ABSTRACT

CHARACTERIZATION OF TIME DEPENDENT PILE CAPACITY IN GLACIAL DEPOSITS BY DYNAMIC LOAD TESTS

by
Upendra L. Karna, P.E.

A study of the effects of time on axial pile capacity in glacial deposits is presented in this report. The dynamic and static load database test results of the Route 21, Viaduct Replacement project are studied and analyzed. In this project two sizes (18-inch and 24-inch dia.) of closed end pipe piles varying in length 100 to 150 feet driven through highly variable glacial deposits were utilized. Within a small reach the subsurface conditions and the behavior of pile capacity with time varied considerably. About 112 piles were tested dynamically by Pile Driving Analyzer. Restriking was performed on fifty-nine piles to establish the soil setup behavior. Restriking was performed generally at two and four weeks after the initial driving. Project area is divided into four soil types to characterize the soil setup behavior. With the exception of one soil type, the pile capacity increased with time. Most of the pile capacity increased within two weeks after driving and after that a moderate increase was observed. Capacity versus time relationship has been evaluated for each soil type and a reasonable setup behavior equation to predict the long term capacity in similar soils has been developed. With one exception, the capacity evaluated by static load tests were about 25 percent higher than the dynamic load tests. The use of dynamic tests to quantify the soil setup behavior in a glacial deposit is realized. The research is substantiated by relevant literature review. Conclusions are drawn and further research is recommended.

CHARACTERIZATION OF TIME DEPENDENT PILE CAPACITY IN GLACIAL DEPOSITS BY DYNAMIC LOAD TESTS

By
Upendra L. Karna, P.E.

A Degree of Engineer Project
Submitted to the Faculty of
New Jersey Institute of Technology
in Partial Fulfillment of the Requirements for the Degree of
Degree of Engineer in Civil Engineering

Department of Civil and Environmental Engineering

May 2001

APPROVAL PAGE

CHARACTERIZATION OF TIME DEPENDENT PILE CAPACITY IN GLACIAL DEPOSITS BY DYNAMIC LOAD TESTS

Upendra L. Karna, P.E.

Dr. Dorairaja Raghu, P.E., Thesis Advisor — Date
Professor of Civil and Environmental Engineering, NJIT

Dr. John R. Schuring, P.E., Committee Member — Date
Chairman, Dept. of Civil and Environmental Engineering, NJIT

Prof. Edward G. Dauenheimer, P.E., Committee Member — Date
Professor of Civil and Environmental Enginering, NJIT

BIOGRAPHICAL SKETCH

Author: Upendra L. Karna, P.E.

Degree: Degree of Engineer in Civil Engineering

Date: May 2001

Undergraduate and Graduate Education:

- Degree of Engineer in Civil Engineering
  New Jersey Institute of Technology, Newark, NJ, May 2001

- Master of Science in Civil Engineering
  Southern Illinois University, Carbondale, IL, 1988

- Bachelor of Engineering in Civil Engineering
  Rajasthan University, Jaipur, India, 1980

Major: Geotechnical Engineering

Presentations and Publications:

"Negative Skin Friction on Coated and Uncoated Model Piles",
Fourth International Conference on Piling and Deep Foundations, Stresa, Italy, April 1991.

Professional Registration:

Licensed Professional Engineer in the States of New Jersey, New York, Pennsylvania and Delaware.

This Degree of Engineer Project
is dedicated to my Parents,
my Wife, Rekha and my Daughter, Uditi

ACKNOWLEDGMENT

I would like to express my sincere appreciation to Dr. Dorairaja Raghu for accepting my request to guide me as an advisor for this study. His help and encouragement for completing this research project is deeply appreciated. I would also like to extend my thanks to Dr. John R. Schuring, P.E. and Prof. Edward G. Dauenheimer, P.E. for serving as committee members for this research project. The graduate school and the Civil and Environmental Engineering Department at the New Jersey Institute of Technology are recognized for their support.

Special thanks are attributed to Arora and Associates, P.C., my employer, for allowing me to have access to various data and reports on record that are utilized for this research work. The help of Arora and Associates, P.C. and their many staff members for assistance in preparing this document is acknowledged and appreciated.

TABLE OF CONTENTS

TABLE OF CONTENTS
(Continued)

TABLE OF CONTENTS
(Continued)

LIST OF TABLES

LIST OF FIGURES

LIST OF FIGURES
(Continued)

LIST OF FIGURES
(Continued)

LIST OF ABBREVIATIONS AND SYMBOLS

AASHTO	-	American Association of State Highway and Transportation officials
Alpha (∝)	-	Adhesion Factor
ASTM	-	American Society for Testing and Materials
BBF	-	Blows per foot
BOR	-	Beginning of Restrike
CAPWAP	-	Case Pile Wave Analysis Program
CPT	-	Cone Penetration Test
CSX	-	Stress Developed in Pile
dB(A)	-	Unit of Sound
DR	-	Relative Density
EB	-	Eastbound
EOD	-	End of Drive
EMX	-	Energy Developed Hammer
FHWA	-	Federal Highway Administration
FPDS	-	Dynamic Test Equipment
g	-	Acceleration due to Gravity
HASP	-	Health and Safety Plan
Ice	-	Internal Combustion Engine
I.D.	-	Internal diameter
Ini Dr	-	Initial Drive
J_{skin}	-	Damping value along Skin Friction

LIST OF ABBREVIATIONS AND SYMBOLS
(Continued)

J_{toe} - Damping value at Toe

NAVFAC - Naval Facility Command

NB - Northbound

NCEER - National Center for Earthquake Engineering Research

NEC - North East Corridor

NJDOT - New Jersey Department of Transportation

PDA - Pile Driving Analyzer

PID - Photo Ionization Detector

pH - Hydrogen Ion Concentration

PPM - Part per Million

Q_{skin} - Quake value along Skin Friction

Q_{toe} - Quake value at Toe

RMX - Maximum Case Method Equation

R_{skin} - Soil Resistance due to Skin Friction

RSP - Standard Case Method Equation

RT - Route

R_{toe} - Soil Resistance at Toe

R_{ult} - Ultimate Soil Resistance

SB - Southbound

SPT - Standard Penetration Test

SPT N - Standard Penetration Test blow count per foot

LIST OF ABBREVIATIONS AND SYMBOLS
(Continued)

TNUWAVE - Dynamic Test Equipment

USA - United States of America

VOC - Volatile Organic Compounds

WB - Westbound

WEAP - Work Energy Application Program

WK - Week

CHAPTER 1

INTRODUCTION

1.1 Introduction

This research deals with the study of time dependent pile capacity characteristics in glacial deposits. In order to study the characteristics of time dependent pile capacity in glacial deposits, pile driving data from a major project "NJ Route 21, Newark Viaduct Replacement" located in the city of Newark, Essex County, New Jersey was used. The project planning and design considerations are briefly discussed. Observations and findings regarding pile capacity with time are studied and analyzed in detail. The whole project was divided in three construction contracts: Advanced Contract, Contract A and Contract B (Contract B & C were combined and later termed as Contract B). At present the Advanced Contract and Contract A foundations have completed. This study is based on the findings associated with pile load test data from the Advanced Contract as well as Contract A. Project key and location plan are presented as Figure 1.1 and Figure 1.2, respectively on the following pages.

The existing NJ Route 21 viaduct connects US Route 1 & 9 and US Route 22 with Broad Street in the city of Newark. Route 1 passes over Route I-78, Amtrak's Northeast Corridor electrified tracks and Contrail's Lehigh high level tracks. The viaduct consists of four lanes with a total width of 51.2 feet, and was constructed in 1932 and rehabilitated in 1967. The existing viaduct replacement is due to insufficient traffic capacity, lack of direct connections with Route I-78, structural deficiencies and substandard features.

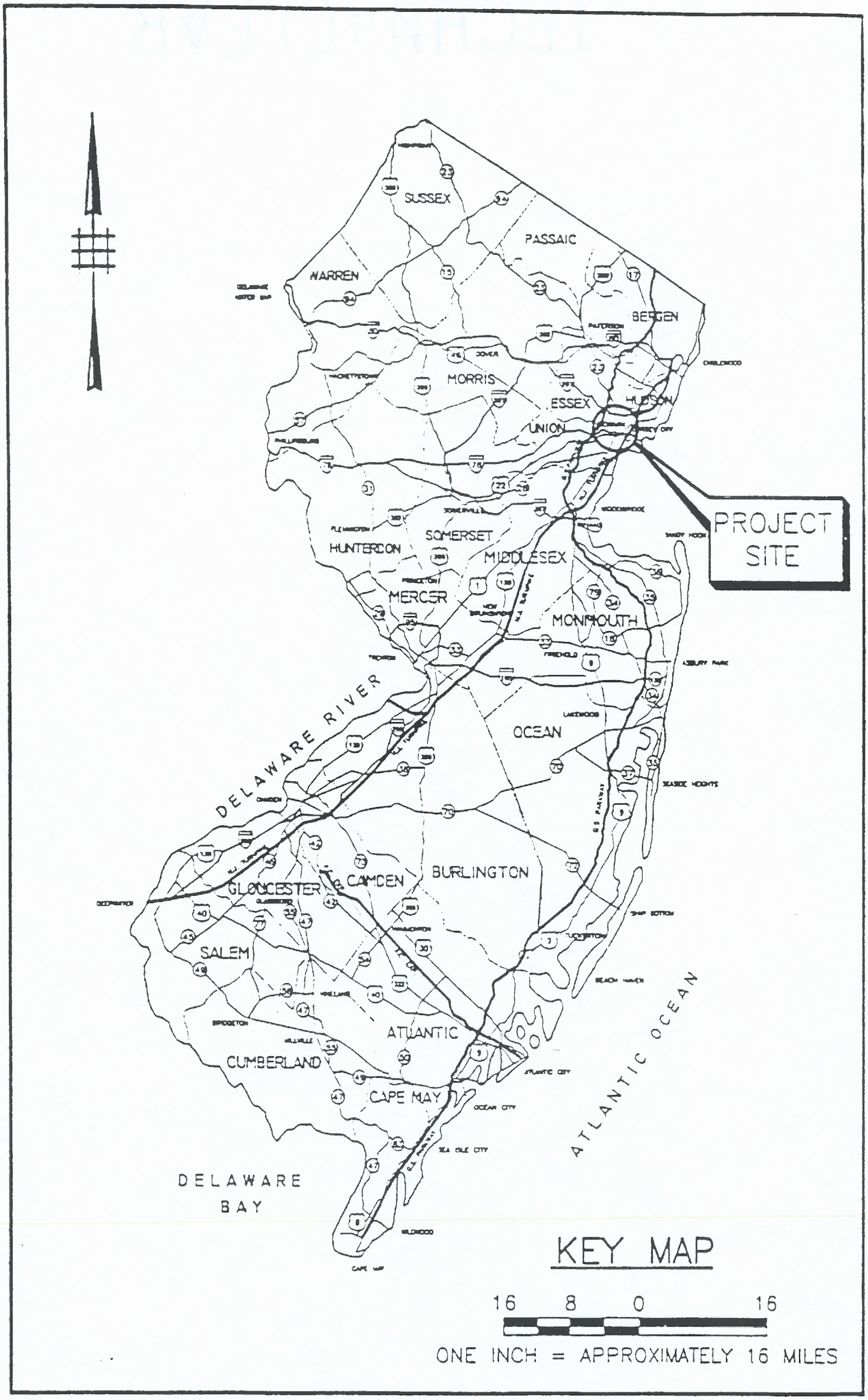


FIGURE 1.1 Key Map

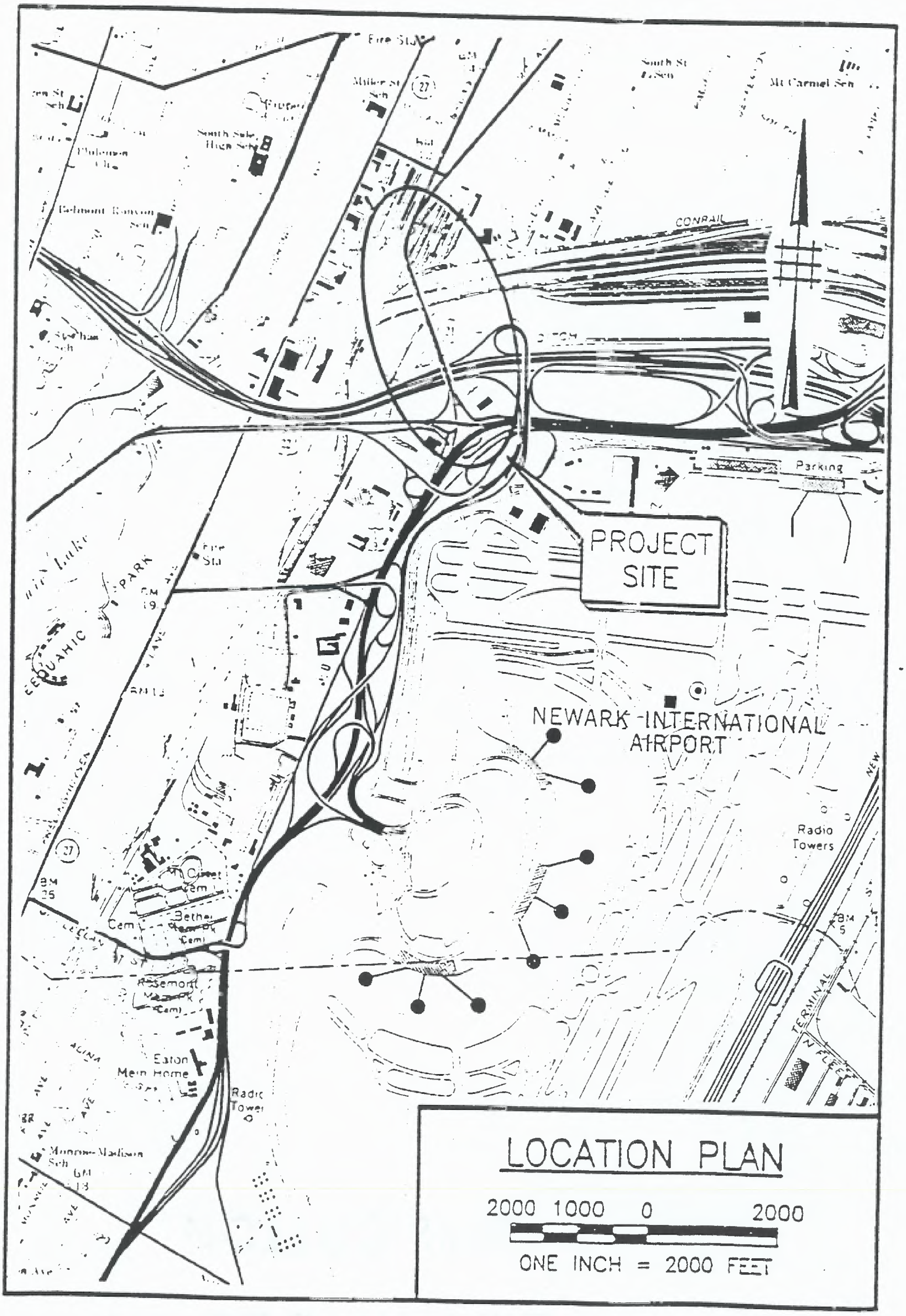


FIGURE 1.2 Location Plan

The proposed alignment for the NJ Route 21 viaduct was selected west of the existing viaduct after studying four different alignment alternatives. Ten ramps have been proposed to connect the viaduct with Route I-78, US Route 1 & 9 and US Route 22 at its southern terminus. Northern terminus of the viaduct will tie into an existing section of NJ Route 21 which is referred to as McCarter Highway. There will also be three ramps in this area which will connect the viaduct with Broad Street and Poinier Street. The proposed viaduct will be a divided roadway with three lanes, an auxiliary lane and a shoulder in each direction, with an overall width of 123 feet.

The construction Contract for the advanced Contract was executed in 1997 which consisted of the foundations for 6 piers, 2 at Route 21 and 4 at I-78 widening. Areas covered by the construction Contract A are mostly of the Section of Route 21 Viaduct from Sta. 10+32.50 to Sta. 28+35, Ramp 11, Ramp 8 and I-78 widening. A total of 63 pier foundations are located in construction Contract A area. The total cost of the project covered by this study is about $100 Million. The construction Contract Plan is presented in Figure 1.3 (see page 5).

Subsurface investigation for this project was investigated by Standard Penetration Test (SPT) borings. The area is underlain by glacial deposit. The grain size and the density of the deposited materials are nonuniform within the studied area.

Closed end pipe piles (24 inch and 18 inch diameter) were recommended and utilized for the foundations. During the execution of the Advanced Contract, anticipated pile capacity was not mobilized within the estimated design depth at some locations as

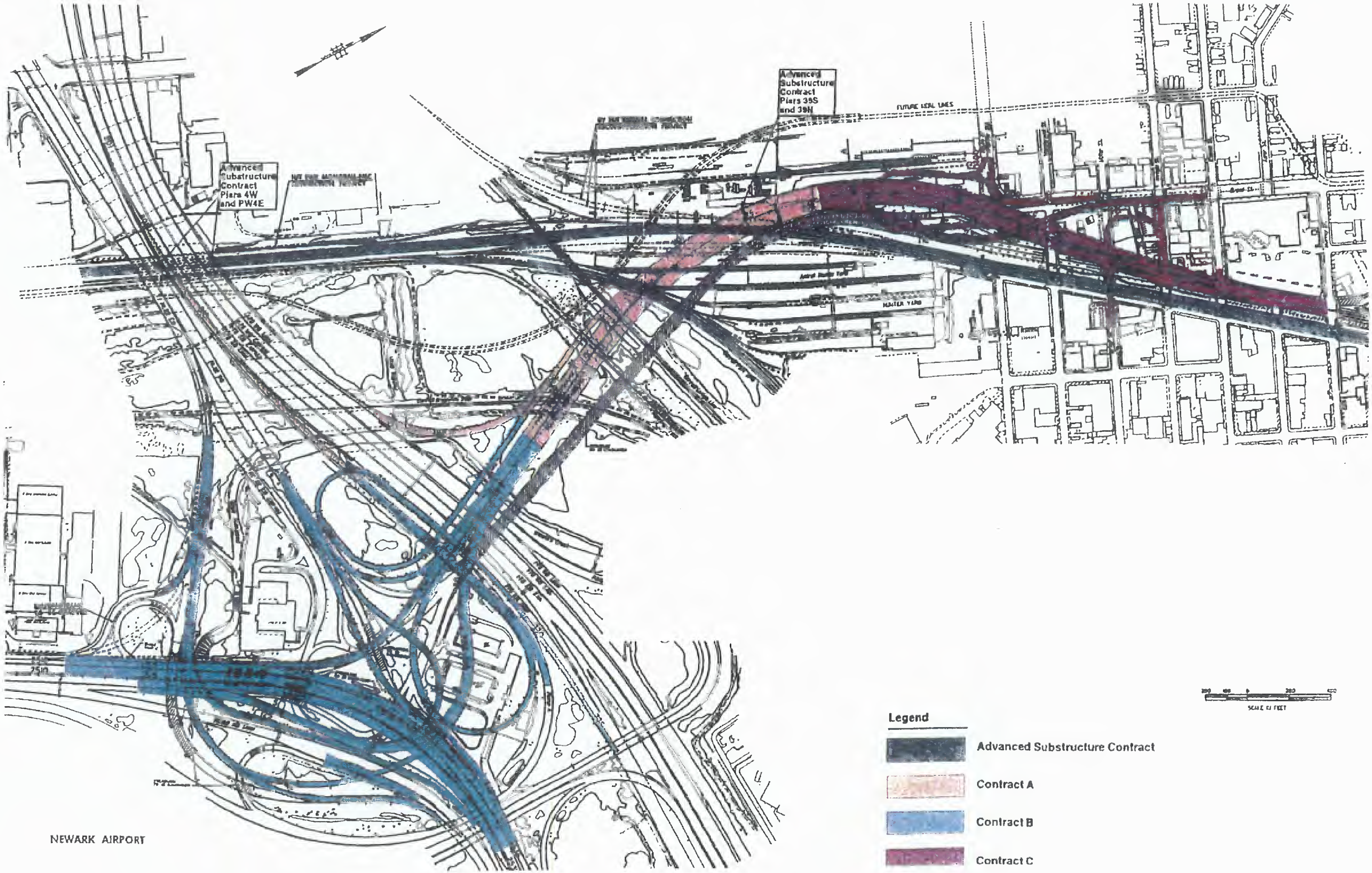


FIGURE 1.3 Construction Contract Plan

indicated by dynamic load testing. Based on the dynamic as well as static load tests conducted, it was realized that the site had very variable soil conditions and soil setup could occur. For the future contracts pilot load tests will be needed to evaluate the setup factor applicable for the site prior to developing production pile driving criteria.

During the execution (1998-1999) of construction Contract A, 112 dynamic load tests monitored by Pile Driving Analyzer (PDA) and applicable Case Pile Wave Analysis Programs (CAPWAP) were conducted. Some static load tests were also conducted. The following observations were made based on the results of the above tests.

1. Set-up, which is defined as the increase in load capacity of the driven pile after the initial driving, occurred. There were large variations in the set-up with significant variations occurring over a very short distance or even within a pile cap.
2. Mobilized skin friction with depth was not consistent even within the same soil type.
3. Length of pile required to mobilize the same ultimate capacity varied significantly even within a short distance.

Based on the above observations production pile driving criteria was established.

Primarily, this study focused on the understanding of the time dependent change in axial pile capacity in glacial deposits. The dynamic and static load test results of the studied area were gathered, evaluated and conclusions were arrived at.

In order to understand the pile capacity characteristics with time, entire area was divided into four soil types depending upon the subsurface soil conditions and are designated as Soil type 1 to Soil type 4. Results of load tests from a particular soil area were analyzed, evaluated and correlated with the subsurface conditions. Various unusual observations and the possible reasons for the anomaly are discussed.

It has been well reported that in a certain type of soils the pile capacity increases with time. In order to study the cause of increase in capacity with time and the variations in the mobilized capacity, a thorough literature review was conducted. Pertinent related articles were gathered, studied and correlated with the observations from this study.

Important findings and the conclusions related with this study have been extracted from these articles. Wherever possible, the observations similar to the article's findings are also discussed. Based on the load test results, it is concluded that the pile capacity will change with time for a glacial deposit. It is recommended that the setup factor should be considered for the future projects in this area for the similar soil conditions. Relevant shortcomings of this study are also discussed. Further study for the subject topic is also recommended to narrow the generalization of the time dependent pile capacity behavioral change in glacial deposits. This should be helpful in evaluating the axial pile capacity in similar subsurface condition.

1.2 Background

Planning for upgrading Route 21 through the city of Newark began in the early 1960's. In the 1972 study titled the "Route 21 Newark Planning Report", the City of Newark recommended (8) the upgrading of Route 21 between Routes I-78 and I-280 to freeway standards.

Based on the recommendation of this study (8), the New Jersey Department of Transportation commissioned the "Newark Highway Access Feasibility Study". This study examined short and long range alternative highway and street improvements within the primary access corridors to Newark. It also examined the feasibility of replacing the existing obsolete and deteriorated viaduct with a modern facility designed for current and anticipated future traffic capacities.

Michael Baker Jr., Inc. (9) was involved since mid 1980's to evaluate the best alternative route and the design of the project. In order to accommodate the projected unconstrained traffic volume, five to six lanes in each direction was intended for the Route 21 viaduct area. However, in order to be consistent with the Route 21 upgrade study of the northern section of Route 21, it was decided to provide only three lanes in each direction for the viaduct portion.

After three to four years of study, Baker prepared an "Engineering Feasibility Study" Report in 1990. After evaluating technical, social, economic and environmental impacts, it was concluded that the viaduct is to be replaced with a new structure. The new alignment should be along and to the west of existing facility with a full interchange with Route I-78.

The proposed viaduct will span I-78, a Conrail Yard, and Amtrak Northeast Corridor Main Line. Direct ramp connections between I-78 and Route 21 viaduct will increase the accessibility into the city of Newark, and distribute traffic more effectively. This direct connection between I-78 and Route 21 would eliminate the need for traffic to use the Hillside Avenue and Turner Boulevard exit ramps from I-78 to access Newark.

Improvements to the Route 21 viaduct facility are essential to the city of Newark's future vitality because of its great importance as a direct link to City's business, commercial and industrial districts. Elimination of the structural deficiencies and substandard geometry of the existing viaduct will also improve public safety.

The proposed west side alternative was developed to meet design criteria to limit the impact on the existing buildings, to provide a smooth transition between the proposed viaduct and the existing interchanges and to minimize the traffic congestion during construction. At the north end of the project, the entire block bounded by Route 21, Broad Street, Poinier Street and Vanderpool Street would be improved with a six lane roadway along Route 21.

1.3 Organization of this Report

This research report discusses the subsurface conditions and various factors that influence the effect of time dependent increase in the load capacity of piles from full scale load test data. The study is primarily based on subsurface conditions where the soil formations are predominantly glacial deposits.

In Chapter 1, introduction and background of the problem are presented. Project description is in Chapter 2 which also contains subsurface conditions. The test pile program is described in Chapter 3. Literature review is discussed in chapter 4. This is followed by a discussion of load test results in Chapter 5. Production pile installation criteria is presented in Chapter 6. Conclusions and recommendations for further study are discussed in

Chapter 7. Supplementary tables, figures and laboratory test results along with a sample static pile capacity analysis can be found in the Appendix A, B, & C.

CHAPTER 2

DESCRIPTION OF THE PROJECT

2.1 General

The northern portion of the project passes through general flat land. This area consists mainly of small buildings, warehouses and local streets. The middle portion of the project traverses through Amtrak and Conrail railway yards. The southern portion of the project lies within a tidal marsh area.

The proposed project includes the replacement of the existing Route 21 viaduct with a new structure, connection to and widening of I-78 in each direction, realignment of existing ramps and connection to Route 1 & 9, Route 22 and the local streets in the City of Newark.

The entire Route 21 viaduct replacement project is divided into three contracts: Advanced Contract, Contract A, and Contract B. Contract A predominantly consists of the Route 21 viaduct from the northern portion to the I-78, the connection ramps to I-78 and the widening of Route I-78 in each direction. The remaining Route 21 viaduct, ramps and street improvements are covered in Contract B. The Advanced Contract consists of the construction of six piers near the New Jersey Transit line project, and it was completed in 1998. Contract A began in 1998 and Contract B in 2000 and the work is now in progress. Most of the construction of Contract A is finished. This report is based on the field test results of the foundation construction of Advanced Contract and Contract A.

2.2 Site Geology

The project site is located within the Piedmont Plain Geological Province of New Jersey.

The geological formation predominantly consists of sedimentary rocks overlain by non-residual materials deposited by the Wisconsin glacier. As discussed in the Engineering Soil Survey Report (59), the deposit is identified as marginal morainic till and stratified drift. The upper portion is predominantly glacial-lacustrine soils deposit.

General characteristics of the deposited material is assorted, relatively homogenous predominantly of sand-sized grains with varying amounts of silt and gravel. Gravel often occurs as layers or beds of varying thickness. Mineralogical composition includes predominantly shale, sandstone and gneiss particles. The color is predominantly red-brown derived from shale siltstone and sandstone.

The underlying rock formation is predominantly sedimentary. The depth to bedrock formation varies from 80 feet to 200 feet. The bedrock formation is believed to be of Pre-cambrian and Paleozoic age.

2.3 Subsurface Investigation

In order to evaluate the subsurface conditions along the proposed structures and the ramps, a subsurface investigation program was conducted in 1993. The investigation program utilized the existing borings of six different series, conducted from 1965 to 1990, in this area. Most of the existing borings were performed along the Route I-78. Most recent set of borings (1990 Series) were performed in order to explore the suitability of the Route 21 alignment selection. A boring location plan containing all borings performed for this study area is presented in Figure 2.1 (see page 13). These borings were performed utilizing

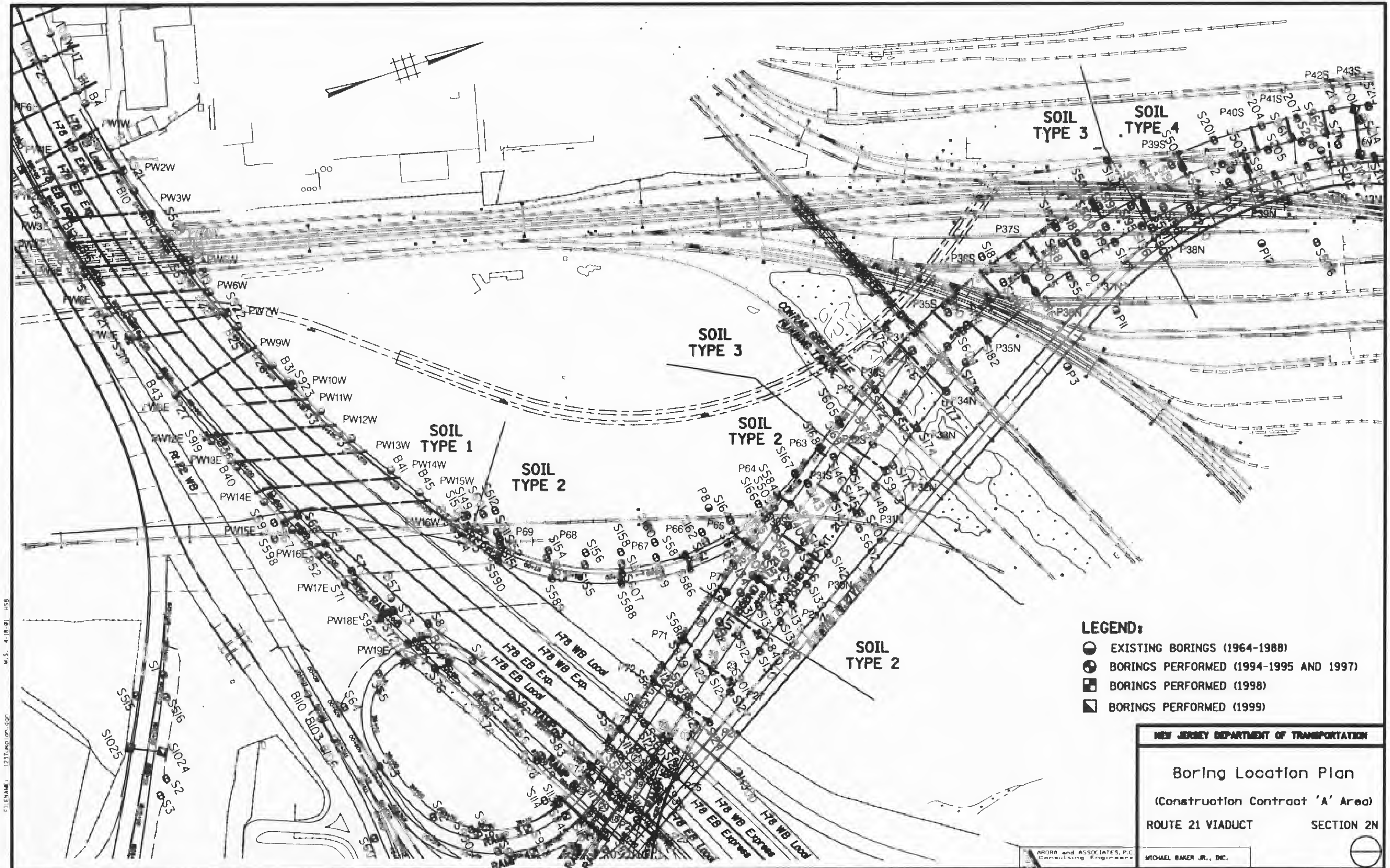


FIGURE 2.1 Boring Location Plan

driven casings with or without drilling mud. Depth of these borings ranged from 26.5 to 123.5 feet.

Subsequently, another subsurface exploration program was planned to investigate the subsurface conditions along the proposed alignment of the Route 21 viaduct and connecting ramps. The exploration program was designed based on the general guidelines prepared by the New Jersey Department of Transportation, and also considered the criteria developed by the American Association of State Highway and Transportation Officials (AASHTO). The depth of the proposed borings was selected based on the preliminary anticipated loads on structures.

Initial field exploration program consisted of 245 borings. During boring operations, due to change in alignment, 48 more borings were added. These borings were performed during October 1994 to March 1995. During the design process some structures were added and alignment of some ramps were modified, and as a result these 25 additional borings were performed, prior to preparing the final Geotechnical Engineering Report. These additional borings were performed during January 1997 to March 1997. Some additional borings were performed during late 1997 and 1998 to explore the subsurface conditions at a greater depth. Depth of borings ranged from 11.5 to 211 feet.

For these investigations, standard penetration test (SPT) borings were performed. SPT boring is a widely used method of subsurface exploration in the USA. Samples obtained during the exploration can be visually identified and further testing can be performed to obtain the true soil properties. The SPT borings utilized four inch nominal I.D. hollow stem auger with or without drilling mud. All disturbed samples were recovered utilizing one and one-half inch nominal I.D. split spoon sampler driven by a 140 pound hammer freely falling thirty inches. The disturbed and undisturbed samples were collected in accordance with NJDOT subsurface exploration criteria at 5 feet intervals. Samples were visually identified in the field according to the Burmister System of soil classification. Representative disturbed and undisturbed samples were labeled and preserved for future testing and identification.

Groundwater table was measured during the boring operation and 24 hours after the boring operation. In order to establish the long term groundwater table, eleven observation wells were installed. In order to evaluate the true hydrostatic pressure, two Casagrande type piezometers at possible drilled shaft locations were also installed.

Environmental Monitoring: The initial environmental screening of the area suspected the site to be contaminated with hazardous materials. Therefore, the field exploration program was planned to be conducted with contaminated waste Level C and D^+ protection. All monitoring was performed in accordance with procedures outlined in the site-specific Health and Safety Plan (HASP) prepared for the project. Personal air monitoring for volatile organic compounds (VOC) was conducted continuously during all intrusive activity using a photo ionization detector (PID). This monitoring was utilized as a method for determining personal protection work levels and waste management protocol. A 50-foot perimeter of each work zone was monitored for total dust particles and VOC. Miniram and PID readings were collected hourly throughout the workday. The oxygen combustible gas meter was at site to monitor fire hazard. Area noise was monitored by noise dosimeter, the limiting value of noise protection was 85 dB(A).

Soils collected in the vicinity of the underground oil storage tank indicated the presence of petroleum products and low level of VOC. However, monitoring instrument (Hnu) levels did not exceed pre-determined site action limits as stated in the HASP. No evidence of contamination was detected from drilling wastes generated.

Approximately 180 cubic yards of soil and 2,500 gallons of liquid waste (generated from decontamination activities) were collected in drums for further laboratory analysis. No hazardous contamination was detected. As a result, these samples were removed from the designated waste management yard as non-hazardous material. It was decided that the future construction activities would not require hazardous waste protection protocol.

2.4 Subsurface Conditions

2.4.1 General

This section describes the general subsurface conditions at the Contract A site of the Route 21 viaduct project, as determined from the subsurface investigations. General brief description of each stratum is presented.

General stratigraphy within the Contract A area and in general the Route 21 project is complex. Significant variations in layer thickness, boring penetration resistance, and composition of grain size distribution are common. Boundaries between strata are not clearly defined in many cases and considerable inter-layering of various glacial deposits are observed. By geological account, this type of heterogeneity is typical in the environment of terminal moraine deposits. The site is located at or near the terminal moraine and the deposits have been complicated with the presence of an ancient channel.

Within the project limits, the groundwater table varied significantly. At the southern end of the project, the groundwater table varied from 3 to 25 feet from the existing ground surface. At the north end of the project, the groundwater table was rather constant at about 8 to 10 feet from the existing ground. In the middle portion of the Route 21 viaduct, the groundwater table was encountered at shallow depths due to the presence of wetlands.

The subsurface investigation indicates that the project site is underlain by five distinct strata:

1) Surface fill
2) Organic Silt & peat (Meadow material)
3) Glacial deposits - ranged from cohesive to granular and mixture of both
4) Decomposed rock
5) Bedrock

Specific differences in various areas will be discussed. A general description of each stratum in descending order as encountered in Contract A is presented below.

Surface Fill

A surface fill layer was encountered at most of the borings. The fill consists of heterogenous mixture of sand, gravel, clay, silt, cinders, wood and other foreign materials. Occasionally a boulder was also encountered. It is believed that this layer was spread over a long period of time by the residue from the surrounding construction activities. The thickness of fill layer ranged between 5 to 15 feet over the Contract A area. SPT blow counts varied from 2 to 90 blows per foot which indicates that the layer is in very loose to very dense condition.

Organic Silt and Peat (Meadow Material)

Meadow material was encountered underneath the fill layer. This stratum consists of gray to black and dark brown, soft to medium stiff organic silt and clay, occasionally with peat. This layer ranged in thickness from 5 to 20 feet. Occasionally this layer was intermixed with the fill layer resulting high blow count. In general, the SPT blow counts ranged from 2 to 38 blows per foot (bpf).

Glacial Deposits

Underlying the meadow material deposit, a thick layer of glacial deposit was encountered. There are two types of glacial deposits encountered: glacial lacustrine and glacial strataified drift. The glacial deposits are very complex and vary significantly in both composition and consistency across the Contract A area. Stratifying these materials is difficult. This is the deposit which affects the pile driving behavior of the project.

In general, a glacial lake deposit which is formed by the alluvial process is encountered underlying the meadow material. This deposit is over-consolidated and occasionally varved,

consisting typically of soft to medium stiff silts, clayey silts and clay, or medium dense to dense fine sand with silt, or an irregular inter-layering of both soil materials. Trace amounts of fine gravel was encountered in some of the borings. This glacial lake deposit, encountered across the Contract A area varies in thickness from 50 to 100 feet.

Underlying the glacial lacustrine deposit, a glacial stratified drift and till deposit was encountered. At the northern portion lacustrine deposit is underlain by till and stratified drift deposit. Whereas at southern portion, lacustrine deposit is underlain by a thin layer of till over stratified drift deposit. Depending upon the location, till deposit consists of hard sandy silt or very dense to medium dense fine to medium silty sand with gravels. At the northern portion of the Contract A area, a significant amount of gravel particles with cobbles are present. The consistency and density inferred from the SPT values are often influenced by gravel content. Therefore, the SPT values may not be indicative of the density of soil at some locations. This layer varies in thickness across the site, from 5 to 65 feet thick at the southern portion, and from 80 to 120 feet at the northern portion of the project. At the souther portion, the lacustrine deposit is predominantly cohesive with lenses of sand indicating occasional fluvial activity within the glacier.

Decomposed Rock

The decomposed rock includes the material that has weathered to soil and partially intact with parent rock. This material is typically described as low plastic clayey silt to medium plastic silt and clay with fine sand and gravel. This description is for the material obtained after fracturing the intact layer present. The SPT N-values in this layer are greater than 100. This layer was not encountered in all borings. However, within the Contract A area the thickness of this layer varies from 3 to 5 feet.

Bedrock

The parent bedrock underlies the decomposed shale, siltstone and sandstone bedrock. In general, the bedrock can be described as red brown soft to medium hard sandy siltstone, shale, and sandstone. The bedrock surface dips down towards the north. Along the south portion of the Contract A area, the depth of bedrock varies between 100 to 150 feet from the ground surface, whereas at the north portion of the viaduct it varies from 150 to 215 feet from the ground surface.

2.4.2 Detailed Stratigraphy

In order to facilitate the study of pile capacity behavior, the entire Contract A area has been divided into four areas where soil conditions and pile driving behavior were observed to be generally similar as follows:

Soil type 1: I-78, EB & WB

Soil type 2: Ramp 8 and Ramp 11, Route 21, NB & SB near Ramp 11 (Piers 29 to 32)

Soil type 3: Route 21, NB & SB (Piers 33 to 38)

Soil type 4: Route 21, NB & SB (Piers 39 to 42)

The regions covered by these soil types are shown in Figure 2.1 on page 13. An abstract of these subsurface conditions has been prepared and presented herewith as Figure 2.2 on page 21. The ground surface elevations presented in this profile and elsewhere in this report are based on National Geodetic Vertical Datum 1929.

Soil type 1

The subsurface condition in this area consists of a thin layer of surface fill and meadow material overlying glacial deposits. These deposits are quite variable in both composition and consistency. The thickness of the glacial deposit varies from 80 to 125 feet. The upper 60 to 100 feet layer is typically a glacial lacustrine deposit which can be described as predominantly medium to dense coarse to fine sand with silt and gravel. Occasionally a cohesive layer of medium to hard consistency from 5 to 10 feet thick was also encountered in some borings.

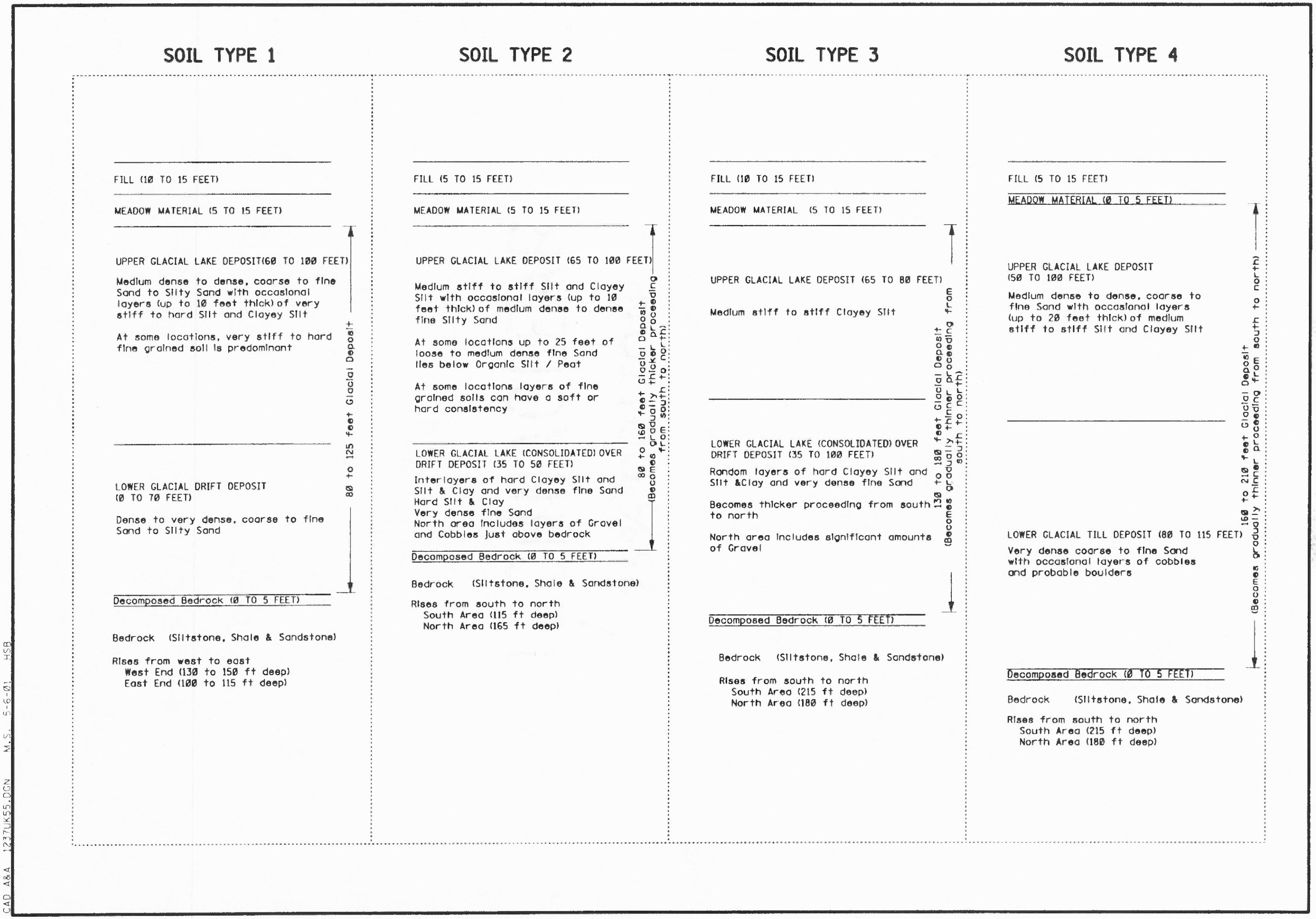


FIGURE 2.2 Abstract of Subsurface Conditions

CAD A&A 1237UK55.DGN M.S. 5-6-01 HSB

The lower portion of the glacial deposit in this area is a relatively thin layer of very dense coarse to fine sand, trace silt with gravel. This lower deposit contained more gravel materials. Silt and clay are generally absent in this deposit. When encountered, the thickness of the layer varied from 10 to 50 feet.

Underlying the glacial fill deposit, a bedrock formation is present. The depth to bedrock along the east side is about 100 to 110 feet, whereas along the west it varies from 130 to 150 feet.

Soil type 2

The general stratigraphy in this area consists of a thin layer of fill and meadow material over a thick glacial deposit. The upper glacial deposit contains a significant amount of cohesive materials. At the south end of the glacial deposit, the thickness ranges from 80 to 100 feet, whereas towards the north, it increases to about 160 feet.

The upper 60 to 100 feet of the glacial deposit is fairly consistent in composition and can be described as mainly a medium stiff to stiff clayey silt with fine sand. Occasionally a trace of fine gravel is also encountered. Mostly the layer is medium stiff, however, at some location, soft and hard layers are also present. Occasionally, a thin layer of fine sand is also present in this upper glacial deposit.

The lower portion of glacial deposit (glacial drift) is about 30 to 50 feet thick and consists of inter-layers of hard medium plastic cohesive soils with fine sand to very dense fine sand with clayey silt. A distinct layer of either hard silt and clay or very dense fine sand is also encountered. Some borings indicate the presence of about 15 feet thick layer of glacial stratified drift deposits containing mainly gravel and boulders just above the bedrock.

At the south end, the bedrock is at about 115 feet from the ground surface, whereas at the north end it increases to a depth of 160 feet. The depth of the lower dense glacial drift deposits follow a similar trend, being deeper at the north end (at about 100 to 130 feet) and shallower at the sough (60 to 80 feet).

<u>Soil type 3</u>

The subsurface condition in this area consists of a thin layer of fill and meadow material over a thick layer of glacial deposit. The thickness of the glacial deposit increases towards the north. At the south end, the thickness of glacial deposit is about 125 feet, whereas at the north end, the thickness is about 175 feet.

The upper glacial deposit (glacial lacustrine) is about 60 to 80 feet thick, and consists of medium stiff to stiff clayey silt with trace of fine sand and gravel. In this layer the material composition is consistent, however, the consistency varies significantly. An occasional layer of medium dense to dense fine sand layer is also present.

The lower portion of the glacial deposit is quite variable in thickness and composition. The thickness increases towards the north. At the south end the thickness is about 30 to 50 feet, whereas towards the north end it increases to about 100 feet. Random layers of stiff to hard silt and clay and dense to very dense fine silty sand with large amounts of gravel are encountered in this deposit. The bedrock surface drops significantly towards the north. In this area, the bedrock is encountered at 150 to 200 feet. The depth to the top of the very dense lower glacial deposit remains somewhat consistent at about 100 to 110 feet in this area.

Soil type 4

Similar to other areas, under a thin layer of surface fill and meadow material deposit, a thick layer of glacial deposit is present in this area. The meadow material thickness is small in this area and even non-existent at the north end of the area. The glacial deposits are very different than those encountered at other areas of the site. The thickness is greatest in this area varying from 160 to 210 feet.

The upper 50 to 100 feet of this glacial deposit typically consists of medium dense to dense medium to fine sand with some silt and occasionally a trace of fine gravel. Occasional layers of medium stiff to stiff silt and clay layer are present.

With depth, this glacial deposit contains a significant amount of gravel. This lower glacial deposit can be described as a glacial till deposit containing mainly coarse to fine sand with silt and gravel. The thickness ranges from 80 to 110 feet. The SPT N-values range from 50 to over 100 indicating a very dense consistency. It is interesting to note that the fine grained cohesive materials are absent in this layer. Occasional pockets of cobbles and big boulders are also present in this layer.

The depth of bedrock increases from south to north. In this area, the bedrock is encountered at 180 to 215 feet. The depth to the very dense lower glacial deposit is about 100 to 110 feet at the south end and about 70 feet at the north end.

2.4.3 Laboratory Testing

Laboratory tests were performed on selected disturbed and undisturbed soil samples retrieved during the soil boring operation. The laboratory testing program consisted of index property testing as well as performance testing and was conducted according to applicable ASTM standards. Index property testing included visual identification, natural unit weight, natural moisture content, grain size distribution, hydrometer, specific gravity, Atterberg limits, organic content, pH and resistivity. Performance testing included one dimensional consolidation, unconfined compression, triaxial shear (unconsolidated undrained and consolidated undrained), direct shear and permeability. In addition, pocket penetrometer tests were performed on various disturbed cohesive soil samples during boring operations. During boring operations, pH values were measured utilizing a field pH meter.

2.5 Geotechnical Analysis and Evaluation

2.5.1 General

The geotechnical analysis considered various deep and shallow foundation schemes. In order to evaluate the most appropriate foundation schemes, various studies and analysis were conducted which were critical for the selection of a foundation scheme.

2.5.2 Soil Parameters

Soil parameters for design were developed by utilizing the laboratory test results and standard penetration tests and are presented in Table 2.1. The geotechnical properties of soils at the northern portion of the project are different from the soils at the southern portion of the project.

The organic content of the meadow material deposits in the northern portion of the project varied from 10.9% to 56.6% with an average organic content of 25%. Similarly, the organic content of the meadow material deposits in the southern portion of the project varied from 10.3% to 78.9% with an average organic content of 35%. The plasticity index of the meadow material deposits in the northern and southern portions varied from 11 to 125 with an average of 53 and from 5 to 75 with an average of 37, respectively. Based on laboratory test results, it was believed that the meadow material deposit in both the northern and southern portions of the project is normally consolidated. At some locations the cohesive soils appeared to be overconsolidated.

TABLE 2.1

Recommended Soil Properties

PROPERTY	TYPE OF SOIL LAYERS					
	Fill	Meadow Material	Cohesionless Alluvial Deposit	Cohesive Alluvial Deposit	Fine Grained Glacial Deposit	Coarse Grained Glacial Deposit
Angle of Internal Friction (φ)	32°	18°	30°	20°	34°	38°
Cohesion (psf)	-	400	-	450	-	-
Total Unit Weight (pcf)	120	100	115	115	125	130
Submerged Unit Weight (pcf)	58	40	53	53	63	68
Specific Gravity of Solids	2.65	2.55	2.65	2.6	2.7	2.75
Coefficient of Base Friction	0.45	0.4	0.45	0.4	0.5	0.55
Horizontal Soil Modulus (pci)	25	20	20	30	125	125

On average, the cohesive alluvial deposits in the northern and southern portions of the project did not contain any gravel. The cohesive alluvial deposits in the northern portion contained 6% sand, 64% silt and 30% clay while the cohesive alluvial deposits in the southern portion contained 7% Sand, 52% silt and 41% clay. The plasticity index of the alluvial cohesive deposits in the northern and southern portions varied from 1 to 16 with an average of 7 and from 3 to 17 with an average of 9, respectively. Based on laboratory test results, it is believed that the alluvial cohesive deposits in both the northern and southern portions of the project are normally consolidated. The low plastic cohesive layer was observed to be varved and highly sensitive.

Non-plastic glacio-lacustrine deposits contained a significant amount of fine cohesive materials throughout the project site. The glacial deposits at the northern portion of the project contained more gravel particles than the southern portion of the project.

The soil properties shown in Table 2.1 were used for the design of the project. Properties provided were based upon field and laboratory test results and various correlations and engineering judgement.

In accordance with AASHTO criteria, the section of the project located north of Route I-78 should be categorized as Type II for seismic design whereas the section of the project located south of Route I-78 should be categorized as Type I for seismic design.

Shallow foundation calculations considered the properties of soils which were encountered within a depth equal to twice the width of the footing. The properties of soils encountered at greater depths were used for deep foundation calculations.

2.5.3 Liquefaction Potential

Based on the general soil formations, it was suspected that the underlying loose sand layers could be susceptible to liquefaction. A loose, relatively fine sand below the groundwater table is susceptible to liquefaction. In general, it has been observed that natural sand deposits having a relative density (D_R) lower than 0.60 are vulnerable to liquefaction. Case histories indicate that liquefaction has occurred within a depth of 50 feet or less. Therefore, a maximum depth of 50 feet was considered in the liquefaction analyses for this project.

Liquefaction will depend on the extent to which the necessary hydraulic gradient, which may induce a quick condition, is developed and maintained. This will depend upon the density of soils, grain size distribution, the nature of ground deformations, the soil permeability, the site geometry, and the duration of the induced vibrations.

Evaluation of liquefaction potential can be conducted either through laboratory testing on undisturbed specimens or using in situ test data. The undisturbed soil samples that were retrieved did not contain the amount required for performing a dynamic shear strength test. As a result, in situ test data utilizing N-values was used for the liquefaction analyses. A procedure based on SPT data is an approximate method since the results depend on the quality of the field test data. Based on the Seed and Idriss (45) method of liquefaction analysis (utilizing SPT data, ground acceleration and effective overburden pressure), the factor of safety against liquefaction ranged from 1.01 to 1.05. At a later stage based on the latest publication based on the modified Seed and Idriss method and the method (57) suggested by the National Center for Earthquake Engineering Research (NCEER), the factor of safety for liquefaction was determined to be greater than 1.28. It was concluded that the subsurface soils within the project limits were not susceptible to liquefaction.

In accordance with AASHTO (2), this project is located in a site within a maximum horizontal ground acceleration of 0.18 g. The following items support the above conclusion.

(i) A dense coarse to fine sand ranging 10 to 15 ft. in thickness was encountered near the ground surface. This dense layer will tend to prevent soil from moving upward during an earthquake.

(ii) The liquefaction susceptible layers consist of a significant amount of low to medium plastic materials. In general, low to medium plastic soils (Plasticity Index 3 to 15) are identified as Clayey Silt to Clay & Silt in the Burmister Soil Classification System. This type of material was encountered often throughout the project and was present within the layer considered for liquefaction susceptibility. The plastic soils will help to retard the generation of excess pore pressure, therefore, preventing a quick sand condition.

(iii) The horizontal resistance of sand during an earthquake is unlikely to be exceeded until a peak ground acceleration exceeds about 0.5 g.

(iv) Where a liquefiable soil layer is encountered between comparatively more permeable layers, free drainage is available to release the excess developed pore water pressure.

(v) Generally liquefaction is considered within a depth of 50 feet. A deep foundation system is thus less vulnerable to damage due to the earthquake.

(vi) Most soils which are considered liquefiable will densify during pile driving. Thus, a driven pile is more suitable in liquefiable soils.

(vii) It has been reported (3) that if the SPT blow count exceeds the numerical value of twice the depth in meters, liquefaction will not occur.

2.5.4 Corrosivity

Corrosion occurs because of small physical and/or chemical differences present in metals or in the environment. In addition to the oxidation process the rate of corrosion in soil is a function of soil resistivity, soil texture, pH value, presence of organic matter, bacterial content and cyclic periods of wetting and drying. The following items were considered when analyzing soil corrosivity:

(i) Fine, even-textured granular soils are less corrosive.

(ii) Soils of uniform composition, such as sand, are less corrosive than a mixture of soils.

(iii) Well aerated, loose soils are less corrosive than poorly aerated, heavy soils, such as clays.

(iv) Highly acidic soils are corrosive. Mildly acidic soils are less corrosive if undisturbed.

(v) The rate of corrosion is not as sensitive to pH as for alkaline soils.

(vi) The presence of cinders in fill may cause corrosion.

(vii) Highly organic soils are more corrosive.

(viii) Soils with anaerobic bacteria (usually heavy water-logged soils) are more corrosive.

(ix) Higher resistivity soils are less corrosive. A soil resistivity value less than 2000 ohm-cm is considered highly corrosive.

(x) Wet soils are usually more corrosive than dry soils.

(xi) Corrosion is usually not a problem in an undisturbed soil zone.

The above mentioned basic considerations give an indication that the project site soils may be corrosive. During the field investigation, various samples were tested for pH by utilizing a field pH meter. Representative disturbed samples were selected within the project limits and tested for electrical resistivity and pH. Utilizing these test results, it was concluded that the fill and meadow material organic soils were corrosive.

A corrosion expert, Ocean City Research Corp. of West Chester, PA, was contracted to review the existing SPT boring logs and laboratory test data. Ocean City Research Corp. performed additional field testing as well as laboratory testing.

The corrosion expert recommended to provide an additional 1/16 inch of steel thickness for galvanic corrosion protection throughout the project. He also recommended that supplemental corrosion control, beyond the additional 1/16 inch corrosion allowance, be provided for Pier Nos. 38S, 38N, 39S and 39N due to possible stray currents in this area. These piers are located along Amtrak's Northeast Corridor in the Waverly Yard area. This supplemental corrosion control consists of applying a barrier coating to the top 20 feet of pile at each pile location. The specific barrier coatings recommended were:

1. Carboline Bitumastic 300M (coal tar epoxy)

   Two coats to 16 mils total dry film thickness

2. Ameron Amercoat 351 (100 percent solids epoxy)

   Two coats to 20 mils total dry film thickness

2.5.5 Specific Considerations

The project area contained many constraints such as existing roadways, high speed railway lines and underground (84-inch and 66-inch diameter sewer lines along Ramp 11) and overhead utility crossings. Special considerations were to be given while selecting the type and construction methodology for the various project foundations.

At locations other than those mentioned above, it was believed that the pile driving could cause the upper loose to medium dense granular soil to settle. In addition, it was believed the underlying predominantly fine sand layer could develop more pore water pressure than it would be able to dissipate during pile driving. Due to this excess pore water pressure the surrounding soils may heave. Whereupon this heave may be followed within a few days by settlement. It was concluded that at this project site (except for foundation units nearby active tracks), there would not be any detrimental effects on the surrounding permanent structures due to subsidence or heave caused by pile driving. As a result, special remedial measures would not be required. However, settlements of the pier foundations nearby the active railway tracks were monitored.

2.6 Foundation Recommendations

2.6.1 General

Shallow as well as deep foundation schemes were considered for this project. Drilled shafts were also evaluated.

Subsurface investigation indicated the presence of fill mixed with foreign deleterious material near the ground surface, loose granular deposits and compressible organic cohesive layers within the influence depth of loading. The presence of these unsuitable materials varied from location to location. These materials were expected to yield intolerable total and differential settlements for shallow foundations. The allowable bearing capacity within the foundation depth was considered low, resulting in larger size footings and the possibility of a significant amount of overexcavation. Variable amounts of differential settlement would have resulted in secondary stresses in the superstructures and substructures, especially for the continuously supported structures. Considering all these factors, shallow foundations were not considered to be suitable for this project.

2.6.2 Foundation Selection Criteria

Based on the subsurface soil conditions, deep foundations were considered feasible for the project. Several alternative types of piles and drilled shafts were evaluated. In the process of selecting the most suitable pile type for this project, the following criterion were considered.

(i) Load carrying capacity

(ii) Constructability

(iii) Performance and Design Life

(iv) Availability

(v) Economic Analysis

(vi) Site constraints

(vii) Past Experience

Timber piles were not considered due to the estimated low load carrying capacity and a smaller length availability.

The advantages and disadvantages of the different types of piles are briefly summarized as follows:

Prestressed Concrete Piles

Advantages

- High design capacity
- Long life
- Moderate material cost

Disadvantages

- Very heavy, requiring special equipment and extra caution for handling
- Difficult driving
- Longer delivery time
- Splicing very difficult
- Difficult to cut
- Structural integrity at splices in question, specifically for heavy capacity piles
- Tensile structural capacity of the pile may be compromised for the seismic uplift condition

Steel H-Piles

Advantages

- Easy to install
- Long life in noncorrosive environment
- Easy to splice
- Readily available

Disadvantages

- Material cost
- Susceptible to corrosion
- Low load carrying capacity

Steel Pipe Piles (Concrete Filled)

Advantages

- High design capacity
- Long life in noncorrosive environment
- Easy to install
- Easy to splice
- Readily available
- Closed end piles displace more soil which enhances frictional capacity
- Due to soil displacement, the relative density of substrata is increased

Disadvantages

- Material cost
- Moderate difficulty in driving
- Susceptible to corrosion

An economic analysis was made. Prestressed concrete piles appeared to be the least expensive. However, due to difficulty in splicing, pile cut off (based on site conditions and construction restrictions) and structural strength deficiency, prestressed concrete piles were not considered suitable for the project. H-piles were not considered suitable due to low bearing capability. Concrete filled pipe piles were considered the most suitable for this project because they would allow the work to proceed more efficiently than the prestressed concrete piles.

In the Northeast Corridor main line crossing area, where the expected superimposed loads were maximum and where site constraints and accessibility impose special considerations, other deep foundations such as drilled shafts were also considered. To obtain the required load carrying capacity, drilled shafts of up to 180 feet length of various sizes were evaluated. Various sizes of drilled shafts were analyzed. Based on the standard design requirement and limited space available between tracks, 4 ft., 5.5 ft. or 12 ft. diameter drilled shafts appeared to be feasible. A separate economic analysis was performed specifically for this area. Based on this economic analysis, 24- inch diameter, concrete filled pipe piles are judged to be best suited for this location.

For the remaining sections of the project, the final selection between 18-inch and 24-inch diameter concrete filled steel pipe piles was made by the structural bridge engineer depending upon the load demands. From a pile driving point of view, a range of batter for the piles between 1 horizontal to 12 vertical (1H:12V) and 1 horizontal to 4 vertical (1H:4V) was utilized for the project.

The load carrying capacity for 18-inch and 24-inch diameter, concrete filled pipe piles are presented in Tables 2.2 to 2.4. The estimated settlements are presented in Tables 2.5 to 2.7. As seen from the Tables 2.2 through 2.4, the pile capacity is derived from both skin friction and bearing, skin friction component of the capacity varying in the range of 30% to 70% of the total capacity. Usually large capacity piles are bearing piles and are driven to refusal into bedrock. For this project, the piles are driven at least 10' or so into the lower glacial deposit and not into bedrock. The arrangement reduces pile lengths considerably, thereby releasing substantial cost savings. Lateral and vertical clearances in this project are very tight. Proximity of railroad tracks and the nearby viaduct places restriction on the intensity of vibrations during pile driving and construction time. These considerations can

Foundation Recommendation for Contract A

TABLE 2.2

North East Corridor (NEC) Main Line Crossing

Capacity for 24" Diam. Concrete Filled Steel Pipe Pile

LOCATION	ESTIMATED PILE LENGTH (FEET)	ULTIMATE CAPACITY (TONS)	SKIN FRICTION PERCENT	ALLOWABLE CAPACITY (TONS)	ALLOW. UPLIFT CAPACITY (TONS)	ALLOWABLE LATERAL LOAD CAP. (TONS)	ULTIMATE LATERAL LOAD CAP. (TONS)	ESTIMATED PILE TIP ELEVATION NB	SB
ROUTE 21 STA. 22+00 TO STA. 24+00	120	500	60%	220	70	7.5	22	-120±	-120±

NOTE: ESTIMATED PILE LENGTH BASED ON ASSUMED BOTTOM OF PIER FOOTING AT EL. 0.0

TABLE 2.3

NJ Route 21 Viaduct Excluding NEC Main Line Crossing

Capacity for 18" & 24" Diam. Concrete Filled Steel Pipe Pile

LOCATION	ESTIMATED PILE LENGTH (FEET)	ULTIMATE CAPACITY (TONS)		SKIN FRICTION PERCENT		ALLOWABLE CAPACITY (TONS)		ALLOWABLE UPLIFT CAPACITY (TONS)		ALLOWABLE LATERAL LOAD CAP. (TONS)		ULTIMATE LATERAL LOAD CAP. (TONS)	
ROUTE 21		18"	24"	18"	24"	18"	24"	18"	24"	18"	24"	18"	24"
STA. 8+40 TO STA. 11+50	125	315	516	70%	60%	130	220	50	55	4	6	11	16
STA. 11+50 TO STA. 14+40	125	292	500	65%	60%	110	200	45	60	4	6	11	16
STA. 14+40 TO STA. 18+90	120	282	500	70%	60%	110	215	45	60	4	6	11	16
STA. 18+90 TO STA. 22+00	125	276	484	70%	60%	110	200	40	55	4	6	11	16
STA. 26+50 TO STA. 27+50	105	288	477	70%	65%	120	200	50	75	6	8	14	20
STA. 27+50 TO STA. 30+00	100	294	515	65%	55%	120	215	45	65	4	6	11	16

NOTE: ESTIMATED PILE LENGTH ASSUMED BASED ON BOTTOM OF PIER FOOTING AT EL. 0.0

UKTABLE3A.WB3

Foundation Recommendation for Contract A

TABLE 2.4

NJ Route 21 Viaduct - Ramps 8 & 11 and Route I-78 Widening

Capacity for 18" & 24" Diam. Concrete Filled Pipe Piles

STRUCTURE	LOCATION	ESTIMATED PILE LENGTH (FEET)	ULTIMATE CAPACITY (TONS)		SKIN FRICTION PERCENT		ALLOWABLE CAPACITY (TONS) *		ALLOWABLE UPLIFT CAPACITY (TONS)		ALLOWABLE LATERAL LOAD CAP. (TONS)		ULTIMATE LATERAL LOAD CAP. (TONS)	
			18"	24"	18"	24"	18"	24"	18"	24"	18"	24"	18"	24"
RAMP 8	STA. 814+00 TO STA. 820+72	80	280	510	40%	30%	120	220	35	45	4	6	11	16
RAMP 11	STA. 1100+77 TO STA. 1112+00	110	240	440	60%	50%	100	185	40	60	4	6	11	14
RAMP 11	STA. 1112+00 TO STA. 1118+50	100	260	440	60%	55%	110	185	35	55	4	6	13	19
RT I-78	STA. 1118+50 TO STA. 1126+00	80	248	450	50%	40%	100	190	40	60	4	6	13	19
RT I-78	STA. 800+77 TO STA. 814+00 STA. 1126+00 TO STA. 1132+83	80	270	500	55%	45%	120	220	50	75	4	6	11	12

* ALLOWABLE PILE CAPACITY FOR RAMP 8 ABUT. - STA. 820+72 IS 95 TONS FOR 18" PILE AND 185 TONS FOR 24" PILE DUE TO NEGATIVE SKIN FRICTION.

NOTE: ESTIMATED PILE LENGTH BASED ON ASSUMED BOTTOM OF PIER FOOTING AT EL. 0.0

UKTABLE14A.WB3

Estimated Pile Settlement (in inches)

TABLE 2.5

North East Corridor (NEC) Main Line Crossing

LOCATION	ESTIMATED PILE LENGTH (FEET)	24" DIAM. CONCRETE FILLED STEEL PIPE PILE	
		SINGLE	GROUP
ROUTE 21 STA. 22+00 TO STA. 24+00	120	0.11	0.39

NOTE: ESTIMATED PILE LENGTH BASED ON ASSUMED BOTTOM OF PIER FOOTING AT EL. 0.0

TABLE 2.6

NJ Route 21 Viaduct Excluding NEC Main Line Crossing

LOCATION	ESTIMATED PILE LENGTH (FEET)	18" DIAM. CONCRETE FILLED STEEL PIPE PILE		24" DIAM. CONCRETE FILLED STEEL PIPE PILE	
		SINGLE	GROUP	SINGLE	GROUP
STA. 8+40 TO STA. 11+50	125	0.14	0.39	0.12	0.32
STA. 11+50 TO STA. 14+40	125	0.16	0.45	0.13	0.35
STA. 14+40 TO STA. 18+90	120	0.14	0.39	0.12	0.32
STA. 18+90 TO STA. 22+00	125	0.13	0.36	0.12	0.32
STA. 26+50 TO STA. 27+50	105	0.16	0.61	0.11	0.37
STA. 27+50 TO STA. 30+00	100	0.14	0.39	0.12	0.32

NOTE: ASSUMED PILE GROUPS WERE USED FOR GROUP SETTLEMENT CALCULATIONS
ESTIMATED PILE LENGTH BASED ON ASSUMED BOTTOM OF PIER FOOTING AT EL. 0.0

TABLE 2.7

NJ Route 21 - Ramps 8 & 11 and Route I-78 Widening

STRUCTURE	LOCATION	ESTIMATED PILE LENGTH (FEET)	18" DIAM. CONCRETE FILLED STEEL PIPE PILE		24" DIAM. CONCRETE FILLED STEEL PIPE PILE	
			SINGLE	GROUP	SINGLE	GROUP
RAMP 8	STA. 814+00 TO STA. 820+72	80	0.13	0.36	0.11	0.29
RAMP 11	STA. 1100+77 TO STA. 1112+00	110	0.16	0.42	0.11	0.29
RAMP 11	STA. 1112+00 TO STA. 1118+00	100	0.16	0.42	0.13	0.35
RT I-78	STA. 1118+50 TO STA. 1126+00	80	0.13	0.36	0.11	0.30
RT I-78	STA. 800+77 TO STA. 814+00 STA. 1126+00 TO STA. 1132+83	80	0.14	0.38	0.12	0.34

NOTE: ASSUMED PILE GROUPS WERE USED FOR GROUP SETTLEMENT CALCULATIONS
ESTIMATED PILE LENGTH BASED ON ASSUMED BOTTOM OF PIER FOOTING AT EL. 0.0

1237\TABLE1&2.WB3

be accommodated by designing piles with friction as well as end bearing, thus reducing pile length. Therefore, the piles were designed for considering friction as well as end bearing.

2.6.3 Pile Design Philosophy

There are basically four methods available to evaluate the axial pile capacity of a pile in cohesionless soils as follows:

(i) Meyerhof Method based on Standard Penetration Test (SPT)

(ii) Nordlund Method

(iii) Effective Stress Method

(iv) Cone Penetration Test (CPT)

The Meyerhof (16) method based on SPT and the method based on CPT value are empirical approaches of evaluating pile capacity. The Nordlund method and effective stress method utilize a semi-empirical approach. Based on actual test results, various correlation charts have been developed for the design of different type of piles utilizing different construction methodology. In the Nordlund method the critical depth concept to evaluate the skin friction is not applied. Therefore in general, the Nordlund method yields a higher capacity than effective stress method.

Literature review indicates a critical depth concept could be appropriate in some cases for design. Vesic (16) pointed out the following: "Beyond a depth of approximately twenty pile diameters both point and skin resistances reach nearly constant final values." These findings depart from the established concepts of linear increase of bearing capacity of deep foundations with depth. Later Vesic provided a basis for the rational explanation of this phenomenon based on the concept of the "rigidity index" of the soil. The rigidity index is defined as the shear stiffness of the soil to its shear strength, considering also the soil compressibility. In a uniform granular soil deposit the shear strength of the soil depends directly upon the vertical effective stress, which increases linearly with depth. The shear stiffness increases approximately with the square root of the vertical effective stress and therefore increases approximately with the square root of the depth. Therefore, as depth increases, the rigidity of the soil (i.e. stiffness/strength) decreases.

Meyerhof explained this phenomenon in another way, involving the estimation of a critical depth. He proposed that beyond a critical depth the mobilized force does not increase with further penetration of pile.

The pile capacities evaluated for this project were based on the method recommended by the U.S. Navy (NAVFAC) Design Manual (33). A sample static pile capacity analysis is presented in Appendix C. This method is basically a semi-empirical method based on effective stress concept and utilizes a critical depth concept for evaluating skin friction as well as end bearing. Various design coefficients have been developed based on actual field observations of various types of piles and are recommended in this manual for design. It is believed that this is a conservative approach for the evaluation of pile capacity. However, it was believed that for a variable site condition, this approach was more suitable. For the same soil conditions and the type of pile, the Norlund method predicted about 25-30 percent more capacity than that proposed by the NAVFAC method.

The assumption that a conservative approach of design (NAVFAC) methodology could be more appropriate was reinforced by the dynamic and static test results conducted during an advanced contract of the project. The dynamic test results indicated great variations in the subsurface conditions. Moreover the capacity mobilized by PDA restrike correlated well with the design value evaluated by NAVFAC method. Therefore, the design recommendations were revised after the difficulties observed during pile driving of the advanced contract. The capacities are shown in Tables 2.2 to 2.4.

Due to heavy structural loads and construction constraints, special consideration was given in developing the foundation scheme near the Northeast Corridor Railway Lines. From a broad geological point of view the subsurface conditions for Contract A area were considered to be in four groups as discussed in the Subsurface Condition Section. However, from a design point of view a close examination of the subsurface condition dictated that it was prudent to divide each section in various groups for the design recommendations. These groups were then individually evaluated based on the substrata thickness and geotechnical properties. The following two types of piles were recommended for this project:

(i) 24-inch diameter, 0.5 inch thick concrete filled pipe piles

(ii) 18-inch diameter, 0.438 inch thick concrete filled pipe piles

The 24 -inch diameter piles were utilized along the Route 21, whereas 18-inch diameter piles were utilized along I-78 Connector for the project.

A closed end pipe pile with a flat toe protection was recommended. Due to heavy structural load demands, it was recommended that the pipe pile material should conform to the Specification of ASTM A252 Grade 3 Steel. The selection of this type of pile was predominantly based on the pile driving analysis performed for the project. A pile driveability evaluation based on the work energy application program (WEAP) predicted that the yield stress of the steel should be at least 45 ksi in order to achieve the desired pile capacity without overstressing the pile. A minimum spacing of two and one half diameters between two piles was established for the project.

The lateral capacity of pile group was evaluated utilizing a computer program "Group Ver.ii" (developed by Ensoft Inc., Austin, TX). The piles were assumed to be fixed at the bottom of the cap. Allowable (working stress) and ultimate lateral (extreme event; seismic stress condition) capacities were evaluated for established tolerable horizontal deflections of 0.25 inch and 1 inch, respectively.

2.6.4 Pile Tip Evaluation

A pile tip elevation at each foundation unit was estimated based on the subsurface conditions encountered correlated with the evaluated pile length to achieve the required capacity. Within a short distance the pile tip elevation varied based on the subsurface conditions. In order to achieve the evaluated capacity it was believed that the pile should penetrate at least ten feet into the glacial till layer.

During the advanced contract (near the Northeast Corridor), some problems were realized in achieving the designed pile capacity. Based on the dynamic as well as static load tests conducted, it was concluded that the piles could experience a significant setup for this project.

The final pile tip elevation established considered the soil setup behavior of the pile. The recommended pile tip elevations for each substructure units are presented in Tables 2.8 to 2.12.

Estimated Pile Tip Elevations

TABLE 2.8

NJ Route 21 Viaduct

APPROXIMATE STATION	PIER NUMBER	STRUCTURE	ESTIMATED PILE TIP ELEVATION NORTHBOUND			ESTIMATED PILE TIP ELEVATION SOUTHBOUND		
			LEFT	CENTER	RIGHT	LEFT	CENTER	RIGHT
10+32	29	RT. 21 VIADUCT	-125±	N/A	-125±	-125±	N/A	-125±
11+55	30	RT. 21 VIADUCT	-135±	N/A	-135±	-135±	N/A	-135±
13+04	31	RT. 21 VIADUCT	-125±	N/A	-125±	-125±	N/A	-125±
14+27	32	RT. 21 VIADUCT	-120±	N/A	-120±	-120±	N/A	-120±
15+63	33	RT. 21 VIADUCT	-115±	N/A	-115±	-115±	N/A	-115±
16+81	34	RT. 21 VIADUCT	-120±	N/A	-120±	-120±	-120±	-120±
17+98	35S	RT. 21 VIADUCT	N/A	N/A	N/A	-120±	-120±	-120±
18+40	35N	RT. 21 VIADUCT	-120±	N/A	-120±	N/A	N/A	N/A
19+40	36S	RT. 21 VIADUCT	N/A	N/A	N/A	N/A	-125±	N/A
20+13	36N	RT. 21 VIADUCT	-125±	N/A	-130±	N/A	N/A	N/A
20+66	37S	RT. 21 VIADUCT	N/A	N/A	N/A	N/A	-125±	N/A
21+46	37N	RT. 21 VIADUCT	N/A	-120±	N/A	N/A	N/A	N/A
22+49	38S	RT. 21 VIADUCT	N/A	N/A	N/A	N/A	-120±	N/A
23+56	38N	RT. 21 VIADUCT	N/A	-120±	N/A	N/A	N/A	N/A
26+62	40S	RT. 21 VIADUCT	N/A	N/A	N/A	-105±	-105±	-105±
27+49	41S	RT. 21 VIADUCT	N/A	N/A	N/A	-105±	-105±	-105±
28+36	42	RT. 21 VIADUCT	-95±	-100±	-100±	-100±	-100±	-95±

NOTE: LEFT, CENTER AND RIGHT ARE LOOKING UPSTATION
THE SHADED AREA REPRESENTS FOUNDATION UNITS ALONG AMTRAK'S NE CORRIDOR

TABLE 2.9

NJ Route 21 Viaduct - Ramp 8

APPROXIMATE STATION	PIER NUMBER	STRUCTURE	ESTIMATED PILE TIP ELEVATION EASTBOUND		
			LEFT	CENTER	RIGHT
814+60	PW14E	RAMP 8	-80±	-80±	-80±
815+55	PW15E	RAMP 8	-85±	-85±	-85±
816+47	PW16E	RAMP 8	-80±	-80±	-80±
817+52	PW17E	RAMP 8	-80±	-80±	-80±
818+88	PW18E	RAMP 8	-80±	-80±	-80±
819+80	PW19E	RAMP 8	-80±	-80±	-80±
820+72	E. ABUT.	RAMP 8	N/A	-85±	N/A

TABLE 2.10

NJ Route 21 Viaduct - Ramp 11

APPROXIMATE STATION	PIER NUMBER	STRUCTURE	ESTIMATED PILE TIP ELEVATION WESTBOUND		
			LEFT	CENTER	RIGHT
1105+64	34	RAMP 11	-110±	N/A	-110±
1107+31	62	RAMP 11	N/A	-110±	N/A
1108+90	63	RAMP 11	N/A	-110±	N/A
1110+35	64	RAMP 11	-115±	N/A	-115±
1111+80	65	RAMP 11	-110±	N/A	-105±
1113+13	66	RAMP 11	N/A	-115±	N/A
1114+78	67	RAMP 11	N/A	-100±	N/A
1116+43	68	RAMP 11	N/A	-100±	N/A
1117+76	69	RAMP 11	N/A	-100±	N/A

NOTE: LEFT, CENTER AND RIGHT ARE LOOKING UPSTATION

Estimated Pile Tip Elevations

TABLE 2.11

NJ Route 21 Viaduct - Route I-78 EB Widening

APPROXIMATE STATION	PIER NUMBER	STRUCTURE	ESTIMATED PILE TIP ELEVATION		
			EASTBOUND		
			LEFT	CENTER	RIGHT
802+69	XF5	RT. I-78	N/A	-80±	N/A
803+78	PF6E	RT. I-78	N/A	-80±	N/A
804+88	PW1E	RT. I-78	N/A	-80±	N/A
805+55	PW2E	RT. I-78	N/A	-75±	N/A
806+33	PW3E	RT. I-78	N/A	-75±	N/A
807+70	PW5E	RT. I-78	N/A	-75±	N/A
808+55	PW6E	RT. I-78	N/A	-80±	N/A
809+81	PW8E	RT. I-78	N/A	-80±	N/A
811+49	PW9E	RT. I-78	N/A	-75±	N/A
812+80	PW12E	RT. I-78	N/A	-75±	N/A
813+66	PW13E	RT. I-78	N/A	-80±	N/A

TABLE 2.12

NJ Route 21 Viaduct - Route I-78 WB Widening

APPROXIMATE STATION	PIER NUMBER	STRUCTURE	ESTIMATED PILE TIP ELEVATION		
			WESTBOUND		
			LEFT	CENTER	RIGHT
1118+79	PW16W	RT. I-78	N/A	-75±	N/A
1120+11	PW15W	RT. I-78	N/A	-75±	N/A
1121+03	PW14W	RT. I-78	N/A	-75±	N/A
1122+05	PW13W	RT. I-78	N/A	-75±	N/A
1123+02	PW12W	RT. I-78	N/A	-80±	N/A
1123+97	PW11W	RT. I-78	N/A	-80±	N/A
1124+45	PW10W	RT. I-78	N/A	-80±	N/A
1125+66	PW9W	RT. I-78	N/A	-80±	N/A
1126+80	PW7W	RT. I-78	N/A	-75±	N/A
1127+55	PW6W	RT. I-78	N/A	-75±	N/A
1128+43	PW5W	RT. I-78	N/A	-75±	N/A
1129+85	PW3W	RT. I-78	N/A	-75±	N/A
1131+06	PW2W	RT. I-78	N/A	-80±	N/A
1132+21	PW1W	RT. I-78	N/A	-85±	N/A

NOTE: LEFT, CENTER AND RIGHT ARE LOOKING UPSTATION

THE SHADED AREAS REPRESENT FOUNDATION UNITS ALONG AMTRAK'S NORTHEAST CORRIDOR

1237\TAB12&13.WB3

2.6.5 Special Considerations During Design

High capacity piles were to be driven nearby the active tracks. It was believed that during the pile driving the upper layer of medium dense granular fill layer nearby the active tracks may experience intolerable settlement. It was also suspected that due to nonhomogenity in the subsurface conditions, the developed pore pressure during pile driving may not dissipate quickly due to the presence of a significant amount of silt. As a result, a minor heave may also occur at some locations.

In order to monitor the settlement and heave, various precautionary measures were recommended based on the severity observed during the construction. The settlement plates, and if required the vibratory monitoring device was recommended along the NE Corridor and other railroad tracks. During pile driving, a settlement or heave greater than 0.25 inch was considered intolerable at the active tracks. Settlement and heave values observed were less than this amount.

CHAPTER 3

TEST PILE PROGRAM

3.1 Load Test Philosophy

It is a customary practice to conduct a test pile program for a major State transportation project. This includes dynamic as well as static load tests. Pile capacity verification based on static load tests is true and accurate. But since static load tests are expensive and time consuming compared to dynamic tests, static tests can not be considered to be feasible for small projects. Dynamic tests can be useful for verifying pile lengths and pile capacities to establish production pile driving criteria and to determine the performance of hammer for driving for this project. Two dynamic load tests at each substructure unit were recommended. As specified by the NJDOT Specification, if the footing size was smaller than 50, only one dynamic load test was utilized. In addition to these tests, seven static load tests were also recommended.

3.1.1 Dynamic Load Test

Ever since the engineers began using piles for the structural supports, attempts were made to find out a rational method to verify the load carrying capacity. It is obvious that this method should be based on measuring the driving energy and the pile response to driving. Equating the kinetic energy of the hammer to the resistance on the pile as it penetrates, the pile capacity can be determined. This type of expression is known as a dynamic formula which includes the effect of pile weight, energy losses and other factors.

Wellington proposed the popular Engineering News formula in 1893 (16). At present there are various other dynamic formulas available such as Hiley, Gates, Janbu and Pacific Coast Uniform Building Code. Various studies have concluded that the pile capacities determined from dynamic formula have shown poor correlations and wide scatter when statistically compared to static load test results. It has also been observed that the dynamic formulae together with observed driving resistance do not predict actual pile capacity. Moreover, this method does not determine the stresses developed in the pile during driving.

The wave equation approach was first developed by E.A.L. Smith in 1960 (16) and has overcome many of the above discussed shortcomings. Over the years various modifications have been made. The Federal Highway Administration (FHWA) sponsored various studies to develop a dynamic analysis based on Wave Energy Application Program (WEAP) and officially released WEAP 86 in 1986. This original approach has been modified by Goble Rausche Likins and Associates, Inc. in 1996 and is known as GRLWEAP.

In a wave equation analysis, the hammer, helmet and pile are modeled by a series of segments each consisting of a concentrated mass and a weightless spring. During driving, the movement of pile segment causes soil resistance forces. The unbalanced force determined by summation of all forces acting on a segment, divided by its mass, yields the acceleration of the segment. The product of acceleration and time step summed over time is the segment velocity. The velocity multiplied by the time step yields a change of segment displacement which then results in a new spring force. This force divided by the cross sectional area is the stress at that point. Similar calculations are made for each segment. From the analysis of the next time step process, the acceleration, velocity, displacement, force and stress of each segment are computed over time. Additional time steps are analyzed until the pile toe begins to rebound.

Soil resistance along the embedded portion of the pile and at the pile toe are represented by both static and dynamic components. Static soil resistance forces are modeled by elasto-plastic springs and the dynamic soil resistance by linear viscous dash pots. The displacement at which the soil changes from elastic to plastic behavior is referred to as the soil "quake".

Permanent set of pile toe is calculated by subtracting a weighted average of the shaft and toe quakes from the maximum pile toe displacement. Inverse of the permanent set is the driving resistance that corresponds to the input ultimate capacity, used for the analysis.

Preparation of input data for wave equation is simple consisting of only the basic driving system characteristics, pile parameters and soil properties. A wave equation analysis can be conducted without much specialized knowledge. However, the interpretation of results require special knowledge. Thus, a dynamic test method was developed.

In a dynamic test method, the strain and acceleration near the pile head are measured as the pile is driven. These dynamic measurements are used to evaluate the pile driving system, pile stress, pile integrity and static pile capacity. The development of the dynamic testing technique first began in 1958 in Case Western University. After long research and testing, commercial testing equipment known as Pile Driving Analyzer (PDA) as shown in Figure 3.1 was made available in 1972. Other dynamic testing equipment (16), such as FPDS equipment and TNOWAVE have been developed in Europe. Dynamic test results were further refined by using signal matching techniques to determine the relative soil

FIGURE 3.1 Pile Driving Analyzer Assembly

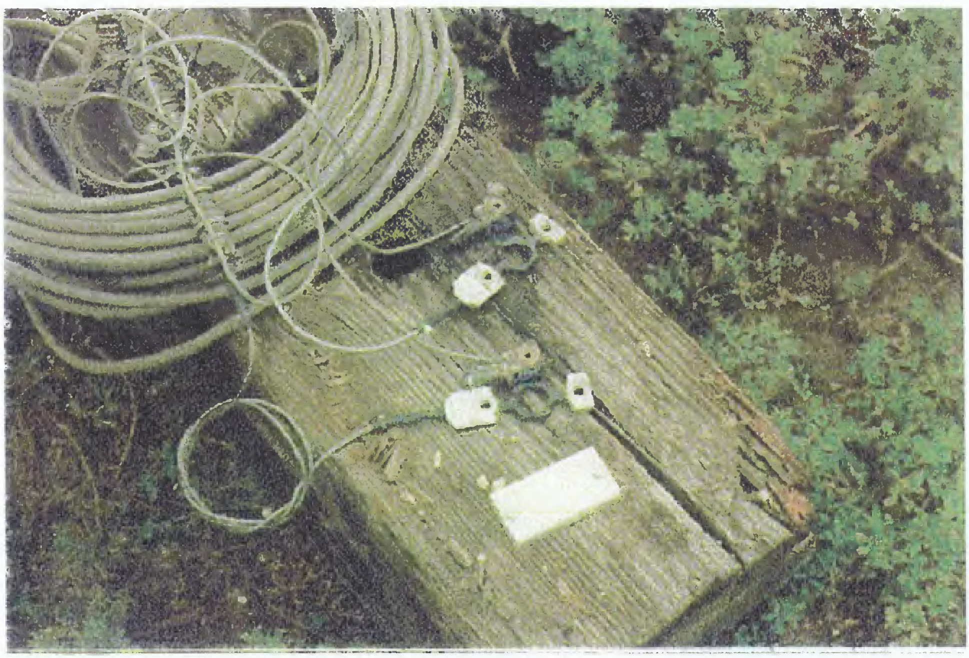


FIGURE 3.2 Strain Gauge and Acceleration Transducers

resistance distribution and dynamic soil properties. This matching technique is known as Case Pile Wave Analysis Program (CAPWAP).

A typical dynamic load testing system consists of a minimum of two strain transducers and two accelerometers bolted as shown in Figures 3.2 and 3.3 at opposite sides of the pile to account for nonuniform hammer impacts and pile bending. Cables from each gauge are combined into a single cable which in turn relays the signals from each hammer blow to Pile Driving Analyzer (PDA) data acquisition system. The PDA converts the strain and acceleration signals to force and velocity records versus time. The PDA utilizes the Case method equations for estimates of static pile capacity, driving stresses, pile integrity and transferred hammer energy. Pile capacity is predicted by PDA in terms of Standard Case Method equation (RSP) and Maximum Case Method equation (RMX). RSP evaluates the capacity of low displacement pile and piles with large shaft resistance. RMX value should be utilized for the large toe resistance and high displacement pile. Because for this condition piles are driven with large toe quake, the toe resistance is often delayed in time.

In the beginning, in order to establish the pile driving criteria, a limited number of dynamic pile load tests with PDA and CAPWAP analysis were recommended for the project. After the implementation of the Advanced Substructure Contract it was realized that the project has variable subsurface conditions within short distances. The dynamic test results indicated that within a short distance, the anticipated pile capacity cannot be mobilized during initial pile driving due to generation of excess pore water pressure. With time, the pile was expected to mobilize the anticipated skin friction. Increase in pile capacity with time is termed as soil set-up. Depending upon the subsurface conditions, the pile capacity

FIGURE 3.3 Strain Gauge and Acceleration Transducers Connected to Pile for Dynamic Load Test

FIGURE 3.4 Dynamic Load Test near Route 21 Rail Lines

would increase up to 50 percent due to soil setup. The increase in pile capacity may be realized in skin friction as well as end bearing. However, a maximum increase in capacity would be realized from mobilized skin friction.

After the above experience, the dynamic load test was recommended for each substructure unit wherever pile test was established for this project. It was recommended that all pile tests should be conducted utilizing PDA and CAPWAP analysis. It was also recommended that a maximum of four weeks time be allowed for the soil setup to develop. The pile driving methodology specified that if pile capacity was not mobilized at or near the recommended pile tip elevation during initial driving, restriking of pile would be conducted. In order to establish the setup, the restriking of pile may be conducted after two weeks and if necessary after four weeks of initial driving. Restriking was to be implemented with a warm hammer. A hammer would be considered warm after striking at least 20 blows to another pile. The restriking was to be performed for a maximum penetration of three inches or 20 hammer blows whichever occurs first. It was recommended that the CAPWAP analysis should be performed for the end of the initial driving and the beginning of the restriking. If required, the restriking would be performed at other blow count depending upon the test results.

Pile load tests were to be conducted as a pilot load test program for this project. The length of production pile was to be decided based on the dynamic load test results and/or static load test conducted for a particular substructure. Production pile driving criteria was to be established based on the refined dynamic soil properties from the pilot load test results.

3.1.2 Static Load Tests

It was realized that during the pilot load test program, some static load tests should be conducted for this project. Depending upon the subsurface soil conditions, seven static load tests were recommended for this project, three along Route 21, one along Ramp 11, and three along I-78 Connector.

It was recommended that the static load test should be conducted utilizing a reaction frame. The tests were to be performed conforming to the Specification ASTM D1143 quick test. It was also recommended that for the frame design as shown in Figure 3.5, the failure load should be assumed to be at least 1.2 times the ultimate capacity recommended for the pile. The failure load criteria was to be evaluated based on the Davisson's method (37).

3.2 Load Test Implementation

3.2.1 General

Soon after commencing work, the Contractor started implementing the pilot load testing program. Initially the load testing started at the I-78 widening. After a couple of weeks, the load testing started at Route 21 viaduct area. As per the contract specification, the production pile installation for an individual footing began after the load test for that particular footing was conducted and production pile driving criteria was established. Therefore, the driving of the test piles and the production piles progressed in an irregular manner. However, in most cases, the Contractor conducted the pilot load test in a particular area and then completing driving the production pile in that area.

All 18-inch diameter pipe piles were installed with either an ICE 60S or ICE 80S open ended diesel hammer. The rated energies for these hammers are 73 Kip-feet and 99 Kip-feet, respectively. Fixed leads were used for vertical pile driving, whereas, swinging lead was used for battered pile driving. The 24-inch diameter piles were installed using an ICE 205S hammer which had a rated energy of 210 Kip-feet. Later on, these piles were driven initially by ICE 44-65 vibratory hammer for a depth of 40 to 80 feet (depending upon the site condition) and after that an impact hammer was used. This decision was made after observing the pile driving records which indicated that the pile capacity would not be significantly impacted by switching to vibratory hammer.

3.2.2 Advanced Substructure Contract

Due to the construction of a New Jersey Transit track, an advance contract was initiated for the project. There were only four substructure units (Pier), two for Route 21 viaduct and two for I-78 widening along the Northeast Corridor track lines. 24-inch diameter piles were employed at Route 21 viaduct and 18-inch diameter pile were utilized at I-78 widening.

The pile testing included dynamic as well as static load tests. The dynamic test was conducted utilizing PDA and CAPWAP analysis. At the Route 21 viaduct site the pile capacity observed to be mobilized near the estimated pile tip elevation during initial driving at one test location. However, the pile capacity was observed to be about ten percent lower at the nearby test location. At two test pile locations which were only about 100 feet from the previous locations the pile capacity was mobilized at much lower elevation than the estimated pile elevation. One restrike with PDA and CAPWAP was performed a week after initial driving which indicated about ten percent increase in pile capacity. One static load test was conducted. A test load of 850 Kips which was more than 2 times the design load was utilized. The failure load was interpreted based on the Davisson's failure criteria which indicated the pile capacity to be about 1300 Kips. The purpose of this load test was to confirm the pile capacity at location which had indicated the increase in pile capacity with time by the dynamic tests.

The dynamic load tests conducted for 18-inch diameter piles at I-78 widening location, indicated that the capacity was mobilized at about five feet lower than estimated tip elevation at one location. At another test location the capacity was reached at much lower tip elevation than the estimated. Due to time constraints, restriking was not performed at this location. The results will be discussed in brief in Chapter 5. A summary of test results are presented in Appendix A as Table A.1.

3.2.3 Construction Contract A

3.2.3.1 Dynamic Load Tests Originally, it was planned to conduct 100 test piles. However, twelve additional tests were required to either replace the damaged test piles or to investigate unusual driving behavior.

The scheduled dynamic test included at least one pile at each foundation. Thus, one test per substructure unit for the contract area was performed. Depending upon the footing size, some footings included more than one dynamic pile test specially in the Route 21 viaduct area. There were 77 tests for 24-inch diameter piles and 35 tests for 18-inch diameter piles for the test program for this contract. Photographs of the dynamic load testing are presented in Figures 3.3 and 3.4.

The dynamic pile tests measured the strain and acceleration at the top of the pile by installing strain gauge and acceleration transducers at the top of the pile. Based on these two measured data, the PDA was able to provide the driving stress, pile capacity, maximum driving energy and assessment of pile integrity along the pile during driving. For selected hammer blows, typically at the end of the initial driving or at the beginning of the restrike, the test data were further refined by signal matching program CAPWAP. The CAPWAP analysis is a rigorous procedure in which the stress wave characteristics of a computer model are matched with those measured in the field to refine the static soil resistance and dynamic properties of the soil. The CAPWAP analysis also separates the contribution of soil resistance into skin friction and toe resistance. A total of 53 restrikes was performed for 24-inch diameter piles. At 38 locations the restriking was performed after two weeks of initial driving. Wherever the capacity was not mobilized after two weeks of initial driving, restriking was performed after about four weeks of initial driving. At some location restrike was performed only after four weeks of initial driving. A total of 21 restrikes were performed for 18-inch diameter piles. At 19 locations restrikes were conducted after two weeks.

3.2.3.2 Static Load Tests The static load tests were performed to determine and evaluate the pile capacity as well as verify the dynamic pile test results. Static load tests were also performed at locations where dynamic pile testing was inconclusive, or to evaluate unanticipated driving conditions.

Static load tests were performed on two of the 18-inch diameter piles and seven of the 24-inch diameter piles. The location and pile designations for the static load test piles are presented with the plots of load versus deformation for typical load tests presented in Appendix B.

The static load tests were conducted using a modified version of the Quick Load Test Method as defined in ASTM D 1143. Static load tests were performed to load levels of up to 135 percent of the computed ultimate capacity of the piles, or to plunging failure, whichever occurred first. Plunging failure was considered to occur when continuous jacking was required to maintain the applied load or when the displacement exceeded 2 inches, whichever occurred first. The piles were loaded in increments of 5 to 7.5 percent of the computed ultimate pile capacity. These loads were maintained until the rate of settlement was less than 0.01 inches per hour, but no longer than one hour. Each increment was maintained for a minimum of 10 minutes. The piles were unloaded in decrements of 25 percent of the maximum applied load.

The load for all tests was applied to the pile with a single hydraulic jack. A 500 ton jack was used for the 18-inch diameter piles and a 800 ton jack was used for the 24-inch diameter piles. Applied load was measured by an electrical load cell positioned between the jack and the main reaction beam. The load cell readings served as the basis for the load measurement and control during the test. The hydraulic jack pressure gauge was also monitored during testing as a check to the applied load measured by the electrical load cell.

A spherical bearing plate was placed on top of the load cell to reduce any errors caused by ram misalignment. The load cell and jack pressure gauge were all calibrated prior to performing the load tests. Periodic recalibration was performed for the test apparatus during pile driving and testing operations.

Vertical displacement of the test piles was measured by three micrometer dial gauges and one piano wire gauge mounted on the test pile in addition to the electrical load cells. The micrometer and piano wire gauges were equally spaced around the pile. These instruments were supported by steel reference beams supported on reference piles which were independent of the reaction frame. The reaction frame was also monitored for movement by an optical survey during load application. The static load test reaction frame consisted of wide flange beams which transferred the applied load to reaction piles. For the test of the 18-inch diameter piles, 10 reaction piles were used while 16 were used for testing the 24-inch diameter piles. Reaction piles were HP 12 x 74 piles which were vibrated, then driven the last 10 feet into the ground. Photographs of the static load test reaction frame and setup are presented in Figures 3.5 and 3.6, respectively. Some additional photographs are presented in Appendix B.

Interpreted failure loads for the static load tests were determined in accordance with Davisson's offset criteria (16), unless plunging failure occurred.

FIGURE 3.5 A View of Static Load Test Reaction Frame

Figure 3.6 Static Load Test Setup

CHAPTER 4

REVIEW OF LITERATURE PERTAINING TO THIS STUDY

It is well known that the piles driven into soft to medium clays or loose saturated silts and silty sands usually exhibit a time dependent increase in load capacity due to the effect of 'soil setup' or 'soil freeze'. A setup factor is defined by Poulos and Davis (38) as the ratio of soil strength a considerable time after driving to that immediately after driving. In a normally consolidated clay the strength will generally increase because of two factors: Thixotropic regain of undrained strength as the structural bonds destroyed by remolding are at least partially restored, and increase resulting from local consolidation of the clay produced by dissipation of excess pore water pressures that arise from the increase in stress in the soil surrounding the pile. In stiff and overconsolidated clay negative pore pressure can develop due to swelling with time.

A setup can also occur in some high permeability sands. However, in this case the process is believed due to the mechanical aging of soils. Many natural deposits such as glacial deposits are sensitive to disturbance. As reported by York, et al (56) these soils experience considerable loss in strength during driving, followed by time dependent strength gain as the soils structure heals at the constant effective stress. If the disturbed soils have been densified, the aged soil will have significant improved strength.

In order to discuss the results of pile load tests and expected soil setup, a review of literature was performed. Important observations and suggested methods for dealing with the problems expected to occur in the various type of soils are discussed. The literature related with soil setup behavior observed for sandy and glacial deposits will be discussed first, followed by the predominantly clay type of soils. Pertinent literature regarding the construction control associated with pile driving problems will also be discussed.

Change in capacity over time of a pile driven in clay has been documented for nearly a century. Over the years it has been realized and reported that the capacity change may occur not only in clay but also in loose sands and mixed typed of soils. Several observations of the pile capacity change in glacial deposits have also been reported.

In order to generate a successful and economical project, it is not only important to design the project with sound engineering knowledge and understanding, but also to know how to implement the design expectation into reality. A study conducted by Sowers (51) to evaluate the factors associated with the geotechnical engineering failures stated that 58 percent of the problems originated in design, 38 percent in construction and only 4 percent in operation. Approximately half of the problems that occurred during construction originated from design, the other half during construction. Sowers further stated that the primary causes for this was absence (12 percent), ignorance (33 percent) and rejection of current technology (55 percent). Of the total, 88 percent could be reduced by acknowledging professional limitations, continuing education, modifying design and construction system and good engineering judgement. Time and money are required to reduce ignorance, and to better utilize our present knowledge and technology in design and construction.

At the 29th Terzaghi Lecture, Focht Jr. (20) discussed a study regarding the reliability of the pile capacity prediction. In this paper, it is stated that nearly 25 percent respondents thought that their pile capacity estimates were reliable within ± 10 to 20 percent, 30 percent within ± 20 to 30 percent and 25 percent within ± 30 to 50 percent. This indicates that a lot of uncertainties are realized in pile capacity design and the construction process by the geotechnical professionals. He identified six critical factors missing in the predicted design values. The six factors are summarized as: Stratigraphy; Properties; Analytical; Historical; Judgement and Intuition. Applying judgement and intuition to address a problem is a significant factor. It has been quoted that "Good judgement comes from experience, and where does experience come from? Experience comes from bad judgement". Good judgement is more than good technical knowledge. He has suggested that a geotechnical engineer must apply good judgement throughout the project design and construction process.

The laboratory tests conducted on long model piles in sand by Hanna, et al. (23) reaffirms the previous studies of the importance of the state of stress and density of the sand at or near the toe of pile in mobilizing the pile capacity during pile installation. He also reported that the shaft friction and end resistance followed a linear relationship with embedment depth of pile. Up to a length to depth ratio of 30 to 40, the resistance increased and beyond that it was virtually constant. This study supports the design methodology of critical depth concept employed for this project. It is the author's conclusion that this approach of pile design is more appropriate for a long pile. Most importantly, Hanna, et al. (23) has demonstrated that no single method of pile analysis will provide a correct interpretation of a pile unless the residual load state of the pile subsequent to the placement of the pile is quantitatively accounted for.

The study conducted by Randolph, et al. (40) by the database of actual load test results demonstrates that the mobilization of actual pile behavior is different than the established limiting friction and end bearing capacity methodology. It has been discussed that in keeping with field observations, shaft friction is observed to be maximum at some distance above the end bearing. However, at the tip of pile it decreases to a minimum. This has been suggested that in order to evaluate end bearing, a limit of skin friction based on critical depth criteria should not be considered.

Fellenius, et al. (18) has reported the pile load test results exhibiting soil setup behavior in highly variable glacial deposits. The subsurface condition stated is very similar to the condition encountered at the Route 21 project, and the pile lengths are also similar. In this paper it has also been reported that with minor exception, driving was generally easy for most pile penetrations. However, after a week of initial driving, capacities increased considerably due to soil setup; the soil setup behavior was not consistent over the site. The pile capacity predicted by dynamic test results matched by CAPWAP analysis compared well with static load test results. Soil setup behavior was confirmed both visually from the wave traces and the CAPWAP results indicating that the increase in the soil resistance was not due to a reduced hammer efficiency or any related influences. A study of capacity increase with time indicates that setup occurred rapidly during the first day after initial driving and then continued at a slow but steady rate for several weeks. Based on this observation, it is assumed that the soil setup is likely to occur for a longer period of time in a variable glacial deposit formation.

A similar case study of a project located nearby this (Route 21) project have reported similar results by York, et al (56). The subsurface condition is identified as medium dense glacial deposits of clean sand. Displacement piles driven into this material showed a time-dependent increase in capacity varying from 40 to 80 percent. From this, it is interesting to note that a significant setup can occur in high permeability soils also. The process by which setup occurs is termed as soil aging. It is known that many natural deposits of pervious sands are sensitive to disturbance. These soils experience considerable loss of strength when their structure is disturbed, followed by a time-dependent gain in strength at a constant effective stress. The strength increase is attributed by the reestablishment of cementation at inter-particle contacts and mechanical aging. It was reported that the setup approached a maximum value within 15 - 25 days.

Relaxation was also reported for a high group of piles in dense deposits. The reason for relaxation was attributed to the cumulative effects of soil displacements and pile- driving vibration compaction of the glacial deposits to a dense state, causing the sand to dilate. This dilation temporarily increased the effective stress at the pile toe. The time for relaxation was observed from a few hours to several days. It was also reported that there may be minor setup after relaxation due to normal process of aging of soil. A similar observation was reported by Svinkin (52) for dense saturated sand. He has stated that the setup behavior in saturated dense sand is complicated and is difficult to generalize even for a given site. Due to the false refusal encountered for many friction piles in dense to very dense fine sand, Moller and Bergdahl (32) measured the induced pore pressure for model piles. They concluded that the false refusal was due to the development of negative pore pressure in dense sand during driving.

A long term gain in the pipe pile capacity in dense marine sand deposits has been reported by Chow, et al. (12) for a project located in France. An 85 percent increase in capacity occurred five years after pile installation. Obviously, this increase in capacity was not only due to the pore pressure development phenomenon around the pile. In granular soil, the stabilization of pore pressure dissipation is expected to occur within hours or days. The paper evaluates the cause for this long term gain. It is suggested that during pile driving arching mechanism develops around the pile, limiting the radial stresses acting on the pile shaft. The long term creep development leads to breakdown of these arching stresses, allowing increase in radial stress and hence ultimately increase in pile capacity. It was also concluded that increased dilation due to sand aging may also contribute in the pile capacity gain. The micro-rearrangement of sand grains during creep may also result in stronger dilation effects during shearing, producing larger increase in the radial stress. In this paper, it was concluded that the effective stress approach provides the most reliable medium term prediction of shaft loads.

The results of a database study to quantify effects of time on pile capacity in sand, clay and mixed soil subsurface conditions have been reported and discussed by Long, et al (30). The study has focused on the time-dependent increase pile capacity due to excess pore pressure dissipation and also due to soil aging. In clay, the setup is attributed by excess pore pressure dissipation, whereas, in sand it is predominantly due to soil aging. This database report has indicated that in clay and mixed profiles, the capacity increased up to six times. The largest increase in capacity developed in 20 to 30 days, beyond that it continued to increase with a smaller rate for half of the pile. For the remaining half it appeared to be constant. It was also observed that 100 days after driving the capacity leveled out. In a sand profile, the soil setup ranged from 30 to 100 percent in 10 days. An empirical relationship for setup behavior in sand has been recommended. In the sand profile, it was concluded that the setup occurred due to soil aging. It was believed that the pile capacity would continue to increase after 10 days, however, with a lesser rate. No major difference was observed in pile capacity gain between a displacement and non-displacement pile.

Thompson, et al. (43) has reported the condition when a real and apparent relaxation can be realized. It is suggested that a permanent decrease in pile capacity with time (real relaxation) is a rare occurrence in glacial deposits. Real relaxation has been observed for end bearing piles in shale bedrock. An increase of the bearing surface area did decrease the end bearing on shale. It is hypothesized that the reason for the decrease of bearing capacity with increasing size is a change in the failure mode from crushing of the intact bedrock to general failure of the bedrock mass. It is reported that an apparent relaxation defined as a decrease in penetration resistance as a change in pile driving performance was commonly observed. The apparent relaxation is primarily associated with single acting diesel hammers. These hammers tend to have a decrease in efficiency after extended hard driving. In the beginning of driving it operates at full efficiency. As a result, the same hammer may drive a pile more effectively on restrike than at the end of initial driving.

A pressure dissipation measurement during pile driving and the comparison with the theoretical values have been reported by Ismael, et al. (25) in clayey silty to sand type of materials. The pore pressure dissipation was observed to occur in four (4) days. The computed and theoretical values agreed well. Based on this study, it is concluded that the soil setup behavior is entirely not related with the pore pressure dissipation in mixed soils.

Soderberg (50) has explained that the pile capacity gain with time in clay and silt in terms of Terzaghi's Consolidation Theory. It is reported that the pile capacity of a friction pile is dependent on the diffusion time of the developed hydrostatic pressure generated by pile driving. The time required to reach a specified state is proportional to the square of the horizontal dimension of the pile and is inversely proportional to the horizontal coefficient of consolidation. It is also discussed that setup behavior is not entirely dependent upon soil characteristics. Based on this study, it is concluded that the horizontal dimension of a pile and the spacing of piles in cluster are more important than the characteristics of the surrounding soil in developing the soil setup behavior.

Randolph, et al. (39) has described the pile installation behavior by a numerical analysis. Consolidation of the soil was analyzed using an elasto-plastic soil model. The analysis was used to predict changes in the strength and water content of soil adjacent to a driven pile, which compared well with the actual measurements. It was also shown that the rate of increase of bearing capacity of a driven pile may be estimated with reasonable accuracy from the rate of increase in shear strength of the soil predicted from the analysis. An important controversial conclusion was also derived in that the shaft capacity of a driven pile in a soil of a given undrained shear strength is effectively independent of the overconsolidation ratio.

An interesting analysis of load test results on driven piles is reported by Ismael (24). The piles were driven through a loose-to-compact calcareous surface sand (fine to medium sand and silt) to a competent dense to very dense siliceous cement sand deposits. It was observed that the surface calcareous sand contributed only 4 to 11 percent of skin friction. A major portion of the calculated pile capacity was derived from tip resistance.

Gain of pile capacity with time due to the soil aging process has been reported by

Jardin, et al. (26). Long piles were driven in dense sand and after 50 days of initial driving, the capacity was observed to be much higher than the initial capacity. With the established aging process, it was concluded that in a 28 year period the pile capacity could increase two times. This was believed due to the long term soil aging process. Long term research in this area was suggested.

Various causes for the improvement of soil properties with age were compiled and explained by Schmertmann (47) at the twenty-fifth Karl Terzaghi Lecture. He has concluded that factors for aging effects are Thixotrapy, secondary compression, particle interference and clay dispersion. This paper proposed that a new pore pressure dissipation theory that provides a mechanism for significant pore pressure reduction and dissipation effects in saturated soils. It also states that the dissipation does not result from hydrodynamic water flow rather the transfer of load from the pore fluid to the soil fabric skeleton due to aging.

The behavior of a driven pile in clay was studied by Coop, et al. (13) on model piles in normally and overconsolidated clays. In this study it was concluded that during undrained condition the magnitude of the increase in the radial effective stress during loading is similar in the normally consolidated and heavily overconsolidated clays. In normally consolidated clay, this increase accounts for the setup of pile capacity. For a drained loading condition, no change in pore pressure was seen to result in an increase in radial effective stress. This suggests that the pile capacity may be lower in drained loading condition contrary to the current theory.

Load test results conducted for a major roadway reconstruction project have been reported by Attwoll, et al. (6). The subsurface conditions varied from clay to sand and mixed soils. Regardless of subsurface conditions, the setup behavior was observed throughout the project. The setup in layered soft to stiff lakebed clays was attributed to the remolding during driving and subsequent reconsolidation. Setup in dense sands was comparable with those in the dense marine deposits. It was also concluded that where a significant amount of the shaft resistance was obtained from interbedded granular strata.

A study performed by Camp III, et al. (11) for the long term capacity gain in stiff cooper marl deposit indicates that the time dependent pile capacity gain depends upon the pile size. The rate of the capacity gain generally decreases as the pile size increases. In this paper it has been suggested that to have better quality control, dynamic testing should be performed at very short intervals of time to correlate the testing data for better pile capacity prediction.

A numerical procedure to predict the pile capacity with time was recommended by Titi and Wathugala (44) by simulating the behavior of pile driving during installation, subsequent consolidation and loading stages. This numerical procedure utilized the theory of strain path method. The finite element nonlinear analysis of porous media was utilized to simulate the subsequent soil consolidation and pile load tests. Both the shaft and end bearing capacity increased with time. However, the major increase was observed to be in shaft capacity.

A method to evaluate the skin friction during pile driving taking into consideration soil degradation has been proposed by Alawneh (1) based on the pull out load tests database in loose to very loose sand soils. It was concluded that at a given location, the earth pressure coefficient is assumed to degrade from a maximum value (near the pile tip) to a minimum value as an exponential or as a power function of the length of pile. The maximum earth pressure coefficient value has been linked to the relative density of sand, effective vertical stress and the pile diameter. It has also been concluded that most of the current design methods are not consistent with the observed pile behavior during driving and axial loading and the shaft friction problem remains an open area for future research.

Whittle and Sutabur (54) has studied the setup behavior in the normal and overconsolidated clays using the strain path method for pile installation and the finite element method for setup of effective stress of soil. It is concluded that earth pressure coefficient (i.e. setup behavior) increases with overconsolidation ratio (OCR). The open ended piles generate less setup (about 10-30 percent) than closed end piles. Low plastic sensitive clays generate the lowest setup stresses at a given OCR, while highly plastic insensitive clays generate the highest value of setup.

Skov, et al. (49) analyzed the pile testing data from three case histories. They recommended a relationship between pile setup and time. Their study concluded that some time after pile installation, pile capacity gain becomes linear if plotted on log scale. Skivin, et al. (48) supported this developed relationship. Paikowski, et al. (37) believed that this method may not be suitable since a large amount of testing over time would be required for each pile size and site to develop the size factor and initial time utilized in the equation.

The results of a full scale investigation conducted in soft sensitive clay soils are reported by Roy, et al. (42). This study concluded that during driving the reduced pore pressure at the pile tip is 1.6 times the total overburden pressure whereas at the pile surface it is 0.8 times the total overburden pressure. Pore pressure is fully dissipated in 600 hours and the consolidation period is governed by consolidation characteristics of the destructed clay. Immediately after driving a decrease in undrained shear strength of the order of 30 to 40 percent was observed within 4 diameter. The strength was fully recovered after pore pressure dissipation.

Four interesting case histories of change in pile capacity with time have been reported by Samon and Authier (45). In two cases, where the piles were driven into deep dense sand, an increase in pile capacity ranging from 33 to 85 percent in a period of 2 to 51 days, respectively, was observed. In the other two cases, where the piles (closed end) were driven to the shale bedrock, the pile capacity was observed to decrease by 11 to 25 percent in a few days after initial driving. It was suggested that the restriking must be performed to account for relaxation for toe bearing pile on shale bedrock.

Pore pressure behavior during pile driving in slightly overconsolidated clay was studied by Azzouz and Morrison (7). It was concluded that clay with low sensitivitiy developed high effective stress on pile shafts.

Long, et al. (29) have studied the most reliable method to predict the pile capacity in the field. A database of approximately 100 load test results is used to quantify the evaluation. The study has included Engineering News formula, Gates formula, Wave equation program (WEAP), measured energy approach (ME), Pile Driving Analyzer (PDA), and Case Pile Wave Analysis Program (CAPWAP). The evaluation has ranked the predicted method based on Wasted Capacity Index (WCI). The WCI is a measure of how inefficiently a method predicts capacity. A precise method will be very efficient and accordingly will have a low WCI. The WCI is calculated from the precision of method and reliability required for the pile foundation. The results show that the use of CAPWAP for the restriking data has the greatest precision, and thus the lowest WCI.

A simplified dynamic method to predict the pile capacity was proposed by Liang and Husein (27). This method is also a dynamic method based on Smith's model and is based on the energy balance concept and utilizes the data from pile driving record. Though it has some merit, the dynamic data is required as input. The elaborate CAPWAP analysis is avoided by utilizing this method. Results of this method will still have to be correlated with those from PDA.

A probabilistic approach was recommended by Liang and Zhou (28) to monitor and control pile driving. This method is based on the energy approach, utilizing the energy delivered to the pile head, the blow count of pile penetration, the maximum velocity at pile head, pile dimensions and elastic properties. Results agreed well with those from CAPWAP analysis and static load tests. The drawback is that an expensive electronic sensor will be required to measure energy. It was suggested that research should be conducted towards developing economical instruments to measure the energy during pile driving.

It has been observed that diesel hammer creates problems in pile driving in certain type of soils. A field evaluation of six diesel hammer's performance was studied by Wu, et al. (53) for the prestressed piles driven into alluvial deposits. They reported that a diesel hammer's performance can not be evaluated by observations only. Dynamic testing should be conducted to evaluate the hammer efficiency in addition to driving energy, otherwise, a wide range of driving resistances could be experienced in the field. In order to maintain the meaningful pile driving criteria, hammer should perform as specified by the manufacturer. Regular maintenance of hammer is very important.

Goble (22) has discussed as to how the dynamic load test method has been developed and what improvement could be expected in the future. In the earlier part of the 20th Century, Engineering News formula was developed to predict the pile capacity. In circa 1950, Smith of Raymond Pile Company developed the first dynamic model analyzed by electronic digital computer. Later, after an extensive research and trial by Case Institute of Technology (now Case Western University) and Ohio Department of Transportation, the Case method of predicting the pile capacity was developed. Later, Pile Driving Analyzer (PDA) was commercially made available to measure force and acceleration to compute the pile capacity by Case method.

Later, it was realized that if the measured motion is input at the pile top and the soil resistances are assumed, it was possible to calculate the force required to generate the input motion. Based on this analogy, a software called CAPWAP was developed. The demonstration project 66 conducted by the Federal Highway Administration in each state endorsed the use of dynamic test results utilizing pile driving analyzer and CAPWAP analysis. At present, for most of the major highway projects, dynamic pile load tests utilizing PDA are routinely performed. It is believed that by utilizing a dynamic load test, a savings of production pile length of about 15 percent can be realized.

CHAPTER 5

DISCUSSION OF LOAD TEST RESULTS

5.1 General

Based on the literature review presented in the previous chapter and engineering judgement, a discussion of test results is presented. Load test results of the construction Contract A and Advanced Contract are discussed in this Section. A limited amount of tests were conducted for the Advanced Contract. Discussion is predominantly based on construction Contract A test results.

Discussion of the load test results is associated with the observations noted in the change in pile capacities with time for the project. It is prudent to briefly discuss the definition and the phenomena associated with the time effects on the pile capacity.

Driving of a pile into the ground changes the condition of the in-situ materials considerably. The state of stress around piles is changed momentarily or even altered depending upon the type of soils present. If groundwater is present, the dynamic force caused by driving develops pore pressure around the pile affecting driving behavior of the pile as well as the long term static capacity of the pile. When a positive pore pressure is developed around the pile and with the passage of time this excess pore pressure is dissipated, this phenomenon is known as soil setup. Due to this increase in pore pressure, the driving resistance is decreased initially, but when the excess pore pressure is dissipated the soil adhesion around the pile increases resulting in increase of soil resistance. The amount of soil setup and the time for setup to occur depends upon the soil type and stress history of the soil. In general, setup generally occurs in clay or silty clay and loose to medium dense silt and silty sand.

Sometimes during pile driving, negative pore pressure is developed around the pile due to the dilation of soil. This phenomenon is called soil relaxation. The development of negative pore pressure temporarily increases the driving resistance. However, with time the pore pressure decreases resulting the decrease in pile capacity. This behavior generally occurs in dense saturated sand and silt, overconsolidated clay and decomposed shale bedrock. In shale this is due to crushing of rock.

The ratio of the increased pile capacity with time with the pile capacity observed during initial driving is defined as 'setup factor'. A setup factor of greater than 1.0 indicates that soil setup has occurred while a value less than one indicates soil relaxation.

5.2 Dynamic Load Test Results

A setup factor was evaluated for all dynamically tested piles where the capacity measurements are available during initial driving and for a restrike at a two week or four week period. The results of these calculations are presented separately for all 18-inch diameter piles and all 24-inch diameter piles. The data is also tabulated and presented in Tables 5.1 and 5.2 for 18-inch and 24-inch diameter piles, respectively.

Time Effects for 18-Inch Diameter Piles:

18-inch diameter piles were utilized along the I-78 widening area only. The subsurface soil condition in this area is represented by Soil type 1 as discussed earlier. As presented in the data table (Table 5.1), soil setup has occurred substantially in this area. The setup factor for a two weeks restrike ranges from 1.08 to 1.89 yielding most of the value

TABLE 5.1
Test Pile Setup Factors
(18 Inch Piles)
For Construction Contract A

PIER NO	PILE NO.	ULTIMATE CAPACITY (Kips)	INITIAL DRIVE (Kips)	2 WEEK RESTRIKE (Kips)	4 WEEK RESTRIKE (Kips)	S. LOAD TEST (Kips)	2-WEEK SETUP FACTOR	4-WEEK SETUP FACTOR	S. LOAD TEST FACTOR	COMMENTS
PW1W	1	540	286	540			1.89			
PW2W	1	540	401	679			1.69			13 Days restrike
PW3W	1	540	401	547			1.36			13 Days restrike
PW5W	1	540	621							Capacity at initial drive; no restrike
	4	540	580							Capacity at initial drive; no restrike
PW6W	3	540	357	454			1.27			13 -day restrike
PW7W	1	540	416	616			1.48			
PW9W	1	496	340	425			1.25			
	5	496	470		698			1.49		34-day restrike
PW10W	2	496	430	611		660	1.42		1.08	Note: 660/611=1.08, 15-d rst.
	4	496	410	525			1.28			15-day restrike
PW11W	1	496	419	544			1.30			
PW12W	1	496	336	433			1.29			13-day restrike
PW13W	1	496	456	544			1.19			16-day restrike
PW14W	1	496	490	744			1.52			16-day restrike
PW15W	1	496	540							Capacity at initial drive; no restrike
	8	496	471	550	80	>670			>1.22	1-d red. 550 kips; 670/550=1.22
PW16W	3	496	510							
	8	496	515							

TABLE 5.1 (Continued)
Test Pile Setup Factors
(18 Inch Piles)
For Construction Contract A

PIER NO	PILE NO.	ULTIMATE CAPACITY (Kips)	INITIAL DRIVE (Kips)	2 WEEK RESTRIKE (Kips)	4 WEEK RESTRIKE (Kips)	S. LOAD TEST (Kips)	2-WEEK SETUP FACTOR	4-WEEK SETUP FACTOR	S. LOAD TEST FACTOR	COMMENTS
E. ABUT.	14	560	581							
	42	560	465	604						2-day restrike
PW1E	1	540	321	467	513		1.45	1.6		15-day restrike, 29-day restrike
PW2E	1	540	336	410			1.22			19-day restrike
PW3E	1	540	692							Capacity at initial drive; no restrike
PW5E	1	540	783							
	4	540	710							
PW6E	1	540	456	493			1.08			13-day restrike
PF6E	1	540	430	546			1.27			13-day restrike
PW8E	1	540	610							Low capacity at planned tip; immediate redrive to capacity
PW9E	2	540	662							Low capacity at planned tip; immediate redrive to capacity
	6	540	569							
PW12E	1	540	321	477	490		1.49	1.53		16-day restrike, 30-day restrike
	8	540	321		385			1.2		26-day restrike
PW13E	1	540	650							Low capacity at planned tip; immediate redrive to capacity
	8	540	675							Low capacity at planned tip; immediate redrive to capacity

NOTE: All piles were in Soil Type-1

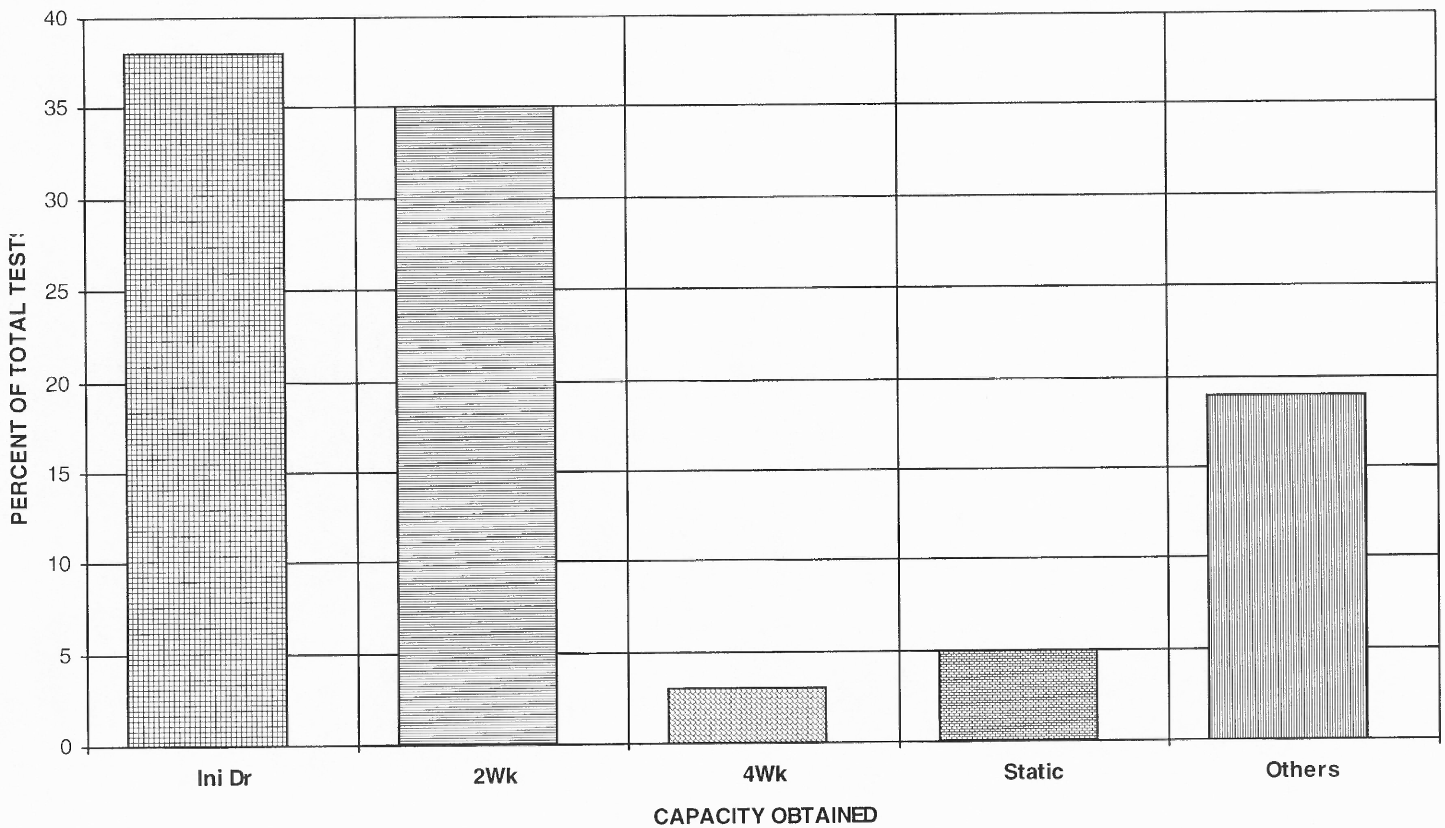


FIGURE 5.1 Pile Setup Distribution (18 Inch Piles)

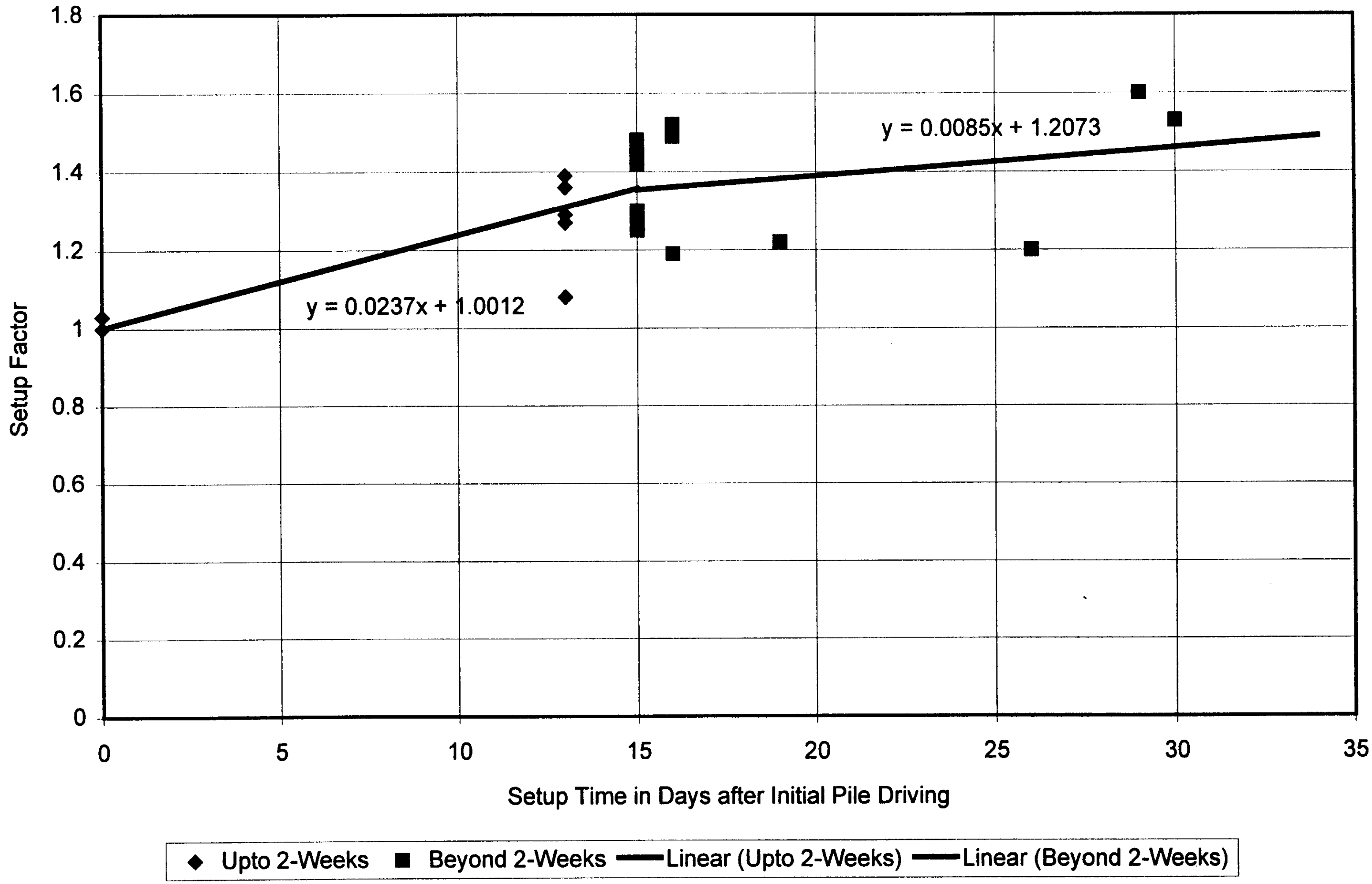


FIGURE 5.2 Setup Factor Versus Time For 18-Inch Piles (Soil Type 1)

from 1.25 to 1.3. The pile setup distribution as presented in Figure 5.1 indicates that a substantial amount of pile capacity was obtained in the two week period. A best fit of curve ignoring the extreme data is plotted by least square method of linear regression, for a two week period and beyond 2 weeks after initial driving. This is presented in Figure 5.2. The probable equation for each period is also presented. This plot indicates that up to a period of two weeks there is about a 2.4 percent increase per day in pile capacity; thereafter, the increase in capacity is at the rate of 1 percent per day. Most of the soil setup occured two weeks after initial driving, however, the capacity would increase for a long period of time. In highly variable glacial deposit the setup could occur for a long period of time as reported by Fellenius, et al. (18). This plot also indicates that after the two week period the setup factor is widely scattered. The setup data table (Table 5.1) also indicates that along the eastbound, a substantial amount of test piles achieved required capacity during the initial driving. It is the author's belief that this could be due to the presence of bedrock at a shallower depth. The soil densification may also have contributed to mobilize capacity during initial driving.

Time Effects for 24-Inch diameter piles:

The 24-inch diameter piles were used in the subsurface condition area designated Soil type 2 to 4 discussed earlier in the Subsurface Condition Section. The soil setup data is presented in Table 5.2. The pile setup distribution as presented in Figure 5.3 indicates that a significant amount of piles mobilized pile capacity during initial driving at or around the estimated pile tip elevation. The time effect characteristics in each soil type will be discussed separately, below.

TABLE 5.2
Test Pile Setup
(24 Inch Piles)
For Construction Contract A

PIER No.	PILE NO.	SOIL TYPE	ULTIMATE CAPACITY (Kips)	INITIAL DRIVE (Kips)	2 WEEK RESTRIKE (Kips)	4 WEEK RESTRIKE (Kips)	S. LOAD TEST (Kips)	2-WEEK SETUP FACTOR	4-WEEK SETUP FACTOR	S. LOAD TEST FACTOR	COMMENTS
29S	1	2	1032	1090		>1400				>1.28	Cap. at ini. dri. no re-strike:
	5	2	1032	1100							
	8	2	1032	1039							Cap. at ini. dri. no re-strike:
30S	8	2	1000	1013							Cap. at ini. dri. no re-strike:
	16	2	1000	1014							Cap. at ini. dri. no re-strike:
31S	1	2	1000	1000							Cap. at ini. dri. no re-strike:
	11	2	1000	1065							Cap. at ini. dri. no re-strike:
	16	2	1000	1010	1329			1.32			
32S	1	2	1000	1120	1233			1.1			12- days re-strike
	16	2	1000	1002	1001			1.1			10 days re-strike- RED. CAP.
33S	1	2	1000	855							
	12	2	1000	837							
34S	2	3	1000	1004							
	11	3	1000	1010							
	16	3	1000	1000							
35S	3	3	1000	865	1060			1.23			5 Days re-strike
	13	3	1000	870	1123			1.29			5-day restrike
36S	1	3	968	848	880	985		1.04	1.16		29-day restrike in 2/4 wk cols.
	4	3	968		1175	1038					2 restrike in 1 day, no ini. value
	21	3	968		990						No initial value
	22	3	968		990						No data
37S	1	3	968	900	1062			1.18			10-day restrike
	16	3	968								No data
	30	3	968	707	820	989	1200	1.16	1.4	1.21	Note: 1200/989=1.21, 13- day re-strike, 27- day re-strike.

TABLE 5.2 (Continued)
Test Pile Setup
(24 Inch Piles)
For Construction Contract A

PIER NO.	PILE NO.	SOIL TYPE	ULTIMATE CAPACITY (Kips)	INITIAL DRIVE (Kips)	2 WEEK RESTRIKE (Kips)	4 WEEK RESTRIKE (Kips)	S. LOAD TEST (Kips)	2-WEEK SETUP FACTOR	4-WEEK SETUP FACTOR	S. LOAD TEST FACTOR	COMMENTS
38S	24	3	1000	798	950	954		1.09	1.2		26 day re-strike
40S	2	4	954	775	795	714		1.03	0.92		4-wk rest. low, but red to ult.
	23	4	954	685	873	900		1.27	1.31		4-wk rest. low, but red to ult.
41S	3	4	954	790	780	742		0.99	0.94		Lower capacity at restrike.
	13	4	954	1020	884	868		0.87	0.85		
42S	3	4	1030	1040	1048			1.01			27-day restrike
	13	4	1030	750	810			1.08			13 day restrike
29N	1	2	1032	1036							Cap. at initial drive; no re-strike
	5	2	1032	1044							Cap. at initial drive; no re-strike
30N	1	2	1000	1075							Cap. at initial drive; no re-strike
	16	2	1000	1045							
31N	1	2	1000	1004							Cap. at initial drive; no re-strike
	3	2	1000	1045							Cap. at initial drive; no re-strike
	9	2	1000	1004							Cap. at initial drive; no re-strike
32N	1	2	1000				>1350			>1.41	3 day redrive to 960kips;
	16	2	1000	1000	1191			1.91			19-day restrike
33N	6	3	1000	1110							Cap. at initial drive; no restrike
	10	3	1000	1021							
34N	1	3	1000	1005							
	8	3	1000	1025							
	18	3	1000	1025							
35N	2	3	1000	840	895			1.07			No initial value
	15	3	1000	960	1190			1.24			No initial value

TABLE 5.2 (Continued)
Test Pile Setup
(24 Inch Piles)
For Construction Contract A

PIER NO.	PILE NO.	SOIL TYPE	ULTIMATE CAPACITY (Kips)	INITIAL DRIVE (Kips)	2 WEEK RESTRIKE (Kips)	4 WEEK RESTRIKE (Kips)	S. LOAD TEST (Kips)	2-WEEK SETUP FACTOR	4-WEEK SETUP FACTOR	S. LOAD TEST FACTOR	COMMENTS
36N	2	3	968	721	824	950		1.14	1.32		28-d rst., 40 d rest. in 2/4 wk col.
	7	3	968	755	1110			1.47			11 day restrike
	17	3	968	650	1160		>1300	1.78		>1.12	5-day restrike
37N	3	3	968		930						No initial value
	13	3	968	919		1020			1.11		27-day restrike
	16	3	968		1075						No initial value
	24	3	968		980						No initial value
	30	3	968		940						No initial value
42N	1	4	1030	735	728	728		0.99	0.99		Lower rest cap. than initial drive, 13-day restrike, 27- day re-strike
	4	4	1030	810			>1000			>1.24	1000/810=1.24
	24	4	1030	865	810	857		0.94	0.99		Lower restrike cap. than drive, 13-day restrike, 27 day restrike
14E	1	1	1020	1030							
	12	1	1020	1030							
15E	1	1	1020	1030							
	12	1	1020	1032							
16E	1	1	1020	1189							
	12	1	1020	1038							
17E	1	2	1020	940	1090			1.16			Same day restrike
	12	2	1020	1028							
18E	1	2	1020	700	885	941	550	1.26	1.34	0.58	14-day restrike, 35-day restrike
	12	2	1020	805	1110	1115		1.38	1.39		14 day restrike, 106-day restrike
19E	4	2	1020	1050							
	9	2	1020	1020							

TABLE 5.2 (Continued)
Test Pile Setup
(24 Inch Piles)
For Construction Contract A

PIER NO.	PILE NO.	SOIL TYPE	ULTIMATE CAPACITY (Kips)	INITIAL DRIVE (Kips)	2 WEEK RESTRIKE (Kips)	4 WEEK RESTRIKE (Kips)	S. LOAD TEST (Kips)	2-WEEK SETUP FACTOR	4-WEEK SETUP FACTOR	S. LOAD TEST FACTOR	COMMENTS
62	6	2	880	905							Cap. at initial drive; no re-strike
63	1	2	880	900			>1186			>1.32	Cap. at initial drive; no restrike; Note 1186/900=1.32
64	1	2	880	702	888	829		1.26	1.18		Only one blow above cap.; rest. 2 wk; not representative 14-d res.
	7	2	880	700	728			1.04			
65	6	2	880	881							Cap. at initial drive; no restrike
	18	2	880	882							Cap. at initial drive; no restrike
19E	9	2	1020	1020							
66	1	2	880	950							Cap. at initial drive; no restrike
67	1	2	880	906							Cap. at initial drive; no restrike
	16	2	880	911							Cap. at initial drive; no restrike
68	1	2	880	902							Cap. at initial drive; no restrike
69	1	2	880		900						No initial value
	8	2	880		950						No initial value
	10	2	880		935						No initial value
	12	2	880	975							Cap. at initial drive; no restrike

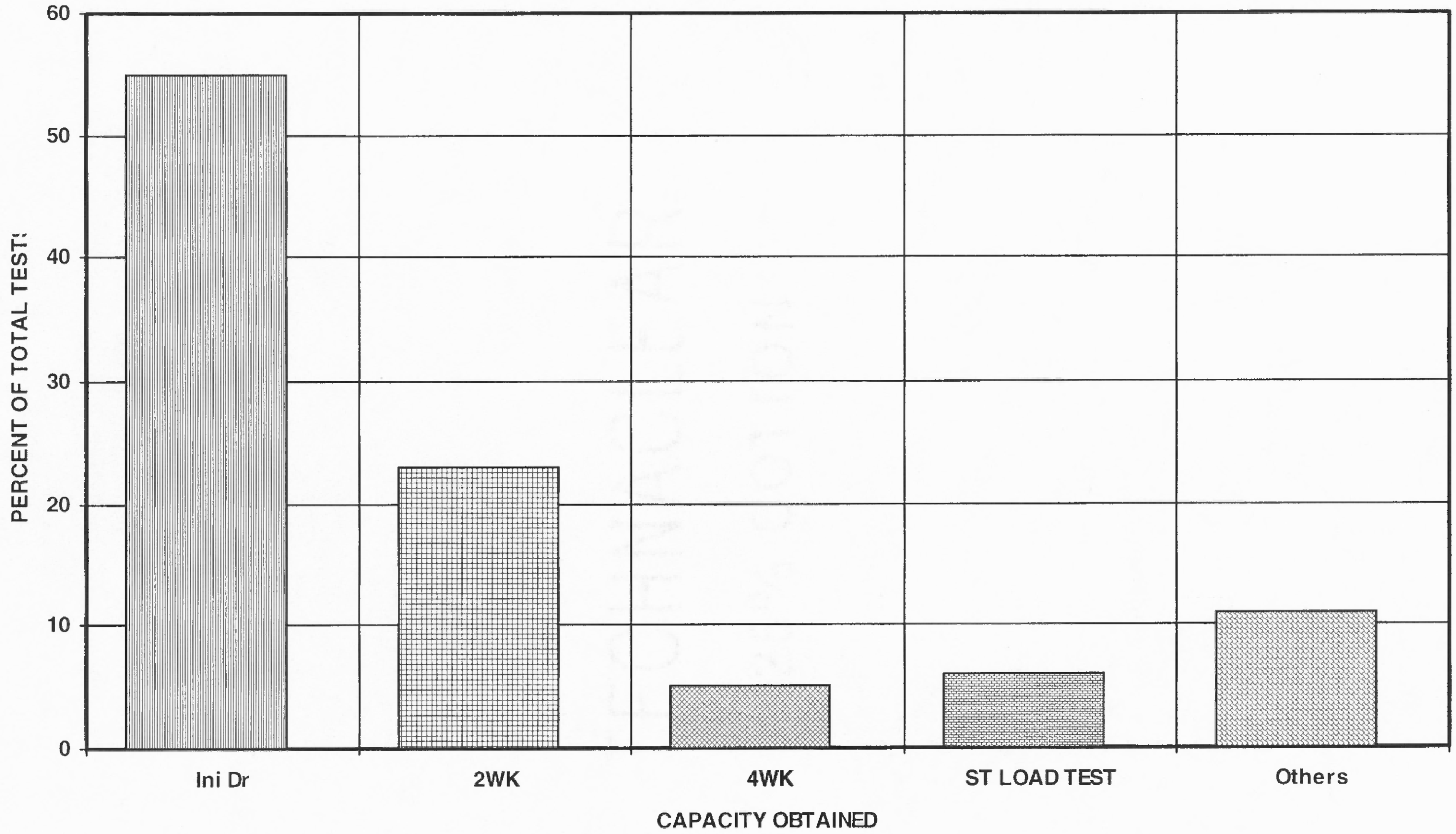


FIGURE 5.3 Pile Setup Distribution (24 Inch Piles)

Mobilized pile capacity data measured during initial driving and restriking in Soil type 2 indicates that in general the setup factor ranges from 1.0 to 1.3. It is observed that in this area most of the pile capacity is mobilized in the lower dense glacial drift deposit, or decomposed rock immediately above the bedrock. Setup did not contribute significantly to the ultimate pile capacity. This is due to the fact that the magnitude of the mobilized skin friction is low (about 10 percent) in the upper glacial deposit (Glacial Lake), therefore, the setup contribution has not added much to the overall pile capacity. Author attributes this to the presence of sand seams in the upper glacial lake deposit. This created a relatively drained condition, thus not exhibiting significant setup. This behavior has been reported by Coop, et al. (13) and Ismael (24).

The setup factor versus time plot as presented in Figure 5.4 indicates that up to a period of two weeks, the gain in capacity will be at the rate of about 1.2 percent per day. Thereafter, the rate will decrease to about 0.7 percent per day. Most of the capacity gain can be realized within two weeks period after initial driving.

In subsurface condition corresponding to soil type 3, the setup factor is very scattered. The setup factor ranged from 1.05 to 1.80 with most of the values around 1.25. The setup behavior has changed the pile capacity significantly. However, the setup behavior is not uniform. A review of dynamic load test data (PDA/CAPWAP) indicates a very interesting observation in this area that the soil setup did not become significant until a depth of about 110 feet. Beyond this depth a significant amount of soil setup is observed. The setup factor up to 110 feet depth averaged 1.50 with a scattered value of 1.1 to 2.1. Whereas, the setup factor in the lower portion averaged 3.0 with a scattered value of 1.5 to 5.70. A

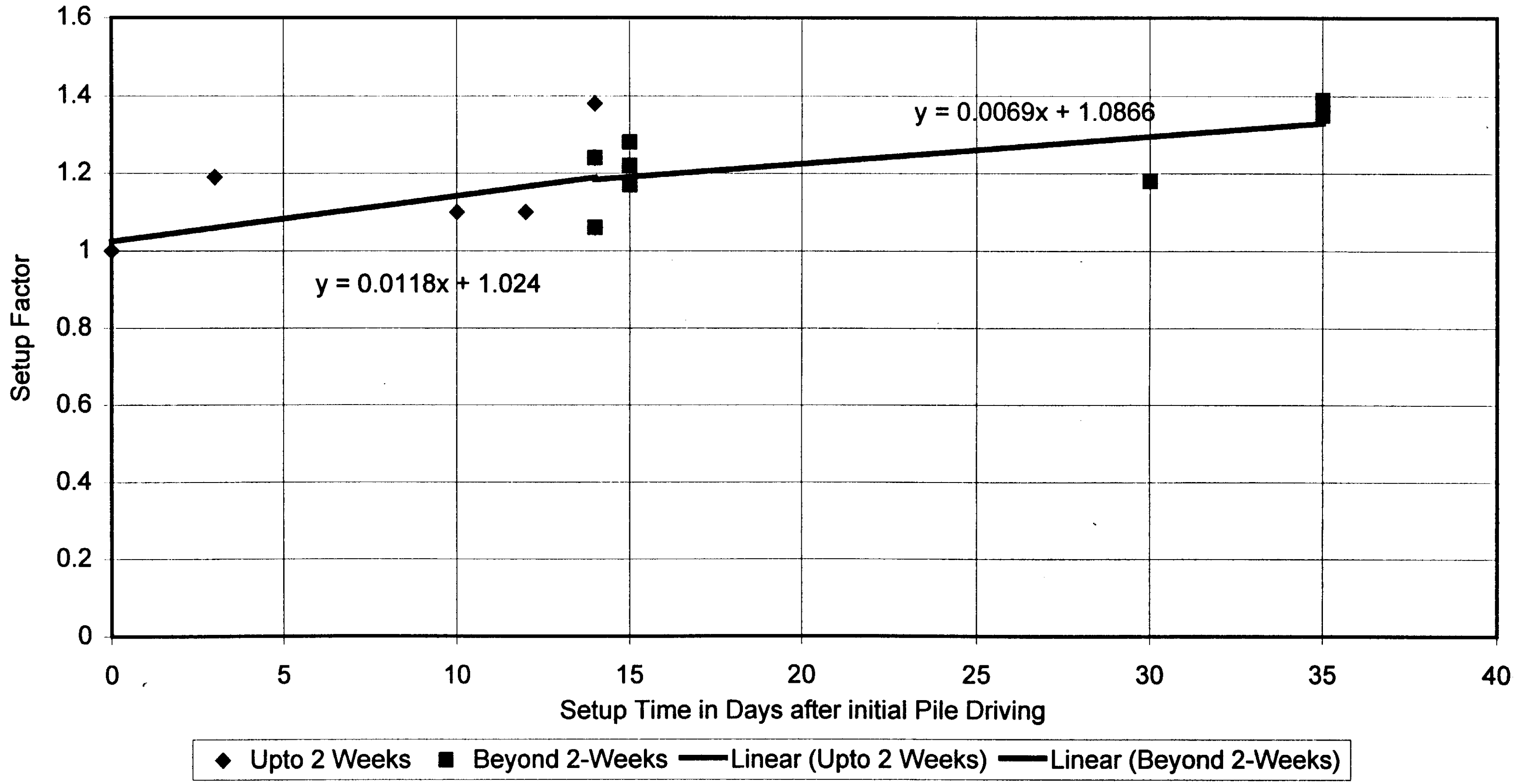


FIGURE 5.4 Setup Factor Versus Time for 24 Inch Pile (Soil Type 2)

close observation of the subsurface condition indicates that the soil has low shear strength properties up to a depth of about 110 feet. In order to develop significant setup, the piles needed to penetrate significantly into the lower dense glacial deposit. Similar observations have been reported by Attwoll, et al. (6). A best fit of curve of soil setup versus time as presented in Figure 5.5 indicates that for up to a two week period, the gain in pile capacity can be realized at a rate of 1.4 percent per day, thereafter the gain can be less than one half percent. The setup factor data for the two week period is scattered. This indicates the nonhomogenity of the underlying lower glacial deposits. Depending upon the subsurface conditions, a substantial setup can be realized within the two week period.

The driving and restriking data has indicated that practically little setup occurred in Soil type 4. Rather some piles exhibited reduction in pile capacity in this area. The setup factor ranged 0.86 to 1.3 with most of the value near 1.0. The setup factor versus time plot as presented in Figure 5.6 indicates that in general, setup was not substantial in this area. Based on this best fit of the curve, the gain in pile capacity can be at the rate of less than half percent per day for a period of two weeks, thereafter, practically no gain in capacity can be realized. The reduction in pile capacity may be due to the inaccuracy of the pile capacity prediction by dynamic test results or due to relaxation. Dense saturated silty sand encountered in this area may develop negative pore pressure during pile driving due to dilation of the dense granular sandy materials. Also, significant amount of gravel and shale fragments present in the lower glacial deposit material may also develop negative pore pressure due to gravel dislodging and swelling of fragmented shale particles. In shale

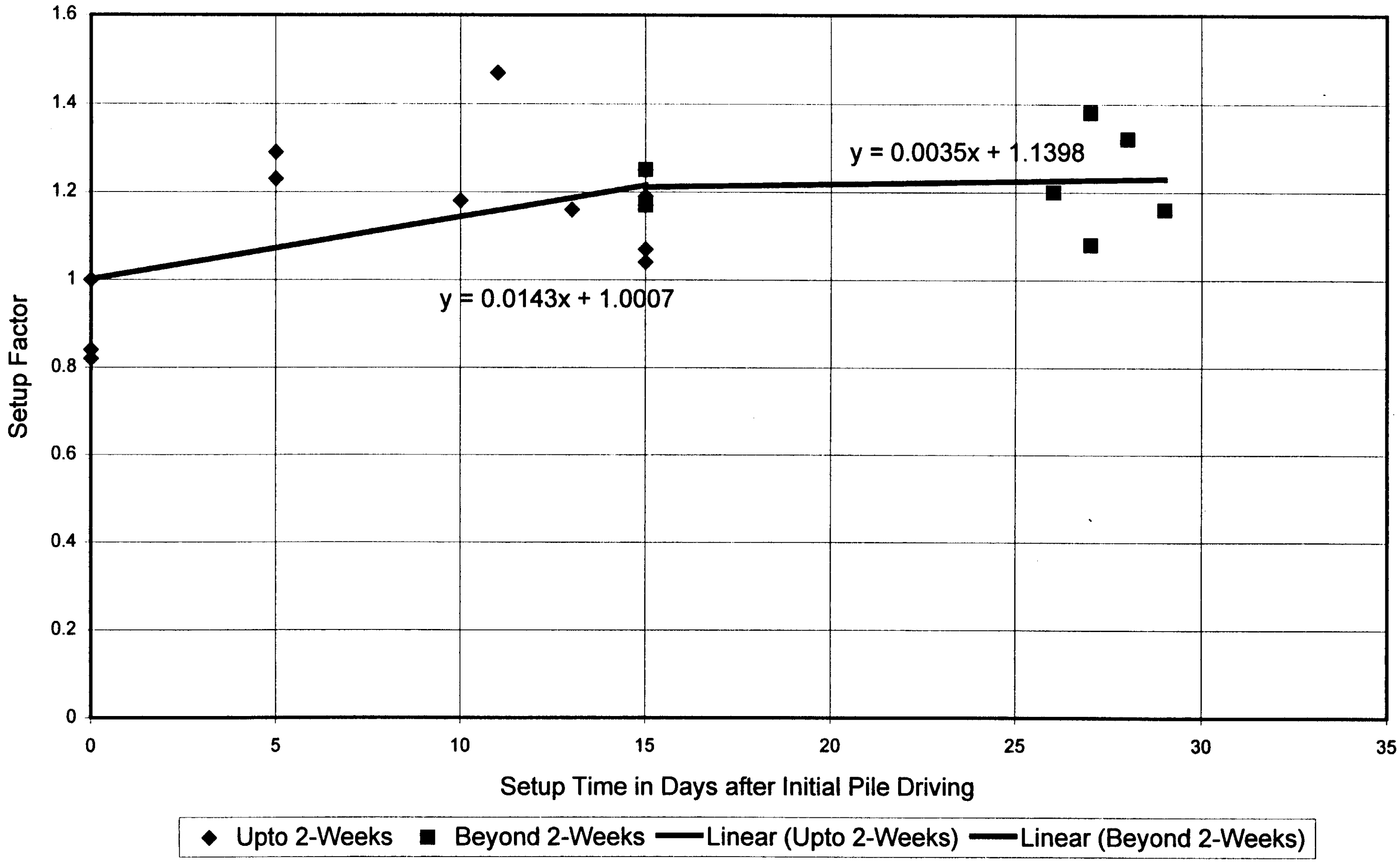


FIGURE 5.5 Setup Factor Versus Time for 24-Inch Piles (Soil Type 3)

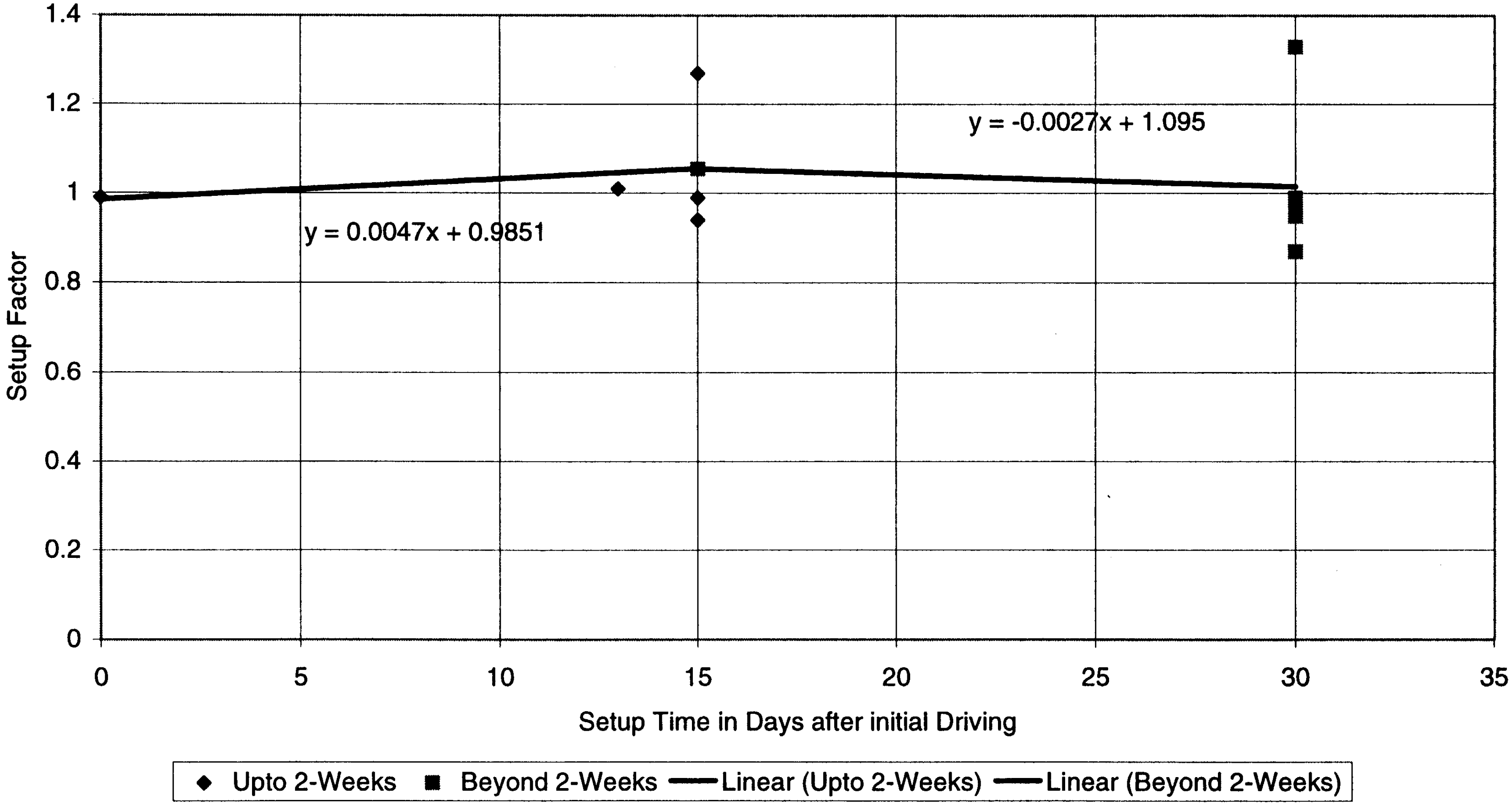


FIGURE 5.6 Setup Factor Versus Time For 24-Inch Piles (Soil Type-4)

bedrock media relaxation can occur as reported by Samon and Authier (45). In soil type 4 area, very little clay was present, therefore, not exhibiting setup behavior in this area.

Analysis of restriking data shows a reduction in the driving resistance indicated by observing a reduced hammer ram stroke. This behavior is due to a lower resistance on restrike offered by soil. This problem occurs in diesel hammer when the hammer is not warm or preignition occurs in the chamber. This phenomenon could have happened in other soil types. But, field notes indicate that this occurred only in soil type 4. The lower resistance during restriking may also indicate that gravel dislodging and/or shale fragment swelling reduced the soil shear strength some time after pile driving.

5.3 Static Load Test Results

Nine static load tests were conducted within the construction Contract A to verify the pile capacity determined by dynamic tests. Two tests in Subsurface Condition Soil type 1, four tests in Soil type 2, two tests in Soil type 3 and one test in Soil type 4 were conducted.

A summary of the load test results are presented in Table 5.3 to discuss the evaluated capacity in relation with those predicted by other methods along with designed ultimate capacity.

TABLE 5.3

Summary of Static Load Tests

Designated Subsurface Area	Pier Designation	Pile Size (inch)	Tip Elevation	Design Ultimate Capacity (Kips)	Predicted Capacity Kips		
					WEAP	PDA/ CAPWAP	Static Load
Soil Type 1	PW10W	18	-85.3	496	610	611	660
"	PW15W	18	-70.2	496	550	550 (I)	670 (M)
Soil Type 2	18E	24	-92.5	1020	1040	941	560
"	63	24	-115.1	880	940	900 (I)	1200 (M)
"	29S	24	-118.0	1032	1110	1090	1390
"	32N	24	-131.8	1000	1110	1000 (I)	1350 (M)
Soil Type 3	36N	24	-146.1	968	1200	1160	1300 (M)
"	37S	24	-130.7	968	900	989	1200
Soil Type 4	42N	24	-116.0	1032	730	810 (I)	1000 (M)

Note: I - Indicates during initial driving

M - Maximum load applied

In some static load tests, the plunging failure criteria as developed by Davisson's method were not defined. In this case, the maximum test load is presented in this table. The PDA and CAPWAP value did not differ much, therefore, in this column a most representative value is presented.

As indicated in Table 5.3, the pile capacity predicted by the static load test results is higher than that predicted based on WEAP and dynamic load tests except at Pier 18E. The above table also indicates that in general the pile capacity predicted by wave equation analysis is a little higher than that predicted by dynamic tests. When comparing the values from static load test to the WEAP analysis, the WEAP analysis under predicted the pile capacity ranging from 8 to 37 percent for a 24-inch diameter pile, and 8 to 22 percent for an 18-inch diameter pile. In the same fashion, the dynamic test results under predicted the capacity by 12 to 35 percent for a 24-inch diameter pile and 8 to 22 percent for an 18-inch pile. This type of variation is not unusual.

This table also indicates a lower variation at Pier PW10W located in subsurface condition Soil type 1 and Pier 36N located in Soil type 3. In order to evaluate this behavior at these locations, the subsurface condition at the pier locations were studied very closely.

The subsurface condition information along I-78 westbound at the static load test location PW10W and PW15W indicates that the depth and thickness of underlying glacial till layer is very irregular. At PW10W, the lower glacial deposit starts at a depth of about EL -67, whereas at PW15W the deposit starts at a depth of about EL -45. The depth of the bedrock is also very irregular. Therefore, it is obvious that in order to achieve the same capacity, the pile at PW10W has to penetrate deeper. The density of the glacial deposits underlying the pile tip at PW10W is much lower than that at the PW15W location. Based on these observations, it is believed that a much higher capacity should be mobilized at PW15W. This is confirmed by the static load test results. The load deformation plot (Figure 5.7) for static test conducted at PW10W location indicates sharp plunging at about 300 ton capacity. But, this behavior was not observed at PW15W location as presented in Figure 5.8. Based on this observation, it is concluded that the comparatively lower capacity

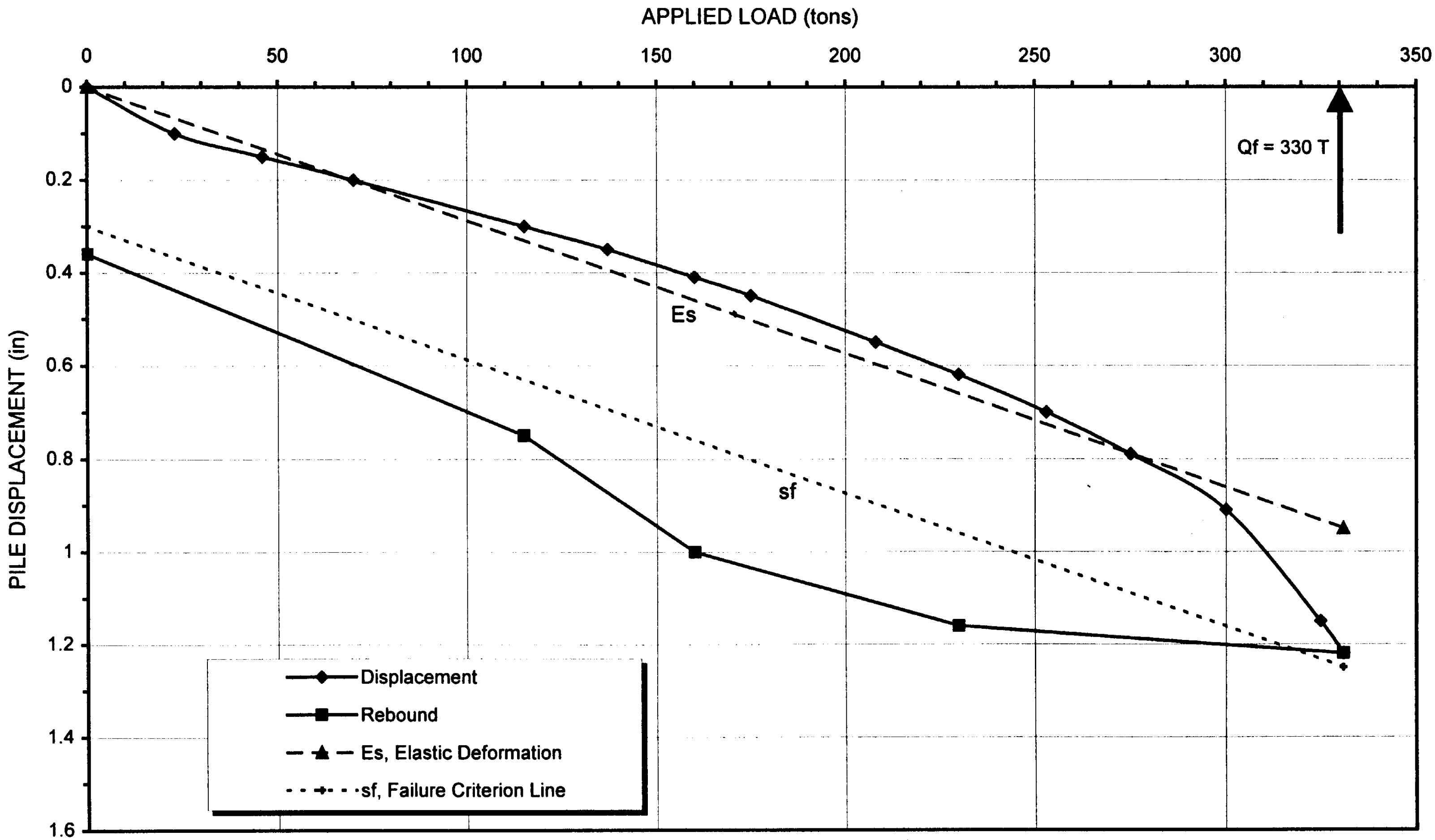


FIGURE 5.7 Static Load Test for Pier 10W, Pile #2

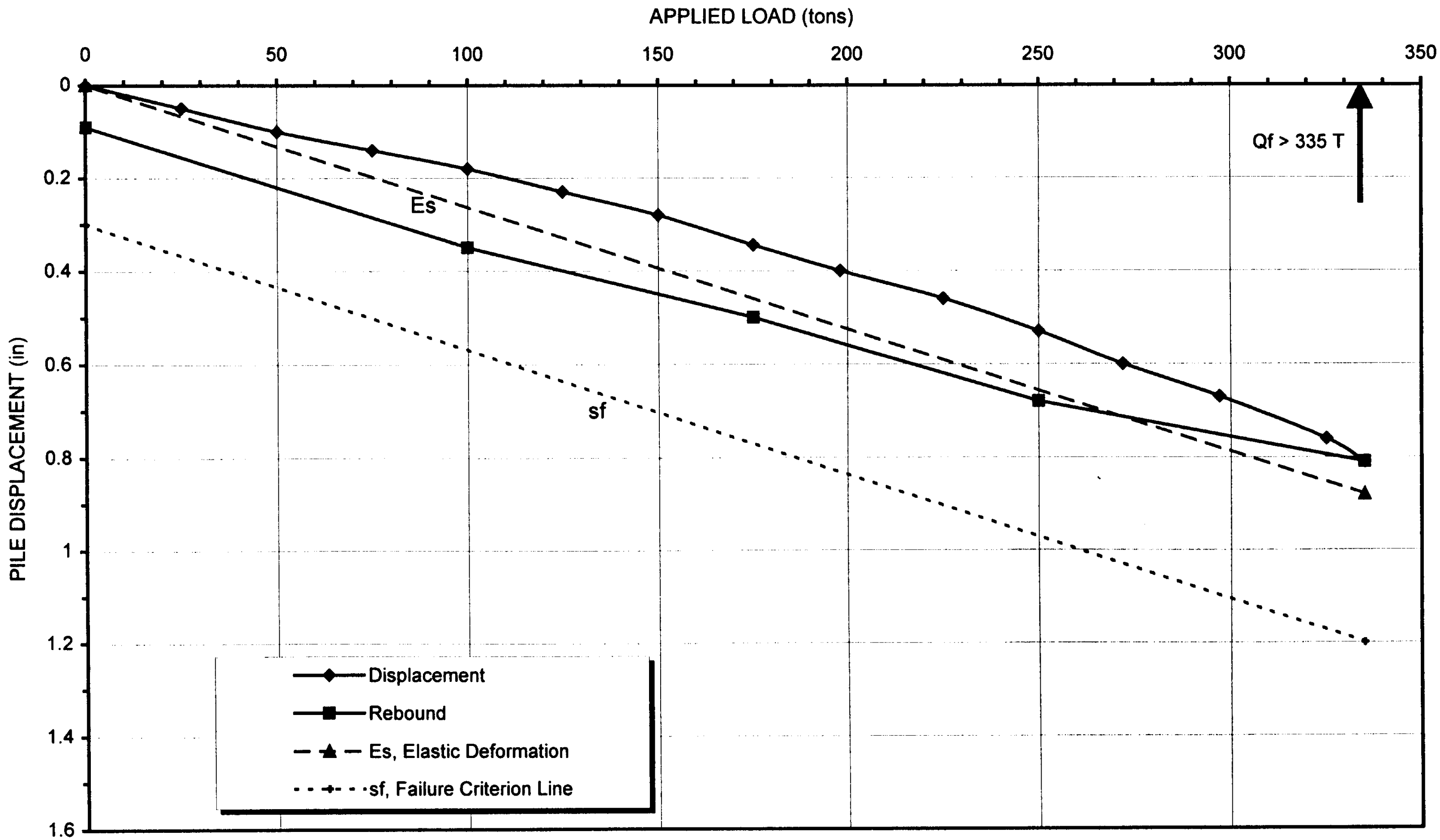


FIGURE 5.8 Static Load Test for Pier 15W, Pile #8

attained at PW10W is predominantly contributed by the density and thickness of the underlying materials below the pile tip.

Another interesting observation is inferred from the dynamic load test results. A lower capacity is obtained at PW15W location by PDA/CAPWAP analysis. This is probably due to the reason that the PW15W pile is shorter and has mobilized smaller skin friction contribution than the PW10W pile.

The subsurface condition near Pier 36N indicates the presence of predominantly gravels at the pile tip elevation. The static load test results as presented in Figure 5.9 indicate that a much higher capacity can be obtained at this location if Davisson's failure criteria is established. Therefore, the comparison of static load tests with those of dynamic load test results would be misleading. It is believed that during driving the gravels have displaced and the surrounding material is densified. During dynamic tests the soil resistance is not realized due to nonhomogenity of the soil present. However, during static load test, densified material indicated higher mobilized capacity. It is concluded that at this location, the pile could have been much shorter.

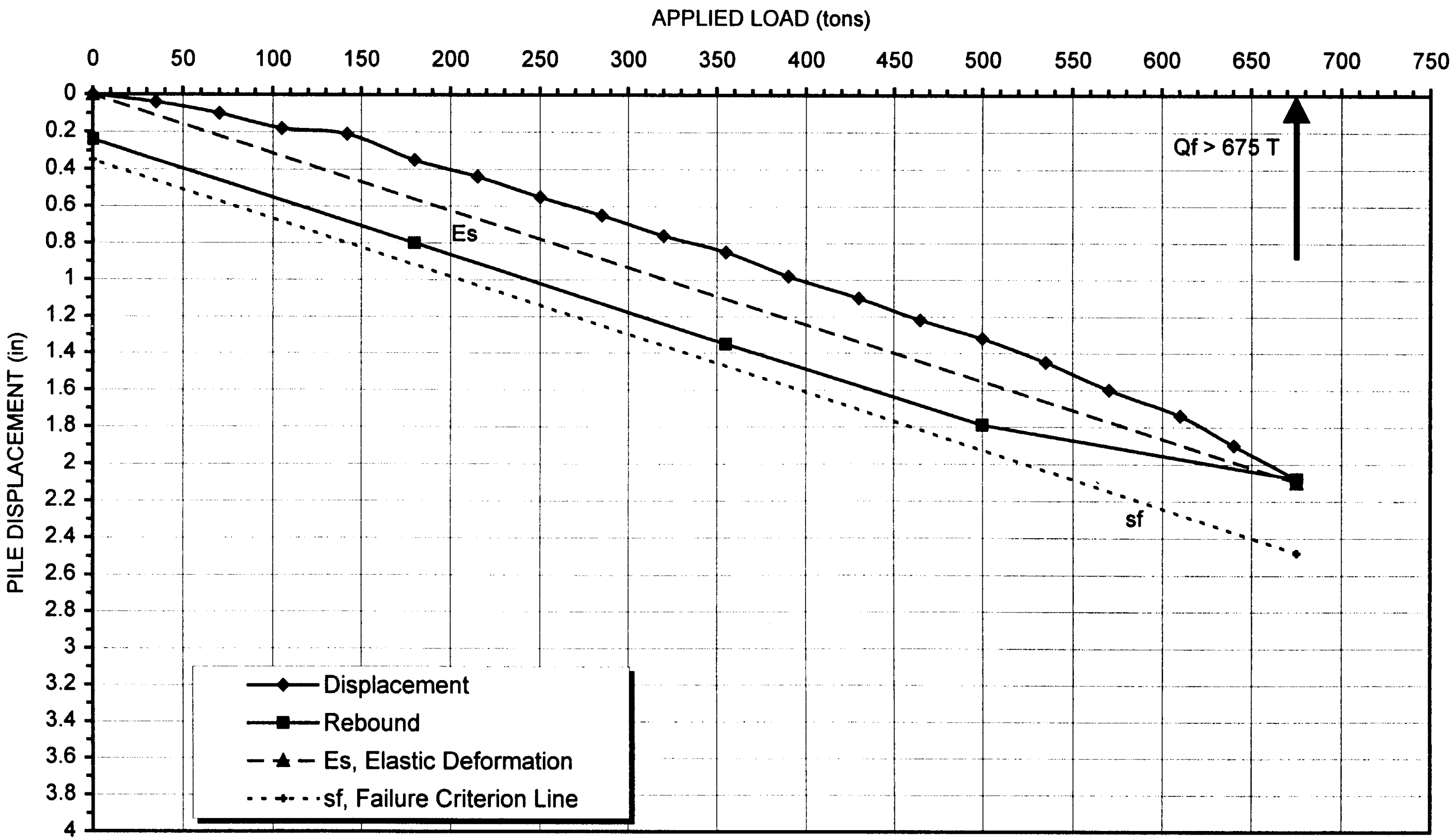


FIGURE 5.9 Static Load Test for Pier 32N, Pile #1

A very unusually low value of pile capacity was indicated at Pier 18E by the static load test as presented in Figure 5.10. In order to evaluate the probable cause for this behavior, a plot of driving resistance with depth and SPT N-values with depth was prepared for this location. The plots are presented as Figures 5.11 and 5.12 in this section. These plots indicate a rapid increase and then rapid decrease in the pile driving resistance and SPT N-values at about 95 to 100 feet below the grade which is approximately the tip of the tested pile. The rapid increase and decrease in the driving resistance indicates the presence of a hard thin layer. The boring log indicates fine grained soils with high SPT N-values underlain by the dense layer. Based on these observations, it is believed that during static load test the pile punched through the thin dense layer into the underlain fine grained soil resulting in a lower value of pile capacity. However, from the dynamic test results, the pile capacity was mobilized at the dense thin layer, resulting in a higher pile capacity. The results of this load test further justifies the importance of a static load test to verify pile capacity. In a variable soil condition, it is important to evaluate the dynamic test results in conjunction with the subsurface soil conditions to predict the true pile capacity, otherwise misleading results could be inferred. Based on the soil boring data and pile driving record, if the presence of a thin layer is detected, the capacity should be confirmed by a static load test.

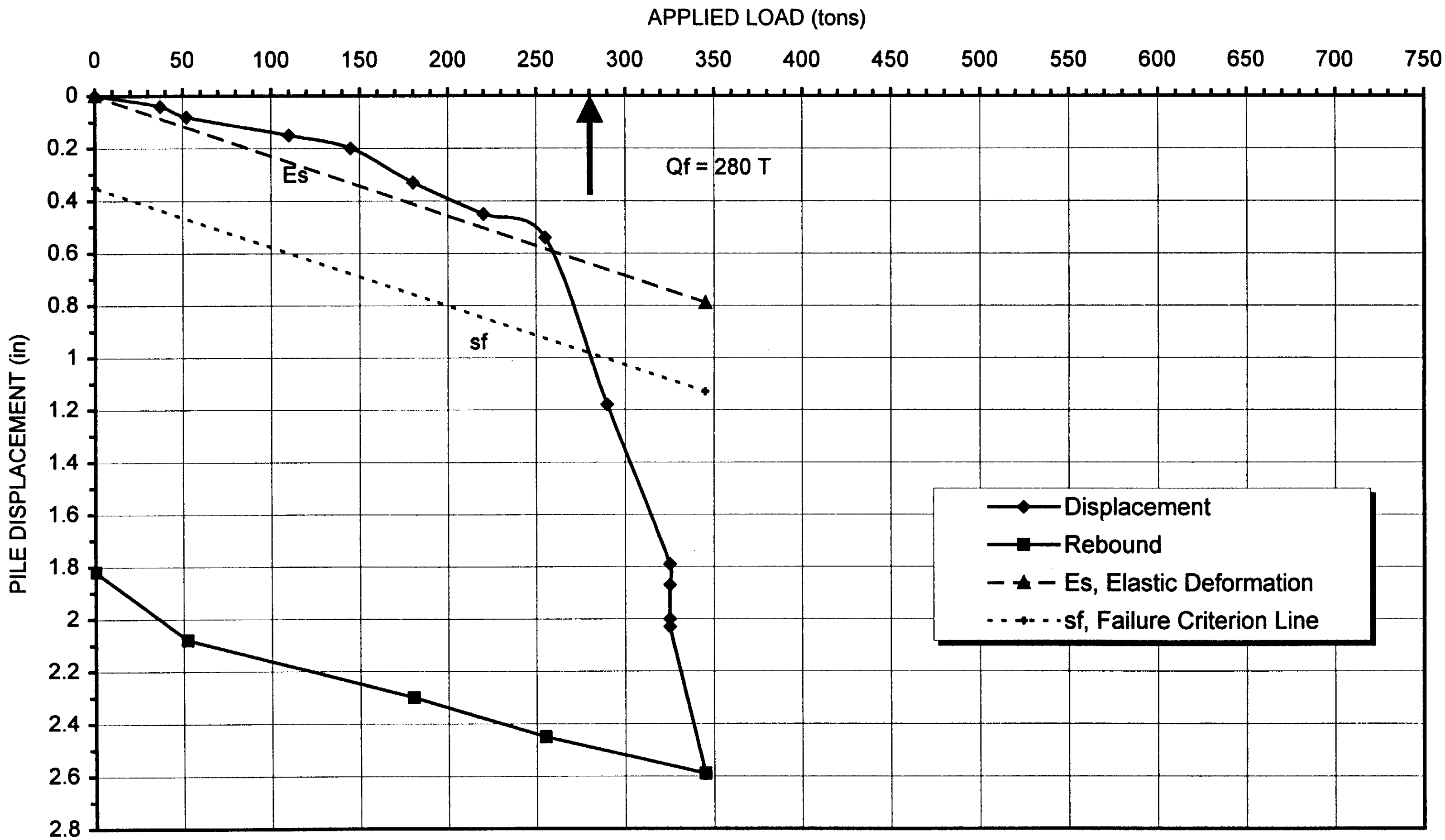
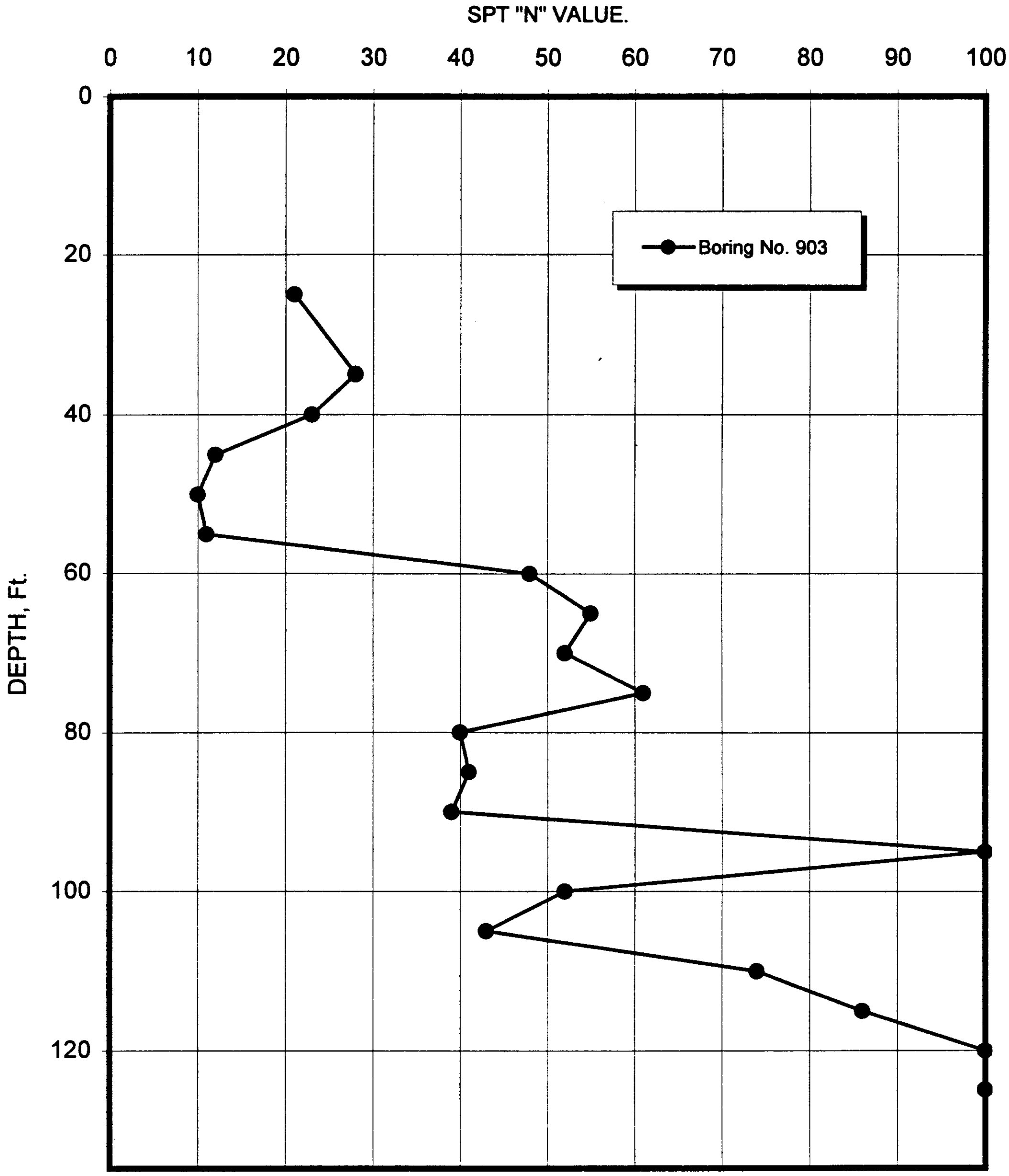
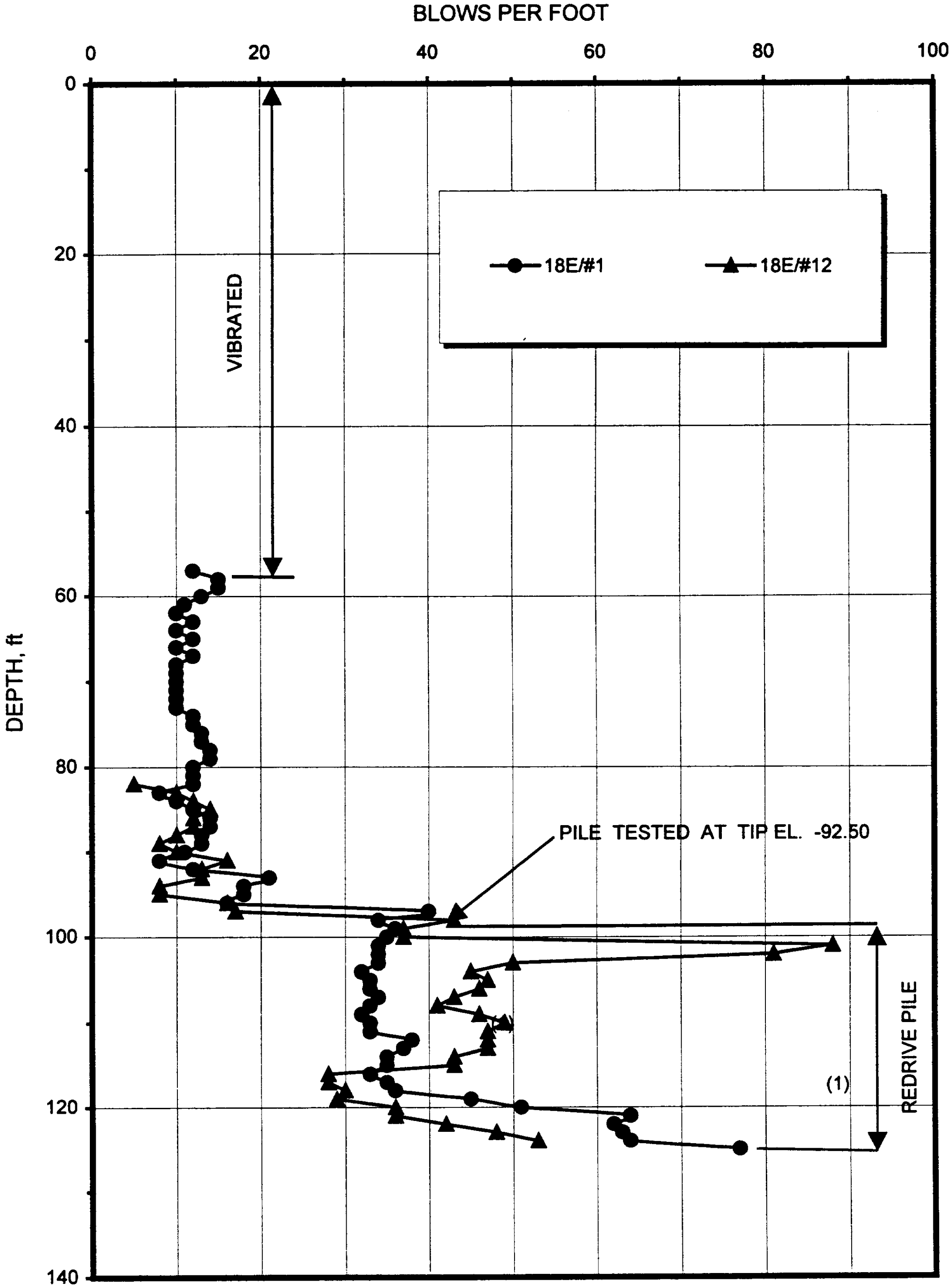


FIGURE 5.10 Static Load Test for Pier 18E, Pile #1



NOTE: Points shown on 100 SPT N line correspond
to 100 Blows/ ft or more.

FIGURE 5.11 SPT N-Value vs Depth for Soil Type 2 (Pier 18E)



(1) RESTRIKE

FIGURE 5.12 Pile Driving Resistance Versus Depth for Soil Type 2

5.4 Mobilization of Pile Capacity

Mobilized ultimate capacity of a test pile was established based on the dynamic test results in coordination with static load test results. Dynamic test results include the results obtained from PDA and CAPWAP analysis either during initial driving or restriking.

Percentage of mobilized total pile capacity varied considerably in the four subsurface conditions categorized for this Contract A area. Amount of mobilized skin friction as well as end bearing also varied significantly. These variations were due to the large variability of the subsurface conditions. Even within a soil area, the pile length and the tip elevation to mobilize the required pile capacity varied significantly.

5.4.1 Skin Friction and End Bearing

CAPWAP analysis yielded the values of mobilized skin friction and end bearing at the tested piles. Required capacity was not mobilized unless the piles significantly penetrated into the lower glacial deposit. Maximum value of skin friction was observed at or near the tip of pile. Similar observations have been reported by Randolph, et al. (40).

18-inch Diameter Pile:

A summary of all the dynamic test results was prepared and presented in Table A.2, Appendix A. During initial driving, the mobilized ultimate capacity along eastbound was generally lower than that along the westbound. The mobilized ultimate capacity along westbound is in the range of about 400 to 500 Kips, whereas along eastbound it is in the range of 300 to 350 Kips. So more setup has occurred along the westbound. Plots of mobilized skin friction versus depth at the initial drive and restrike for all dynamically tested piles are presented in Figures 5.13 and 5.14, respectively. Figure 5.15 indicates the mobilized skin friction for piers where capacity was obtained during initial driving. In these plots the friction resistance mobilized at the end of initial driving is presented as EOD, whereas, BOR value indicates the capacity at the beginning of restrike. CAPWAP analysis was performed for the blow count for EOD and BOR. These plots typically define the trend of the mobilized skin friction. The lower, upper and middle lines simply define the most frequently observed trends with depth. There are some scattered data. This is believed to be due to the variations in the site conditions. These plots also indicate that a significant amount of pile capacity and setup behavior is occurring in the lower dense glacial deposit. A significant number of piles located along the eastbound I-78 roadway mobilized the capacity in initial driving at around the estimated tip elevation.

In general, piles were to be driven below the thin hard layer to achieve the required capacity. A plot of pile driving resistance with depth and SPT N-values (selected borings) with depth as presented in Figures 5.16 and 5.17, respectively indicate the presence of a thin hard layer. In order to achieve the pile capacity for a group of piles it was believed that this thin layer needed to be penetrated. An additional dynamic load test PW9W was performed to confirm this assumption. The production pile driving criteria in this area included a minimum tip elevation and sustained driving requirements to eliminate the possibility of plunging through this thin hard layer for a group of piles.

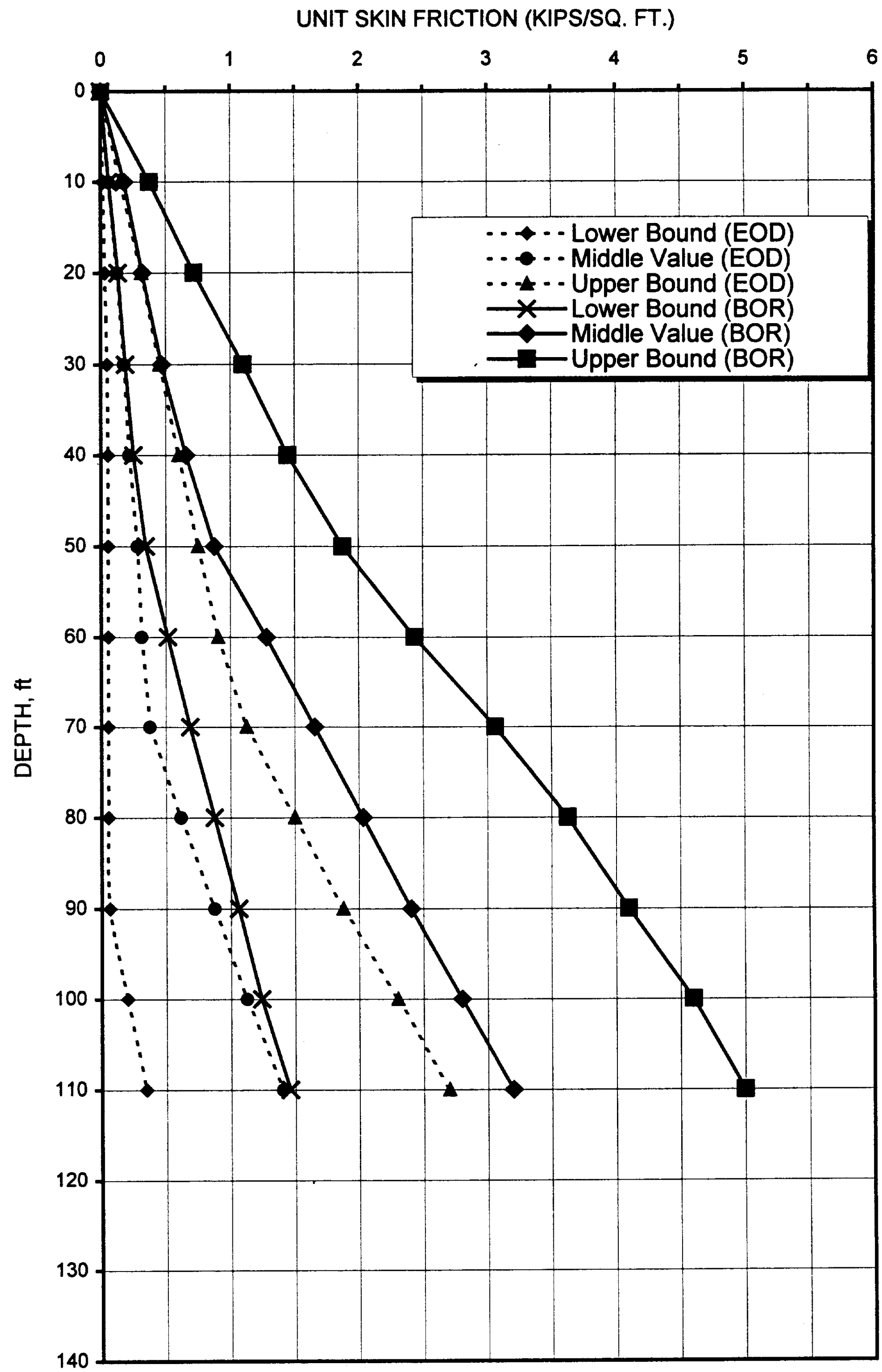


FIGURE 5.13 Unit Skin Friction vs Depth (EOD & BOR) Soil Type 1 (Piers PW1E to PW13E, PF6E, East Abutment)

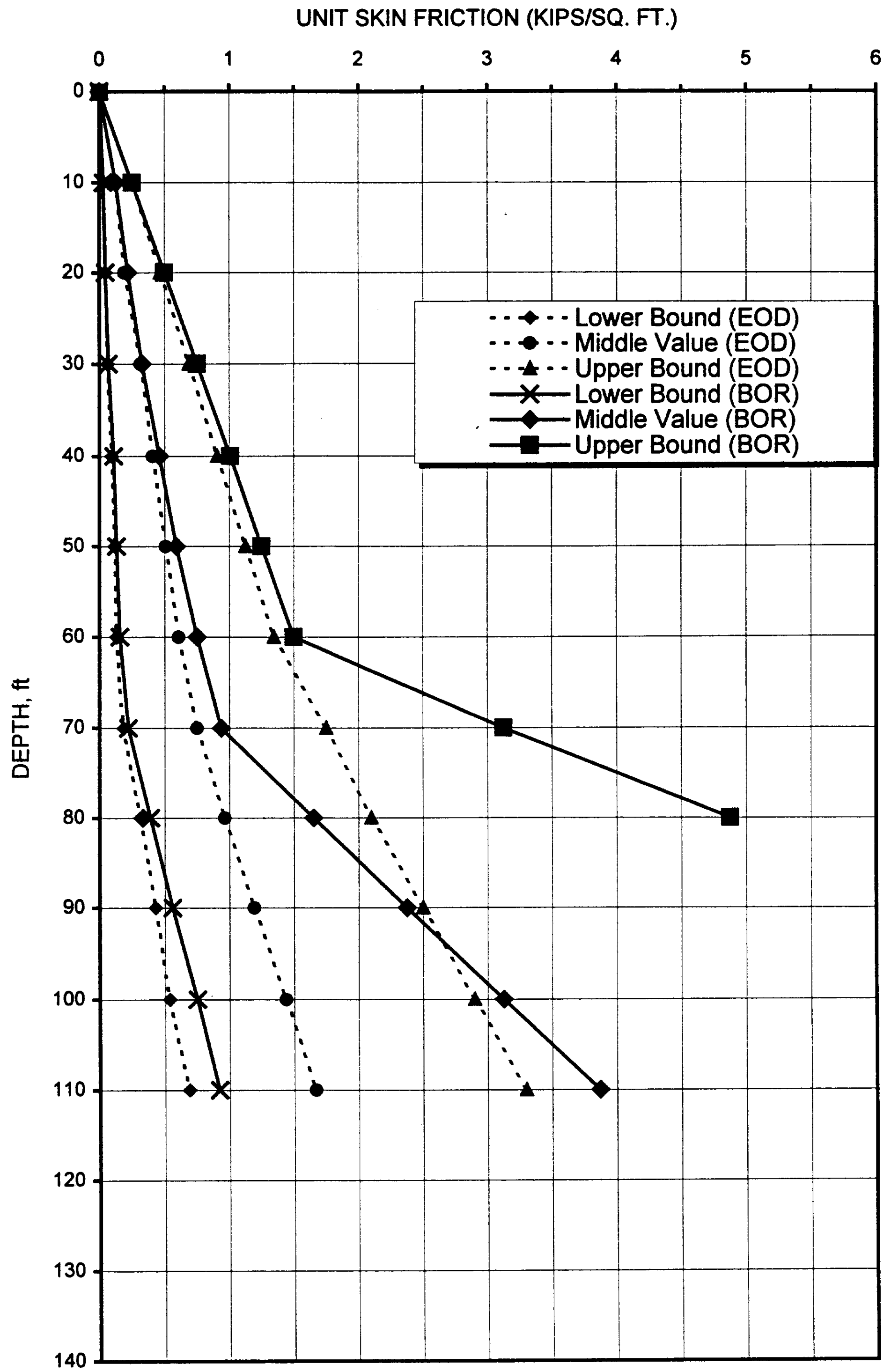


FIGURE 5.14 Unit Skin Friction vs Depth (EOD & BOR) Soil Type 1
(Piers PW1W to PW16W)

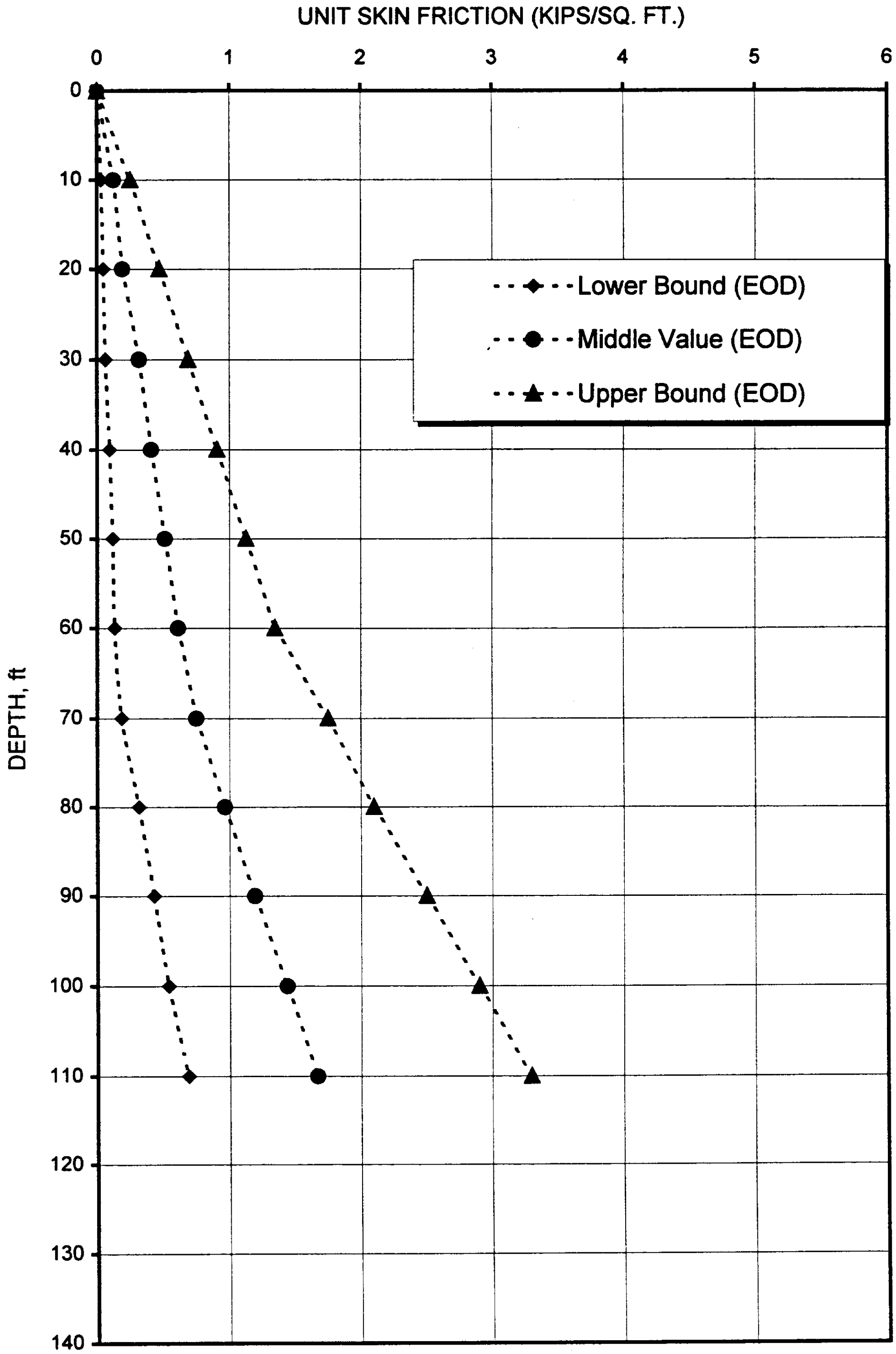
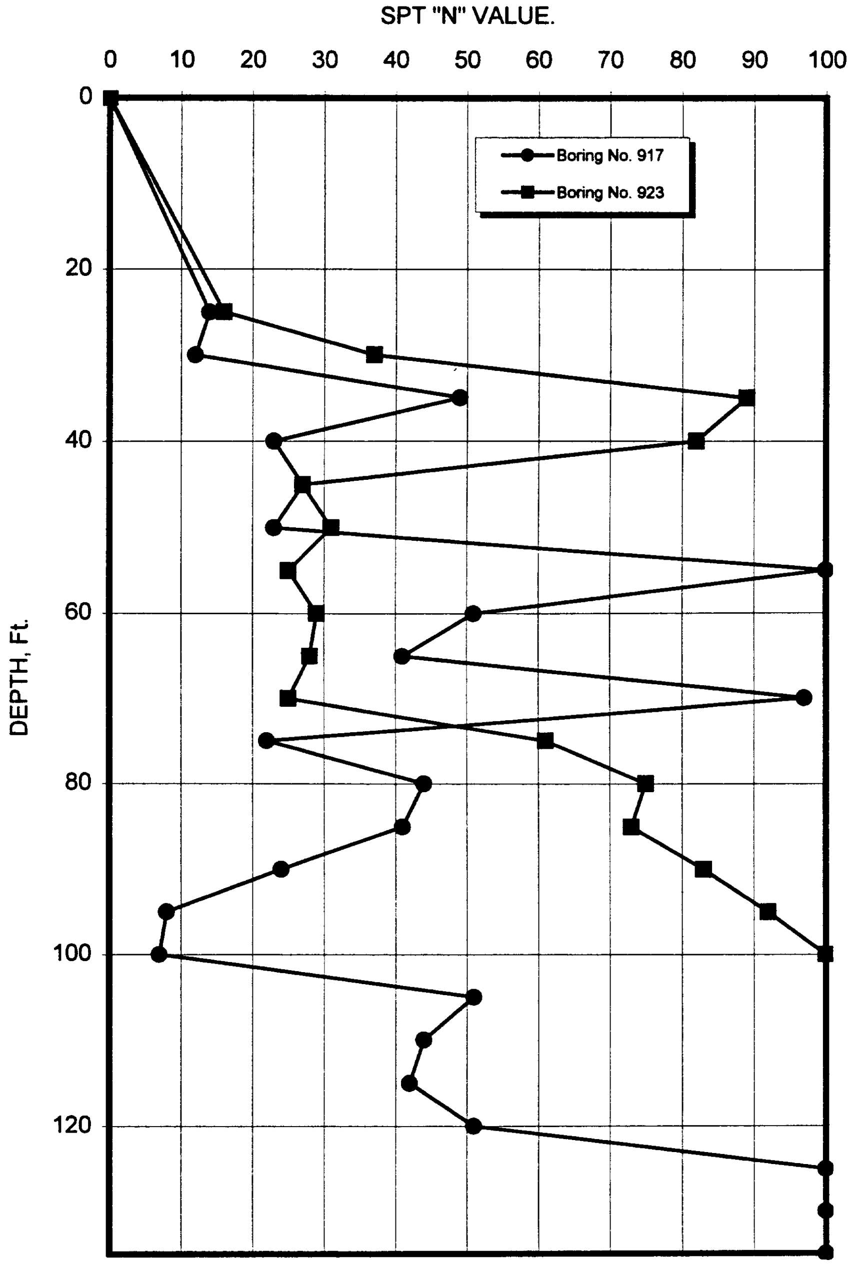


FIGURE 5.15 Unit Skin Friction vs Depth (EOD) Soil Type 1
(Piers 14E to 16E)



NOTE: Points shown on 100 SPT N line correspond to 100 blows/ 1 ft or more.

FIGURE 5.16 SPT N-Value vs Depth for Soil Type 1

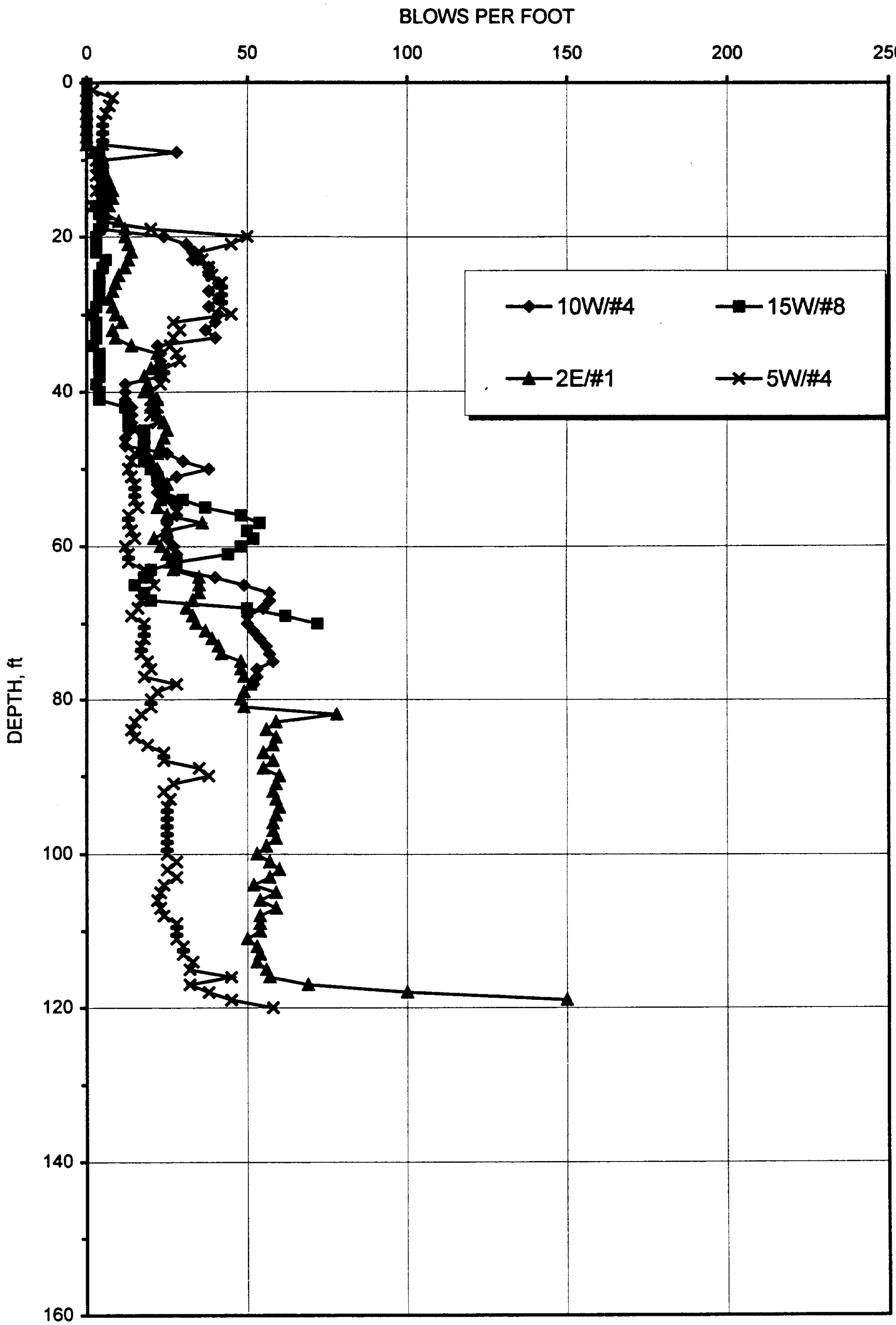


FIGURE 5.17 Pile Driving Resistance vs Depth for Soil Type 1

24-inch Diameter Pile:

A 24-inch diameter pile was driven in Soil types 2 to 4. The plots for the mobilized skin friction with depth in initial driving and restriking are presented in Figures 5.18 to 5.20. These plots for Soil type 2 area indicate that the driving resistance did not increase significantly unless the piles penetrated deep enough into the lower glacial deposits. The CAPWAP results indicate that about 10 percent of pile capacity is mobilized by skin friction in the top glacial deposits ranging from 60 to 100 feet deep below grade. The pile capacity did not mobilize significantly unless the pile reached to depth 90 to 120 feet. Most of the pile capacity ranging about 70 to 80 percent was mobilized by end bearing during the initial driving. A slight decrease in the end bearing was also observed at restrike. After soil setup the end bearing value ranged about 60 to 70 percent. The high percentage of end bearing is attributed to the presence of hard lower glacial deposits and the presence of bedrock underlying this layer.

In Soil type 3, most of the capacity was mobilized below depth of 90 to 120 feet of pile penetration. The lower glacial deposit is thicker. Piles needed to penetrate deeper in lower glacial deposit to achieve the pile capacity. It is also observed that the piles which mobilized capacity during the initial driving exhibited end bearing in the range of 60 to 75 percent. Piles which did not mobilize the capacity in initial driving had to penetrate 25 to 30 feet more into the lower glacial deposit. This lower glacial deposit exhibited more soil setup behavior. After the soil setup, the end bearing capacity of piles decreased and determined to be in the range of 40 to 50 percent.

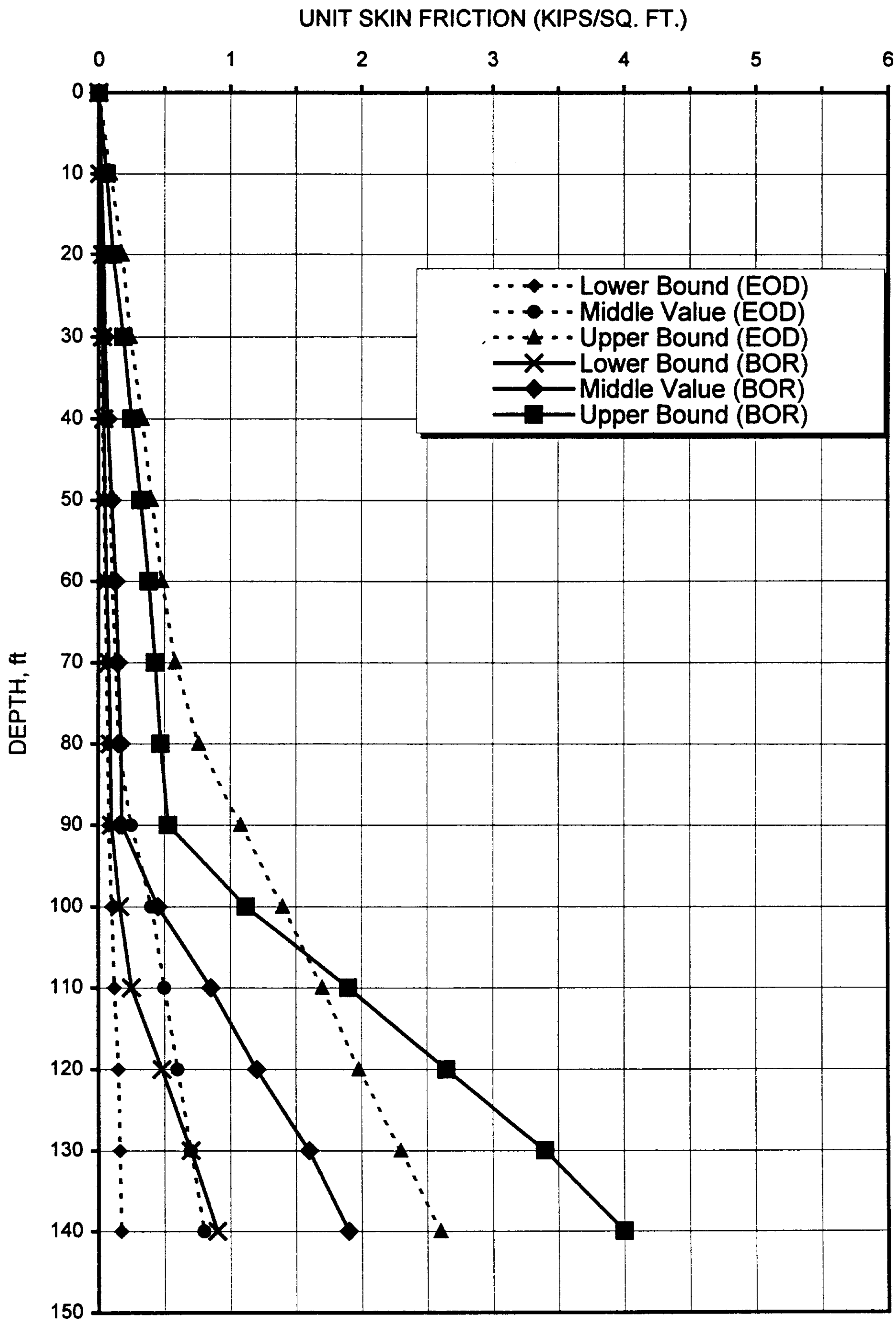


FIGURE 5.18 Unit Skin Friction vs Depth (EOD & BOR) Soil Type 2

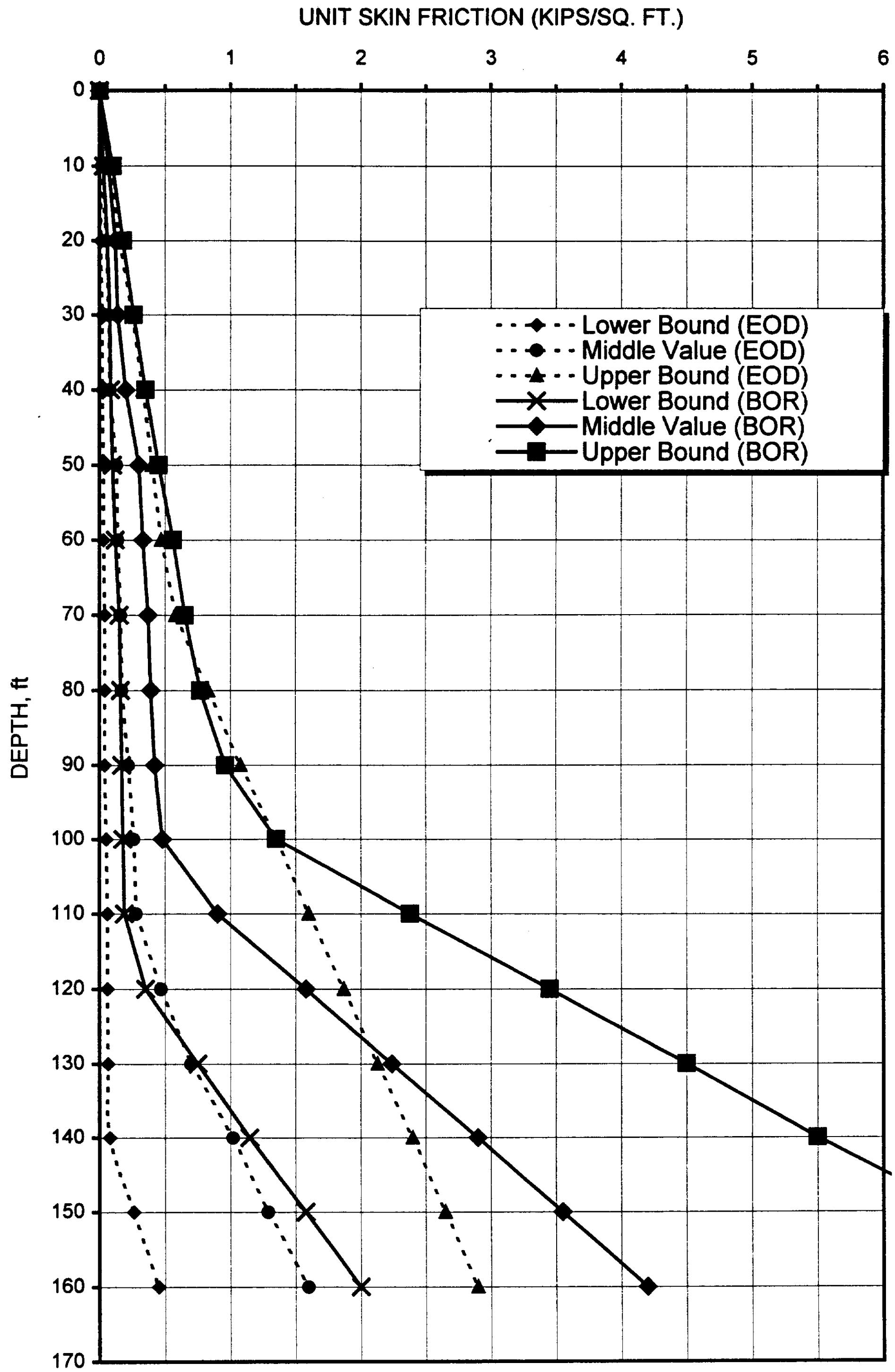


FIGURE 5.19 Unit Skin Friction vs Depth (EOD & BOR) Soil Type 3

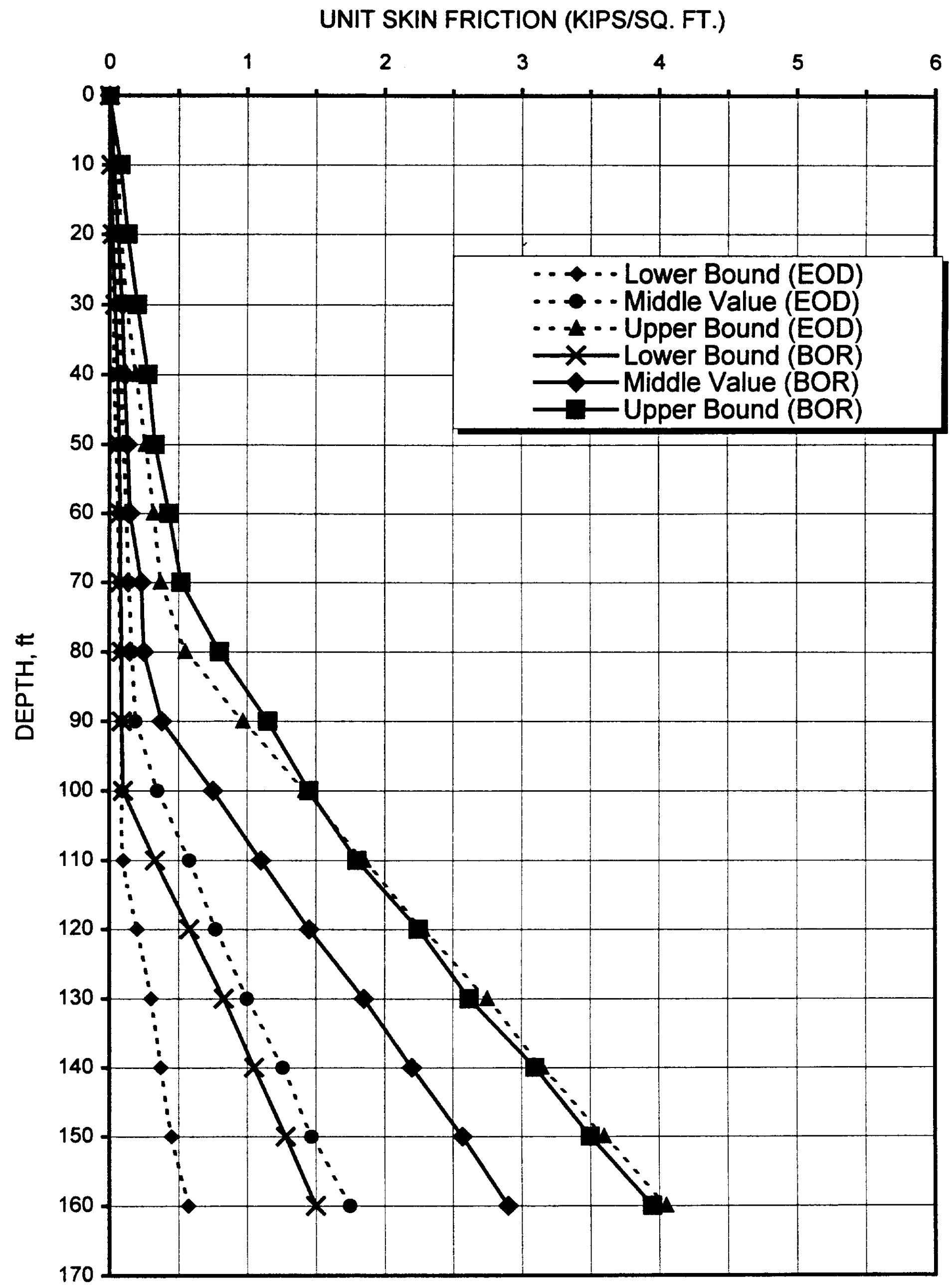
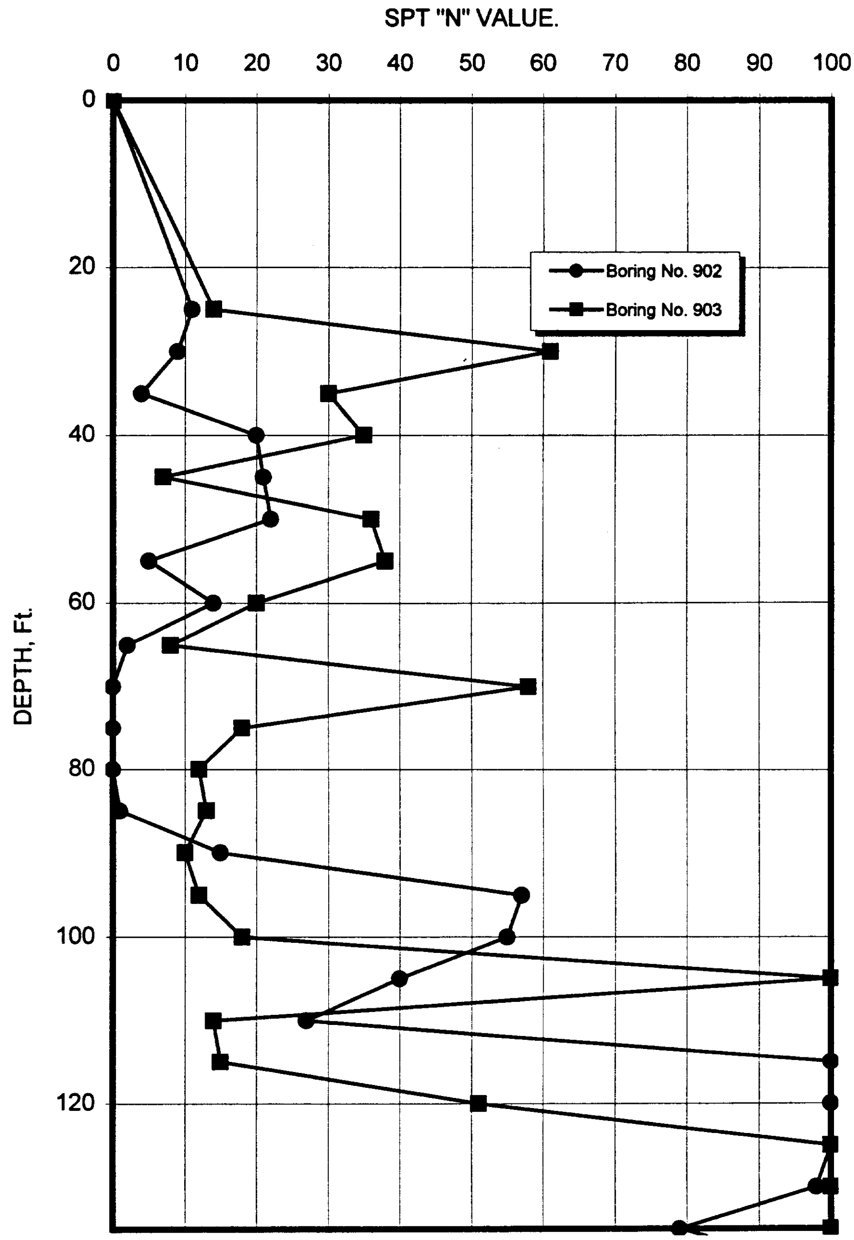


FIGURE 5.20 Unit Skin Friction vs Depth (EOD & BOR) Soil Type 4

The decrease in end bearing during restrike in the upper two types of soil is related with the mechanism of soil resistance mobilization around the pile. After soil setup, more frictional resistance is realized during restrike, which restricts the movement of pile. The pile does not move enough to mobilize the full end bearing. Relatively, a higher pile movement is required to mobilize end bearing value.

A plot of driving resistance with depth and SPT N-values with depth (selected borings and piles) is presented in Figures 5.21 to 5.24. As indicated from SPT N-value data, a generally high value is observed randomly throughout the deposit, resulting in variations in the driving resistance with depth. These variations often occurred over a short distance or even within the area of footing limits.

SPT N-values with depth are plotted from the selected borings for Soil type 4 and presented in Figures 5.25 and 5.26, respectively. The boring data has indicated a significant amount of gravel, boulders and shale fragments to a depth ranging from 120 to 150 feet. In general, higher SPT N-values were observed in this area due to the presence of gravels. However, the required pile capacity was not mobilized in the upper glacial layer unless the pile penetrated through the gravels and boulders layer. Two test piles were damaged while penetrating through the gravel layer. The required pile capacity was revised to be 80 percent of the original capacity. Then the revised pile capacity could be achieved at the gravel and boulder layer without the possibility of damaging the pile. Plot of mobilized skin friction with depth indicates that in general, the soil setup behavior has not occurred significantly in this area.



NOTE: Points shown on 100 SPT line correspond to 100 Blows/ ft or more.

FIGURE 5.21 SPT N-Value vs Depth for Soil Type 2

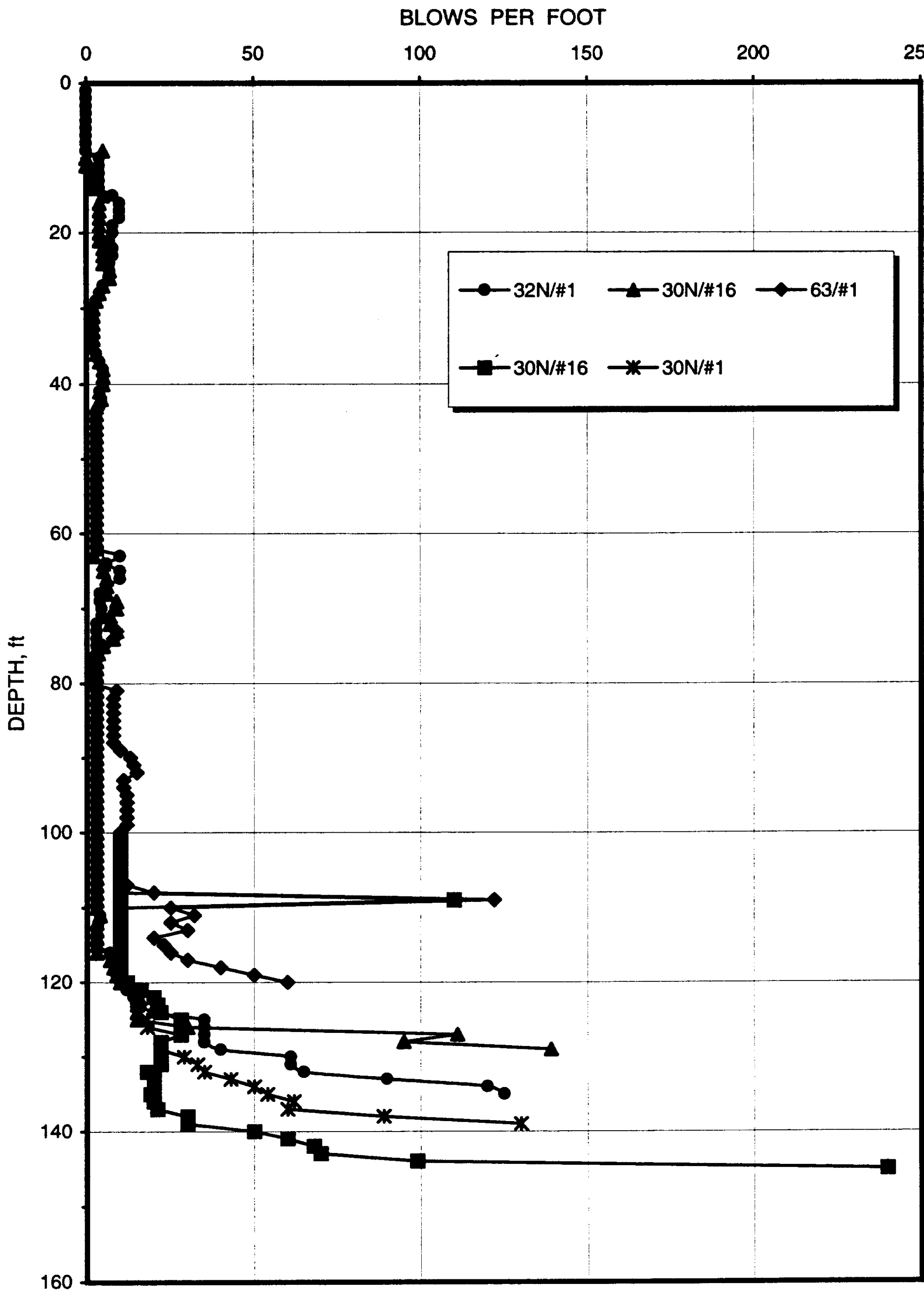
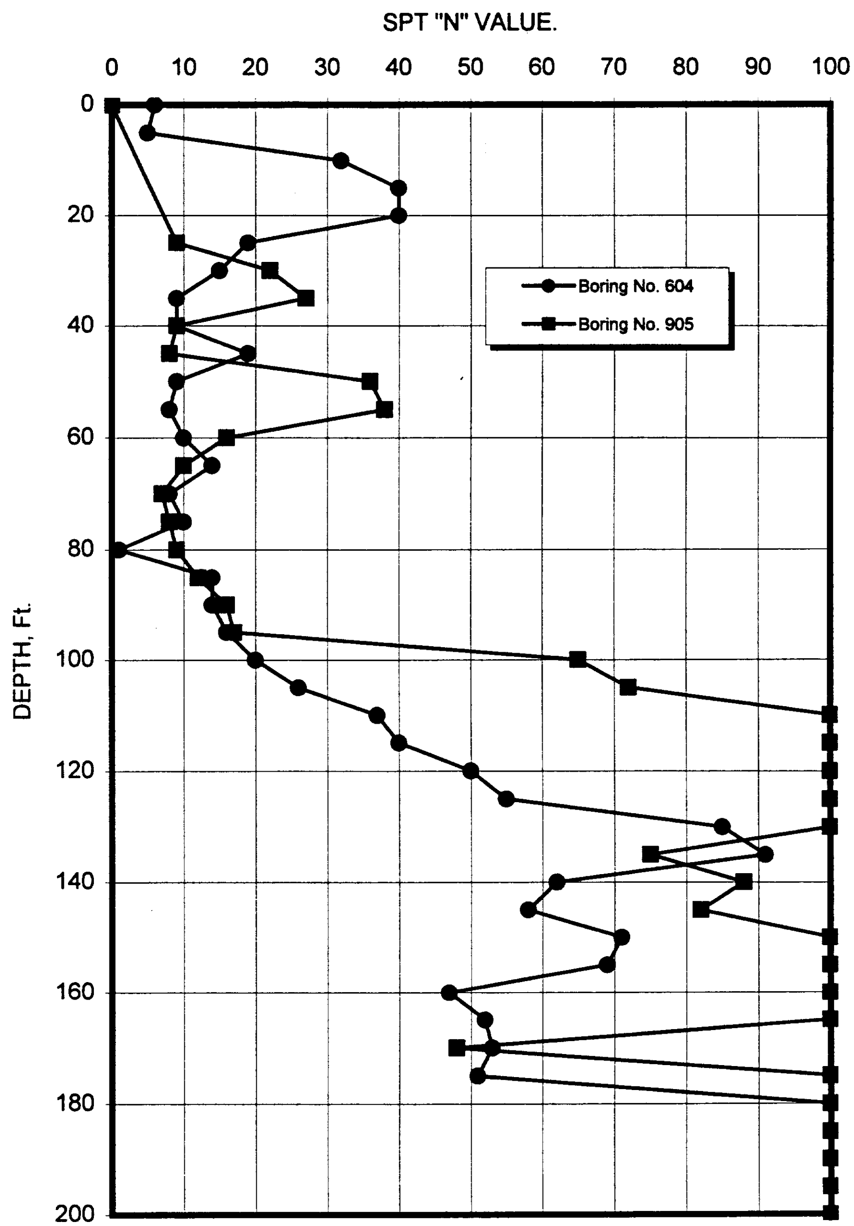


FIGURE 5.22 Pile Driving Resistance Versus Depth for Soil Type 2



Note: Points shown on 100 SPT N line correspond to 100 blows/ 1 ft or more.

FIGURE 5.23 SPT N-Value vs Depth for Soil Type 3

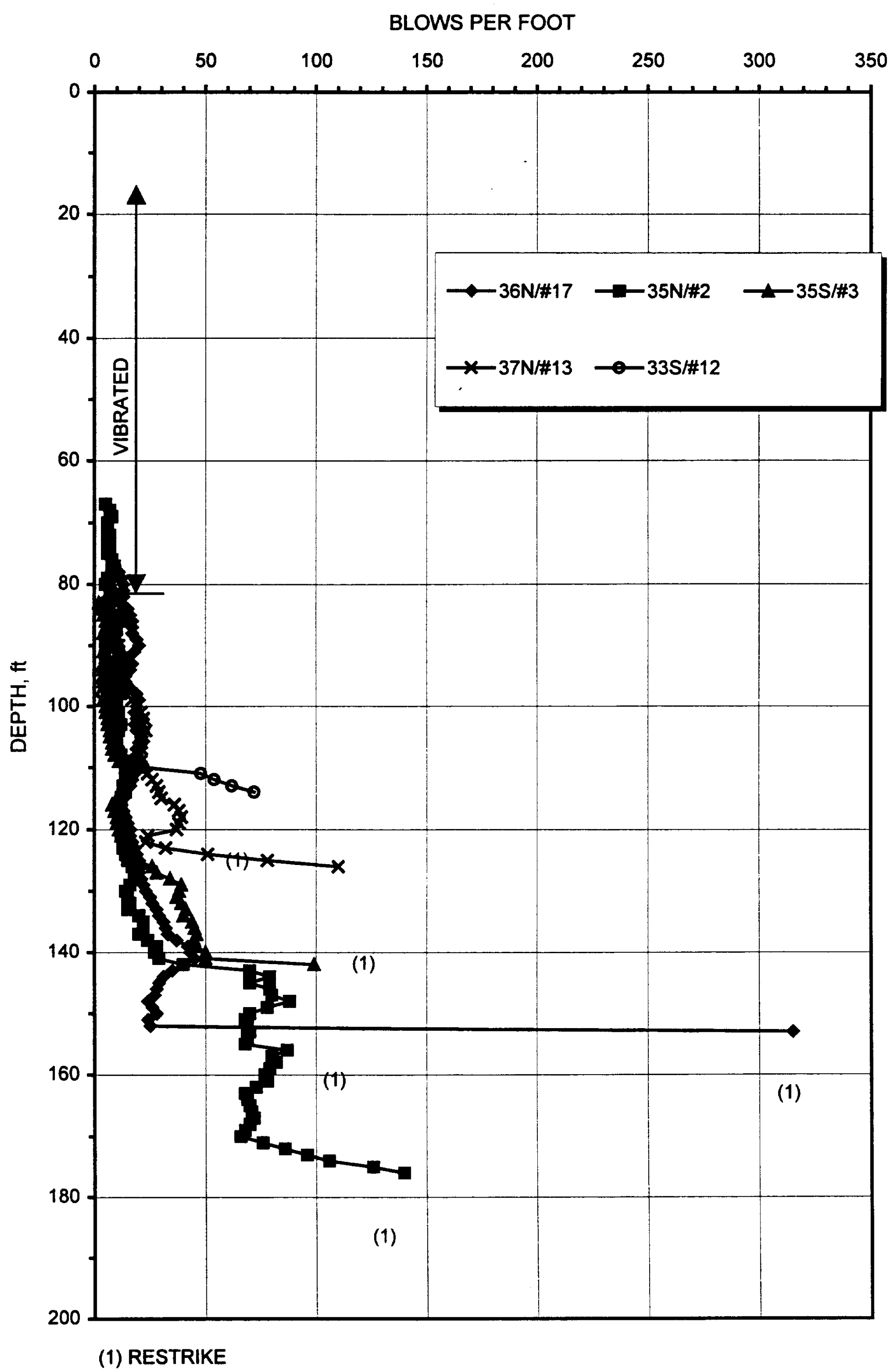
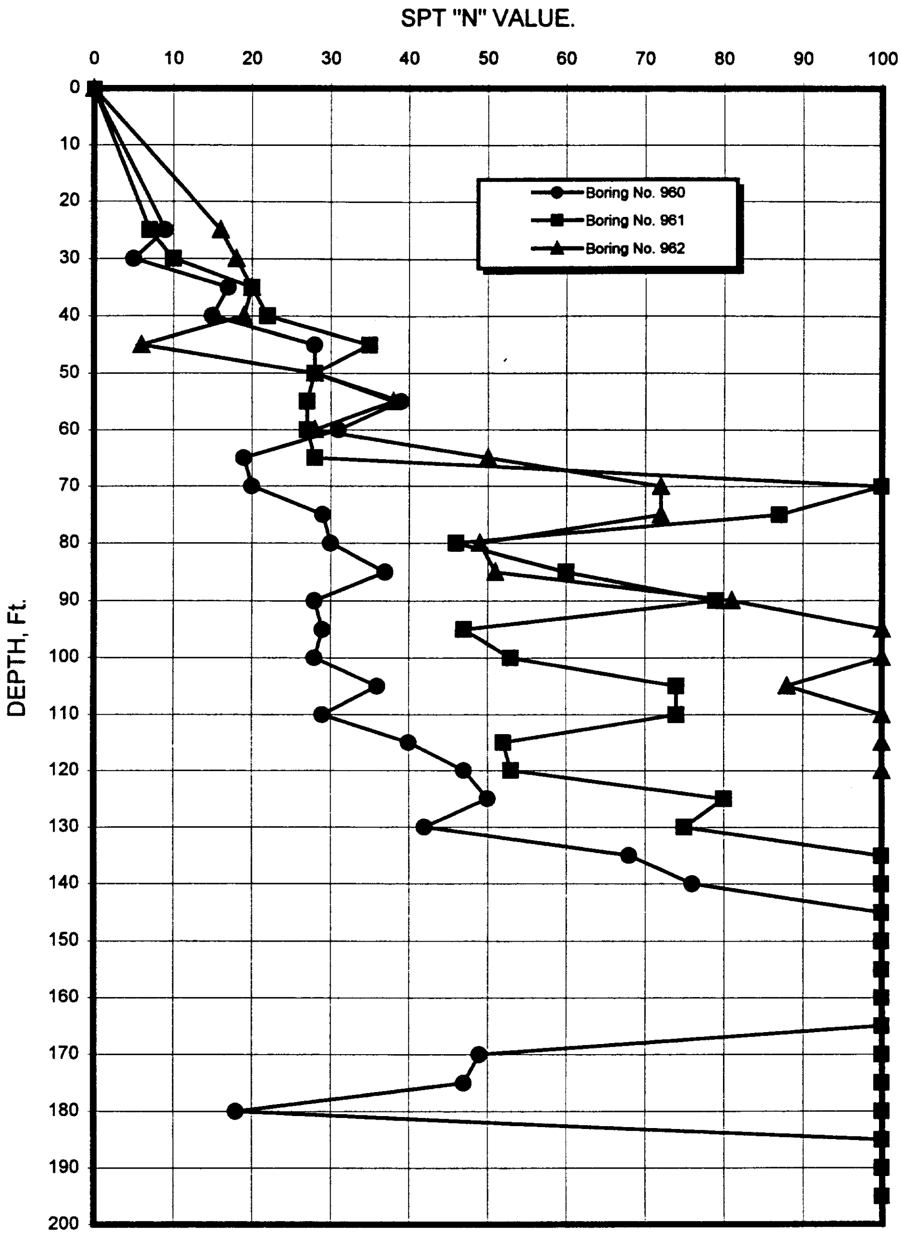


FIGURE 5.24 Pile Driving Resistance vs Depth for Soil Type 3



Note: Points shown on 100 SPT N line correspond to 100 blows/ 1 ft or more.

FIGURE 5.25 SPT N-Value vs Depth for Soil Type 4

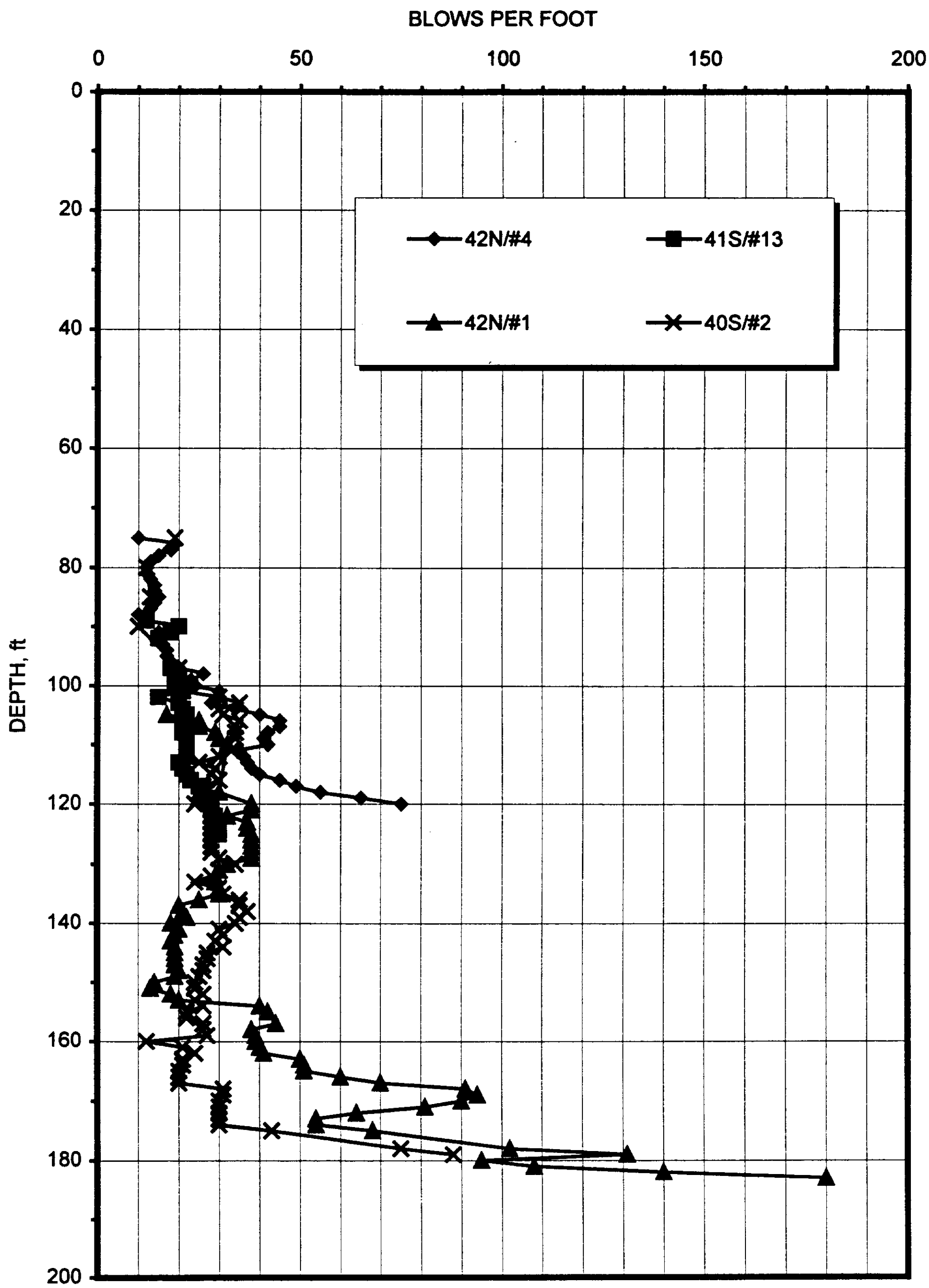


FIGURE 5.26 Pile Driving Resistance Versus Depth for Soil Type 4

The CAPWAP analysis which was performed for a selected blow count during the initial driving (end of driving [EOD]) and in the restriking period (beginning of restrike [BOR]) indicates that after soil setup the end bearing value had decreased slightly. The data is very scattered. However, an average decrease of about 15 percent was observed in Soil type 1 (westbound) and Soil type 2. About a 30 to 40 percent decrease is observed in Soil type 1 (eastbound) and Soil type 3. A 25 percent decrease is observed in Soil type 4.

5.4.2 Dynamic Soil Parameters

The dynamic soil parameters, such as soil damping and quake, are important in evaluating the static pile capacity determination derived from dynamic tests. During pile driving, soil damping occurs at the toe (J_{toe}) and around the pile (J_{skin}) and similar quake at the side (Q_{skin}) and at toe (Q_{toe}) of the piles. These values are obtained from the CAPWAP analysis. A summary of all data for the various areas at the site (in four subsurface conditions) is prepared and presented in Table 5.4.

TABLE 5.4

Summary of Dynamic Soil Properties

Soil Type	Pier Locations	Test Type	Pile Diameter (inches)	J_{Skin} (s/ft)	J_{Toe} (s/ft)	Q_{Skin} (inches)	Q_{Toe} (inches)
Type 1	Piers 14E to 16E	EOD	24	.116	.073	.126	.333
		BOR		No Restrike Tests were Performed			
	Westbound Piers (Piers 1W to 16W)	EOD	18	.149	.079	.139	.411
		BOR		.213	.123	.102	.202
	Eastbound Piers (1E to 13E, PF-6E) and East Abutment	EOD	18	.157	.087	.118	.328
		BOR		.201	.097	.120	.261
Type 2	Piers 29 to 32, 62 to 69, and 17E to 19E	EOD	24	.171	.053	.131	.593
		BOR		.186	.078	.133	.394
Type 3	Piers 33 to 38	EOD	24	.139	.062	.118	.547
		BOR		.141	.125	.196	.260
Type 4	Piers 40 to 42	EOD	24	.130	.069	.140	.477
		BOR		.114	.076	.151	.459

EOD - Test data at end of initial driving

BOR - Test data at beginning of restrike

The following important observations are derived from this table.

- The dynamic soil parameters varied between areas.
- In Soil type 4 area lower values of soil damping values were observed.
- The average side damping ranged between 0.12 (s/ft) to 0.17 (s/ft). The average side damping generally increased between 0.14 (s/ft) to 0.21 (s/ft) at the beginning of restrike. The higher increase was observed in the area where higher setup occurred.
- The side damping data indicates that the soil is in between noncohesive to cohesive.
- The average toe damping at the end of driving is 0.05 (s/ft) to 0.9 (s/ft), lower than the generally reported value 0.15 (s/ft). At the beginning of restrike, the values increase in the range of 0.08 (s/ft) to 0.13 (s/ft).
- For Soil type 4 the average skin damping decreased on restrike whereas toe damping increased slightly. This behavior is similar to a site where in general the soil setup does not occur.
- The average value of side quake ranged from 0.10 (s/ft) to 0.20 (s/ft). This value is higher than the generally representative value 0.10 (s/ft).
- The average toe quake for 18-inch diameter pile ranged from 0.20 (s/ft) to 0.41 (s/ft), whereas for 24-inch diameter pile it ranged from 0.26 (s/ft) to 0.59 (s/ft). This value is closer to d/60 where d is the diameter of pile. The value is representative of fine grained soils and saturated fine sands.

CHAPTER 6

PRODUCTION PILE INSTALLATION

6.1 General

Production pile driving criteria along with the appropriate order length of piles at each foundation unit was developed. In general, the installation criteria included type of hammer, blows for specific stroke and minimum tip elevation. The criteria was developed for initial driving as well as for restriking. During restriking, the pile was considered to achieve the required pile capacity if the established blow count criteria was achieved for a continuous three inch penetration or a maximum of 20 blows, whichever occurred first. In some cases based on the subsurface condition, sustained driving criteria was also established. During the production pile installation, in order to maintain the proper hammer stroke a hand held instrument named 'saximeter' was used. This instrument measures the blow count per minute during driving. Based on hammer blow count per minute by a simple mathematical correlation, the stroke can be determined.

During the pile installation period, at certain locations the pile installation criteria was modified based on the observed unusual driving behavior and the hammer efficiency. In this section the methodology of establishing the pile installation criteria and important observations during pile installation are discussed.

6.2 Pile Installation Criteria

The production pile installation criteria was developed based on the WEAP analysis performed for each soil condition using the dynamic soil parameters, driving hammer efficiency and the distribution of forces from the dynamic load test results. The results of the static load test were also incorporated in the WEAP analysis wherever applicable. A summary of production pile driving criteria for each footing location are presented in Tables 6.1 and 6.2. The initial driving criteria as well as the restriking driving criteria utilized the soil setup behavior. The maximum pile setup occurred in the subsurface condition Soil type 1 and type 3. In these areas the test pile penetrated deeper than that indicated by the designed value, therefore, restriking criteria was also developed. Soil setup also occurred in Soil type 2. In Soil type 4, an appreciable amount of setup did not occur. Most of the test piles achieved the required ultimate capacity during initial driving approximately at designed tip elevation. Therefore, only initial driving criteria was developed. However, the setup behavior and the dynamic test results were used for this initial driving criteria. If pile capacity did not mobilize during the initial driving, restriking was recommended and performed for some piles.

In general, the restrike was to be performed if the initial driving criteria was not achieved within the established pile tip elevation. In general, the time interval between the initial drive and restrike was set at two weeks. However, due to the construction schedule it was not strictly followed.

A sustained driving criteria was also established for some pier footings located in Soil type 1 area where evidence of hard thin layer and variable end bearing conditions were anticipated, based on the subsurface conditions. A sustained driving criteria was considered for penetrating the thin hard layer. This was monitored by evaluating the boring logs in that area. The purpose of this sustained driving criteria was to confirm that the tip of the pile was not resting on a thin hard layer.

TABLE 6.1

Summary of Driving Resistance Criteria
(18-Inch Piles)

PIER/ ABUTMENT	STROKE Ft.	ICE 60S HAMMER INITIAL DRIVING BLOWS/Ft.	ICE 60S HAMMER RESTRIKE BLOWS/In.	ICE 80S HAMMER INITIAL DRIVING BLOWS/Ft.	ICE 80S HAMMER RESTRIKE BLOWS/In.
EAST ABUTMENT	8.5	NOT APPLICABLE		163	20
	9			126	17
	9.5			99	13
	10			80	11
	10.5			66	9
	11			56	7
	11.5			46	6
PW1E, PW2E, PW3E, PW5E, PW5W, PW6E, PW8E, PW9E, PW12E, PW13E	8	148	15	145	14
	8.5	118	12	114	11
	9	97	10	94	9
	9.5	87	9	85	8
	10	73	7	70	6
	10.5	57	6	53	5
	11	43	5	46	4
PF6E, PW1E, PW2W, PW3W, PW6W, PW7W, PW11W, PW12W	7.5	101	20	98	20
	8	79	15	76	14
	8.5	67	13	64	11
	9	55	11	53	9
	9.5	47	9	45	8
	10	43	7	39	6
	10.5	40	6	37	5
PW9W, PW10W, PW13W, PW14W, PW15W, PW16W	8	NOT APPLICABLE		76	14
	8.5			64	11
	9			53	9
	9.5			45	8
	10			39	6
	10.5			34	6
	11			30	5

TABLE 6.2

Summary Of Driving Resistance Criteria
(24-Inch Piles)

PIER/ABUTMENT	STROKE Ft.	ICE 205S HAMMER INITIAL DRIVING (BLOWS/Ft.)	ICE 205S HAMMER RESTRIKE (BLOWS/INCH)
29S.29N	8	20 bpi	NOT APPLICABLE
	8.5	18 bpi	
	9	10 bpi	
	9.5	8 bpi	
	10	20 bpi	
30S, 31S, 32S, 30N, 31N, 32N	7.5	20 bpi	NOT APPLICABLE
	8	20 bpi	
	8.5	15 bpi	
	9	10 bpi, 11bpi (31N)	
	9.5	9 bpi	
	10	8 bpi	
33S, 33N	8	202	20
	8.5	117	18
	9	79	12
	9.5	55	8
	10	43	7
34S, 34N	8	216	20
	8.5	194	18
	9	115	12 For 34S
	9.5	77	8
	10	57	7
35S, 36S, 35N, 36N	8	120	20
	8.5	85	18
	9	63	10,12(36S)
	9.5	45	8
	10	37	7
37S, 37N	8	120	20
	8.5	84	18
	9	60	12
	9.5	43	8
	10	35	7
38S	8	240	20
	8.5	117	20
	9	82	17
	9.5	62	10
	10	45	8
38N	8	240	20
	8.5	170	20
	9	107	17
	9.5	70	10
	10	52	8
40S, 41S, 42S, 42N	8	43	NOT APPLICABLE
	8.5	32	
	9	26	
	9.5	21	
	10	18	
62,63,64,65,66,67, 68, 69* Pier 69- driven prior to receipt of Driving criteria	7.5	18 bpi	NOT APPLICABLE
	8	14 bpi	
	8.5	10 bpi	
	9	8 bpi	
	9.5	7 bpi	
	10	6 bpi	
14E, 15E, 16E, 17E, 18E, 19E	8	20 bpi	
	8.5	14 bpi	
	9	10 bpi	
	9.5	8 bpi	
	10	6 bpi	
	10.5	5 bpi	

A minimum pile tip elevation was also established for the production piles. This was developed to assure that the piles penetrated through the thin dense layer and deep enough into the lower glacial deposit to develop setup, and for tension and lateral load capacity considerations.

All production piles were to be driven by a warm hammer. A hammer is considered warm when it is utilized at least for 20 blows at other pile than driven pile.

6.3 Observations during Pile Installation

In the beginning of the test pile phase, it was realized that within the estimated depth the mobilized capacity was less than 50 percent. Therefore, it was established that during initial driving the pile would penetrate enough to mobilize about 70 percent of the required capacity.

The ICE 205S diesel hammer did not perform well in the upper loose glacial deposit. A similar problem was reported by Wu, et al. (53). This happens due to the development of insufficient soil resistance resulting in a lower stroke. After evaluating the pile dynamic test results, it was believed that a vibratory hammer could be used to penetrate this layer without substantially compromising the pile capacity. However, this would allow penetration of the pile slightly lower than that established. The use of vibratory hammer expedited the construction schedule.

During pile installation, some of the piles did not penetrate to the estimated depth in a group of piles. This partially happened due to the installation sequence of the piles. In such case, the data of all piles in a group was evaluated to see the group effect based on lateral capacity, axial capacity and the influence of pressure buildup on other piles.

At some location (Soil type 3), a high driving resistance with a low stroke was observed. The PDA test data indicated that the hammer efficiency dropped in such case. Hammer operation was found faulty. Hammer repair to prevent preignition was made to correct this problem.

At some location, the actual required capacity which was less than the designed capacity was to be used to finalize the pile tip elevation. This expedited the pile installation process and reduced the pile length.

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER STUDY

The following conclusions are inferred regarding the characteristics of time dependent pile capacity behavior in glacial lacustrine and glacial till deposits by dynamic load test results utilizing the Pile Driving Analyzer. The conclusions are based on the pile load test data of Route 21 project.

7.1 Conclusions

Setup Behavior:

- Setup can occur in glacial lacustrine and till deposits. Depending upon the type and depth of material, in general, a setup factor may range from 1.15 to 1.40 averaging about 1.30. Most of the setup will occur in two weeks after the initial driving. A smaller increase in setup will occur for a longer period of time.
- In glacial lacustrine and till deposits, about 20 percent of setup will occur in 2 weeks at a rate of 1.4 percent a day. Beyond 2 weeks, the rate of gain will be one-half percent a day. The following average setup factor with time relationship can be utilized for the glacial deposits.

$$y = 1.00 + 0.014\ x \text{ (for up to 2 weeks)}$$

$$y = 1.20 + 0.006\ x \text{ (beyond 2 weeks)}$$

Where y = Setup factor $\quad$ x = Days after initial driving

- A smaller pile will exhibit higher setup value. In general about fifty percent higher setup values can be expected for a smaller size pile.

- For a predominantly silty sand glacial till deposit, the rate of setup may range from one-half percent to one percent per day for a period of two weeks and one-quarter to one-half percent beyond that. However, for varved silt and cohesive deposit (lacustrine), the rate of setup may range from one to two percent per day for two weeks and one-half percent to one percent beyond two weeks.
- In a thick layer of cohesive glacial lacustrine deposit, a longer period will be required to experience the soil setup. After a two week period, the rate of soil setup will be 15 percent more than the predominantly sandy soils.
- In order to account for the future capacity gain due to soil setup, it is recommended that during initial driving, the piles should be driven at least two-thirds of the required ultimate capacity. But, the terminated tip location should be verified that it is not resting on a thin dense layer.
- It is recommended that in predominantly sandy glacial till deposits, the restriking should be performed at least two weeks after initial driving to account for most of the soil setup behavior. Whereas, in predominantly silty clay deposit (lacustrine), restriking should be performed after a 4 weeks period.

Subsurface Behavior:

- To evaluate whether a setup will occur or not and how much in a glacial deposit, the subsurface conditions must be thoroughly understood. The soil setup behavior may change significantly depending upon the type of materials present.

- Setup behavior is uniform and consistent in uniform formations. Variation in the layers of soils may complicate the setup behavior. A soft cohesive (lake deposits) and mixed type of soil deposits may exhibit soil setup for a longer period of time.
- A significant pile setup should not be expected in a glacial till deposit containing a significant amount of gravels.
- A uniform silt and varved cohesive glacial lacustrine deposit having sand seams may exhibit a smaller amount of setup than expected due to drained condition.
- The presence of shale and siltstone fragments in glacial till deposit may generate no appreciable increase in soil setup. Depending upon the amount present, it may cause relaxation due to the induced negative pore pressure during driving and due to crushing of grains.
- The SPT N-values may not be a reliable indication about the density of the materials in glacial till or drift deposit. High blow counts may be attributed to the presence of gravels and boulders. The gap graded nature of glacial deposit materials may exhibit high blow count. However, significant displacement may occur during driving. In general, the lower glacial deposit indicated very high blow count indicating very dense material. However, the mobilization of end bearing capacity was not consistent over the site. It is believed that this happened due to the presence of gap graded materials.

Load Tests:

- Actual static pile capacity based on load tests may be considered to be about 25 percent more than the evaluated pile capacity from dynamic load tests (Restrike CAPWAP).

- Dynamic load tests in combination with static load tests should be conducted to confirm the design capacity and to achieve an economical foundation design.
- Pilot load test program must be conducted prior to production pile installation. Some dynamic pile tests should be included as a quality verification tool during the production pile installation period to evaluate the hammer efficiency..
- In variable soil conditions, a dynamic test result should be correlated with the subsurface conditions depicted by borings, otherwise misleading results could be inferred about the depth capacity of pile.
- The test results obtained from the dynamic test are very valuable information for developing an appropriate driving criteria for production piles. This must be considered for a major project and specifically for glacial deposit areas.

Mobilization of Capacity:

- For all practical purposes, in a glacial lacustrine and till deposit, most of the soil setup can be expected from skin friction resistance.
- The location of dense soil layer or bedrock below the pile tip influences significantly the mobilized pile capacity during driving. In glacial deposit this is a very important factor to consider for the design of a displacement type of pile. The pile tip should be deep enough into the dense layer to take advantage of the soil setup.
- Spacing between the piles should be carefully evaluated during the design phase. A wider spacing should be considered for glacial drift deposits where significant amount of gravels are present.

- High capacity for a 24-inch pile was difficult to achieve in some cases and some driving difficulties were also observed. At some areas, due to high soil displacement and densification, some of the inner piles did not penetrate to the desired elevations. Therefore, in glacial lacustrine and till deposits a smaller size pile with lower capacity should be considered.
- In order to develop a better and economical deep pile foundation design, a thorough knowledge of the site geological history is important.
- Thorough subsurface information is important to evaluate the pile design and construction in glacial deposit area. Recovered soil samples from the borings should be identified clearly. Nature of soil particles should be noted during the classification process, such as the type of gravel (pebble, broken shale, etc.) encountered.
- Based on the skin friction mobilization data from dynamic load tests, a critical depth concept method of evaluating the design capacity appears to be appropriate for a glacial deposit area for large capacity deep piles.

General:

- It is the author's estimate that by accounting for soil setup, about 20 - 25 percent savings in pile length has been realized for this project.
- It is recommended that in glacial deposits, some larger size boring other than the SPT size borings should be performed to know the type and size of the materials (gravel) present. In glacial lacustrine deposit, some continuous sampling borings should also be performed.

- It is recommended that if construction schedule allows, the restriking should be delayed as much as possible to allow the soil setup to develop due to soil aging.

7.2 Recommendations for Further Study

- Recommendations presented are based on limited data. In order to develop a comprehensive soil setup behavior, more dynamic and static load test data should be conducted in similar soil conditions and analyzed.
- Further confirmation is required for the soil setup behavior in the drained and undrained subsurface conditions.
- In general, in the upper glacial deposit, the soil resistence and the soil setup was observed not to be significant. This may be due to the higher relative slip (greater elastic shortening) at the upper portion of the pile.
- Soil setup due to soil aging in a glacial deposit is not well understood. This should be further investigated.
- A long term restrike should be performed to develop a refined setup behavior in glacial lake deposits.

APPENDIX A

TABLES

In this Appendix the supplementary tables used for this research work are included. The following tables are included:

Summary of Dynamic Test Results - Advanced Contract

Summary of Dynamic Test Results - Construction Contract A (18-inch piles)

Summary of Dynamic Test Results - Construction Contract A (24-inch piles)

TABLE A-1
Summary of Dynamic Test Results
For Advanced Contract

PIER No.	PILE No.	PILE SIZE (In)	PILE TIP EL. (Ft.)	HAMMER	DRIVE TEST	TEST DATE	CAPWAP							COMMENTS
							R_{ult} (Kips)	R_{skin} (Kips)	R_{toe} (Kips)	J_{skin} (s/ft.)	J_{toe} (s/ft.)	Q_{skin} (In)	Q_{toe} (In)	
39S	28	24	-130.2	Conmaco5300	R	7/10/97	1087.1	465.6	621.5	0.161	0.081	0.097	0.36	
39S	13	24	-109	Conmaco5300	1	8/1/97	844.2	130.7	713.6	0.171	0.062	0.125	0.629	Static load test.
39N	28	24	-175.7	Conmaco5300	1	4/14/97	1401.4	348.5	1053	0.219	0.04	0.1	0.12	
39N	18	24	-177	Conmaco5300	1	7/23/97	1182.1	1182.1	1029	0.127	0.07	0.15	0.33	
PW4E	4	18	-104.9	ICE 60S	1	7/28/97	764.4	764.4	523	0.194	0.078	0.1	0.102	
PW4E	2	18	-120.2	ICE 60S	1	7/10/97	680	680	466.1	0.172	0.12	0.1	0.11	

TABLE A.2

Summary of PDA and CAPWAP Analysis on Test Piles

(18 Inch Piles)

PIER No.	PILE No.	Qult (tons)	PLAN TIP ELEV.	AS-DRIVEN TIP EL.	PDA TEST	BLOW COUNT (bpf)	PDA STROKE (Ft.)	PDA CSX (ksi)	PDA EMX (k-ft.)	PDA R_{ult} (Kips)	CAPWAP R_{ult} (Kips)	CAPWAP R_{skin} (Kips)	CAPWAP R_{toe} (Kips)
PW1W	1	270	-85	-90	INITIAL	30	7.8	30.3	28.4	304	286	146	140
	1	270	-85	-90.3	RESTRIKE	26/3"	8.1	35	30.4	544	540	412	128
PW2W	1	270	-80	-80	INITIAL	75	8.7	32.5	29.1	490	487	320	167
	1	270	-80.3	-80.3	RESTRIKE	28/1.5"	8.5	34.5	34.4	670	679	514	165
PW3W	1	270	-75	-75.5	INITIAL	56	8.2	30.6	24.9	398	401	160	241
	1	270	-75	-75.7	RESTRIKE	23/2"	8.3	35.6	32.3	593	547	305	242
PW5W	1	270	-75	-114	INITIAL	42	9.9	43.5	34.8	560	621	221	400
	4	270	-75	-116.4	INITIAL	24/5"	10.6	36	44.5	557	580	240	340
PW6W	3	270	-75	-75.5	INITIAL	21	9.7	35.8	41.9	330	357	195	162
	3	270	-75	-75.8	RESTRIKE	11/3"	10	33	36	454			
	3	270	-75	-97.9	REDRIVE	26/5"	10.2	32.5	37	534	614	200	414
PW7W	1	270	-75	-75.8	INITIAL	33	9.2	38.3	36.5	394	416	231	185
	1	270	-75	-75.8	RESTRIKE	13/3"	10.8	35.9	47	571	616	282	334
PW9W	1	248	-80	-80.5	INITIAL	46	8.2	36.7	25.6	344	340	221	119
	1	248	-80	-80.8	RESTRIKE	27/3"	7.8	29.1	25	425			
	1	248	-80	-92	REDRIVE	59/5"	8.4	27.1	24	515	561	227	335
	5	248	-80	-66.6	INITIAL	108/9"	8.47	31.19	32.4	480	495	280	215
	5	248	-80	-76.6	REDRIVE	46	10.7	37.6	45	490	470	345	125
	5	248	-80	-76.9	RESTRIKE	34/3"	11	37.7	47	695	698	566	132
PW10W	2	248	-80	-85	INITIAL	39/8"	8.3	29.8	29	426	430	312	118
	2	248	-80	-85.3	RESTRIKE	23/3"	9.5	37.5	40	575	611	513	98
	4	248	-80	-80.4	INITIAL	52	8.1	29.7	27.6	400	410	265	145
	4	248	-80	-80.6	RESTRIKE	17/2"	9.5	37.3	37	530	525	342	183
PW11W	1	248	-80	-80.3	INITIAL	30/6"	8	29.7	32.1	431	419	286	134
	1	248	-80	-80.6	RESTRIKE	17/2"	9.5	37.3	37	530	525	342	183

TABLE A.2 (Continued)

Summary of PDA and CAPWAP Analysis on Test Piles

(18 Inch Piles)

PIER	PILE	Qult	PLAN	AS-	PDA	BLOW	PDA				CAPWAP		
			TIP	DRIVEN	TEST	COUNT	STROKE	CSX	EMX	R_{ult}	R_{ult}	R_{skin}	R_{toe}
No.	No.	(tons)	ELEV.	TIP EL.		(bpf)	(Ft.)	(ksi)	(k-ft.)	(Kips)	(Kips)	(Kips)	(Kips)
PW12W	1	248	-80	-80.3	INITIAL	30/6"	8	29.7	32.1	431	419	286	134
	1	248	-80	-80.8	RESTRIKE	16/3"	10	36.6	47	530	544	402	141
	1	248	-80	-92.53	REDRIVE	44	10.5	31.3	37	590	592	193	399
PW13W	1	248	-75	89.5	INITIAL	30	9.9	35.2	40.4	430	456	166	290
	1	248	-75	89.8	RESTRIKE	13/3"	10.6	38	45	548	544	308	239
PW14W	1	248	-75	-93.5	INITIAL	31	9.8	36	40.3	479	490	140	350
	1	248	-75	-93.8	RESTRIKE	29/3"	11.4	40.8	51	775	744	444	300
PW15W	1	248	-75	-75.1	INITIAL	52/10"	10.7	34.9	41.2	550	540	190	350
	8	248	-75	-68	INITIAL	30	10.3	37.3	39.5	470	471	200	271
	8	248	-75	-70.2	REDRIVE	18/3"	10.3	37.3	38	550	565	247	318
PW16W	3	248	-75	-78	INITIAL	38	10.1	34.3	40.1	502	510	190	320
	8	248	-75	-75	INITIAL	38	10.4	35.4	45.8	512	515	195	350
PF6E	1	270	-80	-79.15	INITIAL	28/5"	8.6	31.4	28.8	434	430	293	137
	1	270	-80	-79.5	RESTRIKE	29/3"	8	31.8	31	542	546	463	83
PW1E	1	270	-80	-88.5	INITIAL	39	7.9	30.2	24	310	321	196	125
	1	270	-80	-88.8	RESTRIKE	24/4"	8.3	30.1	27	427	467	418	49
	1	270	-80	-89	RESTRIKE	31/4"	8.7	36	29	480	513	382	131
	1	270	-80	-117.5	REDRIVE	48/5"	8.3	38.6	28.4	560	570	156	414
PW2E	1	270	-75	-81	INITIAL	49	8.2	30.5	23.5	321	336	198	138
	1	270	-75	-91.3	RESTRIKE	29/3"	8.2	29	27	410			
	1	270	-75	-118.6	REDRIVE	82/7"	8.9	29	26.2	550	587	119	468
PW3E	1	270	-75	-108.5	INITIAL	164	8.3	31.2	27.5	640	692	207	485
PW5E	1	270	-75	-99.5	INITIAL	18/2"	11.1	37.3	43.3	743	783	250	533
	4	270	-75	-103.2	INITIAL	54/10"	11.2	39.3	44.7	689	710	336	374

TABLE A.2 (Continued)

Summary of PDA and CAPWAP Analysis on Test Piles
(18 Inch Piles)

PIER No.	PILE No.	Qult (tons)	PLAN TIP ELEV.	AS-DRIVEN TIP EL.	PDA TEST	BLOW COUNT (bpf)	PDA STROKE (Ft.)	PDA CSX (ksi)	PDA EMX (k-ft.)	PDA R_{ult} (Kips)	CAPWAP R_{ult} (Kips)	CAPWAP R_{skin} (Kips)	CAPWAP R_{toe} (Kips)
PW8E	1	270	-75	-108.5	INITIAL	164	8.3	31.2	27.5	640	692	207	485
PW9E	2	270	-75	-99.5	INITIAL	18/2"	11.1	37.3	4.3	743	783	250	533
	6	270	-75	-103.2	INITIAL	54/10"	11.2	39.3	44.7	689	710	336	374
PW12E	1	270	-75	-77.3	INITIAL	15/6"	7.5	28	27.1	310	321	170	182
	1	270	-75	-77.5	RESTRIKE	12/3"	8.6	32.8	33	475	477	140	221
	1	270	-75	-77.8	RESTRIKE	9/3"	10.2	42.4	41.3	490		256	
	1	270	-75	-90.9	REDRIVE	55/11"	10.2	41.7	41.3	680	670	226	444
	8	270	-75	-75.5	INITIAL	36	7.4	26.4	25	293	321	139	182
	8	270	-75	-75.8	RESTRIKE	11/3"	9.3	32.6	36	385			
	8	270	-75	-89	REDRIVE	39/4"	8	31.7	30	690	678	251	427
PW13E	1	270	-80	-90.5	INITIAL	61	10.5	34.4	40	642	650	200	450
	8	270	-80	-88.5	INITIAL	63	10.6	36.1	39.1	620	675	160	515
E ABUT	14	280	-82	-111.6	INITIAL	80/10"	11.4	32.1	37.2	560	581	255	326
	42	280	-82	-101.3	INITIAL	16/4"	10.9	32.1	38.2	475	465	75	390
	42	280	-82	-102.1	RESTRIKE	52/10"	10.8	36.5	43	600	604	218	386

TABLE A.3

Summary of Dynamic Test Results

(24 Inch Piles)

For Construction Contract A

Pier No.	Pile No.	Q ult. tons	Plan Tip Elev. ft.	Driven Tip Elev. ft.	Drive Test	Blow Count bpf	Stroke ft.	PDA CSX ksi	PDA EMX k-ft.	PDA $R_{ult.}$ Kips	CAPWAP $R_{ult.}$ Kips	CAPWAP R_{skin} Kips	CAPWAP R_{toe} Kips
29S	1	516	-125	-118	Initial	78/8*	10	34.6	95	1089	1090	80	1010
	8	516	-125	-121.5	Initial	219/6*	9.7	35.9	89	1035	1039	157	882
30S	8	500	-135	-122	Initial	102/6*	9.4	34.5	93	1006	1013	173	840
	16	500	-135	-126.5	Initial	44/6*	9.7	34.9	100	1005	1014	201	813
31S	1	500	-125	-117.4	Initial	49/5*	8.9	32	90	1000	1000	157	844
	16	500	125	-122	Initial	81/4*	8.8	30.5	81.8	1010	1010	190	820
	16	500	-125	-122.5	Restrike	20/1*	9.7	39.1	91.7	1240	1329	519	810
32S	1	500	-120	-122	Initial	15	8.1	32	110	600	531	54	477
	1	500	-120	-130	Initial	31/6*	11	38	112	1047	1010	160	850
	1	500	-120	-139.3	Initial	31/4*	11	37.9	120	1120	1190	274	916
	1	500	-120	-139.5	Restrike	50/3*	9.8	38.5	125	1233	1202	541.7	660
	16	500	-120	-120.8	Initial	17	7.4	30.8	83	600	569	98	471
	16	500	-120	-122.3	Redrive	69/6*	9	38	106	1303	1002	232	770
	16	500	-120	-122.8	Restrike	35/2*	8.8	35.2	92	1020	1001	296	705
33S	1	500	-115	-100	Initial	31	7.9	29.1	82.4	784	855	109	746
	1	500	-115	-103.9	Redrive	57/11*	8.9	34.1	90.1	1023	1010	176	834
	12	500	-115	-105.2	Initial	47	8.5	33	93	880	837	142	695
	12	500	-115	-107.4	Redrive	18/3*	8.9	34.8	91.9	1023	1020	152	868
34S	2	500	-120	-141.4	Initial	90/5*	8	28.8	61.9	987	1004	385	619
	16	500	-120	-138.6	Initial	158/7*	7.3	30.7	56.7	980	1000	540	460
35S	3	500	-120	-130.1	Initial	40	8.7	30.3	78	840	865	201	664
	3	500	-120	-130.4	Restrike	31/4*	9.1	35.1	97.1	1060	1060	866	230
	13	500	-120	-128.7	Initial	40	8.7	32.6	86.8	890	870	218	652
	13	500	-120	-129.2	Restrike	54/6*	8.9	35.5	88.2	1140	1123	575	548
36S	1	484	-125	-126	Initial	47	9.2	36.2	93	858	848	307	541
	1	484	-125	-126.3	Restrike	12/3*	8.8	33.9	83.3	880	880	414	466
	1	484	-125	-150	Redrive	114	8.2	33.6	70.8	840	842	372	470
	1	484	-125	-150.3	Restrike	34/3*	8.9	36.6	87.6	965	985	696	289
37S	1	484	-125	-125	Initial	44	9.5	37	94.1	900	900	280	620
	1	484	-125	-127.3	Restrike	30/3*	8	35.8	72.3	1027	1062	642	420
	30	484	-125	-125.7	Initial	22	8.4	33.9	79.7	703	707	196	511

TABLE A.3 (Continued)

Summary of Dynamic Test Results

(24 Inch Piles)

For Construction Contract A

Pier No.	Pile No.	Q ult. tons	Plan Tip Elev. ft.	Driven Tip Elev. ft.	Drive Test	Blow Count bpf	Stroke ft.	PDA CSX ksi	PDA EMX k-ft.	PDA $R_{ult.}$ Kips	CAPWAP $R_{ult.}$ Kips	CAPWAP R_{skin} Kips	CAPWAP R_{toe} Kips
37S	30	484	-125	-126	Restrike	19/4*	7.4	33.5	62.6	760	820	387	433
	30	484	-125	-130.7	Restrike	19/3*	9	41.9	89	956	989	589	400
38S	24	500	120	-121.1	Initial	33	8.7	29.6	84	802	798	122	676
	24	500	-120	-121.4	Restrike	24/3*	8.4	33.1	78.7	932	950	455	495
	24	500	-120	-121.6	Restrike	32/3*	8.8	35.8	82.2	940	954	479	478
40S	2	477	-105	-103	Initial	32	8.5	31	76.1	806	775	215	560
	2	477	-105	-104	Restrike	24	7.8	33.3	75.9	780	795	510	285
	2	477	-105	-119	Redrive	27	8.3	33.3	79.7	745	740	500	240
	2	477	-105	-119.3	Restrike	11/3*	8.3	31.4	75	582	714	504	213
	2	477	-105	-169.1	Redrive	14/2*	9.1	36.6	91.5	954	975	459	516
	23	477	-105	-125.8	Initial	22/10*	8.1	29.7	77.8	719	685	200	485
	23	477	-105	-126	Restrike	17/3*	8	34.4	74.1	860	873	498	375
	23	477	-105	-126.3	Restrike	11/3*	7.8	35.5	83.6	872	900	678	222
	23	477	-105	-168	Redrive	73	9.1	35.5	95.8	1043	1075	339	736
41S	3	477	-105	-148	Initial	27	8.2	32.8	80.4	758	790	375	415
	3	477	-105	-148.3	Restrike	16/3*	7.7	36.5	81.2	750	780	623	157
	3	477	-105	-157	Redrive	23	7.9	36.5	77	710	740	470	270
	3	477	-105	-157.3	Restrike	12/3*	7.8	35.6	73.7	752	742	517	225
	3	477	-105	-162.6	Redrive	68/7*	9.1	35.6	93.9	965	1023	353	671
	13	477	-105	-100	Initial	38	8.5	37.4	92.4	1018	1020	335	685
	13	477	-105	-100.3	Restrike	12/3*	8.5	35	97	884	868	288	580
	13	477	-105	-116	Redrive	71	9.3	36.7	92.8	1087	1087	335	752
42S	3	515	-100	-128.4	Initial	25/5	9.1	32.8	87.4	1038	1040	300	740
	3	515	-100	-128.7	Restrike	16/3*	9.4	36.8	90.4	1048	1048	398	650
	13	515	-100	-128.5	Initial	15/6	8.7	30.9	74.8	816	750	230	520
	13	515	-100	-129	Restrike	13/3*	7.8	32.2	64.2	795	810	520	290
	13	515	-100	-129.3	Restrike	9/3*	7.9	32	69	825			
	13	515	-100	-165.5	Redrive	32/6*	8.6	32	82	1040			
29N	1	516	-125	-122.6	Initial	195/7*	9.5	29.6	82	1015	1036	194	842
	5	516	-125	-121.8	Initial	232/9*	9.5	32.9	93	1017	1044	133	911
30N	1	500	-135	-134.2	Initial	32/3*	9.5	37	105	1051	1075	175	900
	16	500	-135	-135.6	Initial	174/9*	9.6	29.8	82	1120	1045	286	759

TABLE A.3 (Continued)

Summary of Dynamic Test Results

(24 Inch Piles)

For Construction Contract A

Pier No.	Pile No.	Q ult. tons	Plan Tip Elev. ft.	Driven Tip Elev. ft.	Drive Test	Blow Count bpf	Stroke ft.	PDA CSX ksi	PDA EMX k-ft.	PDA $R_{ult.}$ Kips	CAPWAP $R_{ult.}$ Kips	CAPWAP R_{skin} Kips	CAPWAP R_{toe} Kips
31N	1	500	-125	-123	Initial	83/4*	10	33	94	1018	1004	207	797
	9	500	-125	-142	Initial	80	9.9	32.1	92	1017	1004	191	813
32N	1	500	-120	-120.8	Initial	29	7.5	26.1	40	70			
	1	500	-120	-131.8	Redrive	115	9.6	33.7	81	960	940	220	720
	16	500	-120	-125.5	Initial	11	6.5	23.5	73	360			
	16	500	-120	-133	Redrive	77/6*	9.2	34.5	94.4	1000	1000	250	750
	16	500	-120	-133.2	Initial	31/2*	9.3	37.4	98	1055	1191	309	882
33N	6	500	-120	-108.9	Redrive	32/4*	8.9	36.8	106.9	1115	1110	220	890
	15	500	-120	-161	Restrike	36/6*	8.8	41.4	92.2	1100	1110	280	830
34N	1	500	-120	-135	Initial	179	8.1	34.4	67.4	990	1005	333	672
	18	500	-120	-162.4	Initial	145/6*	7.2	32.9	59	985	1025	385	640
35N	2	500	-120	-144.8	Initial	80	8.8	33	77	840	858	308	550
	2	500	-120	-145.1	Restrike	25/3*	8.3	33.4	73.2	900	895	645	250
	2	500	-120	-160.3	Redrive	68/6*	8.9	33.4	83.8	1020	1030	545	485
	15	500	-120	-128.5	Initial	24	7.7	36.9	71	610	584	177	407
	15	500	-120	-136.7	Redrive	53	9.1	37.9	88.4	960	940	520	420
	15	500	-120	-136.7	Restrike	68	9.2	37.9	88.4	1160	1190	900	290
36N	2	484	-120	-136.8	Initial	34	8.9	32.4	84	735	721	200	521
	2	484	-120	-137.1	Restrike	19/3*	8.3	33.7	74.4	800	824	359	465
	2	484	-120	-150.8	Redrive	38	8.5	34.4	76.4	730	724	304	420
	2	484	-120	-151.1	Restrike	21/3*	8.3	36.3	78.3	950	950	760	220
	17	484	-130	-146	Initial	25	8.2	34.5	75.1	650	950	365	285
	17	484	-120	146.-1	Restrike	26/1*	10.5	41	100	1110	1160	916	244
37N	13	484	-120	-121	Initial	76	8.4	36.1	77.4	915	919	271	648
	13	484	-120	-121.3	Restrike	27/3*	8.6	36.5	81.4	1020	1020	670	350
38N	24	500	-120	-110.4	Initial	42/10*	9.6	32.5	90	1006	1007	289	718
42N	1	515	-100	-129.1	Initial	20/8*	.6	29	74.1	761	735	255	480
	1	515	-100	-129.3	Restrike	11/3*	7.3	32	63.4	668	728	430	298
	1	515	-100	-129.6	Restrike	10/4*	7.3	32	70.1	720			
	1	515	-100	-176.4	Redrive	178	7.3	32	72.4	1035			
	4	515	-100	-116	Initial	27	8 .6	35.2	84.1	829	810	310	500

TABLE A.3 (Continued)

Summary of Dynamic Test Results

(24 Inch Piles)

For Construction Contract A

Pier No.	Pile No.	Q ult. tons	Plan Tip Elev. ft.	Driven Tip Elev. ft.	Drive Test	Blow Count bpf	Stroke ft.	PDA			CAPWAP		
								CSX ksi	EMX k-ft.	$R_{ult.}$ Kips	$R_{ult.}$ Kips	R_{skin} Kips	R_{toe} Kips
42N	24	515	-100	-126.2	Initial	30/8*	8.8	30.5	79.9	892	865	265	600
	24	515	-100	-126.5	Restrike	8/3*	7.6	31.8	68.4	790	810	335	475
	24	515	-100	-126.8	Restrike	12/3*	7.6	31.1	69.3	768	857	326	531
	24	515	-100	-164.1	Redrive	72/7*	8.8	33.4	92.7	1021	1079	329	750
PW14E	1	510	-80	-96	Initial	24/3*	8.7	38.6	75.5	1053	1030	509	521
	12	510	-80	-101.5	Initial	39/6*	8.7	33.9	71.1	1076	1030	140	890
PW15E	1	510	-85	-101.7	Initial	64/8*	8.6	33.5	74.7	1019	1 030	350	680
	12	510	-85	-95.6	Initial	44/8*	8.4	31.2	68.7	1037	1032	447	585
PW16E	1	510	-80	-96.4	Initial	71/11*	8.4	32.7	59.5	982	1189	340	849
	12	510	-80	-92	Initial	38/6*	8.4	33.6	71.4	1059	1038	298	740
PW17E	1	510	-80	-90.3	Initial	65	8.5	33.2	67.9	950	940	355	585
	1	510	-80	-90.6	Restrike	41/3*	9	38	67.9	1130	1090	710	380
	1	510	-80	-91.5	Redrive	24/2*	8.5	31.7	67.9	960	965	373	592
	12	510	-80	-107.7	Initial	110	8.3	33.2	69.2	1003	1028	511	517
PW18E	1	510	-80	-91.5	Initial	17	8.3	32.8	77.7	702	700	204	496
	1	510	-80	-91.8	Restrike	16/3*	9	34.8	79	886	885	385	500
	1	510	-80	-92	Restrike	13/3*	9.2	39.9	82.5	900	941	439	502
	1	510	-80	-92.5	Restrike	23/6*	9.2	39.9	82.5	900	941	439	502
	1	510	-80	-120.5	Redrive	75	9.2	39	69.3	1030	1043	477	566
	12	510	-80	-96	Initial	31	8.6	33.7	79.2	840	805	206	599
	12	510	-80	-96.3	Restrike	16/3*	10	36.2	99	1060	1110	496	614
	12	510	-80	-95.5	Restrike	27.9.6	9.6	36.2	75	1050	1115	595	520
	12	510	-80	-119.5	Redrive	44/10*	9.3	31.2	90.2	1030	1020	481	539
	4	510	-80	-109.8	Initial	51/9*	9.6	35.2	86.8	1031	1050	304	746
PW19E	9	510	-110	-113.3	Initial	25/4*	9.4	31.2	71.4	1020	1020	365	655
62	6	440	-110	-106	Initial	62/11*	9.4	32.4	82	920	905	155	750
63	1	440	-115	-115.1	Initial	34/6*	9	32.8	80.7	930	900	150	750
64	1	440	-115	-117.2	Initial	8/2*	7.7	32.7	74	700	702	132	570
	1	440	-115	-117.5	Restrike	25/3*	9.3	35.2	86	841	888	263	625
	1	440	-115	-117.7	Restrike	37/3*	9.1	35.2	86	841	829	199	630
	1	440	-115	-118.5	Redrive	58/3*	8.8	38.8	81	912	930	150	780
	7	440	-115	-116.3	Initial	9/3*	8.7	32. 5	80.1	710	700	157	543

TABLE A.3 (Continued)

Summary of Dynamic Test Results

(24 Inch Piles)

For Construction Contract A

Pier No.	Pile No.	Q ult. tons	Plan Tip Elev. ft.	Driven Tip Elev. ft.	Drive Test	Blow Count bpf	Stroke ft.	PDA			CAPWAP		
								CSX ksi	EMX k-ft.	$R_{ult.}$ Kips	$R_{ult.}$ Kips	R_{skin} Kips	R_{toe} Kips
64	7	440	-115	-116.5	Restrike	19/3*	8.3	32.1	71	721	728	229.3	498.7
	7	440	-115	-117.5	Redrive	26/3*	9.2	36.3	88.3	890	890	188	702
65	6	440	-110	-113.5	Initial	10/3*	8.9	34.4	83	883	881	194	687
	18	440	-110	-117.5	Initial	54/6*	9.3	31.4	76	891	882	126	756
66	1	440	-115	-99.6	Initial	38/6*	9.4	36.1	98	950	950	225	725
67	1	440	-100	-99.7	Initial	33/8*	9.5	34.2	91	948	906	106	800
	16	440	-100	-91.2	Initial	12/3*	9.1	33.2	79	927	911	99	812
68	1	440	-100	-95.1	Initial	26/7*	9.3	37.1	95.6	900	902	238	664
69	12	440	-100	-91	Initial	29/6*	8.9	36.2	78.7	975	975	215	760
29S	6	516	-125	-119.5	Initial	66/6*	9.3	36.8	96.4	1116	1100	143	957
31S	11	500	-125	-120.1	Initial	33/2"	9.2	34.7	93.5	1063	1065	275	790
31N	3	500	-125	-124.3	Initial	40/4"	9	39.4	98.8	1064	1045	155	890
33N	10	500	-115	-160.7	Initial	30/1"	7.3	35.3	70.3	1010	1021	221	800
	16	500	-115	-158	Initial	177/6"	7.6	26.3	53.1	971	1003	353	650
34S	11	500		-139	Initial	85	9	36.5	83.7	1029	1010	365	645
34N	8	500		-138.4	Initial	62/5"	8.2	32.7	61.8	1027	1025	551	474
36S	4	484	-125	-113.3	Restrike	31/1"	8.6	36	90	1100	1175	725	450
	4	484	-125	-113.5	Redrive	30/1"	8.5	34.5	86	1060	1038	450	633
	21	484	-125	-117.4	Restrike	75	8.5	36.9	87.6	1000	990	183	807
36N	7	484	-120	-148.7	Initial	36	8.4	30.8	75.2	755	755	295	460
	7	484	-130	-148.8	Restrike	32/.5"	9.3	40.2	84.1	1090	1110	748	362
37S	15	484	-120	-108	Initial	20	8.5	36.2	88.2	740			
37N	3	484	-120	-102	Restrike	64/3"	8.8	31.6	73.8	926	930	740	190
	3	484	-120	-105.1	Redrive	26/1"	9.5	31.6	83.1	994	1055	290	765
	16	484	-120	-103.9	Restrike	37/1"	9.1	30.4	68.7	985	1075	665	410
	24	484	-120	-101.3	Initial	48/3"	8.9	31.5	69.6	990	980	325	655
	30	484	-120	-126.3	Initial	14/1"	8.7	31.6	69.4	945	940	291	649
69	1	440	-100	-79	Initial	39/3"	8.3	33.4	72.5	927	900	241	659
	8	440	-100	-80.2	Restrike	23/3"	8.7	30.5	65.2	977	950	520	430
	10	440	-100	-78	Restrike	44/3"	8.2	30.7	69.8	955	935	415	520

Note: ICE 206S Hammer was used for 24 Inch diameter Piles.

APPENDIX B

FIGURES

The figures presented in the Appendix are of the secondary importance of this research work. However, these supplemental figures are vital to understand this research work. Some typical footing plans are also included to have a general idea for the test pile setup planning. The following documents are presented in this Appendix:

Inferred Subsurface Profiles

Pile and Footing Plans for Pier 33N

Pier Plan and Elevation for Pier 62

Static Load Tests for Pier 37S

Photographs

- Static Load Testing and Construction

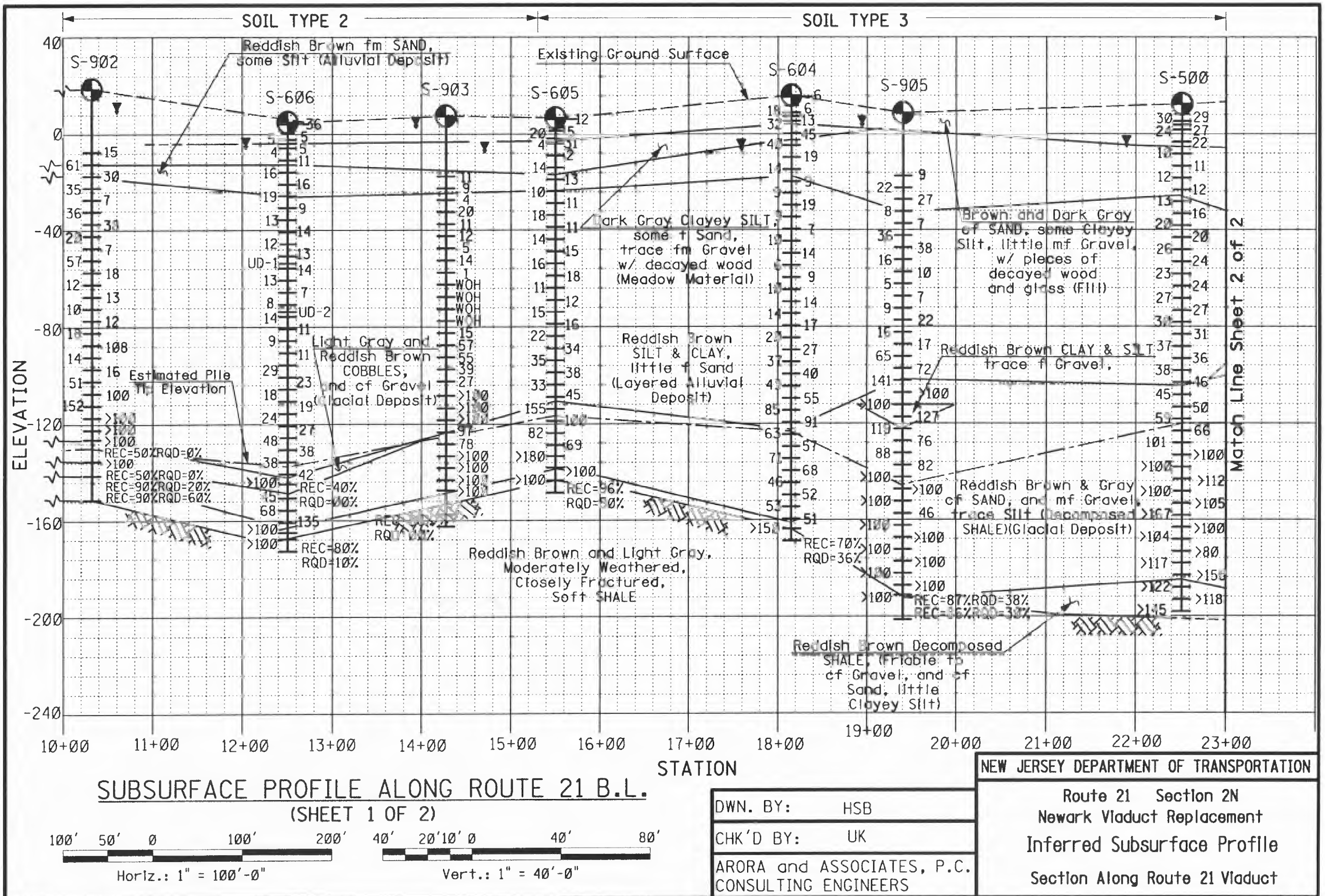


FIGURE B.1 Inferred Subsurface Profile (Rt. 21 Section)

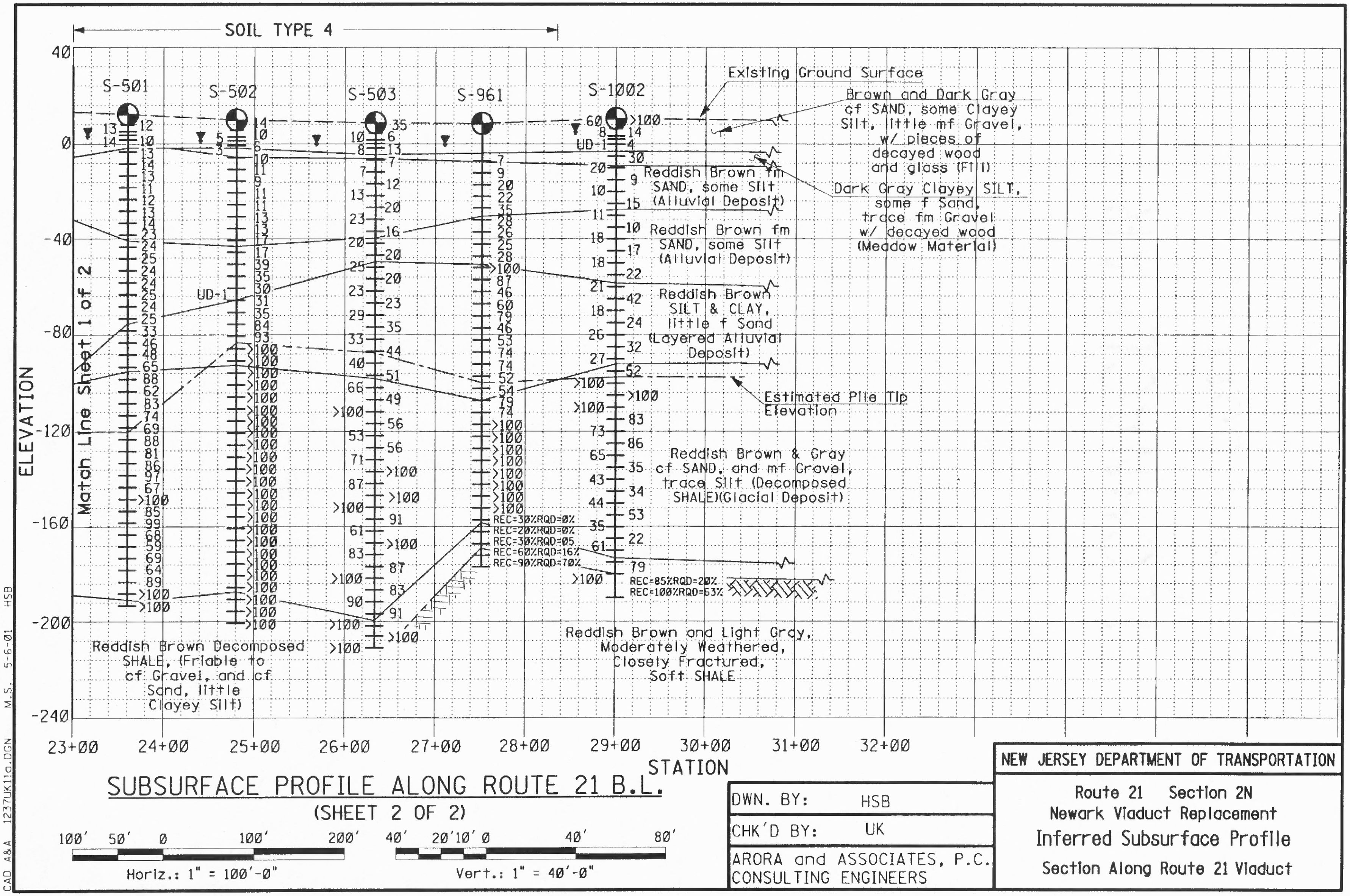


FIGURE B.2 Inferred Subsurface Profile (Rt. 21 Section)

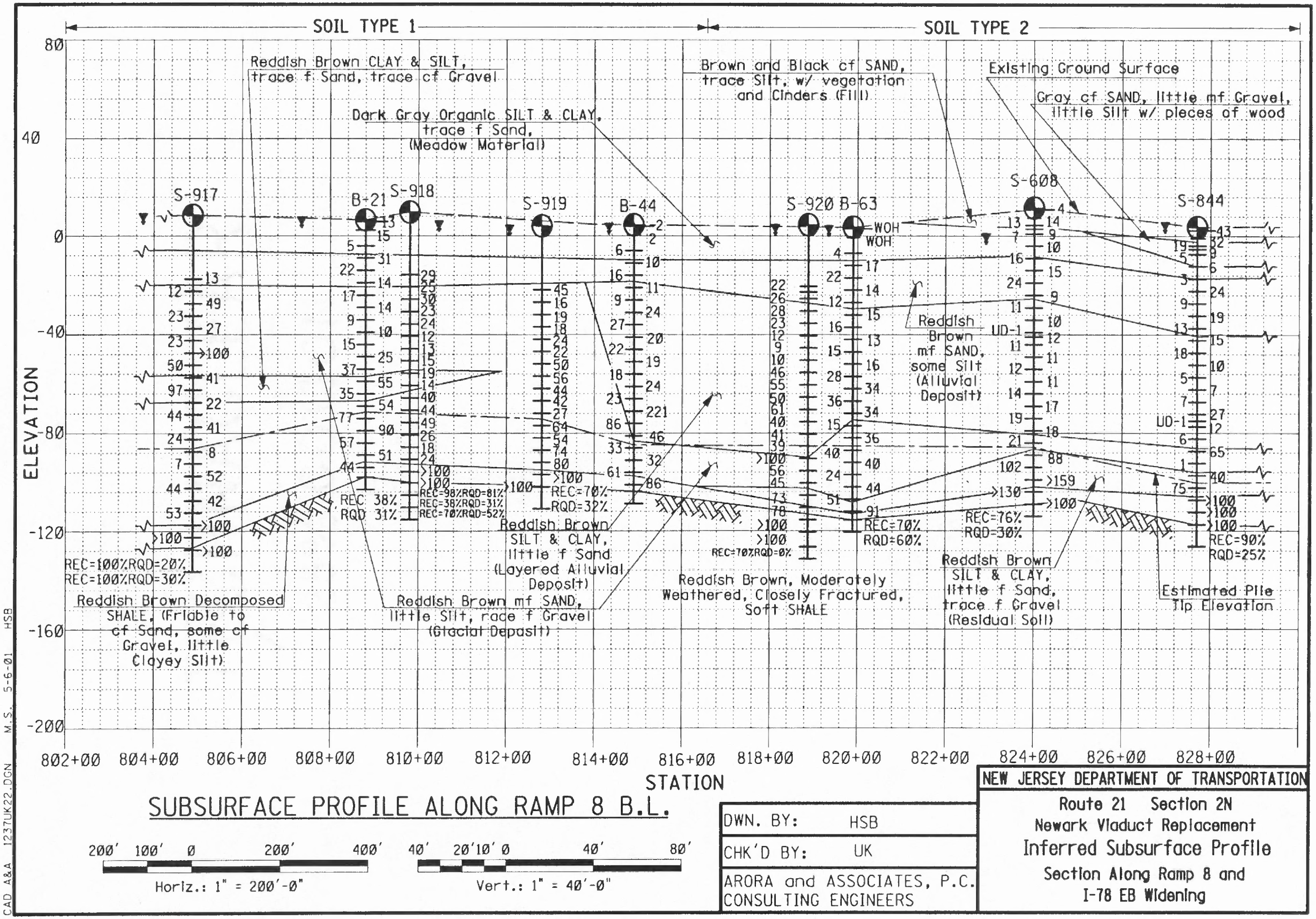
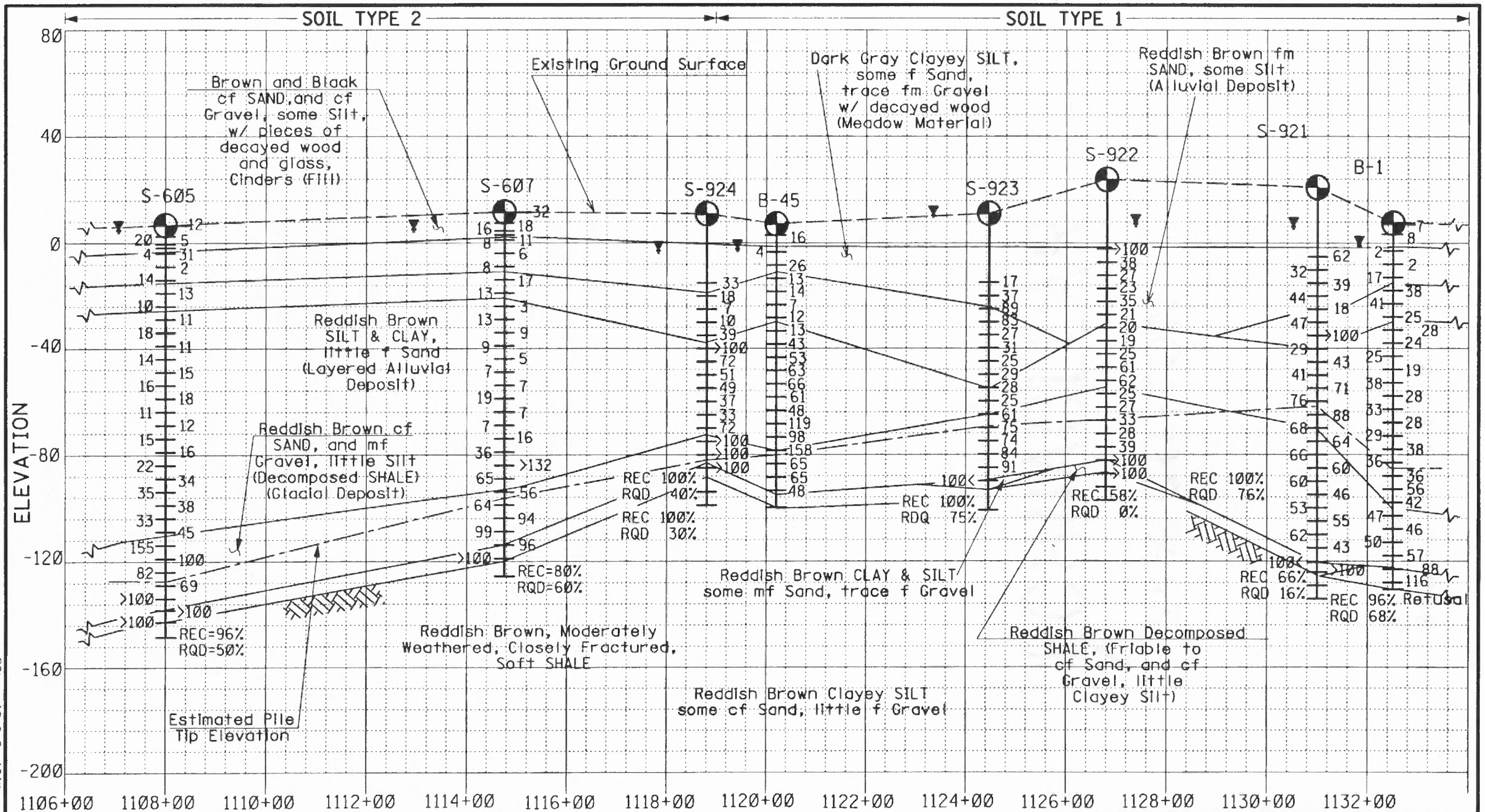


FIGURE B.3 Inferred Subsurface Profile, Ramp 8 and I-78 EB



SUBSURFACE PROFILE ALONG RAMP 11 B.L.

200′ 100′ 0 200′ 400′
Horiz.: 1" = 200′-0"

40′ 20′10′ 0 40′ 80′
Vert.: 1" = 40′-0"

DWN. BY:	HSB
CHK'D BY:	UK
ARORA and ASSOCIATES, P.C. CONSULTING ENGINEERS	

NEW JERSEY DEPARTMENT OF TRANSPORTATION

Route 21 Section 2N
Newark Viaduct Replacement
Inferred Subsurface Profile

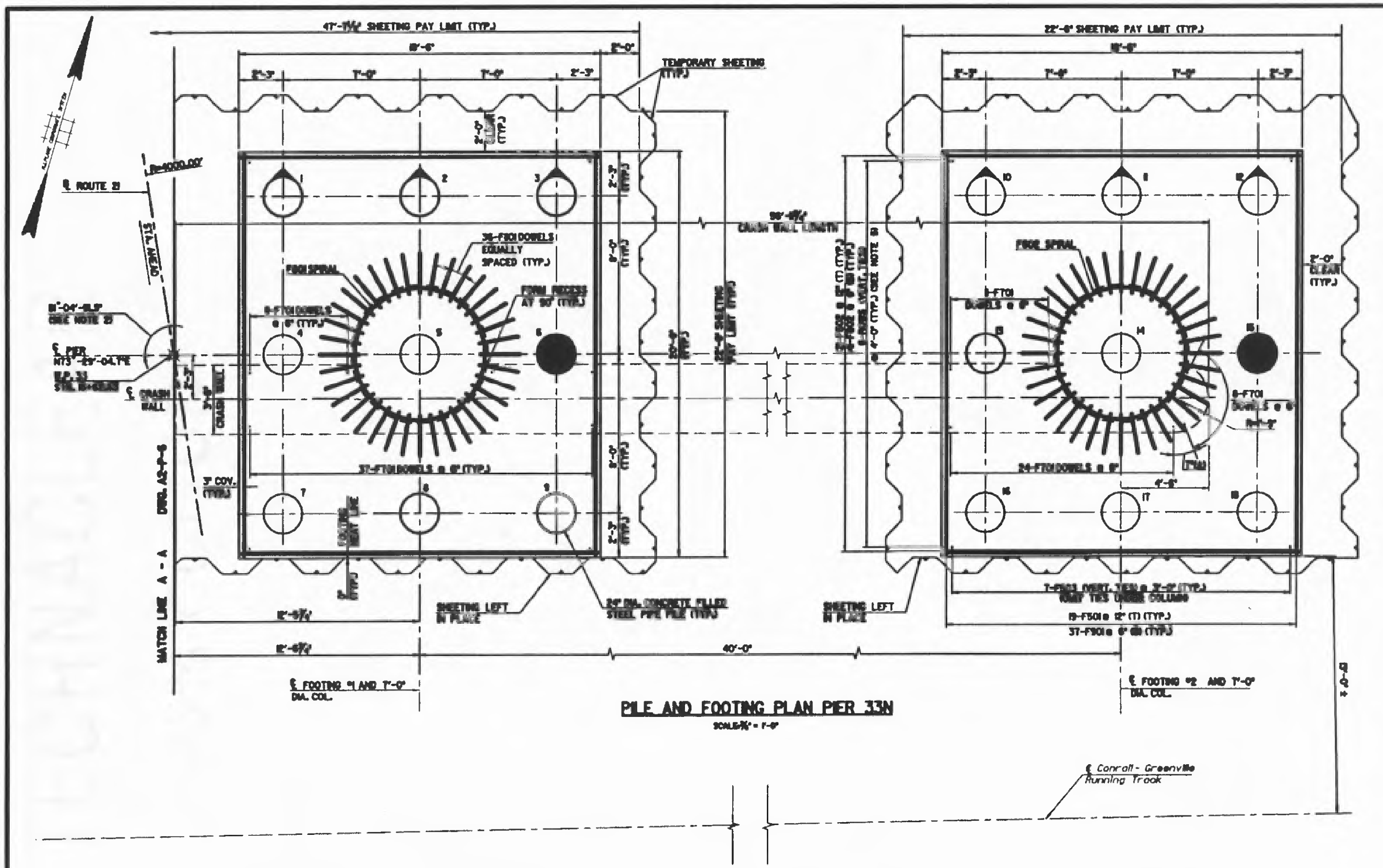
Section Along Ramp 11 and
I-78 WB Widening

CAD A&A 1237uk33.DGN M.S. 5-6-01 HSB

FIGURE B.4 Inferred Subsurface Profile, Ramp 11 and I-78 WB

STRUCTURE NO. 0713 - 151

STRUCTURE NAME: ROUTE 21 VIADUCT



NOTES:

1. FOOTING REINFORCEMENT SHOWN IS TYPICAL FOR EACH FOOTING WITHIN A PIER.
2. ANGLE IS FROM TANGENT TO ROUTE 21 ℄ TO ℄ PIER.
3. THERE MAY BE UTILITIES IN THE VICINITY OF THE NEW CONSTRUCTION. THE CONTRACTOR SHALL LOCATE AND PROTECT ALL UTILITIES PRIOR TO DRIVING PILES.
4. REINFORCEMENT EXTENDING FROM TOP OF PILE INTO FOOTING IS NOT SHOWN FOR CLARITY.
5. WHEN PLACING FOOTING VERTICAL TIES, ALTERNATE HOOKS TOP AND BOTTOM.
6. THE CONTRACTOR IS ALERTED TO THE POTENTIAL CONFLICT BETWEEN THE BATTERED PILES AND THE SHEETING AT PIER 33N. THE CONTRACTOR SHALL TAKE THIS CONDITION INTO ACCOUNT WHEN PREPARING SHEETING DESIGNS.

REFERENCES:

1. FOR WORKING POINT COORDINATES, SEE DWG. C-K-1.
2. FOR PILE REINFORCING, CLOSURE PLATES AND PILE SPLICE DETAILS, SEE DWG. NO. C-P-1.
3. FOR LIMITS OF PAYMENT FOR FOUNDATION EXCAVATION, SEE DWG. NO. C-P-2.
4. FOR PIER 33N BOTTOM OF FOOTING ELEVATION SEE DWG. NO. A2-P-15.
5. FOR PIER 33N PLAN AND ELEVATION, SEE DWG. NO. A2-P-18.
6. FOR PIER 33N REINFORCEMENT SCHEDULE, SEE A2-P-16.
7. FOR VERTICAL LIMITS OF PAYMENT FOR SHEETING, SEE DWG. NO. C-P-2.
8. FOR LIMITS OF STOCKPILING OR REUSE OF EXCAVATED MATERIAL, SEE ROADWAY PLANS.
9. FOR CRASH WALL DETAILS, SEE DWG. NO. C-P-3.

LEGEND:

- INDICATES VERTICAL PILE
- INDICATES TEST PILE
- INDICATES BATTERED PILE IN DIRECTION OF ARROW (1H:2V)

FOUNDATION PILE DATA	
CONCRETE FILLED STEEL PIPE-24" DIA.	PIER 33N
ULTIMATE CAPACITY (TONS)	500
ESTIMATED PILE TIP ELEVATION	-115

QUANTITIES

PAY ITEM NO.	STANDARD ITEM NO.	DESCRIPTION	UNIT	CONTRACT QUANTITY
105	2F21C	FOUNDATION EXCAVATION	C.Y.	275
106	5A21C	CONCRETE IN STRUCTURES, FOOTINGS	C.Y.	183
107	5A21E	REINFORCEMENT STEEL IN STRUCTURES	LBS.	44,100
115	5M21C	TEMPORARY SHEETING	S.F.	1,110
116	5M31C	SHEETING LEFT-IN-PLACE	S.F.	490
133	N5E02	CONCRETE FILLED STEEL PIPE PILES, 24" DIA.	L.F.	1,798
134	N5E03	TEST PILES, CONCRETE FILLED STEEL PIPE PILES, 24" DIA.	L.F.	224
136	N5E04	STEEL PIPE PILES SPLICE, 24" DIA.	UNIT	18
138	N5E07	DYNAMIC MONITORING OF PILES BY PILE DRIVING ANALYZER	UNIT	2

CONTRACT QUANTITIES FOR PILES ARE BASED ON ESTIMATED PILE TIP ELEVATIONS.

NEW JERSEY DEPARTMENT OF TRANSPORTATION

Pile and Footing Plan
Pier 33N

ROUTE 21 SECTION 2N
ROUTE 21 MAINLINE

SCALE: AS SHOWN

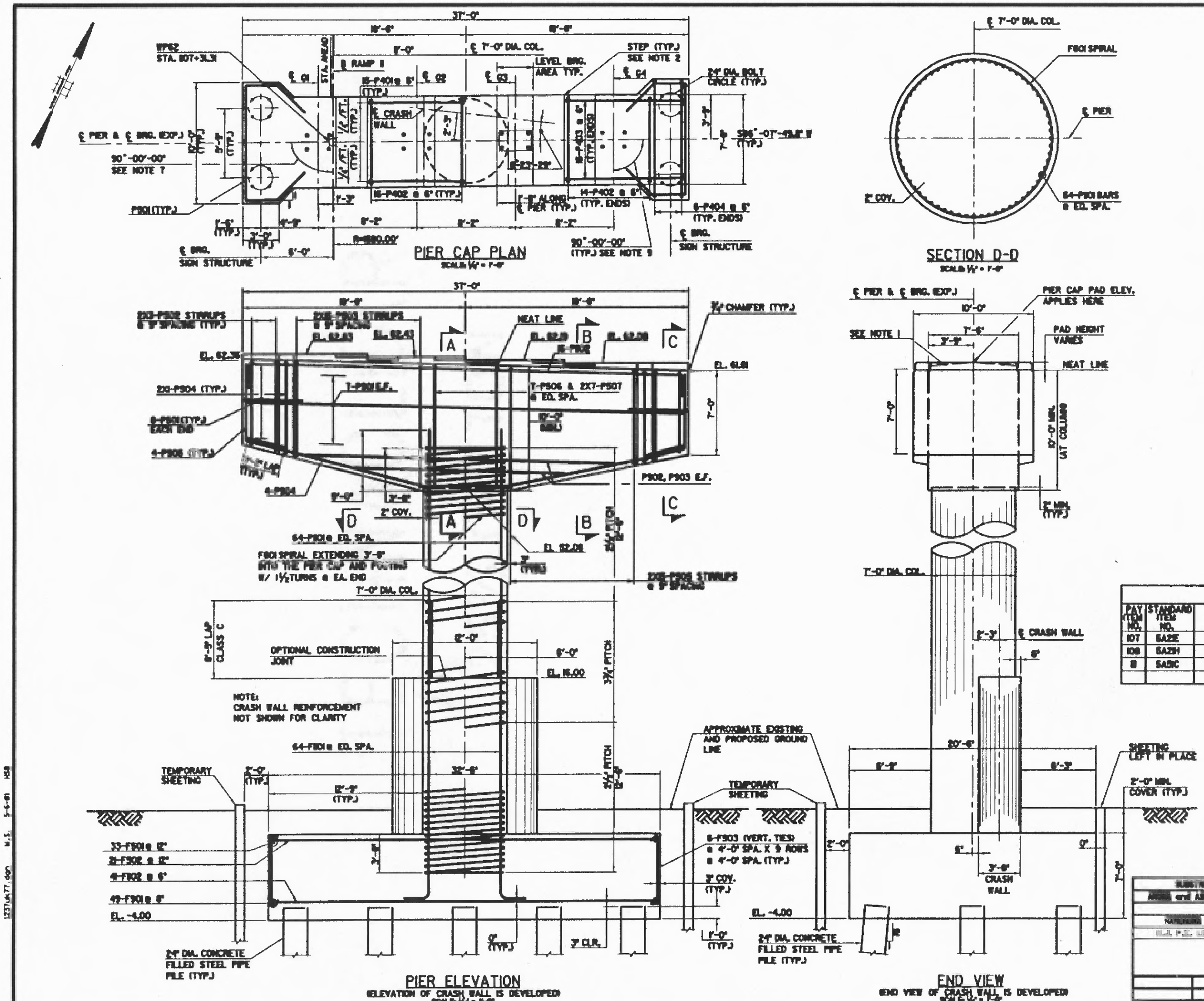
66

B-23 of B-476

J. ROMANO, J. MITCHELL, M. GARN, A. MARINO, F. YERMACK, H. BAL

FIGURE B.5 Pile and Footing Plan, Pier 33N

STRUCTURE NO.	0713 – 151
STRUCTURE NAME:	ROUTE 21 VIADUCT



NOTES:

1. SLOPE PIER CAP SEATS AT 1/4"/FT BETWEEN LEVEL BEARING AREAS.
2. STEP PIER CAP MIDWAY BETWEEN GIRDERS.
3. REINFORCEMENT EXTENDING FROM TOP OF PILE INTO FOOTING IS NOT SHOWN FOR CLARITY.
4. LAP SPLICE OF SPIRAL MAY BE PROVIDED WITHIN CENTER HALF OF COLUMN. LAP SPLICE SHALL BE MINIMUM 48 BAR DIAMETERS. USE FULL STRENGTH LAP WELD ELSEWHERE.
5. ARCHITECTURAL REVEALS NOT SHOWN FOR CLARITY.
6. CONTRACTOR MAY ADJUST THE PITCH OF SPIRAL REINFORCING AS NECESSARY, BUT NOT TO EXCEED 6 INCHES, TO ALLOW FOR THE PLACEMENT OF FOOTING AND PIER CAP REINFORCEMENT.
7. ANGLE IS FROM TANGENT TO RAMP II ℄ TO ℄ OF PIER.
8. FOR ADDITIONAL SIGN STRUCTURE ANCHORAGE INFORMATION SEE NJDOT PLATE NO. 3.9-6. SIGN STRUCTURE IS NOT IN THIS CONTRACT, HOWEVER, CONTRACTOR IS REQUIRED TO PROVIDE ANCHORAGE FOR THE SIGN STRUCTURE IN THE PIER CAP
9. ANGLES GIVEN AT THE INTERSECTION OF ℄ OF BEARINGS AND GIRDERS ARE MEASURED TO TANGENTS OF ARCS AT THE POINTS OF INTERSECTION.

REFERENCES:

1. FOR GENERAL NOTES, SEE GENERAL NOTES AND SUMMARY OF QUANTITIES –ROUTE 21 VIADUCT.
2. FOR PILE AND FOOTING PLAN, SEE DWG. NO. A3-P-1.
3. FOR PLAN OF PIER CAP REINFORCEMENT, SEE DWG. NO. A3-P-8.
4. FOR SECTIONS A-A, B-B AND C-C, SEE DWG. NO. A3-P-8.
5. FOR TYPICAL PIER DETAILS, SEE DWG. NO. C-P-2.
6. FOR ANCHOR BOLT DETAILS, SEE BEARING DETAIL SHEETS AND DWG. NO. A3-P-8.
7. FOR FULL STRENGTH LAP WELD DETAILS, SEE DWG. NO. C-P-2.
8. FOR PLACEMENT OF ARCHITECTURAL REVEALS, SEE DWG. A3-P-8.
9. FOR LIMITS OF EPOXY WATERPROOFING SEAL COAT, SEE DWG. NO. C-P-2.
10. FOR REINFORCEMENT SCHEDULE, SEE DWG. NO. A3-P-08.
11. FOR CRASH WALL DETAILS, SEE DWG. NO. C-P-3.

QUANTITIES

PAY ITEM NO.	STANDARD ITEM NO.	DESCRIPTION	UNIT	CONTRACT QUANTITY
107	5A21E	REINFORCEMENT STEEL IN STRUCTURES	LBS.	25,960
108	5A21H	EPOXY WATERPROOFING SEAL COAT	S.Y.	22
[illegible]	5A51C	CONCRETE IN SUBSTRUCTURES, PIER COLUMNS AND CAPS	C.Y.	[illegible]

CONTRACT A — DWG. A3-P-7

NEW JERSEY DEPARTMENT OF TRANSPORTATION

Pier Plan and Elevation
PIER 62

ROUTE 21 — SECTION 2N — RAMP II

SCALE: AS SHOWN — 137

BRIDGE SHEET NO. B-94 of B-476

EUGENE F. HETZEL, N.J. P.E. LIC. NO. 30525

REVISIONS | BY | CH'K'D | DATE

FIGURE B.6 Pier Plan and Elevation, Pier 62

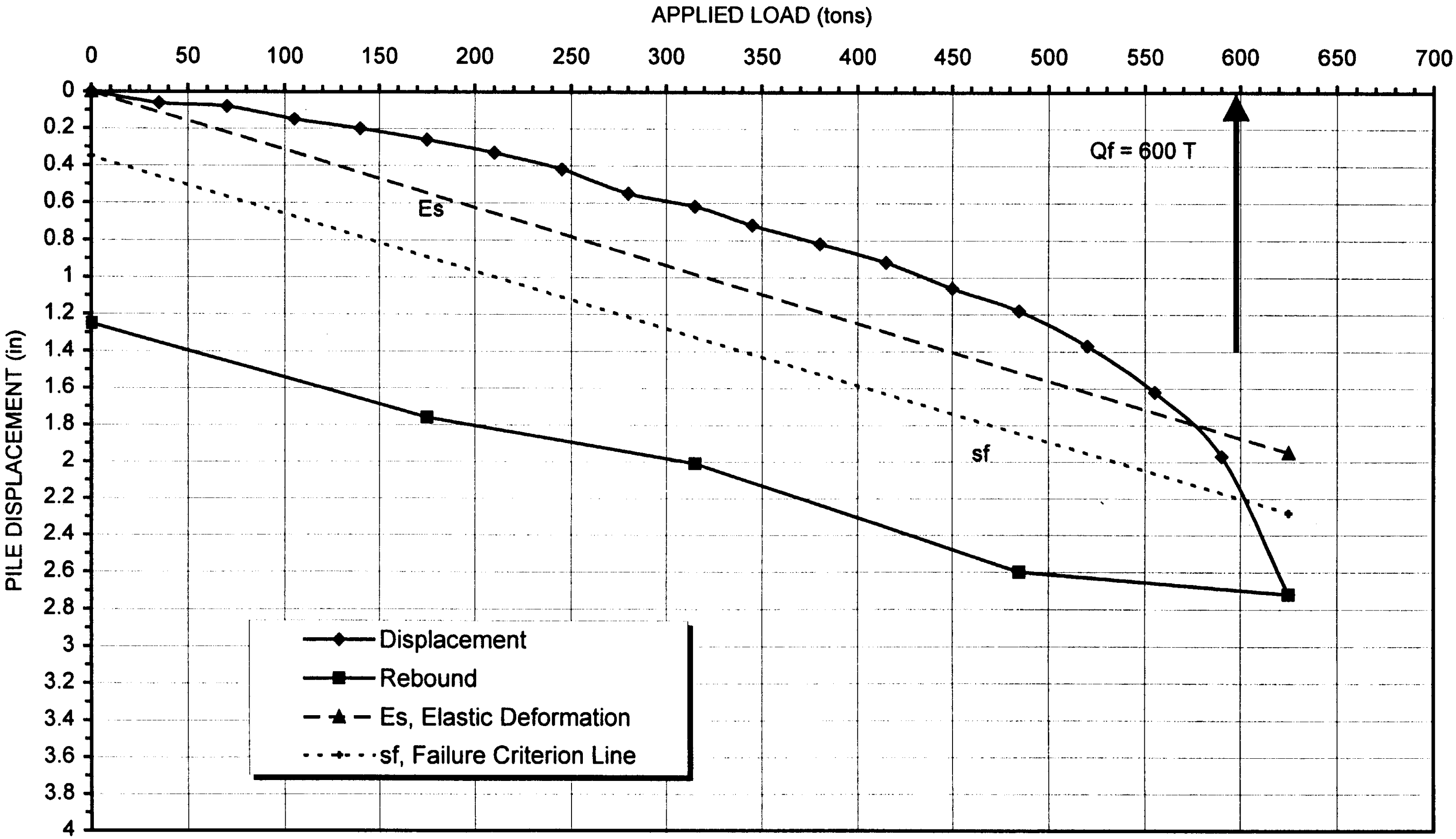


FIGURE B.7 Stattic Load Test for Pier 37S, Pile #30

FIGURE B.8 Static Load Test Data Hydraulic Jack Assembly

FIGURE B.9 Checking of Uplifting of Reaction Frame

FIGURE B.10 Pier Construction for I-78 Widening

FIGURE B.11 Pier Construction for Route 21 Viaduct

APPENDIX C

LAB TEST RESULTS, SELECTED BORING LOG

AND TYPICAL PDA/CAPWAP RESULT

In this Appendix, the summary of laboratory test results of selected borings that are relevant for this research work are included. A large number of borings were performed for the studied area and utilized for this project, however, a typical boring log is included. A typical PDA/CAPWAP analysis along with actual pile driving records are also presented.

Summary of Laboratory Test Results

A typical Boring Log

PDA/CAPWAP and Driving Record for Pier 39N

Static Pile Capacity Analysis

SUMMARY OF LABORATORY TESTS

PROJECT NUMBER: 98-37245-01

PROJECT: ROUTE 21, SECTION 2N, NEWARK VIADUCT

CLIENT: ARORA & ASSOCIATES, P.C./PARSONS BRINCKERHOFF

CONVERSE CONSULTANTS

BORING & SAMPLE NUMBER	DEPTH (M)	CLASSIFICATION	NATURAL WATER CONTENT (%)	ATTERBERG LIMITS		UNCONFINED COMPRESSION		UNIT DRY WEIGHT (KN/Cum)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	CONSOLIDATION	TRIAXIAL COMPRESSION	SPECIFIC GRAVITY	ORGANIC CONTENT (%)
				LIQUID LIMIT	PLASTIC LIMIT	STRESS (TSF)	STRAIN (%)							
S-902														
S-18	33-33.45	Red brown lean clay (CL)	21.1	27	18									
S-903														
S-18	33-33.45	Brown lean clay (CL)	20.0	27	19									
S-20	36-36.45	Red brown silt with sand (ML)							*	*				
S-22	39-39.45	Red brown silty sand (SM)							*					
S-24	42-42.45	Red brown silty sand (SM)							*					
S-917														
S-13	25.5-25.95	Red brown poorly graded sand with silt (SP-SM)							*					
S-17	31.5-31.95	Red brown sandy silt (ML)							*					
S-918														
S-13	25.5-25.95	Red brown silty sand (SM)							*					
S-15	28.5-28.95	Red brown silty sand (SM)							*					

* See Test Curves

SUMMARY OF LABORATORY TESTS

PROJECT NUMBER: 98-37245-01

PROJECT: ROUTE 21, SECTION 2N, NEWARK VIADUCT

CONVERSE CONSULTANTS

CLIENT: ARORA & ASSOCIATES, P.C./PARSONS BRINCKERHOFF

BORING & SAMPLE NUMBER	DEPTH (M)	CLASSIFICATION	NATURAL WATER CONTENT (%)	ATTERBERG LIMITS		UNCONFINED COMPRESSION		UNIT DRY WEIGHT (KN/Cum)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	CONSOLIDATION	TRIAXIAL COMPRESSION	SPECIFIC GRAVITY	ORGANIC CONTENT (%)
				LIQUID LIMIT	PLASTIC LIMIT	STRESS (TSF)	STRAIN (%)							
S-919														
S-11	22.5-22.95	Red brown poorly graded sand with silt (SP-SM)							*					
S-12	24-24.45	Red brown sandy silt (ML)							*					
S-14	27-27.45	Red brown silty sand (SM)							*					
S-921														
S-11	22.5-22.95	Red brown lean clay (CL)	21.6	29	20									
S-13	25.5-25.95	Brown silty clay (CL-ML)	19.9	25	21									
S-14	27-27.45	Red brown silty sand (SM)							*					
S-16	30.0-30.45	Red brown sandy silt (ML)							*					
S-20	36.0-36.45	Red brown silty sand (SM)							*					
S-922														
S-12	24.0-24.45	Red brown silty sand (SM)							*	*				
S-15	28.5-28.95	Red brown silty sand (SM)							*					

* See Test Curves

(SS80667 TBL)

SUMMARY OF LABORATORY TESTS

PROJECT NUMBER: 98-37245-01

PROJECT: ROUTE 21, SECTION 2N, NEWARK VIADUCT

CLIENT: ARORA & ASSOCIATES, P.C./PARSONS BRINCKERHOFF

CONVERSE CONSULTANTS

BORING & SAMPLE NUMBER	DEPTH (M)	CLASSIFICATION	NATURAL WATER CONTENT (%)	ATTERBERG LIMITS		UNCONFINED COMPRESSION		UNIT DRY WEIGHT (KN/Cum)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	CONSOLIDATION	TRIAXIAL COMPRESSION	SPECIFIC GRAVITY	ORGANIC CONTENT (%)
				LIQUID LIMIT	PLASTIC LIMIT	STRESS (TSF)	STRAIN (%)							
S-923														
S-11	22.5-22.95	Red brown silt with sand (ML)						*	*					
S-14	27-27.45	Red brown silty sand (SM)						*						
S-924														
S-13	25.5-25.95	Red brown silty sand (SM)						*						
S-14	27.0-27.45	Red brown silty sand (SM)						*						
S-15	28.5-28.92	Brown silt with sand (ML)	22.4	NP	NP									

* See Test Curves

[illegible]

SUMMARY OF LABORATORY TESTS

PROJECT NUMBER: 98-37245-01

PROJECT: ROUTE 21

CLIENT: ARORA AND ASSOCIATES, INC

CONVERSE CONSULTANTS

BORING & SAMPLE NUMBER	DEPTH (m)	CLASSIFICATION	NATURAL WATER CONTENT (%)	ATTERBERG LIMITS		UNCONFINED COMPRESSION		UNIT DRY WEIGHT (kN/cu.m)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	CONSOLIDATION	TRIAXIAL COMPRESSION	SPECIFIC GRAVITY	ORGANIC CONTENT (%)
				LIQUID LIMIT	PLASTIC LIMIT	STRESS (kPa)	STRAIN (%)							
S-901														
S-20	25.5- 25.95	Red brown lean clay (CL)							*	*				
S-23	30- 30.45	Red brown silt (ML)							*					
S-26														
S-28														
S-907														
UD-1	17.4-18	Red brown lean clay (CL)	25.5 25.1	30	22			 15.5			 *		 2.69	
S-927														
S-8	7.2- 7.65	Gray sandy silty (ML)							*					
UD-2	12.9- 13.5	Red brown silty clay (CL-ML)	22.3 21.0	24	20			 16.4	*	*	 *		 2.69	
UD-3	17.4- 18.0	Red brown lean clay (CL)	26.2 22.1 24.2 31.5	36	21			 16.3 14.7 14.5				 * * *		
UD-4	21.9- 22.5	Red brown silt (ML)	33.2 33.3 32.3	43	28			 14.3 14.4	*	*	 *	 *	 2.75	

* See Test Curves

(SSB0497.TBL)

PRELIMINARY

ENGINEER: Bohdan Pazuniak
DATE ASSIGNED
DATE DUE

JOB NO. 8229.1
JOB NAME Newark Viaduct;
Newark, NJ

DATE REC.
DATE CMP.
REC BY
Page No. 1

SUMMARY OF LABORATORY TEST RESULTS

BORING and SAMPLE No	DEPTH (feet)	CLASSIFICATION	SPECIAL TESTS	NATURAL WATER CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	UNCONFINED COMPRESS: STRESS (tsf)	UNCONFINED COMPRESS: STRAIN (%)	UNIT DRY WGT (pcf)	SPECIFIC GRAVITY	GRAIN SIZE: SIEVE	GRAIN SIZE: HYDRO	pH Test	ORGANIC CONTENT	DIRECT SHEAR: Ø	DIRECT SHEAR: C	PERMEABILITY: K20	PERMEABILITY: cm/s
S-178 S - 15	60'-61.5'			23.0	20	19												
S - 16	65'-66.5'														23.5°	300		
S-179 S - 15	60'-61.5'			17.3	NP						*							
S - 27	120'-121.5'			20.7							*							
S-181 S - 18	75'-76.5'														21°	100		
S - 19	80'-81.5'																	
S-182 S - 6	15'-16'.5			34.6	50	39							17	2				
S - 11	40'-41.5											3.8						
S-184 S - 7	20' - 21.5'			22.4						2.661	*							
S - 14	55'-56.5			22.9	19	14				2.690	*	*						

√ [illegible] NP

* [illegible]

ENGINEER: Bohdan Pazuniak

DATE ASSIGNED:

DATE OUT:

FILE NO.: 8229.1

JOB NAME: Newark Viaduct; Newark, NJ

DATE REC:

DATE CMP:

REC BY:

Page No. 2

SUMMARY OF LABORATORY TEST RESULTS

BORING and SAMPLE No.	DEPTH (feet)	CLASSIFICATION	SPECIAL TESTS	NATURAL WATER CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	UNCON. COMPRESS.: STRESS (tsf)	UNCON. COMPRESS.: STRAIN (%)	UNIT DRY WGT (pcf)	SPECIFIC GRAVITY	GRAIN SIZE: SIEVE	GRAIN SIZE: HYDR.	pH Test	ORGANIC CONTENT	DIRECT SHEAR: Ø	DIRECT SHEAR: C	PERMEABILITY: K20	PERMEABILITY: cm/s
S-186 S - 6	15'-16.5'			353.8	334	209								56.6				
S - 12	45'-46.5'												[illegible].9					
S - 22	95'-96.5'														18.7	205		
S - 23	100'-101.5'																	
S-190 S - 10	35'-36.5'				NP													
S - 17	70'-71.5'									2.724	*	*						
S-191 S - 5	15'-16.5'									2.655	*	*						
S - 20	90'-91.5'									2.692								
S - 22	100'-101.5'														21°	200		
S - 23	105'-106.5'																	
S-192 S - 10	35'-36.5'			22.3							*							

ENGINEER: Bohdan Pazuniak
DATE ASSIGNED
DATE DUE

FILE NO. 8229.1
JOB NAME Newark Viaduct; Newark, NJ

DATE REC
DATE CMP
REC BY
Page No 3

SUMMARY OF LABORATORY TEST RESULTS

BORING and SAMPLE No	DEPTH - feet	CLASSIFICATION	SPECIAL TESTS	NATURAL WATER CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	PLASTIC LIMIT	UNCONFINED COMPRESS: STRESS (psf)	STRAIN (%)	UNIT DRY WGT (pcf)	SPECIFIC GRAVITY	GRAIN SIZE: SIEVE	HYDRO	pH Test	ORGANIC CONTENT	DIRECT SHEAR: Ø	C	PERMEABILITY: K20	cm/s
S-194 UD - 1	68' - 70'			21.0	16	15	582	7										
S - 196 S - 20	85'-86.5'			29.0	34	18				2.707	*	*						
S - 197 S - 9	35'-36.5'									2.685	*	*						
S - 0	40'-41.5'			16.5														
S - 23	105'-106.5'			10.4					118.7								4.7x10^{-3}	
S-199 S - 6	20'-21.5'			23.9	NP													
S - 20	90'-91.5'																	
S - 21	95'-96.5'																	
S-199 S - 19	80'-81.5'			20.7					103.6								1.91x	10^{-}
S - 200 S - 6	20'-21.5'			25.0							*							

ENGINEER: Bohdan Pazuniak
DATE ASSIGNED:
DATE DUE:

FILE No. 8229.1
JOB NAME: Newark Viaduct;
Newark, NJ

DATE REC:
DATE CMP:
REC BY:
Page No 4

SUMMARY OF LABORATORY TEST RESULTS

BORING and SAMPLE No	DEPTH (feet)	CLASSIFICATION	SPECIAL TESTS	NATURAL WATER CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	PLASTIC LIMIT	UNCONF. COMPRESS.: STRESS (tsf)	STRAIN (%)	UNIT DRY WGT (pcf)	SPECIFIC GRAVITY	GRAIN SIZE: SIEVE	HYDRO	pH Test	ORGANIC CONTENT	DIRECT SHEAR: Ø	C	PERMEABILITY: K20	cm/s
S-201 S - 15	65'-66.5														27°	110		
S - 16	70'-71.5'																	
S - 17	75'-76.5'									2.663	*	*						
S-205 S - 11	40'-41.5'			22.8	NP					2.678	*	*						
S-207 S - 8	25'-26.5'			22.1	NP						*							
S-209 S - 6	15'-16.5			18.5	NP					2.67'	*	*						
S-198 S - 20															38.5	206		

* See Test Forms   √ Test Comp and Checked BP   # Test in Progress

Engineer: Bohdan Pazuniak
Date Assigned:
Date Out:

File No.: 8229.1
Job Name: Newark Viaduct; Newark, NJ

Date Rec:
Date Cmp: 5-1-95
Rec by: VK
Page No: 4

SUMMARY OF LABORATORY TEST RESULTS

Boring and Sample No.	Depth (feet)	Classification	Special Tests	Natural Water Content (%)	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Unconf. Compress.: Stress (tsf)	Unconf. Compress.: Strain (%)	Consolidation	Specific Gravity	Grain Size: Sieve	Grain Size: Hydro	pH Test	Organic Content	Direct Shear: Ø	Direct Shear: C	Permeability: K20	Permeability: cm/s
S-149 S - 12	45'-46.5'												7.0					
S-151 UD - 1	15' - 17'			53.0	84	62			*									
S-165 S - 6	15'-16.5'			156.4	NP									35.8				
S - 13	50'-51.5'												8.0					
S-172 S - 14	55'-56.5'			26.8											19	320		
S - 15	60'-61.5'																	

ENGINEER: Bohdan Pazuniak

DATE ASSIGNED:

DATE DUE:

JOB NO.: 8229.1

JOB NAME: Newark Viaduct; Newark, NJ

DATE REC:

DATE CMP: 5-1-95

REC BY: VK

Page No: 3

SUMMARY OF LABORATORY TEST RESULTS

Boring and Sample No.	Depth (feet)	Classification	Special Tests	Natural Water Content (%)	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Uncon. Compress.: Stress (tsf)	Uncon. Compress.: Strain (%)	Unit Dry Wt (pcf)	Specific Gravity	Grain Size: Sieve	Grain Size: Hydro	pH Test	Organic Content	Direct Shear: Ø	Direct Shear: C	Triaxial Test: Ø	Triaxial Test: C
S-116 S - 12	45'-46.5'			? 29.2	NP						*							
S - 122 S - 21	90'-91.5'			24.6	27	17				2.729	*	*						
S-127 S - 16	65'-66.5'			34.2	36	19				2.776	*	*						
S-132 S - 21	90'-91.5'			34.8						2.755								
S-138 UD - 1	17'-19'			31.4	39	24											0	1651
S-142 S - 7	20'-21.5'			35.4	41	21				2.627	*	*						
S-147 S - 14	55'-56.5'			20.4						2.688								
S-149 S - 6	15'-16.5'			101.2	104	78								15.0				

[illegible]

Arora and Associates, P.C.

ROUTE: RT. 21 LOCAL NAME: Newark Viaduct Replacement TEST HOLE NO. S-503
SECTION: 2N
STATION: 26-27 OFFSET: 32' LT REF. LINE: RT. 21 B.L. G. L. EL: 9.76
BORINGS MADE BY: Site Engineers DATE STARTED: 3/22/95
INSPECTOR: Y. S. Choksi DATE COMPLETED: 3/28/95

El. G.W.T.
0 HR. 2.96 DATE: 3/28/95
24 HR. 2.46 DATE: 3/29/95
148.0 ft. P.P. Inst. DATE: 3/28/95

Depth	Casing Blows	Sample No.	Depth (from)	Depth (to)	Blows on Spoon 0/6	6/12	12/18	Rec.	Sample ID and Profiles Changes
		S-1	0.0'	1.5'	8	15	20	1.0'	Dark Gray cf SAND, some Silt, trace mf Gravel
5									Tan and Gray cf SAND, little Silt,
		S-2	5.0'	6.5'	6	4	2	0.3'	trace cf Gravel
		S-3	6.5'	8.0'	3	5	5	1.1'	Reddish Brown mf SAND, little Silt, trace f Gravel
		S-4	8.0'	9.5'	4	6	7	1.3'	Same
10									
		S-5	10.0'	11.5'	4	4	4	1.0'	Same
	A								
	U								
15	G								
	E	S-6	15.0'	16.5'	2	3	4	1.0'	Same
	R								
	S								
20									
		S-7	20.0'	21.5'	3	3	4	1.2'	Reddish Brown f SAND, trace (+) Silt
25									
		S-8	25.0'	26.5'	5	5	7	1.0'	Same
30									
		S-9	30.0'	31.5'	4	6	7	1.1'	Same
35									
		S-10	35.0'	36.5'	7	9	11	0.9'	Same
40									

Nominal I.D. of Drive Pipe 2 1/2" 3 1/2" 4"
Nominal I.D. of Split Barrel Sampler 1 1/2"
Weight of hammer on Drive Pipe 300 lbs.
Weight of hammer on Split Barrel Sampler 140 lbs.
Drop of hammer on Drive Pipe 24"
Drop of hammer on Split Barrel Sampler 30"

The subsurface information shown hereon was obtained for A & A design and estimated purposes. It is made available to authorized users only that they may have access to the same information available to A & A. It is presented in good faith, but is not intended as a substitute for investigations, interpretation or judgement of such authorized users.

Core Dia. ______________

Approximate Change in Strata ______

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

Inferred Change in Strata _ _ _ _ _ _

Arora and Associates, P.C.

ROUTE: RT. 21 LOCAL NAME: Newark Viaduct Replacement TEST HOLE NO. S-503
SECTION: 2N
STATION: 26+27 OFFSET: 32' LT REF. LINE: RT. 21 B.L. G. L. EL: 9.76
BORINGS MADE BY: Site Engineers DATE STARTED: 3/22/95
INSPECTOR: J. Walker DATE COMPLETED: 3/28/95

El. G.W.T.
0 HR. 2.96 DATE: 3/28/95
24 HR. 2.46 DATE: 3/29/95
148.0 ft. P.P. Inst. DATE: 3/28/95

Casing Blows	Sample No.	Depth		Blows on Spoon 0 / / 6	6 / /12	12/ /18	Rec.	Sample ID and Profiles Changes
	S-11	40.0'	41.5'	7	12	11	1.0'	Reddish Brown f SAND, trace Silt
45								
	S-12	45.0'	46.5'	8	10	10	1.1'	Same
50 ROTARY								
	S-13	50.0'	51.5'	9	12	13	1.3'	Same
55 MUD								
	S-14	55.0'	56.5'	7	10	10	1.5'	Reddish Brown f SAND, some Clayey Silt
60								
	S-15	60.0'	61.5'	8	11	12	1.5'	Reddish Brown f SAND, little Clayey Silt
65								
	S-16	65.0'	66.5'	9	12	11	1.5'	Same
70								
	S-17	70.0'	71.5'	10	13	16	1.4'	Same
75								
	S-18	75.0'	76.5'	12	16	19	1.2'	Reddish Brown mf SAND, little Silt, trace f Gravel
80								

Nominal I.D. of Drive Pipe 2 1/2" 3 1/2" 4"
Nominal I.D. of Split Barrel Sampler 1 1/2"
Weight of hammer on Drive Pipe 300 lbs.
Weight of hammer on Split Barrel Sampler 140 lbs.
Drop of hammer on Drive Pipe 24"
Drop of hammer on Split Barrel Sampler 30"

The subsurface information shown hereon was obtained for A & A design and estimated purposes. It is made available to authorized users only that they may have access to the same information available to A & A. It is presented in good faith, but is not intended as a substitute for investigations, interpretation or judgement of such authorized users.

Core Dia. __________

Approximate Change in Strata __________

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

Inferred Change in Strata - - - - - - -

Arora and Associates, P.C.

ROUTE: RT. 21 LOCAL NAME: Newark Viaduct Replacement TEST HOLE NO. S-503

SECTION: 2N

STATION: 26+27 OFFSET: 32' LT REF. LINE: RT. 21 B.L. G. L. EL: 9.76

BORINGS MADE BY: Site Engineers DATE STARTED: 3/22/95

INSPECTOR: Y. S. Choksi DATE COMPLETED: 3/28/95

El. G.W.T.

0 HR. 2.96 DATE: 3/28/95

24 HR. 2.46 DATE: 3/29/95

148.0 ft. P.P. Inst. DATE: 3/28/95

Depth	Casing Blows	Sample No.	Depth		Blows on Spoon 0 / / 6	6 / /12	12/ /18	Rec.	Sample ID and Profiles Changes
	ROTARY	S-19	80.0'	81.5'	15	15	18	1.3'	Same
85		S-20	85.0'	86.5'	17	21	23	1.1'	Reddish Brown cf SAND, little Silt, trace of Gravel
90	MUD	S-21	90.0'	91.5'	21	20	20	1.0'	Same
95		S-22	95.0'	96.5'	19	24	27	1.2'	Same
100		S-23	100.0	101.5	25	41	25	1.0'	Reddish Brown cf SAND, little cf Gravel, trace Silt
105		S-24	105.0	106.5	21	25	24	1.0'	Same
110		S-25	110.0	111.0	48	70	50/ 0.3'	0.6'	Reddish Brown cf SAND, some cf Gravel, trace Silt
115		S-26	115.0	116.5	29	26	30	1.1'	Same
120									

Nominal I.D. of Drive Pipe 2 1/2" 3 1/2" 4"

Nominal I.D. of Split Barrel Sampler 1 1/2"

Weight of hammer on Drive Pipe 300 lbs.

Weight of hammer on Split Barrel Sampler 140 lbs.

Drop of hammer on Drive Pipe 24"

Drop of hammer on Split Barrel Sampler 30"

The subsurface information shown hereon was obtained for A & A design and estimated purposes. It is made available to authorized users only that they may have access to the same information available to A & A. It is presented in good faith, but is not intended as a substitute for investigations, interpretation or judgement of such authorized users.

Core Dia. ____________

Approximate Change in Strata ________

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

Inferred Change in Strata _ _ _ _ _ _

Arora and Associates, P.C.

ROUTE: RT. 21 LOCAL NAME: Newark Viaduct Replacement TEST HOLE NO. S-503
SECTION: 2N
STATION: 26+27 OFFSET: 32' LT REF. LINE: RT. 21 B.L. G. L. EL: 9.76
BORINGS MADE BY: Site Engineers DATE STARTED: 3/22/95
INSPECTOR: Y. S. Choksi DATE COMPLETED: 3/28/95

El. G.W.T.
0 HR. 2.96 DATE: 3/28/95
24 HR. 2.46 DATE: 3/29/95
148.0 ft. P.P. Inst. DATE: 3/28/95

Casing Blows	Sample No.	Depth		Blows on Spoon 0 / / 6	6 / /12	12/ /18	Rec.	Sample ID and Profiles Changes
	S-27	120.0	121.5	23	28	25	1.0'	Reddish Brown cf SAND, little (+) cf Gravel, trace Silt
125								
	S-28	125.0	126.5	18	26	30	1.1'	Same
R								
O								
T								
130 A								
R	S-29	130.0	131.5	26	31	40	1.0'	Same
Y								
M								
135 U								
D	S-30	135.0	136.1	31	50	50/ 0.1'	0.9'	Same
140								
	S-31	140.0	141.5	26	37	50	1.1'	Same
145								
	S-32	145.0	146.3	30	36	60/ 0.3'	1.0'	Same
150								
	S-33	150.0	151.0	38	61	50/ 0.0'	0.7'	Same
155								
	S-34	155.0	156.5	29	41	50	1.1'	Same
160								

Nominal I.D. of Drive Pipe 2 1/2" 3 1/2" 4"
Nominal I.D. of Split Barrel Sampler 1 1/2"
Weight of hammer on Drive Pipe 300 lbs.
Weight of hammer on Split Barrel Sampler 140 lbs.
Drop of hammer on Drive Pipe 24"
Drop of hammer on Split Barrel Sampler 30"

The subsurface information shown hereon was obtained for A & A design and estimated purposes. It is made available to authorized users only that they may have access to the same information available to A & A. It is presented in good faith, but is not intended as a substitute for investigations, interpretation or judgement of such authorized users.

Core Dia. __________

Approximate Change in Strata __________

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

Inferred Change in Strata _ _ _ _ _ _ _

Arora and Associates, P.C.

ROUTE: RT. 21 LOCAL NAME: Newark Viaduct Replacement TEST HOLE NO. S-503
SECTION: 2N
STATION: 26+27 OFFSET: 32' LT REF. LINE: RT. 21 B.L. - G. L. EL: 9.76
BORINGS MADE BY: Site Engineers DATE STARTED: 3/22/95
INSPECTOR: Y. S. Choksi DATE COMPLETED: 3/28/95

El. G.W.T.
0 HR. 2.96 DATE: 3/28/95
24 HR. 2.46 DATE: 3/29/95
148.0 ft. P.P. Inst.
DATE: 3/28/95

Casing Blows	Sample No.	Depth		Blows on Spoon 0 / / 6	6 / /12	12/ /18	Rec.	Sample ID and Profiles Changes
	S-35	160.0	161.5	26	31	30	1.2'	Same
165								
	S-36	165.0	166.4	31	48	68/ 0.4'	1.1'	Same
R								
O								
170 T								
A	S-37	170.0	171.5	28	37	46	1.2'	Same
R								
Y								
175 M								
U	S-38	175.0	176.5	31	37	40	1.3'	Reddish Brown cf SAND, some cf Gravel, trace Silt
D								
180								
	S-39	180.0	181.0	41	70	50/ 0.0'	1.0'	Same
185								
	S-40	185.0	186.5	31	39	44	1.3'	Reddish Brown cf SAND, and cf Gravel, trace Silt
190								
	S-41	190.0	191.5	29	42	48	1.5'	Same
195								
	S-42	195.0	196.5	35	41	50	1.1'	Reddish Brown cf SAND, little cf Gravel, trace Silt
200								

Nominal I.D. of Drive Pipe 2 1/2" 3 1/2" 4"
Nominal I.D. of Split Barrel Sampler 1 1/2"
Weight of hammer on Drive Pipe 300 lbs.
Weight of hammer on Split Barrel Sampler 140 lbs.
Drop of hammer on Drive Pipe 24"
Drop of hammer on Split Barrel Sampler 30"

The subsurface information shown hereon was obtained for A & A design and estimated purposes. It is made available to authorized users only that they may have access to the same information available to A & A. It is presented in good faith, but is not intended as a substitute for investigations, interpretation or judgement of such authorized users.

Core Dia. ____________

Approximate Change in Strata ____________

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

Inferred Change in Strata _ _ _ _ _ _ _

Arora and Associates, P.C.

ROUTE: RT. 21 LOCAL NAME: Newark Viaduct Replacement TEST HOLE NO. S-503
SECTION: 2N
STATION: 26+27 OFFSET: 32' LT REF. LINE: RT. 21 B.L. G. L. EL: 9.76
BORINGS MADE BY: Site Engineers DATE STARTED: 3/22/95 El. G.W.T.
INSPECTOR: Y. S. Choksi DATE COMPLETED: 3/28/95 0 HR. 2.96 DATE: 3/28/95
24 HR. 2.46 DATE: 3/29/95
148.0 ft. P.P. Inst. DATE: 3/29/95

Casing Blows	Sample No.	Depth		Blows on Spoon 0 / /6	6 / /12	12/ /18	Rec.	Sample ID and Profiles Changes	
R	S-43	200.0	201.5	38	52	[illegible]	1.1	Reddish Brown cf SAND, and cf Gravel, trace Silt	
O									
T									
A									
205 R									
Y	S-44	205.0	206.0	47	80	50/ [illegible]	0.7'	Same	
M								- - - - - - - - - -	208.0
U									
210 D									
	S-45	210.0	211.0	[illegible]	90	100/ [illegible]	0.5'	Reddish Brown cf GRAVEL, little cf Sand, trace Silt (Decomposed SHALE)	211.0
								Bottom of Hole	
215									
								Note: 148.0' 3/4" Dia. Casagrande Piezometer installed.	
220									
225									
230									
235									
240									

Nominal I.D. of Drive Pipe 2 1/2" 3 1/2" ✓ 4"
Nominal I.D. of Split Barrel Sampler 1 1/2"
Weight of hammer on Drive Pipe 300 lbs.
Weight of hammer on Split Barrel Sampler 140 lbs.
Drop of hammer on Drive Pipe 24"
Drop of hammer on Split Barrel Sampler 30"

The subsurface information shown hereon was obtained for A & A design and estimated purposes. It is made available to authorized users only that they may have access to the same information available to A & A. It is presented in good faith, but is not intended as a substitute for investigations, interpretation or judgement of such authorized users.

Core Dia. ______________________

Approximate Change in Strata ________

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

Inferred Change in Strata _ _ _ _ _ _

SUMMARY OF DYNAMIC LOAD TEST RESULTS

NJDOT RT.21/78 - Sec. 2N & 5CK

DATE: July 23, 1997

LOCATION: Rte 21/78

HAMMER: Conmaco 5300

PILE NUMBER: Pile 18 Pier 39 North

PILE TYPE: 24" OD x 0.500" Flat Plate at toe

INFORMATION FROM PDA

MAXIMUM STRESS:	32.6	KSI
CAPACITY AT END OF DRIVE:	1073	KIPS
TRANSFERRED ENERGY AT END OF DRIVE:	90	KIP-FT
BLOWS PER MINUTE AT END OF DRIVE:	37	BL/MIN

INFORMATION FROM CAPWAP ANALYSIS

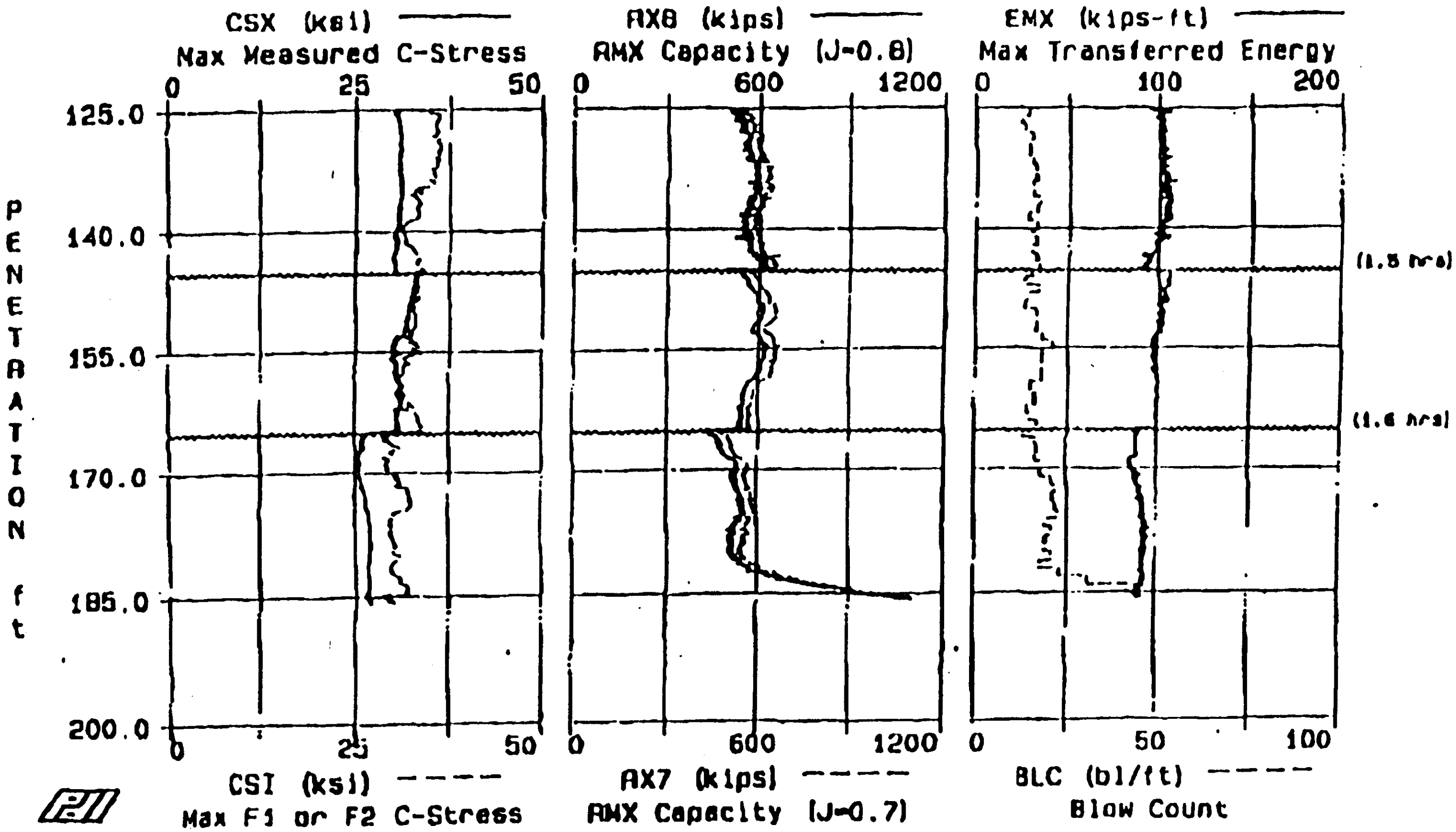
MAXIMUM STRESS:	32.5	KSI
TOTAL CAPWAP CAPACITY:	1182	KIPS
SHAFT RESISTANCE:	153	KIPS
TOE RESISTANCE:	1029	KIPS

Abrden Teferra

GRL AND ASSOCIATES, INC.

GRL & Associates, Inc. 23-Jul-97

NJDOT RT 21/78, P18N (125'-185.67' PENTR), CONM 5300 24"ODX0.5"



Pile: P18N (125'-185.67' PENTR)   Proj: NJDT RT 21/78   Pg1
Info: CONM 5300 24"ODX0.5"   SP: 0.492 k/ft^3
AR: 36.9 in^2   WS: 16810 ft/s
LE: 196.0 ft   EM: 30000 KSI

CSX: Max Measured C-Stress   BPM: Blows Per Minute
CSI: Max F1 or F2 C-Stress   RAU: Capacity - RAU
RX8: RMX Capacity (J=0.8)   RX7: RMX Capacity (J=0.7)
FMX: Max Measured Force   RA2: Capacity - RA2
EMX: Max Transferred Energy

BL# end	bl/ft	depth ft	TY	CSX ksi	CSI ksi	RX8 kips	FMX kips	EMX kips-ft	BPM bl/min	RAU kips	RX7 kips	RA2 kips
2	14	125.00	AV	30.15	30.18	603	1123	87.9	32.3	443	636	694
16	14	126.00	AV	30.45	35.70	523	1124	100.2	34.6	381	557	604
28	12	127.00	AV	30.66	35.67	536	1132	100.6	34.7	413	570	631
41	13	128.00	AV	30.75	35.89	542	1135	102.0	34.7	418	580	636
56	15	129.00	AV	30.90	35.72	563	1141	101.2	34.9	426	598	655
70	14	130.00	AV	30.82	36.38	555	1138	101.2	35.0	413	595	656
84	14	131.00	AV	30.72	35.89	565	1134	102.1	35.1	402	604	663
99	15	132.00	AV	30.63	35.80	575	1131	102.6	35.2	407	613	667
115	16	133.00	AV	30.75	35.32	589	1135	102.0	35.4	413	628	675
131	16	134.00	AV	30.83	35.14	590	1138	103.7	35.5	413	629	664
146	15	135.00	AV	30.88	33.94	585	1140	103.7	35.5	408	625	655
161	15	136.00	AV	30.93	33.01	587	1142	103.5	35.6	409	625	661
178	17	137.00	AV	30.70	33.36	570	1133	105.0	35.6	400	610	642
194	16	138.00	AV	30.60	33.09	563	1130	103.6	35.6	411	599	636
208	14	139.00	AV	30.57	32.53	548	1128	104.2	35.6	403	584	626
224	16	140.00	AV	30.41	31.09	548	1123	101.6	35.5	405	581	631
239	15	141.00	AV	30.25	31.41	556	1117	100.6	35.5	422	590	656
255	16	142.00	AV	30.38	31.73	563	1122	100.6	35.7	428	597	666
270	15	143.00	AV	30.16	32.72	572	1113	98.5	35.6	429	606	687
286	16	144.00	AV	30.11	33.20	604	1111	94.4	35.6	450	637	723
303	17	145.00	AV	30.03	33.31	606	1109	91.2	35.6	445	638	745
318	15	146.00	AV	32.68	33.13	544	1207	105.2	32.7	399	580	641
331	13	147.00	AV	32.61	33.24	562	1204	104.6	35.3	420	595	659
348	17	148.00	AV	32.46	33.19	580	1198	103.5	35.0	437	614	691
362	14	149.00	AV	32.34	32.97	598	1194	103.3	35.0	454	632	705
375	13	150.00	AV	32.05	32.44	608	1183	101.8	35.0	464	644	706
391	16	151.00	AV	31.96	32.76	613	1180	102.1	35.0	469	653	708
407	16	152.00	AV	31.70	32.75	601	1170	101.7	34.9	466	641	716
423	16	153.00	AV	31.50	32.36	581	1163	101.1	34.9	459	620	729
441	18	154.00	AV	30.41	32.18	602	1123	98.2	35.2	472	641	723
462	21	155.00	AV	29.97	32.78	616	1106	97.4	35.6	488	653	749
480	18	156.00	AV	30.10	32.06	618	1111	97.6	35.8	486	653	751
498	18	157.00	AV	30.27	31.46	609	1117	97.8	36.0	478	647	742
516	18	158.00	AV	30.38	31.16	603	1121	97.7	35.8	467	643	736
534	18	159.00	AV	30.59	30.82	584	1129	99.6	35.8	450	628	721
549	15	160.00	AV	30.82	31.10	559	1138	100.1	35.5	422	596	712
565	16	161.00	AV	30.66	31.39	549	1132	99.6	35.2	418	583	713
581	16	162.00	AV	31.10	32.23	548	1148	99.4	35.0	416	580	734
595	14	163.00	AV	30.51	32.48	541	1126	99.4	34.9	400	569	709
610	15	164.00	AV	30.53	33.35	543	1127	99.4	34.8	391	571	713
627	17	165.00	AV	30.34	33.31	535	1120	98.7	34.7	381	562	702
640	13	166.00	AV	26.18	29.46	442	967	88.5	34.8	309	483	544
656	16	167.00	AV	25.92	29.88	468	957	87.9	35.3	313	507	560
673	17	168.00	AV	25.64	29.82	476	947	87.7	35.3	303	511	579
689	16	169.00	AV	25.51	29.10	506	942	86.5	35.4	316	537	611

Pile: P18N (125'-185.67' PENTR)   Proj: NJDT RT 21/78   Pg2
Info: CONM 5300 24"ODX0.5"

BL# end	bl/ft	depth ft	TY	CSX ksi	CSI ksi	RX8 kips	FMX kips	EMX kips-ft	BPM bl/min	RAU kips	RX7 kips	RA2 kips
707	18	170.00	AV	25.45	29.26	529	940	84.6	35.5	393	564	644
725	18	171.00	AV	25.63	30.28	520	946	87.5	35.5	347	557	621
746	21	172.00	AV	25.81	30.95	522	953	88.1	35.7	356	560	619
768	22	173.00	AV	26.18	31.94	535	967	89.8	35.6	364	571	626
789	21	174.00	AV	26.35	32.27	544	973	90.6	35.6	368	582	626
811	22	175.00	AV	26.57	31.33	547	981	92.3	35.5	361	586	616
834	23	176.00	AV	26.79	29.75	543	989	92.1	35.4	362	581	622
853	19	177.00	AV	26.88	29.81	526	992	93.0	35.3	341	568	611
874	21	178.00	AV	26.97	29.80	515	996	93.4	35.3	327	560	612
893	19	179.00	AV	27.07	30.17	509	999	93.4	35.3	328	552	624
912	19	180.00	AV	27.00	30.24	510	997	93.1	35.3	308	543	625
933	21	181.00	AV	25.92	30.35	526	994	92.3	35.4	324	555	639
951	18	182.00	AV	26.83	29.79	556	990	91.9	35.5	351	594	655
974	23	183.00	AV	26.92	30.46	613	994	92.2	35.7	368	651	686
1005	31	184.00	AV	26.84	31.35	719	991	92.5	36.2	384	757	745
1049	44	185.00	AV	26.74	31.07	870	987	90.4	36.7	457	902	878
1103	80	185.67	AV	26.86	29.60	1017	992	89.3	37.0	504	1047	990

BL#	COMMENTS
2	JC = 0.80
302	DATA MERGE: P18N.Q06
625	DATA MERGE: P18N.Q07

DRIVE TIME SUMMARY (23-Jul-97 : P18N.Q07)	DRIVE	minutes	WAIT
BN 2 -> 302, START 12:12:47 -> 12:22:07 STOP,	9.33		
			89.65
BN 303 -> 625, START 13:51:46 -> 14:01:03 STOP,	9.28		
			97.58
BN 627 -> 1103, START 15:38:38 -> 15:51:57 STOP,	13.32		
Total Elapsed time 219.17 minutes Total Time	31.93	minutes	187.23

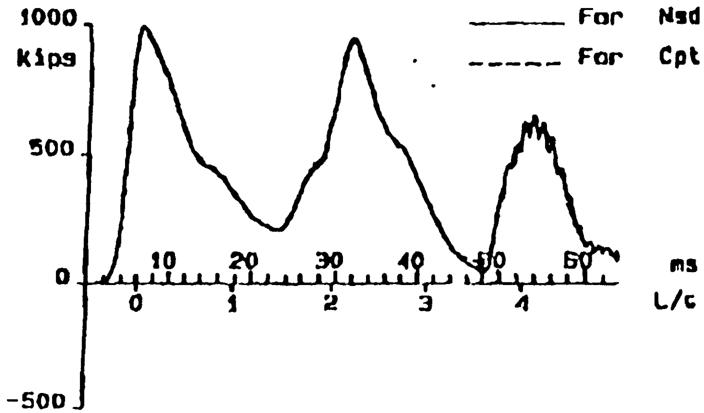
Pile: P18N (125'-185.67' PENTR) Proj: NJDT RT 21/73 Pg1
Info: CONM 5300 24"ODX0.5" SP: 0.492 k/ft^3
AR: 36.9 in^2 WS: 16810 ft/s
LE: 196.0 ft EM: 30000 KSI

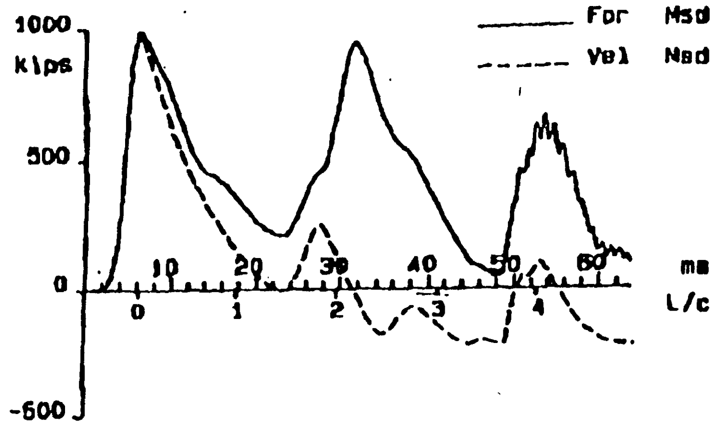
CSX: Max Measured C-Stress BPM: Blows Per Minute
CSI: Max F1 or F2 C-Stress RAU: Capacity - RAU
RX8: RMX Capacity (J=0.8) RX7: RMX Capacity (J=0.7)
FMX: Max Measured Force RA2: Capacity - RA2
EMX: Max Transferred Energy

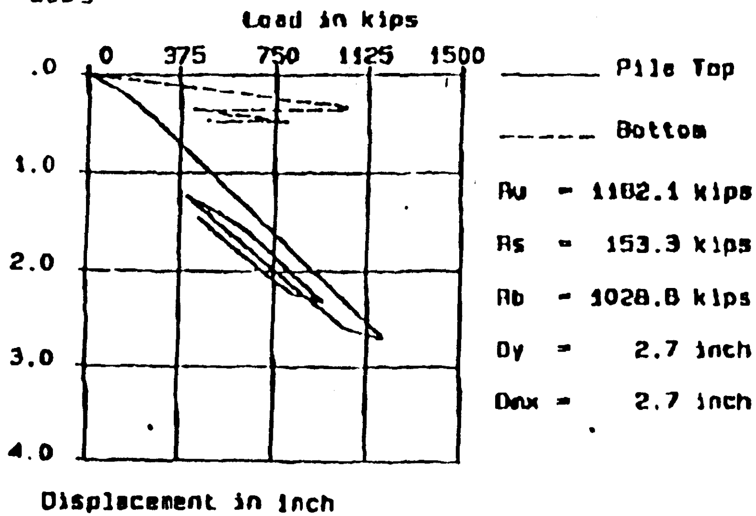
BL#	depth ft	CSX ksi	CSI ksi	RX8 kips	FMX kips	EMX kips-ft	BPM bl/min	RAU kips	RX7 kips	RA2 kips	BLC bl/ft
1047		26.71	28.98	911	986	90.9	36.8	444	942	890	44
1048		26.52	28.90	925	979	87.1	36.6	487	956	960	44
1049	185.00	26.76	29.34	927	988	90.6	36.9	480	960	909	44
1050		26.55	28.85	930	980	88.6	36.7	478	960	900	80
1051		26.32	29.07	950	979	87.1	36.6	528	975	968	80
1052		26.28	28.88	948	970	85.1	36.4	543	979	960	80
1053		26.19	28.44	948	967	84.7	36.4	537	980	960	80
1054		27.03	29.72	954	998	91.2	36.9	497	981	927	80
1055		26.57	29.34	939	981	90.0	36.7	448	972	900	80
1056		26.82	29.50	960	990	90.6	36.9	492	991	980	80
1057		26.74	29.50	960	987	89.4	36.8	493	992	944	80
1058		26.30	29.20	954	971	86.9	36.4	479	985	935	80
1059		26.76	29.66	973	988	88.8	36.7	530	1004	981	80
1060		26.25	28.77	958	969	85.6	36.3	481	988	963	80
1061		26.95	29.66	994	995	88.5	37.0	556	1024	1035	80
1062		26.41	28.79	963	975	88.1	36.8	475	994	943	80
1063		26.63	29.23	978	983	88.7	36.9	492	1009	977	80
1064		26.95	29.58	978	995	90.7	37.1	495	1010	986	80
1065		26.63	29.28	971	983	89.9	36.8	471	1002	912	80
1066		26.63	29.01	979	983	89.4	36.9	493	1012	949	80
1067		27.01	29.72	985	997	91.7	37.1	477	1017	948	80
1068		26.68	29.50	993	985	88.1	37.0	488	1023	978	80
1069		26.98	29.72	1000	996	89.9	37.1	522	1032	985	80
1070		26.93	29.85	997	994	89.7	37.2	509	1029	984	80
1071		26.57	29.04	991	981	88.4	36.9	477	1021	973	80
1072		27.14	29.85	1014	1002	90.4	37.2	526	1043	994	80
1073		26.82	29.50	1011	990	89.8	37.2	500	1040	987	80
1074		26.93	29.58	1023	994	89.5	37.3	504	1051	1040	80
1075		26.90	29.72	1023	993	89.3	37.3	499	1052	1036	80
1076		26.86	29.61	1012	991	89.6	37.2	492	1043	952	80
1077		26.95	29.63	1029	995	90.3	37.2	541	1061	989	80
1078		26.87	29.85	1007	992	91.3	37.0	451	1038	919	80
1079		27.09	29.96	1035	1000	90.1	37.3	509	1064	1039	80
1080		26.98	29.85	1027	996	89.4	37.3	494	1058	987	80
1081		26.79	29.50	1058	989	87.4	37.0	570	1084	1089	80
1082		27.01	29.74	1050	997	89.2	37.2	535	1077	1039	80
1083		26.79	29.53	1035	989	88.7	37.0	493	1066	983	80
1084		26.93	29.58	1038	994	89.9	37.1	495	1069	986	80
1085		26.63	29.09	1037	983	88.3	37.1	495	1067	984	80
1086		26.90	29.72	1066	993	88.0	37.1	555	1092	1087	80
1087		26.98	29.85	1051	996	90.2	37.3	499	1078	990	80
1088		27.30	30.28	1079	1008	89.5	37.3	581	1107	1102	80
1089		26.68	29.28	1069	985	86.6	37.0	562	1094	1038	80
1090		27.20	30.04	1051	1004	93.0	37.5	459	1081	973	80
1091		27.36	30.39	1069	1010	91.2	37.5	505	1099	1049	80
1092		27.09	30.07	1059	1000	90.5	37.4	480	1086	1000	80

NJDOT RT 21/78, PIER 39 NORTH, P18 @ 185.67', BN: 471 — 23-Jul 97

Goble Rausche Likins & Associates, Inc. — CAPWAP(R) Version 1996-2







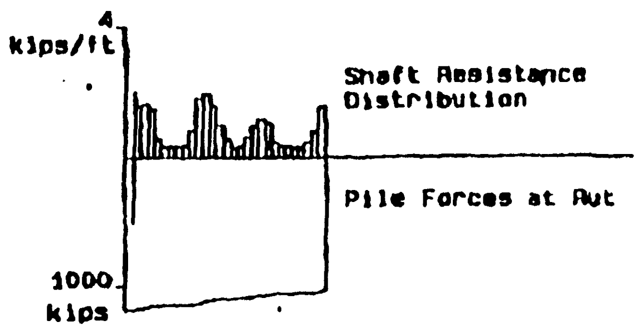
Ru = 1182.1 kips

Rs = 153.3 kips

Rb = 1028.8 kips

Dy = 2.7 inch

Dmx = 2.7 inch



GEORGE HARMS CONSTRUCTION CO., INC.

NJDOT Route 21 & 78 Sections 2H & 6CK

PIPE PILES (18" & 24")

PILE DRIVING RECORD

LOCATION: 39 N

PILE #: 18

DIAMETER: 18"__ 24" X

TIP: POINT__ PLATE X

TYPE: PRODUCTION__ TEST X

TYPE: 81:__ 42__ 43__ 812__ 409__ 824__

WALL THICKNESS: 3/8"__ 1/2" X

POSITION: BAT__ PLUMB X

PILE HAMMER: 60 S

CRANE: 248

PDA MONITORED? Y__ N__

DEPTH STARTED: 48

DEPTH AT COMPLETION: 105

ORIGINAL GROUND ELEV: 10 ±

ESTIMATED TIP ELEV:

ACTUAL TIP ELEV: −93.12

CUT OFF ELEV: −5

REFERENCE ELEV: 9.88

TOTAL PILE LENGTH (A): 115

LENGTH ABOVE CUT-OFF (B): 10.77

LENGTH BELOW CUT-OFF (C): 104.62

REMARKS:

DATE: 7/21/97

TIME STARTED: 11 05

TIME STOPPED:

FOREMAN: JS

INSPECTOR: Bill

NOTE: A − B = C

FEET PEN.	INCHES	BLOWS	BLOWS /FOOT	FEET PEN.	INCHES	BLOWS	BLOWS /FOOT	FEET PEN.	INCHES	BLOWS	BLOWS /FOOT
1	1			37	4			73	1	13	
2	2			38	2			74	2	14	
3	3			39	3			75	4	15	
4	4			40	4			76	4	19	
5	5			41	5			77	5	30	
6	6			42	6			78	6	30	
7	7			43	7			79	7	38	
8	8			44	8			80	8	38	
9	9			45	9			81	9	39	
10	10			46	10			82	10	42	− Splice
11	11			47	11			83	11	40	
12	12			48	12			84	12	38	
13	1			49	1	12		85	1	39	
14	2			50	2	18		86	2	35	
15	3			51	3	22		87	3	33	
16	4			52	4	20		88	4	31	
17	5			53	5	19		89	5	31	
18	6			54	6	14		90	6	28	
19	7			55	7	14		91	7	27	
20	8			56	8	13		92	8	27	
21	9			57	9	13		93	9	[illegible]	
22	10			58	10	13		94	10	[illegible]	
23	11			59	11	13		95	11	[illegible]	
24	12			60	12	14		96	12	[illegible]	
25	1			61	1	13		97	1	[illegible]	
26	2			62	2	12		98	2	35	
27	3			63	3	11		99	3	35	
28	4			64	4	10		100	4	36	
29	5			65	5	11		101	5	37	
30	6			66	6	12		102	6	34	
31	7			67	7	12		103	7	33	
32	8			68	8	13		104	8	33	
33	9			69	9	12		105	SPLICE 9	32	[illegible]
34	10			70	10	12		106	10		Changed
35	11			71	11	13		107	11		Hammer
36	12			72	12	14		108	12		

NOTE: CIRCLED NUMBERS INDICATE ENERGY GAUGE READINGS AND LAST READING SHOWN IS GAUGE AT TIME OF COMPLETION

[illegible] MARINE CONSTRUCTION CO., INC.

NJDOT Route 31 & 78 Sections 3N & 3CX
PIPE PILES (18" & 24")

PILE DRIVING RECORD

LOCATION: 39 N
PILE #: 19
DIAMETER: 18"___ 24" X
TIP: POINT___ PLATE X
TYPE: PRODUCTION___ TEST X
TYPE: #1:___ #2___ #3___ #1R___ #2R___ #3R___
WALL THICKNESS: 3/8"___ 1/2" X
POSITION: BAT___ PLUM X
PILE HAMMER: CONMACO
CRANE: 888
PDA MONITORED? Y X N___
DEPTH STARTED: 105
DEPTH AT COMPLETION: 125

ORIGINAL GROUND ELEV: 10±
ESTIMATED TIP ELEV:
ACTUAL TIP ELEV: 115.12
CUT OFF ELEV: −5
REFERENCE ELEV: 9.38

TOTAL PILE LENGTH (A): 135
LENGTH ABOVE CUT-OFF (B): 20.38
LENGTH BELOW CUT-OFF (C): 114.62
REMARKS: 3 PIECES

DATE: 7/21/97
TIME STARTED: 3:00
TIME STOPPED:
FOREMAN: SS
INSPECTOR: BILL

NOTE: A - B = C

FEET PEN.	INCHES	BLOWS	BLOWS /FOOT	FEET PEN.	INCHES	BLOWS	BLOWS /FOOT	FEET PEN.	INCHES	BLOWS	BLOWS /FOOT
1				37				73			
2				38				74			
3				39				75			
4				40				76			
5				41				77			
6				42				78			
7				43				79			
8	CONT.			44				80			
9		12		45				81			
10		13		46				82			
11		13		47				83			
12		13		48				84			
13		14		49				85			
14		13		50				86			
15		14		51				87			
16		13		52				88			
17		13		53				89			
18		13		54				90			
19		13		55				91			
20		12		56				92			
21		13		57				93			
22		12		58				94			
23		13		59				95			
24		13		60				96			
25		13	STARTED	61				97			
26			BY D.O.T	62				98			
27			[illegible]	63				99			
28			[illegible]	64				100			
29				65				101			
30				66				102			
31				67				103			
32				68				104			
33				69				105			
34				70				106		12	
35				71				107		13	
36				72				108		12	

NOTE: CIRCLED NUMBERS INDICATE DIFFERENT GAUGE READINGS AND LAST READING SHOWN IS SAME AT TIME OF COMPLETION

GEORGE HARMS CONSTRUCTION CO., INC.

NJDOT Route 21 & 78 Section 2H & 6CX
PIPE PILES (18" & 24")

PILE DRIVING RECORD

LOCATION: 39 N
PILE #: 18
DIAMETER: 18"___ 24" X
TIP: POINT___ PLATE X
TYPE: PRODUCTION___ TEST X
TYPE: ___
WALL THICKNESS: 3/8"___ 1/2" X
POSITION: BAT___ PLUMB X
PILE HAMMER: CONMACO
CRANE: 999
PDA MONITORED? Y X N___
DEPTH STARTED: 128
DEPTH AT COMPLETION: 136'9"

ORIGINAL GROUND ELEV: 10 ±
ESTIMATED TIP ELEV:
ACTUAL TIP ELEV: -177.0
CUT OFF ELEV: -.5
REFERENCE ELEV: 7.75

TOTAL PILE LENGTH: 200'
LENGTH ABOVE CUT-OFF: 23.5
LENGTH BELOW CUT-OFF: 176.5
REMARKS: 3 SPLICES

DATE: 7/23/97
TIME STARTED: 12:05
TIME STOPPED:
FOREMAN: J. SNYDER
INSPECTOR: SIK

NOTE: A + B = C

FEET PEN.	INCHES	BLOWS	BLOWS /FOOT	FEET PEN.	INCHES	BLOWS	BLOWS /FOOT	FEET PEN.	INCHES	BLOWS	BLOWS /FOOT
1	4			37		16		73		21	
2	3			38		15		74		21	
3	3			39		15		75		22	
4	4			40		16		76		22	
5	4			41		15		77		21	
6	3			42		15		78		20	
7	3			43		16		79		20	
8	4			44		16		80		19	
9	4			45		16	STOP/ SPLICE	81		19	
10	4			46	1130	13		82		21	
11	4			47		13		83		28	
12	5			48		15		84		31	
13	5			49		16		85			
14	5			50		16	35 BPM	86			
15	6			51		16		87			
16	6			52		17		88			
17	6			53		16		89			
18	6			54		18		90			
19	7			55		20		91			
20	7 STOP 136'9"			56		18		92			
21				57		19		93			
22				58		18		94			
23				59		17		95			
24				60		16		96			
18				61		13		97			
26		13		62		15		98			
27		14		63		15		99			
28		14		64		15		100			
29		14		65		15	[illegible]	101			
30		14		66	300	13		102			
31		14		67		16		103			
32		15		68		16		104			
33		16		69		17		105			
34		16		70		17		106			
35		16		71		19		107			
36		15		72		21		108			

NOTE: CIRCLED NUMBERS INDICATE ENERGY GAUGE READINGS AND LAST READING SHOWN IS SAME AT TIME OF COMPLETION

Pile: P18N (125'-185.67' PENTR)   Proj: NJDT RT 21/78   Pg2
Info: CONN 5300 24"ODX0.5"

BL#	depth ft	CSX ksi	CSI ksi	RX8 kips	FMX kips	EMX kips-ft	BPM bl/min	RAU kips	RX7 kips	RA2 kips	BLC bl/ft
1093		26.82	29.58	1047	990	90.1	37.0	470	1077	988	80
1094		27.30	30.20	1077	1008	91.7	37.4	505	1103	1057	80
1095		26.93	29.55	1069	994	89.9	37.3	506	1096	1047	80
1096		27.03	30.07	1067	998	90.8	37.3	484	1097	991	80
1097		27.14	29.93	1071	1002	90.9	37.3	489	1101	953	80
1098		27.09	29.74	1065	1000	90.6	37.2	491	1096	956	80
1099		26.93	29.36	1083	994	87.8	37.2	538	1112	1035	80
1100		26.98	29.85	1072	996	87.9	37.0	529	1101	1037	80
1101		27.52	30.64	1068	1016	92.6	37.3	476	1035	1000	80
1102		27.17	30.20	1065	1003	90.4	37.2	485	1093	991	80
1103	185.67	27.14	30.18	1091	1002	88.9	37.2	546	1118	1073	80

STOP: 15:51:57

```
DRIVE TIME SUMMARY (23-Jul-97 : P18N.Q07)              DRIVE          WAIT
-----------------------------------------              ----- minutes  ----
BN     2 ->  302, START 12:12:47 -> 12:22:07 STOP,      9.33
                                                                      89.65
BN   303 ->  625, START 13:51:46 -> 14:01:03 STOP,      9.28
                                                                      97.58
BN   627 -> 1103, START 15:36:38 -> 15:51:57 STOP,     13.32
                                             ------------------------------
  Total Elapsed time 219.17 minutes        Total Time  31.93 minutes 187.23
```

Goble Rausche Likins & Associates, Inc. 23-Jul 97

NJDT RT 21/78, PIER 39 NORTH
Pile: P18 @ 185.67'   Blow: 471   Data: CONM 5300 24"OD X 0.500"
Collected: 23-Jul-97 Operator: WONDEM TEFERRA   CAPWAP(R) Ver. 1996-2

CAPWAP FINAL RESULTS

Total CAPWAP Capacity: 1182.1; along Shaft 153.3; at Toe 1028.8 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile at Ru kips	Sum of Ru kips	Unit Resist. w. Respect to Depth kips/ft	Unit Resist. w. Respect to Area kips/f2	Smith Damping Factor s/ft	Quake inch
				1182.1					
1	16.6	6.3	10.3	1171.8	10.3	1.55	.25	.127	.150
2	23.3	12.9	10.7	1161.1	21.0	1.61	.26	.127	.150
3	29.9	19.6	9.7	1151.4	30.7	1.46	.23	.127	.150
4	36.5	26.2	3.7	1147.7	34.4	.56	.09	.127	.150
5	43.2	32.9	2.1	1145.6	36.5	.31	.05	.127	.150
6	49.8	39.5	2.2	1143.4	38.7	.33	.05	.127	.150
7	56.5	46.2	2.3	1141.1	41.0	.34	.05	.127	.150
8	63.1	52.8	2.4	1138.7	43.4	.36	.06	.127	.150
9	69.8	59.4	5.6	1133.2	48.9	.84	.13	.127	.150
10	76.4	66.1	11.8	1121.4	60.7	1.77	.28	.127	.150
11	83.1	72.7	12.9	1108.4	73.7	1.95	.31	.127	.150
12	89.7	79.4	10.3	1098.1	84.0	1.55	.25	.127	.150
13	96.3	86.0	6.6	1091.5	90.6	1.00	.16	.127	.150
14	103.0	92.7	3.6	1087.9	94.2	.54	.09	.127	.150
15	109.6	99.3	2.0	1086.0	96.2	.30	.05	.127	.150
16	116.3	106.0	2.3	1083.7	98.4	.34	.05	.127	.150
17	122.9	112.6	4.1	1079.6	102.5	.62	.10	.127	.150
18	129.6	119.2	6.3	1073.3	108.9	.95	.15	.127	.150
19	136.2	125.9	7.5	1065.8	116.4	1.13	.18	.127	.150
20	142.8	132.5	6.8	1058.9	123.2	1.03	.16	.127	.150
21	149.5	139.2	2.9	1056.0	126.1	.44	.07	.127	.150
22	156.1	145.8	2.3	1053.7	128.4	.34	.05	.127	.150
23	162.8	152.5	2.2	1051.5	130.6	.33	.05	.127	.150
24	169.4	159.1	2.1	1049.4	132.7	.31	.05	.127	.150

Goble Rausche Likins & Associates, Inc. 23-Jul 97

NJDT RT 21/78, PIER 39 NORTH
Pile: P18 @ 185.67' Blow: 471 Data: COMM 5300 24"OD X 0.500"
Collected: 23-Jul-97 Operator: WONDEM TEFERRA CAPWAP(R) Ver. 1996-2

CAPWAP FINAL RESULTS

Total CAPWAP Capacity: 1182.1; along Shaft 153.3; at Toe 1028.8 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile at Ru kips	Sum of Ru kips	Unit Resist. w. Respect to Depth kips/ft	Unit Resist. w. Respect to Area kips/ft2	Smith Damping Factor s/ft	Quake inch
25	176.1	165.7	1.9	1047.6	134.5	.28	.04	.127	.150
26	182.7	172.4	3.1	1044.4	137.7	.47	.07	.127	.150
27	189.4	179.0	5.2	1039.2	142.9	.78	.12	.127	.150
28	196.0	185.7	10.4	1028.8	153.3	1.57	.25	.127	.150
Average Skin Values			5.5			.83	.13	.127	.150
Toe			1028.8				327.65	.070	.330

Soil Model Parameters/Extensions		Skin	Toe
Case Damping Factor		.295	1.092
Unloading Quake	(% of loading quake)	1	6
Unloading Level	(% of Ru)	21	
Resistance Gap (included in Toe Quake)	(inch)		.080
Soil Plug Weight	(kips)		.20

Goble Rausche Likins & Associates, Inc. [illegible]-Jul 97

NJDOT RT 21/78, PIER 39 NORTH
Pile: P28 @ 185.67' Blow: 471 Data: CONN 5300 24"OD X 0.500"
Collected: 23-Jul-97 Operator: NOKDEM THYKRRA CAPWAP(R) Ver. 1996-2

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress kips/in2	max. Tension Stress kips/in2	max. Trnsfd. Energy kips-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	998.9	.0	27.070	.000	92.20	14.8	1.5[illegible]0
5	16.6	1024.2	.0	27.756	.000	88.28	14.4	1.434
10	33.2	952.6	.0	25.815	.000	77.84	14.2	1.377
16	53.2	943.0	.0	25.555	.000	75.60	14.0	1.335
22	73.1	952.5	.0	25.814	.000	72.77	13.4	1.282
28	93.0	885.2	.0	23.990	.000	64.41	13.0	1.212
34	112.9	866.3	.0	23.476	.000	60.11	12.8	1.114
40	132.9	856.2	.0	23.203	.000	54.72	12.4	.987
46	152.8	866.1	-5.2	23.473	-.141	47.25	12.3	.830
52	172.7	1011.0	-43.9	27.399	-1.190	38.80	11.9	.652
58	192.7	1169.3	-43.0	31.689	-1.166	33.47	10.2	.538
59	196.0	1197.6	-42.5	32.456	-1.151	34.10	9.1	.524
Absolute	196.0			32.456		(T=	36.0 ms)	
	166.1				-1.232	(T=	74.3 ms)	

CASE METHOD

	J=0.0	J=0.1	J=0.2	J=0.3	J=0.4	J=0.5	J=0.6	J=0.7	J=0.8	J=0.9
RS1	1224.	1149.	1074.	999.	924.	849.	775.	700.	625.	550.
RMX	1310.	1271.	1233.	1197.	1162.	1131.	1100.	1070.	104[illegible].	1014.
RSU	1274.	1205.	1135.	1065.	995.	925.	855.	786.	716.	646.

RAU 455. RA2 859.

Current CAPWAP Ru= 1182.1; Corresponding J(Rs)= .06; J(Rx)= .34

VMX	VFN	VT1*Z	FT1	FMX	DMX	DFN	EMX	EFN	RLT	REN
14.87	-3.39	979.5	993.2	994.6	1.628	.898	93.3	77.3	1264.	4580.

Goble Rausche Likins & Associates, Inc. 23-Jul 97

NJDT RT 21/78, PIER 39 NORTH
Pile: P18 @ 185.67' Blow: 471 Data: CONM 5300 24"OD X 0.500"
Collected: 23-Jul-97 Operator: WONDEM TEFERRA CAPWAP(R) Ver. 1996-2

PILE PROFILE AND PILE MODEL

Depth ft	Area in2	E-Modulus kips/in2	Spec. Weight kips/ft3	Circumf. ft
.00	36.90	30000.0	.492	6.280
196.00	36.90	30000.0	.492	6.280

Toe Area 3.140 ft2

Segmnt Number	Dist. B.G. ft	Impedance kips/ft/s	Imped. Change %	Tension Slack inch	Tension Eff.	Compression Slack inch	Compression Eff.	Circ. ft
1	3.32	65.85	.00	.000	.000	.000	.000	6.280
41	136.20	65.85	.00	.000	.000	.000	.000	6.280
59	196.00	65.85	.00	.000	.000	.000	.000	6.280

Pile Damping 1.0 %, Time Incr .198 ms, Wave Speed 16810.7 ft/s

ARORA and ASSOCIATES, P.C.
Consulting Engineers

SHEET NO. 2 OF ____

JOB: Rt 21 VIADUCT | PREPARED BY: YSC | DATE: 5/6/95

JOB NO. 1237-085 | CHECKED BY: UK revised | DATE: 6/17/97

SUBJECT: PILE CAPACITY CALCULATION @ S-501 – 2' DIAM CONC. FILLED STEEL PIPE PILE

REF: NAVFAC DM-7.2 MAY 1982 ...... PAGE 7.2-193 TO 200

PIPE DESIGNATION: PP24 WALL THICKNESS = 0.438"

ULTIMATE LOAD CAPACITY IN COMPRESSION

$$Q_{ult} = P_T N_q A_T + \sum_{H=H_0}^{H=H_0+D} (K_{HC})(P_0)(\tan\delta)(S) \text{ ---- (FOR HOMOGENEOUS GRANULAR SOILS)}$$

(A) FRICTION STRENGTH COMPONENT

SEG 1 $= (K_{HC})(P_0)(\tan\delta)(S) \times 40$ — $K_{HC} = 1.25$ $P_0 = 2128$

~~$\delta = \frac{3}{4}\phi = 22.5$ $S = 6.28$~~

$= 1.25 \times 2128 \times 0.41 \times 6.28 \times 40$

Limiting skin friction = 2000 psf

$= 273{,}959$ lbs $= 136.98$ TONS

SEG 2 $= C_A \, 2\pi R \, Z$ .... (FOR COHESIVE SOIL P 196)

$475 \times 6.28 \times 38$ — $C_A = 475$

$= 113354$ lbs $= 56.67$ TONS

SEG 3 $= (K_{HC})(P_0)(\tan\delta)(S)$ — $K_{HC} = 1.25$ $P_0 = 3188$

~~$\delta = \frac{3}{4}\phi = 27.0$ $S = 6.28$~~

$= 1.25 \times 3188 \times 0.51 \times 6.28 \times 10$

$= (\ 2{,}000\ ) \times 6.28 \times 10$ — Limiting Skin friction = 2000 psf

$= 125{,}600$ Lbs. $= 62.80$ TONS

(B) END BEARING COMPONENT

$P_T N_q A_T$ .... (FOR GRANULAR SOILS) $P_T = 3188$ $A_T = 3.14$

$= 3188 \times 65 \times 3.14$ — $N_q = 65$

$= 650{,}671$ lbs $= 325.34$ TONS.

$\therefore Q_{ULT} = 136.98 + 56.67 + 62.80 + 325.34$ TONS

$= 581.79$ TONS

NOTE: The F.S. should be reduced to 2.25 with the use of a dynamic pile driving analyzer. Using $Q_{All} = 220$ Tons, the target $Q_{ult} = 500$ Tons

F.S. = 2.5

$Q_{ALL} = 581.79 / 2.5 = 232.7$ TONS

Recommended Capacity = 220 Tons

ARORA and ASSOCIATES, P.C.
Consulting Engineers

SHEET NO. 1 OF ____

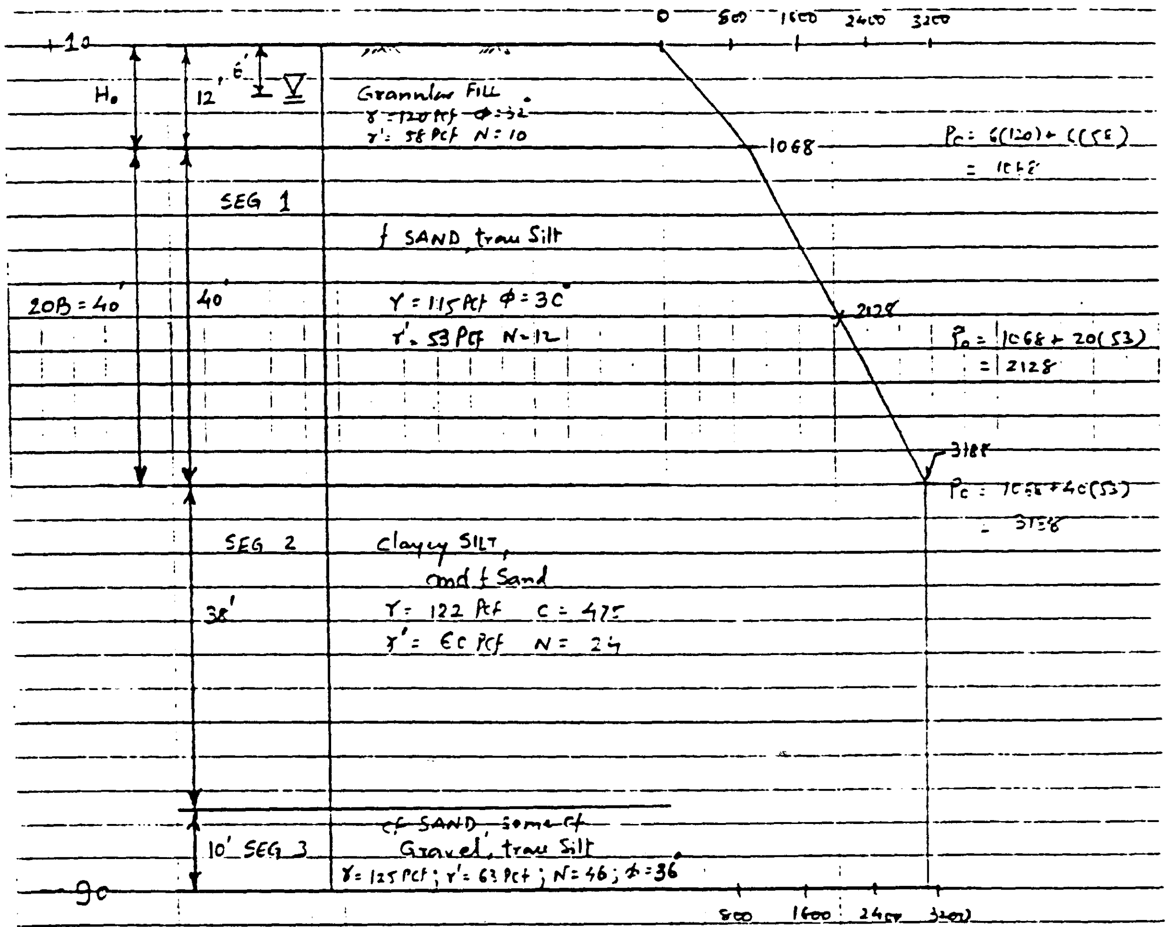
JOB RT 21 VIADUCT PREPARED BY YSC DATE 5/6/95
JOB NO. 1237 - 085 CHECKED BY UK revised DATE 6/17/97
SUBJECT PILE CAPACITY CALCULATION @ S-501 – 2' DIAM CONC. FILLED STEEL PIPE PILE

REF: NAVFAC DM-7.2, MAY 1982 ...... PAGE 7.2-193 TO 200

BORING INFO USED ... S-501

THE FILL LAYER IS NOT CONSIDERED FOR FRICTIONAL RESISTANCE

ASSUME PRESSURE BECOMES CONSTANT AT A DEPTH OF 20 B



FOR 2.0' DIAM. ; 20 B = 40'

TOTAL LENGTH OF PILE = 100' = D

REFERENCES

1. A. S. Alawneh, "Tension Piles in Sand - A Method including Degradation of Shaft Friction during Pile Driving", Transportation Research Record No.1663, pp. 41-49, Sept. 1999.

2. American Association of State Highway and Transportation Officials (AASHTO), "Standard Specifications for Highway Bridges", Sixteenth Edition, 1996.

3. Arora and Associates, P.C., "Geotechnical Engineering Foundation Recommendation Report (Revised)" and Text with Appendices A, B, C, D & E, NJ Route 21, Section 2N, Newark Viaduct Replacement, City of Newark, Essex County, NJ, July 1997.

4. Arora and Associates, P.C., "Geotechnical Engineering Foundation Recommendation Report, Addendum for Contract A (Revised)", NJ Route 21, Section 2N, Newark Viaduct Replacement, City of Newark, Essex County, NJ, July 1998.

5. Arora and Associates, P.C., "Testing Boring Logs for Construction Contract A", NJ Route 21, Section 2N, Newark Viaduct Replacement, City of Newark, Essex County, NJ, July 1998.

6. W. J. Attwooll, D. M. Holloway, K. M. Rollins, M. I. Esrig, S. Sakhai, and D. Hemenway, "Measured Pile Setup during Load Testing and Production Piling", Transportation Research Record No. 1663, pp. 1-7, Sept. 1999.

7. A. S. Azzouz and M. J. Morrison, "Field Measurements on Model Pile in Two Clay Deposits", ASCE Journal of Geotechnical Engineering, Vol. 114, No. 1, pp. 104-121, Jan. 1988.

8. M. Baker Jr. Inc., "Engineering Feasibility Study, NJ Route 21, Section 2N", December 1990.

9. M. Baker Jr., Inc., "Northeast Corridor Crossing Design Appraisal Statement Report", Sept. 1994.

10. J. A. Beavers and C. L. Durr, "Corrosion of Steel Piling in Nonmarine Applications", Transportation Research Board, Report 408, 1998.

11. W. M. Camp III and H. S. Parmar, "Characterization of Pile Capacity with Time in the Cooper Mark-Study of Applicability of a Past Approach to Predict Long-term Pile Capacity", Transportation Research Record No. 1663, pp. 16-24, Sept. 1999.

12. F. C. Chow, R. J. Jardin, F. Bruncy and J. F. Nauroy,. "Effects of Time on Capacity of Pipe Piles in Dense Marine Sand", ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 124, No. 3, March 1998.

13. M. R. Coop and C. P. Worth, "Field Studies of an Instrumented Model Pile in Clay", Geotechnique 39, No. 4, pp. 676-679, 1989.

14. S. Drabkin, H. Lacy and D. S. Kim, "Estimating Settlement of Sand Caused by Construction Vibration", ASCE Journal of Geotechnical Engineering, Vol. 122, No. 11, Nov. 1996.

15. J. M. Duncan, L. T. Evans and P. S. K. Ooi, "Lateral Load Analysis of Single Piles and Drilled Shafts", ASCE Journal of Geotechnical Engineering, Vol. 120, No. 5, June 1994.

16. Federal Highway Administration, "Design and Construction of Driven Pile Foundations",Vol. 1 & II (Revised), Nov. 1998.

17. B. H. Fellenius, "Analysis of Load Tests on Piles Driven through Calcareous Desert Sands", Discussion, ASCE, Journal of Geotechnical and Geoenvironmental Engineering, Feb. 2001.

18. B. H. Fellenious, R. E. Riker, A. J. O'Brien and G. R. Tracy, "Dynamic and Static Testing in Soil Exhibiting Set-up", ASCE Journal of Geotechnical Engineering, Vol. 115, No. 7, July 1989.

19. C. E. Fear and E. C. McRoberts, "Reconsideration of Initiation of Liquefaction in Sandy Soils", ASCE Journal of Geotechnical Engineering, Vol. 121, No. 3, March 1995.

20. J. A. Focht Jr., "Lessons Learned from Missed Predictions", ASCE Journal of Geotechnical Engineering, Vol. 120, No. 10, October 1994.

21. A. M. Ghaly, "Setup and Relaxation in Glacial Sand", Discussion, ASCE Journal of Geotechnical Engineering, April 1996.

22. G. Goble, "Dynamic Pile Monitoring - A Continuing Research Effort", Transportation Record News, No. 207, March - April 2000.

23. T. H. Hanna and R. H. S. Tan, "The Behavior of Long Piles Under Compressive Loads in Sand", Canadian Geotechnical Journal, Vol. 10, No. 3, 1973.

24. N. F. Ismael, "Analysis of Load Tests on Piles Driven through Calcareous Desert Sands", ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 125, No. 10, Oct. 1999.

25. N. F. Ismael and T. W. Klym, "Pore-Water Pressure Induced by Pile Driving", Technical Notes, ASCE Journal of Geotechnical Engineering, Nov. 1979.

26. R. J. Jardine, R. F. Overy and F. C. Chow", Axial Capacity of Offshore Piles in Dense North Sea Sands", ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 124, No. 2, Feb. 1998.

27. R. Y. Kiang and A. I. Huesin, "Simplified Dynamic Method for Pile-Driving Control", ASCE Journal of Geotechnical Engineering, Vol. 119, No. 4, April 1993.

28. R. Y. Liang and J. Zhou, "Probability Method Applied to Dynamic Pile-Driving Control", ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 123, No. 2, Feb. 1997.

29. J. H. Long, D. Bozkurt, J. A. Kerrigan and M. H. Wysockey, "Value of Methods of Predicting Axial Pile Capacity", Transportation Research Record No. 1663, pp. 57-63, 1999.

30. J. H. Long, J. A. Kerrigan and M. H. Wysockey, "Measured Time Effects for Axial Capacity of Driven Piling", Transportation Research Record No. 1663, pp. 8-15, 1999.

31. J. K. Mitchell and Z. V. Solymar, "Time-Dependent Strength Gain in Freshly Deposited or Densified Sand", ASCE Journal of Geotechnical Engineering, Vol. 110, No. 11, Nov. 1984.

32. B. Moller and U. Bergdahl, "Dynamic Pore Pressure during Pile Driving in Fine Sand", 10th International Conference on Soil Mechanics and Foundation Engineering, pp. 791-794, 1981.

33. Naval Facilities Engineering Command, Department of Navy, "Foundations and Earth Structures, Design Manual 7.2 and Soil Mechanics, Design Manual 7.1", 1982.

34. New Jersey Department of Transportation, Bureau of Geotechnical Engineering, "Procedure for Consultants", 1980.

35. New Jersey Department of Transportation, "Standard Specifications for Road and Bridge Construction", 1996.

36. S. G. Paikowski, V. A. LaBelle, and N. M. Hourani, "Dynamic Analysis and Time-Dependent Pile Capacity", 5th International Conference on the Application of Stress-Wave Theory to Piles", Orlando, Florida, 1996.

37. Parsons Brinckerhoff, Quade and Douglas, Inc., "Report on Pile Foundation Testing - Contract A Route 21 - Section 2N, Newark Viaduct Replacement", Jan. 2000.

38. H. G. Poulos and E. H. Davis "Pile Foundation Analysis and Design", published by John Wiley and Sons, 1980.

39. M. F. Randolph, J. P. Carter and C. P. Worth, "Driven Piles in Clay — The Effects of Installation and Subsequent Consolidation", Geotechnique 29, No. 4, pp. 361-393, 1979.

40. M. F. Randolph, J. Dolwin and R. Beck, "Design of Driven Piles in Sand" Geo-, technique 44, No. 3, pp. 427-448, 1994.

41. F. Rausche, G. G. Goble and G. E. Likins, "Dynamic Determination of Pile Capacity", ASCE Journal of Geotechnical Engineering, Vol. III, No. 3, March 1985.

42. M. Roy, R. Blanchet, F. Tavenas and P. L. Rochelle, "Behavior of a Sensitive Clay during Pile Driving", Canadian Geotechnical Journal, Vol. 18, pp. 67-85, 1981.

43. C. D. Thompson and D. E. Thompson, "Real and Apparent Relaxation of Driven Piles", ASCE Journal of Geotechnical Engineering, Vol. III, No. 2, Feb. 1985.

44. H. H. Titi and G. W. Wathugala, "Numerical Procedure for Predicting Pile Capacity - Setup/Freeze", Transportation Research Record 1663, pp. 25-32, 1999.

45. L. Samson and J. Authier, "Change in Pile Capacity with Time: Case Histories", Canadian Geotechnical Journal, Vol. 23, pp. 174-180, 1986.

46. H. B. Seed, "Design Problems in Soil Liquefaction", ASCE Journal of Geotechnical Engineering, Vol. 113, No. 8, August 1987.

47. J. H. Schmertmann, "The Mechanical Aging of Soils", ASCE Journal of Geotechnical Engineering, Vol. 117, No. 9, Sept. 1991.

48. M. R. Skivin, C. M. Morgano and M. Morvant, "Pile Capacity as a Function of Time in Clayey and Sandy Soils", Proc. 5th International Conference and Exhibition on Piling and Deep Foundations", Belgium 1994.

49. R. Skov and H. Denver, "Time Dependence of Bearing Capacity of Piles", Proc. 3rd International Conference on the Application of Stress-Wave Theory, Ottawa, Ontario, Canada, pp. 879-888, 1988.

50. L. O. Soderberg, "Consolidation Theory applied to Foundation Pile Time Effects", Geotechnique, Vol. 12, No. 3, pp. 217-285, 1962.

51. G. F. Sowers, "Human Factors in Civil and Geotechnical Engineering Failures", ASCE Journal of Geotechnical Engineering, Vol. 119, No. 2, Feb. 1993.

52. M. R. Svinkin, "Setup and Relaxation in Glacial Sand", Discussion, ASCE Journal of Geotechnical Engineering, April, 1996.

53. M. J. Wu, T. M. Gurtowski and G. Yaman, "Field Evaluation of Diesel Hammer Performance", Proceeding 11th International Conference of Soil Mechanics and Foundation Engineering, San Francisco, 1985, pp. 1507-1510.

54. A. J. Whittle and T. Sutabutr, "Prediction of Pile Setup in Clay", Transportation Research Record No. 1663, pp. 33-40, 1999.

55. S. D. Werner, "Seismic Guidelines for Ports", ASCE, Technical Council on Lifeline Earthquake Engineering Monograph No. 12, March 1998.

56. D. L. York, W. G. Brusey, F. M. Clemente and S. K. Law, "Setup and Relaxation in Glacial Sand", ASCE Journal of Geotechnical Engineering, Vol. 120, No. 9, Sept. 1994.

57. D. L. York, W. G. Brusey, F. M. Clemente and S. K. Law, "Setup and Relaxation in Glacial Sand", Closure, ASCE Journal of Geotechnical Engineering, April 1996.

58. T. L. Youd and I. M. Idriss, "Proceedings of the NCEER (National Center for Earthquake Engineering Research) Workshop on Evaluation of Liquefaction Resistance of Soils", Technical Report NCEER-97-0022, December 1997.

59. W. H. Zimpfer, F. C. Rogers, D. R. Lueder and G. H. Obear, "Engineering Soil Survey of New Jersey", prepared by Rutgers University as joint highway research project, Report No. 2, Essex County, 1951.