**R E P O R T**

# GEOTECHNICAL DESIGN REPORT FOR BRIDGE NO. C-07-001 (437)

## Bridge Street over Mitchell River
## Chatham, Massachusetts

Prepared for

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## Introduction

The purpose of this report is to provide geotechnical recommendations for the design of the foundations for support of Bridge C-07-001(437) with a movable bascule span carrying Bridge Street over Mitchell River.

The recommended bridge alternative consists of completely demolishing and replacing the existing bridge superstructure and substructures. The new bridge will consist of three 26'-11" approach spans to the west followed by a 15'-6" bascule pier, 33'-9" lift span and two additional 32'-6" approach spans to the east. The proposed bridge replacement will span 195'-0" total from centerline to centerline of bearings on its abutments. The existing profile will be raised in order to provide additional freeboard for a 10 year tidal flood.

The bascule span will be supported on a fully enclosed reinforced concrete bascule pier, which will include walls around the perimeter of the pier, pedestals to support the operating machinery, platforms to provide maintenance access to the equipment, and a footing. The rest pier for the bascule leaf span will be a fully enclosed reinforced concrete type pier, and the piers for the approach spans will be open type pile bents.

## Subsurface Conditions

Five test borings have been drilled for the purpose of this report. Sandy fill, river bottom material, medium dense sand, and very stiff clay underlain by dense to very dense sand layers are present across the site. The groundwater levels in the boreholes are observed to correspond approximately to the water levels during low and high tide conditions in the Mitchell River, which varied between elevations -2.4 feet and 1.6 feet (NAVD 88) during the mean low and mean high water, respectively.

Recommended values of geotechnical parameters for analysis and design of abutments, wingwalls and retaining walls are presented in the following table:

| | |
|---|---|
| Total Unit Weight of Embankment Fill | 125 pcf |
| Retained Soil Internal Friction Angle | 32 degrees |
| Active Earth Pressure Coefficient, $K_a$ | 0.31 |
| "At-Rest" Earth Pressure Coefficient, $K_o$ | 0.47 |
| Passive Earth Pressure Coefficient, $K_p$ | 3.25 |
| Dynamic Earth Pressure Coefficient, $K_{ae}$ | 0.34 |
| Interface Friction Angle – Concrete with Backfill or Foundation Soil | 20 degrees |

The earth pressure coefficients do not include consideration of hydrostatic pressures, and assume that the walls are properly back-drained to prevent build-up of hydrostatic pressures.

For seismic design, the subsurface condition at this site can be classified as Site Class D for stiff soils as determined using SPT N-values (AASHTO Interim 2008 Article 3.10.3). In addition, based on density of the sand layers below the water table, liquefaction of soils at the proposed bridge location will generally not be a concern. Bridge replacement for bridge C-07-001 should be categorized as Seismic Design Category A (SDC A) as per the AASHTO Guide Specifications for LRFD Seismic Bridge Design and subsequently should require minimal seismic design and detailing.

Based on the results of a scour analysis and consideration of the mud line elevations at different substructure locations the recommended maximum scour elevations can be summarized as follows:

| | |
|---|---|
| Abutments | El. -10 feet |
| Approach Piers including Rest Pier | El. -15 feet |
| Bascule Span Pier | El. -21 feet |

**Recommended Foundation System**

Considering the subsurface conditions at the project site and that the approach spans are proposed to be supported on open bent type piers, the most suitable deep foundation for the replacement bridge consists of closed-end concrete filled steel pipe piles. The pile capacity will be derived mostly from soil frictional resistance in the medium dense sand and very stiff clay below the river bottom deposit. In order to obtain the required pile capacities the minimum required pile tip elevation is estimated to range from about Elevation -70 feet to -75 feet. However, the final pile tip elevations to obtain the design factored resistance should be based on the driving criteria established based on the results of dynamic testing (ASTM D4945) performed using the Pile Driving Analyzer (PDA).

Derived soil resistance within the potential scour depth should be ignored when calculating the design factored resistance to ensure that design load can safely be supported below the maximum probable scour depth. However, the ignored soil resistance should still be included when calculating the nominal resistance to be obtained during the dynamic testing. Considering that obstructions may be within the fill layer, to reduce vibrations in locations close to existing structures and subsurface utilities, and to reduce abrasion of pile coating; it is recommended that the piles be driven within pre-augured holes. The bottoms of the pre-augured holes should be at the same elevation as the maximum scour depth. It is recommended that each pile tip be reinforced with a drive point, or shoes to prevent twisting, buckling, or tearing of the pile section.

It is recommended that the steel pipe piles conform to ASTM A252, Grade 3 (45 ksi minimum yield strength), and be coated with a minimum 16 mils application of fusion bonded epoxy that extends up to the maximum scour elevation. In addition, considering the anticipated driving stresses on the piles and to provide a secondary corrosion protection, the recommended minimum shell thickness of the piles should be 0.5 inch.

Based on the anticipated loads and the required structural resistances, it appears that the larger pipe piles with a 16-inche nominal diameter will be selected. Therefore, a summary of the estimated minimum tip elevation, axial compression and uplift resistances are provided in the following table for a 16-inch nominal diameter closed-end concrete filled steel pipe piles.

| Pile Location | Estimated Minimum Tip Elevation (feet) | Total Factored Axial Compression Resistance for Strength Limit State (LRFD Strength 1) (kips) | Nominal Axial Compression Resistance During Dynamic Testing (kips) | Total Factored Uplift Resistance for Strength Limit State (LRFD Strength 1) (kips) |
|---|---|---|---|---|
| West Abutment | -70 | 160 | 265 | 85 |
| West Approach Piers | -70 | 140 | 220 | 70 |
| Bascule / Rest Pier | -70 | 130 | 205 | 65 |
| | -75 | 140 | 220 | 70 |
| East Approach Piers | -70 | 130 | 205 | 65 |
| East Abutment | -70 | 140 | 235 | 70 |

**Construction Considerations**

In accordance with AASHTO LRFD (Table 10.5.5.2.3-1), at least two dynamic tests (ASTM D4945) should be performed at every bridge substructure location (abutments and piers) in order to establish the final driving criteria corresponding to the required nominal pile resistance. As per ASHTO LRFD Table 10.5.5.2.3-1, a resistance factor of 0.65 should be used to estimate the factored pile resistance. Wave equation analyses should be performed to evaluate preliminary driving criteria and define requirements for the size of the pile hammer to drive the piles to their minimum tip elevation.

It is anticipated that cofferdams will be required for construction of the abutments, the bascule pier and the rest pier. Existing piles should be cut-off a minimum of 2 feet below the mud line or completely removed where required to avoid interference with the proposed pile foundations.

## 2.1 SCOPE

The purpose of this report is to provide geotechnical recommendations for the design of the foundations for support of Bridge C-07-001 (437) carrying Bridge Street over Mitchell River.

## 2.2 PROJECT BACKGROUND AND PROPOSED CONSTRUCTION

The project consists of complete replacement of Bridge Number C-07-001 (437) that carries Bridge Street over the Mitchell River in the town of Chatham. The bridge is located on Bridge Street, between Stage Harbor Road and the intersection of Main Street and Morris Island Road (see the locus plan in Appendix A). Elevations in this report are referenced to the North American Vertical Datum of 1988 (NAVD 88).

### 2.2.1 Existing Structure

Originally constructed in the mid 1800's, the bridge has been reconstructed several times, most recently in 1980. In a letter dated January 12, 2010 the Massachusetts Historic Commission determined that the existing structure does not meet the criteria for the National Register of Historic Places as it is less then 50 years of age.

The bridge has a curb-to-curb width of 24'-0" and carries one traffic lane in each direction. For spans two through seven and nine through eleven there are 6'-9" wide sidewalks on both sides of the bridge and 14" x 8" timbers separating the sidewalk from the roadway on both sides. The total out-to-out width of the bridge within these spans is 37'-6". For spans one, eight and twelve there are 3'-9" wide sidewalks on both sides of the bridge and 14" x 8" timbers separating the sidewalk from the roadway on both sides. The total out-to-out width of the bridge within these spans is 31'-6". The superstructure consists of a twelve span timber trestle structure including span eight, the bascule type lift span. The decking is composed of 4" timber decking with a 3" timber wearing surface. The overall length of the superstructure is 192', with a bascule span length of 23'-0". The bridge currently provides a navigable channel width of 19'-4" and an unlimited vertical clearance with the bascule span in the open position and approximately 7'-4" of vertical clearance with the bascule span in the closed position. The substructure consists of concrete abutments with timber piles and timber pier caps founded on timber piles.

The bascule span is elevated by lift hoists that are located in the sidewalks on the east end of the span and an electrical control cabinet is located on the northwest side of the bridge. The bridge is equipped with electrically operated wood frame safety gates and traffic signals on both approaches.

The existing bridge has a 3" diameter electrical conduit mounted to the north side of the superstructure used for bridge operation. At the bascule span, the conduit is submerged in the water and runs along the channel bottom. There are no overhead utility poles across the bridge.

### 2.2.2 Description of Proposed Construction

The recommended bridge alternative consists of completely demolishing and replacing the existing bridge superstructure and substructure. The new bridge will consist of three 26'-11" approach spans to the west followed by a 15'-6" bascule pier, 33'-9" lift span and two additional 32'-6" approach spans to the east. The proposed bridge replacement will span 195'-0" total from centerline to centerline of bearings on its abutments. The existing profile will be raised in order to provide additional freeboard for a 10 year tidal flood.

The approach span superstructure will consist of multiple glulam timber beams with a 5 ¼" glulam panel structural deck and a 3" timber wearing surface. The bascule single leaf superstructure will consist of a 3" timber wearing surface mounted on a custom 8" deep steel open grid deck. The roadway deck will be supported on a steel framing system comprised of stringers, floorbeams, and two variable depth main girders.

The approach span superstructure will be 45'-2 ½" wide out-to-out with 26'-0" clear roadway curb-to-curb. The approach spans will have 7'-0" wide sidewalks on each side of the roadway. The roadway width of the bascule span will match that of the approach spans. However, the sidewalk widths on the bascule span will be 5'-0" wide. The traffic railings separating roadway from sidewalks will consists of crash-tested (TL-2) glulam timber railing with curb. The bascule span will provide a minimum horizontal clearance of 25'-0" between fenders, a minimum vertical clearance of 7'-2" above mean high water with the leaf in the lowered position and unlimited vertical clearance with the leaf in the fully raised position.

The bascule span will be supported on a fully enclosed reinforced concrete bascule pier, which will include walls around the perimeter of the pier, pedestals to support the operating machinery, platforms to provide maintenance access to the equipment, and a footing. The rest pier for the bascule leaf span will be a fully enclosed reinforced concrete type pier, and the piers for the approach spans will be open type pile bents.

## 2.3 REPORT ORGANIZATION

This report is divided into six sections and prepared in general accordance with Section 2.5.2 of the MassDOT Bridge Manual. The Executive Summary is the first section. Following this introductory Section 2, a description of the subsurface conditions, design soil parameters and earthquake considerations are presented in Section 3. Our engineering evaluations and recommended foundation system are presented in Section 4, and construction considerations are presented in Section 5. Finally, the limitations of this study are described in Section 6.

Figures, boring logs, and calculations are included in appendices.

## 3.1 LOCAL GEOLOGY

The geologic history of Cape Cod, where the town of Chatham is located, mostly involves the advance and retreat of the last continental ice sheet and the rise in sea level that followed within the last 25,000 years. During their retreat, the glaciers deposited soil and rock debris called glacial deposits or drift, which filled the bedrock basin in the area. On Cape Cod, the bedrock is buried by glacial deposits ranging in thickness from at least 200 to more than 600 feet. The surficial geology at the bridge location generally consists of thick outwash deposits from melting glaciers, which form stratified drift of various soil particle sizes. Sand and gravel are sorted and stratified by meltwater flowing in streams and draining the glacier. The clay and silt-sized particles are carried by the meltwater streams into relatively calm waters in a glacio-lacustrine (glacial lakes) or glacio-marine (the sea) environment, where they settle out according to the particle sizes. Over several re-advancements and retreats of these ice fronts, the deposits formed the subsurface profile encountered at the bridge site. Finally, more recent saltwater organic sediments and alluvial soils were deposited on top of the glacial outwash, in tidal marshes or estuary and modern day floodplains of major rivers and streams.

## 3.2 SUBSURFACE INVESTIGATION PROGRAM

The subsurface investigation program for this bridge consists of nine borings (denoted as BB-1 through BB-9 on the boring plan in Appendix B), five of which being primary borings (BB-1 through BB-5) and four being complimentary borings (BB-6 through BB-9). Two of the primary borings were located on the embankment behind the existing bridge abutments, with the additional three borings located in the riverbed near the proposed new bridge piers. These five borings were completed under the direction of URS Corporation in April of 2010. Two of the complimentary borings will be located on the embankment with the additional two borings being located currently in the riverbed. The four complementary borings have not been performed as of this report preparation, and will not be performed unless deemed necessary. Additional understanding of the subsurface conditions was obtained from the available reconstruction plans of the existing bridge. The plans, dated May 1980, indicate that four borings were performed prior to construction of the existing bridge. The borings were located at the east abutment as well as bents 2, 4 and 6.

Standard penetration tests (SPT), consisting of a 140-pound hammer dropping 30 inches on a standard 2-inch diameter (OD) split-spoon sampler, were used to establish the soil consistency and collect soil samples at regular intervals. The SPTs were typically performed at five foot intervals of depth, except at BB-2 and BB-3 where SPTs were performed at ten foot intervals after depths of 50 and 60 feet, respectively. Bedrock was not encountered in any of the borings. The original borings for the existing bridge indicate what appeared to be hard or very dense materials at the bottom of each bore hole, identified as “ledge or boulder”. However, our current investigation did not encounter this condition and determine it as part

of a different soil layering at the site. All drilling and sampling operations associated with these borings were performed in the presence of URS's inspector, Mr. Aleksandar Marinkovic. In addition, the soil samples were visually classified by the URS field inspector using the Unified Soil Classification System (USCS) and placed in labeled sample jars. The borings were performed by New Hampshire Boring Inc., of Londonderry, New Hampshire. The boring logs are presented in Appendix D.

Laboratory tests were also performed on selected representative soil samples obtained from the borings for the purpose of confirming the visual field soil classification, and to assist in engineering evaluations. These tests consisted of three grain size analyses (sieve analyses); the results are included in Appendix D.

## 3.3 GENERALIZED SUBSURFACE PROFILES

The subsurface profile below the embankment fill and river bed consists of a loose/soft layer of marsh and river bottom material containing mixtures of fine sand and silt with traces of organics, shells and gravel. This layer represents the natural top soil before the current bridge was built. This layer is underlain by thick inter-bedded layers of medium dense to very dense sand, and stiff to hard clay and silt. The consistencies of these layers generally increase with depth. A generalized subsurface profile was developed longitudinally in the direction of the bridge as shown in Appendix C. The subsurface conditions can generally be described from the ground surface to the limiting depth of the borings as follows:

Fill

The upper 8 to 10 feet of soils behind the existing bridge abutments consist of sandy fill likely placed during construction of the approach roadways and existing bridge abutments. This fill encountered in the borings consisted of loose fine to coarse sand (i.e. SPT N-values of 2 blows per foot) with varying amounts of gravel. It is assumed that this low consistency is not reflective of the embankment consistency in general, but may be due to the drilling action very close to the abutment walls.

River Bottom Sediment

This material primarily comprise of water-sorted fine sands and silts that were deposited in river or marsh tidal estuaries on top of the older alluvium and glacial outwash. It was encountered as about a 4 to 10 foot thick layer below fill in the abutment borings, and at the mudline in the river borings. This deposit consists of loose or soft fine sand and silts with traces of organics, shells and gravel. N-values ranged from 2 to 12 blows per foot.

Medium Dense Sand

Medium dense sandy soil layers between 6 to 21 feet thick were encountered beneath the marsh and river bottom material, down to approximately El.-50 feet. The composition of this material ranged from silty fine sand to fine to coarse sand, and was brown to grey in color. The SPT N-values determined in these layers ranged between 6 and 66 blows per foot, with the majority of the N-values ranging from 10 to 30 blows per foot.

Very Stiff Clay and Silt

Very stiff gray clay and silt layers were also encountered beneath the marsh and river bottom material, and inter-bedded with the sand layers down to approximately El.-50 feet as shown in the surface profile in Appendix C. Where encountered, the thickness of these layers ranges from about 4 to 13 feet. N-values varied from 9 to 45 blows per foot, with the majority over 15 blows per foot.

Dense to Very Dense Sand

A relatively thick gray fine sand layer with some traces of inorganic silt was encountered below approximately El.-50 feet. The top 5 to 10 feet of this layer appeared to have a medium dense consistency, as well as some zones within the layer. However, the general consistency of this layer is regarded as dense with SPT N-values in the range of 25 to 108 blows per foot. Below approximately El.-120 feet, a discontinuous layer of very dense fine to coarse sand was encountered in the deepest boring performed to date (Boring BB-3).

## 3.4 GROUNDWATER CONDITIONS

The Mitchell River at the project site is a tidal estuary that flows to the south on an ebbing tide. The existing channel is lined with rip-rap and rock fill in the vicinity of the bridge and surrounded by salt marshes and low rolling hills. The elevation of the channel water varies from -2.4± feet at mean low tide, and +1.6± feet at mean high tide (see Appendix C figure). Groundwater was measured in the abutment borings BB-1 and BB-2 at approximately 7 and 5.4 feet below the roadway surface, respectively. These measurements correspond with a high tide condition. At the time of drilling BB-5 in the river, the water elevation measured in the boring was 13 feet below the roadway surface, corresponding to a low tide condition.

It should be noted that the groundwater measurements were not taken over an extended period of time; therefore, they do not necessarily reflect potential seasonal or tidal variations in the groundwater level.

## 3.5 DESIGN PARAMETERS FOR SOILS

Recommended values of geotechnical parameters for analysis and design of abutments, wingwalls and retaining walls are presented in the following table:

| | |
|---|---|
| Total Unit Weight of Embankment Fill | 125 pcf |
| Retained Soil Internal Friction Angle | 32 degrees |
| Active Earth Pressure Coefficient, $K_a$ | 0.31 |
| "At-Rest" Earth Pressure Coefficient, $K_o$ | 0.47 |
| Passive Earth Pressure Coefficient, $K_p$ | 3.25 |
| Dynamic Earth Pressure Coefficient, $K_{ae}$ | 0.34 |
| Interface Friction Angle – Concrete with Backfill or Foundation Soil | 20 degrees |

These parameters are based on using gravel borrow for foundation and retaining wall backfill as required in the MassDOT Standard Specifications For Highways and Bridges (Standard Specifications).

The earth pressure coefficients do not include consideration of hydrostatic pressures, and assume that the walls are properly back-drained to prevent build-up of hydrostatic pressures. The surcharge thrust load from traffic on the walls should be computed by multiplying the vertical surcharge pressure by $K_a$ and the height of the wall; the live load surcharge should be applied at mid-height of the wall. If the walls or abutments are prevented from deflecting freely at their crest, the computed backfill earth and surcharge loads, should utilize the coefficient of earth pressure at-rest $K_o$ instead of $K_a$ acting horizontally. The dynamic earth pressure coefficient was derived using the Mononobe-Okabe pseudostatic method for unrestrained structures, with a horizontal peak ground acceleration coefficient of 0.06 in accordance with AASHTO's seismic hazard map for the bridge site. The resultant dynamic pressure should be applied uniformly over the height of the wall.

## 3.6 SEISMIC DESIGN PARAMETERS AND LIQUEFACTION POTENTAIL

For seismic design, the subsurface condition at this site can be classified as Site Class D for stiff soil as determined using SPT N-values (AASHTO Interim 2008 Article 3.10.3). The average SPT N-value in each boring, for the upper 100 feet of the soil profile, ranges between 15 and 50 blows per foot. In addition, based on density of the sand layers below the water table, liquefaction of soils at the proposed bridge location will generally not be a concern.

Bridge replacement for bridge C-07-001 should be categorized as Seismic Design Category A (SDC A) per the AASHTO Guide Specifications for LRFD Seismic Bridge Design and subsequently should require minimal seismic design and detailing.

Seismic Design and Analysis of the structure should be in accordance with AASHTO Criteria and the MHD 2009 LRFD Bridge Manual including the latest updates.

## 3.7 RESULTS OF SCOUR ANALYSES

Based on the results of a scour analysis as presented in the Hydraulic Study Report, dated March 5, 2010, prepared by Massachusetts Highway Department for this bridge, the maximum scour depth during the 100-year tidal flood event could be up to 8.6 feet at the proposed bridge abutments, up to 4.0 feet at the approach span pier locations, and up to 11.5 feet at the proposed bascule span pier. Considering the mud line elevations at different substructure locations the recommended maximum scour elevations can be summarized as follows:

| | |
|---|---|
| Abutments | El. -10 feet |
| Approach Piers including Rest Pier | El. -15 feet |
| Bascule Span Pier | El. -21 feet |

## 4.1 INTRODUCTION

The recommendations for foundation design presented herein are based on our current understanding of the project design requirements, the foundation system considered for the abutments and piers, and subsurface information from the subsurface investigation program perform to-date. Because of the presence of unsuitable river bottom deposits and scour potential, deep pile foundations will be required to support the new replacement bridge abutments and piers. Use of spread footings bearing on the natural sand or clay layer will require cost-prohibitive deep excavations below groundwater. The deep foundations will derive their bearing capacity mostly from the frictional resistance in the natural sand, silt, and clay layers. Piles are anticipated to be driven well into the dense sand deposit below Elevation -55 feet. Specific recommendations for foundation design of the abutments and piers are provided below.

## 4.2 DEEP FOUNDATIONS

### 4.2.1 General

Considering the subsurface conditions at the project site and that the approach spans are proposed to be supported on open bent type piers, the most suitable deep foundation for the replacement bridge consists of closed-end concrete filled steel pipe piles. The pile capacity will be derived mostly from soil frictional resistance in the medium dense sand and very stiff clay below the river bottom deposit. As indicated in the following section, in order to obtain the required pile capacities the minimum required pile tip elevation is estimated to range from about Elevation -70 feet to -75 feet. However, the final pile tip elevations to obtain the design factored resistance should be based on the driving criteria established based on the results of dynamic testing (ASTM D4945) performed using the Pile Driving Analyzer (PDA).

Derived soil resistance within the potential scour depth should be ignored when calculating the design factored resistance to ensure that design load can safely be supported below the maximum probable scour depth. However, the ignored soil resistance should still be included when calculating the nominal resistance to be obtained during the dynamic testing. Considering that obstructions may be within the fill layer, to reduce vibrations in locations close to existing structures and subsurface utilities, and to reduce abrasion of pile coating; it is recommended that the piles be driven within pre-augured holes. The bottoms of the pre-augured holes should be at the same elevation as the maximum scour depth. It is recommended that each pile tip be reinforced with a drive point, or shoes to prevent twisting, buckling, or tearing of the pile section.

Due to the potentially corrosive marine environment, corrosion protection should be considered in the pile design, especially for the exposed portion of the pile and the portion embedded in the fill and river bottom deposit. Corrosion protection of piles driven into undisturbed natural soils is not normally necessary. Therefore, it is recommended that the steel pipe piles conform to ASTM A252, Grade 3 (45 ksi minimum yield strength), and be coated with a minimum 16 mils application of fusion bonded epoxy that extends up to the maximum scour elevation. In addition,

considering the anticipated driving stresses on the piles and to provide a secondary corrosion protection, the recommended minimum shell thickness of the piles should be 0.5 inch.

In the following sections, pile capacity calculations for 14 inches and 16 inches nominal diameter closed-end concrete filled steel pipe piles are presented. However, based on the anticipated loads and the required structural resistances, it appears that the larger pipe piles will be selected.

### 4.2.2 Axial Capacity of Piles

The pile nominal/factored capacity is governed by either the pile structural resistance, or geotechnical resistance of the soils supporting the pile. The structural resistance of fully embedded piles and piles extended above the ground surface can be calculated in accordance with Section 10.7.3.13 of AASHTO LRFD. However, in this section the geotechnical axial capacity of the considered pile sections are only presented. In order to fully utilize the geotechnical resistances presented in this section, it should be verified that the structural pile resistances are greater than the geotechnical resistances.

The pile geotechnical axial resistance can be computed from a combination of end bearing and side resistance using static analysis methods. The following equation from AASHTO LRFD provides the factored axial resistance:

$$R_R = \varphi_{qp}\, q_p\, A_p + \varphi_{qs}\, q_s\, A_s$$

Where

$R_R$ = factored compressive resistance
$\varphi_{qp,\, qs}$ = Resistance Factors (for static analysis methods)
$q_p$ = unit end bearing resistance
$q_s$ = unit side resistance
$A_p$ $A_s$ = Area of Pile Tip and Shaft respectively.

For piles driven into the dense sand layer at the site, the tip resistance factor $\varphi_{qp}$ is 0.45 (static analysis method) for the strength limit state. The end bearing resistance $q_p$ for the dense sand was calculated as 75 ksf.

Based on analysis of the design soil parameters for the various layers present at the site (below the maximum scour elevation), the unit side resistance $q_s$, together with the side resistance factors $\varphi_{qs}$ for the strength limit state (LS) was computed as follows:

| Soil Stratum | $q_s$ | $\varphi_{qs}$ Strength LS (static analysis method) |
|---|---|---|
| Medium Dense Sand | 0.7 ksf | 0.45 |
| Very Stiff Clay | 1.5 ksf | 0.35 |
| Dense Sand | 1.7 ksf | 0.45 |

Substituting these values into the equation for the factored axial resistance by static analysis method, the following equation is derived:

$R_R = \pi D(8.4\ D + 0.32\ L_{MD\ sand} + 0.53\ L_{VS\ clay} + 0.77\ L_{D\ sand})$ Strength Limit State (in kips)

Where

D = Pile Outside Diameter (in feet)

$L_{MD\ sand}$, $L_{VS\ clay}$, $L_{D\ sand}$ = Effective Length of Pile in medium dense sand, very stiff clay, and dense sand, respectively (in feet)

For the extreme limit state, the factored resistance will be equal to the nominal resistance, since the resistance factor is equal to 1.0. The required nominal resistance for dynamic testing is equal to the required factored resistance for strength LS (LRFD Strength 1) divided by a resistance factor of 0.65. Therefore, when dynamic testing is performed, the extreme LS resistance is equal to the nominal resistance obtained by the dynamic testing.

From these equations, the estimated axial resistances and corresponding tip elevations for different diameter pipe piles were calculated and are presented in the following table:

| Pile Location | Pile Diameter (in) | Estimated Minimum Tip Elevation (feet) | Total Factored Axial Compression Resistance for Strength LS (LRFD Strength 1) (kips) | Total Factored Axial Compression Resistance for Extreme LS (kips) |
|---|---|---|---|---|
| West Abutment | 14 | -70 | 135 | 205 |
| | 16 | -70 | 160 | 245 |
| West Approach Piers | 14 | -70 | 120 | 185 |
| | 16 | -70 | 140 | 215 |
| Bascule / Rest Pier | 14 | -70 | 110 | 165 |
| | 16 | -70 | 130 | 200 |
| | 16 | -75 | 140 | 215 |
| East Approach Piers | 14 | -70 | 110 | 165 |
| | 16 | -70 | 130 | 200 |
| East Abutment | 14 | -70 | 115 | 175 |
| | 16 | -70 | 140 | 215 |

If greater pile resistance is required, piles can be driven deeper, and the formulas given above can be used to calculate the estimated minimum pile tip elevation.

To minimize pile group action, AASHTO requires minimum center-to-center pile spacing to be the greater of 2 feet 6 inches or 2.5 pile diameters. The tops of piles should extend at least 12 inches into the concrete stubs, caps, or abutments.

More detailed axial compression pile capacity calculations are presented in Appendix E.

### 4.2.3 Nominal Axial Pile Resistance During Dynamic Testing

As mentioned in Section 4.2.1, soil resistance within the potential scour depth was ignored when calculating the design factored resistance. The nominal resistance to be obtained during the dynamic testing was calculated by adding the estimated nominal pile resistance within the potential scour depth to the design nominal pile resistance (factored resistance/resistance factor). Since the dynamic testing will be performed on at least two piles per substructure, in accordance with AASHTO LRFD Table 10.5.5.2.3-1, a resistance factor of 0.65 should be used to calculate the nominal pile resistance. More detailed calculations for the nominal axial pile resistance during dynamic testing are presented in Appendix E. Based on the calculations presented in Appendix E, the calculated nominal axial pile resistances during dynamic testing are presented in the following table:

| Pile Location | Pile Diameter (in) | Estimated Minimum Tip Elevation (feet) | Total Factored Axial Compression Resistance for Strength Limit State (LRFD Strength 1) (kips) | Nominal Axial Compression Resistance During Dynamic Testing (kips) |
|---|---|---|---|---|
| West Abutment | 14 | -70 | 135 | 225 |
| | 16 | -70 | 160 | 265 |
| West Approach Piers | 14 | -70 | 120 | 190 |
| | 16 | -70 | 140 | 220 |
| Bascule / Rest Pier | 14 | -70 | 110 | 170 |
| | 16 | -70 | 130 | 205 |
| | 16 | -75 | 140 | 220 |
| East Approach Piers | 14 | -70 | 110 | 170 |
| | 16 | -70 | 130 | 205 |
| East Abutment | 14 | -70 | 115 | 195 |
| | 16 | -70 | 140 | 235 |

### 4.2.4 Uplift Capacity of Piles

The pile uplift capacity can similarly be computed using the static analysis method. The following equation provides the factored uplift resistance:

$$R_R = \varphi_{up}\, q_s\, A_s$$

Where

$R_R$ = factored uplift resistance
$\varphi_{up}$ = resistance factor for uplift (static analysis method)
$q_s$ = unit side resistance
$A_s$ = area of pile shaft.

The unit side resistance $q_s$ and side resistance factors $\varphi_{up}$ for the strength limit state (LS) are as follows:

| Soil Stratum | $q_s$ | $\varphi_{up}$ Strength LS |
|---|---|---|
| Medium Dense Sand | 0.7 ksf | 0.35 |
| Very Stiff Clay | 1.5 ksf | 0.25 |
| Dense Sand | 1.7 ksf | 0.35 |

Substituting these values into the equation for the factored uplift resistance, the following equation is derived:

$$R_R = \pi D(0.25\, L_{MD\ sand} + 0.38\, L_{VS\ clay} + 0.6\, L_{D\ sand})$$ Strength Limit State (in kips)

Where

D = Pile Outside Diameter (in feet)
$L_{MD\ sand}$, $L_{VS\ clay}$, $L_{D\ sand}$ = Effective Length of Pile in medium dense sand, very stiff clay, and dense sand, respectively (in feet)

For the extreme limit state, the factored uplift resistance will be equal to 80% of the nominal resistance, since the resistance factor is equal to 0.8. The required nominal resistance for dynamic testing is equal to the required factored resistance for strength LS (LRFD Strength 1) divided by a resistance factor of 0.50. Therefore, when dynamic testing is performed the extreme LS uplift resistance is equal to the factored uplift resistance (LRFD Strength 1) divided by 0.5 and then multiplied by 0.8.

From the above, the uplift resistances for different diameter pipe piles were calculated and are presented in the following table:

| Pile Location | Pile Diameter (in) | Estimated Minimum Tip Elevation for Compression Loading (feet) | Total Factored Uplift Resistance for Strength Limit State (LRFD Strength 1) (kips) | Total Factored Uplift Resistance for Extreme Limit State (kips) |
|---|---|---|---|---|
| West Abutment | 14 | -70 | 70 | 110 |
| | 16 | -70 | 85 | 135 |
| West Approach Piers | 14 | -70 | 60 | 95 |
| | 16 | -70 | 70 | 110 |
| Bascule / Rest Pier | 14 | -70 | 55 | 85 |
| | 16 | -70 | 65 | 100 |
| | 16 | -75 | 70 | 110 |
| East Approach Piers | 14 | -70 | 55 | 130 |
| | 16 | -70 | 65 | 100 |
| East Abutment | 14 | -70 | 60 | 95 |
| | 16 | -70 | 70 | 110 |

The uplift resistance of a pile group shall be taken as the lesser of the sum of individual pile uplift resistances, or uplift resistance of the pile group considered as a block under the stipulation of AASHTO Article 10.7.3.11. Strength limit state resistance factor of 0.5 shall be used for uplift resistance of pile group considered as a block.

More detailed uplift pile capacity calculations are presented in Appendix E.

### 4.2.5 Lateral Capacity of Piles

The lateral load capacity of a pile or pile group can be determined using computer analysis programs for laterally loaded piles such as LPILE or GROUP. For loading parallel to a line of piles, the lateral pile capacity of the shadow piles should be reduced in accordance with factors provided in the latest edition of AASHTO LRFD (e.g. Table 10.7.2.4-1). For loading perpendicular to the centerline of piles, the piles should behave as individual units if they are spaced at a distance of more than 5D, where D is the pile diameter.

A resistance factor of 1.0 shall be used to evaluate the horizontal geotechnical resistance of piles under both the strength and extreme limit state loading conditions. However in most cases, the lateral pile load capacity will be limited by the tolerable lateral movement of the pile foundations, which should be established on the basis of compatible movements of the bridge structural components. The resulting internal forces should be checked against the pile structural capacity. For smaller diameter pipe piles, the top portion of the piles embedded in the cap may need additional reinforcement (i.e. using core steel or rebar) to accommodate the anticipated maximum bending moments due to the lateral load. Analyses for piles for pier foundations that are unsupported above the mud or scour line will have to consider the free unsupported portion of the piles.

### 4.2.6 Settlement of Piles

Settlements of the pile supported abutments and piers due to bridge loading was estimated using the equivalent footing analogy of pile groups in accordance with AASHTO Article 10.7.2.3. Based on the settlement calculations, the elastic settlement of soils is estimated to range from about ½ inch to ¾ inch for the estimated unfactored (service) loads. Considering the nature of subsurface conditions, it is anticipated most of any settlement will occur during construction as loads are applied.

## 5.1 INSTALLATION OF PILES

Piles can be driven from the ground surface with an impact type hammer or with a vibratory hammer to install the piles through the embankment fill and loose/soft river bottom deposits. Vibratory hammer should provide better control when driving through the loose/soft strata, since vibratory hammers do not permit the pile to run. However, an impact pile driving hammer will still have to be used to drive the pile to final tip elevation or refusal.

In accordance with AASHTO LRFD (Table 10.5.5.2.3-1), at least two dynamic tests (ASTM D4945) should be performed at every bridge substructure location (abutments and piers) in order to establish the final driving criteria corresponding to the required nominal pile resistance. As per ASHTO LRFD Table 10.5.5.2.3-1, a resistance factor of 0.65 should be used to estimate the factored pile resistance.

Wave equation analyses should be performed to evaluate preliminary driving criteria and define requirements for the size of the pile hammer to drive the piles to their minimum tip elevation. At a minimum, the pile hammer must have a rated driving capacity of 25,000 foot-pounds of total energy per blow. The maximum driving stresses in the piles should not exceed 0.9 $F_Y$ in either compression or tension. Driving shoes meeting the requirements of ASTM A27 will be required to penetrate to the bearing depth.

## 5.2 EXCAVATIONS FOR FOUNDATIONS

Excavations for the new abutment walls, if possible, should be performed directly behind the existing abutments. It is recommended that the bottom of new abutments be located no deeper than the existing abutments, so that no additional excavation support will be required along the existing abutment backwalls.

Excavations above groundwater level can be accomplished using open cuts with side slopes if space allows. The slopes of open cuts should be per the relevant OSHA, local, and/or federal regulations (see Standard Specifications Section 140.60 Part F) and no steeper than 1.5H:1V. For excavations below groundwater level, or where there is not sufficient space for using open cuts, the excavations should be accomplished using a temporary excavation support system such as steel sheeting or soldier pile/lagging system. The excavation support systems should be designed by a Registered Professional Engineer retained by the contractor.

If excavations extend only slightly below the groundwater level, pumping using filtered sumps to control the groundwater could be feasible. However for deeper excavations below groundwater level, dewatering with wells or a groundwater cut-off wall/cofferdam system is recommended. To minimize soil disturbance and provide a stable working surface, a 4 to 6-inch-thick mud mat of lean concrete, or 12-inches of gravel over filter fabric should be placed to facilitate the foundation construction. The contractor should also be prepared to control rainwater and surface water runoff (see Standard Specifications Section 140.60 Part E).

It is anticipated that cofferdams will be required for construction of the abutments, the bascule pier and the rest pier. Existing piles should be cut-off a minimum of 2 feet below the mud line or completely removed where required to avoid interference with the proposed pile foundations.

## 5.3 EMBANKMENT CONSTRUCTION ADJACENT TO FOUNDATIONS

Due to the planned increase in roadway profile across the bridge, some new embankment construction adjacent to bridge foundations is anticipated. Embankment construction should be performed in general accordance with the construction methods presented in Section 150 of the MassDOT Standard Specifications. Notable requirements are:

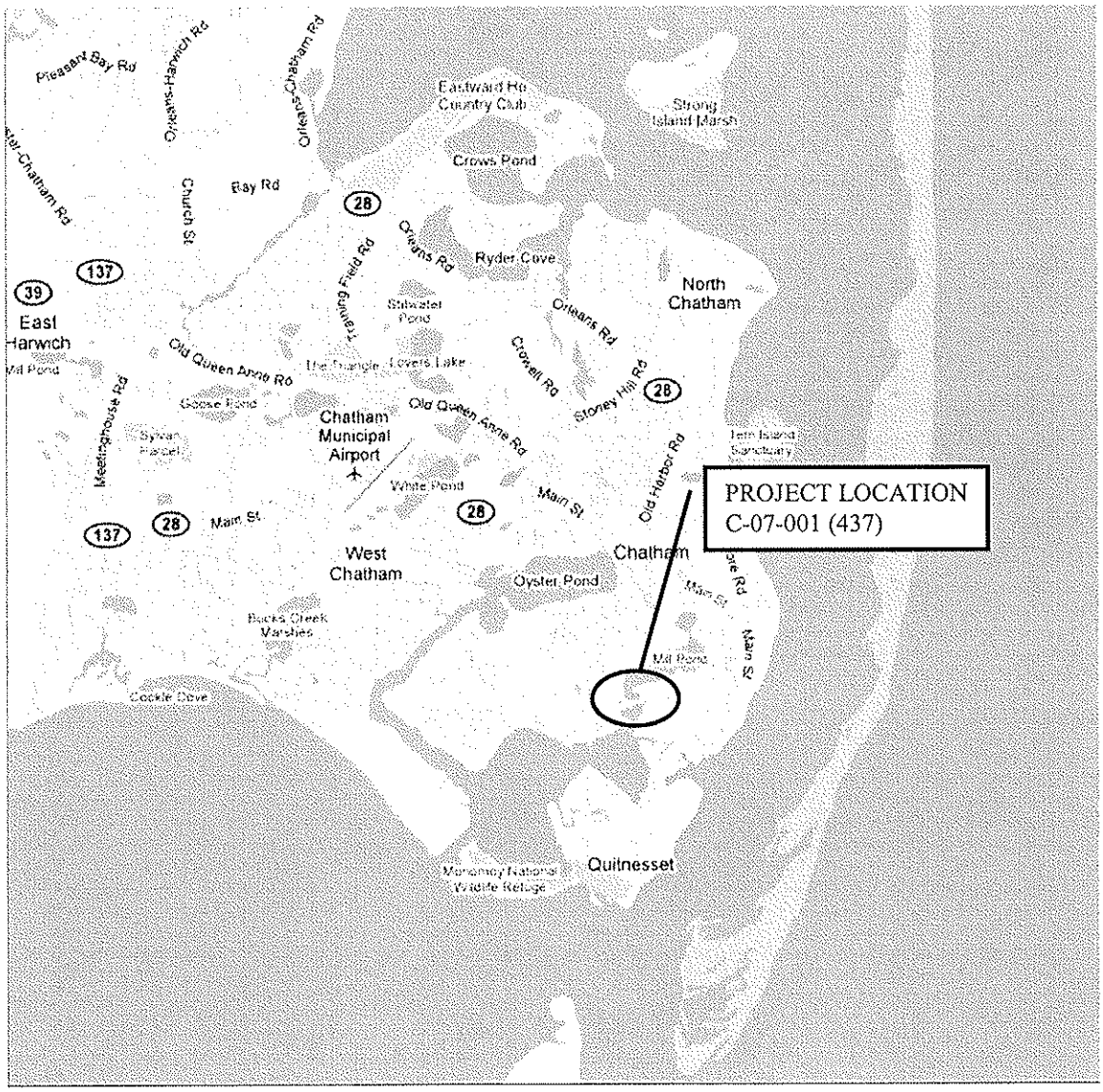
- The use of gravel borrow for backfill around foundations;
- A minimum distance of 20 feet between abutment backwalls and rockfill if used in embankments.

The results and recommendations presented in this report are largely based on subsurface information from a limited number of borings and our use of generally accepted analytical procedures. If further investigation or construction activity reveals significant differences in the subsurface conditions, we should be given the opportunity to review and, if appropriate, modify our recommendations.

# APPENDIX A

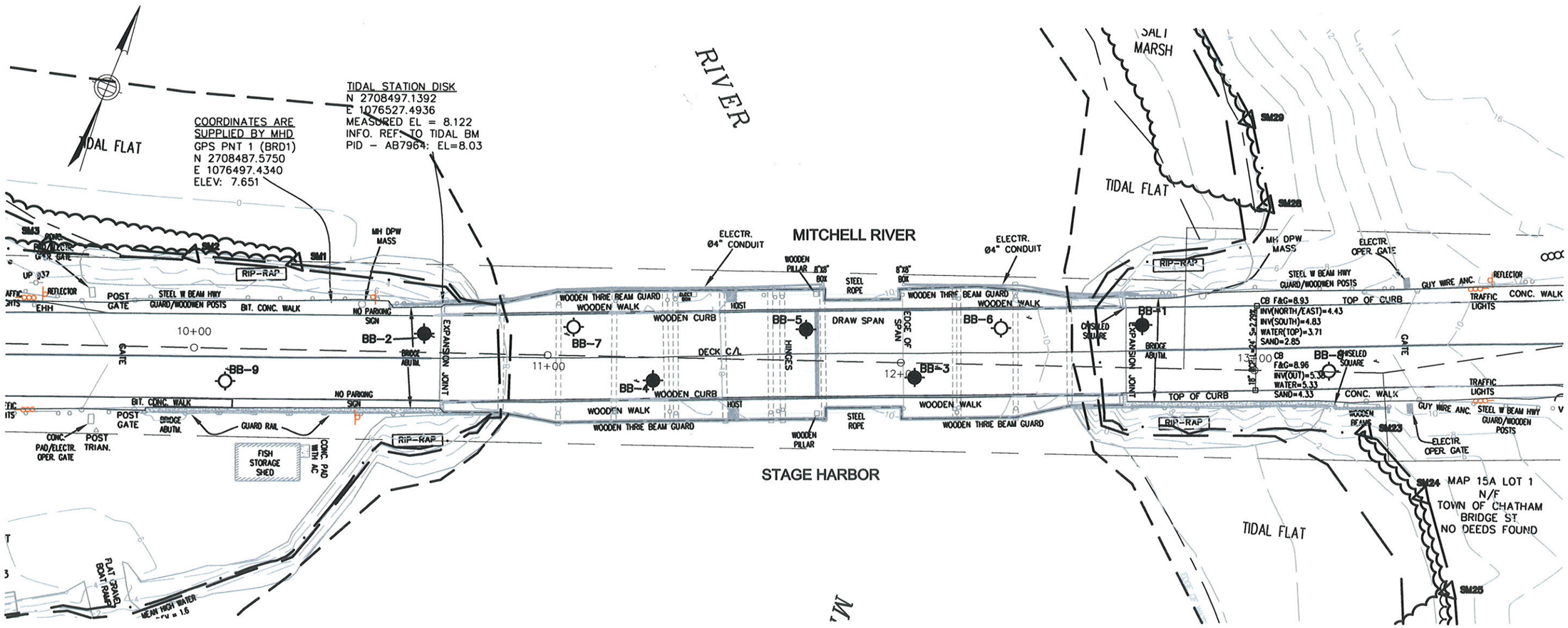
## LOCUS PLAN

Chatham C-07-001 (437)
Bridge Street over the Mitchell River



# APPENDIX B

## BORING LOCATION PLAN




# BORING PLAN

SCALE: 1" = 30'-0"

| BORING LOCATIONS (FEET) | | | | | | |
|---|---|---|---|---|---|---|
| BORING | NORTHING | EASTING | STATION | OFFSET | SURFACE | H.B.E. |
| BB-1 | 2,708,551 | 1,076,718 | 12+68.0 | 12.16 LT | 9.3 | -90.7 |
| BB-2 | 2,708,488 | 1,076,525 | 10+65.0 | 5.33 LT | 7.4 | -92.6 |
| BB-3 | 2,708,518 | 1,076,661 | 12+4.0 | 4.10 RT | 10.7 | -39.3 |
| BB-4 | 2,708,495 | 1,076,590 | 11+30.0 | 6.56 RT | 9.3 | -40.7 |
| BB-5 | 2,708,522 | 1,076,627 | 11+73.0 | 8.85 LT | 10.5 | -39.5 |
| BB-6 | 2,708,526 | 1,076,686 | 12+30.0 | 3.34 RT | 10.2 | -39.80 |
| BB-7 | 2,708,486 | 1,076,562 | 11+0.0 | 7.58 RT | 8.3 | -41.7 |
| BB-8 | 2,708,554 | 1,076,772 | 13+21.0 | 0.22 LT | 9.3 | -40.70 |
| BB-9 | 2,708,459 | 1,076,475 | 10+9.0 | 9.26 RT | 7.2 | -42.75 |

LEGEND:

BB-5 – PRIMARY BORINGS DRILLED IN APRIL 2010

BB-6 – COMPLEMENTARY BORINGS (HAVE NOT BEEN DRILLED)

PROJECT FILE NO. 603690

Moving Massachusetts Forward. massDOT Highway

BORING PLAN FOR PROPOSED BRIDGE IN

## CHATHAM

BRIDGE STREET OVER MITCHELL RIVER

THE COMMONWEALTH OF MASSACHUSETTS
HIGHWAY DEPARTMENT

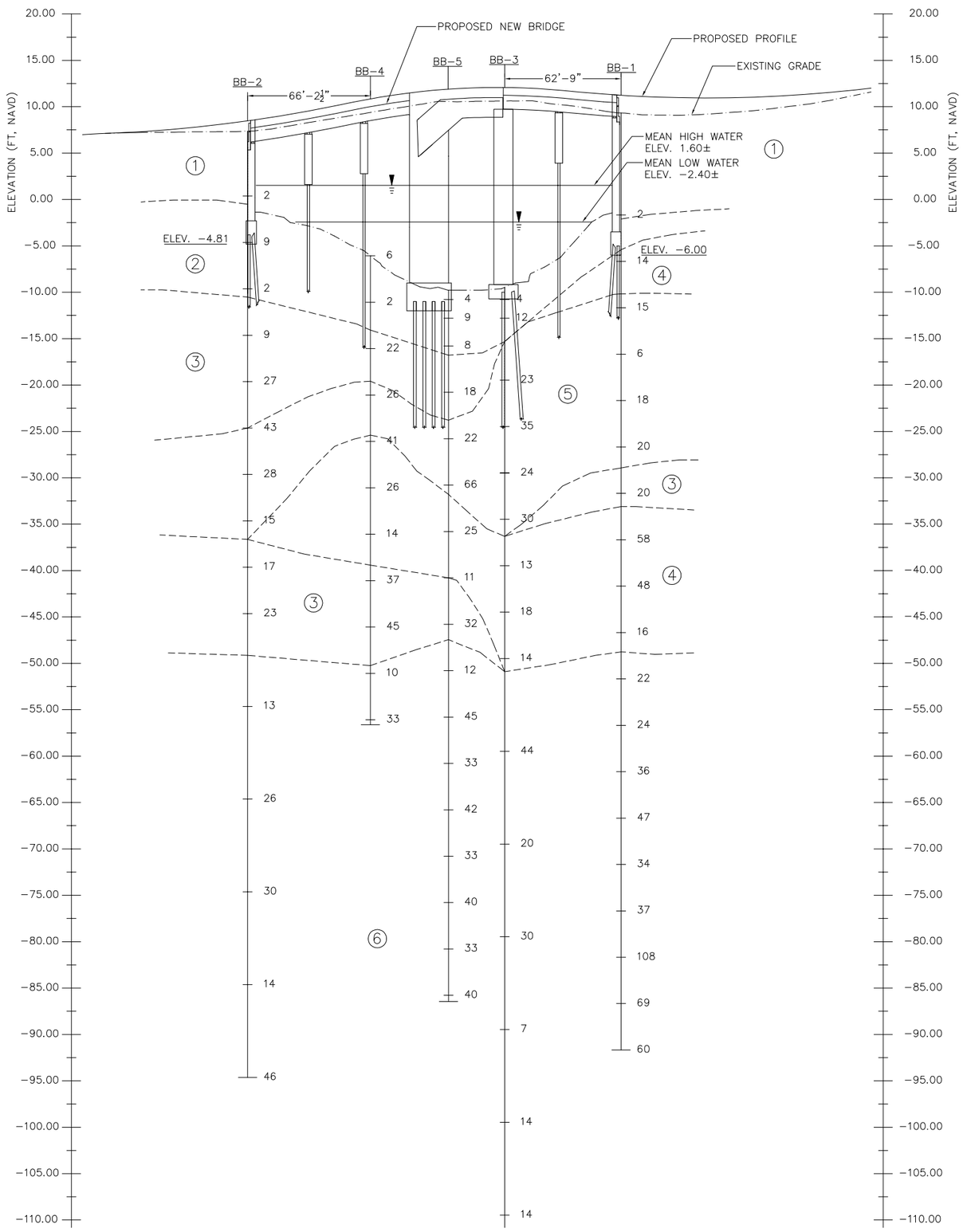
FEBRUARY 2010

BRIDGE NO. C-07-001(437)

PLOTTED: 10/06/10 9:33AM BY:MATT_ALDINGER
DRAWING: P:\J08S-GED\MASSHIGHWAY\2009 SEDRSCM 5\ASSIGN #1 - CHATHAM BRIDGE\500 CADD - GIS - GRAPHICS\510 CADD\STRUC\MISC\CHATHAM BORING PLAN

# APPENDIX C

## GENERALIZED SUBSURFACE PROFILES & BRIDGE LOGITUDINAL SECTION




## GENERAL NOTES:

1. MATERIAL DESCRIPTIONS ARE GENERALIZED AND INCLUDE SAMPLES WITH A NATURAL DEGREE OF VARIATION. SEE BORING LOGS FOR DESCRIPTION OF INDIVIDUAL SAMPLES.
2. DEPTH AND THICKNESSES OF SOIL STRATA BOUNDARIES ARE BASED ON INTERPRETATION OF BORING RESULTS AND ARE SHOWN ONLY TO AID IN VISUALIZING GENERALIZED SUBSURFACE CONDITIONS. ACTUAL STRATA BOUNDARIES BETWEEN BORINGS MAY DIFFER FROM THE CONDITIONS SHOWN HERE.
3. BRIDGE LONGITUDINAL SECTION IS SHOWN ONLY FOR ILLUSTRATIVE PROPOSES
4. FINAL PROFILE WITH THE TITLE BLOCK AND STANDARD SCALE WILL BE INCLUDED WITH THE FINAL REPORT.

## GENERALIZED SOIL DESCRIPTIONS:

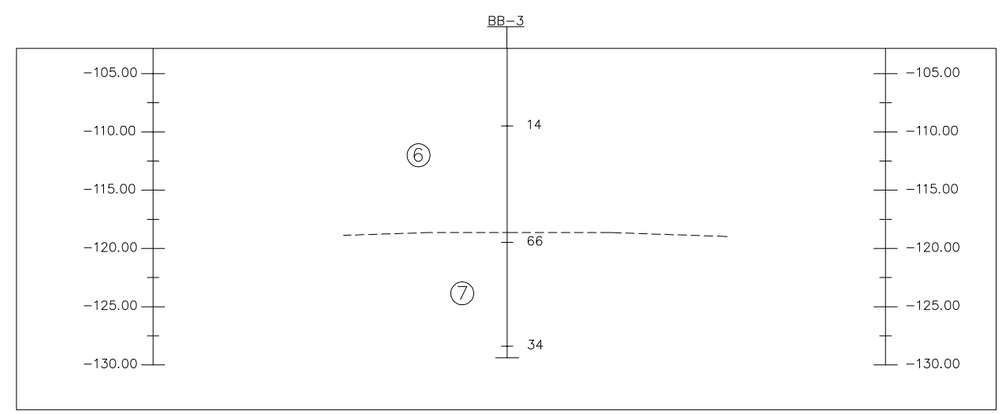
1. FILL – FINE TO COARSE SAND WITH VARYING AMOUNT OF GRAVEL
2. RIVER BOTTOM SEDIMENT – SAND AND SILT, LOOSE/SOFT.
3. VERY STIFF CLAY AND SILT
4. MEDIUM DENSE FINE TO COARSE SAND
5. MEDIUM DENSE SILTY FINE SAND
6. MEDIUM DENSE TO DENSE FINE SAND
7. DENSE TO VERY DENSE FINE TO COARSE SAND

## LEGEND

BB-1 — GEOTECHNICAL BORING DRILLED BY NEW HAMPSHIRE BORING, UNDER URS SUPERVISION

23 — N-VALUE, DEFINED AS NUMBER OF BLOWS OF A 140-LB HAMMER FREE FALLING FOR 30 INCHES REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES AFTER INITIAL 6 INCH PENETRATION

APPROXIMATE STRATA BOUNDARY




# GENERALIZED SUBSURFACE PROFILES & BRIDGE LOGITUDINAL SECTION

DRAFT REPORT

# APPENDIX D

## BORING LOGS & LABORATORY TEST RESULTS

| massDOT | NEW HAMPSHIRE BORING, INC.<br>P.O. Box 165<br>Derry, NH 03038 | Phone: (603)437-1610<br>Fax: (603) 437-0034<br>E Mail: nhb@nhboring.com | **Boring #** BB-1<br>Scale 1" = 5' |
|---|---|---|---|

| | | | |
|---|---|---|---|
| City/Town: Chatham | Bridge: C-07-001 | Project File No: 603690 | Contract No: 58732 |
| Project: Bridge Street over the Mitchell River | | Date & Time Started: 03-30-10 7:00AM | Total Hours: 14.5 |
| Groundwater Depth: 7' | Date & Time: 03-31-10 3:20PM | Date & Time Completed: 03-31-10 3:30PM | |
| Coordinates: N 2708550 | E 1076717 | Driller's Name: Roger Burne | Helper's Name: Patrick Schofield |
| Ground Elevation: 9.3 | Inspector's Name: Alex Marinkovic | Inspector's Company: URS Corporation | |

| Sample Number | Depth Range (Feet) | Blow Counts per 6 Inches Coring Times Minute Per Foot | | | | Recovery Inches | Field Description | Strata Changes |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | ASPHALT | *6" |
| | | | | | | | 1'6" of FINE TO COARSE SAND. | 2' |
| | | | | | | | 3' of CONCRETE AND STEEL REBAR. | 5' |
| | | | | | | | VOID 5' to 8', hole was open. | 8' |
| | | | | | | | From 8' to 9' rip rap of boulders, advanced drove 4" casing to 9' because of debris in hole. Took first sample at 9'. | |
| S-1 | 9' - 11' | 1 | 1 | 1 | 1 | 7" | Wet, very loose, light brown, FINE TO COARSE SAND, trace inorganic silt. | |
| S-2 | 14' - 16' | 15 | 7 | 7 | 5 | 8" | Wet, medium dense, brown, FINE TO COARSE SAND, trace fine to medium gravel, trace inorganic silt. | 18' |
| S-3 | 19' - 21' | 7 | 6 | 9 | 12 | 20" | Wet, medium dense, light brown, FINE SAND AND INORGANIC SILT. | |
| S-4 | 24' - 26' | 3 | 2 | 4 | 5 | 8" | Wet, loose, light brown, FINE SAND, trace fine to medium gravel, some inorganic silt. | |
| S-5 | 29' - 31' | 6 | 7 | 11 | 11 | 9" | Wet, medium dense, light brown, FINE SAND AND INORGANIC SILT. | |

| | | |
|---|---|---|
| Remarks: Sample by cathead.<br>Sample by cathead. 2 bags concrete. | Arrow-Board:<br>Signs: 2<br>Cones: 13 | Protective Device - Stand: Box:<br>Well Depth: Solid Pipe:<br>Stick Up Pipe: Screen Pipe: |

Penetration Resistance (N) Guide:

| Cohesionless Soils (Sands, Gravels) | | Cohesive Soils (Silts, Clays) | |
|---|---|---|---|
| Relative Density | Penetration Resistance | Consistency | Penetration Resistance |
| Very Loose | 0 - 4 | Very Soft | 0 - 2 |
| Loose | 4 - 10 | Soft | 2 - 4 |
| Medium Dense | 10 - 30 | Medium Stiff | 4 - 8 |
| Dense | 30 - 50 | Stiff | 8 - 15 |
| Very Dense | Over 50 | Very Stiff | 15 - 30 |
| N=Sum of Second and Third 6" Blow Counts | | Hard | Over 30 |

Terms Used for Second Entry of Descriptions: and = 40-50%, some = 10-40%, trace = 10% or less

Type of Drill Rig: Diedrich D-50

Casing HW Size: 4" Depth: 99'

Hammer Weight: 300 lbs.

Fall: 30"

Sampler Type: S/S Size: 1 3/8"

Automatic Hammer Weight:

Safety Hammer Weight: 140 lbs.

Donut Hammer Weight:

Fall: 30"

Core Barrel Type: Size:

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|---|---|---|---|

| | | | |
|---|---|---|---|
| City/Town: Chatham | Bridge: C-07-001 | Project File No: 603690 | Contract No: 58732 |
| Project: Bridge Street over the Mitchell River | | Date & Time Started: 03-30-10 7:00AM | Total Hours: 14.5 |
| Groundwater Depth: 7' | Date & Time: 03-31-10 3:20PM | Date & Time Completed: 03-31-10 3:30PM | |
| Coordinates: N 2708550 | E 1076717 | Driller's Name: Roger Burne | Helper's Name: Patrick Schofield |
| Ground Elevation: 9.3 | Inspector's Name: Alex Marinkovic | Inspector's Company: URS Corporation | |

| Sample Number | Depth Range (Feet) | Blow Counts per 6 Inches Coring Times Minute Per Foot | | | | Recovery Inches | Field Description | Strata Changes |
|---|---|---|---|---|---|---|---|---|
| S-6 | 34' - 36' | 9 | 9 | 11 | 12 | 12" | Wet, medium dense, gray, FINE TO COARSE SAND, some inorganic silt. | 39' |
| S-7 | 39' - 41' | 11 | 11 | 9 | 9 | 20" | Wet, very stiff, gray, CLAY. | 41' |
| S-8 | 44- 46 | 21 | 19 | 39 | 41 | 17" | Wet, very dense, gray, FINE TO COARSE SAND, some inorganic silt. | |
| S-9 | 49' - 51' | 14 | 21 | 27 | 21 | 7" | Wet, dense, gray, FINE TO COARSE SAND, trace inorganic silt. | |
| S-10 | 54' - 56' | 11 | 9 | 7 | 11 | 18" | Wet, medium dense, gray, FINE TO COARSE SAND, trace inorganic silt. | 59' |
| S-11 | 59' - 61' | 12 | 11 | 11 | 10 | 12" | Wet, medium dense, gray, FINE SAND AND INORGANIC SILT. | |

Remarks: Sample by cathead.
Sample by cathead. 2 bags concrete.

Arrow-Board:
Signs: 2
Cones: 13

Protective Device - Stand: Box:
Well Depth: Solid Pipe:
Stick Up Pipe: Screen Pipe:

Penetration Resistance (N) Guide:

| Cohesionless Soils (Sands, Gravels) | | Cohesive Soils (Silts, Clays) | |
|---|---|---|---|
| Relative Density | Penetration Resistance | Consistency | Penetration Resistance |
| Very Loose | 0 - 4 | Very Soft | 0 - 2 |
| Loose | 4 - 10 | Soft | 2 - 4 |
| Medium Dense | 10 - 30 | Medium Stiff | 4 - 8 |
| Dense | 30 - 50 | Stiff | 8 - 15 |
| Very Dense | Over 50 | Very Stiff | 15 - 30 |
| N=Sum of Second and Third 6" Blow Counts | | Hard | Over 30 |

Type of Drill Rig: Diedrich D-50
Casing HW Size: 4" Depth: 99'
Hammer Weight: 300 lbs.
Fall: 30"
Sampler Type: S/S Size: 1 3/8"
Automatic Hammer Weight:
Safety Hammer Weight: 140 lbs.
Donut Hammer Weight:
Fall: 30"

Terms Used for Second Entry of Descriptions: and = 40-50%, some = 10-40%, trace = 10% or less

Core Barrel Type: Size:

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|---|---|---|---|

| | | | |
|---|---|---|---|
| City/Town: Chatham | Bridge: C-07-001 | Project File No: 603690 | Contract No: 58732 |
| Project: Bridge Street over the Mitchell River | | Date & Time Started: 03-30-10 7:00AM | Total Hours: 14.5 |
| Groundwater Depth: 7' Date & Time: 03-31-10 3:20PM | | Date & Time Completed: 03-31-10 3:30PM | |
| Coordinates: N 2708550 E 1076717 | Driller's Name: Roger Burne | Helper's Name: Patrick Schofield | |
| Ground Elevation: 9.3 | Inspector's Name: Alex Marinkovic | Inspector's Company: URS Corporation | |

| Sample Number | Depth Range (Feet) | Blow Counts per 6 Inches Coring Times Minute Per Foot | | | | Recovery Inches | Field Description | Strata Changes |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | SILT. | 64' |
| S-12 | 64' - 66' | 4 | 10 | 14 | 18 | 12" | Wet, medium dense, gray, FINE SAND, trace fine to coarse gravel, trace inorganic silt. | 69' |
| S-13 | 69' - 71' | 8 | 14 | 22 | 28 | 14" | Wet, dense, gray, FINE SAND, trace inorganic silt. | |
| S-14 | 74' - 76' | 11 | 21 | 26 | 30 | 18" | Wet, dense, gray, FINE SAND, trace inorganic silt. | |
| S-15 | 79' - 81' | 8 | 18 | 16 | 21 | 14" | Wet, dense, gray, FINE SAND, trace inorganic silt. | |
| S-16 | 84' - 86' | 11 | 18 | 19 | 21 | 11" | Wet, dense, gray, FINE SAND, trace inorganic silt. | |
| S-17 | 89' - 91' | 21 | 39 | 69 | 120 | 18" | Wet, very dense, gray, FINE SAND, trace inorganic silt. | |

Remarks: Sample by cathead.
Sample by cathead. 2 bags concrete.

Arrow-Board:
Signs: 2
Cones: 13

Protective Device - Stand: Box:
Well Depth: Solid Pipe:
Stick Up Pipe: Screen Pipe:

Penetration Resistance (N) Guide:

| Cohesionless Soils (Sands, Gravels) | | Cohesive Soils (Silts, Clays) | |
|---|---|---|---|
| Relative Density | Penetration Resistance | Consistency | Penetration Resistance |
| Very Loose | 0 - 4 | Very Soft | 0 - 2 |
| Loose | 4 - 10 | Soft | 2 - 4 |
| Medium Dense | 10 - 30 | Medium Stiff | 4 - 8 |
| Dense | 30 - 50 | Stiff | 8 - 15 |
| Very Dense | Over 50 | Very Stiff | 15 - 30 |
| N=Sum of Second and Third 6" Blow Counts | | Hard | Over 30 |

Type of Drill Rig: Diedrich D-50
Casing HW Size: 4" Depth: 99'
Hammer Weight: 300 lbs.
Fall: 30"
Sampler Type: S/S Size: 1 3/8"
Automatic Hammer Weight:
Safety Hammer Weight: 140 lbs.
Donut Hammer Weight:
Fall: 30"
Core Barrel Type: Size:

Terms Used for Second Entry of Descriptions: and = 40-50%, some = 10-40%, trace = 10% or less

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|---|---|---|---|

| | | | |
|---|---|---|---|
| City/Town: Chatham | Bridge: C-07-001 | Project File No: 603690 | Contract No: 58732 |
| Project: Bridge Street over the Mitchell River | | Date & Time Started: 03-30-10 7:00AM | Total Hours: 14.5 |
| Groundwater Depth: 7' Date & Time: 03-31-10 3:20PM | | Date & Time Completed: 03-31-10 3:30PM | |
| Coordinates: N 2708550 E 1076717 | Driller's Name: Roger Burne | Helper's Name: Patrick Schofield | |
| Ground Elevation: 9.3 | Inspector's Name: Alex Marinkovic | Inspector's Company: URS Corporation | |

| Sample Number | Depth Range (Feet) | Blow Counts per 6 Inches Coring Times Minute Per Foot | | | | Recovery Inches | Field Description | Strata Changes |
|---|---|---|---|---|---|---|---|---|
| S-18 | 94' - 96' | 17 | 29 | 40 | 46 | 7" | Wet, very dense, gray, FINE SAND, trace inorganic silt. | |
| S-19 | 99' - 101' | 24 | 26 | 34 | 32 | 20" | Wet, very dense, gray, FINE SAND, trace inorganic silt. | 101' |
| | | | | | | | Bottom of Exploration = 101' | |

Remarks: Sample by cathead.
Sample by cathead. 2 bags concrete.

Arrow-Board:
Signs: 2
Cones: 13

Protective Device - Stand: Box:
Well Depth: Solid Pipe:
Stick Up Pipe: Screen Pipe:

Penetration Resistance (N) Guide:

| Cohesionless Soils (Sands, Gravels) | | Cohesive Soils (Silts, Clays) | |
|---|---|---|---|
| Relative Density | Penetration Resistance | Consistency | Penetration Resistance |
| Very Loose | 0 - 4 | Very Soft | 0 - 2 |
| Loose | 4 - 10 | Soft | 2 - 4 |
| Medium Dense | 10 - 30 | Medium Stiff | 4 - 8 |
| Dense | 30 - 50 | Stiff | 8 - 15 |
| Very Dense | Over 50 | Very Stiff | 15 - 30 |
| N=Sum of Second and Third 6" Blow Counts | | Hard | Over 30 |

Type of Drill Rig: Diedrich D-50
Casing HW Size: 4" Depth: 99'
Hammer Weight: 300 lbs.
Fall: 30"
Sampler Type: S/S Size: 1 3/8"
Automatic Hammer Weight:
Safety Hammer Weight: 140 lbs.
Donut Hammer Weight:
Fall: 30"
Core Barrel Type: Size:

Terms Used for Second Entry of Descriptions: and = 40-50%, some = 10-40%, trace = 10% or less

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|---|---|---|---|

| | | | |
|---|---|---|---|
| City/Town: Chatham | Bridge: C-07-001 | Project File No: 603690 | Contract No: 58732 |
| Project: Bridge Street over the Mitchell River | | Date & Time Started: 03-29-10 11:15AM | Total Hours: 17.5 |
| Groundwater Depth: 5.4' Date & Time: 03-31-10 2:30PM | | Date & Time Completed: 03-31-10 2:15PM | |
| Coordinates: N 2708488 E 1076525 | Driller's Name: Manlea Thompson | Helper's Name: Donald Palmer | |
| Ground Elevation: 7.4' | Inspector's Name: Alex Marinkovic | Inspector's Company: URS Corporation | |

| Sample Number | Depth Range (Feet) | Blow Counts per 6 Inches Coring Times Minute Per Foot | Recovery Inches | Field Description | Strata Changes |
|---|---|---|---|---|---|
| | | | | PAVEMENT | * |
| | | | | 9" CONCRETE 1'3" | * 6" |
| | | | | 3'9" VOID | 5' |
| S-1 | 5' - 7' | WOH 1 1/12" | 10" | Wet, very loose, brown, FINE TO COARSE SAND, trace fine gravel, trace wood, trace inorganic silt. | |
| S-2 | 10' - 11' | 2 4 5 4 | 12" | Wet, loose, dark brown, FINE TO COARSE SAND AND FINE TO MEDIUM GRAVEL, trace inorganic silt. | 11' |
| S-2A | 11' - 12' | | 12" | Wet, loose, brown/gray, FINE TO COARSE SAND, some inorganic silt, trace fine gravel. | 13' |
| S-3 | 15' - 17' | WOH/12" 2 4 | 20" | Wet, soft, gray, INORGANIC SILT AND CLAY, trace fine sand. | |
| S-4 | 20' - 22' | 4 5 4 4 | 14" | Wet, stiff, gray, INORGANIC SILT AND CLAY. | 25' |
| S-5 | 25' - 27' | 11 12 15 14 | 17" | Wet, medium dense, gray, FINE SAND AND INORGANIC SILT. | |

Remarks: WOH = Weight of hammer
Changed location; moved boring prior to drilling from N2708488, E1076525 to N2708488, E1076526 boring moved in order to avoid structural members on wooden bridge. Cathead used.

Arrow-Board:
Signs: 2
Cones: 20

Protective Device - Stand: Box:
Well Depth: Solid Pipe:
Stick Up Pipe: Screen Pipe:

Penetration Resistance (N) Guide:

| Cohesionless Soils (Sands, Gravels) | | Cohesive Soils (Silts, Clays) | |
|---|---|---|---|
| Relative Density | Penetration Resistance | Consistency | Penetration Resistance |
| Very Loose | 0 - 4 | Very Soft | 0 - 2 |
| Loose | 4 - 10 | Soft | 2 - 4 |
| Medium Dense | 10 - 30 | Medium Stiff | 4 - 8 |
| Dense | 30 - 50 | Stiff | 8 - 15 |
| Very Dense | Over 50 | Very Stiff | 15 - 30 |
| N=Sum of Second and Third 6" Blow Counts | | Hard | Over 30 |

Type of Drill Rig: CME 550X
Casing HW Size: 4" Depth: 100'
Hammer Weight: 300 lbs.
Fall: 30"
Sampler Type: S/S Size: 1 3/8"
Automatic Hammer Weight: 140 lbs.
Safety Hammer Weight:
Donut Hammer Weight:
Fall: 30"

Terms Used for Second Entry of Descriptions: and = 40-50%, some = 10-40%, trace = 10% or less

Core Barrel Type: Size:

| massDOT | NEW HAMPSHIRE BORING, INC.<br>P.O. Box 165<br>Derry, NH 03038 | Phone: (603)437-1610<br>Fax: (603) 437-0034<br>E Mail: nhb@nhboring.com | **Boring #** BB-2<br>Scale 1" = 5' |
|---|---|---|---|

| | | | |
|---|---|---|---|
| City/Town: Chatham | Bridge: C-07-001 | Project File No: 603690 | Contract No: 58732 |
| Project: Bridge Street over the Mitchell River | | Date & Time Started: 03-29-10 11:15AM | Total Hours: 17.5 |
| Groundwater Depth: 5.4' Date & Time: 03-31-10 2:30PM | | Date & Time Completed: 03-31-10 2:15PM | |
| Coordinates: N 2708488 E 1076525 | Driller's Name: Manlea Thompson | Helper's Name: Donald Palmer | |
| Ground Elevation: 7.4' | Inspector's Name: Alex Marinkovic | Inspector's Company: URS Corporation | |

| Sample Number | Depth Range (Feet) | Blow Counts per 6 Inches Coring Times Minute Per Foot | | | | Recovery Inches | Field Description | Strata Changes |
|---|---|---|---|---|---|---|---|---|
| S-6 | 30' - 31' | 8 | 18 | 25 | 27 | 10" | Wet, hard, gray, INORGANIC SILT AND CLAY, some fine sand. | 31' |
| S-6A | 31' - 32' | | | | | 7" | Wet, dense, brown, FINE TO COARSE SAND, trace fine gravel. | |
| S-7 | 35' - 37' | 9 | 13 | 15 | 14 | 12" | Wet, medium dense, brown, FINE SAND. | |
| S-8 | 40' - 42' | 5 | 7 | 8 | 10 | 12" | Wet, medium dense, gray, FINE SAND. | 43'5" |
| S-9 | 45' - 47' | 4 | 7 | 10 | 16 | 24" | Wet, stiff, gray, CLAY AND INORGANIC SILT, trace fine sand. | |
| S-10 | 50' - 52' | 6 | 9 | 14 | 21 | 24" | Wet, very stiff, gray, CLAY AND INORGANIC SILT, trace fine sand. | 60' |

Remarks: WOH = Weight of hammer
Changed location; moved boring prior to drilling from N2708488, E1076525 to N2708488, E1076526 boring moved in order to avoid structural members on wooden bridge. Cathead used.

Arrow-Board:
Signs: 2
Cones: 20

Protective Device - Stand: Box:
Well Depth: Solid Pipe:
Stick Up Pipe: Screen Pipe:

Penetration Resistance (N) Guide:

| Cohesionless Soils (Sands, Gravels) | | Cohesive Soils (Silts, Clays) | |
|---|---|---|---|
| Relative Density | Penetration Resistance | Consistency | Penetration Resistance |
| Very Loose | 0 - 4 | Very Soft | 0 - 2 |
| Loose | 4 - 10 | Soft | 2 - 4 |
| Medium Dense | 10 - 30 | Medium Stiff | 4 - 8 |
| Dense | 30 - 50 | Stiff | 8 - 15 |
| Very Dense | Over 50 | Very Stiff | 15 - 30 |
| N=Sum of Second and Third 6" Blow Counts | | Hard | Over 30 |

Type of Drill Rig: CME 550X
Casing HW Size: 4" Depth: 100'
Hammer Weight: 300 lbs.
Fall: 30"
Sampler Type: S/S Size: 1 3/8"
Automatic Hammer Weight: 140 lbs.
Safety Hammer Weight:
Donut Hammer Weight:
Fall: 30"

Terms Used for Second Entry of Descriptions: and = 40-50%, some = 10-40%, trace = 10% or less

Core Barrel Type: Size:

| massDOT | NEW HAMPSHIRE BORING, INC.<br>P.O. Box 165<br>Derry, NH 03038 | Phone: (603)437-1610<br>Fax: (603) 437-0034<br>E Mail: nhb@nhboring.com | **Boring #** BB-2<br>Scale 1" = 5' |
|---|---|---|---|

| | | | |
|---|---|---|---|
| City/Town: Chatham | Bridge: C-07-001 | Project File No: 603690 | Contract No: 58732 |
| Project: Bridge Street over the Mitchell River | | Date & Time Started: 03-29-10 11:15AM | Total Hours: 17.5 |
| Groundwater Depth: 5.4' Date & Time: 03-31-10 2:30PM | | Date & Time Completed: 03-31-10 2:15PM | |
| Coordinates: N 2708488 E 1076525 | Driller's Name: Manlea Thompson | Helper's Name: Donald Palmer | |
| Ground Elevation: 7.4' | Inspector's Name: Alex Marinkovic | Inspector's Company: URS Corporation | |

| Sample Number | Depth Range (Feet) | Blow Counts per 6 Inches Coring Times Minute Per Foot | | | | Recovery Inches | Field Description | Strata Changes |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | | 60' |
| S-11 | 60' - 62' | 3 | 5 | 8 | 9 | 24" | Wet, medium dense, gray, FINE SAND, trace inorganic silt. | |
| S-12 | 70' - 72' | 11 | 13 | 13 | 14 | 16" | Wet, medium dense, gray, FINE SAND, trace inorganic silt. | |
| S-13 | 80' - 82' | 16 | 16 | 14 | 32 | 14" | Wet, dense, gray, FINE SAND, trace inorganic silt. | |

Remarks: WOH = Weight of hammer
Changed location; moved boring prior to drilling from N2708488, E1076525 to N2708488, E1076526 boring moved in order to avoid structural members on wooden bridge. Cathead used.

Arrow-Board:
Signs: 2
Cones: 20

Protective Device - Stand: Box:
Well Depth: Solid Pipe:
Stick Up Pipe: Screen Pipe:

Penetration Resistance (N) Guide:

| Cohesionless Soils (Sands, Gravels) | | Cohesive Soils (Silts, Clays) | |
|---|---|---|---|
| Relative Density | Penetration Resistance | Consistency | Penetration Resistance |
| Very Loose | 0 - 4 | Very Soft | 0 - 2 |
| Loose | 4 - 10 | Soft | 2 - 4 |
| Medium Dense | 10 - 30 | Medium Stiff | 4 - 8 |
| Dense | 30 - 50 | Stiff | 8 - 15 |
| Very Dense | Over 50 | Very Stiff | 15 - 30 |
| N=Sum of Second and Third 6" Blow Counts | | Hard | Over 30 |

Type of Drill Rig: CME 550X
Casing HW Size: 4" Depth: 100'
Hammer Weight: 300 lbs.
Fall: 30"
Sampler Type: S/S Size: 1 3/8"
Automatic Hammer Weight: 140 lbs.
Safety Hammer Weight:
Donut Hammer Weight:
Fall: 30"

Terms Used for Second Entry of Descriptions: and = 40-50%, some = 10-40%, trace = 10% or less

Core Barrel Type: Size:

| massDOT | NEW HAMPSHIRE BORING, INC.<br>P.O. Box 165<br>Derry, NH 03038 | Phone: (603)437-1610<br>Fax: (603) 437-0034<br>E Mail: nhb@nhboring.com | **Boring #** BB-2<br>Scale 1" = 5' |
|---|---|---|---|

| | | | |
|---|---|---|---|
| City/Town: Chatham | Bridge: C-07-001 | Project File No: 603690 | Contract No: 58732 |
| Project: Bridge Street over the Mitchell River | | Date & Time Started: 03-29-10 11:15AM | Total Hours: 17.5 |
| Groundwater Depth: 5.4' Date & Time: 03-31-10 2:30PM | | Date & Time Completed: 03-31-10 2:15PM | |
| Coordinates: N 2708488 E 1076525 | Driller's Name: Manlea Thompson | Helper's Name: Donald Palmer | |
| Ground Elevation: 7.4' | Inspector's Name: Alex Marinkovic | Inspector's Company: URS Corporation | |

| Sample Number | Depth Range (Feet) | Blow Counts per 6 Inches Coring Times Minute Per Foot | | | | Recovery Inches | Field Description | Strata Changes |
|---|---|---|---|---|---|---|---|---|
| S-14 | 90' - 92' | 4 | 7 | 7 | 10 | 24" | Wet, medium dense, gray, FINE SAND, trace inorganic silt. | |
| S-15 | 100' - 102' | 13 | 23 | 23 | 27 | 23" | Wet, dense, gray, FINE SAND, trace inorganic silt. | 102' |
| | | | | | | | Bottom of Exploration = 102' | |

Remarks: WOH = Weight of hammer
Changed location; moved boring prior to drilling from N2708488, E1076525 to N2708488, E1076526 boring moved in order to avoid structural members on wooden bridge. Cathead used.

Arrow-Board:
Signs: 2
Cones: 20

Protective Device - Stand: Box:
Well Depth: Solid Pipe:
Stick Up Pipe: Screen Pipe:

Penetration Resistance (N) Guide:

| Cohesionless Soils (Sands, Gravels) | | Cohesive Soils (Silts, Clays) | |
|---|---|---|---|
| Relative Density | Penetration Resistance | Consistency | Penetration Resistance |
| Very Loose | 0 - 4 | Very Soft | 0 - 2 |
| Loose | 4 - 10 | Soft | 2 - 4 |
| Medium Dense | 10 - 30 | Medium Stiff | 4 - 8 |
| Dense | 30 - 50 | Stiff | 8 - 15 |
| Very Dense | Over 50 | Very Stiff | 15 - 30 |
| N=Sum of Second and Third 6" Blow Counts | | Hard | Over 30 |

Type of Drill Rig: CME 550X
Casing HW Size: 4" Depth: 100'
Hammer Weight: 300 lbs.
Fall: 30"
Sampler Type: S/S Size: 1 3/8"
Automatic Hammer Weight: 140 lbs.
Safety Hammer Weight:
Donut Hammer Weight:
Fall: 30"

Terms Used for Second Entry of Descriptions: and = 40-50%, some = 10-40%, trace = 10% or less

Core Barrel Type: Size:

| massDOT | NEW HAMPSHIRE BORING, INC.<br>P.O. Box 165<br>Derry, NH 03038 | Phone: (603)437-1610<br>Fax: (603) 437-0034<br>E Mail: nhb@nhboring.com | **Boring #** BB-3<br>Scale 1" = 5' |
|---|---|---|---|

| | | | |
|---|---|---|---|
| City/Town: Chatham | Bridge: C-07-001 | Project File No: 603690 | Contract No: 58732 |
| Project: Bridge Street over the Mitchell River | | Date & Time Started: 04-02-10 7:00AM | Total Hours: 21 |
| Groundwater Depth: 13.7' Tidal Date & Time: 04-06-10 7:00AM | | Date & Time Completed: 04-06-10 3:00PM | |
| Coordinates: N 2708522 E 1076660559 | Driller's Name: Roger Burne | Helper's Name: Patrick Schofield | |
| Ground Elevation: 10.7' | Inspector's Name: Alex Marinkovic | Inspector's Company: URS Corporation | |

| Sample Number | Depth Range (Feet) | Blow Counts per 6 Inches Coring Times Minute Per Foot | | | | Recovery Inches | Field Description | Strata Changes |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | Wooden BRIDGE DECK | * |
| | | | | | | | Rollerbit through wooden bridge deck, sedaments samples at surface of mudline 20'.<br><br>VOID - AIR. 13'6" | * 7" |
| | | | | | | | Top of WATER SURFACE | 13'7" |
| | | | | | | | Top of MUDLINE. | 20' |
| SD-1 | 20' - 22' | 2 | 1 | 3 | 4 | | FINE SAND, trace shells. | |
| SD-1 | 22' - 24' | 5 | 6 | 6 | 7 | | FINE SAND, trace shells. | |
| | | | | | | | Began sampling with 1 3/8" split spoon at 29'. | 29' |
| S-1 | 29' - 31' | 10 | 11 | 12 | 11 | 14" | Wet, medium dense, gray, FINE SAND AND INORGANIC SILT, trace fine to medium gravel. | |

Remarks: Sample with cathead.

Changed location; moved boring prior to drilling from N29708518, E1076661 to N2708522, E1076665, boring moved in order to avoid structural members in wooden bridge.

Arrow-Board:
Signs: 2
Cones: 15

Protective Device - Stand: Box:
Well Depth: Solid Pipe:
Stick Up Pipe: Screen Pipe:

Type of Drill Rig: Diedrich D-50

Casing HW Size: 4" Depth: 60'
Casing NW Size: 3" Depth: 138'
Hammer Weight: 300 lbs.
Fall: 30"

Sampler Type: S/S Size: 1 3/8"
Sediment Sample Size: 3"
Automatic Hammer Weight: 140 lbs.
Safety Hammer Weight:
Fall: 30"

Core Barrel Type: Size:

Penetration Resistance (N) Guide:

| Cohesionless Soils (Sands, Gravels) | | Cohesive Soils (Silts, Clays) | |
|---|---|---|---|
| Relative Density | Penetration Resistance | Consistency | Penetration Resistance |
| Very Loose | 0 - 4 | Very Soft | 0 - 2 |
| Loose | 4 - 10 | Soft | 2 - 4 |
| Medium Dense | 10 - 30 | Medium Stiff | 4 - 8 |
| Dense | 30 - 50 | Stiff | 8 - 15 |
| Very Dense | Over 50 | Very Stiff | 15 - 30 |
| N=Sum of Second and Third 6" Blow Counts | | Hard | Over 30 |

Terms Used for Second Entry of Descriptions: and = 40-50%, some = 10-40%, trace = 10% or less

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|---|---|---|---|

| | | | |
|---|---|---|---|
| City/Town: Chatham | Bridge: C-07-001 | Project File No: 603690 | Contract No: 58732 |
| Project: Bridge Street over the Mitchell River | | Date & Time Started: 04-02-10 7:00AM | Total Hours: 21 |
| Groundwater Depth: 13.7' Tidal Date & Time: 04-06-10 7:00AM | | Date & Time Completed: 04-06-10 3:00PM | |
| Coordinates: N 2708522 E 1076660559 | Driller's Name: Roger Burne | Helper's Name: Patrick Schofield | |
| Ground Elevation: 10.7' | Inspector's Name: Alex Marinkovic | Inspector's Company: URS Corporation | |

| Sample Number | Depth Range (Feet) | Blow Counts per 6 Inches Coring Times Minute Per Foot | | | | Recovery Inches | Field Description | Strata Changes |
|---|---|---|---|---|---|---|---|---|
| S-2 | 34' - 36' | 14 | 17 | 18 | 24 | 12" | Wet, medium dense, brown, FINE SAND, trace inorganic silt. | |
| S-3 | 39' - 41' | 11 | 12 | 12 | 17 | 18" | Wet, medium dense, gray, FINE SAND AND INORGANIC SILT. | |
| S-4 | 44' - 46' | 11 | 14 | 16 | 17 | 7" | Wet, medium dense, gray, FINE SAND, trace inorganic silt. | |
| S-5 | 49' - 51' | 6 | 6 | 7 | 12 | 12" | Wet, medium dense, gray, FINE TO COARSE SAND, trace inorganic silt. | |
| S-6 | 54' - 56' | 8 | 7 | 11 | 10 | 9" | Wet, medium dense, gray, FINE TO COARSE SAND, trace inorganic silt. | |
| S-7 | 59' - 61' | 7 | 7 | 7 | 10 | 18" | Wet, medium dense, gray, COARSE TO FINE SAND, trace inorganic silt.<br>Began sampling every 10' interval per onsite engineer. | |

Remarks: Sample with cathead.

Changed location; moved boring prior to drilling from N29708518, E1076661 to N2708522, E1076665, boring moved in order to avoid structural members in wooden bridge.

Arrow-Board:
Signs: 2
Cones: 15

Protective Device - Stand: Box:
Well Depth: Solid Pipe:
Stick Up Pipe: Screen Pipe:

Penetration Resistance (N) Guide:

| Cohesionless Soils (Sands, Gravels) | | Cohesive Soils (Silts, Clays) | |
|---|---|---|---|
| Relative Density | Penetration Resistance | Consistency | Penetration Resistance |
| Very Loose | 0 - 4 | Very Soft | 0 - 2 |
| Loose | 4 - 10 | Soft | 2 - 4 |
| Medium Dense | 10 - 30 | Medium Stiff | 4 - 8 |
| Dense | 30 - 50 | Stiff | 8 - 15 |
| Very Dense | Over 50 | Very Stiff | 15 - 30 |
| N=Sum of Second and Third 6" Blow Counts | | Hard | Over 30 |

Terms Used for Second Entry of Descriptions: and = 40-50%, some = 10-40%, trace = 10% or less

Type of Drill Rig: Diedrich D-50
Casing HW Size: 4" Depth: 60'
Casing NW Size: 3" Depth: 138'
Hammer Weight: 300 lbs.
Fall: 30"
Sampler Type: S/S Size: 1 3/8"
Sediment Sample Size: 3"
Automatic Hammer Weight: 140 lbs.
Safety Hammer Weight:
Fall: 30"
Core Barrel Type: Size:

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|---|---|---|---|

| | | | |
|---|---|---|---|
| City/Town: Chatham | Bridge: C-07-001 | Project File No: 603690 | Contract No: 58732 |
| Project: Bridge Street over the Mitchell River | | Date & Time Started: 04-02-10 7:00AM | Total Hours: 21 |
| Groundwater Depth: 13.7' Tidal Date & Time: 04-06-10 7:00AM | | Date & Time Completed: 04-06-10 3:00PM | |
| Coordinates: N 2708522 E 1076660559 | Driller's Name: Roger Burne | Helper's Name: Patrick Schofield | |
| Ground Elevation: 10.7' | Inspector's Name: Alex Marinkovic | Inspector's Company: URS Corporation | |

| Sample Number | Depth Range (Feet) | Blow Counts per 6 Inches Coring Times Minute Per Foot | | | | Recovery Inches | Field Description | Strata Changes |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | | 69' |
| S-8 | 69' - 71' | 11 | 12 | 32 | 87 | 14" | Wet, dense, gray, FINE SAND, trace inorganic silt. | |
| S-9 | 79' - 81' | 12 | 6 | 14 | 40 | 20" | Wet, medium dense, gray, FINE SAND, trace inorganic silt. | |
| S-10 | 89' - 91' | 17 | 11 | 19 | 38 | 24" | Wet, medium dense, gray, FINE SAND, trace inorganic silt. | |

Remarks: Sample with cathead.

Changed location; moved boring prior to drilling from N29708518, E1076661 to N2708522, E1076665, boring moved in order to avoid structural members in wooden bridge.

Arrow-Board:
Signs: 2
Cones: 15

Protective Device - Stand: Box:
Well Depth: Solid Pipe:
Stick Up Pipe: Screen Pipe:

Penetration Resistance (N) Guide:

| Cohesionless Soils (Sands, Gravels) | | Cohesive Soils (Silts, Clays) | |
|---|---|---|---|
| Relative Density | Penetration Resistance | Consistency | Penetration Resistance |
| Very Loose | 0 - 4 | Very Soft | 0 - 2 |
| Loose | 4 - 10 | Soft | 2 - 4 |
| Medium Dense | 10 - 30 | Medium Stiff | 4 - 8 |
| Dense | 30 - 50 | Stiff | 8 - 15 |
| Very Dense | Over 50 | Very Stiff | 15 - 30 |
| N=Sum of Second and Third 6" Blow Counts | | Hard | Over 30 |

Terms Used for Second Entry of Descriptions: and = 40-50%, some = 10-40%, trace = 10% or less

Type of Drill Rig: Diedrich D-50

Casing HW Size: 4" Depth: 60'
Casing NW Size: 3" Depth: 138'
Hammer Weight: 300 lbs.
Fall: 30"

Sampler Type: S/S Size: 1 3/8"
Sediment Sample Size: 3"
Automatic Hammer Weight: 140 lbs.
Safety Hammer Weight:
Fall: 30"

Core Barrel Type: Size:

| MassDOT | NEW HAMPSHIRE BORING, INC.<br>P.O. Box 165<br>Derry, NH 03038 | Phone: (603)437-1610<br>Fax: (603) 437-0034<br>E Mail: nhb@nhboring.com | **Boring #** BB-3<br>Scale 1" = 5' |
|---|---|---|---|

| | | | |
|---|---|---|---|
| City/Town: Chatham | Bridge: C-07-001 | Project File No: 603690 | Contract No: 58732 |
| Project: Bridge Street over the Mitchell River | | Date & Time Started: 04-02-10 7:00AM | Total Hours: 21 |
| Groundwater Depth: 13.7' Tidal Date & Time: 04-06-10 7:00AM | | Date & Time Completed: 04-06-10 3:00PM | |
| Coordinates: N 2708522 E 1076660559 | Driller's Name: Roger Burne | Helper's Name: Patrick Schofield | |
| Ground Elevation: 10.7' | Inspector's Name: Alex Marinkovic | Inspector's Company: URS Corporation | |

| Sample Number | Depth Range (Feet) | Blow Counts per 6 Inches Coring Times Minute Per Foot | | | | Recovery Inches | Field Description | Strata Changes |
|---|---|---|---|---|---|---|---|---|
| S-11 | 99' - 100' | 3 | 5 | 2 | 4 | 10" | Wet, loose, gray, FINE SAND with clay lenses. | 100' |
| S-11A | 100' - 101' | | | | | 10" | Wet, medium dense, gray, FINE SAND, trace inorganic silt. | |
| S-12 | 109' - 111' | 5 | 7 | 7 | 16 | 20" | Wet, medium dense, gray, FINE SAND, trace inorganic silt. | |
| S-13 | 119' - 121' | 7 | 6 | 8 | 11 | 21" | Wet, medium dense, gray, FINE SAND, trace inorganic silt. | |

Remarks: Sample with cathead.

Changed location; moved boring prior to drilling from N29708518, E1076661 to N2708522, E1076665, boring moved in order to avoid structural members in wooden bridge.

Arrow-Board:
Signs: 2
Cones: 15

Protective Device - Stand: Box:
Well Depth: Solid Pipe:
Stick Up Pipe: Screen Pipe:

Penetration Resistance (N) Guide:

| Cohesionless Soils (Sands, Gravels) | | Cohesive Soils (Silts, Clays) | |
|---|---|---|---|
| Relative Density | Penetration Resistance | Consistency | Penetration Resistance |
| Very Loose | 0 - 4 | Very Soft | 0 - 2 |
| Loose | 4 - 10 | Soft | 2 - 4 |
| Medium Dense | 10 - 30 | Medium Stiff | 4 - 8 |
| Dense | 30 - 50 | Stiff | 8 - 15 |
| Very Dense | Over 50 | Very Stiff | 15 - 30 |
| N=Sum of Second and Third 6" Blow Counts | | Hard | Over 30 |

Terms Used for Second Entry of Descriptions: and = 40-50%, some = 10-40%, trace = 10% or less

Type of Drill Rig: Diedrich D-50

Casing HW Size: 4" Depth: 60'
Casing NW Size: 3" Depth: 138'
Hammer Weight: 300 lbs.
Fall: 30"

Sampler Type: S/S Size: 1 3/8"
Sediment Sample Size: 3"
Automatic Hammer Weight: 140 lbs.
Safety Hammer Weight:
Fall: 30"

Core Barrel Type: Size:

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|---|---|---|---|

| | | | |
|---|---|---|---|
| City/Town: Chatham | Bridge: C-07-001 | Project File No: 603690 | Contract No: 58732 |
| Project: Bridge Street over the Mitchell River | | Date & Time Started: 04-02-10 7:00AM | Total Hours: 21 |
| Groundwater Depth: 13.7' Tidal Date & Time: 04-06-10 7:00AM | | Date & Time Completed: 04-06-10 3:00PM | |
| Coordinates: N 2708522 E 1076660559 | Driller's Name: Roger Burne | Helper's Name: Patrick Schofield | |
| Ground Elevation: 10.7' | Inspector's Name: Alex Marinkovic | Inspector's Company: URS Corporation | |

| Sample Number | Depth Range (Feet) | Blow Counts per 6 Inches Coring Times Minute Per Foot | | | | Recovery Inches | Field Description | Strata Changes |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | | 129' |
| S-14 | 129' - 131' | 14 | 25 | 41 | 24 | 20" | Wet, very dense, gray, FINE TO COARSE SAND. | |
| S-15 | 138' - 140' | 11 | 17 | 17 | 21 | 18" | Wet, dense, gray, FINE TO COARSE SAND. | 140' |
| | | | | | | | Bottom of Exploration = 140' | |

Remarks: Sample with cathead.

Changed location; moved boring prior to drilling from N29708518, E1076661 to N2708522, E1076665, boring moved in order to avoid structural members in wooden bridge.

Arrow-Board:
Signs: 2
Cones: 15

Protective Device - Stand: Box:
Well Depth: Solid Pipe:
Stick Up Pipe: Screen Pipe:

Penetration Resistance (N) Guide:

| Cohesionless Soils (Sands, Gravels) | | Cohesive Soils (Silts, Clays) | |
|---|---|---|---|
| Relative Density | Penetration Resistance | Consistency | Penetration Resistance |
| Very Loose | 0 - 4 | Very Soft | 0 - 2 |
| Loose | 4 - 10 | Soft | 2 - 4 |
| Medium Dense | 10 - 30 | Medium Stiff | 4 - 8 |
| Dense | 30 - 50 | Stiff | 8 - 15 |
| Very Dense | Over 50 | Very Stiff | 15 - 30 |
| N=Sum of Second and Third 6" Blow Counts | | Hard | Over 30 |

Terms Used for Second Entry of Descriptions: and = 40-50%, some = 10-40%, trace = 10% or less

Type of Drill Rig: Diedrich D-50

Casing HW Size: 4" Depth: 60'
Casing NW Size: 3" Depth: 138'
Hammer Weight: 300 lbs.
Fall: 30"

Sampler Type: S/S Size: 1 3/8"
Sediment Sample Size: 3"
Automatic Hammer Weight: 140 lbs.
Safety Hammer Weight:
Fall: 30"

Core Barrel Type: Size:

| massDOT | NEW HAMPSHIRE BORING, INC.<br>P.O. Box 165<br>Derry, NH 03038 | Phone: (603)437-1610<br>Fax: (603) 437-0034<br>E Mail: nhb@nhboring.com | **Boring #** BB-4<br>Scale 1" = 5' |
|---|---|---|---|

| | | | |
|---|---|---|---|
| City/Town: Chatham | Bridge: C-07-001 | Project File No: 603690 | Contract No: 58732 |
| Project: Bridge Street over the Mitchell River | | Date & Time Started: 04-01-10 7:00AM | Total Hours: 9 |
| Groundwater Depth: 13'1" Tidal | Date & Time: 04-01-10 7:00AM | Date & Time Completed: 04-01-10 4:00PM | |
| Coordinates: N 2708493 | E 1076585 | Driller's Name: Roger Burne | Helper's Name: Patrick Schofield |
| Ground Elevation: 9.3 | Inspector's Name: Alex Marinkovic | Inspector's Company: URS Corporation | |

| Sample Number | Depth Range (Feet) | Blow Counts per 6 Inches Coring Times Minute Per Foot | | | | Recovery Inches | Field Description | Strata Changes |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | WOODEN BRIDGE DECK 7" | * * 7" |
| | | | | | | | Rollerbit through wooden bridge deck to mudline at 14', began sampling at 14'.<br><br>VOID - AIR. 13'5" | |
| | | | | | | | MUDLINE | 14' |
| S-1 | 14' - 16' | 3 | 3 | 3 | 4 | 12" | Wet, loose, gray, FINE TO COARSE SAND, trace organic silt, trace shells. | 19' |
| S-2 | 19' - 21' | 1 | 1 | 1 | 1 | 18" | Wet, very soft, INORGANIC SILT AND FINE SAND. | |
| ST-1 | 21' - 23' | | | | | 24" | Shelby tube sample taken from 21' to 23'. | |
| S-3 | 24' - 26' | 7 | 11 | 11 | 16 | 20" | Wet, very stiff, gray, CLAY, trace fine sand seams. | |
| S-4 | 29' - 31' | 11 | 12 | 14 | 13 | 18" | Wet, medium dense, gray, FINE SAND AND INORGANIC SILT. | |

Remarks: * Drilling through bridge into water. Sample by cathead.
Changed location; moved boring prior to drilling from N2708495, E1076590 to N2708493, E1076585, boring moved in order to avoid structural members on wooden bridge. Pressure pushed 3" X 2' long shiby tube for sample.

Arrow-Board:
Signs: 2
Cones: 9

Protective Device - Stand: Box:
Well Depth: Solid Pipe:
Stick Up Pipe: Screen Pipe:

Penetration Resistance (N) Guide:

| Cohesionless Soils (Sands, Gravels) | | Cohesive Soils (Silts, Clays) | |
|---|---|---|---|
| Relative Density | Penetration Resistance | Consistency | Penetration Resistance |
| Very Loose | 0 - 4 | Very Soft | 0 - 2 |
| Loose | 4 - 10 | Soft | 2 - 4 |
| Medium Dense | 10 - 30 | Medium Stiff | 4 - 8 |
| Dense | 30 - 50 | Stiff | 8 - 15 |
| Very Dense | Over 50 | Very Stiff | 15 - 30 |
| N=Sum of Second and Third 6" Blow Counts | | Hard | Over 30 |

Type of Drill Rig: Diedrich D-50
Casing HW Size: 4" Depth: 64'
Hammer Weight: 300 lbs.
Fall: 30"
Sampler Type: S/S Size: 1 3/8"
Sampler Type for: ST=2"
Sediment Sample Size: 3"
Safety Hammer Weight: 140 lbs.
Fall: 30"
Core Barrel Type: Size:

Terms Used for Second Entry of Descriptions: and = 40-50%, some = 10-40%, trace = 10% or less

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|---|---|---|---|

| | | | |
|---|---|---|---|
| City/Town: Chatham | Bridge: C-07-001 | Project File No: 603690 | Contract No: 58732 |
| Project: Bridge Street over the Mitchell River | | Date & Time Started: 04-01-10 7:00AM | Total Hours: 9 |
| Groundwater Depth: 13'1" Tidal | Date & Time: 04-01-10 7:00AM | Date & Time Completed: 04-01-10 4:00PM | |
| Coordinates: N 2708493 | E 1076585 | Driller's Name: Roger Burne | Helper's Name: Patrick Schofield |
| Ground Elevation: 9.3 | Inspector's Name: Alex Marinkovic | Inspector's Company: URS Corporation | |

| Sample Number | Depth Range (Feet) | Blow Counts per 6 Inches Coring Times Minute Per Foot | | | | Recovery Inches | Field Description | Strata Changes |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | | 34' |
| S-5 | 34' - 36' | 16 | 17 | 24 | 24 | 14" | Wet, dense, brown, FINE TO COARSE SAND, trace organic silt. | |
| S-6 | 39' - 41' | 8 | 11 | 15 | 17 | 16" | Wet, medium dense, light brown, FINE TO COARSE SAND, trace inorganic silt. | 44' |
| S-7 | 44' - 46' | 5 | 6 | 8 | 12 | 12" | Wet, medium dense, gray, COARSE TO FINE SAND, trace inorganic silt. | 48' |
| S-8 | 49' - 51' | 9 | 15 | 22 | 27 | 20" | Wet, hard, gray, CLAY. | |
| S-9 | 54' - 56' | 12 | 18 | 27 | 28 | 24" | Wet, hard, gray, CLAY. | 59' |
| S-10 | 59' - 61' | 4 | 5 | 5 | 8 | 18" | Wet, medium dense, gray, FINE SAND AND INORGANIC SILT. | |

Remarks: * Drilling through bridge into water. Sample by cathead.
Changed location; moved boring prior to drilling from N2708495, E1076590 to N2708493, E1076585, boring moved in order to avoid structural members on wooden bridge. Pressure pushed 3" X 2' long shlby tube for sample.

Arrow-Board:
Signs: 2
Cones: 9

Protective Device - Stand: Box:
Well Depth: Solid Pipe:
Stick Up Pipe: Screen Pipe:

Penetration Resistance (N) Guide:

| Cohesionless Soils (Sands, Gravels) | | Cohesive Soils (Silts, Clays) | |
|---|---|---|---|
| Relative Density | Penetration Resistance | Consistency | Penetration Resistance |
| Very Loose | 0 - 4 | Very Soft | 0 - 2 |
| Loose | 4 - 10 | Soft | 2 - 4 |
| Medium Dense | 10 - 30 | Medium Stiff | 4 - 8 |
| Dense | 30 - 50 | Stiff | 8 - 15 |
| Very Dense | Over 50 | Very Stiff | 15 - 30 |
| N=Sum of Second and Third 6" Blow Counts | | Hard | Over 30 |

Type of Drill Rig: Diedrich D-50
Casing HW Size: 4" Depth: 64'
Hammer Weight: 300 lbs.
Fall: 30"
Sampler Type: S/S Size: 1 3/8"
Sampler Type for: ST=2"
Sediment Sample Size: 3"
Safety Hammer Weight: 140 lbs.
Fall: 30"
Core Barrel Type: Size:

Terms Used for Second Entry of Descriptions: and = 40-50%, some = 10-40%, trace = 10% or less

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|---|---|---|---|

| | | | |
|---|---|---|---|
| City/Town: Chatham | Bridge: C-07-001 | Project File No: 603690 | Contract No: 58732 |
| Project: Bridge Street over the Mitchell River | | Date & Time Started: 04-01-10 7:00AM | Total Hours: 9 |
| Groundwater Depth: 13'1" Tidal | Date & Time: 04-01-10 7:00AM | Date & Time Completed: 04-01-10 4:00PM | |
| Coordinates: N 2708493 | E 1076585 | Driller's Name: Roger Burne | Helper's Name: Patrick Schofield |
| Ground Elevation: 9.3 | Inspector's Name: Alex Marinkovic | Inspector's Company: URS Corporation | |

| Sample Number | Depth Range (Feet) | Blow Counts per 6 Inches Coring Times Minute Per Foot | Recovery Inches | Field Description | Strata Changes |
|---|---|---|---|---|---|
| S-11 | 64' - 66' | 13 16 17 16 | 20" | Wet, medium dense, gray, FINE SAND AND INORGANIC SILT. | 66' |
| | | | | Bottom of Exploration = 66' | |

Remarks: * Drilling through bridge into water. Sample by cathead.
Changed location; moved boring prior to drilling from N2708495, E1076590 to N2708493, E1076585, boring moved in order to avoid structural members on wooden bridge. Pressure pushed 3" X 2' long shlby tube for sample.

Arrow-Board:
Signs: 2
Cones: 9

Protective Device - Stand: Box:
Well Depth: Solid Pipe:
Stick Up Pipe: Screen Pipe:

Penetration Resistance (N) Guide:

| Cohesionless Soils (Sands, Gravels) | | Cohesive Soils (Silts, Clays) | |
|---|---|---|---|
| Relative Density | Penetration Resistance | Consistency | Penetration Resistance |
| Very Loose | 0 - 4 | Very Soft | 0 - 2 |
| Loose | 4 - 10 | Soft | 2 - 4 |
| Medium Dense | 10 - 30 | Medium Stiff | 4 - 8 |
| Dense | 30 - 50 | Stiff | 8 - 15 |
| Very Dense | Over 50 | Very Stiff | 15 - 30 |
| N=Sum of Second and Third 6" Blow Counts | | Hard | Over 30 |

Type of Drill Rig: Diedrich D-50
Casing HW Size: 4" Depth: 64'
Hammer Weight: 300 lbs.
Fall: 30"
Sampler Type: S/S Size: 1 3/8"
Sampler Type for: ST=2"
Sediment Sample Size: 3"
Safety Hammer Weight: 140 lbs.
Fall: 30"

Terms Used for Second Entry of Descriptions: and = 40-50%, some = 10-40%, trace = 10% or less

Core Barrel Type: Size:

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|---|---|---|---|

| | | | |
|---|---|---|---|
| City/Town: Chatham | Bridge: C-07-001 | Project File No: 603690 | Contract No: 58732 |
| Project: Bridge Street over the Mitchell River | | Date & Time Started: 04-01-10 7:00AM | Total Hours: 11.5 |
| Groundwater Depth: 13' Tidal | Date & Time: 04-01-10 7:30PM | Date & Time Completed: 04-01-10 6:30PM | |
| Coordinates: N 2708522 | E 1076627 | Driller's Name: Manlea Thompson | Helper's Name: Donald Palmer |
| Ground Elevation: 10.5' | Inspector's Name: Alex Marinkovic | Inspector's Company: URS Corporation | |

| Sample Number | Depth Range (Feet) | Blow Counts per 6 Inches Coring Times Minute Per Foot | | | | Recovery Inches | Field Description | Strata Changes |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | PAVEMENT * 6" | * |
| | | | | | | | CONCRETE 9" | 1'3" |
| | | | | | | | BRIDGE DECK 7". Rollerbit through bridge deck, sedaments samples at surface of mudline 20'.<br><br>VOID - AIR. 18'2" | 20' |
| SD-1 | 20' - 22' | 1 | 1 | 3 | 5 | 12" | Wet, loose, gray, FINE SAND, trace inorganic silt. | |
| SD-2 | 22' - 24 | 3 | 4 | 5 | 7 | 12" | Wet, loose, gray, FINE SAND, trace inorganic silt.<br><br>Began sampling with 1 3/8" split spoon at 25'. | 25' |
| S-1 | 25' - 27' | 3 | 4 | 4 | 5 | 14" | Wet, loose, gray, FINE SAND, trace inorganic silt. | 27'6" |
| S-2 | 30' - 32' | 5 | 8 | 10 | 12 | 20" | Wet, very stiff, gray, CLAY AND INORGANIC SILT, trace fine sand. | |

Remarks: Sample by cathead.
Changed location: moved boring prior to drilling from N29708522, E1076627 to N2708521, E1076627, boring moved in order to avoid structural members on a wooden bridge.

Arrow-Board:
Signs:
Cones:

Protective Device - Stand: Box:
Well Depth: Solid Pipe:
Stick Up Pipe: Screen Pipe:

Penetration Resistance (N) Guide:

| Cohesionless Soils (Sands, Gravels) | | Cohesive Soils (Silts, Clays) | |
|---|---|---|---|
| Relative Density | Penetration Resistance | Consistency | Penetration Resistance |
| Very Loose | 0 - 4 | Very Soft | 0 - 2 |
| Loose | 4 - 10 | Soft | 2 - 4 |
| Medium Dense | 10 - 30 | Medium Stiff | 4 - 8 |
| Dense | 30 - 50 | Stiff | 8 - 15 |
| Very Dense | Over 50 | Very Stiff | 15 - 30 |
| N=Sum of Second and Third 6" Blow Counts | | Hard | Over 30 |

Type of Drill Rig: CME-550X
Casing HW Size: 4" Depth: 95'
Hammer Weight: 300 lbs.
Fall: 30"
Sampler Type: S/S Size: 1 3/8"
Sampler Type for: S/S Sediment Sample Size: 3"
Automatic Hammer Weight: 140 lbs.
Fall: 30"

Terms Used for Second Entry of Descriptions: and = 40-50%, some = 10-40%, trace = 10% or less

Core Barrel Type: Size:

| massDOT | NEW HAMPSHIRE BORING, INC.<br>P.O. Box 165<br>Derry, NH 03038 | Phone: (603)437-1610<br>Fax: (603) 437-0034<br>E Mail: nhb@nhboring.com | **Boring #** BB-5<br>Scale 1" = 5' |
|---|---|---|---|

| | | | |
|---|---|---|---|
| City/Town: Chatham | Bridge: C-07-001 | Project File No: 603690 | Contract No: 58732 |
| Project: Bridge Street over the Mitchell River | | Date & Time Started: 04-01-10 7:00AM | Total Hours: 11.5 |
| Groundwater Depth: 13' Tidal | Date & Time: 04-01-10 7:30PM | Date & Time Completed: 04-01-10 6:30PM | |
| Coordinates: N 2708522 | E 1076627 | Driller's Name: Manlea Thompson | Helper's Name: Donald Palmer |
| Ground Elevation: 10.5' | Inspector's Name: Alex Marinkovic | Inspector's Company: URS Corporation | |

| Sample Number | Depth Range (Feet) | Blow Counts per 6 Inches Coring Times Minute Per Foot | | | | Recovery Inches | Field Description | Strata Changes |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | | 34' |
| S-3 | 35' - 37' | 8 | 11 | 11 | 14 | 13" | Wet, medium dense, gray, FINE SAND, trace inorganic silt. (Strong fuel odor contamination) | 38' |
| | | | | | | | CLAY area, determined by drill process. | 39' |
| S-4 | 40- 42 | 30 | 32 | 34 | 31 | 20" | Wet, very dense, brown, FINE SAND, some inorganic silt. | |
| S-5 | 45' - 47' | 6 | 11 | 14 | 14 | 13" | Wet, medium dense, gray, FINE TO COARSE SAND, trace inorganic silt. | |
| S-6 | 50' - 51' | 5 | 5 | 6 | 6 | 12" | Wet, medium dense, gray, FINE TO COARSE SAND, trace inorganic silt. | 51' |
| S-6A | 51' - 52' | | | | | 12" | Wet, stiff, gray, CLAY AND INORGANIC SILT, trace fine sand. | |
| S-7 | 55' - 57' | 10 | 12 | 20 | 22 | 24" | Wet, hard, gray, CLAY AND INORGANIC SILT, trace fine sand. | 57'6" |
| S-8 | 60' - 62' | 4 | 5 | 7 | 8 | 19" | Wet, medium dense, gray, FINE SAND, trace inorganic silt. | |

Remarks: Sample by cathead.
Changed location: moved boring prior to drilling from N29708522, E1076627 to N2708521, E1076627, boring moved in order to avoid structural members on a wooden bridge.

Arrow-Board:
Signs:
Cones:

Protective Device - Stand: Box:
Well Depth: Solid Pipe:
Stick Up Pipe: Screen Pipe:

Penetration Resistance (N) Guide:

| Cohesionless Soils (Sands, Gravels) | | Cohesive Soils (Silts, Clays) | |
|---|---|---|---|
| Relative Density | Penetration Resistance | Consistency | Penetration Resistance |
| Very Loose | 0 - 4 | Very Soft | 0 - 2 |
| Loose | 4 - 10 | Soft | 2 - 4 |
| Medium Dense | 10 - 30 | Medium Stiff | 4 - 8 |
| Dense | 30 - 50 | Stiff | 8 - 15 |
| Very Dense | Over 50 | Very Stiff | 15 - 30 |
| N=Sum of Second and Third 6" Blow Counts | | Hard | Over 30 |

Type of Drill Rig: CME-550X
Casing HW Size: 4" Depth: 95'
Hammer Weight: 300 lbs.
Fall: 30"
Sampler Type: S/S Size: 1 3/8"
Sampler Type for: S/S Sediment Sample Size: 3"
Automatic Hammer Weight: 140 lbs.
Fall: 30"

Terms Used for Second Entry of Descriptions: and = 40-50%, some = 10-40%, trace = 10% or less

Core Barrel Type: Size:

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|---|---|---|---|

| | | | |
|---|---|---|---|
| City/Town: Chatham | Bridge: C-07-001 | Project File No: 603690 | Contract No: 58732 |
| Project: Bridge Street over the Mitchell River | | Date & Time Started: 04-01-10 7:00AM | Total Hours: 11.5 |
| Groundwater Depth: 13' Tidal | Date & Time: 04-01-10 7:30PM | Date & Time Completed: 04-01-10 6:30PM | |
| Coordinates: N 2708522 | E 1076627 | Driller's Name: Manlea Thompson | Helper's Name: Donald Palmer |
| Ground Elevation: 10.5' | Inspector's Name: Alex Marinkovic | Inspector's Company: URS Corporation | |

| Sample Number | Depth Range (Feet) | Blow Counts per 6 Inches Coring Times Minute Per Foot | | | | Recovery Inches | Field Description | Strata Changes |
|---|---|---|---|---|---|---|---|---|
| S-9 | 65' - 67' | 12 | 16 | 29 | 31 | 15" | Wet, dense, gray, FINE SAND, trace inorganic silt. | |
| S-10 | 70' - 72' | 11 | 12 | 21 | 23 | 18" | Wet, dense, gray, FINE SAND, trace inorganic silt. | |
| S-11 | 75' - 77' | 11 | 16 | 26 | 34 | 24" | Wet, dense, gray, FINE SAND, trace inorganic silt. | |
| S-12 | 80' - 82' | 10 | 13 | 20 | 31 | 17" | Wet, dense, gray, FINE SAND, trace inorganic silt. | |
| S-13 | 85' - 90' | 15 | 18 | 22 | 30 | 14" | Wet, dense, gray, FINE SAND, trace inorganic silt. | |
| S-14 | 90' - 92' | 13 | 14 | 19 | 24 | 16" | Wet, dense, gray, FINE SAND, trace inorganic silt. | |

Remarks: Sample by cathead.
Changed location: moved boring prior to drilling from N29708522, E1076627 to N2708521, E1076627, boring moved in order to avoid structural members on a wooden bridge.

Arrow-Board:
Signs:
Cones:

Protective Device - Stand: Box:
Well Depth: Solid Pipe:
Stick Up Pipe: Screen Pipe:

Penetration Resistance (N) Guide:

| Cohesionless Soils (Sands, Gravels) | | Cohesive Soils (Silts, Clays) | |
|---|---|---|---|
| Relative Density | Penetration Resistance | Consistency | Penetration Resistance |
| Very Loose | 0 - 4 | Very Soft | 0 - 2 |
| Loose | 4 - 10 | Soft | 2 - 4 |
| Medium Dense | 10 - 30 | Medium Stiff | 4 - 8 |
| Dense | 30 - 50 | Stiff | 8 - 15 |
| Very Dense | Over 50 | Very Stiff | 15 - 30 |
| N=Sum of Second and Third 6" Blow Counts | | Hard | Over 30 |

Type of Drill Rig: CME-550X
Casing HW Size: 4" Depth: 95'
Hammer Weight: 300 lbs.
Fall: 30"
Sampler Type: S/S Size: 1 3/8"
Sampler Type for: S/S Sediment Sample Size: 3"
Automatic Hammer Weight: 140 lbs.
Fall: 30"

Terms Used for Second Entry of Descriptions: and = 40-50%, some = 10-40%, trace = 10% or less

Core Barrel Type: Size:

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|---|---|---|---|
| City/Town: Chatham | Bridge: C-07-001 | Project File No: 603690 | Contract No: 58732 |
| Project: Bridge Street over the Mitchell River | | Date & Time Started: 04-01-10 7:00AM | Total Hours: 11.5 |
| Groundwater Depth: 13' Tidal | Date & Time: 04-01-10 7:30PM | Date & Time Completed: 04-01-10 6:30PM | |
| Coordinates: N 2708522 | E 1076627 | Driller's Name: Manlea Thompson | Helper's Name: Donald Palmer |
| Ground Elevation: 10.5' | Inspector's Name: Alex Marinkovic | Inspector's Company: URS Corporation | |

| Sample Number | Depth Range (Feet) | Blow Counts per 6 Inches Coring Times Minute Per Foot | Recovery Inches | Field Description | Strata Changes |
|---|---|---|---|---|---|
| S-15 | 95' - 97' | 11 17 23 25 | 12" | Wet, dense, gray, FINE SAND, trace inorganic silt. | 97' |
| | | | | Bottom of Exploration = 97' | |

Remarks: Sample by cathead.
Changed location: moved boring prior to drilling from N29708522, E1076627 to N2708521, E1076627, boring moved in order to avoid structural members on a wooden bridge.

Arrow-Board:
Signs:
Cones:

Protective Device - Stand: Box:
Well Depth: Solid Pipe:
Stick Up Pipe: Screen Pipe:

Penetration Resistance (N) Guide:

| Cohesionless Soils (Sands, Gravels) | | Cohesive Soils (Silts, Clays) | |
|---|---|---|---|
| Relative Density | Penetration Resistance | Consistency | Penetration Resistance |
| Very Loose | 0 - 4 | Very Soft | 0 - 2 |
| Loose | 4 - 10 | Soft | 2 - 4 |
| Medium Dense | 10 - 30 | Medium Stiff | 4 - 8 |
| Dense | 30 - 50 | Stiff | 8 - 15 |
| Very Dense | Over 50 | Very Stiff | 15 - 30 |
| N=Sum of Second and Third 6" Blow Counts | | Hard | Over 30 |

Type of Drill Rig: CME-550X
Casing HW Size: 4" Depth: 95'
Hammer Weight: 300 lbs.
Fall: 30"
Sampler Type: S/S Size: 1 3/8"
Sampler Type for: S/S Sediment Sample Size: 3"
Automatic Hammer Weight: 140 lbs.
Fall: 30"
Core Barrel Type: Size:

Terms Used for Second Entry of Descriptions: and = 40-50%, some = 10-40%, trace = 10% or less

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URS BOSTON

URS Corporation
260 Franklin Street, Suite 300
Boston, MA 02110-3197
Attn: Mr. Aleksander Marinkovi

Report Date: 4/22/10

Project: **Mitchell River Bridge-Chatham, MA**
Briggs #:

Tested: 4/16/10
Received: 4/2/10

| 1. Sample No. | Description | Source of Material |
|---|---|---|
| M-18747 | Clayey-Silty Sand | BB-4 (21"-23") |

2. Particle Size Analysis {ASTM D422}

| Sieve Size Standard | Sieve Size Alternate | Results {% Passing by Wt.} | Specifications |
|---|---|---|---|
| 100 mm | 4" | 100 | |
| 90 mm | 3-1/2" | 100 | |
| 75 mm | 3" | 100 | |
| 63 mm | 2-1/2" | 100 | |
| 50 mm | 2" | 100 | |
| 37.5 mm | 1-1/2" | 100 | |
| 25 mm | 1 | 99 | |
| 19 mm | 3/4" | 99 | |
| 12.5 mm | 1/2" | 99 | |
| 9.5 mm | 3/8" | 98 | |
| 4.75 mm | #4 | 98 | |
| 2.00 mm | #10 | 98 | |
| 0.850 mm | #20 | 97 | |
| 0.425 mm | #40 | 96 | |
| 0.250 mm | #60 | 93 | |
| 0.106 mm | #140 | 82 | |
| 0.053 mm | #270 | 68 | |
| 0.0326 mm | | 57 | |
| 0.0207 mm | | 44 | |
| 0.0127 mm | | 32 | |
| 0.0092 mm | | 26 | |
| 0.0066 mm | | 20 | |
| 0.0034 mm | | 11 | |
| 0.0014 mm | | 7 | |

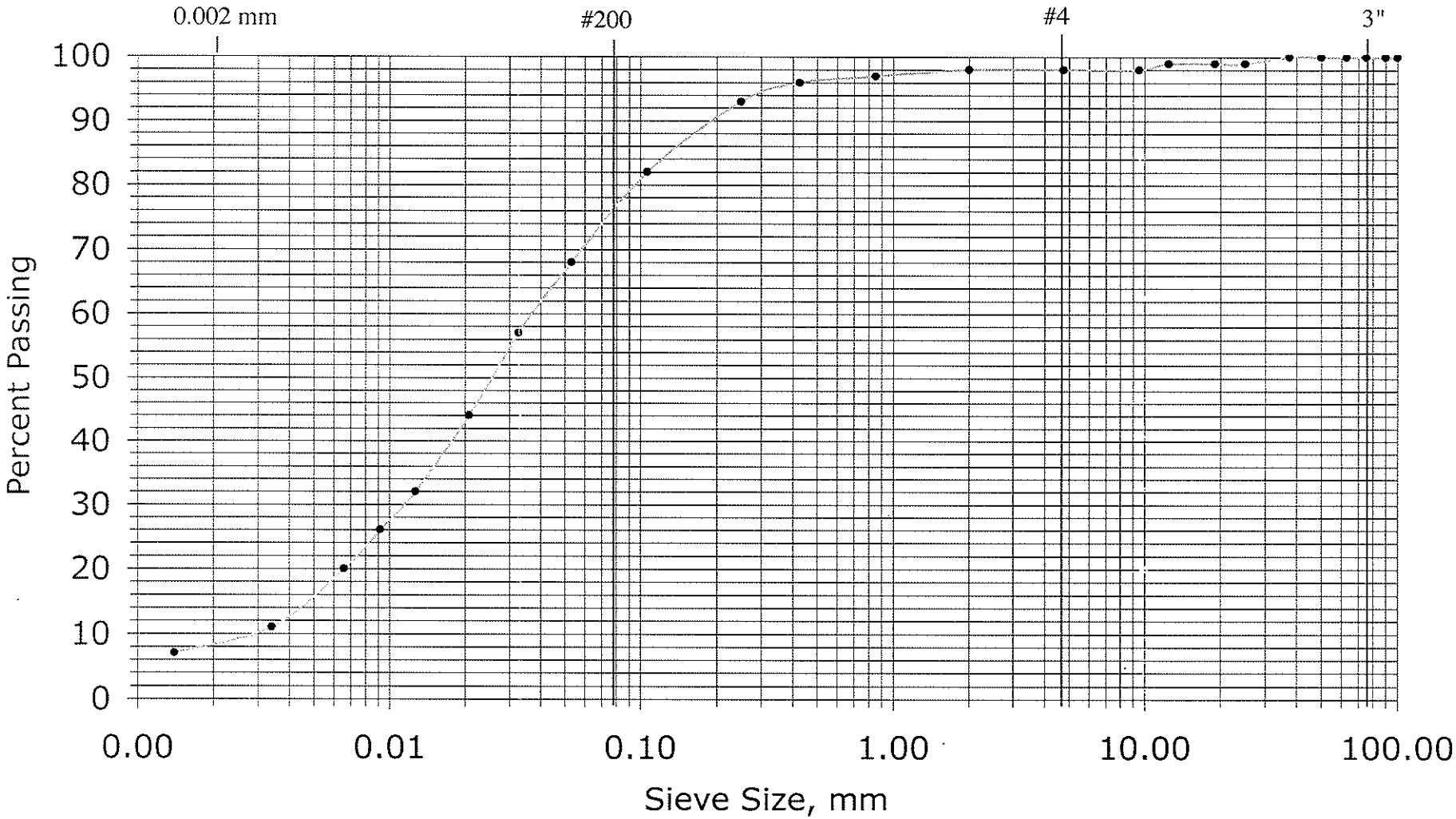
BRIGGS ENGINEERING & TESTING
*A Division of PK Associates, Inc.*

Sean Skorohod
Director of Testing Services
Construction Technology Division

Briggs Engineering & Testing
A Division of PK Associates, Inc.

| | |
|---|---|
| Project: | Mitchell River Bridge-Chatham, MA |
| Date Tested: | 4/16/10 |
| Lab Ref. No.: | M-18747 |

## Particle Size Analysis



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*A Division of PK Associates, Inc.*

URS Corporation
260 Franklin Street, Suite 300
Boston, MA 02110-3197
Attn: Mr. Aleksander Marinkovi

Report Date: 4/22/10

Project: **Mitchell River Bridge-Chatham, MA**
Briggs #:

Tested: 4/16/10
Received: 4/2/10

1.

| Sample No. | Description | Source of Material |
|---|---|---|
| M-18748 | Silty Sand | BB-3 (0-4')<br>Rec. 24" x2 runs |

2. Particle Size Analysis {ASTM D422}

| Sieve Size | | Results | Specifications |
|---|---|---|---|
| Standard | Alternate | {% Passing by Wt.} | |
| 100 mm | 4" | 100 | |
| 90 mm | 3-1/2" | 100 | |
| 75 mm | 3" | 100 | |
| 63 mm | 2-1/2" | 100 | |
| 50 mm | 2" | 100 | |
| 37.5 mm | 1-1/2" | 100 | |
| 25 mm | 1 | 100 | |
| 19 mm | 3/4" | 99 | |
| 12.5 mm | 1/2" | 98 | |
| 9.5 mm | 3/8" | 98 | |
| 4.75 mm | #4 | 96 | |
| 2.00 mm | #10 | 90 | |
| 0.850 mm | #20 | 80 | |
| 0.425 mm | #40 | 64 | |
| 0.250 mm | #60 | 44 | |
| 0.106 mm | #140 | 12 | |
| 0.053 mm | #270 | 7 | |
| 0.0376 mm | | 6 | |
| 0.0240 mm | | 5 | |
| 0.0139 mm | | 4 | |
| 0.0099 mm | | 3 | |
| 0.0070 mm | | 3 | |
| 0.0035 mm | | 1 | |
| 0.0015 mm | | 0 | |

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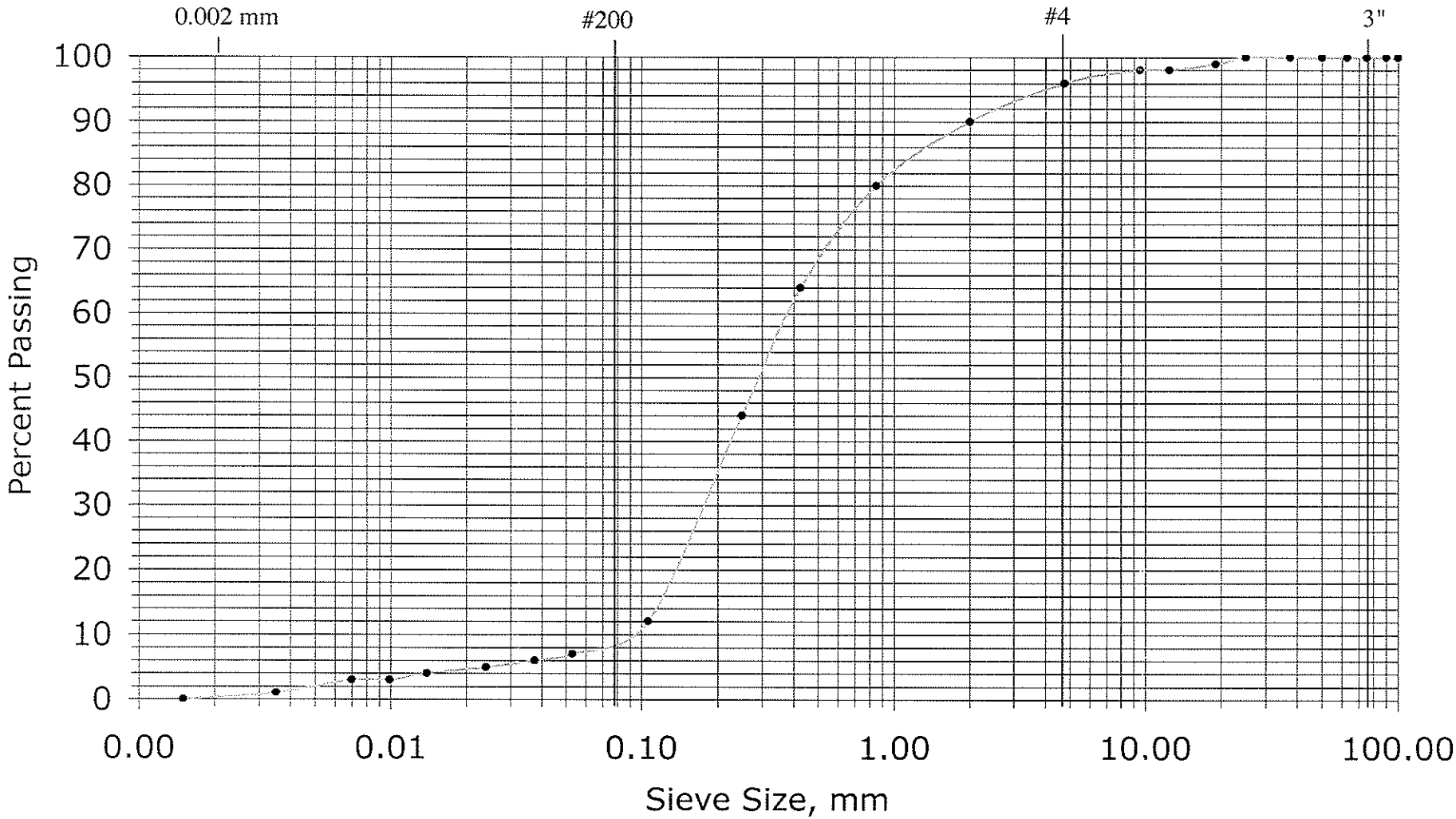
| | |
|---|---|
| Project: | Mitchell River Bridge-Chatham, MA |
| Date Tested: | 4/16/10 |
| Lab Ref. No.: | M-18748 |

## Particle Size Analysis



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Report Date: 4/22/10

Project: **Mitchell River Bridge-Chatham, MA**
Briggs #:

Tested: 4/16/10
Received: 4/2/10

| 1. Sample No. | Description | Source of Material |
|---|---|---|
| M-18749 | Silty Sand | BB-5 (22-24') Rec=12" |

2. Particle Size Analysis {ASTM D422}

| Sieve Size | | Results | Specifications |
|---|---|---|---|
| Standard | Alternate | {% Passing by Wt.} | |
| 100 mm | 4" | 100 | |
| 90 mm | 3-1/2" | 100 | |
| 75 mm | 3" | 100 | |
| 63 mm | 2-1/2" | 100 | |
| 50 mm | 2" | 100 | |
| 37.5 mm | 1-1/2" | 100 | |
| 25 mm | 1 | 100 | |
| 19 mm | 3/4" | 99 | |
| 12.5 mm | 1/2" | 99 | |
| 9.5 mm | 3/8" | 99 | |
| 4.75 mm | #4 | 99 | |
| 2.00 mm | #10 | 97 | |
| 0.850 mm | #20 | 94 | |
| 0.425 mm | #40 | 86 | |
| 0.250 mm | #60 | 68 | |
| 0.106 mm | #140 | 35 | |
| 0.053 mm | #270 | 23 | |
| 0.0348 mm | | 18 | |
| 0.0231 mm | | 11 | |
| 0.0137 mm | | 6 | |
| 0.0098 mm | | 5 | |
| 0.0070 mm | | 3 | |
| 0.0035 mm | | 2 | |
| 0.0014 mm | | 1 | |

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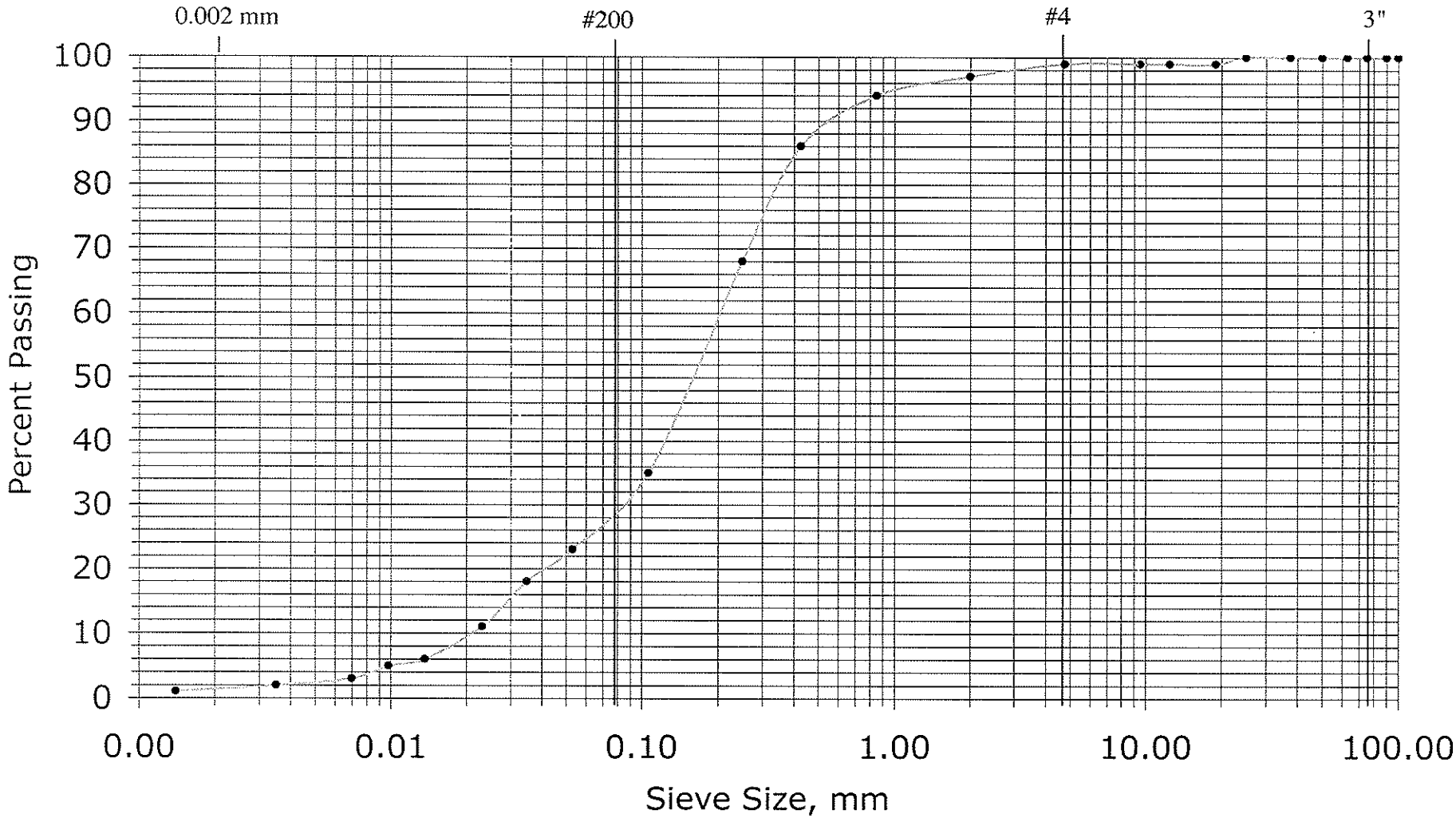
|---|---|
| Project: | Mitchell River Bridge-Chatham, MA |
| Date Tested: | 4/16/10 |
| Lab Ref. No.: | M-18749 |

## Particle Size Analysis



# APPENDIX E

## PILE CAPACITY CALCULATIONS

OBJECTIVE: ESTIMATE FACTORED RESISTANCE OF PIPE PILE FOUNDATION FOR AXIAL COMPRESSION AND UPLIFT

METHOD: CHECK CAPACITY BASED GEOTECHNICAL RESISTANCE

REFERENCE:

* AASHTO, 2007 LRFD BRIDGE DESIGN SPECIFICATIONS, 2008 INTERIM REVISIONS, 4TH EDITION.
* ASCE, 1993 DESIGN OF PILE FOUNDATIONS, TM/DESIGN GUIDES, USACOE NO. 1

→ GEOTECHNICAL AXIAL RESISTANCE:

ASSUMPTIONS: IGNORE SOIL RESISTANCE IN FILL, RIVER BOTTOM MATERIAL & 100-YR DESIGN FLOOD SCOUR ZONE

FOR ABUTMENTS → IGNORE RESISTANCE ABOVE EL. -10 FT
FOR APPROACH & REST PIERS → IGNORE RESISTANCE ABOVE EL. -15 FT
FOR BASCULE SPAN PIER → IGNORE RESISTANCE ABOVE EL. -21 FT

Resistance Factors

$$R_R = \phi_{qp}\, q_p A_p + \phi_{qs}\, q_s A_s \quad \text{- AASHTO 10.7.3.8.6a-2}$$

$R_R$: Factored resistance.
$q_p A_p$: tip resistance (end bearing)
$q_s A_s$: side resistance (friction)

* Side resistance in clay: Use α Method

SPT-N Values from 9 to 45, with average N = 25 blows/ft. → Very stiff.

$$q_s = \alpha\, S_u$$

$S_u$: average undrained shear strength
$\alpha$: adhesion factor

Reference

*Table 45.2*
***Relation of Consistency of Clay, Number of Blows N on Sampling Spoon, and Unconfined Compressive Strength***

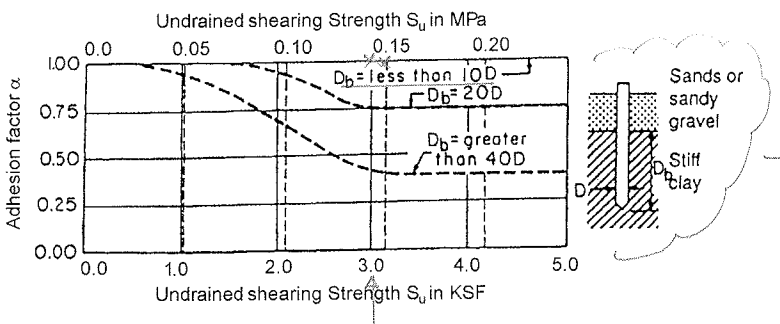
Ref. Terzaghi & Peck, Soil Mechanics in Engineering Practice

| Consistency | Very Soft | Soft | Medium | Stiff | Very Stiff | Hard |
|---|---|---|---|---|---|---|
| $N$ | <2 | 2–4 | 4–8 | 8–15 | 15–30 | >30 |
| $q_u$ (tons/ft² → Su (Ksf)) | <0.25 | 0.25–0.50 | 0.50–1.00 | 1.00–2.00 | 2.00–4.00 | >4.00 |

use Su ≈ 3.0 Ksf

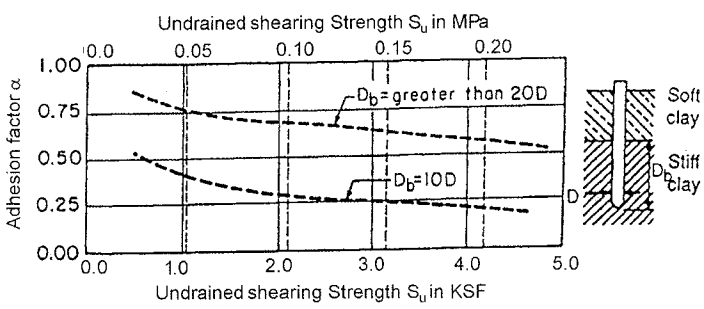
clay layer thickness is up to ~13 feet

Assume 14-inch diameter piles → $\frac{D_b}{D} \approx \frac{(13)(12)}{14} \approx 11$ → use $D_b \leq 10D$ (for charts below)




→ α = 1.00

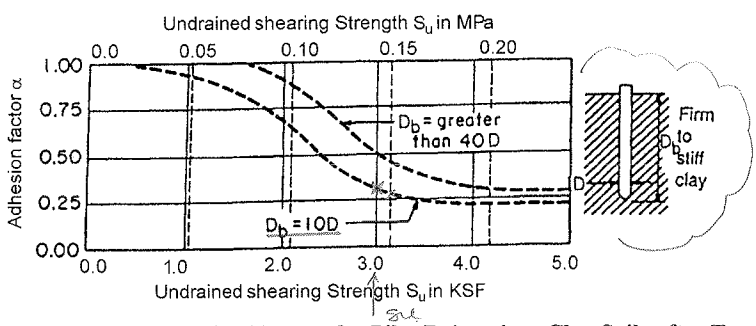
Su




Average α = 0.65

use α = 0.5 – ASCE

AASHTO LRFD




→ α = 0.30

Su

**Figure 10.7.3.8.6b-1 Design Curves for Adhesion Factors for Piles Driven into Clay Soils after Tomlinson (*1980*).**

* Side resistance in clay (cont'd)

$$q_s = \alpha S_u = (0.5)(3.0 \text{ ksf}) = 1.5 \text{ ksf.}$$

* Side resistance in sand : use Nordlund / Thurman Method

For Medium Dense Sand → above El. -55 ft → SPT-N values from 6 to 66
(silty fine sand to well graded sand) with average N ≈ 24 blows/ft

$\phi_f = 30° - 35°$ → use $\phi_f \approx 32°$ (soil friction angle)

For Dense Sand → below El. -55 ft → SPT-N values from 5 to 108
(mostly uniform fine sand) with average N ≈ 37 blows/ft

$\phi_f = 34° - 38°$ → use $\phi_f \approx 34°$ (soil friction angle)
↑ conservative

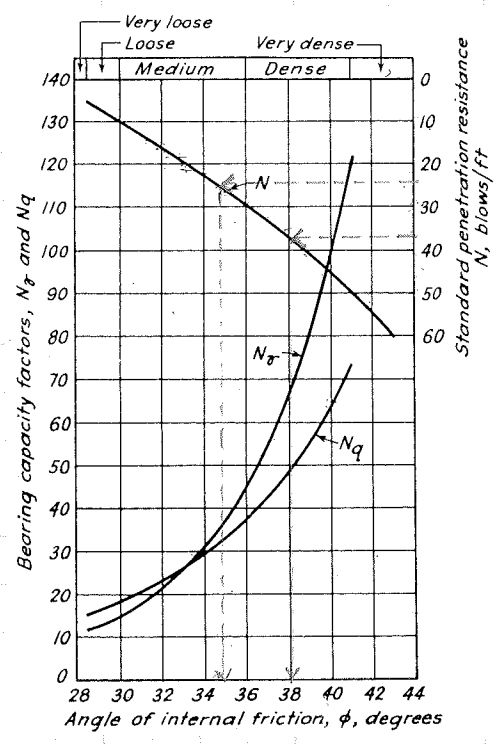



**Table 4.1 Representative Values of $\phi_d$ for Sands and Silts**

| Material | Degrees | |
|---|---|---|
| | Loose | Dense |
| Sand, round grains, uniform | 27.5 | 34 |
| Sand, angular grains, well graded | 33 | 45 |
| Sandy gravel | 35 | 50 |
| Silty sand | 27–33 | 30–35 |
| Inorganic silt | 27–30 | 30–34 |

Ref. Peck, Hanson & Thornburn - Foundation Engineering

* Side resistance in sand (cont'd)

$$q_s = K_\delta C_F \sigma_v' \frac{\sin(\delta+\omega)}{\cos\omega}$$ - AASHTO 10.7.3.8.6f-1

$\omega = 0°$ for straight pipe piles

$V$ = pile displacement volume

$= \frac{1}{4}\pi\left(\frac{14}{12}\right)^2 = 1.1\ ft^3/ft$

→ for 14" diameter pipe pile (assume closed end/plugged)

$K_\delta = 1.15$ for $\phi_f = 30°$
$K_\delta = 1.75$ for $\phi_f = 35°$ } Interpolate

For $\phi_f = 32°$ (med. dense) → $K_\delta = 1.4$
For $\phi_f = 34°$ (dense) → $K_\delta = 1.6$

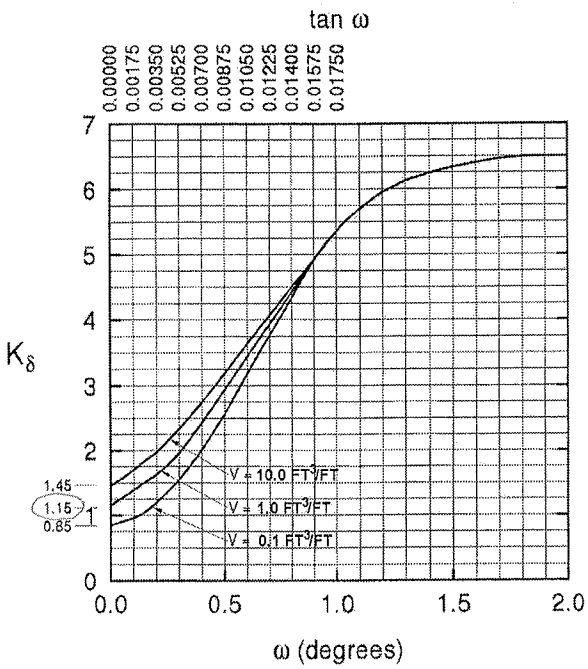



**Figure 10.7.3.8.6f-2 Design Curve for Evaluating $K_\delta$ for Piles where $\phi_f = 30°$ (*Hannigan et al., 2005* after *Nordlund, 1979*).**

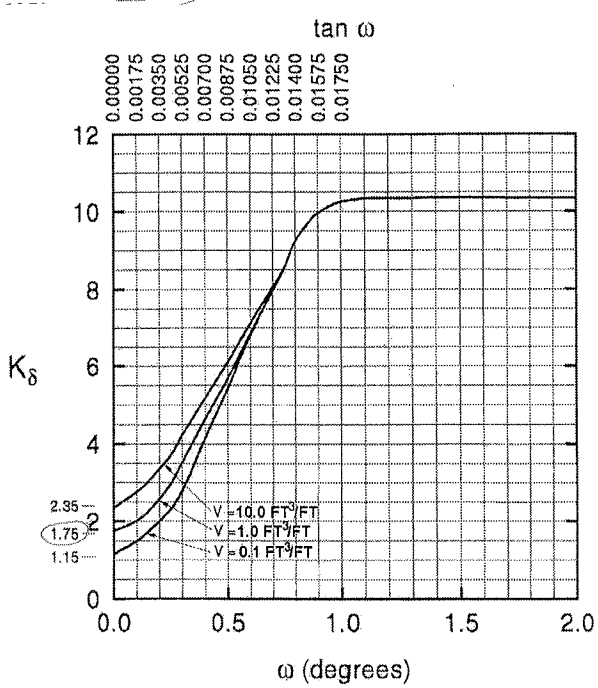



**Figure 10.7.3.8.6f-3 Design Curve for Evaluating $K_\delta$ for Piles where $\phi_f = 35°$ (*Hannigan et al., 2005* after *Nordlund, 1979*).**

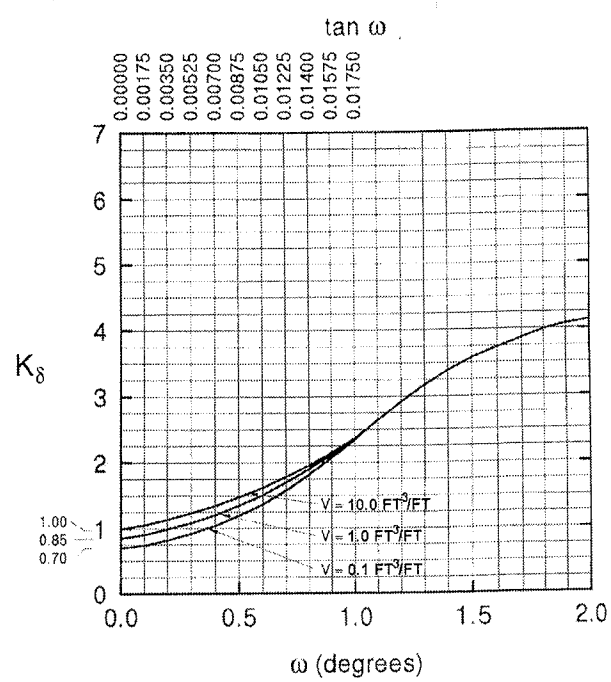
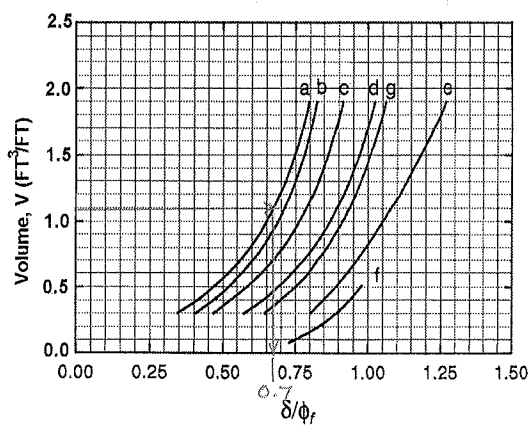



**Figure 10.7.3.8.6f-1 Design Curve for Evaluating $K_\delta$ for Piles where $\phi_f = 25°$ (*Hannigan et al., 2005* after *Nordlund, 1979*).**




a. Closed end pipe and non-tapered portion of monotube piles
b. Timber piles
c. Precast concrete piles
d. Raymond step-taper piles
e. Raymond uniform taper piles
f. H-piles
g. Tapered portion of monotube piles

**Figure 10.7.3.8.6f-6 Relation of $\delta/\phi_f$ and Pile Displacement, *V*, for Various Types of Piles (*Hannigan et al., 2005* after *Nordlund, 1979*).**

For closed end pipe with $V \approx 1.1\ ft^3/ft$

→ soil/pile interface friction angle

$\delta/\phi_f = 0.7$

→ soil friction angle

For $\phi_f = 32°$ (medium dense)

→ $\delta = 22°$

For $\phi_f = 34°$ (dense)

→ $\delta = 24°$

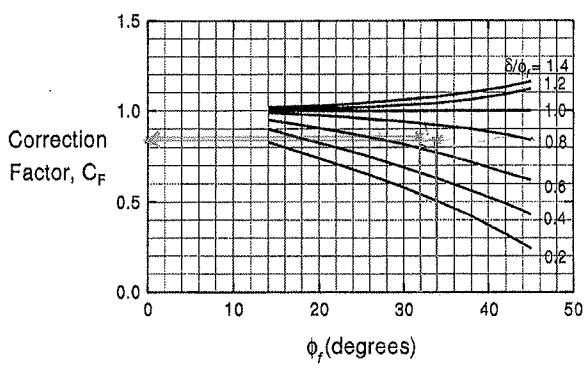



**Figure 10.7.3.8.6f-5 Correction Factor for $K_\delta$ where $\delta \neq \phi_f$ (*Hannigan et al., 2005* after *Nordlund, 1979*).**

For $\delta/\phi_f = 0.7$

→ med. dense

$\phi_f = 32°$ & $34°$

→ dense

→ $C_F \approx 0.85$

$\sigma_v'$ = effective overburden stress at midpoint of soil layer

↓

at El. −35 ft for med. dense layer

El. −60 ft for dense layer

(assume 10 ft penetration)

$= 25\,(125 - 62.4) = 1565$ psf (lowest mudline at El. −10 ft) For med. dense layer

$\gamma_{effective}$

$= 50\,(125 - 62.4) = 3130$ psf For dense layer

* Side resistance in sand (cont'd)

$q_s = K_s C_F \sigma_v' \sin\delta$ (for $\omega = 0°$ – non tapered piles)

$= (1.4)(0.85)(1565)\sin 22° = 698$ psf (Say 700 psf for med. dense sand)

$= (1.6)(0.85)(3130)\sin 24° = 1{,}731$ psf (Say 1,700 psf for dense sand.)

* Tip resistance in dense sand: use Nordlund / Thurman method

$q_p = \alpha_t N_q' \sigma_v' \leq q_L$ AASHTO 10.7.3.8.6f-2

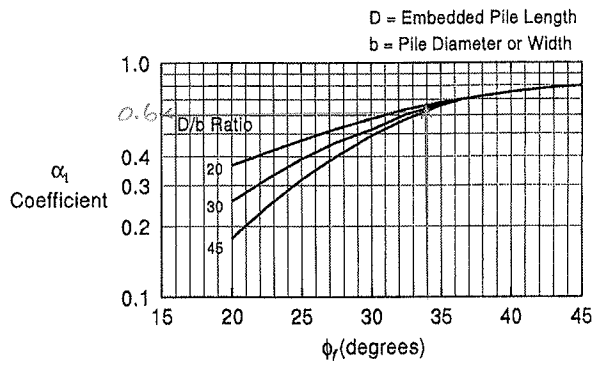



**Figure 10.7.3.8.6f-7 $\alpha_t$ Coefficient (*Hannigan et al., 2005* modified after *Bowles, 1977*).**

minimum pile embedment ~ 55 ft (from El. –10 to –65 ft)

D/b ratio $\approx \frac{55}{14/12} = 47$ (assume 14" pile)

∴ use D/b ratio of 45 ±

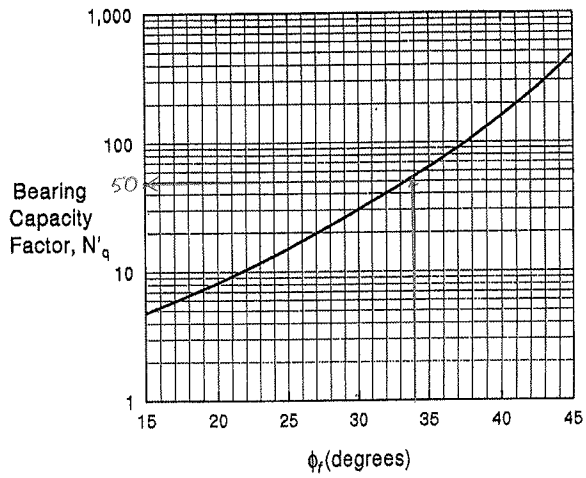
For $\phi_f = 34°$ (dense sand) → $\alpha_t \approx 0.6$

$\sigma_v'$ = effective overburden stress at pile tip (assume mudline at El. –10 ft)

$= 55(\underbrace{125 - 62.4}_{\gamma_{effective}}) = 3443$ psf > 3.2 ksf (max.)

→ use $\sigma_v' = 3.2$ ksf (max).

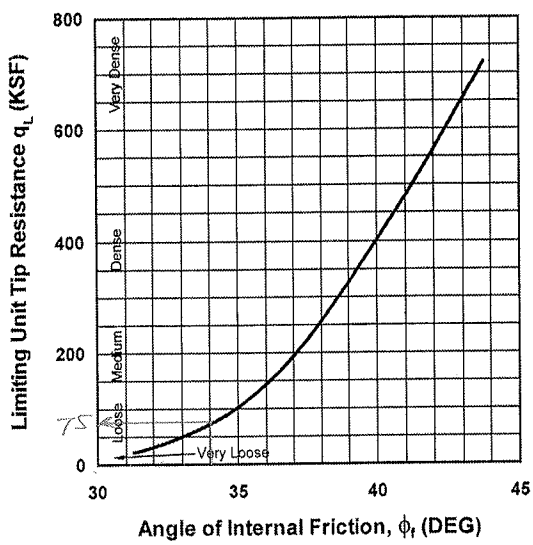
* Tip resistance in dense sand (cont'd):




Bearing capacity factor

$N_q' = 50$ - for $\phi_f \approx 34°$

**Figure 10.7.3.8.6f-8 Bearing Capacity Factor, $N'_q$, (*Hannigan et al., 2005* modified after *Bowles, 1977*).**




Limiting tip resistance

$q_L = 75$ ksf.

**Figure 10.7.3.8.6f-9 Limiting Unit Pile Tip Resistance (*Hannigan et al., 2005* after *Meyerhof, 1976*).**

$$q_p = \alpha_t N_q' \sigma_v' \leq q_L$$

$$= (0.6)(50)(3.2) = 96 \text{ ksf} > 75 \text{ ksf}$$

$\rightarrow$ use $q_p = 75$ ksf.

Reference

→ GEOTECHNICAL AXIAL RESISTANCE IN COMPRESSION

$$R_R = \phi_{qp}\, q_p A_p + \phi_{qs}\, q_s A_s$$ – AASHTO 10.7.3.8.6a-2

→ GEOTECHNICAL UPLIFT RESISTANCE IN TENSILE

$$R_R = \phi_{up}\, q_s A_s$$ – AASHTO 10.7.3.10-1

where:

$A_p = \frac{1}{4}\pi D^2$ (D: pile diameter)

$A_s = \pi D L$ (L: depth/length)

$q_s$: Side resistance (friction)

$\phi$: Resistance factors.

| Stratum | $q_s$ (Ksf) | $q_p$ (Ksf) | Resistance Factors (AASHTO Table 10.5.5.2.3-1) | | | |
|---|---|---|---|---|---|---|
| | | | Strength Limit State | | Extreme Limit State | |
| | | | Compression | Uplift | Compression | Uplift |
| Med. Dense Sand | 0.7 | — | 0.45 | 0.35 | 1.0 | 0.8 |
| V. Stiff clay | 1.5 | — | 0.35 | 0.25 | 1.0 | 0.8 |
| Dense Sand | 1.7 | 75 | 0.45 | 0.35 | 1.0 | 0.8 |

* Axial Resistance:

$$R_R = \pi D\,(8.4D + 0.32 L_{MD\,sand} + 0.53 L_{vs\,clay} + 0.77 L_{D\,sand})$$ Strength LS

$$= \pi D\,(18.7D + 0.7 L_{MD\,sand} + 1.5 L_{vs\,clay} + 1.7 L_{D\,sand})$$ Extreme LS

* Uplift Resistance:

$$R_R = \pi D\,(0.25 L_{MD\,sand} + 0.38 L_{vs\,clay} + 0.6 L_{D\,sand})$$ Strength LS

$$= \pi D\,(0.56 L_{MD\,sand} + 1.2 L_{vs\,clay} + 1.36 L_{D\,sand})$$ Extreme LS

**CHATHAM BRIDGE**
**ESTIMATES OF PILE FACTORED RESISTANCE - AXIAL COMPRESSION**

| Soil Strata | Unit Side Resistance $q_s$ (ksf) | Unit Tip Resistance $q_p$ (ksf) | Resistance Factors | |
|---|---|---|---|---|
| | | | Strength Limit State | Extreme Limit State |
| Medium Dense Sand | 0.7 | -- | 0.45 | 1.0 |
| Very Stiff Clay | 1.5 | -- | 0.35 | 1.0 |
| Dense Sand | 1.7 | 75 | 0.45 | 1.0 |

| Pile Location | Top Bearing Layer Elevation (ft) | Top of Dense Sand Elevation (ft) | Pile Tip in Dense Sand Elevation (ft) | Clay Thickness (ft) Above Dense Sand |
|---|---|---|---|---|
| West Abutment | -10 | -55 | -65 | 25 |
| West Approach Piers | -15 | -55 | -65 | 14 |
| Bascule Pier | -21 | -55 | -65 | 10 |
| East Approach Piers | -15 | -55 | -65 | 0 |
| East Abutment | -10 | -55 | -65 | 4 |

| Pile Location | Pile Diameter (in) | Strength Limit State | | | Strength Limit State | | |
|---|---|---|---|---|---|---|---|
| | | Factored Tip Resistance (kips) | Factored Side Resistance (kips) | Total Factored Resistance (kips) | Factored Tip Resistance (kips) | Factored Side Resistance (kips) | Total Factored Resistance (kips) |
| West Abutment | 14 | 36 | 99 | 135 | 80 | 251 | 331 |
| | 16 | 47 | 113 | 161 | 105 | 287 | 392 |
| West Approach Piers | 14 | 36 | 85 | 121 | 80 | 206 | 286 |
| | 16 | 47 | 97 | 144 | 105 | 235 | 340 |
| Bascule Pier | 14 | 36 | 75 | 111 | 80 | 179 | 259 |
| | 16 | 47 | 86 | 133 | 105 | 204 | 309 |
| East Approach Piers | 14 | 36 | 74 | 110 | 80 | 165 | 245 |
| | 16 | 47 | 85 | 132 | 105 | 188 | 293 |
| East Abutment | 14 | 36 | 83 | 119 | 80 | 189 | 270 |
| | 16 | 47 | 95 | 142 | 105 | 217 | 321 |

**CHATHAM BRIDGE**
**ESTIMATES OF PILE FACTORED RESISTANCE - UPLIFT**

| Soil Strata | Unit Side Resistance $q_s$ (ksf) | Unit Tip Resistance $q_p$ (ksf) | Resistance Factors | |
|---|---|---|---|---|
| | | | Strength Limit State | Extreme Limit State |
| Medium Dense Sand | 0.7 | -- | 0.35 | 0.8 |
| Very Stiff Clay | 1.5 | -- | 0.25 | 0.8 |
| Dense Sand | 1.7 | -- | 0.35 | 0.8 |

| Pile Location | Top Bearing Layer Elevation (ft) | Top of Dense Sand Elevation (ft) | Pile Tip in Dense Sand Elevation (ft) | Clay Thickness (ft) Above Dense Sand |
|---|---|---|---|---|
| West Abutment | -10 | -55 | -65 | 25 |
| West Approach Piers | -15 | -55 | -65 | 14 |
| Bascule Pier | -21 | -55 | -65 | 10 |
| East Approach Piers | -15 | -55 | -65 | 0 |
| East Abutment | -10 | -55 | -65 | 4 |

| Pile Location | Pile Diameter (in) | Strength Limit State | | | Strength Limit State | | |
|---|---|---|---|---|---|---|---|
| | | Factored Tip Resistance (kips) | Factored Side Resistance (kips) | Total Factored Resistance (kips) | Factored Tip Resistance (kips) | Factored Side Resistance (kips) | Total Factored Resistance (kips) |
| West Abutment | 14 | 0 | 74 | 74 | 0 | 201 | 201 |
| | 16 | 0 | 85 | 85 | 0 | 230 | 230 |
| West Approach Piers | 14 | 0 | 64 | 64 | 0 | 165 | 165 |
| | 16 | 0 | 74 | 74 | 0 | 188 | 188 |
| Bascule Pier | 14 | 0 | 57 | 57 | 0 | 143 | 143 |
| | 16 | 0 | 65 | 65 | 0 | 164 | 164 |
| East Approach Piers | 14 | 0 | 58 | 58 | 0 | 132 | 132 |
| | 16 | 0 | 66 | 66 | 0 | 151 | 151 |
| East Abutment | 14 | 0 | 64 | 64 | 0 | 152 | 152 |
| | 16 | 0 | 73 | 73 | 0 | 173 | 173 |

→ ESTIMATE POTENTIAL DOWNDRAG LOAD FROM RIVER BOTTOM MATERIAL:

→ use Nordlund/Thurman Method for side friction

$$q_{sd} = K_\delta C_F \sigma_V' \sin\delta$$ (see charts pages 4 & 5)

For loose sand and silt → $\phi_f = 27^\circ$ (see Table 4.1, page 3)

$\delta = 0.7(27) = 19^\circ$

$$\sigma_V' = \underbrace{11}_{\text{average mid-layer depth between embankment and river bottom}}(125) - \overbrace{8}^{\text{average low \& high water in river}}(62.4) = 876 \text{ psf}$$

For $\phi_f = 27^\circ$ → $K_\delta = 1.0$

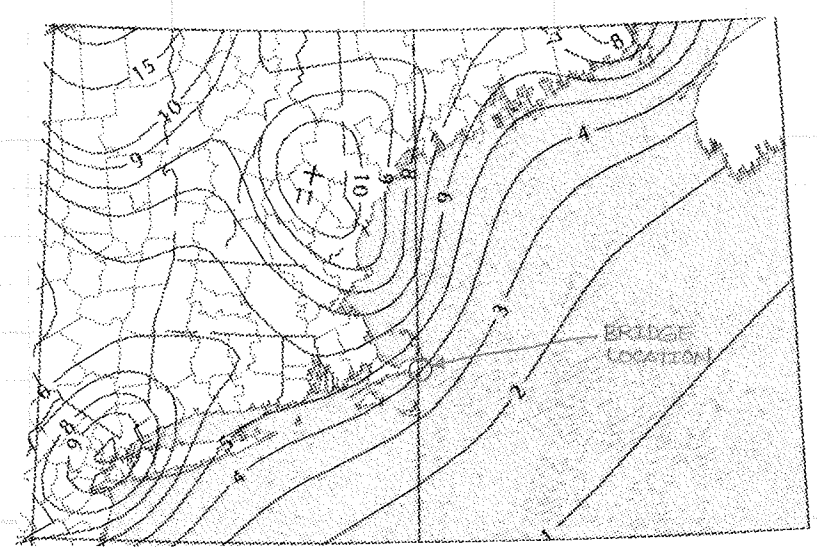
For $\delta/\phi_f = 0.7$ → $C_F = 0.9$

$$\therefore q_{sd} = (1.0)(0.9)(876)\sin 19^\circ = 257 \text{ psf}$$ say 260 psf

→ CHECK LIQUEFACTION POTENTIAL OF SOILS AT BRIDGE SITE

→ use empirical method based on AASHTO Appendix A10

$$\frac{(\tau_R)_{av}}{\sigma_o'} = 0.65 r_d \left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_o}{\sigma_o'}\right)$$ – AASHTO A10.1-1




AASHTO Figure 3.10.2.1-1

peak ground acceleration coeff.

$a_{max} = 0.04g$ ← (for site class B)

$a_{max} = 0.064g$ (for site class D)

– See attached page

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 41.670000
Longitude = -069.960000
Site Class B
Data are based on a 0.05 deg grid spacing.

| Period (sec) | Sa (g) | |
|---|---|---|
| 0.0 | 0.040 | PGA - Site Class B |
| 0.2 | 0.089 | Ss - Site Class B |
| 1.0 | 0.029 | S1 - Site Class B |

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
Latitude = 41.670000
Longitude = -069.960000
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40
Data are based on a 0.05 deg grid spacing.

| Period (sec) | Sa (g) | |
|---|---|---|
| 0.0 | 0.064 | As - Site Class D |
| 0.2 | 0.143 | SDs - Site Class D |
| 1.0 | 0.068 | SD1 - Site Class D |

→ CHECK LIQUEFACTION POTENTIAL (CONT'D)

Most critical zone appears to be loose silty fine sand soil at El. -15 in BB-1 (see soil profile).

→ SPT N value = 6 blows/foot.

$\sigma_o$ = total overburden pressure = (20)(125) = 2,500 psf.

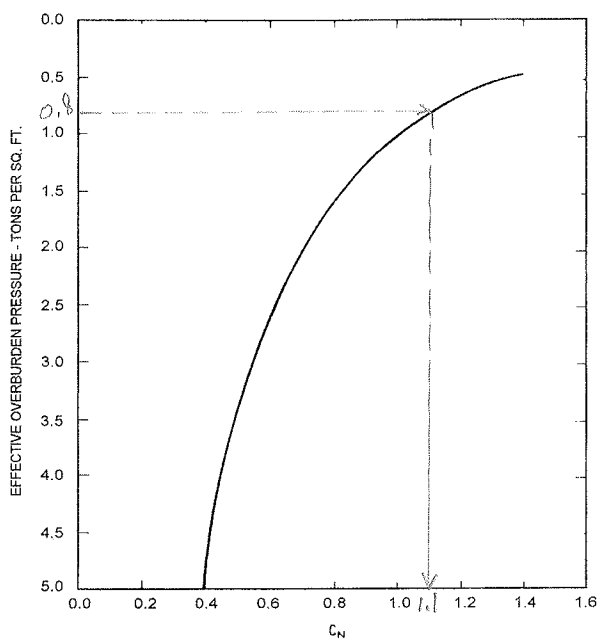
(average depth between embankment and river)

$\sigma_o'$ = initial effective overburden pressure = 2,500 − 15(62.4) = 1,564 psf

(average high & low water depth)

$r_d$ = stress reduction factor (1.0 at ground surface, 0.9 at depth of 30 ft)

≈ 0.93 at depth of ~20 ft




For $\sigma_o'$ = 1,564 psf = 0.8 tsf

$C_N$ = 1.1 (correction factor)

**Figure A10.1-2 Relationship Between $C_N$ and Effective Overburden Pressure.**

$N_1 = N\,C_N$ – AASHTO A10.1-2 (Modified penetration resistance)

$= (6)(1.1) = 7$ blows/ft.

→ CHECK LIQUEFACTION POTENTIAL (CONT'D)

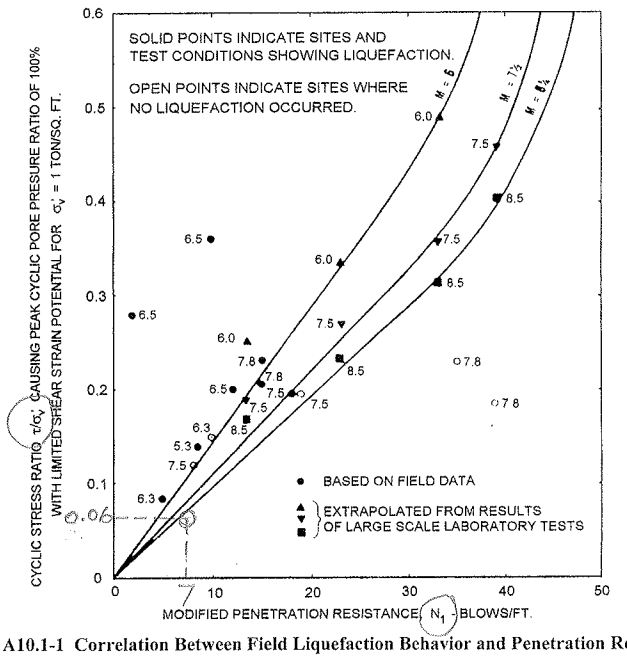



Figure A10.1-1 Correlation Between Field Liquefaction Behavior and Penetration Resistance.

$$\frac{\tau}{\sigma'_v} = (0.65)(0.93)(0.064)\left(\frac{2,500}{1,564}\right)$$

$$= 0.06$$

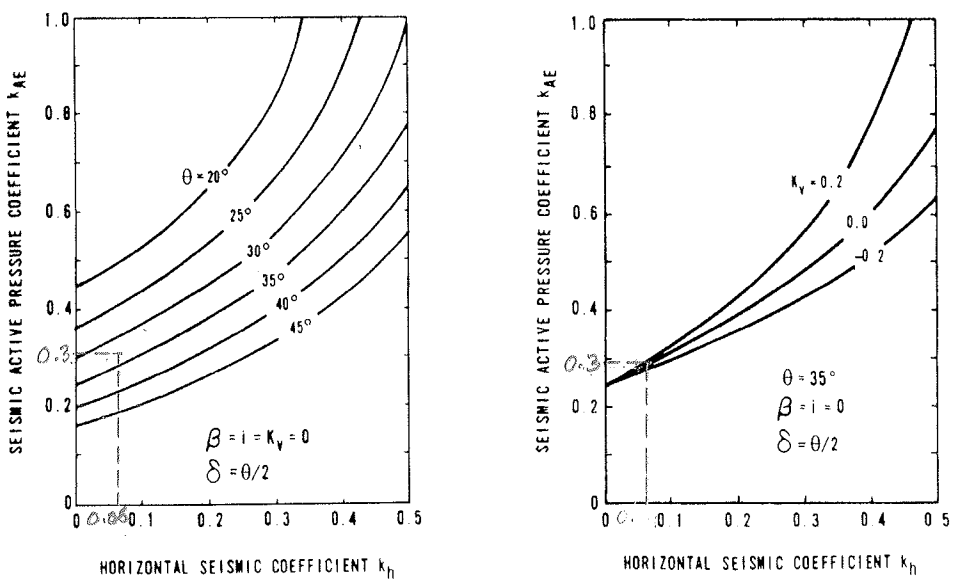
$N_1 = 7$ blows/ft

point plotted right-hand side of curves

→ Not susceptible to liquefaction

→ ESTIMATE DYNAMIC/SEISMIC ACTIVE EARTH PRESSURE COEFFICIENT

use Mononobe - Okabe Analysis AASHTO Appendix A11




For $k_h = 0.06g$ ($a_{max}$)

$\theta = 32°$ ($\phi_f$ backfill)

$K_{ae} \approx 0.3$

Figure A11.1.1.1-2 Effects of Seismic Coefficient and Soil Friction Angle on Seismic Active Pressure Coefficient.

→ CALCULATE DYNAMIC / SEISMIC ACTIVE EARTH PRESSURE COEFFICIENT

→ USE AASHTO EQ. A11.1.1-2 ATTACHED

$$K_{ae} = \frac{\cos^2(\phi-\theta-\beta)}{\cos\theta\cos^2\beta\cos(\delta+\beta+\theta)} \times \left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\theta-i)}{\cos(\delta+\beta+\theta)\cos(i-\beta)}}\right]^{-2}$$

assume $\phi$ = soil friction angle = 32°

$K_v = 0 \rightarrow \theta = \arctan(\overset{0.06}{K_h}/(1-\underset{0}{K_v}))$

$\theta = \arctan(0.06) = \underline{3.4°}$

$\delta$ = wall interface friction angle = 0°

$i$ = backfill slope angle = 0° (horizontal backslope)

$\beta$ = slope of wall to vertical = 0° (vertical wall.

$$K_{ae} = \frac{\cos^2(32-3.4)}{\cos 3.4 \cos 3.4} \times \left[1+\sqrt{\frac{\sin 32 \sin(32-3.4)}{\cos 3.4}}\right]^{-2}$$

$$= \frac{\cos^2(28.6)}{\cos^2(3.4)} \times \underbrace{\left[1+\sqrt{\frac{\sin 32 \sin 28.6}{\cos 3.4}}\right]^{-2}}_{1.5041}$$

$\underline{K_{ae} = 0.34}$

Equilibrium considerations of the soil wedge behind the abutment, as shown in Figure 1, then lead to a value, $E_{AE}$, of the active force exerted on the soil mass by the abutment and vice versa. When the abutment is at the point of failure $E_{AE}$ is given by the expression:

$$E_{AE} = \frac{1}{2}\gamma H^2 (1-k_v) K_{AE} \qquad \text{(A11.1.1.1-1)}$$

where the seismic active pressure coefficient $K_{AE}$ is

$$K_{AE} = \frac{\cos^2(\phi-\theta-\beta)}{\cos\theta\cos^2\beta\cos(\delta+\beta+\theta)} \times \left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\theta-i)}{\cos(\delta+\beta+\theta)\cos(i-\beta)}}\right]^{-2} \qquad \text{(A11.1.1.1-2)}$$

and where

$\gamma$ = unit weight of soil (kcf)

$H$ = height of soil face (ft.)

$\phi$ = angle of friction of soil (°)

$\theta$ = arc tan $(k_h/(1-k_v))$ (°)

$\delta$ = angle of friction between soil and abutment (°)

$k_h$ = horizontal acceleration coefficient (dim.)

$k_v$ = vertical acceleration coefficient (dim.)

$i$ = backfill slope angle (°)

$\beta$ = slope of wall to the vertical, negative as shown (°)

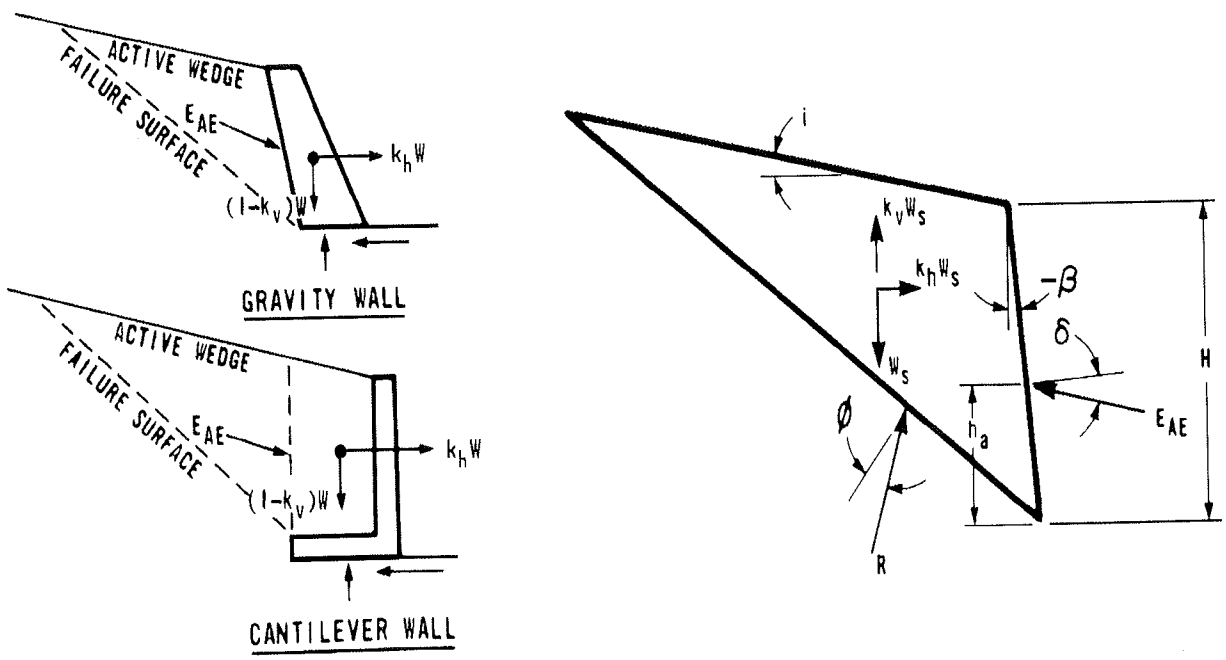
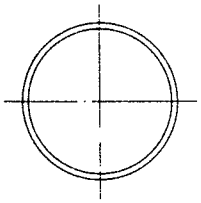



**Figure A11.1.1.1-1 Active Wedge Force Diagram.**

# APPENDIX C-2 (CONT'D)



## PIPE PILES

Dimensions and Properties for Designing

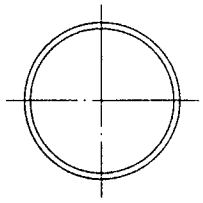
| Designation and Outside Diameter | Wall Thickness | Area A | Weight per Foot | Section Properties | | | Area of Exterior Surface | Inside Cross-Sectional Area | Inside Volume | External Collapse Index |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | I | S | r | | | | |
| in. | in. | $in^2$ | lb. | $in.^4$ | $in.^3$ | in. | $ft^2/ft$ | $in.^2$ | $yd^3/ft$ | * |
| PP14 (cont'd) | .219 | 9.48 | 32.23 | 225 | 32.2 | 4.87 | 3.67 | 144 | .0372 | 185 |
| | .230 | 9.95 | 33.82 | 236 | 33.7 | 4.87 | 3.67 | 144 | .0370 | 215 |
| | .250 | 10.8 | 36.71 | 255 | 36.5 | 4.86 | 3.67 | 143 | .0368 | 277 |
| | .281 | 12.1 | 41.17 | 285 | 40.7 | 4.85 | 3.67 | 142 | .0365 | 395 |
| | .312 | 13.4 | 45.61 | 314 | 44.9 | 4.84 | 3.67 | 141 | .0361 | 542 |
| | .344 | 14.8 | 50.17 | 344 | 49.2 | 4.83 | 3.67 | 139 | .0358 | 691 |
| | .375 | 16.1 | 54.57 | 373 | 53.3 | 4.82 | 3.67 | 138 | .0355 | 835 |
| | .438 | 18.7 | 63.44 | 429 | 61.4 | 4.80 | 3.67 | 135 | .0348 | 1,130 |
| | .469 | 19.9 | 67.78 | 457 | 65.3 | 4.79 | 3.67 | 134 | .0345 | 1,280 |
| | .500 | 21.2 | 72.09 | 484 | 69.1 | 4.78 | 3.67 | 133 | .0341 | 1,460 |
| PP16 | .134 | 6.68 | 22.71 | 210 | 26.3 | 5.61 | 4.19 | 194 | .0500 | 28 |
| | .141 | 7.02 | 23.88 | 221 | 27.6 | 5.61 | 4.19 | 194 | .0499 | 33 |
| | .150 | 7.47 | 25.39 | 235 | 29.3 | 5.60 | 4.19 | 194 | .0498 | 39 |
| | .156 | 7.76 | 26.40 | 244 | 30.5 | 5.60 | 4.19 | 193 | .0497 | 44 |
| | .164 | 8.16 | 27.74 | 256 | 32.0 | 5.60 | 4.19 | 193 | .0496 | 52 |
| | .172 | 8.55 | 29.08 | 268 | 33.5 | 5.60 | 4.19 | 193 | .0495 | 60 |
| | .179 | 8.90 | 30.25 | 278 | 34.8 | 5.59 | 4.19 | 192 | .0494 | 67 |
| | .188 | 9.34 | 31.75 | 292 | 36.5 | 5.59 | 4.19 | 192 | .0493 | 78 |
| | .203 | 10.1 | 34.25 | 314 | 39.3 | 5.59 | 4.19 | 191 | .0491 | 98 |
| | .219 | 10.9 | 36.91 | 338 | 42.3 | 5.58 | 4.19 | 190 | .0489 | 124 |
| | .230 | 11.4 | 38.74 | 354 | 44.3 | 5.58 | 4.19 | 190 | .0488 | 144 |
| | .250 | 12.4 | 42.05 | 384 | 48.0 | 5.57 | 4.19 | 189 | .0485 | 185 |
| | .281 | 13.9 | 47.17 | 429 | 53.6 | 5.56 | 4.19 | 187 | .0481 | 264 |
| | .312 | 15.4 | 52.27 | 473 | 59.2 | 5.55 | 4.19 | 186 | .0478 | 362 |
| | .344 | 16.9 | 57.52 | 519 | 64.8 | 5.54 | 4.19 | 184 | .0474 | 487 |
| | .375 | 18.4 | 62.58 | 562 | 70.3 | 5.53 | 4.19 | 183 | .0470 | 617 |
| | .438 | 21.4 | 72.80 | 649 | 81.1 | 5.50 | 4.19 | 180 | .0462 | 874 |
| | .469 | 22.9 | 77.79 | 691 | 86.3 | 5.49 | 4.19 | 178 | .0458 | 1,000 |
| | .500 | 24.3 | 82.77 | 732 | 91.5 | 5.48 | 4.19 | 177 | .0455 | 1,130 |

Material Specifications - ASTM A252

Example of suggested method of designation: PP8-5/8 x .109

*The External Collapse Index is a non-dimensional function of the diameter to wall thickness ratio and is for general guidance only. The higher the number, the greater is the resistance to collapse.

# APPENDIX C-2 (CONT'D)



## PIPE PILES

Dimensions and Properties for Designing

| Designation and Outside Diameter | Wall Thickness | Area A | Weight per Foot | Section Properties | | | Area of Exterior Surface | Inside Cross-Sectional Area | Inside Volume | External Collapse Index |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | I | S | r | | | | |
| in. | in. | $in^2$ | lb. | $in.^4$ | $in.^3$ | in. | $ft^2/ft$ | $in.^2$ | $yd^3/ft$ | * |
| PP18 | .141 | 7.91 | 26.89 | 315 | 35.0 | 6.31 | 4.71 | 247 | .0634 | 23 |
| | .172 | 9.63 | 32.75 | 383 | 42.5 | 6.30 | 4.71 | 245 | .0630 | 42 |
| | .188 | 10.5 | 35.76 | 417 | 46.4 | 6.30 | 4.71 | 244 | .0627 | 55 |
| | .203 | 11.3 | 38.58 | 449 | 49.9 | 6.29 | 4.71 | 243 | .0625 | 69 |
| | .219 | 12.2 | 41.59 | 484 | 53.7 | 6.29 | 4.71 | 242 | .0623 | 87 |
| | .230 | 12.8 | 43.65 | 507 | 56.3 | 6.28 | 4.71 | 242 | .0621 | 101 |
| | .250 | 13.9 | 47.39 | 549 | 61.0 | 6.28 | 4.71 | 241 | .0619 | 129 |
| | .281 | 15.6 | 53.18 | 614 | 68.2 | 6.27 | 4.71 | 239 | .0614 | 184 |
| | .312 | 17.3 | 58.94 | 678 | 75.4 | 6.25 | 4.71 | 237 | .0610 | 253 |
| | .344 | 19.1 | 64.87 | 744 | 82.6 | 6.24 | 4.71 | 235 | .0605 | 341 |
| | .375 | 20.8 | 70.59 | 807 | 89.6 | 6.23 | 4.71 | 234 | .0601 | 443 |
| | .406 | 22.4 | 76.29 | 869 | 96.5 | 6.22 | 4.71 | 232 | .0597 | 559 |
| | .438 | 24.2 | 82.15 | 932 | 104 | 6.21 | 4.71 | 230 | .0592 | 675 |
| | .469 | 25.8 | 87.81 | 993 | 110 | 6.20 | 4.71 | 229 | .0588 | 788 |
| | .500 | 27.5 | 93.45 | 1,050 | 117 | 6.19 | 4.71 | 227 | .0584 | 900 |
| PP20 | .141 | 8.80 | 29.91 | 434 | 43.4 | 7.02 | 5.24 | 305 | .0785 | 17 |
| | .172 | 10.7 | 36.42 | 527 | 52.7 | 7.01 | 5.24 | 303 | .0780 | 30 |
| | .188 | 11.7 | 39.78 | 574 | 57.4 | 7.00 | 5.24 | 302 | .0778 | 40 |
| | .203 | 12.6 | 42.92 | 619 | 61.9 | 7.00 | 5.24 | 302 | .0776 | 50 |
| | .219 | 13.6 | 46.27 | 666 | 66.6 | 6.99 | 5.24 | 301 | .0773 | 63 |
| | .250 | 15.5 | 52.73 | 756 | 75.6 | 6.98 | 5.24 | 299 | .0768 | 94 |
| | .281 | 17.4 | 59.18 | 846 | 84.6 | 6.97 | 5.24 | 297 | .0763 | 134 |
| | .312 | 19.3 | 65.60 | 935 | 93.5 | 6.96 | 5.24 | 295 | .0758 | 184 |
| | .344 | 21.2 | 72.21 | 1,030 | 103 | 6.95 | 5.24 | 293 | .0753 | 247 |

Material Specifications - ASTM A252

Example of suggested method of designation: PP8-5/8 x .109

*The External Collapse Index is a non-dimensional function of the diameter to wall thickness ratio and is for general guidance only. The higher the number, the greater is the resistance to collapse.