Design and Construction of Driven Pile Foundations—Lessons Learned on the Central Artery/Tunnel Project

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Turner-Fairbank Highway Research Center
6300 Georgetown Pike
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FOREWORD

The purpose of this report is to document the issues related to the design and construction of driven pile foundations at the Central Artery/Tunnel project. Construction issues that are presented include pile heave and the heave of an adjacent building during pile driving. Mitigation measures, including the installation of wick drains and the use of preaugering, proved to be ineffective. The results of 15 dynamic and static load tests are also presented and suggest that the piles have more capacity than what they were designed for. The information presented in this report will be of interest to geotechnical engineers working with driven pile foundation systems.

Gary L. Henderson
Director, Office of Infrastructure
Research and Development

NOTICE

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16. Abstract
Five contracts from the Central Artery/Tunnel (CA/T) project in Boston, MA, were reviewed to document issues related to design and construction of driven pile foundations. Given the soft and compressible marine clays in the Boston area, driven pile foundations were selected to support specific structures, including retaining walls, abutments, roadway slabs, transition structures, and ramps. This report presents the results of a study to assess the lessons learned from pile driving on the CA/T. This study focused on an evaluation of static and dynamic load test data and a case study of significant movement of an adjacent building during pile driving. The load test results showed that the piles have more capacity than what they were designed for. At the site of significant movement of an adjacent building, installation of wick drains and preaugering to mitigate additional movement proved to be ineffective. Detailed settlement, inclinometer, and piezometer data are presented.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised March 2003)

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CHAPTER 1. INTRODUCTION

Pile foundations are used extensively for the support of buildings, bridges, and other structures to safely transfer structural loads to the ground and to avoid excess settlement or lateral movement. They are very effective in transferring structural loads through weak or compressible soil layers into the more competent soils and rocks below. A “driven pile foundation” is a specific type of pile foundation where structural elements are driven into the ground using a large hammer. They are commonly constructed of timber, precast prestressed concrete (PPC), and steel (H-sections and pipes).

Historically, piles have been used extensively for the support of structures in Boston, MA. This is mostly a result of the need to transfer loads through the loose fill and compressible marine clays that are common in the Boston area. Driven piles, in particular, have been a preferred foundation system because of their relative ease of installation and low cost. They have played an important role in the Central Artery/Tunnel (CA/T) project.

ROLE OF DRIVEN PILE FOUNDATIONS ON THE CA/T PROJECT

The CA/T project is recognized as one of the largest and most complex highway projects in the United States. The project involved the replacement of Boston’s deteriorating six-lane, elevated central artery (Interstate (I) 93) with an underground highway; construction of two new bridges over the Charles River (the Leverett Circle Connector Bridge and the Leonard P. Zakim Bunker Hill Bridge); and the extension of I–90 to Boston’s Logan International Airport and Route 1A. The project has been under construction since late 1991 and is scheduled to be completed in 2005.(1)

Driven pile foundations were used on the CA/T for the support of road and tunnel slabs, bridge abutments, egress ramps, retaining walls, and utilities. Because of the large scale of the project, the construction of the CA/T project was actually bid under 73 separate contracts. Five of these contracts were selected for this study, where a large number of piles were installed, and 15 pile load tests were performed. The locations of the individual contracts are shown in figure 1 and summarized in table 1. A description of the five contracts and associated pile-supported structures is also given below.

1. **Contract C07D1** is located adjacent to Logan Airport in East Boston and included construction of a part of the I–90 Logan Airport Interchange roadway network. New roadways, an egress ramp, retained fill sections, a viaduct structure, and retaining walls were all constructed as part of the contract.(2) Driven piles were used primarily to support the egress ramp superstructure, abutments, roadway slabs, and retaining walls.

2. **Contract C07D2** is located adjacent to Logan Airport in East Boston and included construction of a portion of the I–90 Logan Airport Interchange. Major new structures included highway sections, a viaduct structure, a reinforced concrete open depressed roadway (boat section), and at-grade approach roadways.(2) Driven piles were used to support the boat section, walls and abutments, and portions of the viaduct.

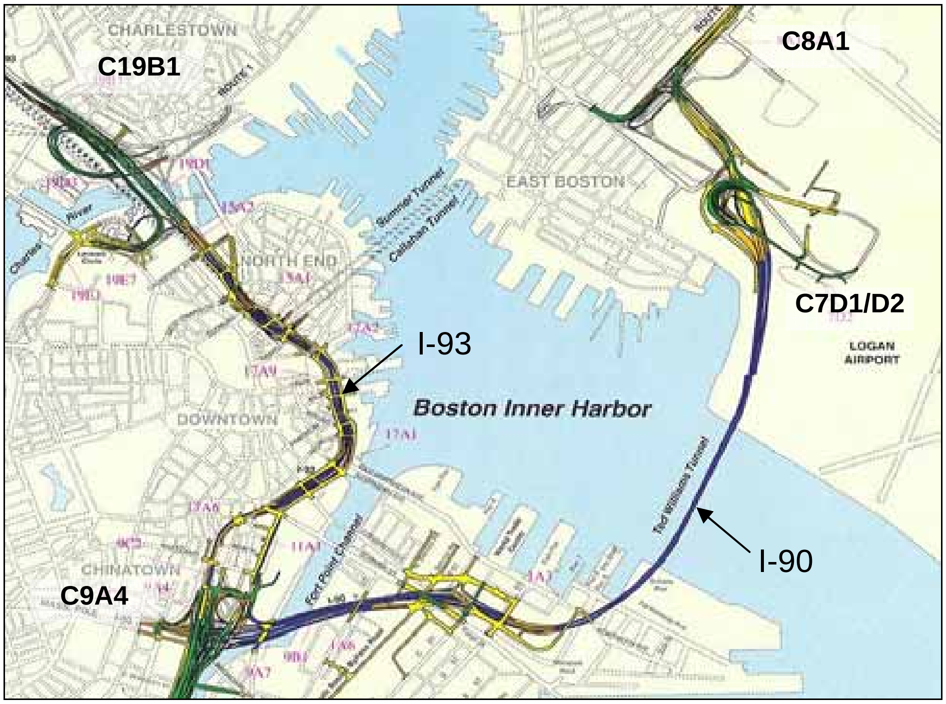


Figure 1. Locations of selected contracts from the CA/T project.[(3)]

Table 1. Summary of selected contracts using driven pile foundations.

Contract	Location	Description
C07D1	Logan Airport	I–90 Logan Airport Interchange
C07D2	Logan Airport	I–90 Logan Airport Interchange
C08A1	Logan Airport	I–90 and Route 1A Interchange
C09A4	Downtown	I–93/I–90 Interchange, I-93 Northbound
C19B1	Charlestown	I–93 Viaducts and Ramps North of the Charles River

3. **Contract C08A1** is located just north of Logan Airport in East Boston and included construction of the I–90 and Route 1A interchange. This contract involved new roadways, retained fill structures, a viaduct, a boat section, and a new subway station.[(2)] Both vertical and inclined piles were used to support retaining walls and abutments.

4. **Contract C09A4** is located just west of the Fort Point Channel in downtown Boston. The contract encompassed construction of the I–90 and I–93 interchange, and the northbound section of I–93. Major new structures included surface roads, boat sections, tunnel sections, viaducts, and a bridge.[(2)] Piles were used to support five approach structures that provide a transition from on-grade roadways to the viaduct sections. Piles were also used to support utility pipelines.

5. **Contract C19B1** is located just north of the Charles River in Charlestown. The contract included the construction of viaduct and ramp structures forming an interchange connecting Route 1, Storrow Drive, and I–93 roadways. Major new structures included roadway transition structures, boat sections, retaining walls, and a stormwater pump station.[(2)] Piles

were used to support the ramp structures that transition from on-grade roadways to the viaduct or boat sections.

OBJECTIVES

The overall objective of this report is to document the lessons learned from the installation of driven piles on the CA/T project. This includes review and analysis of pile design criteria and specifications, pile driving equipment and methods, issues encountered during construction, dynamic and static load test data, and cost data for different pile types and site conditions.

SCOPE

This report consists of six chapters, the first of which presents introductory and background information about the contracts where significant pile driving occurred. The second chapter discusses the criteria and specifications used for pile design and construction on the CA/T project. The third chapter documents the equipment and methods used for pile driving. Major construction issues encountered during driving, such as pile and soil heave, are also discussed. The fourth chapter presents the results of pile load tests performed on test piles using static and dynamic test methods, including a discussion of axial capacity, dynamic soil parameters, and pile driving criteria. The fifth chapter presents the unit costs for pile driving and preaugering for the different pile types used, as identified in the original construction bids. Finally, the sixth chapter summarizes the important findings of this study.

CHAPTER 2. DRIVEN PILE DESIGN CRITERIA AND SPECIFICATIONS

This chapter presents the pile design criteria and specifications used on the CA/T project in contracts C07D1, C07D2, C08A1, C09A4, and C19B1. These include information on the types of piles used, capacity requirements, minimum preaugering depths, and testing requirements. The subsurface conditions on which the design criteria were based are also discussed.

SUBSURFACE CONDITIONS

Representative soil profiles from each of the contract sites are shown in figures 2 through 6 based on the interpretation of geotechnical borings. (See references 4, 5, 6, 7, 8, 9, and 10)

As shown in figures 2 through 5, the conditions encountered at sites in East Boston (C07D1, C07D2, and C08A1) and in downtown Boston (C09A4) are similar. The subsurface conditions at these locations typically consisted of fill overlying layers of organic silt, inorganic sand or silt, marine clay, glacial soils, and bedrock. The subsurface conditions shown in figure 6 for the C19B1 site in Charlestown, however, were different from the other four sites. Organic soils and marine clays were only encountered to a limited extent at the site. Also, the thickness of the fill layer was greater relative to the other sites.

The physical properties and geological origin of the soils encountered at the contract sites are described below.[11-12]

Bedrock: The bedrock in the area consists of argillite from the Cambridge formation. The condition of the bedrock varies considerably with location, even within a given site. Evaluation of rock core samples indicates that the rock is typically in a soft and weathered condition and contains a significant amount of fracturing. However, hard and sound bedrock was found at some locations.

Glacial Soils: The glacial soils were deposited during the last glaciation approximately 12,000 years ago. These deposits include glacial till, and glaciomarine, glaciolacustrine, and glaciofluvial soils. Till is characterized by a mass of unsorted debris that contains angular particles composed of a wide variety of grain sizes, ranging from clay-sized particles to large boulders. Glaciomarine or glaciolacustrine deposits generally consist of clay, silt, and sand, whereas glaciofluvial deposits contain coarser grained sand and gravel. The glacial soils are typically dense in nature as indicated by high standard penetration test (SPT) resistance, and the piles were typically terminated in these deposits.

Marine Soils: Marine soils were deposited over the glacial soils during glacial retreat in a quiescent deepwater environment. The marine clay layer, as shown in figures 2 through 5, is the thickest unit in the profile, but was encountered only to a limited extent at the Charlestown site. The clay is generally overconsolidated in the upper portions of the layer and is characterized by relatively higher strengths. The overconsolidation is a result of past desiccation that occurred during a period of low sea level. By comparison, the deeper portions of the clay layer are much softer and penetration of the SPT split spoon can sometimes occur with just the weight of the drilling rods alone.

Inorganic Soils: Inorganic silts and sands are typically encountered overlying the marine soils. These soils were deposited by alluvial processes.

Organic Soils: The organic soils that are encountered below the fill generally consist of organic silt and may contain layers of peat or fine sand. These soils are the result of former tidal marshes that existed along the coastal areas.

Fill Soils: Fill material was placed in the more recent past to raise the grade for urban development. The fill layer is highly variable in its thickness and composition, ranging from silts and clays to sands and gravels. The consistency or density is also variable as indicated by the SPT blow counts. The variability in the fill is attributed to the characteristics of the particular borrow source material and the methods of placement.

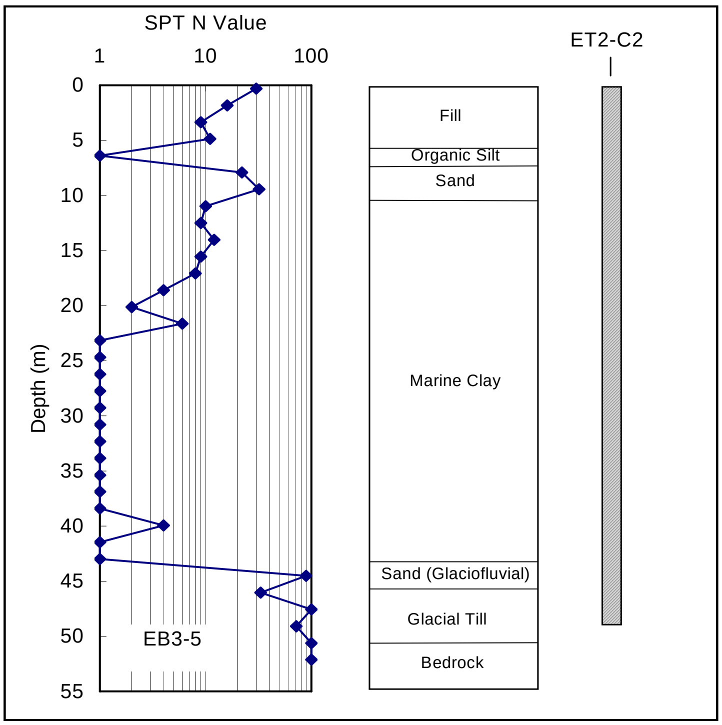


Figure 2. Soil profile at the contract C07D1 site as encountered in Boring EB3-5.

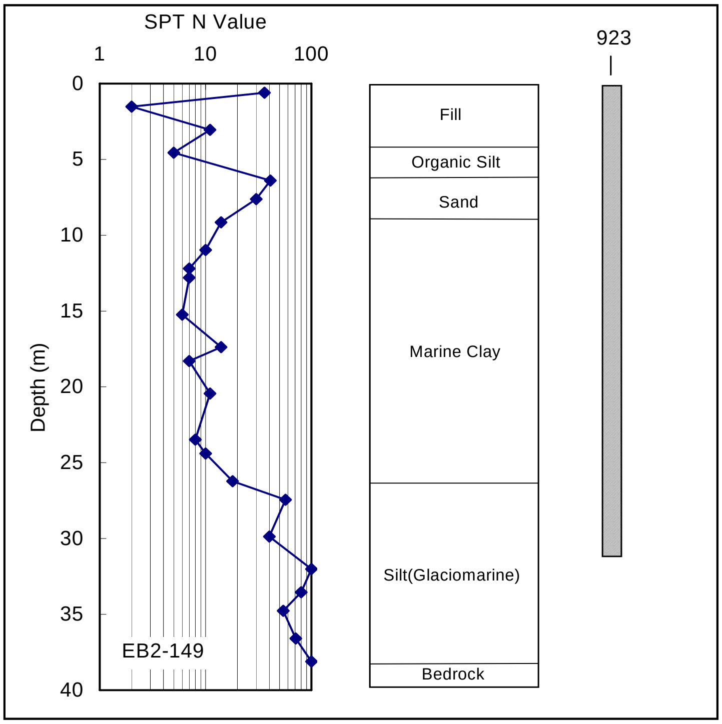


Figure 3. Soil profile at the contract C07D2 site as encountered in Boring EB2-149.

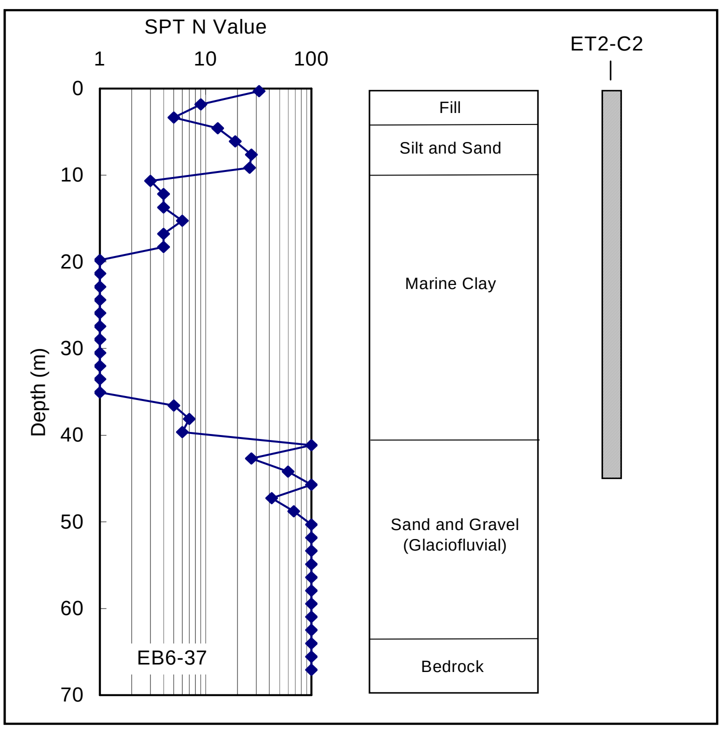


Figure 4. Soil profile at the contract C08A1 site as encountered in Boring EB6-37.

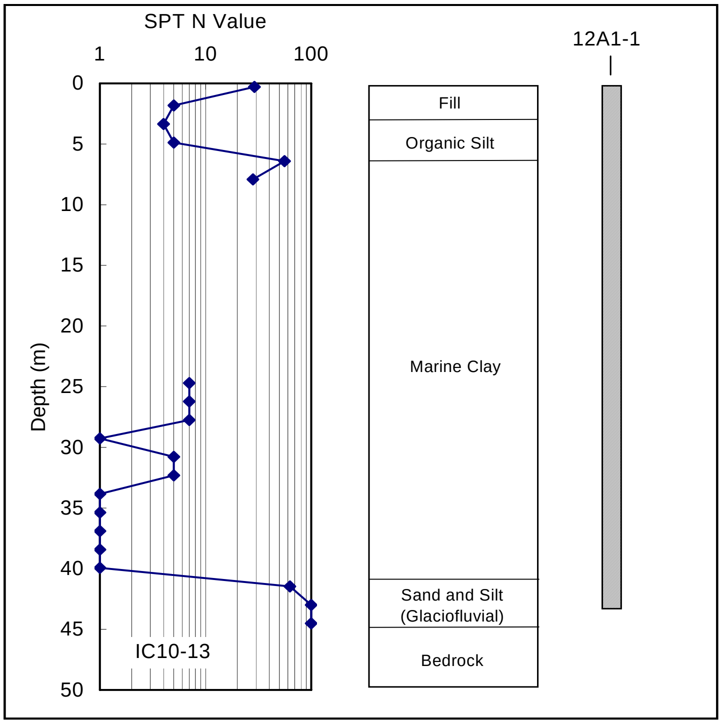


Figure 5. Soil profile at the contract C09A4 site as encountered in Boring IC10-13.

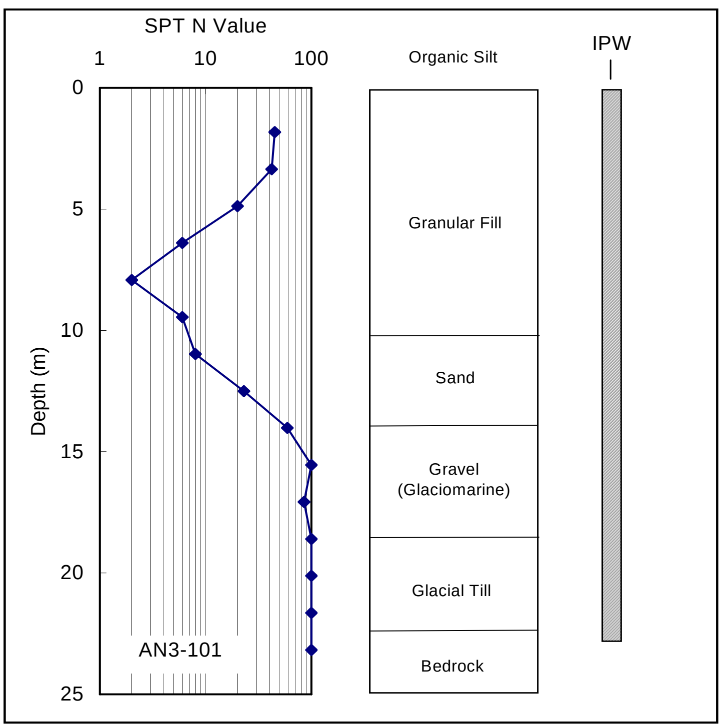


Figure 6. Soil profile at the contract C19B1 site as encountered in Boring AN3-101.

DESIGN CRITERIA AND SPECIFICATIONS

The variable fill and compressible clay soils encountered at depth necessitated the use of deep foundations. Driven piles were selected, and design criteria and specifications were developed for their installation, ultimate capacity, and testing. Because the CA/T project was located in Massachusetts, the design criteria were required to satisfy the regulations given in the Massachusetts State building code.(13) The technical content of the State code is based on the 1993 edition of the Building Officials and Code Administrators (BOCA) national building code.

The specifications that were used for each CA/T contract are contained in two documents of the Massachusetts Highway Department (MHD). The first document includes the general requirements for all CA/T contracts and is entitled *Supplemental Specifications and CA/T Supplemental Specifications to Construction Details of the Standard Specifications for Highways and Bridges (Division II) for Central Artery (I-93)/Tunnel (I-90) Project in the City of Boston*.(14)

The specifications pertaining to individual contracts are covered in a second document concerning special provisions.(15) The special provisions are necessary given the uniqueness of the environmental conditions, soil conditions, and structure types found in each contract. The special provisions present specific details regarding the pile types, pile capacity requirements, and minimum preaugering depths.

Information selected from the specification regarding pile types, preaugering criteria, pile driving criteria, and axial load and test criteria is highlighted below.

Pile Types

Two types of piles were specified on the selected contracts of the CA/T: (1) PPC piles, and (2) concrete-filled steel pipe piles. The PPC piles were fabricated using 34.5- to 41.3-megapascal (MPa) (28-day strength) concrete and were prestressed to 5.2 to 8.3 MPa. The design drawings of typical 30-centimeter (cm)- and 41-cm-diameter square PPC piles are shown in figures 7 and 8, respectively.

To prevent damage to the pile tips during driving in very dense materials, the PPC piles were also fitted with 1.5-meter (m)-long steel H-pile “stingers.” In the 41-cm-diameter PPC piles, an HP14x89 section was used as the stinger. The stingers were welded to a steel plate that was cast into the pile toe, as shown in figure 8. Stingers were used intermittently on the 30-m-diameter PPC piles, consisting of HP10 by 42 sections.

The concrete-filled steel pipe piles were 31 to 61 cm in diameter, with wall thicknesses ranging from 0.95 to 1.3 cm. The piles were driven closed-ended by welding a steel cone or flat plate onto the pile tip prior to driving. Once the pile was driven to the required depth, the pile was filled with concrete.

A summary of the pile types used on the CA/T is given in table 2, along with the estimated quantities driven. The quantities are based on the contractor’s bid quantities that were obtained directly from Bechtel/Parsons Brinckerhoff. As shown in table 2, the 41-cm-diameter PPC piles were the dominant pile type used, accounting for more than 70 percent of the total length of pile driven.

Table 2. Summary of pile types used on the selected CA/T contracts.

Pile Type	Estimated Length of Pile Driven (m)					
	C07D1	C07D2	C08A1	C09A4	C19B1	Total
32-cm pipe	-	-	-	-	5,550	5,550
41-cm pipe	-	-	-	5,578	-	5,578
61-cm pipe	-	-	-	-	296	296
30-cm square PPC	7,969	3,981	792	3,658	2,177	18,577
41-cm square PPC	32,918	19,879	8,406	14,326	6,279	81,808

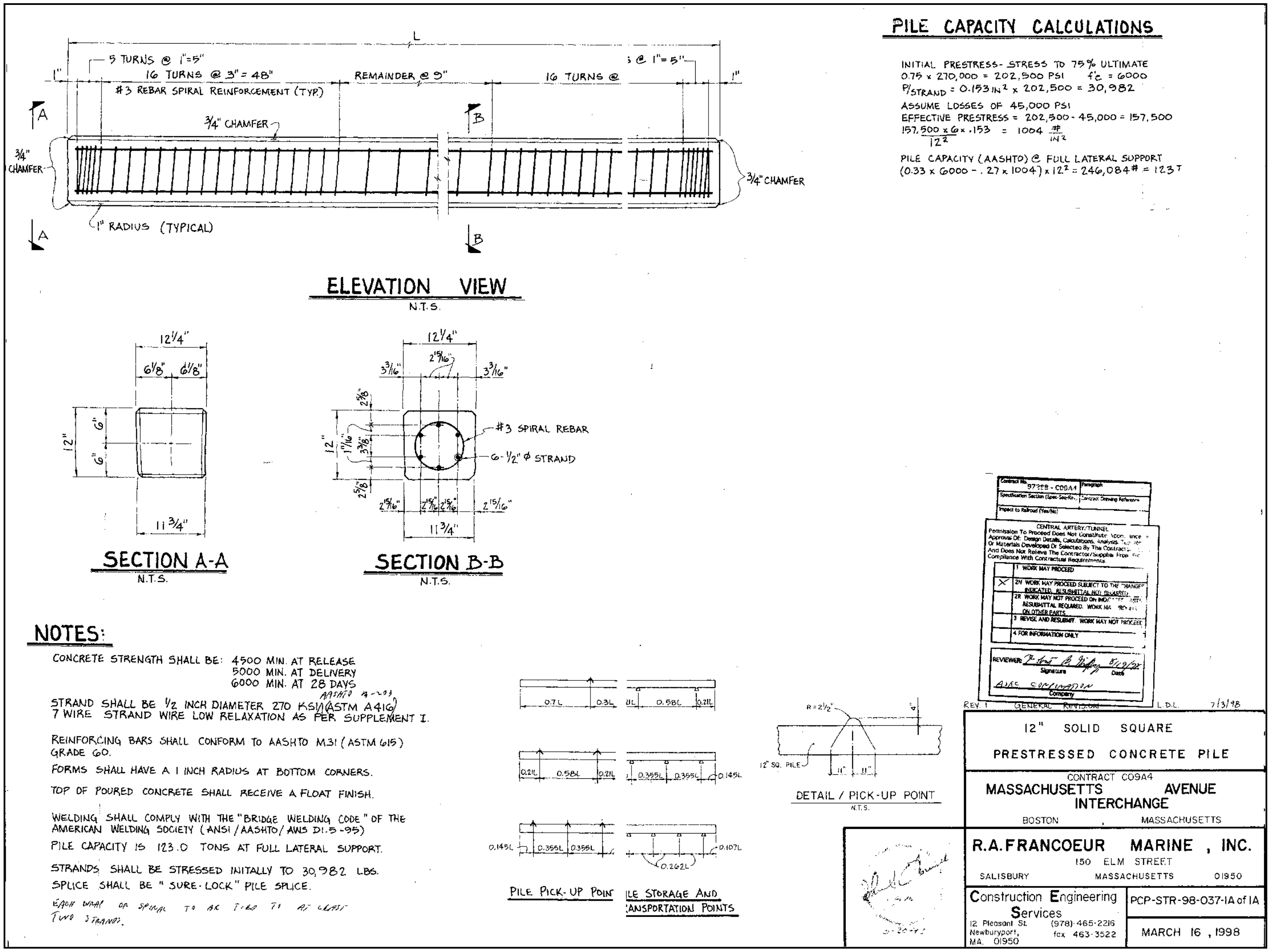
Preaugering Criteria

Preaugering was specified for all piles that were installed in embankments or within the specified limits of adjacent structures. Settlement problems observed at the Hilton hotel (contract C07D1) initiated the use of preaugering to reduce the potential for soil heave caused by pile installation. Soil heave is discussed further in chapter 3. The required depth of preaugering varied depending on the contract and pile location, but ranged from 7.6 to 32.0 m below the ground surface.

Pile Driving Criteria

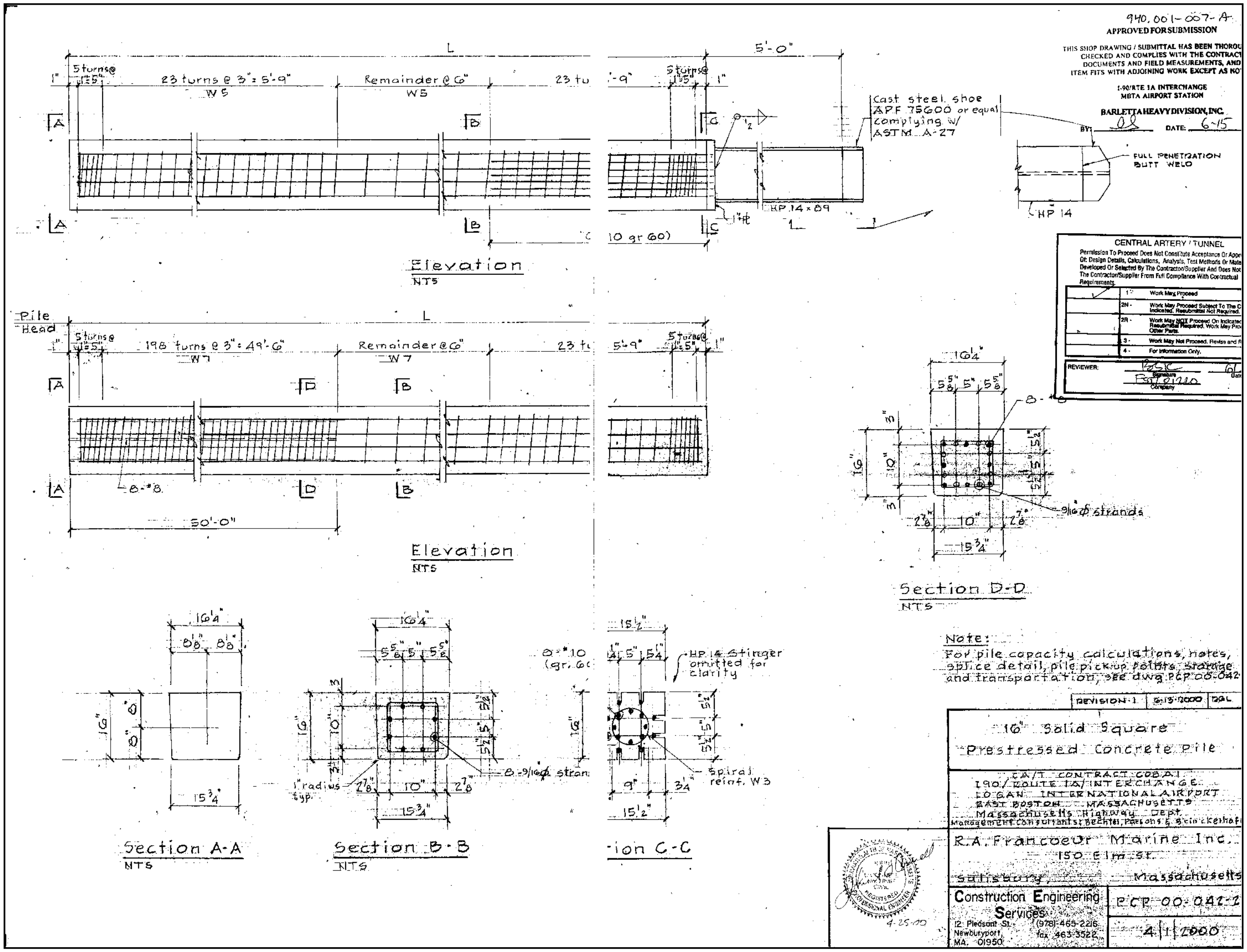
The specifications required that a Wave Equation Analysis of Piles (WEAP) be used to select the pile driving equipment. The WEAP model estimates hammer performance, driving stresses, and driving resistance for an assumed hammer configuration, pile type, and soil profile. The acceptability of the hammer system was based on the successful demonstration that the pile could be driven to the required capacity or tip elevation without damage to the pile, within a penetration resistance of 3 to 15 blows per 2.5 cm.

The pile driving resistance criteria estimated from the WEAP analysis was also used as the initial driving criteria for the installation of the test piles. Additional WEAP analyses were required for changes in the hammer type, pile type or size, or for significant variations in the soil profile. It was also specified that the WEAP analyses be rerun with modifications to the input parameters to match the results obtained from the dynamic or static load test results. Modifications to the driving criteria could be made as appropriate, based on the results of the pile load tests.



1 foot = 0.30 m
1 inch = 25.4 mm

Figure 7. Typical pile details for a 30-cm-diameter PPC pile.



1 foot = 0.30 m
1 inch = 25.4 mm

Figure 8. Typical pile details for a 41-cm-diameter PPC pile with stinger.

Axial Load and Pile Load Test Criteria

The required allowable axial capacities that were identified in the special provisions are summarized in table 3. Allowable axial load capacities ranged from 311 to 1,583 kilonewtons (kN). Lateral load criteria were not identified in the selected contracts.

Table 3. Summary of pile types and axial capacity (requirements identified in the selected contracts).

Pile Type	Required Allowable Axial Capacity (kN)
32-cm pipe	890
41-cm pipe	1,583
61-cm pipe	311
30-cm square PPC	356–756
41-cm square PPC	534–1,379

The axial capacity of the piles was verified using pile load tests, which were specified in section 940.62 of the general specifications.(14) The required ultimate capacities for the load tests were specified by applying a minimum factor of safety of 2.0 to the required allowable values. A factor of safety of 2.25 was specified in contract C19B1, which is consistent with the recommended American Association of State Highway and Transportation Officials (AASHTO) criteria for piles designed and evaluated based only on a subsurface exploration, static analysis, WEAP analysis, and dynamic pile testing.(16)

Dynamic load testing was required for test piles and for a portion of the production piles to monitor driving-induced stresses in the piles, evaluate hammer efficiency and performance, estimate the soil-resistance distribution, and evaluate the pile capacity during initial installation driving and restrikes. A waiting period of 12 to 36 hours (h) was required after pile installation before restrike tests could be performed.

Static load tests were required for test piles to confirm that the minimum specified allowable capacity was achieved and to better estimate or establish higher allowable design capacities. Section 1817.4.1 of the Massachusetts State building code says that the load reaching the top of the bearing stratum under maximum test load for a single pile or pile group must not be less than 100 percent of the allowable design load for end-bearing piles. Therefore, the specifications required that the static load test demonstrate that 100 percent of the design load was transferred to the bearing layer. If any of the test criteria were not met, the contractor was required to perform additional static load test(s).

CHAPTER 3. CONSTRUCTION EQUIPMENT AND METHODS

This chapter presents a description of the equipment and methods used during pile driving operations at the CA/T project in the selected contracts. This includes a general overview of impact hammers, how a pile is installed, and how to tell when a pile has reached the desired capacity. Construction issues associated with pile driving during this project are also presented. Pile heave was identified as an issue during construction of the arrivals tunnel at Logan Airport, which required a significant number of piles to be redriven. At another site at the airport, soil heave resulting from pile driving caused significant movement of an adjacent building and required changes to the installation process, including preaugering the piles to a depth of 26 m.

EQUIPMENT AND METHODS

Impact hammers were used to drive all of the piles for the CA/T project. An impact hammer consists of a heavy ram weight that is raised mechanically or hydraulically to some height (termed "stroke") and dropped onto the head of the pile. During impact, the kinetic energy of the falling ram is transferred to the pile, causing the pile to penetrate the ground.

Many different pile driving hammers are commercially available, and the major distinction between hammers is how the ram is raised and how it impacts the pile. The size of the hammer is characterized by its maximum potential energy, referred to as the "rated energy." The rated energy can be expressed as the product of the hammer weight and the maximum stroke. However, the actual energy transferred to the pile is much less a result of energy losses within the driving system and pile. The average transferred energies range from 25 percent for a diesel hammer on a concrete pile to 50 percent for an air hammer on a steel pile.[17]

Three types of hammers were used on the selected contracts: (1) a single-acting diesel, (2) a double-acting diesel, and (3) a single-acting hydraulic. The manufacturers and characteristics of the hammers used in these contracts are summarized in table 4, along with the pile types driven. Schematics of the three types of hammers are shown in figures 9 through 11.

Table 4. Summary of pile driving equipment used on the selected contracts.

Make and Model	Type	Action	Rated Energy (kN-m)	Pile Types Driven	Contracts	Designation
Delmag™ D 46-32	Diesel	Double	153.5	41-cm PPC	C07D1	I
HPSI 2000	Hydraulic	Single	108.5	41-cm PPC	C07D1, C07D2	II
ICE 1070	Diesel	Double	98.5	31-cm PPC, 41-cm PPC, 41-cm pipe	C08A1, C09A4	III
HPSI 1000	Hydraulic	Single	67.8	41-cm PPC	C19B1	IV
Delmag D 19-42	Diesel	Single	58.0	32-cm pipe	C19B1	V
Delmag D 30-32	Diesel	Single	99.9	32-cm pipe	C19B1	VI

A single-acting diesel hammer (figure 9) works by initially raising the hammer with a cable and then releasing the ram. As the ram free-falls within the cylinder, fuel is injected into the combustion chamber beneath the ram and the fuel/air mixture becomes pressurized. Once the ram strikes the anvil at the bottom of the cylinder, the fuel/air mixture ignites, pushing the ram back to the top of the stroke. This process will continue as long as fuel is injected into the combustion chamber and the stroke is sufficient to ignite the fuel.

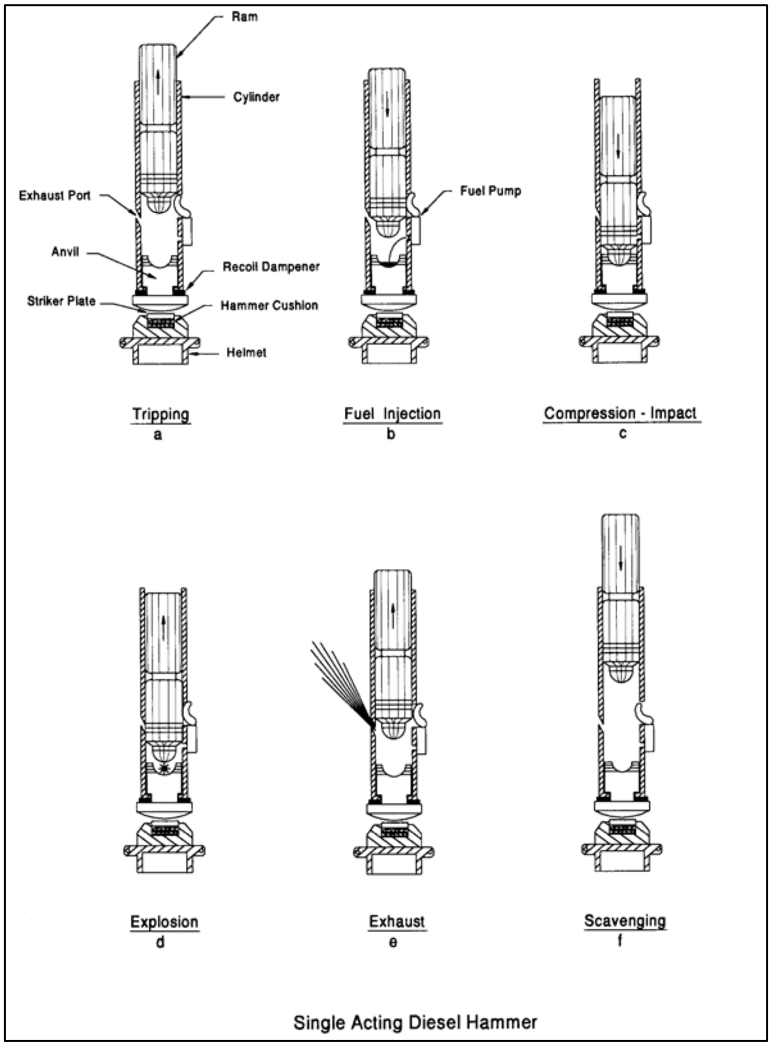


Figure 9. Single-acting diesel hammer.[17]

A double-acting diesel hammer (figure 10) works like the single-acting diesel hammer except that the system is closed at the top of the ram. As the ram rebounds to the top of the stroke, gasses are compressed in the bounce chamber at the top of the hammer. The bounce chamber temporarily stores and redirects energy to the top of the ram, allowing the stroke height to be reduced and the blow rate to be increased. Bounce chamber pressure is monitored during pile driving because it is correlated with hammer energy. The stroke of the hammer, and thus the energy, is controlled using the fuel pump. This is effective for avoiding bouncing of the hammer during the upstroke, which can lead to unstable driving conditions and damage to the hammer.[17]

A single-acting hydraulic hammer (figure 11) uses a hydraulic actuator and pump to retract the ram to the top of the stroke. Once the ram is at the top of the stroke, the ram is released and free-falls under gravity, striking the anvil. An advantage of hydraulic hammers is that the free-fall height, and thus the energy delivered to the pile, can be controlled more accurately.

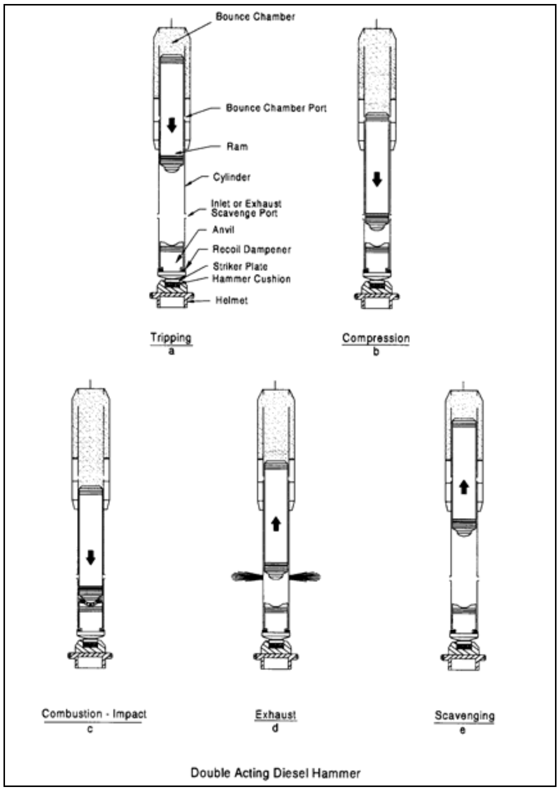


Figure 10. Double-acting diesel hammer.[17]

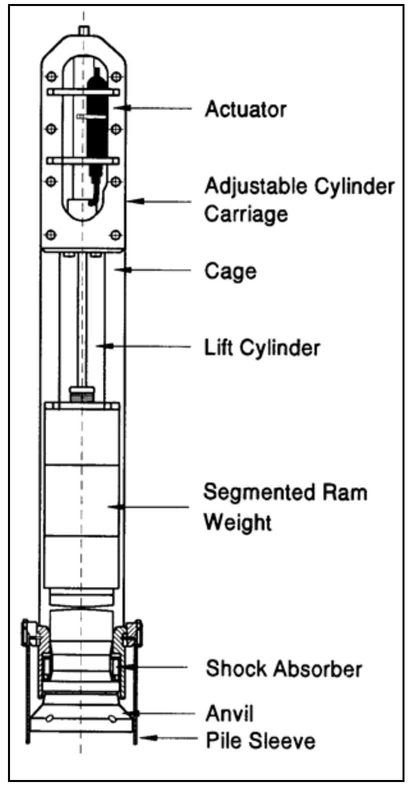


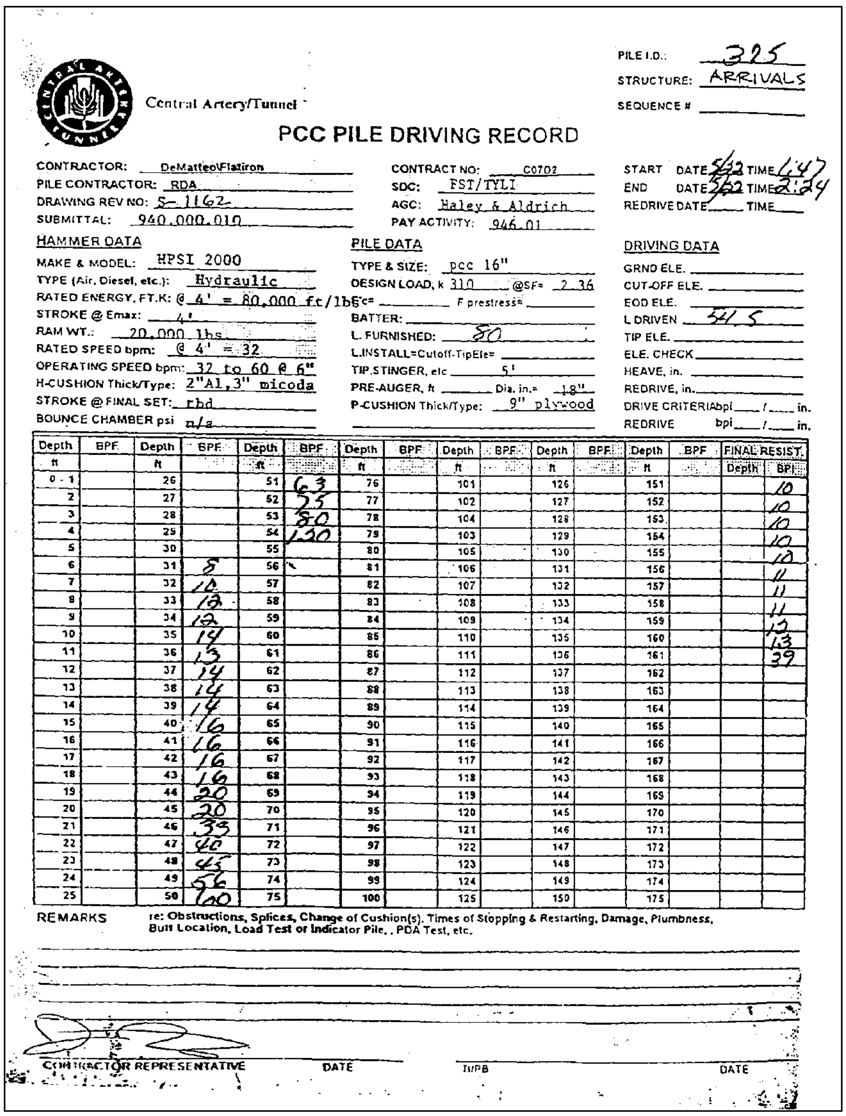
Figure 11. Single-acting hydraulic hammer.[17]

In preparation for driving, a pile is first hoisted to an upright position using the crane and is placed into the leads of the pile driver. The leads are braces that help position the piles in place and maintain alignment of the hammer-pile system so that a concentric blow is delivered to the pile for each impact. Once the pile is positioned at the desired location, the hammer is lowered onto the pile butt. A pile cushion consisting of wood, metal, or composite material is placed between the pile and the hammer prior to driving to reduce stresses within the pile during driving.

Once the pile is in position, pile driving is initiated and the number of hammer blows per 0.3 m of penetration is recorded. Toward the end of driving, blows are recorded for every 2.5 cm of penetration. Pile driving is terminated when a set of driving criteria is met. Pile driving criteria are generally based on the following: (1) the minimum required embedment depth, (2) the minimum number of blows required to achieve capacity, and (3) the maximum number of blows to avoid damage to the pile. All information that is associated with pile driving activities (e.g., hammer types, pile types, pile lengths, blow counts, etc.) is recorded on a pile driving log.

A typical pile driving log is shown in figure 12. This particular record is for the installation of a 24-m-long, 41-cm-diameter PPC pile installed at the airport as part of contract C07D2. A hydraulic hammer with an 89-kN ram and a 1.2-m stroke was used. The number of blows per 0.3 m of driving was recorded from an embedment depth of 9.5 m to a final depth of 16.5 m. At a depth of 16.5 m, the hammer blows required to drive the pile 2.5 cm were recorded in the right-hand column of the record. Driving was stopped after a final blow count of 39 blows per 2.5 cm was recorded.

Once a pile has been installed, the hammer may be used to drive the pile again at a later time. Additional driving that is performed after initial installation is referred to as a redrive or restrike. A redrive may be necessary for two reasons: (1) to evaluate the long-term capacity of the pile (i.e., pile setup or pile relaxation), or (2) to reestablish elevations and capacity in piles that have been subject to heave. Both of these issues were significant for the CA/T project, and they are discussed in the next section.



Central Artery/Tunnel

PILE I.D.: 325
STRUCTURE: ARRIVALS
SEQUENCE #

PCC PILE DRIVING RECORD

CONTRACTOR: DeMatteo\Flatiron
PILE CONTRACTOR: RDA
DRAWING REV NO: S-1162
SUBMITTAL: 940.000.010

CONTRACT NO: C07D2
SDC: FST/TYLI
AGC: Haley & Aldrich
PAY ACTIVITY: 946.01

START DATE 5/22 TIME 1:47
END DATE 5/22 TIME 2:24
REDRIVE DATE TIME

HAMMER DATA
MAKE & MODEL: HPSI 2000
TYPE (Air, Diesel, etc.): Hydraulic
RATED ENERGY, FT.K: @ 4' = 80,000 ft/lbs
STROKE @ Emax: 4'
RAM WT.: 20,000 lbs
RATED SPEED bpm: @ 4' = 32
OPERATING SPEED bpm: 32 to 60 @ 6"
H-CUSHION Thick/Type: 2"Al,3" micoda
STROKE @ FINAL SET: tbd
BOUNCE CHAMBER psi n/a

PILE DATA
TYPE & SIZE: pcc 16"
DESIGN LOAD, k 310 @SF= 2.36
f'c= F prestress=
BATTER:
L. FURNISHED: 80
L.INSTALL=Cutoff-TipEle=
TIP,STINGER, etc 5'
PRE-AUGER, ft Dia. in.= 18"
P-CUSHION Thick/Type: 9" plywood

DRIVING DATA
GRND ELE.
CUT-OFF ELE.
EOD ELE.
L DRIVEN 54.5
TIP ELE.
ELE. CHECK
HEAVE, in.
REDRIVE, in.
DRIVE CRITERIA bpi / in.
REDRIVE bpi / in.

Depth ft	BPF	Depth ft	BPF	Depth ft	BPF	Depth ft	BPF	Depth ft	BPF	Depth ft	BPF	Depth ft	BPF	FINAL RESIST. Depth	BPI
0 - 1		26		51	63	76		101		126		151			10
2		27		52	75	77		102		127		152			10
3		28		53	80	78		104		128		153			10
4		29		54	120	79		103		129		154			10
5		30		55		80		105		130		155			10
6		31	8	56		81		106		131		156			11
7		32	12	57		82		107		132		157			11
8		33	12	58		83		108		133		158			11
9		34	12	59		84		109		134		159			12
10		35	14	60		85		110		135		160			13
11		36	13	61		86		111		136		161			39
12		37	14	62		87		112		137		162			
13		38	14	63		88		113		138		163			
14		39	14	64		89		114		139		164			
15		40	16	65		90		115		140		165			
16		41	16	66		91		116		141		166			
17		42	16	67		92		117		142		167			
18		43	16	68		93		118		143		168			
19		44	20	69		94		119		144		169			
20		45	20	70		95		120		145		170			
21		46	33	71		96		121		146		171			
22		47	40	72		97		122		147		172			
23		48	45	73		98		123		148		173			
24		49	56	74		99		124		149		174			
25		50	100	75		100		125		150		175			

REMARKS re: Obstructions, Splices, Change of Cushion(s), Times of Stopping & Restarting, Damage, Plumbness, Butt Location, Load Test or Indicator Pile, , PDA Test, etc.

CONTRACTOR REPRESENTATIVE DATE TI/PB DATE

Figure 12. Typical pile driving record.

CONSTRUCTION-RELATED ISSUES

Pile Heave

Pile heave is a phenomenon where displacement of soil from pile penetration causes vertical or horizontal movement in nearby, previously driven piles. Pile heave generally occurs in insensitive clays that behave as incompressible materials during pile driving.[17] In these soils, the elevation of adjacent piles is often continuously monitored during driving to look for heave. If a pile moves in excess of some predetermined criterion, the pile is redriven to redevelop the required penetration and capacity. From a cost perspective, pile heave is important because redriving piles can require significant additional time and effort.

Pile Layout and Soil Conditions

Of the contracts reviewed, pile heave was an issue during construction of the arrivals tunnel at Logan Airport (contract C07D2). The location of the C07D2 site is shown in figure 1. A plan view of the arrivals tunnel structure showing the pile locations is shown in figure 13. The tunnel structure is approximately 159 m in length and is located where ramp 1A-A splits from the arrivals road. The tunnel was constructed using the cut-and-cover method, and thus a portion of the overburden soil was excavated prior to pile driving.

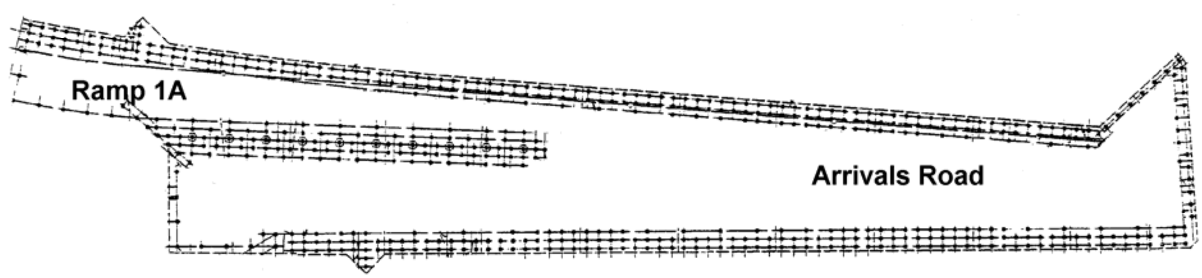


Figure 13. Site plan, piling layout for the arrivals tunnel at Logan Airport.[18]

Approximately 576 piles were driven beneath the alignment of the tunnel structure. The piles, consisting of 41-cm-diameter PPC piles, were designed to support a concrete mat foundation in addition to a viaduct located above the tunnel. They were generally installed in a grid-like pattern, with a spacing of approximately 1.2 m by 1.8 m center to center (figure 13).

The general subsurface conditions based on borings advanced in the area prior to excavation consist of approximately 3 to 6.1 m of cohesive and/or granular fill, overlying 1.5 to 3 m of organic silt and sand, overlying 12.2 to 42.7 m of soft marine clay, overlying 0.9 to 2.8 m of glacial silts and sands, underlain by bedrock.[6] Excavation was accomplished into the clay layer, resulting in a clay layer thickness of about 6.1 m at the southeastern end of the structure to around 3.7 m at the northwestern end.[19]

The piles were designed for end bearing in the dense glacial silts and sands, and were preaugered to about the bottom of the marine clay layer to minimize heave and displacement of these soils. The preauger depths were approximately 30 to 70 percent of the final embedment depths of the piles. Preaugering was done using a 46-cm-diameter auger, which is the equivalent circular diameter of the 41-cm square pile. The piles were driven using an HPSI 2000 hydraulic hammer.

Field Observations

Pile heave was monitored during construction by field engineers. As described in the Massachusetts State building code and project specifications, piles identified with vertical displacement exceeding 1.3 cm required redriving. According to field records, 391 of the 576 piles (68 percent) installed required redriving. Of those 391 piles, 337 piles (86 percent) were driven in one redrive event, 53 piles (14 percent) required a second redrive event, and 1 pile required a third redrive event. The impact on the construction schedule or costs was not identified. Despite the use of partial preaugering, a significant portion of the piles showed excessive heave and required substantial redrive efforts. Heave is attributed to the displacement of the underlying glacial soils that were not preaugered.

Pile heave issues were not identified on the other CA/T contracts. Since partial preaugering was used on the majority of these contracts, the difference may be related to the spacing between piles. Table 5 summarizes the pile spacing used on the selected contracts. As shown in table 5, the pile spacing of 1.2 m used at the arrivals tunnel structure is significantly less than the spacing used for structures of comparable size. Therefore, it is anticipated that a pile spacing of greater than about 1.8 m may limit pile heave to within the 1.3-cm criterion.

Table 5. Summary of pile spacing from selected contracts.

Contract	Structure	Foundation	Bent Spacing (m)	Pile Spacing (m)
C07D1	Ramp ET	Slab	2.7	2.7
		Pile cap	1.4	1.4
	Egress Ramps	Pile cap	1.8	1.8
C07D2	Arrivals Tunnel	Pile cap	1.8	1.2
		Pile cap	1.8	1.2
		Pile cap	1.4	1.2
C08A1	South Abutment	Pile cap	3.05	1.82.4
	East Abutment	Pile cap	1.1–2.7	1.4–2.6
	West Abutment	Pile cap	1.1–2.1	1.4–2.7
C09A4	Utilities	Pile cap	2.0–2.7	1.8
	Approach No. 1	Slab	3.7	5.6
		Pile cap	1.4	2.6
		Pile cap	NA	1.4
		Pile cap	NA	1.5
	Approach No. 2	Slab	4.57	3.1–4.6
	Approach No. 5	Slab	3.7–4.9	2.1–4.3
C19B1	NS-SN	Slab	3.7	4.9
	Ramp CT	Slab	3.1	4.6
	Ramp LT	Slab	2.9–3.2	2.4–3.1

NA = not applicable or available

Soil Heave

Soil heave caused by pile driving was primarily responsible for the significant movement observed at a building adjacent to the construction of the east abutment and east approach to ramp ET at Logan Airport (contract C07D1). Shortly after the start of pile driving, settlement in excess of 2.5 cm was measured at the perimeter of the building and cracking was observed on the structure itself. These observations prompted the installation of additional geotechnical instrumentation, installation of wick drains to dissipate excess pore pressure generated during pile driving, and preaugering of the piles to reduce soil displacement. Despite these efforts, heave continued to a maximum vertical displacement of 8.8 cm. (See references 20, 21, 22, and 23.)

Pile Layout and Soil Conditions

The location of the project in relation to the building is shown in figure 14. The portion of the east approach that is adjacent to the building consists of two major structures, including an abutment and a pile-supported slab. Both structures are supported by 41-cm-diameter PPC piles. The layout of the pile foundation system is also shown in figure 14. The piles for the slab are arranged in a grid-like pattern with a spacing of about 2.7 m center to center. A total of 353 piles support the structures.

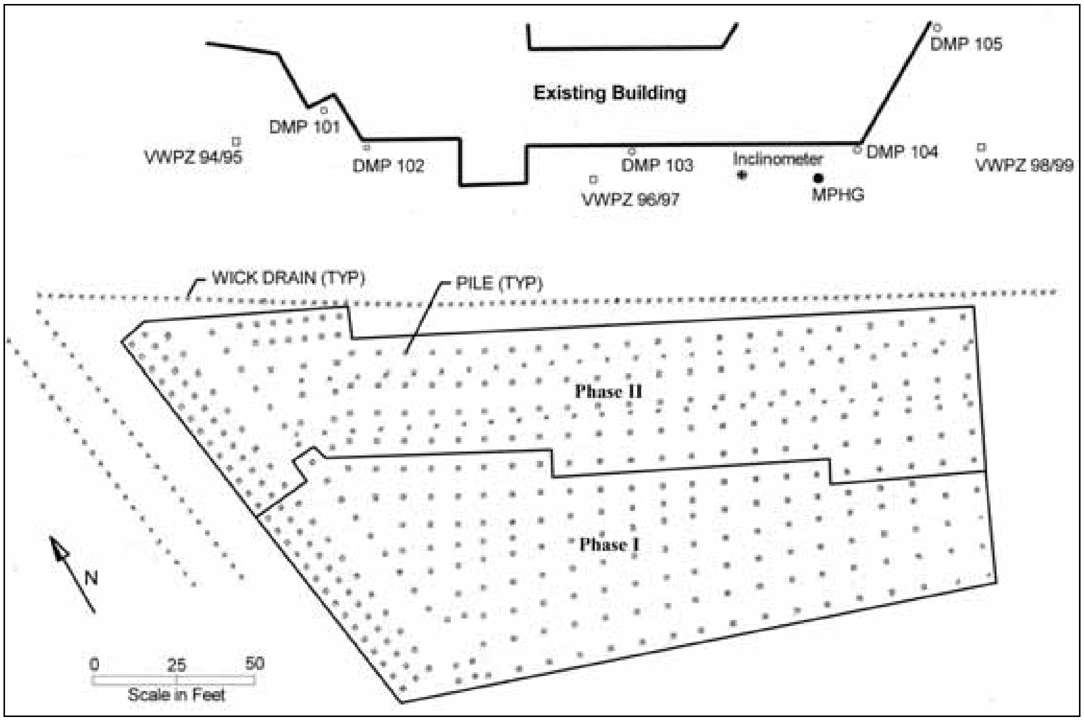


Figure 14. Site plan showing locations of piles, building footprint, and geotechnical instrumentation.

Prior to construction activities, five deformation monitoring points (DMPs) were installed along the front perimeter of the building closest to the work area. The DMPs consisted of 13-cm-long hex bolts fixed to the building. These points, designated DMP-101 through DMP-105, were monitored for vertical movement. The DMPs were monitored initially by the contractor and subsequently monitored by an independent consultant.

The subsurface conditions based on borings advanced in the area consist of approximately 3 to 4.6 m of fill, overlying 3 to 6.1 m of organic silt and sand, overlying 27.4 to 33.5 m of soft marine clay, overlying 6.1 to 12.2 m of glacial silt and sand, underlain by bedrock. The piles were designed as end bearing piles to be driven into the dense underlying glacial materials. The glacial soils were encountered at depths of approximately 39.6 to 45.7 m below the ground surface and bedrock was encountered at a depth of approximately 48.8 m.

Field Observations (Phase I Pile Driving)

Pile driving for the east approach was executed in two phases. The first phase began on April 5, 1995, and concluded on June 10, 1995. The second phase began on July 13, 1995, and concluded on August 17, 1995. The piles were driven using a Delmag D46-32 single-acting diesel hammer. The extent of the first phase of pile driving is shown in figure 15. This first phase of work was performed no closer than 27.4 m from the building. The majority of the piles for the slab were installed from the west side of the site working toward the east during the periods of April 5 to April 23, and May 15 to June 2. The majority of the piles for the abutment were installed at the west end of the site during the period of April 23 to May 15.

Settlement data obtained by the contractor during the first phase of pile driving are shown in figure 15. On April 21, 1995, after approximately 2 weeks of pile driving on the west side of the site, initial heave displacements of 0.9 and 0.7 cm were measured in DMP-102 and DMP-103, respectively. Notable heave was observed at DMP-101 and DMP-104 on May 1, which registered displacements of 1.3 and 0.8 cm, respectively. An initial heave displacement of 0.4 cm was measured in DMP-105 on May 9. The heave increased steadily to maximum values as pile driving commenced toward the east side of the site.

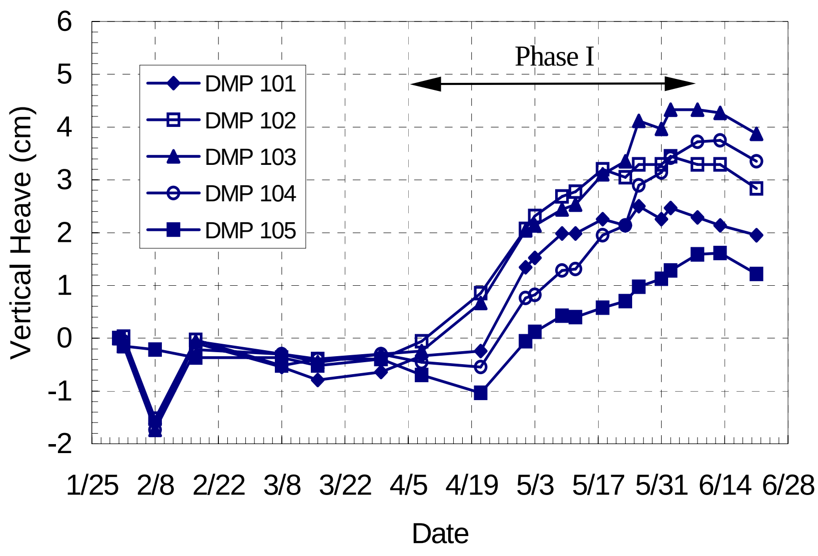


Figure 15. Settlement data obtained during first phase of pile driving.

A summary of the maximum heave values attributed to the first phase of driving is given in table 6. The greatest amount of heave occurred in DMP-103, which was centrally located relative to the pile grid. On June 2, 1995, 1 week before completion of construction, the heave measured in DMPs 101 through 103 began to level off and subside.

Table 6. Maximum building heave (in cm) observed during pile driving.

Construction Phase	DMP 101	DMP 102	DMP 103	DMP 104	DMP 105
Phase I	2.5	3.5	4.3	3.8	1.6
Phase II	3.6	4.8	5.3	3.7	1.3

As a result of the excessive heave (greater than 2.5 cm) observed in the first phase of pile driving, mitigation measures were implemented for the second phase of work. This was critical considering that the second phase involved driving piles even closer to the building. The geotechnical consultant recommended three approaches to limiting heave based on schedule and cost constraints.[24] These included: (1) installation and monitoring of pore pressures in the clay during driving and adjusting mitigating measures as appropriate; (2) installation of wick drains between the Hilton and the work area to intercept and aid in the reduction of pore pressures beneath the Hilton that may be generated from pile driving; and (3) based on the performance of the wick drains, preauger phase II piles to limit soil displacement.

Field Observations (Phase II Pile Driving)

Prior to the start of the second phase of pile driving, three double-nested vibrating wire piezometers (VWPZ) were installed to measure pore pressures. These piezometers were installed in close proximity to three of the existing deformation monitoring points (DMP-102 through DMP-104). Additional instrumentation was also installed following the start of the second phase of work, including a multipoint heave gauge (MPHG) to measure vertical movement with depth and an inclinometer to measure lateral movement. The locations of the additional geotechnical instrumentation are shown in figure 14.

The second phase of pile driving began on July 13, 1995, and concluded on August 17, 1995. The extent of the work area is also shown in figure 14. Pile driving generally progressed from the west side of the site toward the east. The location of the second phase of work was no closer than 15.2 m from the existing building.

Shortly after the start of driving, 200 wick drains were installed from July 20 to July 28, 1995, around the western and northern perimeters of the work area. The drains were installed through the clay layer at a spacing of 1.2 m center to center.

Settlement data for the second phase of work, shown in figure 16, demonstrate that heave began to increase at DMP-101 through DMP-104 approximately 1 week after the start of pile driving. Based on the review of initial settlement data, preaugering was implemented from August 4, 1995, through the completion of construction. Preaugering was accomplished using a 41-cm-diameter auger to a depth of 26 m, which is approximately 50 to 60 percent of the pile's final embedment depth. The auger diameter is 11 percent less than the 46-cm equivalent circular diameter for a 41-cm square pile.

As shown in figure 16, heave continued to increase even after preaugering was initiated. Net heave values of 3.3 to 13.5 cm (table 6) were observed from the start of preaugering to the completion of pile driving, resulting in total heave values ranging from 2.6 to 8.8 cm.

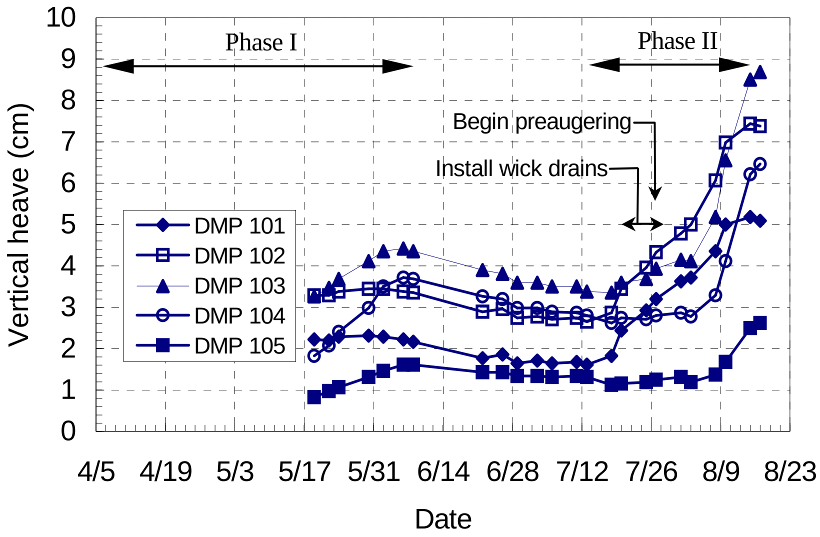


Figure 16. Settlement data obtained during second phase of pile driving.

Data from the multipoint heave gauge showed that the magnitude of the heave was relatively constant within the upper 30 m, as shown in figure 17. However, vertical displacement decreases dramatically below this depth to approximately zero at the bedrock depth of approximately 50 m. The maximum heave of approximately 5.1 cm at a depth of 3 m below the ground surface is also consistent with the maximum value of 5.3 cm recorded at DMP-103.

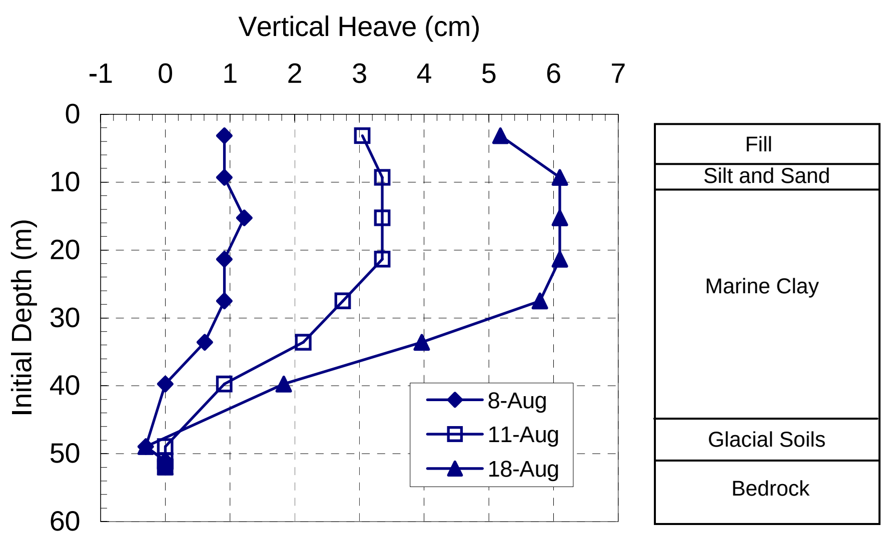


Figure 17. Multipoint heave gauge data obtained during second phase of pile driving.

The excess pore pressures recorded during the second phase of pile driving are presented in figure 17. The six gauges shown in figure 18 correspond to three pairs (55894–55895, 55896–55897, and 55898–55899) located adjacent to DMP-102, DMP-103, and DMP-104, respectively. There was an increase in the excess pore pressures throughout the pile driving, with maximum values ranging from 0.6 to 12.8 m of head with an average of 5.9 m. The greatest head was measured in VWPZ-55896 at a location nearest DMP-103. These data suggest that the wick drains were not effective in dissipating all excess pore pressures generated during pile driving.

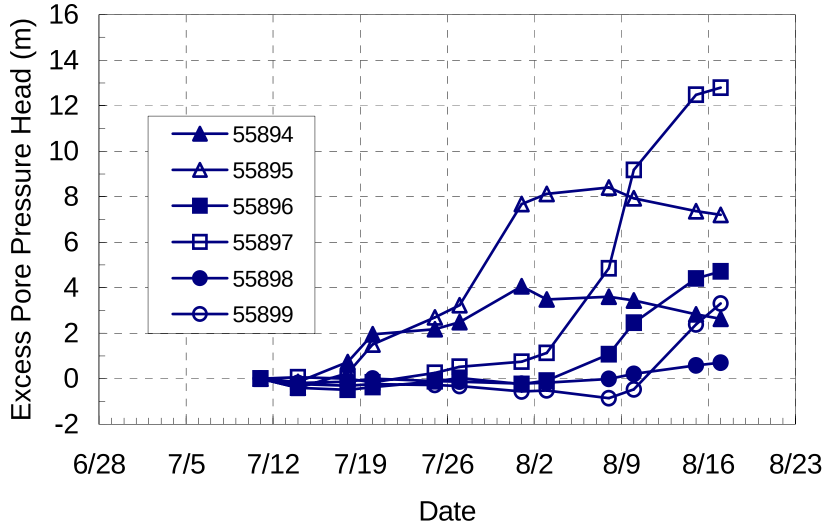


Figure 18. Pore pressure data obtained during second phase of pile driving.

The inclinometer data that were obtained adjacent to the building are shown in figure 19. These data showed increasing lateral movement in the direction of the building during pile driving. The maximum net lateral deformations were relatively constant with depth within the upper 30 m of the profile. A maximum deformation of approximately 6 cm was recorded at a depth of approximately 34 m. Similar to the vertical deformations, the lateral deformations decreased sharply below this depth to zero at the bedrock depth. These data suggest that the lateral deformations are of the same magnitude and behavior as the vertical deformations.

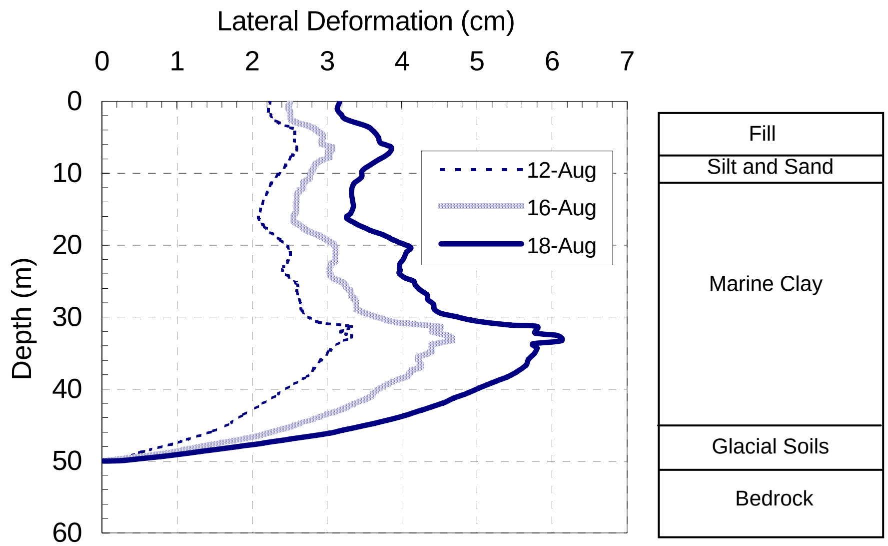


Figure 19. Inclinometer data obtained during second phase of pile driving.

Summary

Soil heave was recognized early on as a potential problem and following phase I driving in contract C07D1, several mitigation efforts were initiated. These included installing wick drains to promote rapid dissipation of excess pore pressures and preaugering piles through a portion of the soft clay layer to a depth of 26 m. Additional instrumentation was installed, including piezometers, MPHG, and an inclinometer. Despite these efforts, heave during phase II pile driving continued to increase to a maximum displacement of 8.8 cm. The piezometer data indicate that the wick drains were not effective in rapidly dissipating pore pressures generated during pile driving. The deformation data indicated that soil heave can still occur in piles that are preaugered over a portion of their embedded depth.

CHAPTER 4. DYNAMIC AND STATIC PILE LOAD TEST DATA

This chapter presents the methodology and results of dynamic and static pile load test data for the selected contracts. At least two static load tests were performed per contract, and the results of 15 tests are presented herein. The Pile Driving Analyzer® (PDA) was also used on these piles for comparison, and analyses were performed periodically during production pile installation. Issues related to design loads and load test criteria are discussed, including factors of safety and load transfer requirements. A comparison is made between the results of the static load tests and the CAse Pile Wave Analysis Program (CAPWAP®) analyses. The CAPWAP data suggest that the quake values generally exceed the values typically recommended in wave equation analyses. A review of the literature is presented to evaluate the significance of this finding. High blow counts recorded during the end of driving also suggest that the majority of the estimated pile capacities from CAPWAP are conservative.

LOAD TEST METHODS

Dynamic Load Test Methods

Approximately 160 dynamic pile load tests were performed to evaluate pile capacity, driving stresses, and hammer performance during the installation of test piles and production piles. The data presented in this report were obtained from project files. (See references 25, 26, 27, 28, 29, 30, 31, 32, 33, 34.)

The PDA was used to record, digitize, and processes the force and acceleration signals measured at the pile head. These signals were used to estimate static capacity using the Case Method, a simplified field procedure for estimating pile capacity, as well as the more rigorous CAPWAP. The dynamic load test results discussed in this report are primarily from the CAPWAP analyses. A description of the fundamentals of dynamic testing, including CAPWAP, is presented in *Design and Construction of Driven Pile Foundations* (Federal Highway Administration (FHWA) report no. FHWA-HI-97-013).[17] The dynamic testing was carried out in general accordance with project specifications section 940.62.C,[14] Dynamic Load Tests, and D4945-89 of the American Society for Testing and Materials (ASTM). D4945-89 is entitled "Standard Method for High Strain Testing of Piles."[35]

CAPWAP is an iterative curve-fitting technique where the pile response determined in a wave equation model is matched to the measured response of the actual pile for a single hammer blow. The pile model consists of a series of continuous segments and the total resistance of the embedded portion of the pile is represented by a series of springs (static resistance) and dashpots (dynamic resistance). Static resistance is formulated from an idealized elastoplastic soil model, where the quake parameter defines the displacement at which the soil changes from elastic to plastic behavior. The dynamic resistance is formulated using a viscous damping model that is a function of a damping parameter and the velocity.

First, the forces and accelerations acting on the actual pile during initial impact are recorded with a strain gauge and accelerometer mounted at the pile head. The measured acceleration is used as input to the pile model along with reasonable estimates of soil resistance, quake, and damping parameters. The force-time signal at the pile head is calculated using the model and is compared to the measured force-time signal. The soil-resistance distribution, quake, and damping parameters are subsequently modified until agreement is reached between the measured and calculated signals. An example of a comparison between a measured and calculated force signal from one of the test piles is shown in figure 20. Once an acceptable match is achieved, the solution yields an estimate of ultimate static capacity, the distribution of soil resistance along the pile, and the quake and damping parameters.

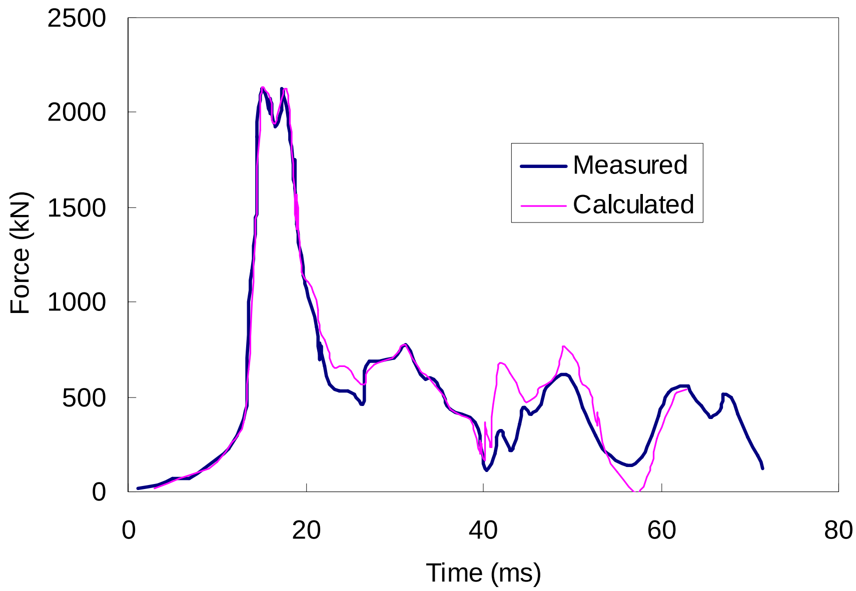


Figure 20. Example of CAPWAP signal matching, test pile 16A1-1.[33]

Static Load Test Methods

Static load tests were performed during the test phase of each contract to verify the design assumptions and load-carrying capacity of the piles. Telltale rods installed at various depths within the piles were used to evaluate the load transfer behavior of the piles with regard to the surrounding soil and bearing stratum. The static tests were carried out in general accordance with project specifications section 940.62.B.4,[14] Short Duration Test, and the ASTM's D1143-81, which is entitled "Standard Test Method for Piles Under Static Axial Compression Load."[36] The static load test data presented in this report were obtained from the project files. (See references 37 through 50.)

Static loads were applied and maintained using a hydraulic jack and were measured with a load cell. A typical load test arrangement is shown in figure 21. Reaction to the jack load is provided by a steel frame that is attached to an array of steel H-piles located at least 3 m away from the test pile. Pile head deflections were measured relative to a fixed reference beam using dial gauges. Telltale measurements were made in reference to the pile head or the reference beam using dial gauges. Pile head and telltale deflection data were recorded for each loading increment.

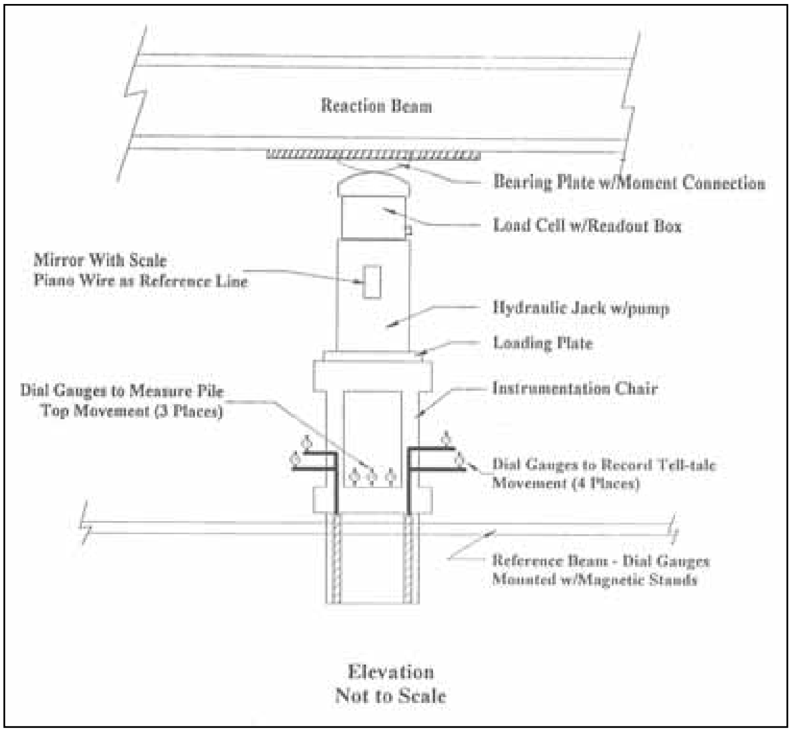


Figure 21. Typical static load test arrangement showing instrumentation.[51]

An excerpt from the loading procedures for short-duration load test section 940.62 is given below[14]:

> a) Apply 25 percent of the allowable design load every one-half hour up to the greater of the following: [two alternatives are described; the most general is 200 percent of the design load]. Longer time increments may be used, but each time increment should be the same. At 100 percent of the design load, unload to zero and hold for one-half hour; then reload to 100 percent and continue 25 percent incremental loads. At 150 percent, unload to zero and hold for one-half hour; then reload to 150 percent and continue 25 percent incremental loads. In no case shall the load be changed if the rate of settlement is not decreasing with time.
>
> b) At the maximum applied load, maintain the load for a minimum of one hour and until the settlement (measured at the lowest point of the pile at which measurements are made) over a one-hour period is not greater than 0.254 mm (0.01 inch).
>
> c) Remove 25 percent of the load every 15 minutes until zero load is reached. Longer time increments may be used, but each shall be the same.
>
> d) Measure rebound at zero load for a minimum of one hour.

e) After 200 percent of the load has been applied and removed, and the test has shown that the pile has additional capacity, i.e., it has not reached ultimate capacity, continue testing as follows. Reload the test pile to the 200 percent design load level in increments of 50 percent of the allowable design load, allowing 20 minutes between increments. Then increase the load in increments of 10 percent until either the pile or the frame reach their allowable structural capacity, or the pile can no longer support the added load. If failure at maximum load does not occur, hold load for one hour. At maximum achieved load, remove the load in four equal decrements, allowing 15 minutes between decrements.

The capacity of the test piles was selected as the greater capacity defined by two failure criteria. The first criteria establishes the allowable design capacity as “50 percent of the applied test load which results in a net settlement of the top of the pile of up to 1.3 cm, after rebound, for a minimum of one hour at zero load.” The second criterion uses Davisson’s criteria as described below.

The Davisson offset limit load criterion was used on the project to define the ultimate capacity, or failure, of the test piles.[52] The ultimate load is interpreted as the point at which the displacement of the pile head meets a limit that is offset to the elastic compression line of the pile. For piles less than 61 cm in diameter, the limit is defined by the following linear relationship:

$$S_f = S_e + (0.38 + 0.008D) \tag{1}$$

where,

S_f = Movement of pile top (cm).
D = Pile diameter or width (cm).
S_e = Elastic compression of total pile length (cm).

The elastic compression in this case refers to the pile deflection that would occur if 100 percent of the applied load was transferred to the toe of the pile (i.e., zero shaft friction), and is given by the following equation:

$$S_e = \frac{QL}{AE} \tag{2}$$

where,

Q = Applied load.
L = Total length of pile.
A = Cross-sectional area of the pile.
E = Modulus of elasticity of the pile.

The average load in the pile at the midpoint between two telltale locations was estimated from the elastic shortening of the pile using the following equation:

$$Q_{avg} = AE \frac{D_1 - D_2}{\Delta L} \tag{3}$$

where,

- A = Area of pile.
- E = Modulus of elasticity of the pile.
- D_1 = Deflection at upper telltale location.
- D_2 = Deflection at lower telltale location.
- ΔL = Distance between the upper and lower telltale locations.

Both equations 2 and 3 require the modulus of elasticity of the pile. The specifications require that the elastic modulus be determined via compression tests performed on representative concrete samples (ASTM C 469-87a). However, this method is not really applicable to the concrete-filled steel pipe piles. It was common practice on the CA/T project to use the upper telltale and pile head deflections to calculate the modulus of the pile using equation 3. This approach was justified by assuming that any preaugering that was performed prior to pile installation would reduce the shaft friction, especially near the pile head. In some cases, the elastic modulus of the PPC piles was determined from a combination of telltale and compression test data using engineering judgment.

LOAD TEST RESULTS

More than 160 dynamic tests were performed on the selected contracts to evaluate pile capacity during both the testing and production phases. Of these 160 tests, the results of 28 tests are presented in this report because they correspond to static load tests on 15 piles. Information about each pile tested is shown in table 7, and pile driving information is presented in table 8.

Table 7. Summary of pile and preauger information.

Test Pile Name	Contract	Pile Type	Preauger Depth (m)	Preauger Diameter (cm)
ET2-C2	C07D1	41-cm PPC	0	NA[1]
ET4-3B	C07D1	41-cm PPC	0	NA
375	C07D2	41-cm PPC	9.1	45.7
923	C07D2	41-cm PPC	24.1	45.7
I90 EB SA	C08A1	41-cm PPC	NI[2]	40.6
14	C08A1	41-cm PPC	27.4	40.6
12A1-1	C09A4	31-cm PPC	30.5	45.7
12A2-1	C09A4	31-cm PPC	32.0	45.7
16A1-1	C09A4	41-cm PPC	30.5	45.7
I2	C09A4	41-cm PPC	30.5	40.6
3	C09A4	41-cm pipe	24.4	40.6
7	C09A4	41-cm pipe	24.4	40.6
IPE	C19B1	32-cm pipe	7.6	30.5
IPW	C19B1	32-cm pipe	12.2	30.5
NS-SN	C19B1	41-cm PPC	8.2	40.6

Notes:
1. NA = Not applicable.
2. NI = Data not identified.

Table 8. Summary of pile driving information.

Test Pile Name	Test Type[1]	Hammer Type[2]	Embedment Depth (m)	Minimum Transferred Energy (kN-m)	Recorded Penetration Resistance (blows/2.5 cm)	Permanent Set (cm)
ET2-C2	EOD	I	47.5	NI[3]	7,7,7	0.36
	34DR	–	–	58.0	11	0.23
ET4-3B	EOD	II	41.1	NI	8,7,10	0.25
	NI	–	–	50.8	14	0.18
375	EOD	II	16.8	50.2	12,13,39	0.08
	7DR	–	–	54.2	> 12	< 0.20
923	EOD	II	32.9	46.1	7,7,7	0.36
	7DR	–	–	51.5	> 8	0.33
I90 EB SA	EOD	III	46.6	25.8	12,10,10	0.25
	1DR	–	–	25.8	13	0.20
14	EOD	III	45.4	25.8	10,10,16	0.15
	1DR	–	–	23.1	21	0.13
12A1-1	EOD	III	41.8	20.7	4,4,5	0.51
	1DR	–	–	28.6	> 7	> 0.36
12A2-1	EOD	III	38.7	15.3	3,4,4	0.64
	1DR	–	–	18.6	8	0.33
16A1-1	EOD	III	43.3	24.4	6,7,7	0.36
	3DR	–	–	17.1	11	0.23
I2	EOD	III	37.2	27.1	4,4,4	0.64
	1DR	–	–	19.0	5	0.51

Table 8. Summary of pile driving information (continued).

Test Pile Name	Test Type[1]	Hammer Type[2]	Embedment Depth (m)	Minimum Transferred Energy (kN-m)	Recorded Penetration Resistance (blows/2.5 cm)	Permanent Set (cm)
3	EOD	III	39.6	57.1	11,12,14	0.18
	1DR	–	–	49.9	30	0.08
7	EOD	III	38.1	49.8	11,11,11	0.23
	3DR	–	–	50.2	> 16	< 0.15
IPE	EOD	V	19.5	39.6	5,5,5	0.51
	1DR	–	–	53.0	7	0.36
IPW	EOD	VI	22.6	43.3	5,5,5	0.51
	1DR	–	–	59.7	8	0.33
NS-SN	EOD	IV	13.4	27.1	8,15,16	0.15
	7DR	–	–	24.4	26	0.10

Notes:
1. EOD = End of initial driving, #DR = # days before restrike.
2. Hammer types: I = Delmag D 46-32, II = HPSI 2000, III = ICE 1070, IV = HPSI 1000, V = Delmag D 19-42, VI = Delamag D 30-32.
3. NI = Data not identified.

Dynamic Results and Interpretation

Dynamic tests were performed both at the end of initial driving of the pile (EOD) and at the beginning of restrike (BOR), typically 1 to 7 days (1DR, 7DR, etc.) after installation. In most cases, the dynamic tests were performed before the static load tests. Test piles ET2-C2 and ET4-3B, however, were dynamically tested during a restrike after a static load test was performed. The ultimate capacities of the 15 test piles as determined by CAPWAP analysis are summarized in table 9. The table lists when the test was performed, as well as the predicted shaft and toe resistance.

Table 9. Summary of CAPWAP capacity data.

Test Pile Name	Test Type[1]	Recorded Penetration Resistance (blows/2.5 cm)	Ultimate Capacity[2] (kN)		
			Shaft	Toe	Total
ET2-C2	EOD	7,7,7	NI[3]	NI	NI
	34DR	11	(2,028)	(1,219)	(3,247)
ET4-3B	EOD	8,7,10	NI	NI	NI
	NI	14	(1,744)	(1,975)	(3,719)
375	EOD	12,13,39	(890)	(3,336)	(4,226)
	7DR	> 12	(1,245)	(3,514)	(4,759)
923	EOD	7,7,7	667	1,904	2,571
	7DR	> 8	(1,664)	(1,708)	(3,372)
I90 EB SA	EOD	12,10,10	934	712	1,646
	1DR	13	(1,156)	(1,112)	(2,268)
14	EOD	10,10,16	(449)	(2,237)	(2,687)
	1DR	21	(894)	(1,926)	(2,820)
12A1-1	EOD	4,4,5	685	979	1,664

Table 9. Summary of CAPWAP capacity data (continued).

Test Pile Name	Test Type[1]	Recorded Penetration Resistance (blows/2.5 cm)	Ultimate Capacity[2] (kN)		
	1DR	> 7	(1,103)	(743)	(1,846)
12A2-1	EOD	3,4,4	316	845	1,161
	1DR	8	1,023	431	1,454
16A1-1	EOD	6,7,7	956	1,063	2,015
	3DR	11	(983)	(876)	(1,859)
I2	EOD	4,4,4	400	1,130	1,530
	1DR	5	1,526	489	2,015
3	EOD	11,12,14	(983)	(2,086)	(3,069)
	1DR	30	(1,228)	(1,690)	(2,918)
7	EOD	11,11,11	(80)	(2,740)	(2,820)
	3DR	> 16	(983)	(1,984)	(2,962)
IPE	EOD	5,5,5	489	1,334	1,824
	1DR	7	645	1,535	2,180
IPW	EOD	5,5,5	778	1,223	2,002
	1DR	8	1,290	1,468	2,758
NS-SN	EOD	8,15,16	(583)	(1,806)	(2,389)
	7DR	26	(858)	(1,935)	(2,793)

Notes:

1. EOD = End of initial driving, #DR = # days before restrike.
2. Values shown in parentheses denote conservative values.
3. NI = Data not identified.

Many of the capacities are listed in parentheses, which indicates that the values are most likely conservative (i.e., the true ultimate capacity is larger). It is recognized in the literature that dynamic capacities can be underestimated if the hammer energy is insufficient to completely mobilize the soil resistance.(53) Specifically, research has shown that blow counts in excess of 10 blows per 2.5 cm may not cause enough displacement to fully mobilize the soil resistance.(53,54) As shown in table 8, the majority of the piles during restrike exceeded 10 blows per 2.5 cm and are thus likely to be lower than the true ultimate capacity of the piles.

The conservativeness of the CAPWAP capacities in certain piles can be illustrated by comparing the load versus displacement curve at the toe evaluated with CAPWAP to that obtained in a static load test. The toe load-displacement curves from test pile 16A1-1 are shown in figure 22. Blow counts of seven blows per 2.5 cm were recorded for this pile during initial driving. The static load test data shown in figure 22 were extrapolated from the telltale data. As shown in figure 22, the maximum resistance mobilized by the pile toe from CAPWAP is approximately 1060 kN. At least 1670 kN were mobilized in the static load test; however, the ultimate value is actually higher since failure was not reached.

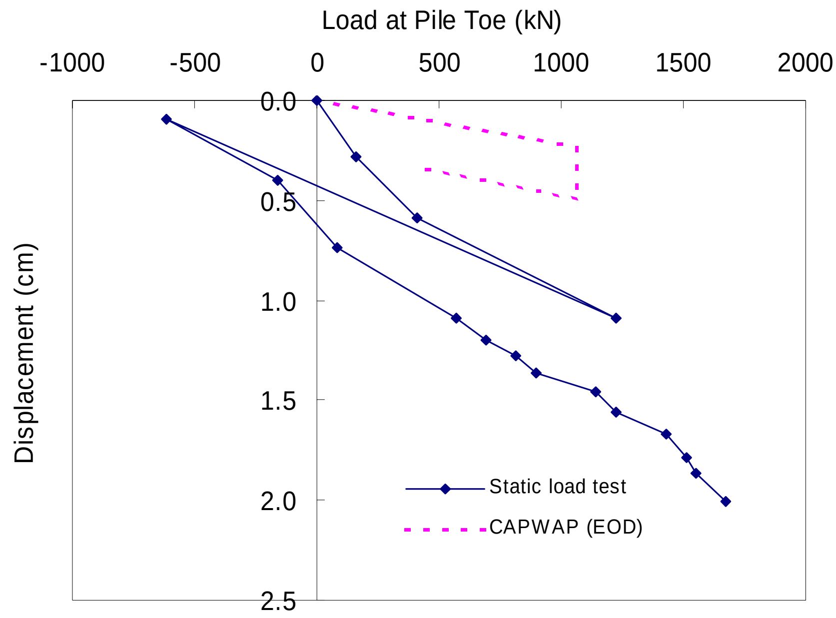


Figure 22. Load-displacement curves for pile toe, test pile 16A1-1.

Soil quake and damping parameters obtained from the CAPWAP analyses are summarized in table 10. It is often assumed that the quake values are approximately 0.25 cm in typical wave equation analyses. The toe quake values in this study range from 0.25 to 1.19, with an average of 1.6 cm. Large toe quake values on the order of up to 2.5 cm have been observed in the literature.(55,56) However, the quake values in this study appear to be within typical values.(57)

Table 10. Summary of CAPWAP soil parameters.

Test Pile Name	Test Type[1]	Quake (cm)		Damping (s/m)	
		Shaft	Toe	Shaft	Toe
ET2-C2	EOD	–	–	–	–
	34DR	0.43	0.84	0.72	0.23
ET4-3B	EOD	–	–	–	–
	-	0.56	0.36	0.89	0.82
375	EOD	0.64	1.19	0.33	0.07
	7DR	0.51	0.86	0.23	0.20
923	EOD	0.38	1.14	0.72	0.43
	7DR	0.23	0.81	0.46	0.43
I90 EB SA	EOD	0.13	0.89	0.16	0.56
	1DR	0.38	0.56	0.69	0.69
14	EOD	0.25	0.76	0.39	0.43
	1DR	0.25	0.41	0.59	0.43
12A1-1	EOD	–	–	–	–
	1DR	0.38	0.56	0.75	0.16
12A2-1	EOD	–	–	–	–
	1DR	0.25	0.51	0.49	0.33
16A1-1	EOD	–	–	–	–
	3DR	0.25	0.10	1.41	1.15
I2	EOD	0.25	0.51	0.75	0.26
	1DR	0.13	0.25	0.46	0.10
3	EOD	0.48	0.64	0.13	0.10
	1DR	0.15	0.56	0.33	0.10
7	EOD	0.23	0.64	0.46	0.10
	3DR	0.25	0.36	0.52	0.10
IPE	EOD	0.25	0.69	0.62	0.23
	1DR	0.38	0.89	0.59	0.23
IPW	EOD	0.38	0.64	0.43	0.23
	1DR	0.25	0.36	0.59	0.20
NS-SN	EOD	0.30	0.91	0.52	0.33
	7DR	0.13	0.46	0.72	0.49

Notes:
1. EOD = End of initial driving, #DR = # days before restrike.
2. s/m = seconds/meter.

Comparison of CAPWAP Data

A comparison between the EOD and BOR CAPWAP capacities is shown in figure 23. The line on the figure indicates where the EOD and BOR capacities are equal. Data points that are plotted to the left of the line show an increase in the capacity over time, whereas data that fall below the line show a decrease in capacity. In the four piles (12A2-1, I2, IPE, and IPW) where the soil resistance was believed to be fully mobilized for both the EOD and BOR, the data show an increase of 20 to 38 percent occurring over 1 day. The overall increase in capacity is attributed to an increase in the shaft resistance.

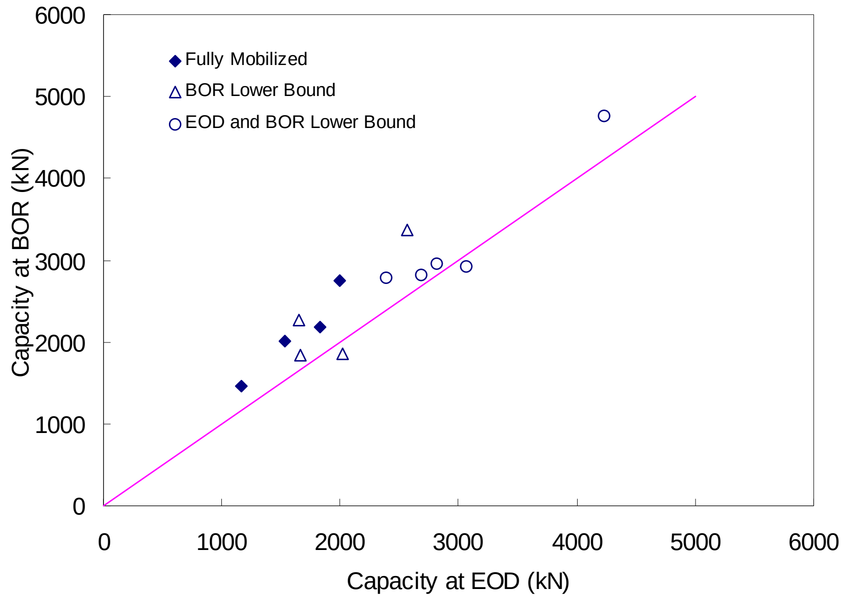


Figure 23. CAPWAP capacities at end of initial driving (EOD) and beginning of restrike (BOR).

Static Load Test Data

Static load tests were performed on 15 piles approximately 1 to 12 weeks after their installation. The test results are summarized in table 11. In general, two types of load deflection behavior were observed in the static load tests (figures 24 through 27).

Table 11. Summary of static load test data.

Test Pile Name	Time After Pile Installation (days)	Maximum Applied Load (kN)	Maximum Pile Head Displacement (cm)
ET2-C2	13	3,122	1.7
ET4-3B	20	3,558	2.4
375	15	3,447	1.6
923	33	3,447	2.4
I90 EB SA	23	3,781	1.6
14	6	3,105	2.2
12A1-1	30	1,512	1.4
12A2-1	24	1,014	0.5
16A1-1	17	3,612	2.6
I2	6	3,558	1.7
3	9	3,959	2.4
7	10	3,167	2.0
IPE	84	2,384	1.3
IPW	10	2,891	4.1
NS-SN	30	2,535	1.3

Test pile 12A1-1 (figure 24) represents a condition where the axial deflection of the pile is less than the theoretical elastic compression (assuming zero shaft friction). This pile was loaded to

1,557 kN in five steps and at no point during the loading did the deflection exceed the estimated elastic compression of the pile. This behavior is attributed to shaft friction, which reduces the compressive forces in the pile and limits the settlement. The significant contribution of shaft friction is also apparent in the load distribution curve shown in figure 25, which shows the load in the pile decreasing with depth. This behavior is typical of test piles ET2-C2, ET4-3B, I90-EB-SA, 12A1-1, 12A2-1, I2, and 3.

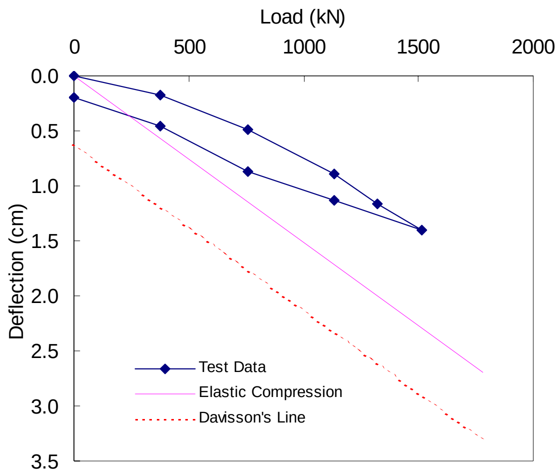


Figure 24. Deflection of pile head during static load testing of pile 12A1-1.

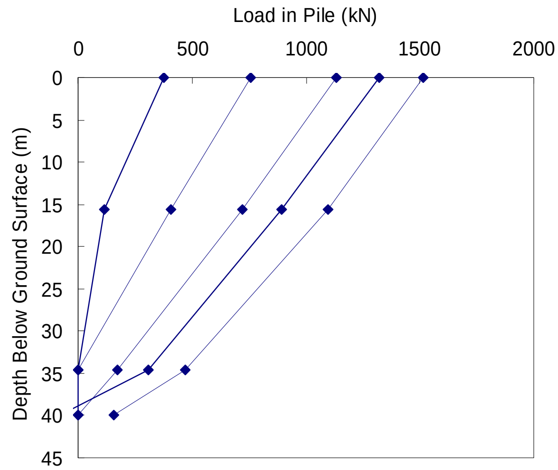


Figure 25. Distribution of load in pile 12A1-1.

Test pile 14 (figure 26) represents a condition where the axial deflection is approximately equal to the theoretical elastic compression. This suggests that more of the applied loads are being distributed to the toe of the pile with less relative contribution of shaft friction. This is apparent in figure 27, which shows negligible changes in the load within the pile with depth. This behavior is typical of test piles 375, 923, 14, 16A1-1, 7, IPE, and IPW.

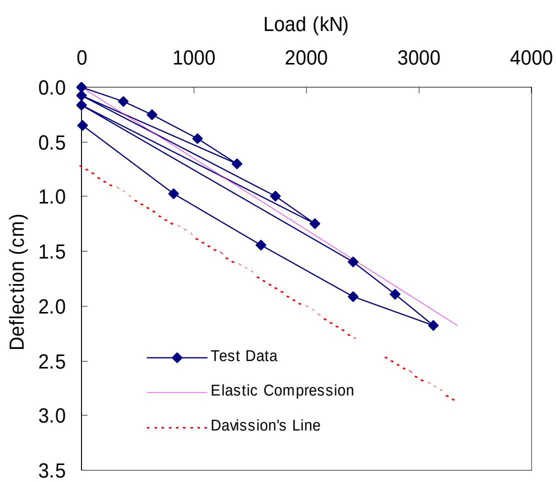


Figure 26. Deflection of pile head during static load testing of pile 14.

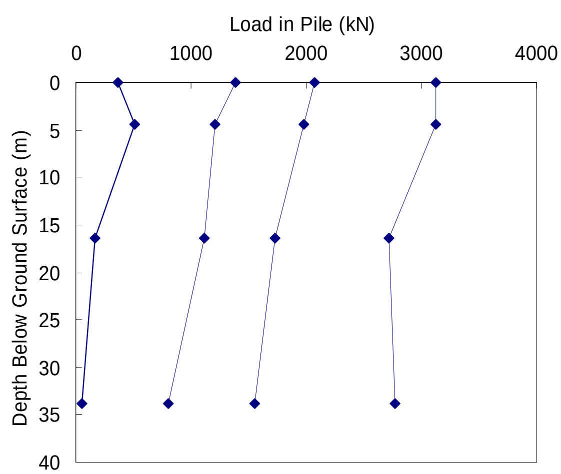


Figure 27. Distribution of load in pile 14.

Of the 15 static load tests, only one test pile (IPW) was loaded to failure according to Davisson's criteria. These data are shown in figures 28 and 29. This pile showed a significant increase in the deflection at approximately 2,580 kN, subsequently crossing the Davisson's line at approximately 2,670 kN at a displacement of around 2.5 cm. The telltale data obtained near the toe of the pile indicated that the pile failed in plunging.

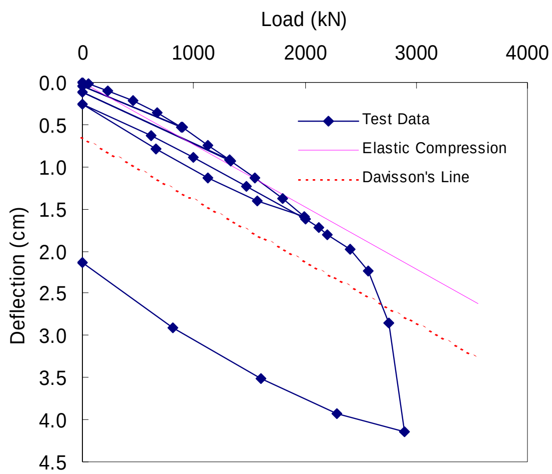


Figure 28. Deflection of pile head during static load testing of pile IPW.

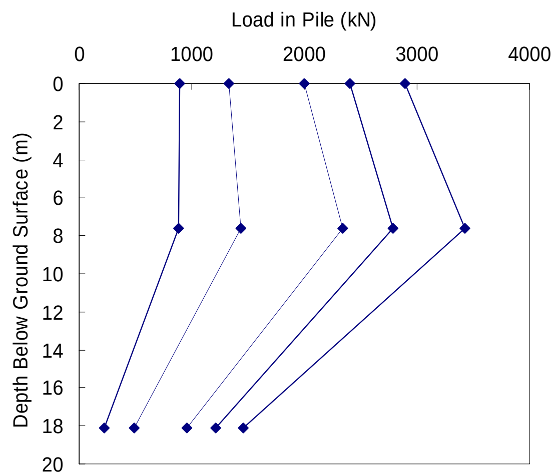


Figure 29. Distribution of load in pile IPW.

All test piles achieved the required ultimate capacities in the static load tests. The required ultimate capacities were determined by multiplying the allowable design capacity by a factor of safety of at least 2.0, as specified in the project specifications. A slightly higher factor of safety of 2.25 was used in contract C19B1. Three of the 15 static tests did not demonstrate that 100 percent of the design load was transferred to the bearing soils. Two of the piles (12A1-1 and 12A2-1) could not transfer the load to the bearing soils because of the high skin friction (figures 24 and 25). Test pile I2 could not demonstrate load transfer because the bottom telltale was not functioning.

Comparison of Dynamic and Static Load Test Data

The capacities determined by CAPWAP and from the static load tests are summarized in table 12, along with the required ultimate capacities. Of the 15 test piles, only one pile (IPW) was loaded to failure in a static load test. Likewise, only four BOR CAPWAP analyses and eight EOD CAPWAP analyses mobilized the full soil resistance. This means that the true ultimate capacity of the majority of the piles tested was not reached, and this makes a comparison of static load test and CAPWAP results difficult.

Test pile IPW was brought to failure in the static load test. Coincidentally, it is anticipated that the CAPWAP capacities for this pile also represent the fully mobilized soil resistance because of the relatively low blow counts (i.e., < 10) observed during driving. Based on a comparison of all data for pile IPW, its capacity increased by approximately 35 percent soon after installation, yielding a factor of safety of approximately 3.0. Note that this pile was preaugered to a depth of approximately half of the embedment depth. The capacity of 2,669 kN determined in the static load test is slightly less than the restrike capacity of 2,758 kN. However, this difference is partly attributed to modifications that were made to the pile after the dynamic testing, but prior to static testing. These modifications included removal of 0.6 m of overburden at the pile location and filling of the steel pipe pile with concrete, both of which would decrease the capacity of the pile measured in the static load test.

Table 12. Summary of dynamic and static load test data.

Test Pile Name	Required Allowable Capacity (kN)	Required Minimum Factor of Safety	Required Ultimate Capacity (kN)	CAPWAP Ultimate Capacity[1] (kN)		Ultimate Capacity From Static Load Test (kN)
				EOD	BOR	
ET2-C2	1,379	2.00	2,758	NI[2]	(3,247)	(3,122)
ET4-3B	1,379	2.00	2,758	NI	(3,719)	(3,558)
375	1,379	2.00	2,758	(4,226)	(4,759)	(3,447)
923	1,379	2.00	2,758	2,571	(3,372)	(3,447)
I90 EB SA	1,379	2.00	2,758	1,646	(2,268)	(3,781)
14	1,379	2.00	2,758	(2,687)	(2,820)	(3,105)
12A1-1	756	2.00	1,512	1,664	(1,846)	(1,512)
12A2-1	507	2.00	1,014	1,161	1,454	(1,014)
16A1-1	1,245	2.00	2,491	2,015	(1,859)	(3,612)
I2	1,245	2.00	2,491	1,530	2,015	(3,558)
3	1,583	2.00	3,167	(3,069)	(2,918)	(3,959)
7	1,583	2.00	3,167	(2,820)	(2,962)	(3,167)
IPE	890	2.25	2,002	1,824	2,180	(2,384)
IPW	890	2.25	2,002	2,002	2,758	2,669
NS-SN	1,112	2.25	2,504	(2,389)	(2,793)	(2,535)

Notes:

1. Capacities shown in parenthesis denote values that are conservative (dynamic load tests) or where failure was not achieved (static load tests).
2. NI = Data not identified.

CHAPTER 5. COST DATA OF DRIVEN PILES

This chapter presents a summary of the costs associated with pile driving operations on the CA/T project. The costs presented in this report were obtained directly from the contractor and represent the contractor's bid estimates identified in the individual contracts. The primary purpose of the cost data is to document the approximate cost of pile driving on the CA/T project; however, the data may also be useful to design engineers for planning purposes.

The contractor's bid costs for pile driving are summarized in table 13 by pile type. Unless noted, the costs in table 13 do not include costs for preaugering or costs associated with the mobilization or demobilization of the contractor's equipment. Steel pipe piles had the highest unit costs, ranging from $213 per meter for the 81.3-cm pile to $819 for the 154.9-cm pile. Unit costs for the PPC piles were lower, ranging from $72 to $197 per meter for the 30-cm PPC piles and $95 to $262 per meter for the 41-cm piles. As one would expect, the unit costs tended to decrease with the increasing size of the contract. The contractor's bid costs for preaugering are summarized in table 14. Preaugering was not performed in contract C07D1, and preaugering costs were not identified in the contract C07D2 bid. As shown in table 14, the additional cost of preaugering ranged from $33 to $49 per meter.

Table 13. Summary of contractor's bid costs for pile driving.

Contract	Pile Type	Estimated Length of Pile Installed (m)	Estimated Cost of Installation	Cost per meter of Pile[1]
C19B1	32-cm concrete-filled steel pipe	550	$1,183,650	$213.19
C09A4	41-cm concrete-filled steel pipe	5,578	$1,647,000	$295.27[2]
C19B1	61-cm concrete-filled steel pipe	296	$242,500	$819.26
C08A1	30-cm square PPC with stinger	792	$156,000	$196.97
C19B1	30-cm square PPC with stinger	2,177	$285,720	$131.24
C09A4	30-cm square PPC	3,658	$600,000	$164.02[2]
C07D2	30-cm square PPC with stinger	3,981	$289,510	$72.72
C07D1	30-cm square PPC with stinger	7,955	$652,500	$82.02
C19B1	41-cm square PPC with stinger	6,279	$824,000	$131.23
C08A1	41-cm square PPC with stinger	8,406	$2,206,400	$262.48
C09A4	41-cm square PPC with stinger	14,326	$3,290,000	$229.65[2]
C07D2	41-cm square PPC with stinger	19,879	$2,396,800	$120.57
C07D1	41-cm square PPC with stinger	32,918	$3,132,000	$95.15

Notes:

1. Unit costs include the costs of materials and labor for pile driving only. Preaugering is not included unless otherwise noted. See table 14 for preaugering unit costs. Mobilization and/or demobilization costs are not included.
2. Unit costs include the costs of preaugering.

Table 14. Summary of contractor's bid costs for preaugering.

Contract	Preaugering Depth Range (m)	Estimated Total Preaugering Depth (m)	Estimated Cost of Preaugering	Estimated Cost per meter
C08A1	0 to 30.5	2,134	$70,000	$32.80
C19B1	0 to 30.5	3,712	$182,655	$49.21

CHAPTER 6. LESSONS LEARNED

This chapter presents a summary of the lessons learned from driven piles on the CA/T project. The conclusions presented below are based on the evaluation of field records, project specifications, and pile load test data compiled from the project files. Five contracts were evaluated, including three located in East Boston/Logan Airport, one located in downtown Boston, and one located in Charlestown. Significant findings are summarized below:

- The dominant pile type used on the CA/T project was a 41-cm square PPC pile. Based on the contractor’s bid estimates, the PPC piles were also the most economical pile type.
- Pile heave in excess of the 1.3-cm criteria was identified on one cut-and-cover tunnel structure requiring 445 restrike events for the 576 piles used in the structure. The heave occurred even though preaugering of the marine clay layer was performed. Pile heave issues were not identified at other structures where the pile spacing was greater than about 1.8 m.
- Installation of displacement piles in contract C07D1 caused excessive movement of an adjacent structure. Despite the use of wick drains and partial preaugering, vertical displacement continued up to 8.8 cm. The wick drains were not effective in rapidly dissipating excess pore pressures from pile driving.
- The heave issues observed in contract C07D1 prompted the use of preaugering on subsequent contracts. Preaugering was performed over a portion, generally 30 to 70 percent, of the final pile embedment depth.
- Pile capacities evaluated using dynamic methods were conservative in hard driving conditions (i.e., penetration resistance greater than 10 blows per 2.5 cm) where the soil resistance may not be fully mobilized.
- Quake values from CAPWAP analyses ranged from 0.25 to 1.19 cm, with an average value of 0.64 cm. These values are higher than the values typically used in wave equation analyses; however, they are within the range of published values.
- Comparison of CAPWAP data evaluated at the end of initial driving and during restrike shows that the capacity of the piles increased over time by at least 20 percent from an increase in shaft resistance.
- Only 1 out of 15 piles tested in a static load test was brought to failure according to Davisson’s criteria, because the specifications did not specifically require that the pile be brought to failure.
- Three of the 15 piles did not successfully demonstrate that 100 percent of the design load was transferred to the bearing soils. Two piles did not meet the criteria because of high shaft friction, and the third did not meet the criteria because of a malfunctioning bottom telltale.
- Comparison of dynamic and static load test capacities was only possible on one pile (IPW), which reached the Davisson’s failure criteria in the static load test. CAPWAP and static capacities were in good agreement for this pile.

REFERENCES

1. Massachusetts Turnpike Authority (2000), Project Summary, http://www.bigdig.com/thtml/summary.htm.

2. Massachusetts Turnpike Authority (2000), Project Contract Lists, http://www.bigdig.com/thtml/contlist.htm.

3. Massachusetts Turnpike Authority (2000), Maps and Plans, http://www.bigdig.com/thtml/maps01.htm.

4. GZA GeoEnvironmental, Inc. (1991), *Central Artery (I-93)/Tunnel (I-90) Project, Geotechnical Data Report, South Bay Interchange, Design Sections D009B/D009C*, Boston, MA.

5. GZA GeoEnvironmental, Inc. (1992), *Central Artery (I-93)/Tunnel (I-90) Project, Geotechnical Data Report, South Bay Interchange, Design Section D009A*, Boston, MA.

6. Haley and Aldrich, Inc. (1991), *Final Geotechnical Data Report, Central Artery (I-93)/Tunnel (I-90) Project, Design Sections D007C and D007D (C07D2)*, Boston, MA.

7. Haley and Aldrich, Inc. (1996), *Final Geotechnical Report, Central Artery (I-93)/Tunnel (I-90) Project, Design Section D008A*, Boston, MA.

8. Maguire Group, Inc., and Frederic R. Harris, Inc. (1995), *Final Report on Soil Stabilization and Testing Program, Central Artery (I-93)/Tunnel (I-90) Project, D009A*, Boston, MA.

9. Maguire Group, Inc., and Frederic R. Harris, Inc. (1995), *Supplemental Geotechnical Data Report, Central Artery (I-93)/Tunnel (I-90) Project, Design Section D009A*, Boston, MA.

10. Stone and Webster, Inc. (1996), *Final Geotechnical Data Report, Central Artery (I-93)/Tunnel (I-90) Project, Design Section D019B, I-93 Viaducts and Ramps North of Charles River*, Boston, MA.

11. Barosh, P.J.; Kaye, C.A.; and Woodhouse, D. (1989), "Geology of the Boston Basin and Vicinity." *Civil Engineering Practice: Journal of the Boston Society of Civil Engineers*, 4(1), 39-52.

12. McGinn, A.J., and O'Rourke, T.D. (2003), *Performance of Deep Mixing Methods at Fort Point Channel*, Cornell University, Ithaca, NY.

13. Commonwealth of Massachusetts (1997), *The Massachusetts State Building Code, Users Guide to 780 CMR* (6th Edition), William F. Galvin, Boston, MA.

14. Massachusetts Highway Department (1996), *MHD Supplemental Specifications and CA/T Supplemental Specifications to Construction Details of the Standard Specifications for Highways and Bridges (Division II) for Central Artery (I-93)/Tunnel (I-90) Project in the City of Boston* [Section 940 covers driven piles], September 3, 1996.

15. Massachusetts Highway Department (1998), *Special Provisions to Construction Details for the Standard Specifications for Highways and Bridges and the Supplemental Specifications (Division II) for Central Artery/Tunnel Project I-93 Viaducts and Ramps North of Charles River (C19B1) in the City of Boston* [Section 940 covers driven piles], April 29, 1998.

16. AASHTO (2002), *Standard Specifications for Highway Bridges*, Washington, DC.

17. FHWA (1998), *Design and Construction of Driven Foundations*, Report No. FHWA-HI-97-013, Washington, DC.

18. Massachusetts Highway Department (1996), Pile Layout Plan: I-90 Logan Airport Interchange Arrivals Road, Tunnel, Contract C07D2, Drawing No. S-1162.

19. Geosciences Testing and Research, Inc. (1997), Letter to RDA Construction Regarding Dynamic Load Test Results, Arrivals Tunnel: Round 1, May 5, 1997.

20. Massachusetts Highway Department (1995), Interoffice Memorandum to File Regarding Hilton Heave, June 5, 1995.

21. Massachusetts Highway Department (1995), Interoffice Memorandum to Lauren Cragg Regarding Hilton Heave, June 28, 1995.

22. Massachusetts Highway Department (1995), Interoffice Memorandum to File Regarding Hilton Heave, August 3, 1995.

23. Massachusetts Highway Department (1995), Interoffice Memorandum to File Regarding Hilton Heave, August 23, 1995.

24. GZA GeoEnvironmental, Inc. (1995), Letter to FST/TYLI Regarding D007D (C07D1) I-90 Logan Airport Interchange, Contract No. 93096, East Approach to Ramp E-T Pile Driving, July 10, 1995.

25. Field Measurements, Inc. (2000), Letter to R.A. Francoeur Marine, Inc., Regarding Pile Load Test Program, Dynamic Load Test Results, CA/T Contract C08A1, August 24, 2000.

26. Geosciences Testing and Research, Inc. (1997), Letter to RDA Construction Regarding Arrivals Tunnel Test Program, Dynamic Test Results (Piles 258 and 375), C07D2, May 30, 1997.

27. Geosciences Testing and Research, Inc. (1997), Letter to RDA Construction Regarding Ramp 1AA Test Program, Dynamic Test Results (Piles 923, 955, and 999), C07D2, October 9, 1997.

28. Geosciences Testing and Research, Inc. (1998), Letter to P.A. Frisco Regarding Dynamic Pile Testing Results, NS-SN Transition Structure, CA/T C19B1 Project, October 7, 1998.

29. Geosciences Testing and Research, Inc. (2000), Letter to P.A. Frisco Regarding Dynamic Pile Testing Results, LT Wall Structure Indicator Piles, CA/T C19B1 Project, September 27, 2000.

30. Geosciences Testing and Research, Inc. (2001), Letter to Modern Continental Construction Regarding Dynamic Pile Testing Results, CT Wall Structure Indicator Pile IPW, CA/T C19B1 Project, March 23, 2001.

31. Geosciences Testing and Research, Inc. (2002), Letter to R.A. Francoeur Marine, Inc., Regarding Static Load Test Report for Pile 14, C08A1, 1ANB-A/D West Abutment, September 6, 2002.

32. GZA GeoEnvironmental, Inc. (1998), Letter to Slattery/Interbeton/J.F. White/Perini Regarding Dynamic Pile Testing Results, October 9, 1998.

33. GZA GeoEnvironmental, Inc. (1999), Letter to Slattery/Interbeton/J.F. White/Perini Regarding Dynamic Pile Testing Results, CA/T Contract C09A4, Approach No. 5, July 26, 1999.

34. Perini Corporation (1994), Letter of Transmittal Containing CAPWAP Results, Contract C07D1, May 2, 1994.

35. ASTM (1996a), "Standard Test Method for High-Strain Dynamic Testing of Piles," D4945-89, *1996 Annual Book of ASTM Standards*, ASTM International, West Conshohocken, PA.

36. ASTM (1996b), "Standard Test Method for Piles Under Static Axial Compression Load," D1143-81, *1996 Annual Book of ASTM Standards*, ASTM International, West Conshohocken, PA.

37. Field Measurements, Inc. (2000), Letter to R.A. Francoeur Marine, Inc., Regarding Pile Load Test Program, Compression Pile Load Test Results, Pile I90EB South, CA/T C08A1, September 19, 2000.

38. Geosciences Testing and Research, Inc. (1997), Letter to RDA Construction Regarding Static Load Test Report, Pile 375, C07D2 Arrivals Tunnel, East Boston, MA, June 10, 1997.

39. Geosciences Testing and Research, Inc. (1997), Letter to RDA Construction Regarding Static Load Test Report, Pile 923, C07D2 Ramp 1A-A, East Boston, MA, November 4, 1997.

40. Geosciences Testing and Research, Inc. (1998), Letter to P.A. Frisco, Inc., Regarding Static Load Test Report, Test Pile NS-SN Transition Bridge, C19B1, November 4, 1998.

41. Geosciences Testing and Research, Inc. (2000), Letter to Modern Continental Construction Regarding Static Load Test Results, CA/T C19B1, LT Retaining Wall, December 20, 2000.

42. Geosciences Testing and Research, Inc. (2001), Letter to Modern Continental Construction Regarding Static Load Test Results for Pile IPW, CA/T C19B1, CT Retaining Wall, April 3, 2001.

43. Geosciences Testing and Research, Inc. (2002), Letter to R.A. Francoeur Marine, Inc., Regarding Static Load Test Report for Pile 14, C08A1, 1ANB-A/D West Abutment, September 6, 2002.

44. GZA GeoEnvironmental, Inc. (1998), Letter to Slattery/Interbeton/J.F. White/Perini Regarding Compression Load Test Pile, Utility Foundations (Test Pile 3), July 13, 1998.

45. GZA GeoEnvironmental, Inc. (1998), Letter to Slattery/Interbeton/J.F. White/Perini Regarding Compression Load Test Pile, Utility Foundations (Test Pile 7), July 26, 1998.

46. GZA GeoEnvironmental, Inc. (1998), Letter to Slattery/Interbeton/J.F. White/Perini Regarding Compression Load Test Pile, Approach 1 (Pile 12A1-1), November 9, 1998.

47. GZA GeoEnvironmental, Inc. (1998), Letter to Slattery/Interbeton/J.F. White/Perini Regarding Compression Load Test Pile, Approach 2 (Pile 12A2-1), November 2, 1998.

48. GZA GeoEnvironmental, Inc. (1999), Letter to Slattery/Interbeton/J.F. White/Perini Regarding Compression Load Test Pile, Approach 5 (Indicator Pile 2), August 11, 1999.

49. McPhail Associates, Inc. (1994), *Pile Load Test Report, Pier ET2, Logan Airport Egress Ramps, Contract C07D1, East Boston, Massachusetts*.

50. McPhail Associates, Inc. (1994), *Pile Load Test Report, Pier ET4, Logan Airport Egress Ramps, Contract C07D1, East Boston, Massachusetts*.

51. Geosciences Testing and Research, Inc. (1997), Letter to RDA Construction Regarding Static Load Test Frame/Instrumentation, Contract C07D2, January 13, 1997.

52. Davisson, M.T. (1972), "High-Capacity Piles." *Proceedings of Lecture Series on Innovations in Foundation Construction*, Chicago, IL, 81-112.

53. Rausche, M.F.; Goble, G.G.; and Likins, G. (1985), "Dynamic Determination of Pile Capacity." *Journal of Geotechnical Engineering*, 111(3), 367-383.

54. Fellenius, B.H.; Riker, R.E.; O'Brien, A.J.; and Tracy, G.R. (1989), "Dynamic and Static Testing in Soil Exhibiting Set-Up," *Journal of Geotechnical Engineering*, 115(7).

55. Authier, J., and Fellenius, B.H. (1980), "Quake Values Determined From Dynamic Measurements," *Proceedings of the First International Conference on the Application of Stress-Wave Theory to Piles*, Stockholm, Sweden, 197-216.

56. Likins, G.E. (1983), "Pile Installation Difficulties in Soils With Large Quakes," *Symposium on Dynamic Measurements of Piles and Piers*, Philadelphia, PA, May 1983.

57. Liang, R.Y., and Zhou, J. (1997), "Probability Method Applied to Dynamic Pile-Driving Control," *Journal of Geotechnical and Geoenvironmental Engineering*, 123(2), 137-144.