	D36-23	-	42(log) 38-39(PDA)	4	-128.0	108/ft	278	280	250-280	0.5	169	-	Pile spalling.
20May86	167	D36-23	-	38-40	4	-140.5	371/ft (28/1")	-	160-180	180	0.6	205	-	Redrive. Stop due to high driving resistance.
30Jun86	167	D36-23	-	40	4	-140.6	21/.75"	260-280	260-280	260-280	0.6	211 204	-	Early in restrike. Late in restrike.
24Jul86	STATIC LOAD TEST ==>												240	<340 tons req'd.
24Sep86	148	D46/23 (Change of hammer)												
	Early in restrike:													
			-	43-47	4	-140.6	42/1"	235	240	235	0.6	196	-	Restrike.
	Late in restrike:													
			-	43-44	4	-141.4	34/1"	284	280	284	0.6	149	-	

Definitions, Notes: See Table 1A.

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TABLE 1C - DATA SUMMARY: W. APPROACH, TP1(REPL.), STA 56+95 (20" PSC PILE)

Date	Ldrv	Hammer	Stroke	BPM	Fuel	TipEl	Blowcnt	........Ru-PDA, tons...........				Ru-CWP	Ru-Tst	Notes
	ft		ft		Set	ft		PDA-Sum	GZA-Notes	DriveLog	J	tons	tons	
07-08Oct86	135	D46/23												
	Near end of driving:													
			-	45-46	4	-106.0	9/9"	-	100	-	0.5	-	-	Reduce fuel set.
	End of driving:													
			-	48-49	3	-109.0	30/9"	Note->	-	160	0.5	-	-	Stop due to damaged sensor. PDA tape: 153-166 tons capacity.
09Oct86	135	D46/23	6.5-7.5	45	3	-127.5	42/12", 24/15"	-	95	95-100	0.5	81	-	Stop due to pile length limitation. Splice - steel extension,W14x283, L=55'.
23Dec86	189	D46/23	7.5-8.5	39.5	4	-181.7	55/8"	340	340-350	340	0.5	226	-	Redrive after splicing.
24Dec86	189	D46/23	-	41-43	4	-182.1	Note->	324	-	315-350	0.5	264	-	Restrike: 23/1.5" 9/1" 8/1" 8/1" 10/1"
15Jan87	STATIC LOAD TEST ==>												420	>340 tons req'd.

Definitions, Notes: See Table 1A.

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TABLE 1D - DATA SUMMARY: TRESTLE TP2, BENT 9, STA 37+20 [24" PSC PILE]

Date	Ldrv	Hammer	Stroke	BPM	Fuel	TipEl	Blowcnt	Ru-PDA, tons				Ru-CWP	Ru-Tst	Notes
	ft		ft		Set	ft		PDA-Sum	GZA-Notes	DriveLog	J	tons	tons	
16Oct - 20Oct86	110	D46/23	7.5	41-46	3	-100.0	64/11"	-	200-230	150-200	0.5	200	-	Initial drive. Stop due to pile length limitation.
11Dec86	110	D46/23												
	First 30 blows:		5.5	-	2									
	Last 10 blows:		8.0	-	4	-100.1	40/1.5"	220	220	220	0.5	205	-	Restrike prior to splice.
														Add PSC section.
17Dec86	140	D46/23												
	Initial drive:		9.0	39	-	-110.0	7/1"	360	330-360	330-360	0.5	237	-	Redrive after splice. Stop due to high PDA capacity.
	Cont. drive:		9.0	40	-	-110.5	5/1"-8/1"	430-460	400-420	-	0.5	271	-	
												242		Second CAPWAP on same data.
29Jan87	140	D46/23												
	Initial rstrk:		7.7	42.4	4			294	290	-	.63-.65	308	-	Restrike.
	Cont. rstrk:		8.5	40.4	4	-110.6	32/1.5"	330	330	340	.63-.65	370	-	
17Feb87	STATIC LOAD TEST ==>												180	< 330 tons req'd.

Definitions, Notes: See Table 1A.

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TABLE 1E - DATA SUMMARY: TRESTLE TP4, BENT 26, STA 51+40 [24" PSC PILE]

Date	Ldrv	Hammer	Stroke	BPM	Fuel	TipEl	Blowcnt	Ru-PDA, tons				Ru-CWP	Ru-Tst	Notes
	ft		ft		Set	ft		PDA-Sum	GZA-Notes	DriveLog	J	tons	tons	
21Oct86	110	D46/23	-	41	3	-105.0	45/11"	-	200-230	150-200	0.5	207	-	Initial drive. Stop due to pile length limitation. Add PSC section.
13Nov86	179	D46/23												Redrive after splice.
Beginning of drive:			-	45	3	-106.2	166/12"	278	-	-	0.5	188	-	
End of drive:			-	39	4	-111.5	69/4"	556	500-540	500-550	0.5	252	-	Continue driving.
14Nov86	179	D46/23	-	39	4	-145.0	102/ft	212	200-220	200-250	0.9	260	-	Hammer problems.
17Nov86	179	D46/23	-	38	4	-168.0	81/4"	172	180-200	200-250	0.9	291	-	Stop due to pile length limitation.
18Nov86	179	D46/23	-	42	-	-168.3	11/1", 14/1", 5/.25"	180	180	190	0.9	323	-	Restrike.
01Dec86	STATIC LOAD TEST ==>												520	>330 tons req'd.
04Dec86	173	D46/23	-	43	-	-168.4	25/1"	210	200-205	-	0.7	227	-	Restrike. Pile spalled.

Definitions, Notes: See Table 1A.

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TABLE 2 - SUMMARY OF STATIC LOAD TEST RESULTS - W.APPROACH AND TRESTLE

Test Pile	Estimated Tip Elevation, ft	Ultimate Capacity, tons	Notes
West Approach:			
TP1, Sta. 56+80	-72	83	Failed (<340 tons). Pile broke during subsequent redrive, with tip at el. -97 ft.
TP1(Repl.), Sta. 56+95	-182	420	Passed (>340 tons).
TP2, sta. 69+00	-141	240	Failed (<340 tons).
Trestle:			
TP2, Sta. 37+20	-111	180	Failed (<330 tons).
TP4, Sta. 51+40	-168	520	Passed (>330 tons).

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TABLE 3 COMPARISON OF PDA AND CAPWAP CAPACITY PREDICTIONS WITH LOAD TEST RESULTS

Test Pile Sta.	Load Test		PDA										CAPWAP								Note
			Final Drive					Restrike					Final Drive				Restrike				
		Ru		Ru		Error			Ru		Error		Ru		Error		Ru		Error		
	Date	Tons	J	Low	High	Tons	%	J	Low	High	Tons	%	Low	High	Tons	%	Low	High	Tons	%	
56+80	21Apr86	83	0.2	346	346	263	317%	0.2	517	517	434	523%	192	192	109	131%	135	196	83	99%	(1)
								0.6	110	125	35	42%					102	102	19	23%	(2)
69+00	24Jul86	240	0.6	160	180	-70	-29%	0.6	260	280	30	13%	205	205	-35	-15%	204	211	-33	-14%	(1)
								0.6	235	284	20	8%					149	196	-68	-28%	(2)
51+40	01Dec86	520	0.9	172	172	-348	-67%	0.9	180	180	-340	-65%	291	291	-229	-44%	323	323	-197	-38%	(1)
								0.7	210	210	-310	-60%					227	227	-293	-56%	(2)
56+95	15Jan87	420	0.5	340	340	-80	-19%	0.5	324	324	-96	-23%	226	226	-194	-46%	264	264	-156	-37%	(1)
37+20	17Feb87	180	0.5	400	460	250	139%	0.6	294	330	132	73%	242	271	77	43%	308	370	159	88%	(1)
SUMMARY >			Error:					Error:					Error:				Error:				
			Average				114%	Average				101%	Average			56%	Average			48%	
			Range from				-67%	Range from				-65%	Range from			-46%	Range from			-56%	
			to				317%	to				523%	to			131%	to			99%	

Notes:

Test Pile Sta. = Nominal stationing of test pile.

Ru = Ultimate axial compressive capacity of pile, in tons.

PDA = Pile Driving Analyzer, used to predict pile capacity at the time of driving.

CAPWAP = Computer analysis of PDA measurements, resulting in a capacity prediction.

J = Damping factor used in PDA capacity prediction.

Error = Difference between pile capacity as predicted by PDA or CAPWAP and as measured in static load test. Where more than one prediction was recorded, the average of the low and high prediction was used. The error is expressed both in tons and percent of static load test capacity.

Avg. Error = Average percentage error, using absolute values.

(1) Restrike prior to load test.

(2) Restrike after load test.

TABLE 4 - DATA SUMMARY: PIER 12, 2ND TEST PILE: HP 14 X 117

Date	Ldrv ft	Hammer	Stroke ft	BPM	Fuel Set	TipEl ft	Blowcnt	Ru-PDA, tons PDA-Sum	GZA-Notes	DriveLog	J	Ru-CWP tons	Ru-Tst tons	Notes
25Aug86	151.0	Vibro ICE-812	-	-	-	-135.6	-	-	-	-	-	-	-	Initial drive, prior to final splice.
26Aug86	181.3'+45'Follower(HP14x117)=226.3':													
		D36-23	NR	42*	NR	-167.3	12/1"	390-550	390-500	395-517	0.5	419	-	Final drive, with impact hammer. Stopped due to follower racking.
		* PDA record indicates 36-37 BPM at end of driving.												
28Aug86	226.3	D36-23	NR	**	#4	-167.4	20/1"	452	450	450	0.5	419	-	Restrike prior to load test.
		** PDA record indicates 38-39 BPM.												
18Sep86	STATIC LOAD TEST ==>												>400	Did not reach failure load.

Definitions, Notes: See Table 1A.

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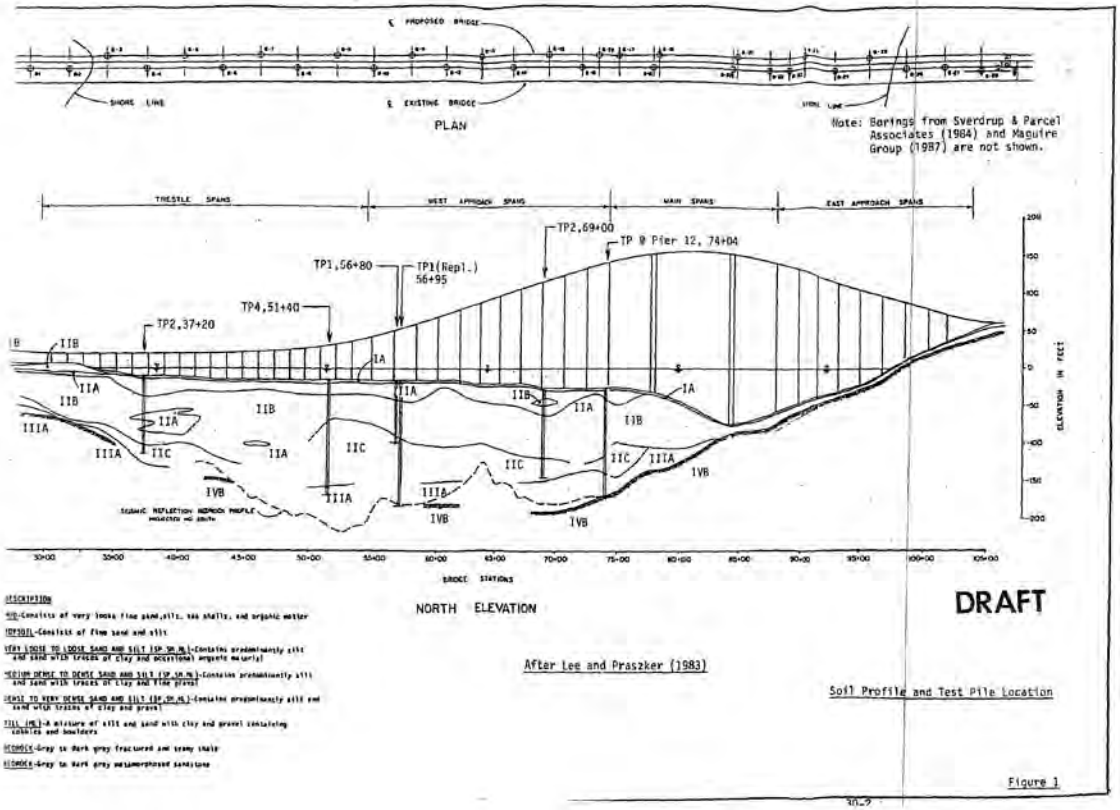
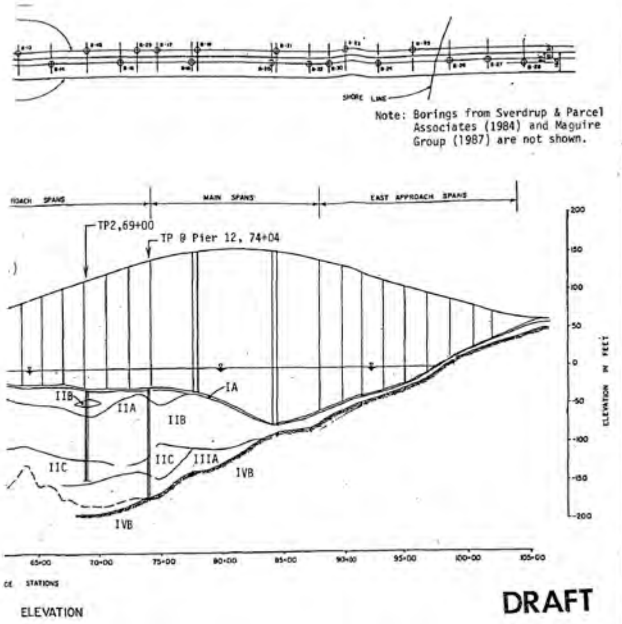
FIGURES



After Lee and Praszker (1983)

Soil Profile and Test Pile Location

Figure 1



Note: Borings from Sverdrup & Parcel Associates (1984) and Maguire Group (1987) are not shown.

ELEVATION

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After Lee and Praszker (1983)

Soil Profile and Test Pile Location

Figure 1

30-3

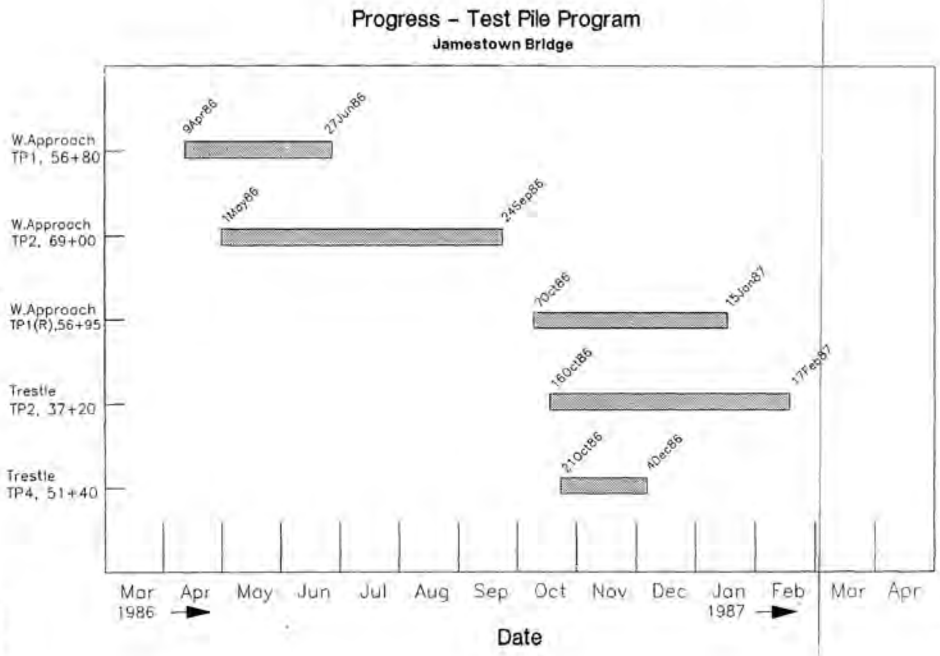
Progress - Test Pile Program
Jamestown Bridge
W.Approach TP1, 56+80
9Apr86
27Jun86
W.Approach TP2, 69+00
1May86
24Sep86
W.Approach TP1(R),56+95
7Oct86
15Jan87
Trestle TP2, 37+20
16Oct86
17Feb87
Trestle TP4, 51+40
21Oct86
4Dec86
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Jan
Feb
Mar
Apr
1986 →
1987 →
Date

Figure 2

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Test Pile Drive Log

W.APPROACH, TP1, STA 56+80

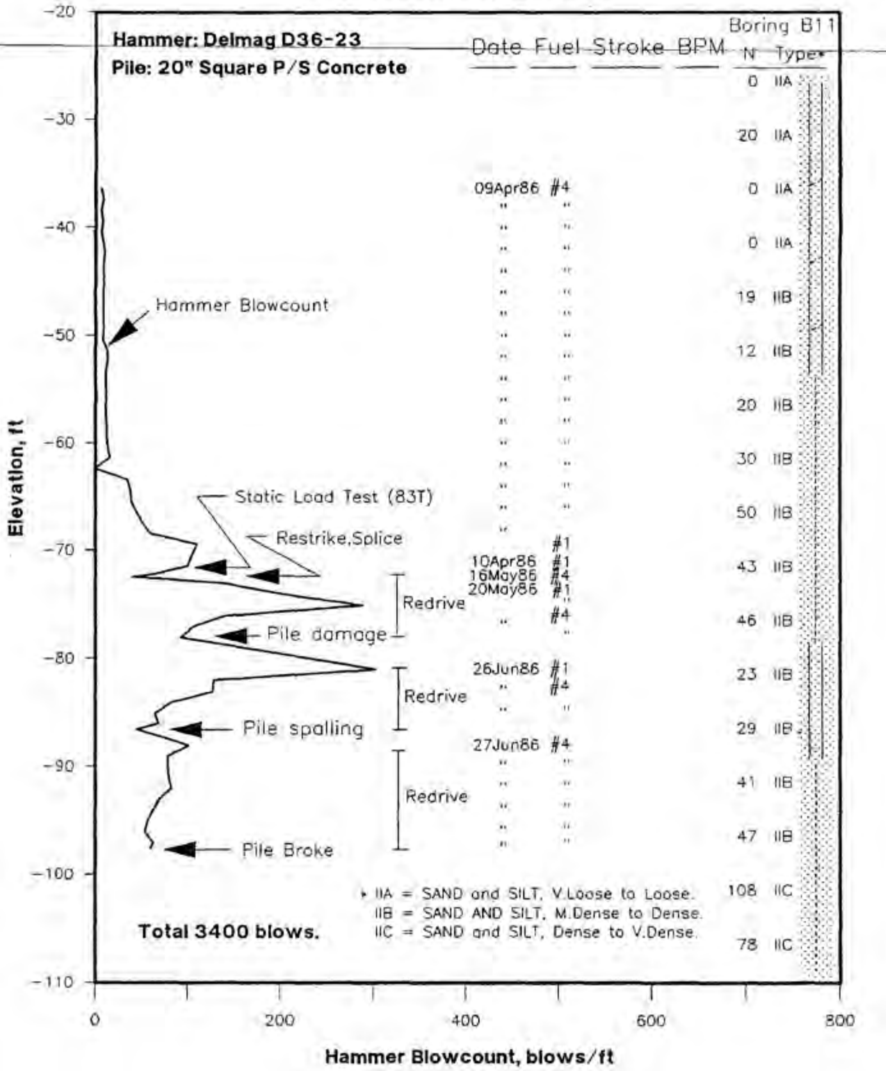

Figure 3A

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Test Pile Drive Log

W.APPROACH, TP2, STA 69+00

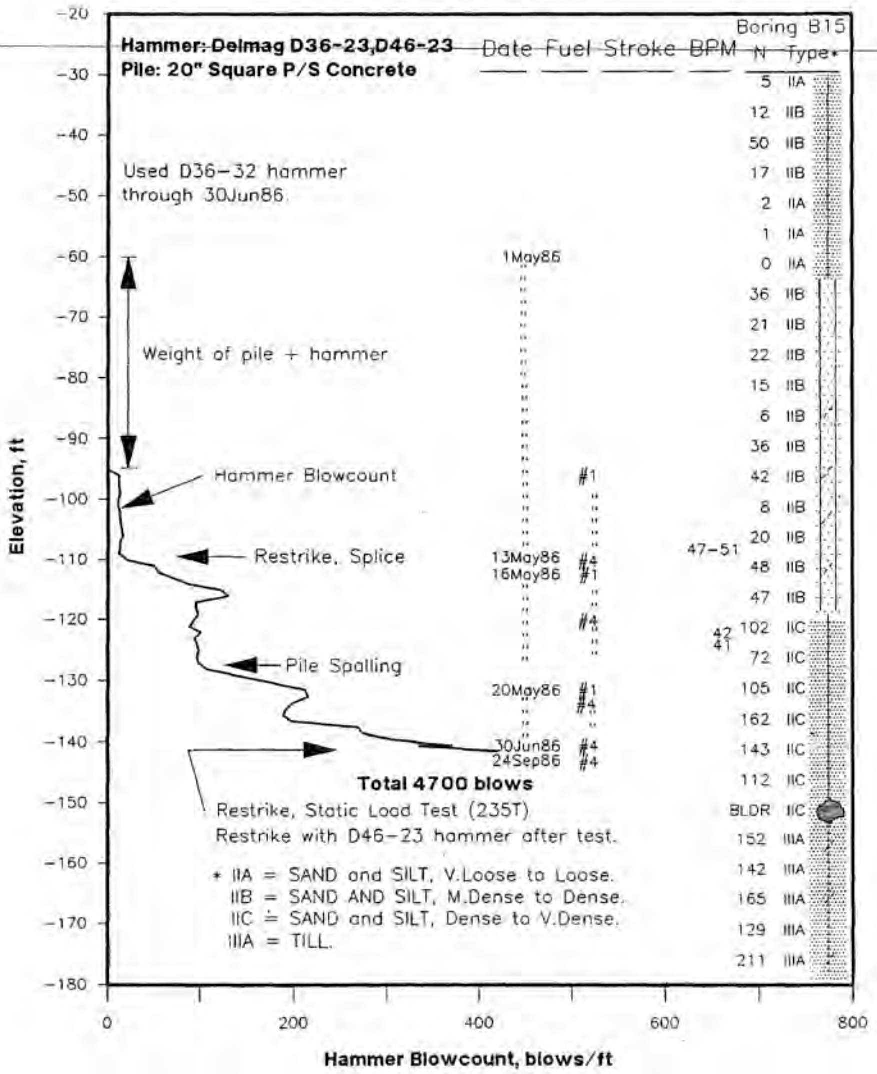


Figure 3B

DRAFT

Test Pile Drive Log

W.APPROACH, TP1(REPLACEMENT), STA 56+95

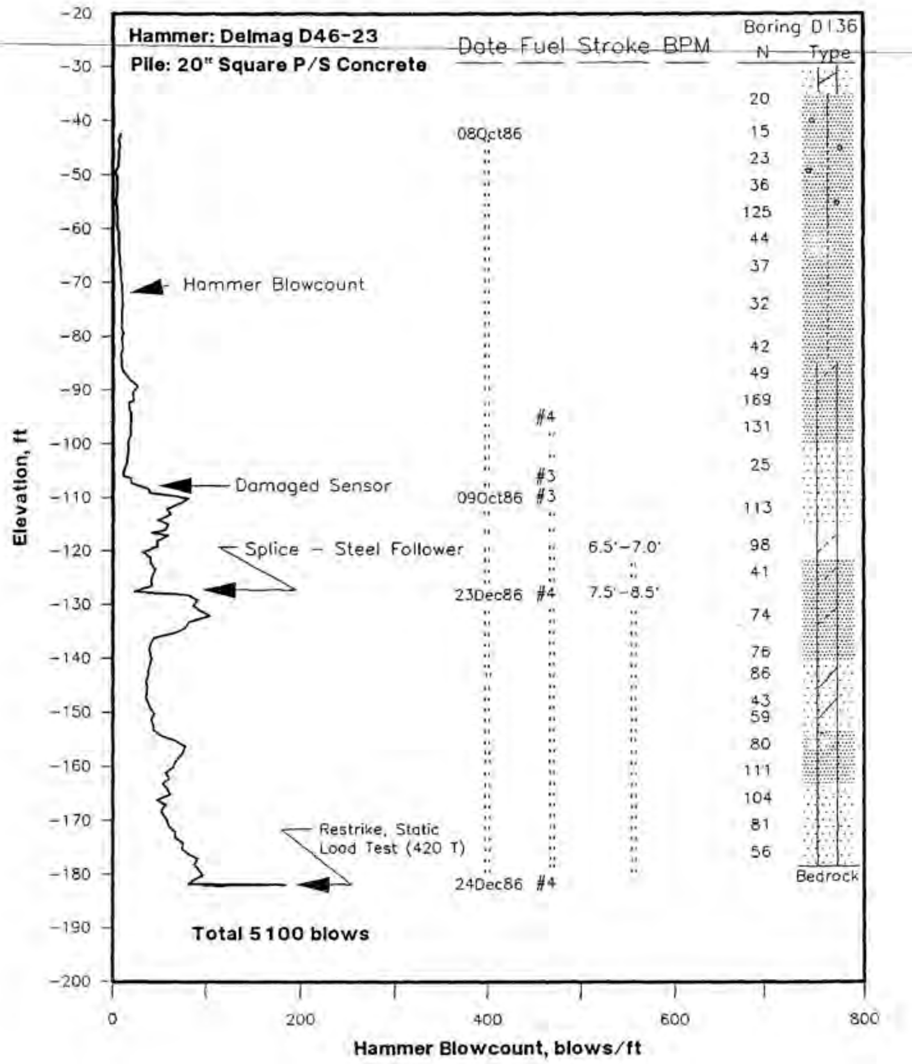
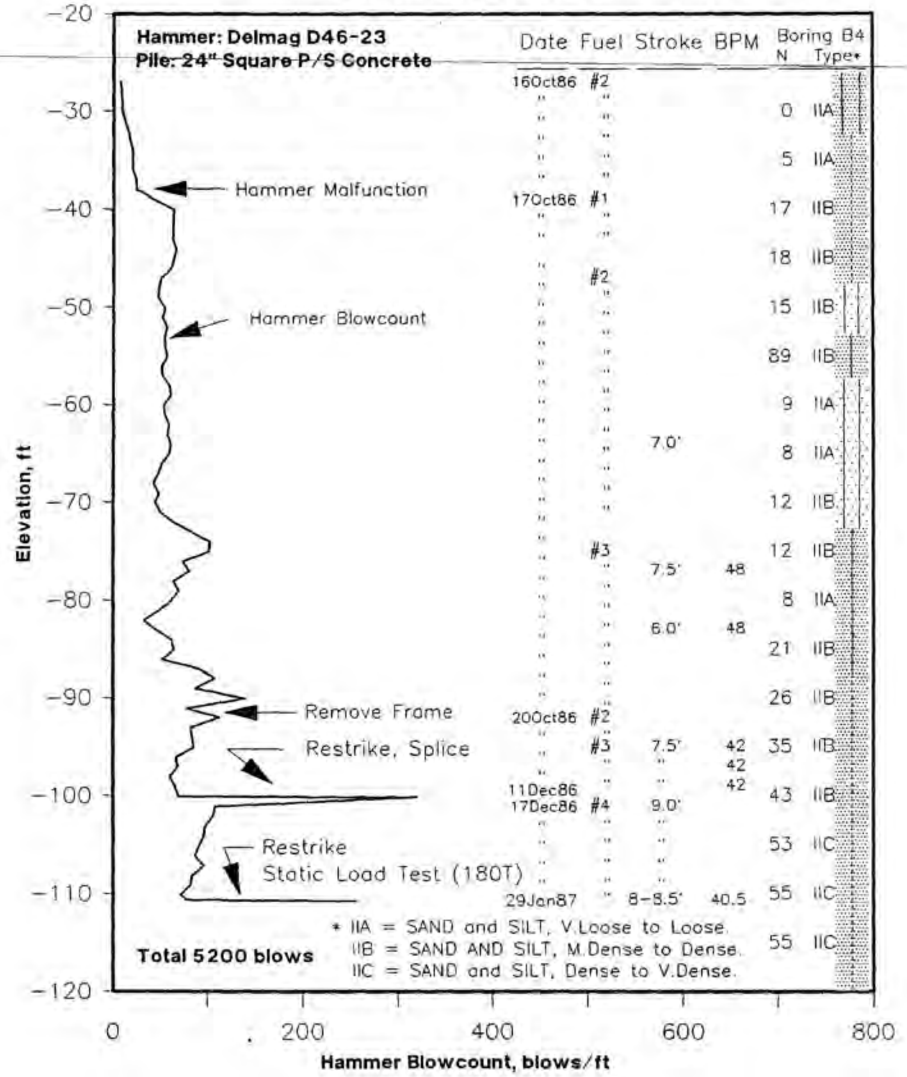


Figure 3C

DRAFT

Test Pile Drive Log

TRESTLE, TP2, STA 37+20



30-7

Figure 3D

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Test Pile Drive Log

TRESTLE, TP4, STA 51+40

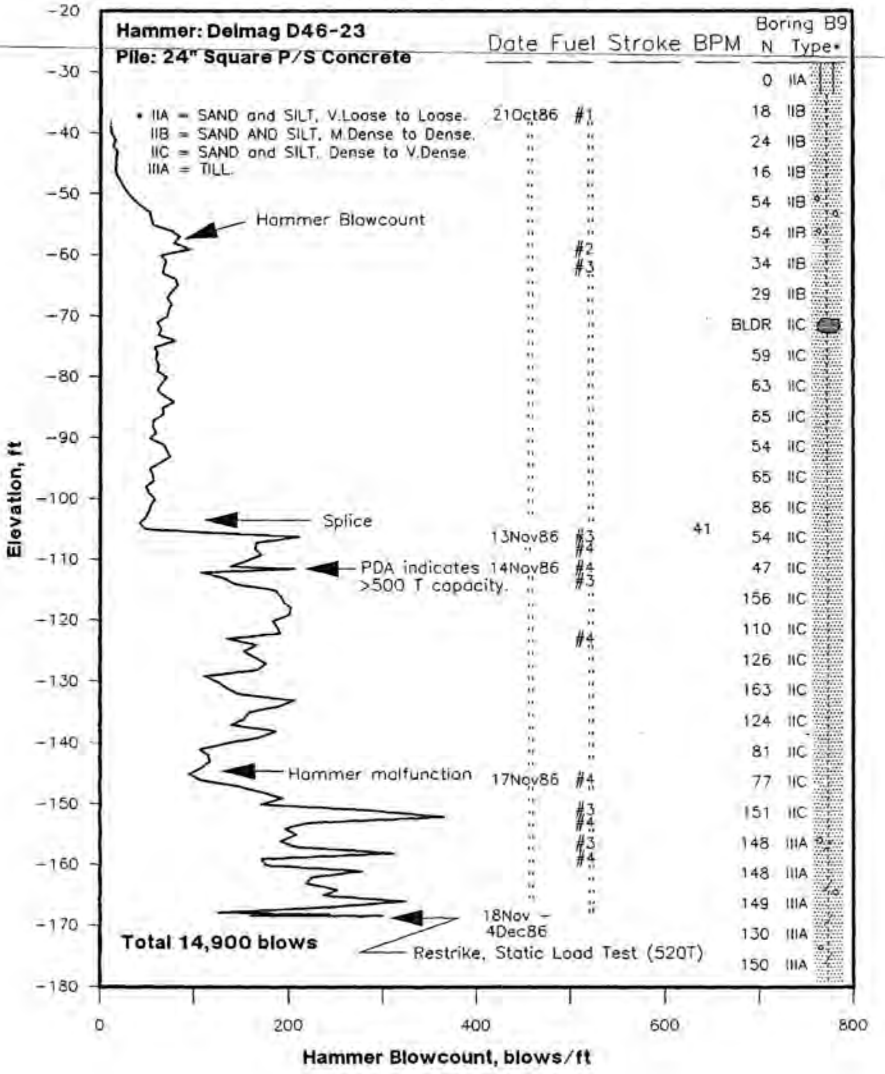
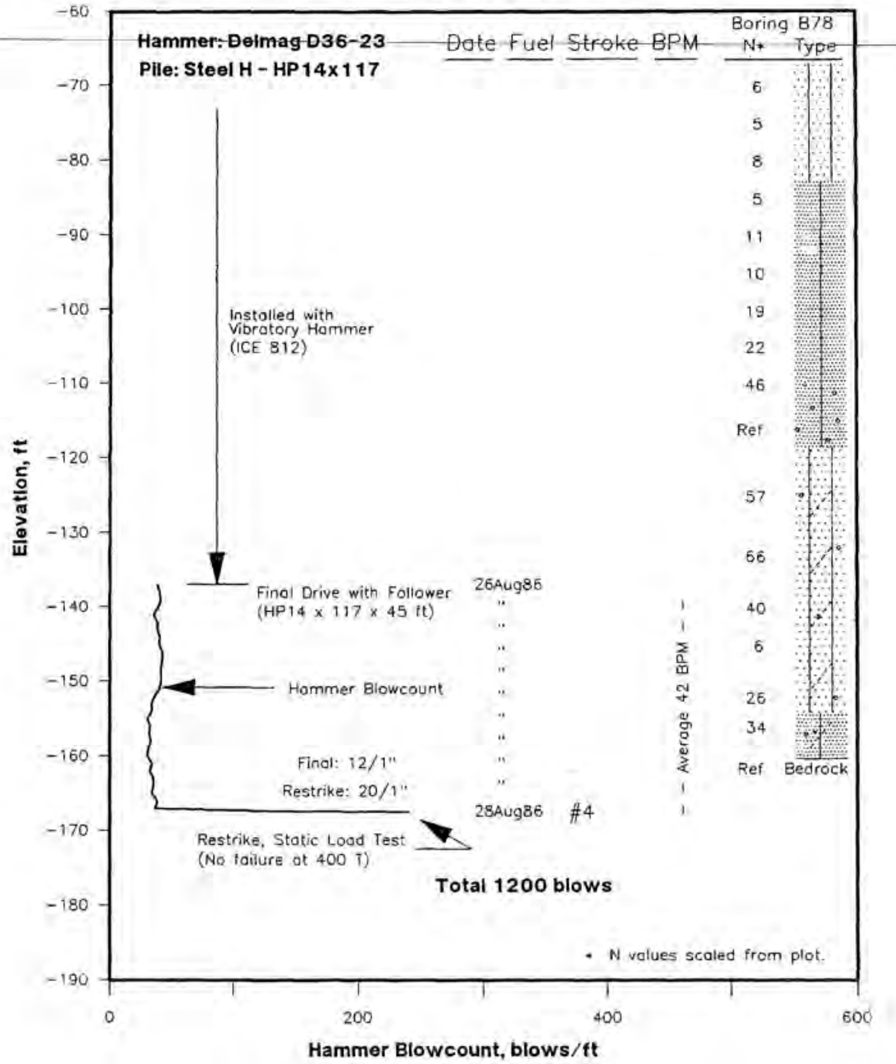


Figure 3E

DRAFT

Test Pile Drive Log

PIER 12, 2ND TEST PILE, STA 74+04



30-9

Figure 3F

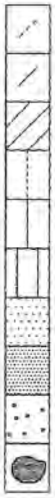
Notes: Test Pile Drive Logs

1. Logs are summaries and condensations of data. Refer to ~~original records for complete details.~~
2. Fuel = Fuel setting on hammer. Range is #1 (lowest) to #4 (highest).
3. Stroke = Ram fall, in feet.
4. BPM = Hammer blow rate in blows per minute.
5. Boring = Soil boring. Refer to complete reports for additional borings and more detail.

   N = Standard Penetration Test N-value.

   "IIA",etc. = Layer designation (see text of Report), Designations are from Sverdrup & Parcel Associates (1982) and Lee and Proszker (1983). No designations have been given for D136, which is shown on the log for TP1(Repl.), Sta. 56+95, and B78, which is shown on the log for the H-Pile test at Pier 12; these borings were performed after the referenced reports were issued.

   Symbols:



Trace to little clay

Some clay to clayey

Clay, to "and clay"

Trace to little silt

Some silt to silty

Silt, to "and silt"

Trace to little sand

Sand, to some sand

Gravel

Boulder

Figure 3G

DRAFT

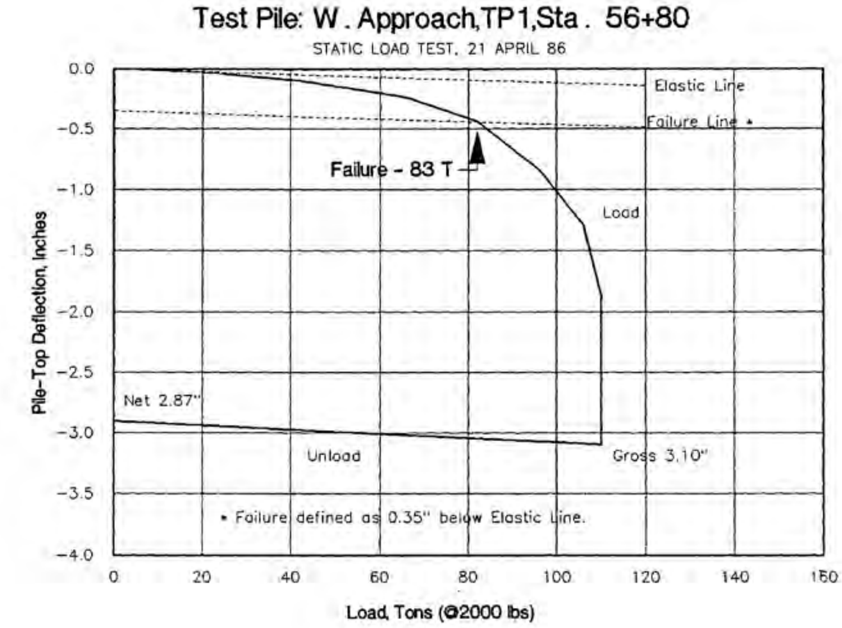


Figure 4A

DRAFT

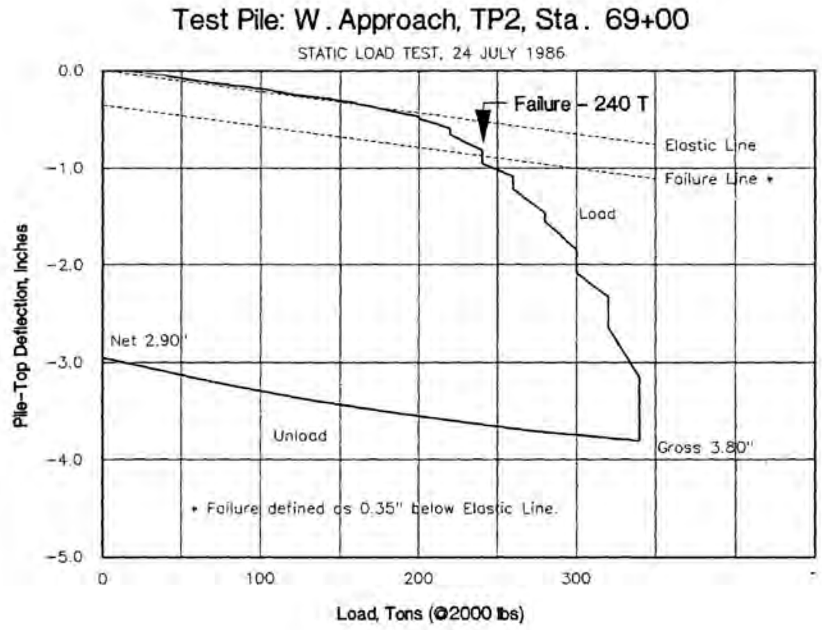


Figure 4B

DRAFT

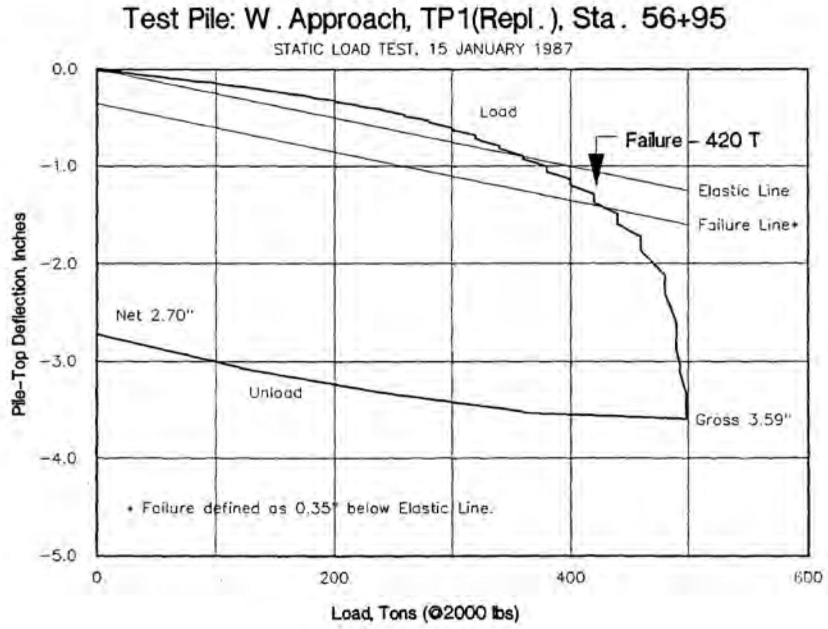


Figure 4C

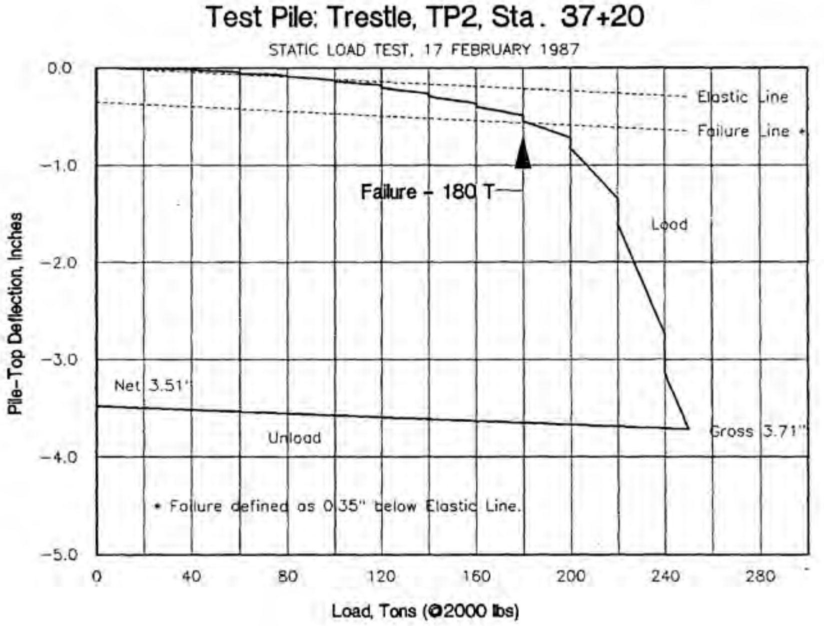
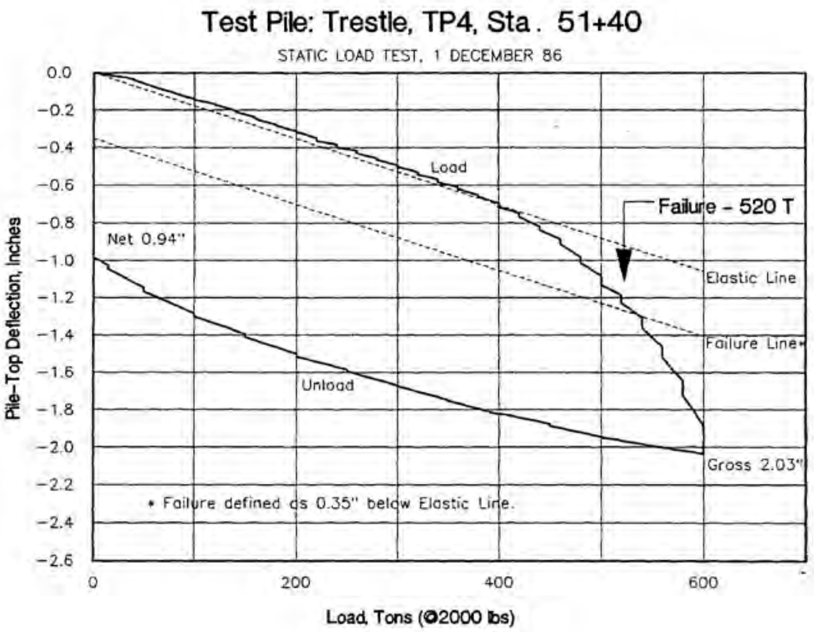

Test Pile: Trestle, TP2, Sta. 37+20
STATIC LOAD TEST, 17 FEBRUARY 1987
Pile-Top Deflection, Inches
0.0
-1.0
-2.0
-3.0
-4.0
-5.0
0
40
80
120
160
200
240
280
Load, Tons (@2000 lbs)
Elastic Line
Failure Line *
Failure - 180 T
Load
Net 3.51"
Gross 3.71"
Unload
* Failure defined as 0.35" below Elastic Line.

Figure 4D



Test Pile: Trestle, TP4, Sta. 51+40

STATIC LOAD TEST, 1 DECEMBER 86

Failure – 520 T

* Failure defined as 0.35" below Elastic Line.

Figure 4E

DRAFT

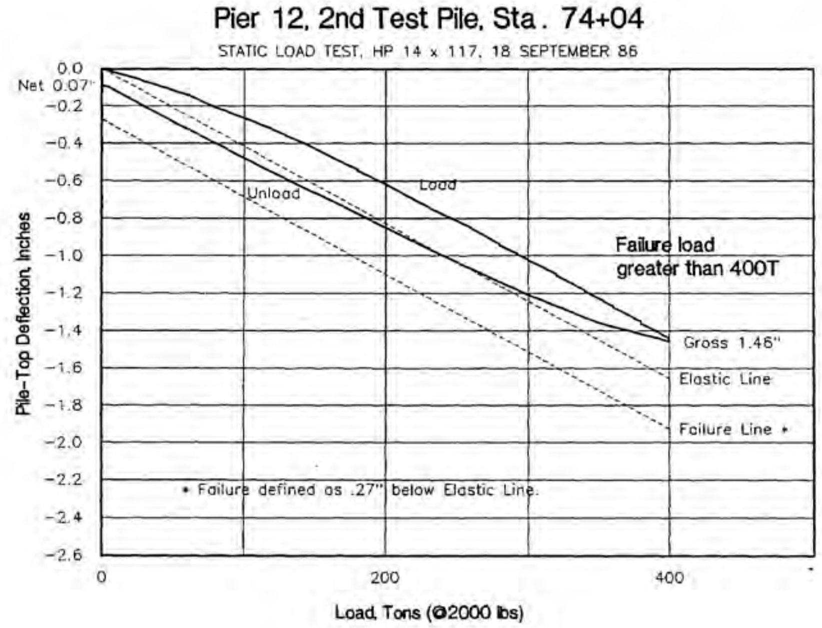


Figure 4F

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