REPORT ON
GEOTECHNICAL DESIGN REPORT
REPLACEMENT BRIDGE OVER PRESUMPSCOT RIVER AND MAINE CENTRAL RAILROAD
MAINEDOT PIN 15094.00
ROUTES 26/100 - FALMOUTH, MAINE

by

Haley & Aldrich, Inc.
Portland, Maine

for

Maine Department of Transportation
Augusta, Maine

File No. 35524-010
25 November 2009

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25 November 2009
File No. 35524-010

Maine Department of Transportation
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Attention: Laura Krusinski, P.E.
Senior Geotechnical Engineer

Subject: Geotechnical Design Report
Replacement Bridge over Presumpscot River and Maine Central Railroad
MaineDOT PIN 15094.00
Routes 26/100 - Falmouth, Maine

Ladies and Gentlemen:

We are pleased to submit herewith our report titled, "Geotechnical Design Report, Replacement Bridge over Presumpscot River and Maine Central Railroad," prepared in accordance with our proposal, dated 15 April 2009 and with the provisions of our GCA Agreement with MaineDOT, No. CT20070810000000003861.

This Geotechnical Design Report (GDR) is a compilation of the results of subsurface investigations and geotechnical laboratory testing programs and provides geotechnical design recommendations in support of the subject project.

The remainder of this report is divided into the following sections:

- Section 1 – Text: Introduction, Geologic Setting, Subsurface Investigation Programs, Generalized Subsurface Conditions, Laboratory Testing Program, Strength and Compressibility Characteristics of Marine Clay, Geotechnical Design Recommendations, Construction Considerations, and Limitations of Recommendations.
- Section 2 – Illustrations: Project Locus, Site and Subsurface Exploration Location Plan, Geologic Profile, Compressibility and Shear Strength Data (North Approach), Compressibility and Shear Strength Data (South Approach), Corrected Standard Penetration Test Values for Liquefaction Assessment (Alluvial Deposit), Initial Liquefaction Assessment, North of River (AASHTO, Site Class E), Initial Liquefaction Assessment, South of River (AASHTO, Site Class D), Site Specific Liquefaction Assessment (North of River), Site Specific Liquefaction Assessment (South of River), Table I – Preliminary Phase Explorations, Table II – Design Phase Explorations, Table III – Preliminary Phase In-Situ Vane Shear Test Results, Table IV – Design Phase In-Situ Vane Shear Test Results, Table V – Pier 1 Pile Cap Loads, Table VI – Pier 2 Pile Cap Loads, Test Boring Logs, Observation Well Installation and Groundwater Monitoring Reports, and Laboratory Test Results.
- Section 3 – Appendices: Preliminary and Design Phase Memoranda, Calculations.

Thank you for the opportunity to help support MaineDOT on this significant bridge project. We look forward to providing continued assistance to the Department during the bidding and construction phases of the project.

Sincerely yours,
HALEY & ALDRICH, INC.

Bryan C. Steinert, P.E.
Staff Engineer

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Technical Reviewer

Wayne A. Chadbourne, P.E.
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STATE OF MAINE
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Enclosures

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1. INTRODUCTION

This report present the results of preliminary and design phase geotechnical investigations, field and laboratory testing, engineering evaluations, and geotechnical design recommendations conducted by Haley & Aldrich, Inc. (Haley & Aldrich) for the Maine Department of Transportation (MaineDOT) for the proposed replacement of the Route 26/100 bridge over the Presumpscot River (river) and Maine Central Railroad (MCRR) in Falmouth, Maine (see Sheet 1, Project Locus).

1.1 Existing Site Conditions

1.1.1 Existing Bridge Structure

Based on our review of historic plans provided by MaineDOT, it is our understanding that the existing bridge structure was constructed in 1932 and 1933. The 15-span, approximately 800-ft long bridge structure is supported on two abutments and 26 piers. The portion of the bridge that crosses the river consists of two, 125-ft long spans with a single pier located in the river. The portion of the bridge that crosses the MCRR tracks consists of two, 75-ft long spans. The substructures are supported on timber piles with a design capacity of approximately 15 tons. The timber piles range in length from 20 to 60 ft. The existing approaches (north and south) were constructed as fill embankments with a maximum height of approximately 23 ft relative to pre-construction (1932) grades.

1.1.2 Terrain

The existing ground surface varies significantly within the project limits, between STA 105+00 (southern project limit) and STA 124+50 (northern project limit). The existing roadway profile slopes down at an approximate 4 percent grade from STA 105+00 (El. 66) to STA 109+00 (El. 50). At approximately STA 109+00 the proposed roadway alignment diverges from the existing alignment and slopes down the existing approach embankment to El. 32 in the vicinity of STA 111+00. Between STA 111+00 and the river (STA 112+00, approximate) the ground surface gently slopes down to roughly El. 22. The limits of the river, as defined and referenced herein, are generally between STA 112+00 and STA 114+50. Flood levels in the river, as determined by TY Lin, are summarized below.

Discharge	Headwater (ft, NAVD 88)
$Q_{1.1}$	El. 18.4 (“Normal Water”)
Q_{50}	El. 27.6
Q_{100}	El. 28.7
Q_{500}	El. 30.9

As part of the hydraulic evaluation, TY Lin also considered the effects of scour at the proposed bridge location. The results of their evaluation are summarized below. Please note that no scour is anticipated at Abutment No. 1, Pier 3, or Abutment No. 2.

Substructure	Approximate Existing Ground Surface (ft, NAVD 88)	Design Scour Level (ft, NAVD 88)
Pier 1	El. -4.0	El. -14.4
Pier 2	El. 26.0	El. 19.3

The floodplain of the river extends north from STA 114+50 to approximately STA 116+50. The ground surface within these limits remains relatively flat, varying between El. 24 and El. 26. North of STA

116+50 the ground surface rises steeply to El. 40 near STA 117+50 where it continues to rise gently within the limits of the MCRR right-of-way to El. 46 at STA 118+50 (approximate). North of STA 118+50 the proposed alignment climbs up the existing north approach embankment to El. 70 in the vicinity of STA 120+00. The proposed alignment merges back into the existing roadway alignment and site grades remain flat to the northern project limit (STA 124+50). Refer to Sheet 3, Geologic Profile, for a graphic interpretation of existing ground surface levels along the centerline of the proposed bridge/roadway alignment.

1.2 Proposed Bridge Structure

The replacement bridge and approach roadway alignment will be offset to the east of the existing bridge alignment as shown on Sheet 2, Site and Subsurface Exploration Location Plan. The total length of the proposed project alignment is 1,950 linear feet (lf) which, consists of a 720-ft long bridge structure and 1,230 lf of approach roadway that will be widened/reconstructed in order to realign the existing approaches with the new bridge alignment.

The bridge superstructure will be approximately 40 ft wide and will consist of two 11-ft wide travel lanes, two 5-ft wide outside shoulders, and one 5-ft wide sidewalk located on the east side of the structure. The bridge superstructure will be constructed using composite welded steel beams (five beam lines) running parallel to the long dimension of the bridge, with a 7½-in. thick composite concrete deck and a 3-in. thick bituminous concrete wearing surface separated by a high performance waterproofing membrane. The bridge superstructure will be supported on two abutments (one stub, one full-height) and three piers at the locations shown in the table below.

Substructure	Station at Centerline Of Alignment (ft)
Abutment No. 1	111+42
Pier 1	113+02
Pier 2	115+02
Pier 3	117+02
Abutment No. 2	118+62

The bridge approach roadways will be approximately 40 to 50-ft wide and will consist of two 11-ft wide travel lanes, two 5-ft wide outside shoulders, and one 5-ft wide sidewalk located on the east side of the roadway. Lane widths will vary somewhat along the north approach due to the presence of turning lanes allowing vehicle access to existing residential and commercial properties. Existing grades within the limits of the existing roadway along the north and south approaches will be raised by approximately 3 to 4 ft (relative to existing roadway grades). Maximum raises in grade ranging from 26 ft (north approach) to 33 ft (south approach) will be required to construct the widened portion of the approaches outside the limits of the existing roadway.

1.3 Horizontal Coordinate System and Elevation Datum

Plan locations of test borings are reported as northing and easting coordinates relative to the Maine State Plane Coordinate System, North American Datum of 1983 (NAD 83), Maine 2000 West Zone. The project elevation datum and elevations referenced herein are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).

2. GEOLOGIC SETTING

Falmouth is located in the Coastal Lowlands Region of southwestern Maine. During Late Wisconsinan time, the glacial ice margin had reached the present coast of Maine, approximately 16,000 years ago. The weight of the glacier caused downwarping of the earth's crust and the coastal region was submerged into the ocean. During glacial melting, large quantities of sediment were carried by glacial streams into the ocean at the glacier margin. The glaciomarine sediments settled to the sea floor and were subject to tidal movements and other marine processes between 11,000 and 12,000 years ago. The accumulated sediments consisted of sand, silt and clay and comprise the Presumpscot Formation. This formation typically overlies glacial till sediment composed of a heterogeneous mixture of sand, silt, clay and gravel, and may include cobbles, boulders and rock debris. Glacial till is deposited directly by the glacier without sorting and reworking by glacial meltwater and typically overlies bedrock (Thompson and Marvinney, 2008).

The evolution of rivers and streams began as a result of deglaciation, eventually producing a formation of organic deposits in low-lying peat bogs, marshes and swamps. Stream alluvium deposited along the Presumpscot River low-lying floodplain consists primarily of silty, sandy sediments and organics (Thompson and Marvinney 2008). Surficial deposits encountered in preliminary and design phase geotechnical test borings at the site include alluvial deposits, glaciomarine deposits of the Presumpscot Formation and/or glacial till.

Bedrock at the site is mapped as the Berwick Formation which, consists of deep ocean sediments deposited during Early Silurian time (Portland West Quadrangle, Maine, revised 2003). As a result of structural deformation of the earth's crust during the Acadian Orogeny, these sedimentary rocks were metamorphosed. Bedrock encountered in preliminary and design phase geotechnical test borings at the site consisted of metamorphic schist and gneiss. Major faulting and shearing occurred after the Acadian Orogeny, producing part of the Norumbega Fault Zone. Rocks within this fault zone show indications of right-lateral strike slip movement (Hussey and Marvinney, 2003).

The Norumbega Fault Zone separates the Central Maine Sequence to the northwest from the Coastal Litho-Tectonic Belt to the southeast. During the Mesosoic era, a later period of faulting occurred indicating vertical movement, forming the Nonesuch River Fault. The Berwick Formation is separated from the sheared Eliot Formation by the Nonesuch River Fault. An inferred fault is mapped within the site vicinity along the Presumpscot River valley, as a continuation of the Nonesuch River Fault (Hussey, Bothner and Thompson, 2007). However, no postglacial tectonic movement along existing bedrock faults has been recorded in Maine (Ebel, 1989).

3. SUBSURFACE INVESTIGATION PROGRAMS

3.1 Historic Explorations by Others

Previous explorations were conducted at the site in association with the original construction of the bridge. Four "wash borings" were drilled in the vicinity of substructures located on the south side of the river, within the river and immediately north of the river in 1932/1933. No information was provided on the historic contract documents relative to drilling means/methods, the depth of the explorations or the soil conditions encountered.

3.2 Recent Explorations by Haley & Aldrich

Haley & Aldrich conducted preliminary (Fall 2008) and design phase (Spring 2009) geotechnical investigation programs at the site. All test borings were drilled by Maine Test Borings of Brewer, Maine. In total, seventeen test borings were drilled along the existing and proposed bridge alignments in order to identify general subsurface conditions. "As-drilled" locations of the preliminary and design phase test borings are shown on Sheet 2. Coordinate location data and ground surface elevations at exploration locations are provided on individual test boring logs provided in Section 2 (Sheet Nos. 17-93) and are listed in Tables I and II (Sheet Nos. 11 and 12). All soil and bedrock samples were classified in accordance with MaineDOT classification system and were preserved in glass jars and wooden boxes. The samples that were not submitted for laboratory testing are available for review upon request. The soil and bedrock samples are being stored at the Haley & Aldrich laboratory facility in Portland, Maine. A discussion of drilling means and methods for each phase of geotechnical investigation is provided below.

3.2.1 Preliminary Phase Explorations

A total of six test borings, designated BB-FRR-101 through BB-FRR-102 and BB-FPR-101 through BB-FPR-104, were drilled along and immediately east of the existing bridge structure. Test boring locations were laid out in the field by Haley & Aldrich by taping/pacing distances from existing site features. "As-drilled" test boring locations and ground surface elevations were determined in the field by MaineDOT using GPS survey equipment.

Subsurface explorations were drilled using either a track-mounted Mobile Drill B-50 drill rig (BB-FRR-101, BB-FRR-102, BB-FPR-101, and BB-FPR-102), a CME 45 skid-mounted drill rig placed on an anchored barge (BB-FPR-103) or a CME 550X ATV mounted drill rig (BB-FPR-104). Test borings were drilled to depths ranging from approximately 97 to 180 ft below ground surface (BGS) using 3.0-in. (NW-size) or 4.0-in. (HW-size) inside diameter (ID) steel casing. Soil samples were generally collected continuously through the fill soils and at standard, 5-ft intervals, thereafter by driving a 1-3/8-in. ID split-spoon sampler with a 140-lb hammer dropped from a height of 30 in., as indicated on the test boring logs. All drilling and sampling was performed in accordance with MaineDOT specifications.

Each drill rig was equipped with a standard rope and cathead and safety hammer per MaineDOT requirements (Appendix A of MaineDOT Geotechnical Drilling Contract Specifications, revised June 2007). A theoretical hammer efficiency factor of 0.6 was assumed for the rope and cathead/safety hammer system for the all of the drill rigs.

The number of hammer blows required to advance the sampler through each 6 in. interval was recorded and is provided on the test boring logs. The uncorrected SPT N-value is defined as the total number of blows required to advance the sampler through the middle 12 in. of the 24-in. sampling interval. The energy-corrected SPT N-value (N_{60}) is equal to the uncorrected N-value multiplied by the hammer efficiency factor divided by 0.6 (i.e., 60 percent theoretical hammer efficiency).

In-situ vane shear tests were conducted within the marine clay deposit in several of the test borings. A standard, 3 in. by 6 in. rectangular vane (Acker style) attached to an approximate 2-ft long, ¾-in. diameter rod extension was attached to AW-size (1 ¾-in. OD) drill rods and used to perform the tests. The vane was pushed (by hand) approximately 1 ft below the bottom of the borehole and was rotated using a calibrated torque wrench. Results of the vane shear testing, including raw torque values and calibrated shear strengths, are summarized in Table III (Sheet 13) and are provided on the individual test boring logs in Section 2 (Sheet Nos. 17-93).

A total of four, undisturbed samples of marine clay were obtained in test borings BB-FRR-102, BB-FPR-101 and BB-FPR-102. The samples were obtained by advancing a 3-in. OD thin-wall Shelby Tube into the clay using a piston sampler. Bentonite drilling mud was used while advancing the test borings in order to minimize soil disturbance.

Each test boring, with the exception of BB-FPR-101, was advanced a minimum of 10 ft into bedrock using a 2.0-in. (NQ-size) ID diamond-tipped core barrel. Bedrock was not sampled in test boring BB-FPR-101 due to a damaged casing drive shoe. Test borings were typically advanced greater than 10 ft into bedrock when the recovered core samples were highly fractured and/or weathered.

Two observation wells were installed in completed boreholes BB-FPR-101 and BB-FPR-102 to provide information on the static groundwater levels at the site and to determine whether the groundwater levels at the site are affected by water level fluctuations in the nearby Presumpscot River. The observation wells consisted of 2-in. ID, machine-slotted PVC pipe and solid PVC riser pipe extending approximately 3 ft above existing ground surface. The observation wells were outfitted with a steel guardpipe and steel lock/cap assembly. Observation well installation and groundwater monitoring reports are provided in Section 2.

3.2.2 Design Phase Explorations

Preliminary engineering evaluations were conducted to assess how the subsurface conditions encountered in the preliminary phase explorations affected the overall design and construction of the proposed replacement bridge. Subsequently, it was determined that additional (design phase) explorations were required in order to refine and update the preliminary engineering analyses. A design phase exploration program was developed and submitted to MaineDOT in a memorandum dated 14 January 2009 for review and approval prior to the commencement of drilling.

3.2.2.1 Test Boring Location

A design phase exploration program was developed based on 1) the preferred roadway alignment and superstructure type identified by TY Lin in their Preliminary Design Report (PDR) dated April 2009 and 2) the requirements of the MaineDOT Bridge Design Guide (BDG) Section 2.10.4. A total of twelve design phase test borings were proposed in order to provide subsurface information along the preferred alignment and at specific substructure locations listed below.

- One test boring at each of the following substructure locations: Abutment No. 1 (BB-FPR-205), Pier 2 (BB-FPR-208), Pier 3 (BB-FRR-201), and Abutment No. 2 (BB-FRR-202).
- Two test borings along the preferred north approach embankment alignment (BB-FRR-203 and BB-FRR-204).

- Three test borings along the preferred south approach embankment alignment (BB-FPR-201, BB-FPR-202 and BB-FPR-203).
- Three test borings between Abutment No. 1 and the river (BB-FPR-204, BB-FPR-206 and BB-FPR-207).

3.2.2.2 Test Boring Execution

The test boring locations were laid out in the field by Haley & Aldrich using GPS survey equipment. "As-drilled" test boring locations and ground surface elevations at test boring locations were determined in the field by MaineDOT using GPS and/or optical survey equipment.

In general, subsurface explorations were drilled using similar drill rig equipment to that used for the preliminary phase with the exception that a truck-mounted Mobile Drill B-47 drill rig was used to drill test borings BB-FRR-203 and BB-FRR-204. All other test borings were drilled with a CME 550X ATV or track-mounted Mobile Drill B-47 mounted drill rig. Test borings were drilled to depths ranging from approximately 16 to 180 ft BGS. Boreholes were advanced and soil samples were collected using similar means and methods that were used to conduct the preliminary phase test borings with the exception that soil samples were generally collected continuously through the fill and alluvial soils and then at standard, 5-ft intervals, thereafter.

Similar to the preliminary phase geotechnical investigation, in-situ vane shear tests were also conducted within the marine clay layer in several of the test borings. In-situ vane shear tests were conducted with either a 65 mm by 130 mm or a 55 mm by 110 mm Geonor rectangular vane (per MaineDOT requirements) attached to a 2-ft long, 12-mm diameter rod extension, attached to a string of 5/8-in. outside diameter (OD) hollow chrome-moly rods. At each in-situ vane shear test location, the vane was pushed (by hand) until the bottom of the vane was approximately 1 to 2 ft below the bottom of the borehole. The vane was then rotated at a rate of about 90 degrees per minute using a calibrated torque wrench. Results of the vane shear testing, including raw torque values and calibrated shear strengths, are summarized in Table IV (Sheet 14) and are provided on the test boring logs in Section 2 (Sheet Nos. 17-93).

A total of five, relatively undisturbed samples of marine clay were obtained in test borings BB-FPR-205, BB-FRR-202 and BB-FRR-203. The samples were obtained similarly to those collected in the preliminary phase investigation.

Test borings drilled at proposed substructure locations (BB-FPR-205, BB-FPR-208, BB-FRR-201 and BB-FRR-202) were advanced a minimum of 10 ft into bedrock using a 2.0-in. (NQ-size) ID diamond-tipped core barrel. Test borings were typically advanced greater than 10 ft into bedrock when the recovered core samples were highly fractured and/or weathered.

4. GENERALIZED SUBSURFACE CONDITIONS

The subsurface conditions encountered at the site consist of the following geologic units presented in order of increasing depth below ground surface: topsoil/fill, interbedded marine deposits, alluvial deposit, marine clay, marine sand, glacial till and bedrock. Refer to Sheet 3 for a graphic interpretation of the subsurface soil conditions along the proposed project alignment and Tables I and II (Sheet Nos. 11 and 12) for a summary of the soil units and encountered thicknesses. A description of each soil unit is provided separately, below. Detailed soil descriptions are provided on the test boring logs in Section 2 (Sheet Nos. 17-93).

Please note that soil descriptions provided on the test boring logs, summarized below and shown on the geologic profile (Sheet 3) do not represent actual field conditions other than at the specific test boring locations. The actual conditions will vary from those described and shown herein.

4.1 Soil Unit and Bedrock Descriptions

Topsoil / Fill

A thin layer of man-placed fill soils and/or topsoil was encountered in each test boring. The layer ranged in thickness from approximately 0.3 to 8 ft. The topsoil/fill soils were typically very loose to medium dense.

Interbedded Marine Deposit

Interbedded marine deposits were encountered along the north and south approaches in test borings BB-FPR-201, BB-FRR-203 and BB-FRR-204. Along the north approach the deposit was approximately 8 to 9 ft thick and was encountered beneath a surficial layer of fill and overlying marine clay. The soil unit generally consisted of fine sand and/or silt. The granular (sand) portions of the deposit were loose to medium dense while the cohesive (silt) portions were medium stiff to very stiff. Along the south approach (BB-FPR-201) the deposit was approximately 20-ft thick and was encountered directly beneath the existing roadway pavement section and overlying glacial till. The stratum consisted of alternating layers of sand and gravel with some thin lenses of silt and clay. The layer was typically loose to very dense.

Alluvial Deposit

Interbedded layers of fine sand, silt, clay and occasional organics were encountered within the flood plain north and south of the river. Where encountered, the deposit ranged in thickness from approximately 9 ft to 24 ft. The soil was generally very loose to medium dense.

Marine Clay Deposit

A marine clay deposit was encountered in each test boring with the exception of those drilled along the proposed south approach (BB-FPR-201, BB-FPR-202 and BB-FPR-203). In the vicinity of the MCRR tracks the clay ranges in thickness from approximately 50 to 80 ft, increasing in thickness from south to north, and is typically soft to medium stiff. The marine clay encountered beneath the alluvial deposit within the flood plain of the river and on the south side of the river ranged in thickness from approximately 10 to 24 ft, decreasing in thickness from north to south, and is generally medium stiff.

Marine Sand Deposit

A deposit of poorly graded fine to medium sand with silt was encountered in each test boring with the exception of BB-FPR-104 and BB-FPR-201 through BB-FPR-204, drilled south of the river. The marine sand was encountered directly beneath the marine clay layer and ranged in thickness from approximately 33 to 63 ft, increasing in thickness from south to north. The marine sand was typically very loose to dense.

Glacial Till

A heterogeneous mixture of clay, silt, sand and gravel was encountered in each test boring with the exception of BB-FRR-203, BB-FRR-204 and BB-FPR-206. Cobbles and boulders are often present within the glacial till deposit and were encountered as noted on the test boring logs. In general, glacial till was encountered directly beneath the marine sand layer with the following exceptions: test borings BB-FPR-104, BB-FPR-204 and BB-FPR-207 where it was overlain by marine clay, and test boring BB-FPR-201where it was overlain by interbedded marine deposits. The deposit ranged in thickness from approximately 26 ft in the vicinity of the MCRR tracks to approximately 35 ft directly north of the river to approximately 100 ft south of the river.

Bedrock

Bedrock was sampled in each preliminary phase test boring with the exception of BB-FPR-101 and in design phase test borings drilled at proposed substructure locations. Where encountered, the top of bedrock surface ranged from approximately 85 to 170 ft BGS. In general, the bedrock surface is fairly flat but slopes down slightly from south to north. Bedrock encountered at the site consists of very soft to hard, fresh to highly weathered, gray GNEISS/SCHIST. At some test boring locations, up to 2 ft of weathered bedrock was encountered overlying more competent bedrock sampled and described herein.

Rock quality designation (RQD) is a common parameter that is used to help assess the competency of sampled bedrock. RQD is defined as the sum of pieces of recovered bedrock greater than 4 in. in length divided by the total length of recovered bedrock. RQD values for bedrock encountered at the site ranged from 0 to 73 percent.

4.2 Groundwater Conditions

Two groundwater observation wells were installed in completed preliminary phase boreholes BB-FPR-101 and BB-FPR-102. Water levels were measured between El. 20 and El. 22 at BB-FPR-101 and between El. 15 and El. 17 at BB-FPR-102. Please note that the measured water levels in BB-FPR-102 appeared to typically be within 1 to 2 ft of the normal ($Q_{1.1}$) water level in the river (El. 18.4). Qualitatively, the water levels measured in BB-FPR-101 do not appear to be as directly influenced by the water level in the river as compared to BB-FPR-102.

Groundwater levels can be expected to fluctuate, subject to seasonal variation, local soil conditions, topography and precipitation. Water levels encountered during construction may differ from those observed in the test borings or observation wells. Observation well installation and groundwater monitoring reports are included in Section 2 (Sheet Nos. 94-97).

5. LABORATORY TESTING PROGRAM

Preliminary and design phase laboratory testing programs were undertaken to assist in soil classification/identification, determination of engineering properties, and evaluating reuse potential of representative soil samples collected during the field investigations. In general, laboratory testing was performed on disturbed soil samples collected during SPT and Shelby Tube sampling. All laboratory soil testing was performed by GeoTesting Express of Boxborough, Massachusetts. Geotechnical laboratory testing was performed in accordance with applicable American Society for Testing Materials (ASTM) testing procedures. All soil samples were transported to GeoTesting Express by Haley & Aldrich personnel. Preliminary and design phase laboratory testing and results are summarized below. All laboratory test results are provided in Section 2 (Sheet Nos. 98-167).

5.1 Preliminary Phase Laboratory Testing & Results

A laboratory testing program was conducted upon completion of the preliminary phase geotechnical investigation. The testing program included four natural water content tests, four Atterberg Limits tests, and four constant rate of strain consolidation (CRSC) tests (used to determine compressibility and stress history characteristics of marine clay). Prior to CRSC testing, radiography tests were conducted on tube samples to aid in assessing the sample quality, general material type and presence of areas of disturbance and variations in soils retrieved. A summary of laboratory test results completed on collected samples of marine clay is provided below.

- Natural Water Content: 31% to 40%
- Atterberg Limits:
    - Liquid Limit (LL): 23% to 38%
    - Plastic Limit (PL): 14% to 18%
    - Plasticity Index (PI): 9% to 20%
- Total Unit Weight: 108 pcf to 118 pcf

In addition to the laboratory testing performed and summarized above, four grain size analyses were conducted on samples of marine sand collected from test boring BB-FPR-103. The testing was completed in order to aid in assessing the reuse potential of soil generated from excavating the cofferdam used to construct the Pier 1 substructure. The results of grain size analyses are summarized below.

Test Boring (Sample No.)	Sample Depth (ft, BGS)	Percent Gravel	Percent Sand (course/med./fine)	Percent Fines[1]	USCS Classification
BB-FPR-103 (D10)	18.0-20.0	2.7	78.8 (0/1/78)	18.5	SP
BB-FPR-103 (D11)	28.0-30.0	0.0	87.8 (1/5/82)	12.2	SP
BB-FPR-103 (D12)	38.0-40.0	0.2	98.9 (1/55/43)	0.9	SP
BB-FPR-103 (D13)	48.0-50.0	0.4	94.6 (4/48/43)	5.0	SP

[1]- Refers to the percentage passing the No. 200 (0.075 mm) sieve.

5.2 Design Phase Laboratory Soil Testing & Results

The testing program included eight natural water content tests, eight Atterberg Limits tests, and four CRSC tests. Similar to the preliminary phase laboratory testing, radiography tests were conducted on tube samples to aid in assessing the sample quality, general material type and presence of areas of disturbance and variations in soils retrieved prior to CRSC testing. A summary of laboratory test results completed on collected samples of marine clay is provided below.

- Natural Water Content: 31% to 53%
- Atterberg Limits:
  - Liquid Limit (LL): 23% to 48%
  - Plastic Limit (PL): 15% to 23%
  - Plasticity Index (PI): 8% to 25%
- Total Unit Weight: 105 pcf to 114 pcf

6. STRENGTH AND COMPRESSIBILITY CHARACTERISTICS OF MARINE CLAY

The undrained shear strength of the marine clay stratum was estimated using in-situ vane shear tests conducted during drilling of the preliminary and design phase test borings. Measured peak undrained shear strengths varied from approximately 200 to 1,100 pounds per square foot (psf).

The stress history of the deposit was estimated by comparing measured undrained shear strength values and estimated values of maximum past pressure from the CRSC tests to estimate the overconsolidation ratio (OCR) of the marine clay. Using the design shear strength profile, an empirical approach known as Stress History and Normalized Engineering Properties (SHANSEP) was used to establish a profile of maximum past pressure versus depth as a function of the shear strength profile. The design maximum past pressure profile is shown in comparison to the laboratory consolidation data and the existing effective overburden pressure in Sheet 4 (north approach) and Sheet 5 (south approach).

The stress-strain or compressibility characteristics of clay deposits are highly dependent upon their stress history. Overconsolidation is a condition that results from the clay deposit having been exposed, at some time in the geologic past, to stresses greater then the present in-place stresses. If the clay deposit is stressed within the limits of the maximum previous stress (i.e., maximum past pressure), the magnitude of settlement will be a function of the recompression ratio (RR) of the clay. If the applied stress exceeds the maximum previous stress, the magnitude of settlement will be a function of the virgin compression ratio (CR). Measured values of CR are typically 10 to 25 times greater than RR, and consolidation settlement is directly correlated with the value of CR or RR. Therefore, the estimated settlement for normally consolidated clay would be 10 to 25 times greater than that of overconsolidated clay for the same stress increase. Measured CR and RR values from the clay samples tested on both sides of the river ranged from 0.23 to 0.32, and from 0.002 to 0.031, respectively.

The data indicates that the marine clay is lightly to moderately overconsolidated along the entire project alignment. The upper 5 to 10 ft of the marine clay stratum along the entire alignment consists of an overconsolidated “crust”, which is overconsolidated by 3,000 psf at the north approach and possibly more at the south approach, likely due to historical drying and desiccation. Based on the consolidation test results conducted in test borings drilled along the north approach, we estimate that the marine clay deposit below the crust in this area is overconsolidated by approximately 500 psf. Therefore, the marine clay would be highly compressible under an embankment load that results in greater than 500 psf stress increase in the clay deposit below the crust. This was considered the critical compressibility profile for design of the north approach embankment in our evaluations. Based on the consolidation test results conducted in test borings BB-FRR-102 and BB-FPR-102, we estimate that the marine clay deposit (within the flood plain along the south approach) is overconsolidated by at least 1,700 psf. The larger overconsolidation in the floodplain portion of the south approach is likely due to post-glacial erosion, considering that the preconsolidation pressure values are similar to the estimated values at similar elevations along the north approach.

7. GEOTECHNICAL DESIGN RECOMMENDATIONS

Geotechnical design recommendations for the subject project, as discussed and provided herein, were developed in accordance with the following documents:

- AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications, Fourth Edition, 2007 with Interim Revisions through 2009 and
- MaineDOT Bridge Design Guide (BDG), August 2003.

Preliminary and design phase memoranda as well as supplemental engineering calculations are provided for reference in Section 3.

7.1 Approach Embankment Design Considerations

Subsurface soil conditions along the roadway alignment will significantly affect the planning and design of the proposed construction. Portions of the proposed approach embankments within the limits of the existing embankments will be raised by approximately 3 to 4 ft. Maximum raises in grade outside of the existing roadway range from 26 ft (north approach) to 33 ft (south approach). Engineering evaluations were conducted in order to assess the feasibility of constructing the approach embankments as it relates to the presence of a 75 to 80-ft thick layer of soft to medium stiff, slightly overconsolidated marine clay at the north end of the project and a surficial, 10 to 25-ft thick, very loose to medium dense, alluvial soil deposit at the south end of the project. Refer to memorandums dated 26 December 2008 and 16 January 2009 provided in Section 3 for additional details related to the north and south approach embankment evaluations.

7.1.1 Normal Weight Earthfill

Consolidation settlement and global embankment stability evaluations were conducted in order to assess the feasibility of constructing the proposed approach embankments using normal weight earthfill.

7.1.1.1 Consolidation Settlement

Approach embankment construction using normal weight earthfill will cause consolidation settlement of the underlying marine clay particularly along the north approach and to a lesser extent along the south approach. Estimates of the magnitude of ground surface settlement (primary consolidation; secondary settlement not included) using normal weight earthfill to construct the approach embankments were calculated and are summarized below.

Station (ft)	Estimated Consolidation Settlement (in.)
111+25	9
119+00	19
119+50	12
120+00	7

The use of lightweight fill material would result in a reduction in the magnitude of settlement. Although several lightweight fill alternatives exist, each would still require some thickness of normal weight earthfill cover in order to construct the roadway pavement section to provide acceptable roadway performance. Lightweight fill alternatives are discussed in subsequent sections of this report.

7.1.1.2 Global Embankment Stability

Approach embankment construction using normal weight earthfill could cause excessive vertical and lateral strains eventually resulting in a shear failure of the foundation soil and subsequent failure of the embankments. A series of computer-assisted, two-dimensional global stability evaluations were performed using the computer program Slide 5.0. The existing approach embankments were evaluated and factors of safety calculated in order to provide a basis for comparison to the proposed embankment construction. The results of the existing site conditions are summarized below.

Location	Calculated Factor of Safety
Existing South Approach Embankment	1.4
Existing North Approach Embankment	1.2

Note: Approach embankments were evaluated along the centerline of the existing roadway.

It is our opinion that the calculated factors of safety summarized above are conservative. The slope stability software used for these analyses models the approach embankments as an infinitely wide embankment; which is conservative based on the finite width of the embankments. Therefore, actual factors of safety for a three-dimensional model would be somewhat higher. We used a factor of safety of 1.3 as the basis for embankment design. We believe this corresponds to a factor of safety of approximately 1.5 for the three-dimensional condition.

Proposed approach embankments were initially evaluated assuming that normal weight earthfill was used as embankment fill for both the north and south approaches. The results of global embankment stability analyses using normal weight earthfill are summarized below.

Location	Calculated Factor of Safety
South Approach Embankment at STA 111+45 (approx.)	1.0
North Approach Embankment at STA 118+65 (approx.)	0.8

Note: Approach embankments were evaluated along the centerline of the Existing roadway.

It is our opinion that the calculated factor of safety for normal weight earthfill is too low. Increasing the factor of safety could be accomplished by either reducing the driving forces or increasing the resisting forces (e.g., by increasing the strength of the marine clay (north) and alluvial (south) soils).

7.1.1.3 Embankment Construction Techniques

Based on the results of consolidation settlement and global stability evaluations summarized above, conventional staged embankment construction utilizing surcharging and prefabricated vertical drains (PV drains) is considered the only way to safely construct the approach embankments without causing excessive post-construction settlement and/or shear failure of the foundation soils (assuming normal weight earthfill is used to construct the embankments.

This construction technique involves the installation of PV drains through the marine clay profile to accelerate drainage of water from the clay (consolidation process). The earthfill embankments would be constructed in stages based on bearing capacity considerations and a surcharge (additional thickness of normal weight earthfill) would be left in place. This would allow the marine clay to gain strength through consolidation thereby increasing calculated factors of safety against embankment instability and forcing settlement to occur before the roadway is completed.

This construction sequence would allow normal weight earthfill to be used to construct the approach embankments which would be considerably less expensive than using lightweight fill material. However, this process would require more time to complete embankment construction in order to allow the consolidation process and strength gain process to occur. In addition, it is our understanding that the existing bridge will remain in service during construction of the new bridge. Therefore, due to the proximity of the proposed approach embankment to the existing bridge substructures, the use of normal weight earthfill and staged embankment construction methods would cause additional loading (downdrag) of the existing timber pile foundations that would result in excessive settlement of the existing bridge structure.

Per our discussions with TY Lin and MaineDOT, it is our opinion that this embankment construction alternative is not considered to be feasible for the project due to the extended construction duration and the potential negative impacts it would have on the existing bridge substructures.

7.1.2 Approach Embankment Design Conclusions

Based on the results of our evaluations considering the use of normal weight earthfill to construct the north and south approach embankments we conclude the following:

- Settlement considerations, and to a lesser extent global stability, controlled the design of the north approach embankment.
- Global stability considerations controlled the design of the south approach embankment.

7.1.3 Lightweight Fill

Based on the results of preliminary evaluations as described above, the use of lightweight fill material is required to minimize post-construction settlement of the north approach embankment and to reduce driving forces (thereby increasing calculated factors of safety against embankment instability) of the south approach embankment.

We considered the use of the following lightweight fill materials for this project:

Lightweight Fill Material	Total Unit Weight (lb/ft^3)
Tire Derived Aggregate (TDA)	60
Expanded Shale (ES)	55 to 70
Expanded Polystyrene (geofoam)	2 to 4

Due to the high total unit weight of both TDA and ES (as compared to geofoam), excessive post-construction settlement was still anticipated along the north approach embankment and driving forces were not reduced enough to provide acceptable factors of safety against rotational failure of the south approach embankment.

Therefore, settlement and global embankment stability evaluations, similar to those performed using normal weight earthfill were completed using geofoam in order to determine the type and extent of lightweight fill required to satisfy settlement and global stability requirements.

North and south approach embankment stability evaluations were conducted modeling several proposed bridge abutment locations with various combinations of geofoam and normal weight earthfill. Additional details are provided below.

7.1.3.1 South Approach Embankment and Abutment No. 1

Global stability evaluations were conducted modeling four proposed bridge abutment location alternatives:

- Alternative No. 1A (STA 111+42): using geofoam extending 60 ft behind the abutment and within the approach embankment, with a rockfill toe berm in front of the pile-supported stub abutment, and a riprap slope (on top of the toe berm) extending into the river.
- Alternative No. 1B (STA 111+42): using geofoam extending 70 ft behind the abutment and within the approach embankment, with a wrapped face reinforced soil mass behind the geofoam cell, a full-height, vertical-sided MSE wall in front of the stub abutment and a riprap slope extending into the river for scour protection.
- Alternative No. 2A (STA 111+00): using geofoam extending 25 ft behind the abutment, with a rockfill toe berm in front of the pile-supported stub abutment and a riprap slope (on top of the toe berm) extending into the river.
- Alternative No. 2B (STA 111+00): using geofoam extending 25 ft behind the abutment, with a wrapped face reinforced soil mass behind the geofoam cell, a full-height, vertical sided MSE wall in front of the pile-supported stub abutment, and a riprap slope extending into the river for scour protection.

Analyses indicated that a combination of geofoam and normal-weight fill is practicable for both Alternatives No. 1 and 2. However, the global stability of the abutment and approach embankment in Alternative No. 1 is sensitive to the stability of the alluvial soils between the abutment and the river. Therefore, additional measures would be needed to improve the properties of the alluvial soils in this area (i.e., ground improvement) or to increase the volume of geofoam to ensure stability during the design life of the bridge.

Cost implications for each of the abutment location alternatives were evaluated and compared to the cost of an equivalent bridge superstructure. Based on our evaluation, it was determined that it would be more cost effective to construct the abutment at STA 111+42 (Alternative No. 1) primarily due to the increased cost of the bridge superstructure associated with Alternative No. 2. Furthermore, it was determined that it would be more cost effective to construct a pile-supported stub abutment on a geofoam fill embankment with a rockfill toe berm (Alternative 1A) as compared to constructing an MSE wall (Alternative 1B).

Therefore, we recommend that Alternative No. 1A, as described above, be used as the basis for design.

7.1.3.2 North Approach Embankment and Abutment No. 2

Preliminary-level evaluations suggested that global stability of the approach embankments control both the plan location and design of the approach embankment and bridge abutment. Global stability evaluations were conducted modeling three proposed bridge abutment location alternatives:

- Alternative No. 1 = Station 118+65
- Alternative No. 2 = Station 119+05
- Alternative No. 3 = Station 119+45

Each alternative was evaluated using both normal-weight earthfill and lightweight fill to construct the approach embankment. Based on the results of the global stability evaluations, each abutment location alternative was found to be technically feasible by using various quantities of geofoam behind the proposed bridge abutment and within the approach embankment.

Cost implications for each abutment location alternative was evaluated and compared to the cost of an equivalent bridge superstructure. Results indicated that constructing the north abutment at location Alternative No. 1 was the most cost effective solution.

7.1.4 Expanded Polystyrene (EPS) Geofoam

The geofoam will be protected by a nominal 5-ft thick layer of normal-weight earth fill will and a relatively thin (4 to 6 in.) concrete distribution slab. The geofoam blocks will experience elastic compression under the weight of overlying embankment fill, pavement base/subbase/asphalt materials and related surcharge loads. Long term (creep) deformation can also occur if the elastic strain within the geofoam mass exceeds 1 percent strain. In both cases, the magnitude of vertical deformation (elastic and creep) is related to the elastic modulus (stiffness) of the specific grade of geofoam. In order to minimize the total vertical deformation of the approach embankments, we concluded that the type of geofoam used to construct the approach embankment would need to strain less than 1 percent under dead and live loads.

ASTM defines several different grades of geofoam. A summary of the physical properties of select grades of geofoam are provided below.

Geofoam Grade	Minimum Density (pcf)	Compressive Resistance At 1 Percent Deformation (psi)	Elastic Modulus (psi)
EPS19	1.15	5.8	580
EPS22	1.35	7.3	730
EPS29	1.80	10.9	1,090
EPS39	2.40	15.0	1,500

For the purposes of our evaluations, a uniform vertical load equal to 1,000 psf was applied to the top of the geofoam mass. This load includes the dead load equivalent to approximately 6 ft of normal weight earthfill overlying the geofoam mass and an assumed live load surcharge equal to 250 psf. Based on this applied load, elastic stress and strain were calculated for various grades of geofoam. The calculated elastic strain for each grade of geofoam is summarized below.

Geofoam Grade	Elastic Strain (percent)
EPS19	0.95 to 1.20
EPS22	0.75 to 0.95
EPS29	0.51 to 0.64
EPS39	0.37 to 0.46

The range of calculated elastic strain within the geofoam indicate that creep deformation on the order of 3 to 4 in. at the north approach embankment would be anticipated if EPS19, and possibly if EPS22, grade geofoam was used, because elastic strains are approaching or in excess of 1 percent. Therefore, we recommend that a material with the minimum physical properties of EPS29 be used to construct the north and south approach embankments in order to minimize post-construction creep deformations.

Elastic compression of the geofoam blocks was calculated along the length of the north and south approach embankments based on the physical properties of EPS29 and are summarized below.

Station (ft)	Approximate Geofoam Thickness (ft)	Elastic Compression (in.)	Long Term (Creep) Compression (in.)
110+75 to 110+90	12	1	negligible
110+90 to 111+08	16	1¼ to 1½	negligible
111+08 to 111+15	20	1½ to 1¾	negligible
111+15 to 111+35	22	1½ to 1¾	negligible
118+74 to 118+90	16	< 1	negligible
118+90 to 119+00	27	1¾ to 2¼	negligible
119+00 to 119+50	20	1¼ to 1¾	negligible
119+50 to 120+25	9	¾ to 1	negligible
120+25 to 120+75	2	0 to ¼	negligible

The elastic compression of the geofoam blocks will generally occur during embankment construction, prior to roadway paving (i.e., construction of the concrete distribution slab and placement of embankment fill and pavement base/subbase materials). Since the elastic compression of the geofoam will occur prior to paving, we do not anticipate elastic deformations of the geofoam will impact roadway/pavement performance.

It should be noted that the thickness of the geofoam also varies transverse to the project baseline. Therefore, there may be some differential deformation within the embankment (again, deformations will take place during embankment construction). Furthermore, the geofoam outside the limits of the travel lanes will not be subjected to the full design loading condition and will likely deform less than the values shown above.

Please refer to a design memorandum dated 8 September 2009 and supporting calculations in Section 3 for additional details.

7.1.5 Estimated EPS Geofoam Fill Volumes

Based on the proposed bridge alignment and existing and proposed roadway grades, we have estimated the volume of geofoam fill that will be required to construct the north and south approach embankments as follows:

Location	Estimated Volume of EPS Geofoam (cy)
Phase I Geofoam South	2,600 cy
Phase I Geofoam North	2,875 cy
Phase II Geofoam South	100 cy
Phase II Geofoam North	1,025 cy
	Total = 6,660 cy

Please recall that we initially estimated that approximately 6,200 cy of geofoam would be needed in the Preliminary Design Report (PDR) prepared by TY Lin. The primary reason for the increase between the preliminary and design phases is the presence of geofoam on the west side of the proposed approach embankments and beneath the existing bridge structure (phase II geofoam) as well as the proposed grading.

7.2 Liquefaction Evaluations

The liquefaction susceptibility of the granular soil deposits at the subject site was evaluated based on the subsurface conditions encountered in the preliminary and design-phase test borings drilled for the project. The liquefaction evaluations discussed herein have been conducted in general accordance with the requirements of LRFD Specifications, Appendix A10, "Seismic Analysis and Design of Foundations."

An initial liquefaction evaluation was conducted that included all of the geologic strata based on the results of the preliminary test borings. The results of this evaluation indicated that the alluvial deposit at the site was potentially susceptible to liquefaction during a design-level earthquake. Therefore, supplemental explorations and evaluations were conducted to further identify the liquefaction potential of the alluvial soils. Our evaluations were conducted in two phases: 1) initial liquefaction evaluation and 2) site-specific liquefaction evaluation. Our results and conclusions are presented in the following subsections.

7.2.1 Seismic Site Class

Based on the corrected SPT blow count (granular soils) and undrained shear strength (cohesive soils) obtained from the preliminary and design phase test borings, the south portion of the alignment is considered Site Class "D" and the northern portion of the alignment is considered Site Class "E" in accordance with Table 3.10.3.1-1. Based on Site Class and geographic location, values of peak ground acceleration were developed in accordance with AASHTO LRFD for use in the initial liquefaction evaluation discussed below.

7.2.2 Initial Liquefaction Evaluation

The liquefaction susceptibility of the granular soils at the site was determined by comparing the equivalent uniform cyclic stress ratio (CSR) imposed by the design earthquake to the cyclic resistance ratio (CRR) of the in-situ soils at each sample location. Liquefaction of the in-situ granular soils would occur when the CRR is less than or equal to the CSR. In the instance where the CRR equals the CSR the factor of safety against liquefaction (FS_{liq}) is equal to 1.0. In Appendix A10 of the LRFD Specifications, it is suggested that a FS_{liq} value of 1.5 or greater is desirable to establish "a reasonable margin of safety against liquefaction in the case of important bridge sites."

CRR is a function of clean sand-corrected blow counts, $N1_{60\text{-}CS}$, following the simplified empirical methodology (referred to as "simplified method") originally developed by Seed et al. (1985), and most recently updated by Idriss and Boulanger (2008). $N1_{60\text{-}CS}$ values consist of field SPT N-values that have been corrected for in-situ effective overburden pressure, borehole diameter, hammer type, drill rod length, and percent passing the No. 200 sieve (i.e., fines). $N1_{60\text{-}CS}$ values were calculated for the alluvial deposits encountered on both sides of the river for use in the liquefaction evaluation. Alluvial deposits with greater than 50 percent passing the No. 200 sieve (i.e., silt and clay soils) were not considered in this evaluation because the simplified method is not intended to be used for silt and clay soils. These soils are not considered liquefiable during the design earthquake at the site. The calculated values and a summary of the correction factors are presented on Figure 6.

The "baseline" CRR vs. $N1_{60\text{-}CS}$ correlation is based on an earthquake magnitude (M) equal to 7.5 and an effective overburden pressure of 1 atmosphere (atm) (approximately 2,000 psf). Therefore, correction factors developed by Seed and recently updated by Idriss and Boulanger were used to account for the design earthquake magnitude for this site (assumed M = 6.5 for initial evaluation; typically the maximum considered in the northeast) and the actual effective overburden pressure at each sample location.

The CSR is calculated in accordance with the simplified method as a function of the peak horizontal ground acceleration of the design earthquake and an empirically based stress reduction factor. Values of peak ground acceleration used in the initial evaluation were developed in accordance with the seismic design methodology of the LRFD Specifications. The seismic hazard level defined in the LRFD Specifications corresponds to a 7 percent probability of exceedance in 75 years, or a 1,000-year earthquake event. This seismic hazard defined by the LRFD Specifications is based on the 2003 version of the United States Geological Survey (USGS) probabilistic database. The northern and southern sides of the river were classified as Site Class “E” and Site Class “D”, respectively, as described previously in this report. The stress reduction factor was calculated in accordance with Idriss and Boulanger (2008).

7.2.3 Results of Initial Liquefaction Evaluation

Calculated values of CRR and CSR based on the simplified method and the resulting values of FS_{liq} for each sample in the near-surface alluvial soils have been graphically summarized on Figures 7 and 8 for the areas north and south of the river, respectively. The results of the initial liquefaction evaluation show that corrected SPT blow counts measured within the near-surface alluvial deposit on the north and south sides of the river result in FS_{liq} values generally less than 1.0.

Considering that the results indicated a potential for widespread liquefaction, slope stability evaluations were conducted assuming that the alluvial deposits had liquefied and had a reduced, residual undrained shear strength of 200 psf. The results of these evaluations indicated that the post-earthquake slope stability safety factor would be less than 1.0, and liquefaction-induced lateral spreading was likely to occur.

Based on these results, we concluded that post-earthquake slope stability was a concern for the north and south approach embankments, and foundation/substructure design would be impacted by additional lateral soil loads resulting from lateral spreading. Accordingly, the following issues were evaluated by Haley & Aldrich and TY Lin during the design development process:

- Reduction/elimination of lateral pile capacity during the design earthquake event at Piers 1 and 2;
- Forces and moments induced on foundations and superstructure at Piers 1 and 2 by lateral spreading of the near-surface alluvial deposit; and
- Ground improvement alternatives to remediate liquefaction potential as an alternative to designing the structural elements for earthquake induced loading.

The general conclusion was that the design and cost impact of the liquefaction hazard as defined by the initial evaluation conducted based on the simplified method would be substantial, specifically related to lateral spreading impacts on Piers 1 and 2, which would have required nearly twice as many piles to resist lateral spreading loads as would be needed for the static case (with no lateral spreading forces). Therefore, a site-specific liquefaction evaluation was considered warranted to refine the results prior to moving forward with substantial additional foundation/ground improvement measures.

7.2.4 Site-Specific Liquefaction Evaluation

A site-specific liquefaction evaluation was conducted by performing site-specific response analyses (SSRA) for representative subsurface profiles for the north and south sides of the river. The results of the SSRA would be used to directly calculate maximum shear stresses in the alluvial deposit resulting from a 1,000-year earthquake event, which are converted to CSR values for use in the liquefaction evaluations. The SSRA conducted for the liquefaction evaluation included the following steps:

1. Use deaggregations obtained from the 2003 USGS database to prepare bedrock uniform hazard spectrum (UHS) for the site (based on site latitude/longitude coordinates) for a 1,000-year earthquake event (LRFD Specifications hazard level).

2. Determine representative soil profiles for the north and south sides of the river for use in the ground response analyses (see table below for summary of profiles).

3. Estimate lower-bound and upper-bound shear wave velocity ranges using empirical correlations (based on soil type and strength) for the geologic strata to be used in the ground response analyses (see table below for summary of values).

4. Use two earthquake input ground motions (recorded ground motions from seismographs for previous earthquakes), and scale the ground motions so that spectral content “matches” with the UHS to develop input ground motions (i.e., seismograms) scaled for the appropriate earthquake hazard level.

5. Use the computer software Proshake to perform one-dimensional equivalent linear ground response analyses to determine peak ground acceleration and peak shear stress values in the alluvial deposits.

6. Use the peak shear stress values to develop site-specific CSR values developed using lower-bound and upper-bound shear wave velocity profiles, and develop a mean CSR profile for comparison to CRR values and determination of $FS_{liq.}$

The generalized one-dimensional soil profiles and shear wave velocity ranges used in the SSRA are summarized in the following table.

Geologic Stratum	Idealized Stratum Thickness (ft)		Range in Shear Wave Velocity (ft/sec)	
	North of River	South of River	North of River	South of River
Fill/Alluvial Deposit	30	20	150-600	150-530
Marine Clay	30	20	250-550	250-550
Marine Sand	60	10	700-1,150	750-950
Glacial Till	40	80	1,000-2,000	1,000-2,000
Bedrock	Infinite	Infinite	2,500-3,500	2,500-3,500

Four sets of site-specific CSR values were generated each for the north and south sides of the river, corresponding to two input ground motions analyzed using the lower-bound and upper-bound shear wave velocity profiles on each side of the river. The mean CSR was calculated as the average of the four CSR profiles for each side of the river.

Modifications were also made to the CRR values used for the site-specific liquefaction evaluations. The corrected $N1_{60\text{-}CS}$ values presented on Figure 6 remained unchanged, but the deaggregations obtained from the 2003 USGS database indicated that the 1,000-year earthquake hazard level at the site is controlled by an earthquake with a magnitude of between 5.7 and 6.0. Therefore, the assumed magnitude was reduced from 6.5 to 6.0, which increased the CRR values. This also allowed for the increase of the scaling factor that accounts for the magnitude (defined as magnitude scaling factor [MSF]) from 1.3 to 2.0 in accordance with Youd et al (2001) for an M = 6.0 earthquake.

7.2.5 Results of Site-Specific Liquefaction Evaluation

Calculated values of CRR and CSR and the resulting values of FS_{liq} (calculated as CRR divided by mean CSR) for each sample in the near-surface alluvial soils have been graphically summarized on Figures 9 and 10 for the areas north and south of the river, respectively.

The results presented on Figure 9 for the north side of the river indicate that all of the encountered soils result in FS_{liq} values greater than 1.5. Therefore, we conclude that the potential for liquefaction to occur on the north side of the river is very low, and there will be no liquefaction-induced instability or loading in the vicinity of the proposed structure.

The results presented on Figure 10 for the south side of the river indicate that all of the encountered soils except for one data point (boring BB-FPR-206, sample depth 6 to 8 ft BGS; FS_{liq} = 0.96) result in FS_{liq} values greater than 1.0, with values generally ranging between 1.1 and 1.7. We conclude that the isolated data point indicating a safety factor below 1.0 is not representative of the general conditions. Although the calculated FS_{liq} values are typically below the minimum desirable level of 1.5 indicated in the LRFD Specifications, we conclude that the potential for liquefaction-related impacts to the proposed bridge is low, and ground improvement is not warranted.

7.3 Bridge Abutment and Pier Foundation Design Recommendations

As shown on the interpretive geologic profile (Sheet 3), the subsurface conditions along the centerline of the proposed bridge alignment consist primarily of variable thicknesses of alluvial soils, marine clay, marine sand, glacial till and bedrock. The glacial till soils and bedrock are considered suitable for support of the bridge superstructure. Based on the depth to the suitable foundation bearing strata and the magnitude of the design loads, we consider driven pile foundations as the most practicable foundation alternative.

Specifically, the following driven pile foundation alternatives were evaluated to determine the most practicable and cost-effective system for the project:

- HP14x73 and HP14x117 steel H-piles (H-piles, non-displacement pile)
- 12-¾-in. diameter (0.375-in. wall thickness) and 16-in. diameter (½-in. wall thickness) concrete filled steel pipe piles driven with closed end (pipe piles, displacement pile)
- 16-in. square precast prestressed concrete (PPC) piles (displacement pile)

Each of the pile types listed above was found to be technically feasible as discussed in our 16 January 2009 memorandum (see Section 3). However, the use of steel H-piles driven to practicable refusal into dense glacial till or in/on bedrock was identified as the preferred option and is recommended to support the proposed bridge structure. Specific pile design recommendations are provided below (all Articles and Tables referenced below refer to AASHTO LRFD Bridge Design Specifications).

7.3.1 Axial Compression Pile Resistance

Since the piles will be driven to end bearing in/on bedrock, the structural resistance of the pile will control the design, as discussed in Article 10.7.3.2.3. Therefore, we recommend that the steel H-piles be designed for a nominal compressive resistance based on the structural resistance of the pile, in accordance with Article 6.9.4.1. The structural resistance factor (Article 6.5.4.2) for axial resistance of piles in compression and subject to damage due to severe driving is 0.5, therefore:

Steel H-pile Section	Factored Structural Resistance[1] (kip)
HP14x73	535
HP14x117	860
[1] – Values provided have not been reduced to account for downdrag or loss of cross-sectional area caused by corrosion of the steel.	

Downdrag occurs when the soil adjacent to an installed pile (typically the soft, compressible marine clay soils) moves downward relative to the pile (in this case, caused by compression of the soft marine clay/silt soils under the weight of newly placed fill material). Maximum raises in grade ranging from 26 ft (north approach) to 33 ft (south approach) will be required to construct the widened portion of the approaches outside the limits of the existing roadway. As discussed previously, geofoam will be used to construct the approach embankments in order to minimize post-construction settlement (north approach) and provide adequate factors of safety against slope instability (south approach). As a result, downdrag loading on piles located at Abutment No. 1 and Abutment No. 2 is considered negligible. In addition, since grades will remain virtually the same at Pier substructure locations, downdrag loading on piles at these locations is also considered to be negligible.

The geotechnical engineering design of the proposed piles also included consideration of corrosion in accordance with AASHTO LRFD requirements. Based on our visual review of the soil samples and our experience on similar projects with similar soil conditions; it is our opinion that the in-situ soils have low corrosive potential. Therefore, the net factored pile resistances provided above do not include a reduction in pile cross sectional area for steel degradation.

We recommend the pile tips be protected using cast steel driving shoes to prevent damage when driving through the dense glacial till and to/into bedrock.

7.3.2 Pile Group Evaluations

Based on the results of the initial liquefaction evaluation and subsequent development of additional lateral forces caused by lateral spreading of alluvial soils north of the river, TY Lin requested that Haley & Aldrich perform pile group evaluations for Pier 1 and Pier 2 substructures. TY Lin performed pile group evaluations for Abutment No. 1, Pier 3, and Abutment No. 2 substructures. The structural design of all substructure pile caps was completed by TY Lin with coordination from Haley & Aldrich at Pier 1 and Pier 2. Pier loading information was developed by TY Lin for the service, strength and extreme event limit states. The loads are summarized in Table V (Pier 1) and Table VI (Pier 2) and were provided at the proposed top of pile cap level (Pier 1 = El. -7.0 and Pier 2 = El. 18.0). In addition, tolerable deflection criteria were provided by TY Lin for assessment of pile groups under the service limit state.

Pile group analyses were performed using the computer program FB-MultiPier (FB-Pier Version 4). FB-MultiPier is a nonlinear finite element analysis program that is capable of analyzing multiple bridge pier structures interconnected by bridge spans. The program couples nonlinear structural finite element analysis with nonlinear static soil models for axial, lateral and torsional soil behavior to provide a system of analysis for coupled bridge pier structures and foundation systems.

The results of the initial liquefaction evaluation suggested that the alluvial soil deposit present north of the river was susceptible to liquefaction and could result in lateral spreading of the material into the river. As a result, the Pier 1 and Pier 2 substructures could be subjected to additional lateral forces. Lateral forces estimated to act on Pier 1 and Pier 2 substructures are summarized below.

Substructure	Ground Improvement North of River (Y/N)	Estimated Additional Lateral Force Caused by Lateral Spreading (kip)
Pier 1	N	800
Pier 1	Y	450 to 600
Pier 2	N	550 to 560
Pier 2	Y	Negligible

The additional lateral forces summarized above were modeled at each substructure (Pier 1 and Pier 2) location in combination with the superstructure loads provided by TY Lin. In order to determine the most cost-effective substructure design, several preliminary foundation design alternatives were investigated that considered the following:

- Designing substructures to resist additional lateral spreading forces.
- Minimizing lateral spreading forces acting on substructures by dropping foundation elements below the liquefiable zone.
- Installing ground improvement to negate additional forces acting on substructures.
- Cost benefit of supporting each concrete column with an individual group of piles compared to the cost of supporting both columns on one group of piles.

Based on the results of the preliminary pile group evaluations and subsequent cost estimates, we concluded the following:

- The cost impact of the liquefaction hazard (designing foundation to resist forces, installing ground improvement, dropping foundation down) as defined by the initial liquefaction evaluation was substantial compared to the static case.
- Cost savings could be realized if the pier columns were supported by one group of piles rather than two.
- Design of pile groups was controlled by moments acting on the piles at the base of the pier shafts during the extreme event which caused axial tension in the piles.

As a result of the cost impact of the liquefaction hazard, a site-specific liquefaction evaluation was conducted. The site-specific liquefaction evaluation concluded that the potential for liquefaction to occur on the north and south sides of the river was very low and low, respectively. Therefore, it was not necessary to account for additional lateral spreading forces in the substructure design evaluations. In addition, subsequent pile group evaluations were conducted modeling one large group of steel H-piles (HP14x117 and HP14x73) supporting both pier columns.

The factored geotechnical uplift resistance for an individual pile located within the Pier 1 and Pier 2 substructures under extreme event limit state loading is capable of resisting the applied loads. However, based on discussions with TY Lin, it is our understanding that the structural pile to pile cap connection is only capable of resisting approximately 65 kips. Therefore, the Pier 1 and Pier 2 pile groups were designed such that the maximum tension demand in an individual pile did not exceed 65 kips. Based on our evaluations, the design of the pile groups at Piers 1 and 2 are controlled by axial uplift forces during extreme event loading

The results of our evaluations show that the optimal Pier 1 pile group consists of a five by five pile group. The center-to-center pile spacing transverse and parallel to the alignment is 5.75 ft and 5.25 ft, respectively. The overall pile cap dimensions are approximately 25 ft by 85 ft. Perimeter piles are battered at 1H:12V in order to resist lateral loads.

The results of our evaluations show that the optimal Pier 2 pile group consists of six by six group of HP14x73 steel H-piles. The center-to-center pile spacing transverse and parallel to the alignment is 8.25 ft and 5.83 ft, respectively. The overall pile cap dimensions are approximately 34 ft by 45 ft. Perimeter piles are battered at 3H:12V in order to resist lateral loads.

A summary of the Pier 1 and Pier 2 pile group reactions are provided in Section 3.

7.3.3 Axial Tension Pile Resistance

As reported above, axial tension (uplift) demand (approximately 130 kips for an individual pile) during extreme event limit state loading was the controlling factor in determining the size, spacing, and number of piles required to support Pier 1 and Pier 2. Although not the controlling factor, piles also experience uplift at Pier 3 during strength and extreme event limit state loading. Based on conversations with TY Lin, it is our understanding that piles installed to support Abutment No. 1 and Abutment No. 2 do not experience uplift forces. In general, uplift in the piles is cause by overturning moments acting on the pile caps, at the base of the pier shafts.

Uplift loads will be resisted geotechnically by friction between the pile and the surrounding soil along the embedded pile length (piles driven to end bearing in/on bedrock as discussed in Section 7.3.1). The nominal uplift resistance of steel H-piles was evaluated in accordance with Article 10.7.3.8.6 with the strength limit state resistance factors specified in Article 10.5.5.2.3 and the extreme event limit state resistance factors specified in Article 10.5.5.3.3. The strength limit state resistance factor for driven piles subjected to uplift is 0.2. For uplift resistance during the extreme event limit state, the resistance factor is equal to 0.8. According to the methodology outlined herein, the factored geotechnical tension (uplift) resistance for individual HP14x73 steel H-pile proposed at each substructure location is as follows:

Substructure	Factored Geotechnical Uplift Resistance (per pile)[1]	
	Strength Limit State	Extreme Event Limit State
Pier 1	30 kips	110 kips
Pier 2	55 kips	220 kips
Pier 3	55 kips	220 kips

[1] – Piles are not subject to tension loading during service limit state loading.

7.3.4 Pile Settlement and Elastic Pile Compression

Pile settlement due to elastic shortening of the steel H-piles as well as pile tip settlement was evaluated based on the maximum factored Service Limit State loads generated from pile group evaluations. Estimates of elastic pile compression for an individual pile at each substructure are summarized below.

Substructure	Maximum Factored Service Limit State Load[1] (kip)	Approximate Elastic Pile Compression (in.)
Abutment No. 1	96	< 0.1
Pier 1	372	½
Pier 2	173	¼
Pier 3	364	½
Abutment No. 2	198	¼

[1] – Based on pile group evaluations performed by TY Lin and Haley & Aldrich.

The values do not include pile tip settlement, which is considered to be negligible for two primary reasons: 1) the relatively small load transmitted to the pile tip during service limit state loading and 2) the piles will be driven (installed) to resistances in excess of the maximum factored service limit state loads shown above and will therefore likely penetrate through any fractured, weathered or decomposed bedrock that would otherwise be present at the pile tip. The elastic shortening of the piles is anticipated to occur primarily during construction, soon after the superstructure loads are applied.

7.3.5 Pile Tip Elevations

As discussed previously, the piles are expected to develop the vast majority of their axial compressive resistance through end bearing in/on bedrock. In addition, the nominal and factored axial tension resistances summarized above are based on a fully embedded pile i.e., pile penetrating overburden soils. We do not anticipate that the piles will penetrate appreciably into the bedrock therefore, the recommended tip elevations for estimating pile lengths are based on interpolated bedrock elevations encountered in the preliminary and design phase test borings. For estimating bid quantities, we recommend the following pile tip elevations at each substructure location:

Substructure	Estimated Pile Tip Elevation
Abutment No. 1	El. -105
Pier 1	El. -98
Pier 2	El. -107
Pier 3	El. -105
Abutment No. 2	El. -123

We recommend that the order lengths of the piles reflect a minimum additional 5 ft of length in order to accommodate dynamic pile testing instrumentation.

7.4 Abutment, Panel Wall and MSE Wall Design Recommendations

7.4.1 Abutments & Panel Walls

As previously noted, a large portion of the approach embankments will be constructed using geofoam. Due to the proximity of the railroad right-of-way and adjacent property lines east of the north approach embankment, it was necessary to include a vertical-sided approach embankment. Although the geofoam blocks are self-supporting, they do require a vertical facing or soil cover for protection.

The geofoam also requires protection from traffic loads and petroleum based products in the event a spill occurs on the overlying roadway. Therefore, we recommend a nominal 5-ft thick layer of normal-weight earth fill be provided over the geofoam. The thickness of the earth fill is controlled by the depth of embedment required for guardrail posts. In addition, we recommend that a relatively thin (4 to 6 in.) concrete distribution slab be constructed over the geofoam cell, within the limits of the travel lanes, to distribute traffic loads. A high-density polyethylene (HDPE) liner should also be provided to protect the geofoam from petroleum based solvent (or other products that can degrade the geofoam) spills.

The combination of an earth retaining structure on top of a geofoam facing system created a demand for a unique wall system. The upper portion of the wall will be designed to resist lateral earth pressures from the retained normal-weight earth fill as well as traffic loads and impact loads applied to the guard rail. The lower portion of the wall will not be subjected to significant lateral pressures, primarily due to the self-supporting nature of the geofoam material.

A variety of wall systems were considered for the subject project and were evaluated based on technical feasibility, constructability, and cost. Refer to our memorandum dated 10 July 2009, which is provided Section 3, for additional details.

Based on the results of our evaluations and subsequent discussions with MaineDOT and TY Lin, we recommend that a wall system consisting of vertical precast prestressed concrete (PPC) panels supported on a continuous grade beam/footing be used to protect the geofoam and retain the normal weight earthfill. A similar "PPC panel wall" system was used to retain and protect a geofoam fill embankment as part of the Interstate 15 (I-15) Reconstruction project in Salt Lake City, Utah. The PPC panels were nominally 6-in. thick and 8-ft wide; the height of the panels varied but were generally less than 25 ft.

The PPC panels proposed for this project are restrained near the top by means of an approximate 5-ft long, 1-in. diameter threaded bar that is structurally connected to the distribution slab by a connection angle and shear stud that is cast directly into the distribution slab. Furthermore, since the geofoam blocks are self-supporting, we recommend that an approximate 3-in. wide air gap be provided between the outside face of the geofoam blocks and the inside face of the PPC panels. The air gap will eliminate the need to design the panel walls and abutments for additional lateral loading caused by elastic compression and volumetric changes as the geofoam is loaded. A summary of the proposed PPC panel walls is provided below.

Panel Wall No.	Location
Panel Wall 1 (south approach, west wall)	STA 111+01.04 (20.50' LT) to STA 111+17.16 (20.50' LT)
Panel Wall 2 (south approach, east wall)	STA 111+46.03 (25.50' RT) to STA 111+61.88 (25.50' RT)
Panel Wall 3 (north approach, west wall)	STA 118+46.87 (20.50' LT) to STA 118+78.87 (20.50' LT)
Panel Wall 4 (north approach, east wall)	STA 118+91.55 (25.50' RT) to STA 120+77.61 (29.27' RT)

We recommend that the grade beam/footing supporting the panel walls wall be designed based on a factored bearing resistance at the strength limit state equal to 1,500 psf. It is estimated that the panel walls could experience up to 1 in. of elastic and/or consolidation settlement of the marine and/or alluvial foundation soils. We anticipate that most of the predicted wall settlement will primarily occur during construction of the panel walls.

We recommend that the portion of the abutments and panel walls extending above the concrete distribution slab should be designed for lateral earth pressures using an equivalent fluid unit weight of 36 pounds per cubic foot (pcf) which assumes an active earth pressure coefficient of 0.3 and a soil unit weight of 120 pcf. This recommendation assumes the granular soil above the distribution slab will be drained and no unbalanced hydrostatic pressures will develop behind the abutments/panel walls. In addition, the panel walls should be designed for a live load surcharge equivalent to 2 ft of earthfill (equivalent to an area load of 250 psf; in accordance with Article A.11.1). A uniform horizontal load of 125 psf should be applied to the panel wall and abutment above the distribution slab to account for the live load surcharge. In accordance with Article A.11.1, the portion of the panel walls and abutments above the distribution slab should be designed for a uniform horizontal load equal to 55 psf to account for seismic soil loading.

We recommend that lateral loads acting on the concrete distribution slab, caused by the threaded bar connection with the panel walls, be resisted by friction between the distribution slab and both the overlying (granular borrow) and underlying (leveling sand) materials. We recommend that a coefficient of friction (tan δ) equal to 0.49 (for δ = 26°) be used to calculate the ultimate sliding resistance for the north and south approach embankment distribution slabs (above and below distribution slabs).

In addition to the lateral restraint provided in the upper portion of the panel walls by the threaded bar and anchor, lateral loads on proposed panel walls can be resisted by a combination of friction along the bases of the footings and passive pressure on the vertical faces of below grade footings. Frictional resistance should be calculated using a coefficient of friction (tan δ) between the footings and in-situ soil equal to 0.31 (for δ = 17°). In addition, approximately 6 kips per panel (750 lb/lf) can be used in passive resistance assuming a 2-ft (min.) thick grade beam/footing and a minimum 5 ft of normal weight earthfill is placed on top of the geofoam.

7.4.2 Mechanically Stabilized Earth (MSE) Walls

We recommend that a conventional MSE Wall be used to transition from the geofoam cored north approach embankment/Panel Wall 4 (STA 120+77.61) to the existing approach roadway (STA 122+00). We anticipate that the MSE wall will range from approximately 10 to 12 ft in height (4 to 8 ft of exposed face).

Design of the MSE wall system should be provided by the Contractor. We recommend that the system be designed in accordance with the AASHTO LRFD Bridge Design Specifications and the MaineDOT BDG.

Evaluations were conducted in order to assess the external stability of the MSE wall. Stability analyses included sliding, overturning (eccentricity), bearing capacity and global stability. The analyses were made assuming a reinforced soil zone extending approximately 70 percent of the maximum wall height behind the face of the wall, and that the reinforced soil and MSE wall facing would act as a rigid body. We recommend that the MSE wall be designed based on a factored bearing resistance at the strength limit state equal to 3,000 psf. In addition, the calculations indicate that the MSE wall meets the minimum requirements for external stability. The effect of seismic loading on MSE wall external stability (sliding, eccentricity, and bearing capacity) was checked and calculations indicate seismic stability of the wall is adequate. Calculations related to external stability analyses are included in Section 3 of this report. The MSE wall vendor is responsible for the design of the internal stability of the wall.

It is estimated that the placement of the approach embankment fill behind the MSE walls will cause consolidation and densification of the underlying marine sand and clay soils and settlement of the MSE walls. Approximately 1-¼-in. of wall settlement is expected due to elastic compression of the foundation soils. The MSE wall is expected to be able to tolerate this magnitude of total and differential settlement. We anticipate that most of the predicted wall settlement will occur during construction of the approach embankment and MSE wall

The following wall design comments and recommendations are offered for consideration:

- A concrete leveling pad/footing should be provided to support the wall face elements. The leveling pad should be at least 2-ft wide and designed to bear at a minimum depth of 4.5 ft below ground surface. The leveling pad should bear on a minimum of 12 in. of crushed stone (MaineDOT 703.31).
- The wall reinforcing should extend a minimum of 0.7 times the effective wall height behind the face elements.
- The wall should be designed by a licensed Professional Engineer in the State of Maine.
- A foundation drain (filter protected perforated drain pipe – minimum 4-in. diameter) should be provided behind the wall face elements to remove any water that may collect behind the wall.

7.4.3 Wrapped Face Reinforced Soil Mass

Based on the results of our evaluations at the south approach, we determined the plan and elevation limits of geofoam required to satisfy minimum factors of safety against global stability. The optimal geofoam configuration included a cell with a vertical face located approximately 65 ft south of Abutment No. 1 (STA 111+42). Due to the vertical nature of the geofoam cell, we recommend that a geotextile reinforced soil mass be constructed behind the cell in order to eliminate unbalanced lateral earth pressures on the geofoam mass.

7.4.4 Frost Protection

The minimum depth of embedment/cover for panel wall footings and MSE walls was evaluated in accordance with Section 5.2.1 of the MaineDOT BDG and Sections 10.6.1.2 and 11.10.2.2 of the LRFD Bridge Design Specifications. Based on a design freezing index equal to 1,300 freezing degree days, we recommend that the footings and walls bear a minimum of 4.5 ft below the lowest adjacent ground surface exposed to freezing.

8. CONSTRUCTION CONSIDERATIONS

8.1 Temporary Earth Support

Based on the anticipated elevation of the bottom of abutment and pier pile caps, existing site grades adjacent to the proposed substructures, and the proximity of railroad property lines relative to Pier 3 and Abutment No. 2, temporary earth support systems will likely be required to construct the substructures. In addition, due to the phased approach embankment construction described in Section 8.1, we anticipate that temporary earth support systems will be required in order to construct the new roadway alignment while the existing roadway remains in service.

Based on the subsurface soil, rock, and groundwater conditions at the site, we anticipate that the most cost effective excavation support system(s) for construction of the substructures will consist of the following:

Substructure	Approximate Maximum Height of Retained Soil (ft)	Potential Excavation Support System(s)
Abutment No. 1	NA	NA
Pier 1[1]	28 to 32 ft	steel sheeting
Pier 2	12 to 14 ft	steel sheeting, soldier piles & lagging
Pier 3	11 to 16 ft	steel sheeting, soldier piles & lagging
Abutment No. 2	8 to 11 ft	steel sheeting

[1] – approximate maximum height of retained soil is measured after completion of cofferdam excavation and prior to placement of tremie seal. Once tremie seal is in place, approximate maximum height of Retained soil ranges from 8 to 12 ft.

In general, temporary earth support system(s) are the responsibility of the Contractor and should be designed by a Licensed Professional Engineer in the State of Maine. We recommend that temporary earth support system(s) be designed to support all appropriate combinations of earth, geofoam, water and surcharge loads (from traffic, construction equipment, material stockpiles and other sources) imposed on the system(s) during all phases of the construction period. The proposed locations of Pier 3 and Abutment No. 2 are within approximately 60 ft (west) and 20 ft (east) of the centerline of the existing MCRR tracks. As a result, we recommend that the temporary earth support system(s) at these locations be designed and the railroad tracks instrumented/monitored in accordance with the latest edition of the Manual for Railway Engineering published by the American Railway Engineering and Maintenance-of-Way Association (AREMA). The temporary earth support system(s) should be designed such that new batter piles can be installed without interference. The Contractor is responsible for choosing an applicable factor of safety for the earth support system(s). The Contractor’s design shall also consider the means and methods and construction sequencing proposed by the Contractor. We recommend that design calculations and shop drawings be prepared by the Contractor and stamped by a Licensed Professional Engineer in the State of Maine and be submitted to MaineDOT for review prior to construction.

Based on the nature and phasing of the proposed construction we anticipate that some portions of the cofferdam used to construct the Pier 1 substructure and the temporary earth support systems used to construct the north and south approach embankments will be cutoff and left in place.

8.2 Dynamic Pile Load Testing Program

We recommend that the factored structural resistances be confirmed in the field using dynamic methods. The piles should be driven to a nominal resistance equal to the maximum factored axial compressive pile load divided by a resistance factor equal to 0.65 (Table 10.5.5.2.3-1). In accordance with Section 10.7.3.8.3, the minimum number of piles that should be dynamically tested to confirm factored structural resistance is based on the total number of piles (approximately 125 total) at the site and the variability (moderate) in subsurface soil conditions. We recommend that Contractor perform three dynamic pile load tests with 24-hour (minimum) restrike tests at each substructure location to evaluate hammer system efficiencies, driving stresses in the pile, and the nominal resistance of the piles. We recommend that the first and second dynamic pile load tests at each substructure location be completed on the first plumb and battered production piles driven. The one remaining test at each substructure location should be completed at a different location within the pile group, after approximately one half of the production piles have been installed at the pile cap. CAPWAP analysis should be performed on a select number of indicator piles installed during the dynamic test program.

8.3 Reuse of Excavated On-Site Soils

The volume of soil to be excavated to facilitate the construction of the Pier 1 substructure was estimated based on the dimensions of the pile cap and tremie seal and the subsurface conditions encountered in test boring BB-FPR-103. We have estimated that approximately 2,925 cy (375 alluvial, 1,250 cy marine clay, 1,300 cy marine sand) will be generated from the cofferdam excavation.

Based on the results of the grain size analyses conducted on samples of marine sand collected from test boring BB-FPR-103, we have determined that the material meets the minimum requirements of MaineDOT Standard Specification 703.19 - Granular Borrow. As a result, the excavated marine sand soils could be reused to construct portions of the north and south approach embankments. We estimate that the entire 1,300 cy of material could be used to construct a portion of the approach embankment south of STA 110+50 (behind the geofoam cell and wrapped-face reinforced soil mass).

Based on visual observation of collected samples of alluvial and marine clay soils, it is our opinion that these soils do not meet the minimum requirements of Granular Borrow. However, excavated alluvial and marine clay soils could still be reused to construct portions of the north and south approach embankments in accordance with MaineDOT Standard Detail 203(01) - Muck Excavation and Waste Disposal provided that the approach embankment fills are designed with slopes flatter than 2H:1V.

Potential areas for reusing the excavated alluvial and marine clay soils in accordance with MaineDOT Standard Detail 203(01) are: 1) on the west side of the north approach between STA 118+50 and the existing abutment and 2) on the east side of the south approach embankment between STA 109+50 and Abutment No. 1. In addition, the existing building owned by MaineDOT and located along the north approach does contain below-grade space. As currently planned, this building will be demolished as part of the proposed construction and the below-grade space will require filling. We estimate that approximately 150 cy of dredge material could be used to fill this area. We recommend that either the alluvial and/or marine clay soils be used to fill this area since neither could be used as granular borrow for embankment construction.

Based on our review of the plans and cross sections, it is our opinion that the location best suited for reusing the alluvial/marine clay dredge material is the east side of the south approach embankment, generally between STA 109+50 and Abutment No. 1. The existing ground surface level west of the existing north approach embankment and the approach embankment side slopes are relatively flat and in our opinion would not be a good location for alluvial/marine clay dredge reuse. We evaluated flattening the existing 2H:1V slope along the length of alignment between STA 109+50 and Abutment No. 1 in order to accommodate the 1,625 cy of alluvial and marine clay dredge soils in accordance with Standard Detail 203(01). We estimate that flattening the slope to 2.25H:1V will provide sufficient capacity (2,114 cy) to accommodate all of the alluvial and marine clay dredge soil. However, under this scenario, approximately 489 cy of embankment fill would still be needed to construct the flattened slope.

It is our understanding that MaineDOT has conducted a supplemental exploration program in the Presumpscot River in order to collect soil samples to submit for chemical testing. If the results of the chemical testing show that the soils are chemically impacted, MaineDOT will determine restrictions on reuse of these soils.

It is important to keep in mind the Pier 1 cofferdam may be excavated with a clamshell-type bucket and will likely take place in-the-wet. The dredge soils will be saturated and will likely require a lay down/stockpile area that can be used by the Contractor to moisture-condition (air dry) the dredge soils. Conditioning will be needed in order to achieve a moisture content suitable for placement and compaction of the soils.

8.4 Submittal Reviews

The contract drawings and specifications should be written so that the requirements of the documents are consistent with the design intent of the geotechnical recommendations outlined herein. Haley & Aldrich has worked with the design team to prepare the specifications and contract drawings related to the following topics:

- Temporary Lateral Support of Excavation
- geofoam
- Pile Installation and Testing

The contract specifications require that the Contractor and the Contractor's engineer perform analyses and submit results to MaineDOT for review. The design team should be allowed to review the geotechnical-related submittals to ensure that the Contractor's analyses/submittals are in accordance with the intent of the design. This will enable us to observe compliance with the design concepts, assumptions and specifications, and to facilitate design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

8.5 Construction Monitoring

The geotechnical design and earthwork recommendations contained herein are based on the known and predictable behavior of a properly engineered and constructed foundation. Monitoring of the foundation and approach embankment construction is required to enable the geotechnical engineer to keep in contact with procedures and techniques used in construction. Therefore, it is recommended that an individual representing MaineDOT, qualified by geotechnical training and experience be present at the site to provide monitoring during the approach embankment and foundation construction activities listed below:

- Placement of lightweight fill within approach embankments.
- Dynamic testing of the indicator piles and review of the PDA results.
- Installation of the production piles.

9. LIMITATIONS OF RECOMMENDATIONS

This report is prepared for the exclusive use of MaineDOT relative to the Replacement Bridge over Presumpscot River and Maine Central Railroad, Routes 26/100, in Falmouth, Maine. There are no intended beneficiaries other than MaineDOT. Haley & Aldrich shall owe no duty whatsoever to any other person or entity on account of the Agreement or the report. Use of this report by any person or entity other than MaineDOT for any purpose whatsoever is expressly forbidden unless such other person or entity obtains written authorization from MaineDOT and from Haley & Aldrich indicating that the Report is adequate for such other use. Use of this report by such other person or entity without the written authorization of MaineDOT and Haley & Aldrich shall be at such other person's or entities sole risk, and shall be without legal exposure or liability to Haley & Aldrich.

The analyses and recommendations are based, in part, upon the data obtained from the referenced subsurface explorations. The nature and extent of variations between explorations may not become evident until construction. If variations then appear, it may be necessary to reevaluate the recommendations of this report.

The planned construction will be supported on or in the soil at the site and below grade structures may be close to or penetrate the design groundwater level for the project. Recommendations for foundation and/or floor drainage, moisture protection, and/or waterproofing have been included herein, when appropriate. These recommendations address the conventional geotechnical engineering-related aspects of design and construction and are not intended to provide an environment that would prohibit infestation of mold or other biological pollutants. Our work scope did not include the development of criteria or procedures to minimize the risk of mold or other biological pollutant infestations in or near any structure.

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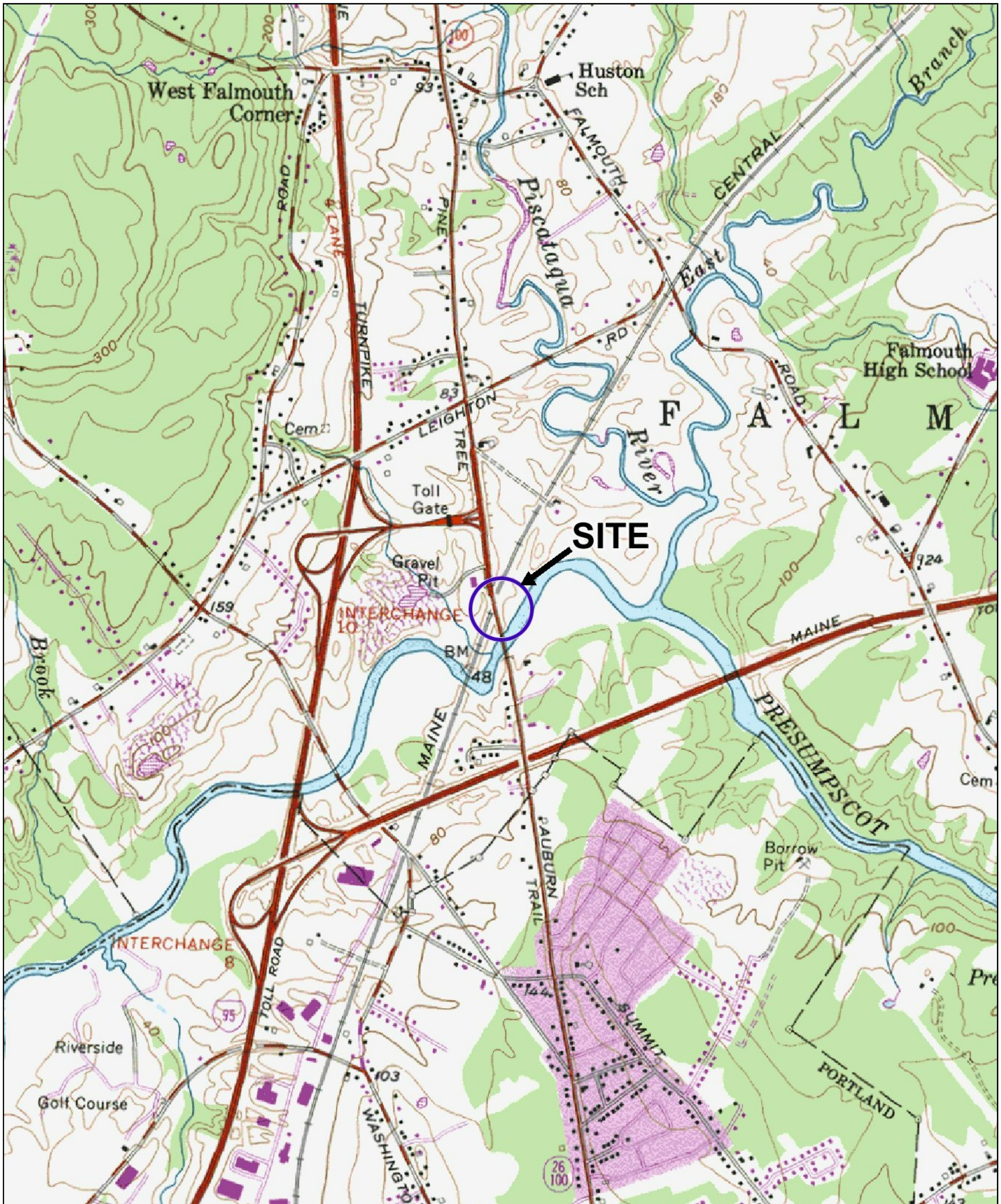
SECTION 2



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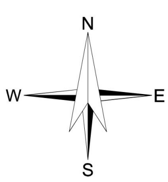

U.S.G.S. QUADRANGLE: PORTLAND WEST, ME

HALEY & ALDRICH

REPLACEMENT BRIDGE OVER PRESUMPSCOT RIVER AND MCRR
MAINEDOT PIN 15094.00
ROUTES 26/100 - FALMOUTH, MAINE

PROJECT LOCUS

SCALE: 1:24,000
NOVEMBER 2009

SHEET 1

35524-010 1.PDF

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LEGEND

BB-FPR-104 EL. 29.4	DESIGNATION, LOCATION AND GROUND SURFACE ELEVATION OF PRELIMINARY PHASE TEST BORINGS DRILLED BY MAINE TEST BORINGS OF BREWER, MAINE IN OCTOBER AND NOVEMBER 2008
BB-FPR-201 EL. 51.0	DESIGNATION, LOCATION AND GROUND SURFACE ELEVATION OF DESIGN PHASE TEST BORINGS DRILLED BY MAINE TEST BORINGS OF BREWER, MAINE IN APRIL AND MAY 2009
	ELEVATION CONTOUR OF EXISTING GROUND SURFACE
(OW)	DENOTES OBSERVATION WELL INSTALLED IN COMPLETED BOREHOLE
A A	DESIGNATION, LOCATION AND ORIENTATION OF SUBSURFACE PROFILE

NOTES:

1. BASEPLAN TAKEN FROM THE ELECTRONIC MICROSTATION FILE ENTITLED "3DMAPPING_25NOV08.DGN", DATED 25 NOVEMBER 2008 PREPARED BY STATE OF MAINE DEPARTMENT OF TRANSPORTATION (MAINEDOT).
2. EXISTING SITE CONDITIONS, CONTOURS OF EXISTING GROUND SURFACE ELEVATIONS, BATHYMETRIC INFORMATION IN THE RIVER AND LOCATION AND ORIENTATION OF EXISTING SITE FEATURES ARE TAKEN FROM ELECTRONIC MICROSTATION FILES PROVIDED BY MAINEDOT, DATE 31 DECEMBER 2008.
3. PROPOSED SITE CONDITIONS AND THE LOCATION AND ORIENTATION OF PROPOSED SITE FEATURES ARE TAKEN FROM ELECTRONIC MICROSTATION FILES PROVIDED BY TY LIN INTERNATIONAL ON 29 JANUARY 2009.
4. AS-DRILLED LOCATIONS OF TEST BORINGS WERE DETERMINED IN THE FIELD BY MAINEDOT USING GPS SURVEY EQUIPMENT.
5. ELEVATIONS ARE IN FEET AND REFERENCE THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD 88).
6. SUBSURFACE EXPLORATIONS WERE MONITORED IN THE FIELD BY HALEY & ALDRICH PERSONNEL.
7. REFER TO SECTION 2 OF REPORT ENTITLED "REPORT ON GEOTECHNICAL DESIGN REPORT, REPLACEMENT BRIDGE OVER PRESUMPSCOT RIVER AND MAINE CENTRAL RAILROAD, MAINE DOT PIN 15094.00, ROUTES 26/100 FALMOUTH, MAINE" PREPARED BY HALEY & ALDRICH, DATED 25 NOVEMBER 2009.

W S N E

0 60 120 180 240

SCALE IN FEET

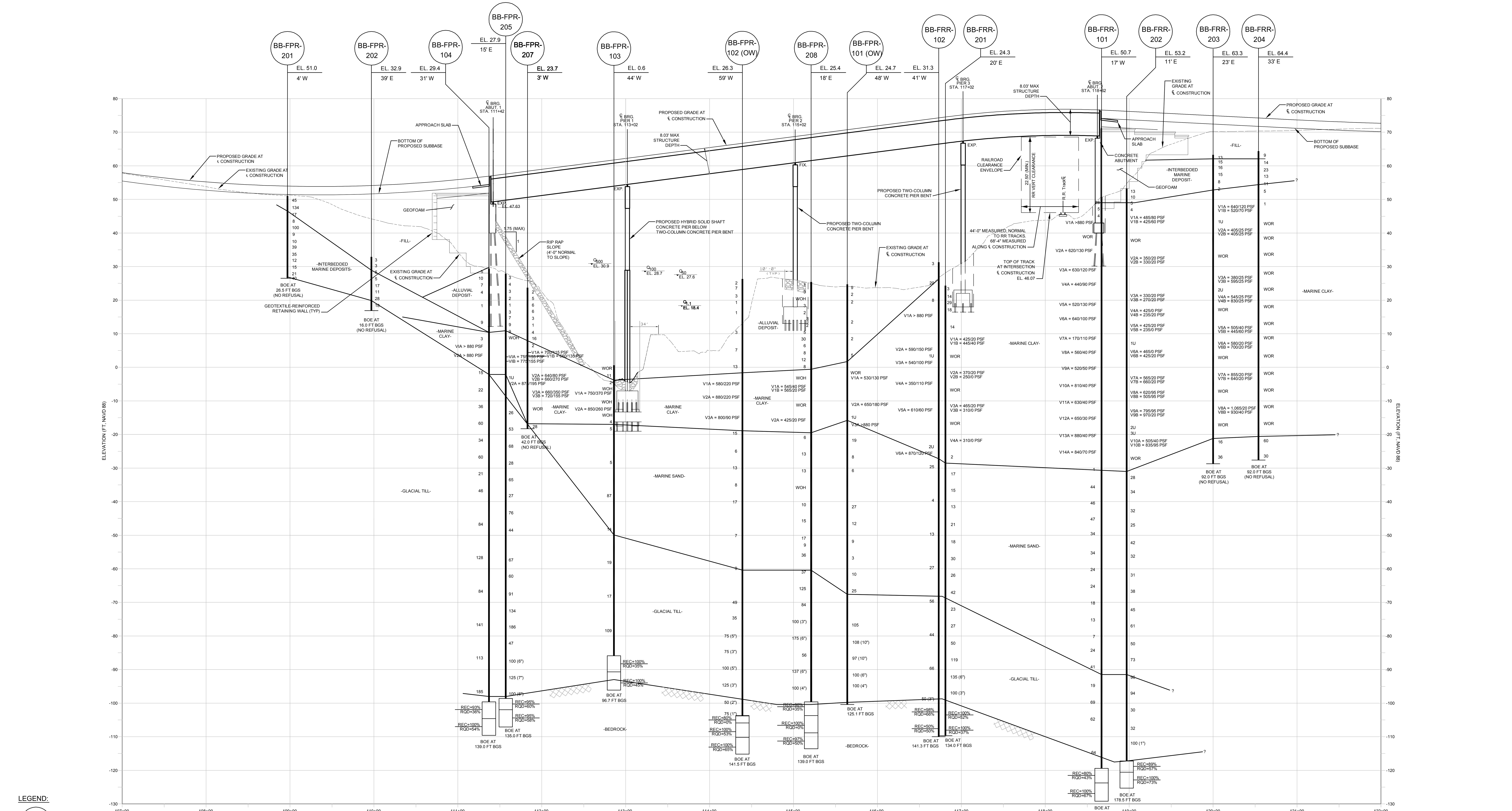
HALEY & ALDRICH

PROPOSED REPLACEMENT BRIDGE
OVER PRESUMPSCOT RIVER AND MCRR
ROUTES 26/100 - FALMOUTH, MAINE
MAINEDOT PIN 15094.00

SITE AND SUBSURFACE EXPLORATION LOCATION PLAN

SCALE: AS SHOWN
NOVEMBER 2009

SHEET 2



LEGEND:

BB-FPR-205 — DESIGNATION AND LOCATION OF TEST BORING DRILLED BY MAINE TEST BORINGS OF BREWER, MAINE IN 2008 AND 2009

EL. 27.9 — GROUND SURFACE ELEVATION AT TEST BORING LOCATION

15' E — APPROXIMATE PERPENDICULAR OFFSET DISTANCE AND DIRECTION MEASURED FROM THE SUBSURFACE PROFILE LINE (CONSTRUCTION CENTERLINE)

2 — CORRECTED STANDARD PENETRATION TEST (SPT) N-VALUE (N_{60} BLOWS PER FOOT); WOR = WEIGHT OF RODS; WOH = WEIGHT OF HAMMER.

V1A = 755/155 PSF — DENOTES IN-SITU VANE SHEAR TEST PERFORMED AT DEPTH SHOWN WITH CORRECTED PEAK/RESIDUAL SHEAR STRENGTHS PROVIDED.

REC=95% / REC=60%; REC=95% / REC=58% — PERCENT RECOVERY / PERCENT ROCK QUALITY DESIGNATION (RQD)

BOE AT 135.0 FT BGS — DENOTES BOTTOM OF EXPLORATION AT APPROXIMATE DEPTH SHOWN

GROUNDWATER LEVEL MEASURED IN COMPLETED OBSERVATION WELL ON DATE SPECIFIED

(OW) — DENOTES OBSERVATION WELL INSTALLED IN COMPLETED BOREHOLE

Q_{50} EL. 28.7 — FLOOD ELEVATION FOR DESIGN YEAR STORM EVENT SHOWN, DETERMINED BY TY LIN INTERNATIONAL

NOTES:

1. SEE SHEET 2 FOR THE LOCATION AND ORIENTATION OF THE SUBSURFACE PROFILE (PROPOSED CONSTRUCTION CENTERLINE).
2. THE LOCATION AND ORIENTATION OF PROPOSED BRIDGE / ROADWAY GRADES ALONG THE PROPOSED CONSTRUCTION CENTERLINE ARE TAKEN FROM THE ELECTRONIC MICROSTATION FILE ENTITLED, "PROFILE.DGN", PROVIDED TO HALEY & ALDRICH, INC. BY TY LIN INTERNATIONAL ON 12 MAY 2009.
3. GROUND SURFACE ELEVATIONS AT "AS-DRILLED" TEST BORING LOCATIONS WERE DETERMINED BY THE MAINE DEPARTMENT OF TRANSPORTATION USING GPS SURVEY EQUIPMENT.
4. "AS-DRILLED" LOCATIONS OF THE TEST BORINGS WERE DETERMINED IN THE FIELD BY THE MAINE DEPARTMENT OF TRANSPORTATION USING GPS SURVEY EQUIPMENT.
5. LINES REPRESENTING CHANGES IN STRATA SHOWN ON THE PROFILE ARE BASED ON LINEAR INTERPOLATION BETWEEN SUBSURFACE EXPLORATIONS AND DEPICT ANTICIPATED SUBSURFACE CONDITIONS ALONG THE CONSTRUCTION CENTERLINE. THE ACTUAL CONDITIONS WILL DIFFER FROM THOSE SHOWN.
6. ELEVATIONS ARE IN FEET AND REFERENCE THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD 1988).

HORIZ. 0 60 120 180 240
VERT. 0 15 30 45 60
SCALE IN FEET

HALEY & ALDRICH

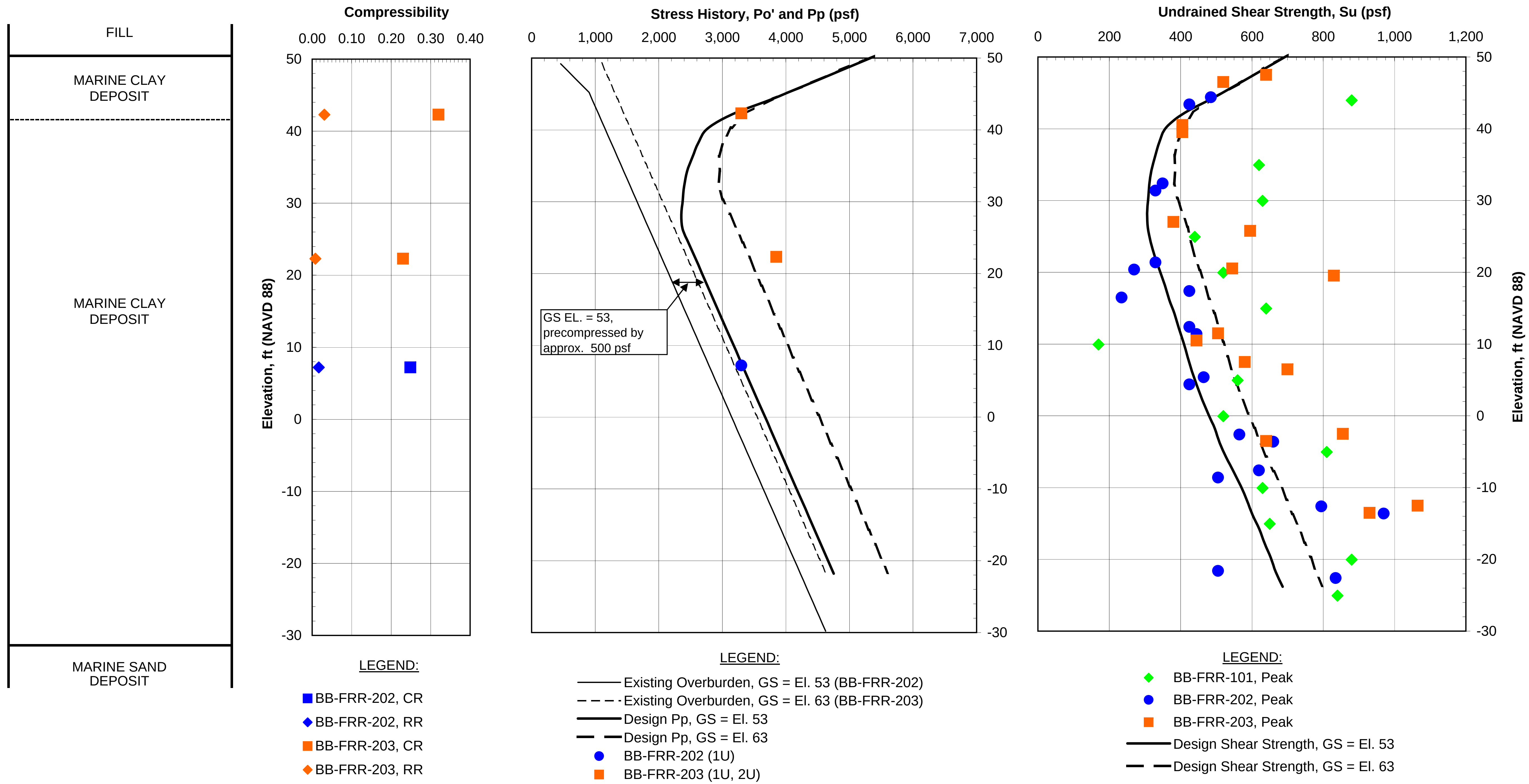
PROPOSED REPLACEMENT BRIDGE
OVER PRESUMPSCOT RIVER AND MCRR
ROUTES 26/100 - FALMOUTH, MAINE
MAINEDOT PIN 15094.00

GEOLOGIC PROFILE A-A

SCALE: AS SHOWN
NOVEMBER 2009

SHEET 3

G:\35524\GLOBAL\CAD\35524-000_004D_PROFILE.DWG



NOTES:

1. Typical soil profile and in-situ field vane test results from test borings BB-FRR-101, BB-FRR-202, BB-FRR-203; see Tables III and IV for tabulated results and additional details of vane shear testing.
2. Compressibility data from laboratory constant rate of strain consolidation tests performed on specimens of marine clay trimmed from Shelby tube samples obtained from test borings BB-FRR-202 and BB-FRR-203.
3. Design shear strength profiles developed by establishing best-fit curves through the BB-FRR-101 and BB-FRR-202 correlated shear strengths for ground surface El. 53 and through the BB-FRR-203 correlated shear strengths for ground surface El. 63, as shown.
4. Design preconsolidation pressure profiles established using an assumed ratio of undrained shear strength (Su) over preconsolidation pressure (Pp) equal to 0.13.
5. Design Su/Pp ratio was developed by comparison of in-situ vane shear test results to corresponding consolidation test results at similar depths.
6. RR = Recompression ratio; CR = Virgin compression ratio; Po' = Existing Effective Overburden Pressure.

HALEY & ALDRICH

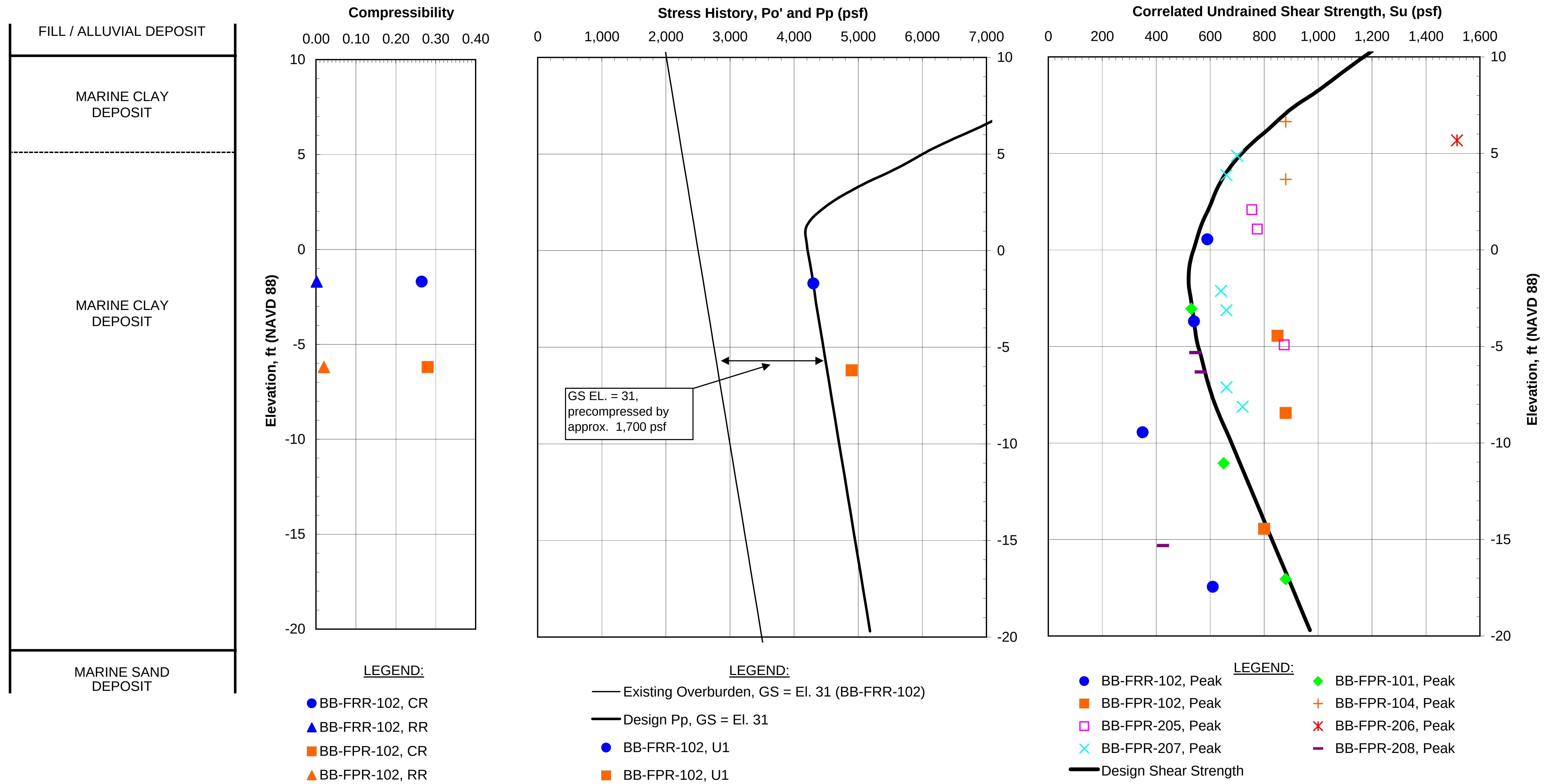
REPLACEMENT BRIDGE OVER
PRESUMPSCOT RIVER AND MCRR
ROUTES 100/26 - FALMOUTH, MAINE

COMPRESSIBILITY AND
SHEAR STRENGTH DATA
(NORTH APPROACH)

NOT TO SCALE
NOVEMBER 2009

SHEET 4

G:\PROJECTS\35524 - Presumpscot River Bridge\010\Approach Embankments\[2009_1113_HAI_North Approach Compressibility Figure_f.xls]Fig 4 - North app



NOTES:

1. Typical soil profile and in-situ field vane test results from test borings BB-FRR-102, BB-FPR-101, BB-FPR-102, BB-FPR-104, BB-FPR-205, BB-FPR-206, BB-FPR-207 and BB-FPR-208; see Tables III and IV for tabulated results and additional details of vane shear testing.
2. Compressibility data from laboratory constant rate of strain consolidation tests performed on specimens of marine clay trimmed from Shelby tube samples obtained from test borings BB-FRR-102 and BB-FPR-102.
3. Design shear strength profiles developed by establishing best-fit curves through the correlated shear strengths for the borings shown above.
4. Design preconsolidation pressure profiles established using an assumed ratio of undrained shear strength (Su) over preconsolidation pressure (Pp) equal to 0.12.
5. Design Su/Pp ratio was developed by comparison of in-situ vane shear test results to corresponding consolidation test results at similar depths.
6. RR = Recompression ratio; CR = Virgin compression ratio; Po' = Existing Effective Overburden Pressure.

HALEY & ALDRICH

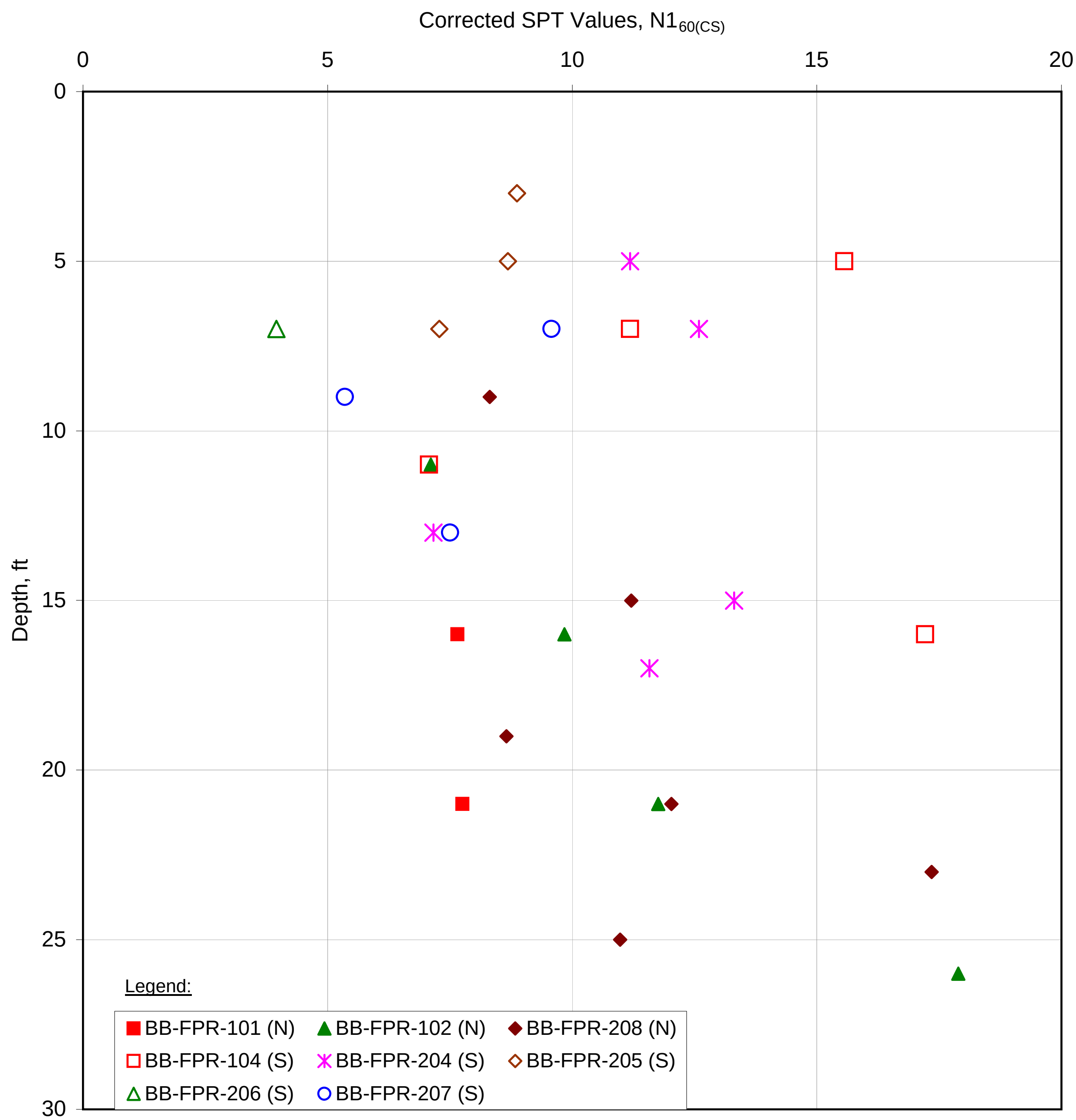
REPLACEMENT BRIDGE OVER
PRESUMPSCOT RIVER AND MCRR
ROUTES 100/26 - FALMOUTH, MAINE

COMPRESSIBILITY AND
SHEAR STRENGTH DATA
(SOUTH APPROACH)

NOT TO SCALE
NOVEMBER 2009

SHEET 5

G:\PROJECTS\35524 - Presumpscot River Bridge\010\Approach Embankments\[2009_1113_HAI_South Approach Compressibility Figure_f2.xls]Fig 5 - South Abutment



Notes:

1 - $N1_{60(CS)}$ = SPT N-value corrected for overburden, drilling and sampling methods, and fines content for use in evaluating liquefaction resistance.

2 - All borings were drilled as cased borings with rope/cathead and safety hammer.

3 - $N1_{60(CS)}$ = [Nm (field value) x C_E x C_B x C_R x C_S x C_N] +$\Delta N1_{60(CS)}$, where C_E = energy ratio correction, C_B = borehole diameter correction, C_R = rod length correction, C_S = sampler correction, C_N = overburden correction factor, and $\Delta N1_{60(CS)}$ = correction for fines content.

4 - Blow counts were corrected in accordance with Idriss and Boulanger (2008) and Youd et al (2001).

5 - Samples with greater than 50 percent passing the No. 200 sieve (i.e., silts and clays) are not shown.

6 - (N) after boring designation in legend indicates north of river boring, (S) indicates south of river boring.

HALEY & ALDRICH

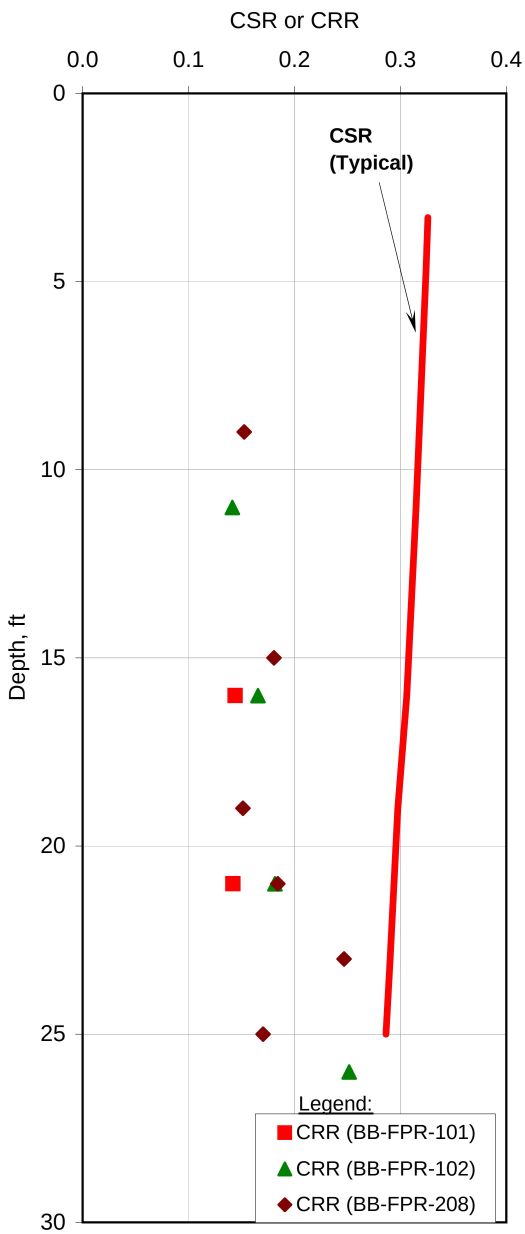
REPLACEMENT BRIDGE OVER
PRESUMPSCOT RIVER AND MCRR
ROUTES 100/26 - FALMOUTH, MAINE

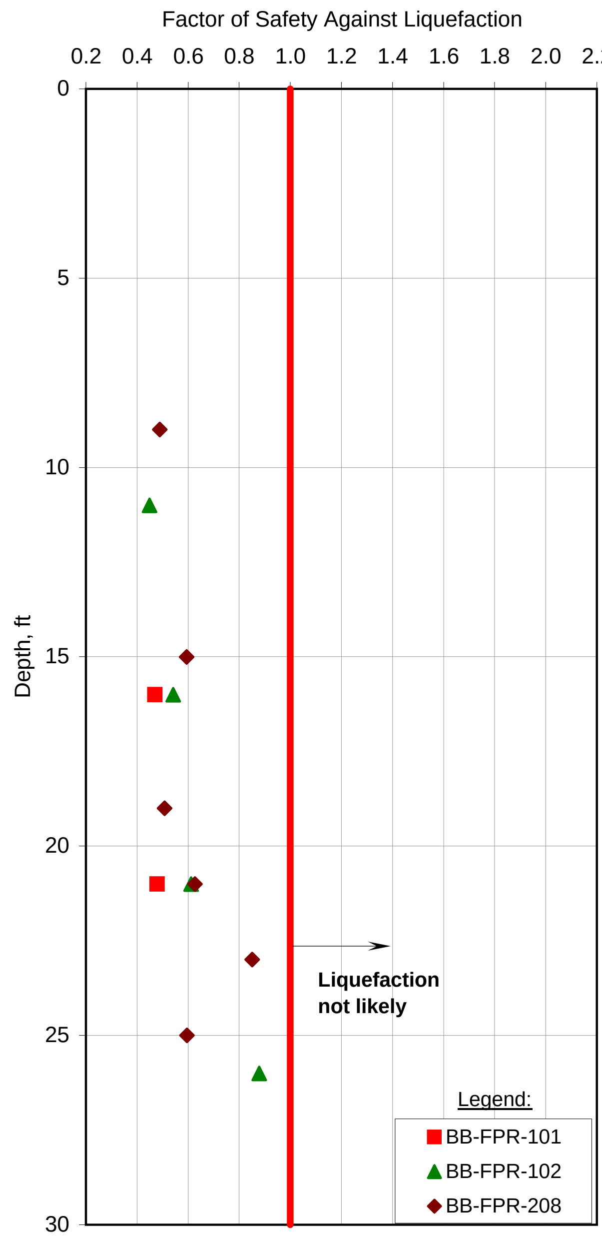
CORRECTED STANDARD PENETRATION TEST VALUES FOR LIQUEFACTION ASSESSMENT, ALLUVIAL DEPOSIT

NOT TO SCALE
NOVEMBER 2009

SHEET 6

G:\PROJECTS\35524 - Presumpscot River Bridge\010\Liquefaction Evaluation\[2009_1112_HAI_PrelimLiquefaction AASHTO2008_f2.xls]Blowcount Chart





Notes:

1 - Cyclic Stress Ratio (CSR) = $CSR_{M=M,s'vc=1\ atm}$, corresponds to cyclic shear stress induced by design earthquake.

2 - Cyclic Resistance Ratio (CRR) = $CRR_{M=M,s'vc=1\ atm}$ (corrected for magnitude and overburden), corresponds to resistance of soil layer to cyclic shear stress (based on Standard PenetrationTest results and fines content).

3 - Factor of safety against liquefaction triggering = CRR / CSR.

4 - Considered earthquake magnitude 6.5 with PGA = 0.218 g (assumes AASHTO 2007, Site Class E, Design Spectrum).

5 - Initial liquefaction analyses used simplified empirical procedures by Idriss and Boulanger (2008).

6 - Some data points not within the range of values shown are not displayed on charts.

HALEY & ALDRICH

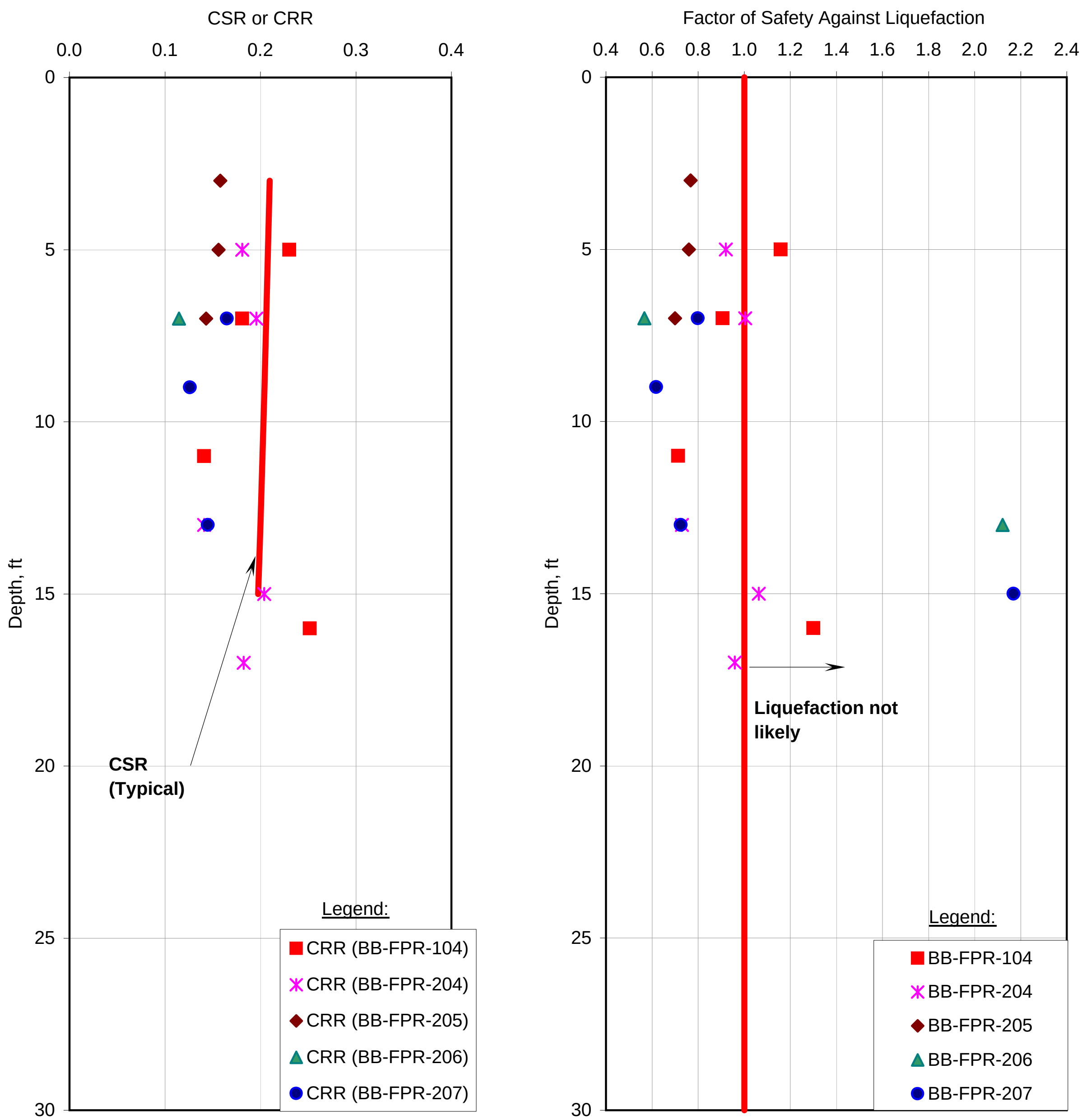
REPLACEMENT BRIDGE OVER
PRESUMPSCOT RIVER AND MCRR
ROUTES 100/26 - FALMOUTH, MAINE

INITIAL LIQUEFACTION
ASSESSMENT, NORTH OF RIVER
(AASHTO 2007, Site Class E)

NOT TO SCALE
NOVEMBER 2009

SHEET 7

G:\PROJECTS\35524 - Presumpscot River Bridge\010\Liquefaction Evaluation\[2009_1112_HAI_PrelimLiquefaction AASHTO2008_f2.xls]North of River Chart



Notes:

1 - Cyclic Stress Ratio (CSR) = $CSR_{M=M,s'vc=1\ atm}$, corresponds to cyclic shear stress induced by design earthquake.

2 - Cyclic Resistance Ratio (CRR) = $CRR_{M=M,s'vc=1\ atm}$ (corrected for magnitude and overburden), corresponds to resistance of soil layer to cyclic shear stress (based on Standard PenetrationTest results and fines content).

3 - Factor of safety against liquefaction triggering = CRR / CSR.

4 - Considered earthquake magnitude 6.5 with PGA = 0.140 g (assumes AASHTO 2007, Site Class D, Design Spectrum).

5 - Initial liquefaction analyses used simplified empirical procedures by Idriss and Boulanger (2008).

6 - Some data points not within the range of values shown are not displayed on charts.

HALEY & ALDRICH

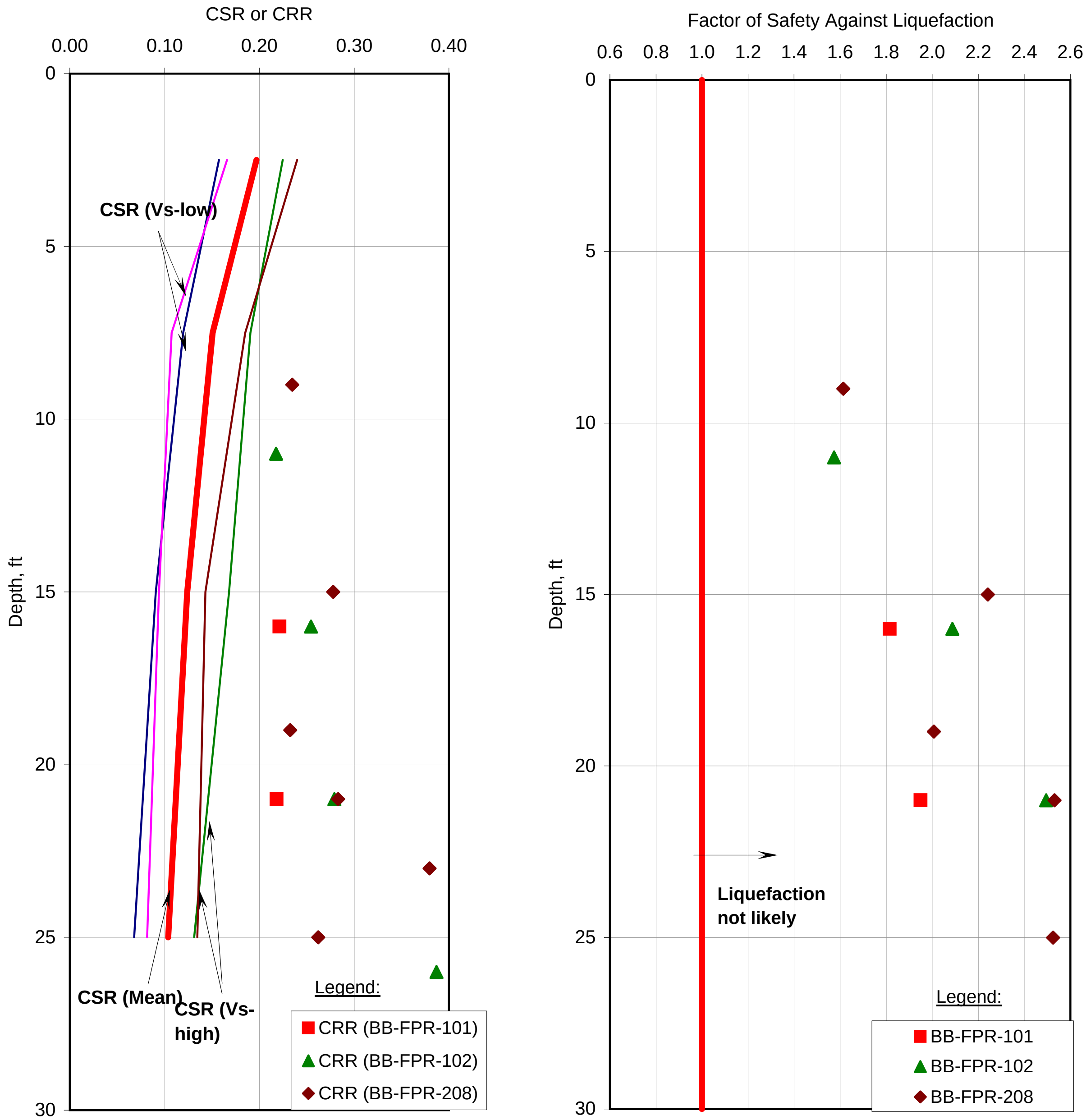
REPLACEMENT BRIDGE OVER
PRESUMPSCOT RIVER AND MCRR
ROUTES 100/26 - FALMOUTH, MAINE

INITIAL LIQUEFACTION
ASSESSMENT, SOUTH OF RIVER
(AASHTO 2007, Site Class D)

NOT TO SCALE
NOVEMBER 2009

SHEET 8

G:\PROJECTS\35524 - Presumpscot River Bridge\010\Liquefaction Evaluation\[2009_1112_HAI_PrelimLiquefaction AASHTO2008_f2.xls]South of River Chart



Notes:

1 - Cyclic Stress Ratio (CSR) = $CSR_{M=M,s'vc=1\ atm}$, corresponds to cyclic shear stress calculated using site-specific response analysis (SSRA).

2 - Cyclic Resistance Ratio (CRR) = $CRR_{M=M,s'vc=1\ atm}$ (corrected for magnitude and overburden), corresponds to resistance of soil layer to cyclic shear stress (based on Standard Penetration Test results and fines content).

3 - Factor of safety against liquefaction triggering = CRR / CSR (mean).

4 - Considered earthquake magnitude 6.0 based on deaggregation of seismic hazard for 7% in 75-year earthquake recurrence period.

5 - Liquefaction analyses used shear stress output from SSRA (Proshake) using lower and upper bound shear wave velocity profiles.

6 - Some data points not within the range of values shown are not displayed on charts.

HALEY & ALDRICH

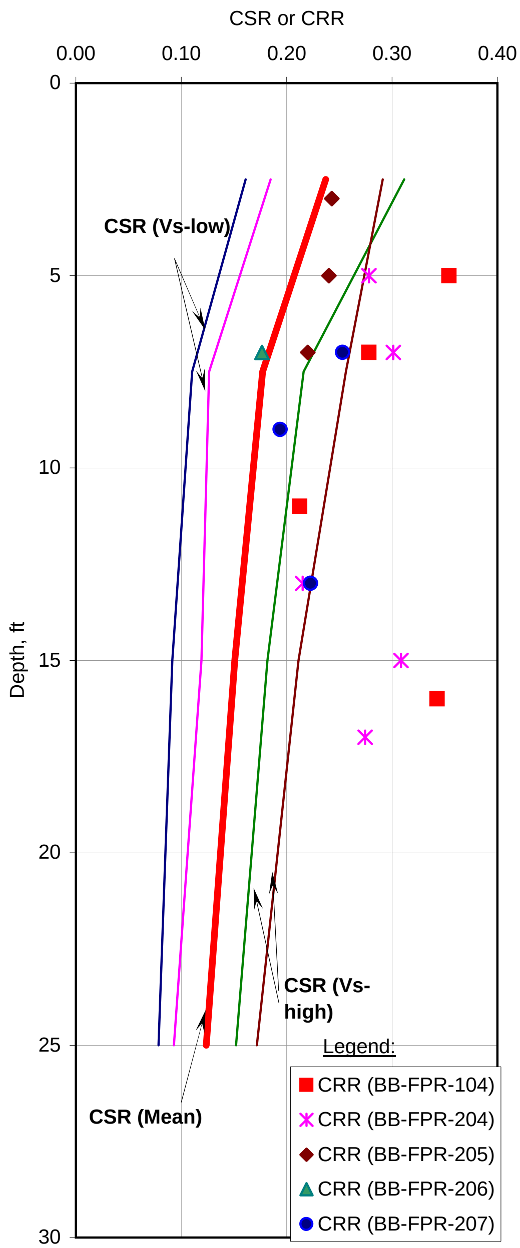
REPLACEMENT BRIDGE OVER
PRESUMPSCOT RIVER AND MCRR
ROUTES 100/26 - FALMOUTH, MAINE

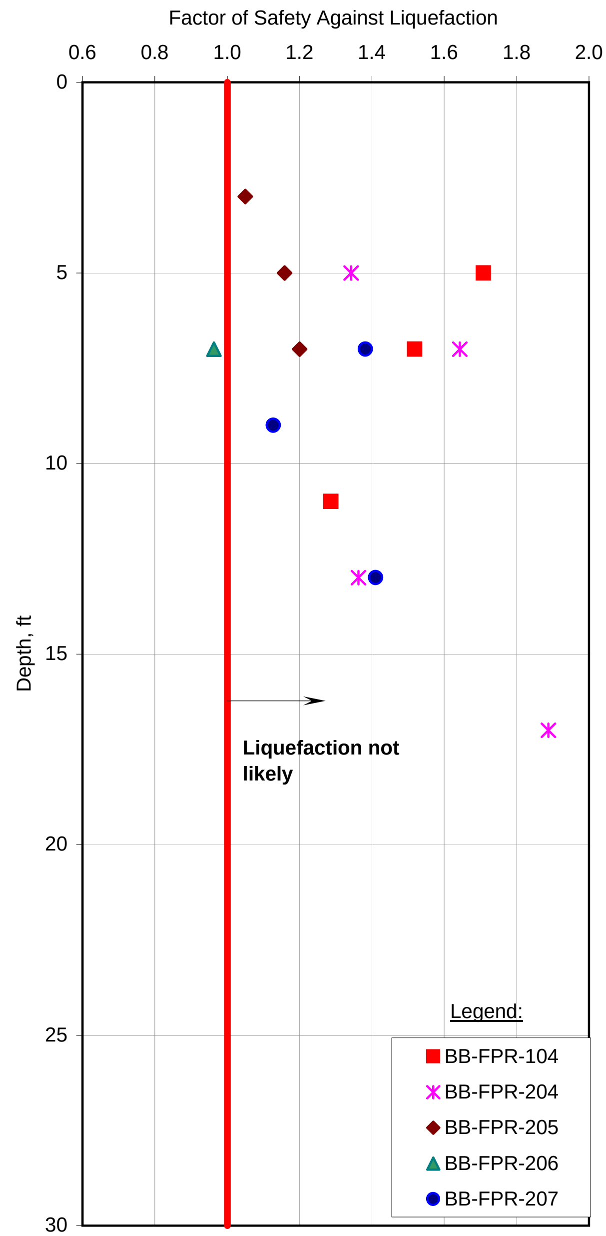
SITE-SPECIFIC LIQUEFACTION ASSESSMENT, NORTH OF RIVER

NOT TO SCALE
NOVEMBER 2009

SHEET 9

G:\PROJECTS\35524 - Presumpscot River Bridge\010\Liquefaction Evaluation\Site Specific\[2009-1112-MKY-Site Specific LIQ_revARB-f2.xls]North of River Chart





Notes:

1 - Cyclic Stress Ratio (CSR) = CSR$_{M=M,s'vc=1\ atm}$, corresponds to cyclic shear stress calculated using site-specific response analysis (SSRA).

2 - Cyclic Resistance Ratio (CRR) = CRR$_{M=M,s'vc=1\ atm}$ (corrected for magnitude and overburden), corresponds to resistance of soil layer to cyclic shear stress (based on Standard Penetration Test results and fines content).

3 - Factor of safety against liquefaction triggering = CRR / CSR (mean).

4 - Considered earthquake magnitude 6.0 based on deaggregation of seismic hazard for 7% in 75-year earthquake recurrence period.

5 - Liquefaction analyses used shear stress output from SSRA (Proshake) using lower and upper bound shear wave velocity profiles.

6 - Some data points not within the range of values shown are not displayed on charts.

HALEY & ALDRICH

REPLACEMENT BRIDGE OVER
PRESUMPSCOT RIVER AND MCRR
ROUTES 100/26 - FALMOUTH, MAINE

SITE-SPECIFIC LIQUEFACTION ASSESSMENT, SOUTH OF RIVER

NOT TO SCALE
NOVEMBER 2009

SHEET 10

G:\PROJECTS\35524 - Presumpscot River Bridge\010\Liquefaction Evaluation\Site Specific\[2009-1112-MKY-Site Specific LIQ_revARB-f2.xls]South of River Chart

TABLE I
Preliminary Phase Explorations
Replacement Bridge over Presumpscot River and Maine Central Railroad
Routes 26/100 - Falmouth, Maine

MaineDOT Pin: 15094.00
Haley & Aldrich File No.: 35524-000

LOCATION DATA:

Test Boring No.[1]	Approximate Station (STA, ft)	Approximate Offset Distance (ft)	Coordinates[2]	
			Northing	Easting
BB-FRR-101	118 + 67	17' W	326,956	1,005,192
BB-FRR-102	116 + 73	41' W	326,767	1,005,237
BB-FPR-101	115 + 64	48' W	326,663	1,005,264
BB-FPR-102	114 + 39	59' W	326,555	1,005,297
BB-FPR-103	112 + 86	44' W	326,399	1,005,333
BB-FPR-104	111 + 37	31' W	326,256	1,005,371

SUBSURFACE DATA:

Test Boring No.[1]	Ground Surface Elevation[3,4]	Approximate Strata Thickness[5] (ft)						Approximate Elevation of Top of Weathered Bedrock	Approximate Elevation of Top of Competent Bedrock	Approximate Elevation of Bottom of Exploration
		Topsoil	Fill	Alluvial Deposit	Marine Clay	Marine Sand	Glacial Till			
BB-FRR-101	50.7	NE	4.0	NE	79.0	57.9	27.9	-118.1	-119.4	-129.2
BB-FRR-102	31.3	NE	5.0	NE	53.5	46.5	23.5	-97.2	-98.7	-110.0
BB-FPR-101	24.7	2.3	NE	20.7	17.5	51.8	32.8	-100.4	NE	-100.4
BB-FPR-102	26.3	2.7	NE	24.8	17.7	47.3	37.5	-103.7	-103.7	-115.2
BB-FPR-103	0.6	NE	NE	4.4	13.2	32.9	34.5	-84.4	-85.9	-96.1
BB-FPR-104	29.4	NE	4.0	15.0	9.0	NE	99	-97.6	-99.6	-109.6

Notes:
1 Test boring locations are shown on Figure 2, Site and Subsurface Exploration Location Plan.
2 As-drilled coordinates of test borings were determined by MaineDOT using GPS survey equipment and are provided in NAD83, Maine 2000 West Zone coordinate system.
3 Ground surface elevations at test boring locations were determined in the field by MaineDOT using GPS survey equipment.
4 Elevations are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).
5 "NE" indicates stratum was not encountered in test boring.

TABLE II
Design Phase Explorations
Replacement Bridge over Presumpscot River and Maine Central Railroad
Routes 26/100 - Falmouth, Maine

MaineDOT Pin: 15094.00
Haley & Aldrich File No.: 35524-010

LOCATION DATA:

Test Boring No.[1]	Approximate Station (STA, ft)	Approximate Perpendicular Offset Distance (ft)	Coordinates[2]	
			Northing	Easting
BB-FRR-201	116 + 81	20' E	326,795	1,005,290
BB-FRR-202	118 + 97	11' E	326,994	1,005,206
BB-FRR-203	120 + 00	23' E	327,094	1,005,180
BB-FRR-204	120 + 54	33' E	327,147	1,005,173
BB-FPR-201	108 + 97	4' W	326,022	1,005,421
BB-FPR-202	109 + 97	39' E	326,125	1,005,456
BB-FPR-203	110 + 35	82' E	326,168	1,005,495
BB-FPR-204	111 + 20	72' W	326,234	1,005,332
BB-FPR-205	111 + 57	15' E	326,283	1,005,412
BB-FPR-206	111 + 58	71' E	326,292	1,005,467
BB-FPR-207	111 + 83	3' W	326,305	1,005,390
BB-FPR-208	115 + 21	18' E	326,640	1,005,336

SUBSURFACE DATA:

Test Boring No.[1]	Ground Surface Elevation[3,4]	Approximate Strata Thickness[5] (ft)							Approximate Elevation of Top of Weathered Bedrock	Approximate Elevation of Top of Competent Bedrock	Approximate Elevation of Bottom of Exploration
		Bituminous Concrete / Topsoil	Fill	Alluvial Deposit	Marine Deposit			Glacial Till			
					Interbedded	Clay	Sand				
BB-FRR-201	24.3	1.0	NE	NE	NE	51.8	36.2	35.5	NE	-100.2	-109.7
BB-FRR-202	53.2	NE	1.8	NE	NE	81.2	63.0	24.4	NE	-117.2	-125.3
BB-FRR-203	63.3	NE	1.4	NE	9.1	74.0	>7.5	NE	NE	NE	-28.7
BB-FRR-204	64.4	0.3	2.0	NE	7.7	75.0	>7.0	NE	NE	NE	-27.6
BB-FPR-201	51.0	0.4	4.1	NE	19.6	NE	NE	>2.4	NE	NE	24.5
BB-FPR-202	32.9	NE	4.0	9.0	NE	NE	NE	>3.0	NE	NE	16.9
BB-FPR-203	31.2	2.0	NE	13.2	NE	NE	NE	>4.8	NE	NE	11.2
BB-FPR-204	28.1	NE	8.0	10.0	NE	19.0	NE	>4.0	NE	NE	-12.9
BB-FPR-205	27.9	1.0	NE	16.0	NE	16.5	3.0	90.0	NE	-98.6	-107.1
BB-FPR-206	26.4	2.0	NE	12.5	NE	10.5	>12.0	NE	NE	NE	-10.6
BB-FPR-207	23.7	0.5	NE	16.0	NE	24.0	NE	>1.5	NE	NE	-18.3
BB-FPR-208	25.4	2.5	NE	23.5	NE	20.3	33.7	42.5	-97.1	-99.6	-113.6

Notes:
[1] Test boring locations are shown on Figure 2, Site and Subsurface Exploration Location Plan.
[2] As-drilled coordinates of test borings were determined by MaineDOT using GPS survey equipment and are provided in NAD83, Maine 2000 West Zone coordinate system.
[3] Ground surface elevations at test boring locations were determined in the field by MaineDOT using GPS survey equipment.
[4] Elevations are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).
[5] "NE" indicates stratum was not encountered in test boring.

TABLE III
Preliminary Phase In-Situ Vane Shear Test Results
Replacement Bridge over Presumpscot River and Maine Central Railroad
Routes 26/100 - Falmouth, Maine

MaineDOT Pin: 15094.00
Haley & Aldrich File No.: 35524-000

Test Boring No.[1]	Estimated Ground Surface Elevation[2,3]	Vane Size (in. x in.)	Test No.	Depth below ground surface (ft)	Approx. Elevation[3] (ft)	V_{max} [4] (in-lbs)	$V_{remolded}$ [4] (in-lbs)	S_u [5] (psf)	$S_{u(remolded)}$ [5] (psf)
BB-FRR-101	50.7	3 in. X 6 in.	V_1	6.5 - 7.0	44.2 - 43.7	>600	-	>880	-
			V_2	15.5 - 16.0	35.2 - 34.7	419	85	620	130
			V_3	20.5 - 21.0	30.2 - 29.7	430	80	630	120
			V_4	25.5 - 26.0	25.2 - 24.7	300	60	440	90
			V_5	30.5 - 31.0	20.2 - 19.7	350	90	520	130
			V_6	35.5 - 36.0	15.2 - 14.7	435	70	640	100
			V_7	40.5 - 41.0	10.2 - 9.7	115	75	170	110
			V_8	45.5 - 46.0	5.2 - 4.7	380	30	560	40
			V_9	50.5 - 51.0	0.2 - -0.3	350	35	520	50
			V_{10}	55.5 - 56.0	-4.8 - -5.3	550	25	810	40
			V_{11}	60.5 - 61.0	-9.8 - -10.3	430	25	630	40
			V_{12}	65.5 - 66.0	-14.8 - -15.3	445	20	650	30
			V_{13}	70.5 - 71.0	-19.8 - -20.3	>600	30	>880	40
			V_{14}	75.5 - 76.0	-24.8 - -25.3	570	50	840	70
BB-FRR-102	31.3	3 in. X 6 in.	V_1	20.5 - 21.0	10.8 - 10.3	>600	120	>880	180
			V_2	30.5 - 31.0	0.8 - 0.3	400	100	590	150
			V_3	34.5 - 35.5	-3.2 - -4.2	365	70	540	100
			V_4	40.5 - 41.0	-9.2 - -9.7	235	75	350	110
			V_5	48.5 - 49.0	-17.2 - -17.7	415	40	610	60
			V_6	56.5 - 57.0	-25.2 - -25.7	590	80	870	120
BB-FPR-101	24.7	3 in. X 6 in.	V_1	27.5 - 28.0	-2.8 - -3.3	360	90	530	130
			V_2	35.5 - 36.0	-10.8 - -11.3	440	120	650	180
			V_3	41.5 - 42.0	-16.8 - -17.3	>600	-	>880	-
BB-FPR-102	26.3	3 in. X 6 in.	V_1	30.5 - 31.0	-4.2 - -4.7	580	150	850	220
			V_2	34.5 - 35.0	-8.2 - -8.7	>600	150	>880	220
			V_3	40.5 - 41.0	-14.2 - -14.7	540	60	800	90
BB-FPR-103	0.6	3 in. X 6 in.	V_1	8.5 - 9.0	-7.9 - -8.4	510	250	750	370
			V_2	12.5 - 13.0	-11.9 - -12.4	580	175	850	260
BB-FPR-104	29.4	3 in. X 6 in.	V_1	22.5 - 23.0	6.9 - 6.4	>600	-	880	-
			V_2	25.5 - 26.0	3.9 - 3.4	>600	-	880	-

Notes:
1 Test boring locations are shown on Figure 2, Site and Subsurface Exploration Location Plan.
2 Ground surface elevations at test boring locations were determined in the field by MaineDOT using optical surveying methods.
3 Elevations are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).
4 Vane shear measurements are shown on the test boring reports presented in Appendix A.
5 V_{max} and $V_{remolded}$ represent direct peak and remolded vane torque values, respectively.
6 S_u and $S_{u(remolded)}$ represent corrected undrained peak and residual undrained shear strengths, respectively, rounded to the nearest 10 psf.
7 in-lbs = inch-pounds of torque, psf = pounds per square foot.

TABLE IV
Design Phase In-Situ Vane Shear Test Results
Replacement Bridge over Presumpscot River and Maine Central Railroad
Routes 26/100 - Falmouth, Maine

MaineDOT Pin: 15094.00
Haley & Aldrich File No.: 35524-010

Test Boring No.[1]	Estimated Ground Surface Elevation[2,3]	Vane Size (mm x mm)	Test No.	Depth below ground surface (ft)	Approx. Elevation[3]	V_{max} [4] (in-lbs)	$V_{remolded}$ [4] (in-lbs)	S_u [5] (psf)	$S_{u(remolded)}$ [5] (psf)
BB-FRR-201	24.3	55x110	V1A	15.6 - 16.0	8.7 - 8.3	110	5	425	20
			V1B	16.6 - 17.0	7.7 - 7.3	115	10	445	40
			V2A	25.6 - 26.0	-1.3 - -1.7	95	5	370	20
			V2B	26.6 - 27.0	-2.3 - -2.7	65	0	250	0
			V3A	35.6 - 36.0	-11.3 - -11.7	120	5	465	20
			V3B	36.6 - 37.0	-12.3 - -12.7	80	0	310	0
			V4A	45.6 - 46.0	-21.3 - -21.7	80	0	310	0
BB-FRR-202	53.2	55x110	V1A	8.6 - 9.0	44.6 - 44.2	125	20	485	80
			V1B	9.6 - 10.0	43.6 - 43.2	110	15	425	60
			V2A	20.6 - 21.0	32.6 - 32.2	90	5	350	20
			V2B	21.6 - 22.0	31.6 - 31.2	85	5	330	20
			V3A	31.6 - 32.0	21.6 - 21.2	85	5	330	20
			V3B	32.6 - 33.0	20.6 - 20.2	70	5	270	20
			V4A	35.6 - 36.0	17.6 - 17.2	110	0	425	0
			V4B	36.5 - 36.9	16.7 - 16.3	60	5	235	20
			V5A	40.6 - 41.0	12.6 - 12.2	110	5	425	20
			V5B	41.6 - 42.0	11.6 - 11.2	115	0	445	0
			V6A	47.6 - 48.0	5.6 - 5.2	120	0	465	0
			V6B	48.6 - 49.0	4.6 - 4.2	110	5	425	20
			V7A	55.6 - 56.0	-2.4 - -2.8	145	5	565	20
			V7B	56.6 - 57.0	-3.4 - -3.8	170	5	660	20
			V8A	60.6 - 61.0	-7.4 - -7.8	160	25	620	95
			V8B	61.6 - 62.0	-8.4 - -8.8	130	25	505	95
			V9A	65.6 - 66.0	-12.4 - -12.8	205	25	795	95
			V9B	66.6 - 67.0	-13.4 - -13.8	250	5	970	20
			V10A	74.6 - 75.0	-21.4 - -21.8	130	10	505	40
			V10B	75.6 - 76.0	-22.4 - -22.8	215	25	835	95
BB-FRR-203	63.3	65x130	V1A	15.6 - 16.0	47.7 - 47.3	270	50	640	120
			V1B	16.6 - 17.0	46.7 - 46.3	220	30	520	70
			V2A	22.6 - 23.0	40.7 - 40.3	170	10	405	25
			V2B	23.6 - 24.0	39.7 - 39.3	170	10	405	25
			V3A	36.1 - 36.5	27.2 - 26.8	160	10	380	25
			V3B	37.1 - 37.5	26.2 - 25.8	250	10	595	25
			V4A	42.6 - 43.0	20.7 - 20.3	230	10	545	25
			V4B	43.6 - 44.0	19.7 - 19.3	350	10	830	25
		55x110	V5A	51.6 - 52.0	11.7 - 11.3	130	10	505	40
			V5B	52.6 - 53.0	10.7 - 10.3	115	15	445	60
			V6A	55.6 - 56.0	7.7 - 7.3	150	5	580	20
			V6B	56.6 - 57.0	6.7 - 6.3	180	5	700	20
			V7A	65.6 - 66.0	-2.3 - -2.7	220	5	855	20
			V7B	66.6 - 67.0	-3.3 - -3.7	165	5	640	20
			V8A	75.6 - 76.0	-12.3 - -12.7	275	5	1,065	20
			V8B	76.6 - 77.0	-13.3 - -13.7	240	10	930	40
BB-FPR-205	27.9	55x110	V1A	25.6 - 26.0	2.3 - 1.9	195	40	755	155
			V1B	26.6 - 27.0	1.3 - 0.9	200	40	775	155
			V2A	32.6 - 33.0	-4.7 - -5.1	225	50	875	195
BB-FPR-206	26.4	55x110	V1A	10.5 - 10.9	15.9 - 15.5	160	45	620	175
			V2A	20.5 - 20.9	5.9 - 5.5	390	30	1,515	115
BB-FPR-207	23.7	55x110	V1A	18.6 - 19.0	5.1 - 4.7	180	30	700	115
			V1B	19.6 - 20.0	4.1 - 3.7	170	35	660	135
			V2A	25.6 - 26.0	-1.9 - -2.3	165	20	640	80
			V2B	26.6 - 27.0	-2.9 - -3.3	170	70	660	270
			V3A	30.6 - 31.0	-6.9 - -7.3	170	90	660	350
			V3B	31.6 - 32.0	-7.9 - -8.3	185	40	720	155
BB-FPR-208	25.4	55x110	V1A	30.5 - 30.9	-5.1 - -5.5	140	10	545	40
			V1B	31.5 - 31.9	-6.1 - -6.5	145	5	565	20
			V2A	40.5 - 40.9	-15.1 - -15.5	110	5	425	20

Notes:

1 Test boring locations are shown on Figure 2, Site and Subsurface Exploration Location Plan.

2 Ground surface elevations at test boring locations were determined in the field by MaineDOT using GPS survey equipment.

3 Elevations are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).

4 Vane shear measurements are shown on the test boring reports presented in Appendix A.

5 V_{max} and $V_{remolded}$ represent direct peak and remolded vane torque values, respectively.

6 S_u and $S_{u(remolded)}$ represent corrected undrained peak and residual undrained shear strengths, respectively, rounded to the nearest 10 psf.

7 in-lbs = inch-pounds of torque, psf = pounds per square foot.

8 Torque was measured in foot-pounds for test borings BB-FRR-201, BB-FRR-202, and BB-FPR-208; measured values have been multiplied by 12 to convert to in-lbs as shown above.

9 "A" and "B" designations indicate vanes conducted concurrently with borehole at same bottom elevation.

TABLE V
Pier 1 Pile Cap Loads
Replacement Bridge over Presumpscot River and Maine Central Railroad
Routes 26/100 - Falmouth, Maine

MaineDOT Pin: 15094.00
Haley & Aldrich File No.: 35524-010

Load Case Description	Py (kip)	Vx (kip)	Vz (kip)	Mx (kip-ft)	Mz (kip-ft)
Strength Loading[1,4]:					
Force Concurrent with Max Py	5,824	26	26	2,283	-1,839
Force Concurrent with abs(Max) Vx[3]	4,799	211	46	1,155	-1,324
Force Concurrent with abs(Max) Vz	4,239	109	115	2,360	-1,121
Force Concurrent with abs(Max) Mx[3]	4,184	83	112	7,331	-4,403
Force Concurrent with abs(Max) Mz	4,530	-169	-75	-1,451	1,559
Service Loading[1,4]:					
Force Concurrent with Max Py	4,668	18	27	709	-41
Force Concurrent with abs(Max) Vx	2,774	149	40	959	-939
Force Concurrent with abs(Max) Vz	3,532	-50	-65	-1,319	665
Force Concurrent with abs(Max) Mx	4,335	40	61	1,418	-329
Force Concurrent with abs(Max) Mz	3,671	-119	-35	-606	1,062
Seismic Event Loading[1,2,4] (R=1.0):					
Force Concurrent with Max Py	4,668	-277	314	18,872	9,018
Force Concurrent with abs(Max) Vx	2,774	-357	313	18,778	10,443
Force Concurrent with abs(Max) Vz	3,532	315	315	18,996	-9,346
Force Concurrent with abs(Max) Mx	4,335	315	315	18,996	-9,346
Force Concurrent with abs(Max) Mz	3,671	-351	311	18,599	10,457

Notes:
[1] - Loads provided by TY Lin International to Haley & Aldrich on 7 July 2009.
[2] - Seismic loads were extrapolated by TY Lin International from Pier 3 evaluations also conducted by TY Lin International.
[3] - Loads revised by TY Lin International and provided to Haley & Aldrich on 22 September 2009.
[4] - Loads shown are in TY Lin's coordinate system and were converted to FB MultiPier coordinate system by Haley & Aldrich.

TABLE VI
Pier 2 Pile Cap Loads
Replacement Bridge over Presumpscot River and Maine Central Railroad
Routes 26/100 - Falmouth, Maine

MaineDOT Pin: 15094.00
Haley & Aldrich File No.: 35524-010

Load Case Description	Py (kip)	Vx (kip)	Vz (kip)	Mx (kip-ft)	Mz (kip-ft)
Strength Loading[1,4]:					
Force Concurrent with Max Py	2,570	55	-48	-2,569	-1,336
Force Concurrent with abs(Max) Vx	1,884	-124	48	2,206	2,168
Force Concurrent with abs(Max) Vz	1,564	-66	83	3,333	1,146
Force Concurrent with abs(Max) Mx	2,367	96	-71	-3,410	-1,888
Force Concurrent with abs(Max) Mz	1,206	111	-41	-1,447	-2,331
Column Design Forces (At Base with MM[3])	2,301	89	-71	-4,438	-1,955
Service Loading[1,4]:					
Force Concurrent with Max Py	1,925	65	-35	-1,909	-1,514
Force Concurrent with abs(Max) Vx	1,414	-115	37	1,694	2,133
Force Concurrent with abs(Max) Vz	1,439	88	-54	-26	-1,882
Force Concurrent with abs(Max) Mx	1,439	88	-54	-26	-1,882
Force Concurrent with abs(Max) Mz	1,414	-115	37	1,694	2,133
Seismic Event Loading[1,2,4] (R=1.0):					
Force Concurrent with Max Py	2,077	-130	77	3,343	2,970
Force Concurrent with abs(Max) Vx	1,353	-173	77	3,336	3,506
Force Concurrent with abs(Max) Vz	1,682	59	-238	-10,130	-1,131
Force Concurrent with abs(Max) Mx	1,947	70	-238	-10,150	-1,237
Force Concurrent with abs(Max) Mz	1,353	-173	77	3,336	3,506
Column Design Forces (At Base with MM[3])	832	-43	-238	-11,609	899
Plastic Hinging Forces				7,704	9,551

Notes:
[1] - Loads provided by TY Lin International to Haley & Aldrich on 16 June 2009
[2] - Seismic loads revised by TY Lin International and provided to Haley & Aldrich on 19 June 2009.
[3] - MM = moment magnification.
[4] - Loads shown are in TY Lin's coordinate system and were converted to FB MultiPier coordinate system by Haley & Aldrich.

Maine Department of Transportation	Project: Proposed Replacement Bridge Over Presumpscot River and MCRR	Boring No.: BB-FPR-101
Soil/Rock Exploration Log US CUSTOMARY UNITS	Location: Routes 26/100, Falmouth, Maine	PIN: 15094.00

Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	24.7	**Auger ID/OD:**	-
Operator:	D. McKeen	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E. Beirne	**Rig Type:**	Mobile Drill B-47 Bombardier	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/15/08 to 10/22/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	-
Boring Location:	E1005264, N326663 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	7.8 (10/23/08, 0730)
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/13	0.0 - 2.0	1-7-2-1	9	9	Open			Dark brown, moist, stiff, sandy SILT, trace coarse gravel, no structure, layer of silty SAND from 0.8 to 1.2 ft, occasional organics throughout -TOPSOIL-(ML)	
								22.4		2.3	
	2D	24/ 24	2.3 - 4.3	2-1-1-1	2	2				Olive-brown, moist, very soft, sandy SILT, no structure (ML) -ALLUVIAL DEPOSIT-	
5	3D	24/18	4.0 - 6.0	1-1-1-1	2	2				Olive-brown, moist to wet, very soft, silty CLAY, little fine sand, no structure, occasional organics (CL) -ALLUVIAL DEPOSIT-	
10								14.7		10.0	
	4D ~~MV~~	24/20	10.0 - 12.0 ~~10.5 - 11.0~~	1-1-1-2	2	2				Olive-brown to gray-brown, mottled, wet, very soft, sandy CLAY to clayey fine SAND, no structure -ALLUVIAL DEPOSIT-(CL/SC) Note: Attempted vane at 10.0 ft, unable to push.	
15								9.7		15.0	
	5D	24/23	15.0 - 17.0	1-1-1-1	2	2				Gray, wet to saturated, very loose, SAND, little silt, poorly graded, sulfurous odor, frequent organics, micaceous -ALLUVIAL DEPOSIT-(SP)	
20	6D	24/17	20.0 - 22.0	3-3-2-5	5	5				Gray, wet, loose, SAND, poorly graded, slight sulfurous odor, large wood fragments, coarse sand in tip of spoon -ALLUVIAL DEPOSIT-(SP)	
								1.7		23.0	
										Note: Approximate strata change at 23.0 ft based on drill action.	
							21				
25	7D	24/24	25.0 - 27.0	WOR-WOR-WOR-WOH			22			Gray, wet, very soft, lean CLAY, trace fine sand, no structure, black organic streaking below approximately 26.2 ft	

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 17

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Boring No.: BB-FPR-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge Over Presumpscot River and MCRR
Location: Routes 26/100, Falmouth, Maine

Boring No.: BB-FPR-101
PIN: 15094.00

Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	24.7	**Auger ID/OD:**	-
Operator:	D. McKeen	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E. Beirne	**Rig Type:**	Mobile Drill B-47 Bombardier	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/15/08 to 10/22/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	-
Boring Location:	E1005264, N326663 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	7.8 (10/23/08, 0730)
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							15			-MARINE DEPOSIT-(CL)	
	V1A		27.5 - 28.0	Su=530/130 psf			17			3x6 in. vane raw torque readings: V1A: 360/90 in-lbs	
							17				
							16				
30							Push				
35	V2A		35.5 - 36.0	Su=650/180 psf			Open			3x6 in. vane raw torque readings: V2A: 440/120 in-lbs	
	MU	24/0	36.5 - 38.5								
	1U	24/23	38.5 - 40.5								C#CRC-3 WC=31.2% LL=33 PL=17 PI=16
40	MV		40.5 - 41.0					-15.8		40.5 Note: Attempted vane at 40.0 ft, unable to push, probable sand layer.	
	V3A		41.5 - 42.0	Su=>880 psf						3x6 in. vane raw torque readings: V3A: >600 in-lbs Note: Unable to turn vane for remolded value at 42.0 ft due to probable sand layer.	
45	8D MV	24/16	45.0 - 47.0 45.5 - 46.0	7-11-8-8	19	19				Gray, wet to saturated, medium dense SAND, little clay, poorly graded, no structure, small clayey sand layer at tip of sample -MARINE DEPOSIT-(SP) Note: Attempted vane at 45.0 ft, unable to push.	
							1				
50	9D	24/14	50.0 - 52.0	5-4-4-5	8	8	3			Gray, wet, loose, SAND, poorly graded, no structure -MARINE DEPOSIT-(SP)	
							8				

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 18

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge Over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FPR-101
PIN: 15094.00

Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	24.7	**Auger ID/OD:**	-
Operator:	D. McKeen	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E. Beirne	**Rig Type:**	Mobile Drill B-47 Bombardier	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/15/08 to 10/22/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	-
Boring Location:	E1005264, N326663 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	7.8 (10/23/08, 0730)
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
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WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
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Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
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WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							16				
							21				
55	10D	24/5	54.4 - 56.4	5-3-3-4	6	6	1			Gray, wet, loose, SAND, trace fine gravel, poorly graded, no structure -MARINE DEPOSIT-(SP)	
							5				
							8				
							10				
							12				
60							3			Note: Approximately 3.0 ft of soil measured inside casing after drill rods were removed.	
							6				
							9				
							12				
							18				
							81				
65	11D	24/5	65.0 - 67.0	10-9-18-6	27	27	83			Gray, wet, medium dense, SAND, trace silt, little coarse to fine gravel, well graded, no structure -MARINE DEPOSIT-(SW)	
							110				
							63				
							61				
							66				
70	12D	24/12	70.0 - 72.0	5-6-6-8	12	12	64			Gray, wet, medium dense, SAND, little clay, trace fine gravel, well graded, no structure, layer of clayey sand to sandy clay from approximately 70.5 to 71.0 ft -MARINE DEPOSIT-(SW)	
							57				
							70				
							73				
							103				
75	13D	24/18	75.0 - 77.0	10-6-3-3	9	9	114			Gray, saturated, loose to stiff, alternating layers of sandy CLAY and clayey SAND, trace fine gravel, no structure -MARINE DEPOSIT-(CL/SC)	
							80				
							98				

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 19

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Proposed Replacement Bridge Over Presumpscot River and MCRR **Location:** Routes 26/100 Falmouth, Maine	**Boring No.:** BB-FPR-101 **PIN:** 15094.00

Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	24.7	**Auger ID/OD:**	-
Operator:	D. McKeen	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E. Beirne	**Rig Type:**	Mobile Drill B-47 Bombardier	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/15/08 to 10/22/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	-
Boring Location:	E1005264, N326663 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	7.8 (10/23/08, 0730)
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt

R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person

S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected

$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							123				
							123				
80	14D	24/12	80.0 - 82.0	2-1-2-6	3	3	136			Gray, saturated, very loose, SAND, poorly graded, no structure -MARINE DEPOSIT-(SP)	
							150				
							180				
							190				
							180				
85	15D	24/10	85.0 - 87.0	6-5-5-10	10	10	220			Gray, wet, medium dense, SAND, little fine gravel, well graded, no structure, poorly graded medium to fine SAND in tip of spoon -MARINE DEPOSIT-(SP)	
							214				
							229				
							221				
							186				
90	16D	24/14	90.0 - 92.0	10-11-14-17	25	25	225			Gray, wet to saturated, medium dense, SAND, trace fine gravel, poorly graded, no structure -MARINE DEPOSIT-(SP)	
							220				
							408	-67.6		92.3 Note: Approximate strata change based on casing blow counts. Driller noted gravelly sand layers between 92.0 and 100 ft.	
							465				
							467				
95							Open				
										Note: Advanced roller bit through cobble from 99.0 to 99.9 ft.	
100	17D	24/12	100.0 - 102.0	31-30-75-48	105	105	44			Gray, wet, very dense, gravelly SAND, trace clay, well graded, bonded, large piece of gravel stuck in tip of spoon -GLACIAL TILL-(SW)	
							56				
							73			Note: Encountered several gravel layers and cobbles between 102.0 and 105.0 ft.	
							135				

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 20

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge Over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FPR-101
PIN: 15094.00

Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	24.7	**Auger ID/OD:**	-
Operator:	D. McKeen	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E. Beirne	**Rig Type:**	Mobile Drill B-47 Bombardier	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/15/08 to 10/22/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	-
Boring Location:	E1005264, N326663 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	7.8 (10/23/08, 0730)
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
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MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
105	18D	16/10	104.5 - 105.8	57-58-50/4"			188			Gray, moist to wet, very dense, SAND, some coarse to fine gravel, little clay, well graded, bonded to somewhat bonded -GLACIAL TILL-(SW)	
							73				
							52				
							107			Note: Encountered cobble at 107.1 ft.	
							45				
110	19D	16/11	109.5 - 110.8	37-47-50/4"			51			Gray, wet to saturated, very dense to hard, alternating layers of sandy SILT and silty SAND, poorly graded, no structure -GLACIAL TILL-(ML/SM)	
							Wash Ahead				
115	20D	12/12	114.5 - 115.5	47-100/6"						Gray, moist, very dense, silty SAND, trace fine gravel, poorly graded, bonded -GLACIAL TILL-(SP)	
120	21D	4/4	119.5 - 119.8	100/4"						Gray, moist, very dense, gravelly SAND, trace silt, poorly graded, bonded -GLACIAL TILL-(SP)	
125								-100.4		Note: Encountered obstruction at 124.4 ft, advanced roller bit to 125.1 ft. Probable bedrock fragments observed in wash water. Attempted to advance casing to top of bedrock, casing stopped at 120.8 ft due to crushed drive shoe. 125.1	
										Bottom of Exploration at 125.1 feet Below Ground Surface.	
										Note: Groundwater observation well installed in completed borehole. See Observation Well Installation Report for Details.	
130											

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 21

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge Over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FPR-102
PIN: 15094.00

Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	26.3	**Auger ID/OD:**	-
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E. Beirne	**Rig Type:**	Mobile Drill B-47 Bombardier	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/23/08 to 10/28/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E1005297, N326555 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	9.1 (10/28/09, 0705)
Hammer Efficiency Factor: .6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/10	0.0 - 2.0	1-WOH-2-1	2	2	Open			Gray-brown to black, moist, very loose to loose(2 to 2.7 ft), SAND, poorly graded, interbedded with sandy ORGANIC SOIL, no structure, organic matter and wood fragments throughout -TOPSOIL-(SP/OL/OH)	
	2D	24/22	2.0 - 4.0	1-3-4-3	7	7	9	23.6		2.7	
							14			Gray-brown, moist to wet, loose, SAND, little silt, poorly graded, no structure, occasional organics -ALLUVIAL DEPOSIT-(SP)	
	3D	24/18	4.0 - 6.0	2-2-1-1	3	3	7	22.3		4.0	
5							7			Brown, moist to wet, soft, sandy SILT, organics throughout, no structure -ALLUVIAL DEPOSIT-(ML)	
	4D	24/16	6.0 - 8.0	1-1-WOH-1	1	1	7			Brown, wet to saturated, soft, sandy SILT, organics throughout, no structure -ALLUVIAL DEPOSIT-(ML)	
							8				
							9				
							8				
10	5D	24/17	10.0 - 12.0	1-WOH-1-WOH	1	1	18			Brown, wet to saturated, very loose to very soft, interbedded layers of silty SAND, poorly graded and sandy SILT, mottled, few organics -ALLUVIAL DEPOSIT-(SM/ML)	
							27				
							22				
							21				
							21				
15	6D	24/19	15.0 - 17.0	3-1-2-4	3	3	30	11.3		15.0 Gray-brown to gray, wet to saturated, very loose, SAND, some silt, poorly graded, layers of organic matter throughout, sulfurous odor, no structure -ALLUVIAL DEPOSIT-(SP)	
							40				
							41				
							48				
							47				
20	7D	24/14	20.0 - 22.0	2-3-4-7	7	7	43			Gray, wet to saturated, loose SAND, trace silt, poorly graded, few layers of organic matter, slight sulfurous odor, no structure -ALLUVIAL DEPOSIT-(SP)	
							50				
							58				
							70				
							66				
25	8D	24/6	25.0 - 27.0	5-7-6-9	13	13	58			Gray, wet to saturated, medium dense, SAND, trace silt, trace fine gravel, poorly graded, one organic layer, no structure, coarser with depth	

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 22

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Proposed Replacement Bridge Over Presumpscot River and MCRR	**Boring No.:**	BB-FPR-102		
Location:	Routes 26/100 Falmouth, Maine	**PIN:**	15094.00		
Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	26.3	**Auger ID/OD:**	-
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E. Beirne	**Rig Type:**	Mobile Drill B-47 Bombardier	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/23/08 to 10/28/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E1005297, N326555 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	9.1 (10/28/09, 0705)
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							71			-ALLUVIAL DEPOSIT-(SP)	
							59	-1.2		27.5	
							57			Note: Wash water and casing blows indicate clay beginning at approximately 27.5 ft.	
							59				
30	V1A		30.5 - 31.0	Su=580/220 psf			37			3x6 in. vane raw torque readings: V1A: 580/150 in-lbs	
	1U	24/22	31.5 - 33.5				37				C#CRC-1 WC=36.6% LL=38 PL=18 PI=20
							32				
							34				
	9D	24/24	34.0 - 36.0	push thru vane			39			Gray, wet to saturated, medium stiff, silty CLAY, little medium to fine sand, occasional sand seams -MARINE DEPOSIT-(CL) 3x6 in. vane raw torque readings: V2A: >600/150 in-lbs	
35	V2A		34.5 - 35.0	Su=880/220 psf			46				
							39				
							36				
							37				
							33				
40	10D	24/24	40.0 - 42.0	push thru vane			46			Gray, wet to saturated, medium stiff, silty CLAY, trace fine sand, black organic staining from approximately 41.0 ft, no structure -MARINE DEPOSIT-(CL) 3x6 in. vane raw torque readings: V3A: 540/60 in-lbs	
	V3A		40.5 - 41.0	Su=800/90 psf			40				
							36				
							32				
							44				
45	11D	24/18	45.0 - 47.0	8-9-6-13	15	15	16	-18.9		Note: Attempted vane at 45.0 ft, unable to push. 45.2	
	MV		45.5 - 46.0				24			Gray, wet, medium dense, SAND, little silty clay, poorly graded, no structure -MARINE DEPOSIT-(SP)	
							25				
							46			Note: Approximately 2.0 ft of soil measured inside of casing after drill rods were removed.	
	MD	24/0	49.5 - 51.5	2-3-3-5	6	6	53			No recovery	
50							10				
							16				

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 23

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge Over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FPR-102
PIN: 15094.00

Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	26.3	**Auger ID/OD:**	-
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E. Beirne	**Rig Type:**	Mobile Drill B-47 Bombardier	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/23/08 to 10/28/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E1005297, N326555 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	9.1 (10/28/09, 0705)
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							24				
							39				
	MD	24/0	54.5 - 56.5	6-6-7-12	13	13	35			No recovery. Redrove split spoon for 2nd attempt to retrieve sample at 54.5 ft. Gray, wet to saturated, medium dense, SAND, trace silty clay, poorly graded, no structure -MARINE DEPOSIT-(SP)	
55							11				
							23				
							32				
							61				
	14D	24/24	59.5 - 61.5	5-3-5-6	8	8	95			Gray, wet to saturated, loose SAND, little silty CLAY, poorly graded, no structure, sandy clay for last 3 in. of spoon -MARINE DEPOSIT-(SP)	
60							35				
							36				
							38				
							33				
	15D	24/5	64.5 - 66.5	6-8-9-7	17	17	54			Gray, saturated, medium dense, SAND, little silt, poorly graded, no structure -MARINE DEPOSIT-(SP)	
65							61				
							47				
							77				
							76				
							125				
70							173				
							174				
							177				
							92				
	16D	24/15	74.5 - 76.5	2-3-4-4	7	7	80			Gray, wet, loose SAND, trace fine gravel, no structure -MARINE DEPOSIT-(SP)	
75							79				
							74				
							64				

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 24

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge Over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FPR-102
PIN: 15094.00

Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	26.3	**Auger ID/OD:**	-
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E. Beirne	**Rig Type:**	Mobile Drill B-47 Bombardier	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/23/08 to 10/28/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E1005297, N326555 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	9.1 (10/28/09, 0705)
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							64				
							79				
80							138				
							108				
							77				
							72				
	17D	24/16	84.5 - 86.5	1-3-6-9	9	9	71			Gray, wet to saturated, loose, SAND, trace silt and fine gravel, well graded, no structure -MARINE DEPOSIT-(SP)	
85							93				
							124				
							182				
							227				
							235				
90							284				
							256				
							315	-66.2		92.5 Note: Approximate strata change based on casing blows.	
							330				
	18D	24/15	94.5 - 96.5	21-25-24-11	49	49	379			Gray, moist, dense, silty SAND, little coarse to fine gravel, poorly graded, bonded, one layer of poorly graded medium to fine SAND -GLACIAL TILL-(SM)	
95							Wash Ahead				
										Note: Cobble encountered from 98.0 to 98.4 ft.	
	19D	24/6	99.5 - 101.5	35-26-9-12	35	35				Gray, wet, medium dense, SAND, some coarse to fine gravel, poorly graded, trace silt, somewhat bonded -GLACIAL TILL-(SP)	
100							Wash Ahead	-74.2		100.5 Gray, wet, medium dense, sandy GRAVEL, well graded, no structure -GLACIAL TILL-(GW)	

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 25

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Proposed Replacement Bridge Over Presumpscot River and MCRR Location: Routes 26/100 Falmouth, Maine	Boring No.: BB-FPR-102 PIN: 15094.00

Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	26.3	**Auger ID/OD:**	-
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E. Beirne	**Rig Type:**	Mobile Drill B-47 Bombardier	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/23/08 to 10/28/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E1005297, N326555 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	9.1 (10/28/09, 0705)
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
105	20D	10/5	104.5 - 105.3	65-75/5"						Gray, wet to saturated, very dense, silty SAND, trace fine gravel, poorly graded, bonded to no structure, fine sand and silt last 5 in. of sample -GLACIAL TILL-(SM)	
110	21D	8/5	109.5 - 110.2	83-75/3"						Gray, wet to moist, very dense, SAND, little silt and coarse to fine gravel, poorly graded, bonded -GLACIAL TILL-(SP)	
115	22D	5/4	114.5 - 114.9	100/5"						Gray, wet, very dense, silty SAND, little fine gravel, poorly graded, somewhat bonded -GLACIAL TILL-(SM)	
120	MD	3/0	119.5 - 119.8	125/3"						No recovery	
125	24D	7/6	124.5 - 125.1	84-50/2"			Wash Ahead			Gray, moist, very dense, SAND, trace silt and coarse to fine gravel, poorly graded, some original rock fabric present, weathered gravel	
130								-103.7		130.0 Top of Bedrock at El.-103.7	

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 26

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Proposed Replacement Bridge Over Presumpscot River and MCRR	**Location:**	Routes 26/100 Falmouth, Maine	**Boring No.:**	BB-FPR-102
				PIN:	15094.00
Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	26.3	**Auger ID/OD:**	-
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E. Beirne	**Rig Type:**	Mobile Drill B-47 Bombardier	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/23/08 to 10/28/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E1005297, N326555 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	9.1 (10/28/09, 0705)
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	26D	1/2	130.0 - 130.1	75/1"			NQ			Very soft to moderately hard, slightly to highly weathered, dark gray SCHIST. Joints are extremely close to close, low angle to vertical, planar, stepped and undulating, smooth to rough, disintegrated to decomposed, tight to open, silt infilling Rock Mass Quality=Very Poor -BERWICK FORMATION- R1:Core Times (min:sec): 130.1-131.1' (5:00), 131.1-132.2' (5:00)	
	R1	25/20	130.1 - 132.2	RQD = 0%							
	R2	53/53	132.1 - 136.5	RQD = 53%						Very soft to hard, fresh to highly weathered, dark gray, SCHIST. Joints are low angle to moderately dipping, very close to moderately spaced, planar, stepped and undulating, smooth to rough, fresh to decomposed, very tight to partly open, some silt infilling. 132.1 to 133.1 ft highly decomposed, very micaceous Rock Mass Quality=Fair -BERWICK FORMATION- R2:Core Times (min:sec): 132.1-133.1' (3:00), 133.1-134.1' (3:00), 134.1-135.1' (2:00), 135.1-136.1' (3:00), 136.1-136.5' (1:00)	
135											
	R3	60/60	136.5 - 141.5	RQD = 65%						Very soft to hard, fresh to highly weathered, gray, fine-grained to aphanitic SCHIST. Joints are moderately dipping to horizontal, very close to moderately spaced, planar to undulating, smooth to rough, fresh to decomposed, tight to partly open. 136.5 to 137.3 ft highly weathered. One vertical joint with calcite mineralization. Rock Mass Quality=Fair -BERWICK FORMATION- R3:Core Times (min:sec): 136.5-137.5' (3:00), 137.5-138.5' (2:00), 138.5-139.5' (2:00), 139.5-140.5' (2:00), 140.5-141.5' (3:00)	
140											
								-115.2		141.5	
										Bottom of Exploration at 141.5 feet Below Ground Surface. Note: Observation well installed in completed borehole. See Observation Well Installation Report for details.	
145											
150											
155											

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 27

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Proposed Replacement Bridge Over Presumpscot River and MCRR	**Location:**	Routes 26/100 Falmouth, Maine	**Boring No.:**	BB-FPR-103
				PIN:	15094.00
Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	0.6	**Auger ID/OD:**	-
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E. Beirne/B. Steinert	**Rig Type:**	CME 45 Skid	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/07/08 to 10/14/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E1005333, N326399 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	Not observed
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/3	0.0 - 2.0	WOR-WOR-WOR-WOH			1			Gray, wet, very loose, SAND, poorly graded, wood in tip -ALLUVIAL DEPOSIT-(SP) Note: Poor recovery, pushing wood	
							2				
	2D	24/4	2.0 - 4.0	WOH-3-8-15	11	11	2			Gray, saturated, medium dense, SAND, trace gravel and silt, poorly graded, wood in tip -ALLUVIAL DEPOSIT-(SP) Note: Poor recovery, pushing wood.	
							29				
	3D	24/20	4.0 - 6.0	16-1-1-2	2	2	11	-3.8		Black, wet, medium dense, SAND, little silt, trace gravel, poorly graded (SP)	
5							20			4.4	
	4D ~~MV~~	24/24	6.0 - 8.0 ~~6.5 - 7.0~~	WOR-WOH-WOH-WOH			12			Gray, wet, very soft, silty CLAY, no structure -MARINE DEPOSIT-(CL) Note: Attempted vane at 6.0 ft, unable to push.	
							Open				
	5D ~~V1A~~	24/24	8.0 - 10.0 ~~8.5 - 9.0~~	push thru vane ~~Su=750/370 psf~~						Gray, wet, medium stiff, silty CLAY, no structure -MARINE DEPOSIT-(CL) 3x6 in. vane raw torque readings: V1A: 510/250 in-lbs	
10	6D	24/24	10.0 - 12.0	WOR-WOH-WOH-WOH						Gray, wet, medium stiff, silty CLAY, trace fine sand, no structure, fine sand seam at 10.8 ft -MARINE DEPOSIT-(CL)	
	7D ~~V2A~~	24/24	12.0 - 14.0 ~~12.5 - 13.0~~	push thru vane ~~Su=850/260 psf~~						Gray, wet, medium stiff, silty CLAY, no structure, frequent sand partings, black organic streaking 3x6 in. vane raw torque readings: V2A: 580/175 in-lbs	
	8D	24/24	14.0 - 16.0	WOR-WOH-WOH-1						Gray, wet, medium stiff, silty CLAY, little fine sand, no structure, frequent fine sand partings, black organic streaking -MARINE DEPOSIT-(CL)	
15											
	9D ~~MV~~	24/24	16.0 - 18.0 ~~16.5 - 17.0~~	WOR-2-2-9	4	4				Gray, saturated to wet, medium stiff, sandy CLAY to clayey SAND, interbedded, no structure Note: Attempted vane at 16.0 ft, unable to push.	
								-17.0		17.6	
	10D	24/14	18.0 - 20.0	1-2-3-4	5	5	Push			Gray, saturated, loose, fine SAND, trace fine gravel and medium sand, little silt, poorly graded, no structure, iron staining from 19.8 to 20.0 ft. (SP)	G#150313 A-2-4(0)
20							5				
							16				
							25				
							40				
							51				
25							58				

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 28

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Proposed Replacement Bridge Over Presumpscot River and MCRR Location: Routes 26/100 Falmouth, Maine	Boring No.: BB-FPR-103 PIN: 15094.00

Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	0.6	**Auger ID/OD:**	-
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E. Beirne/B. Steinert	**Rig Type:**	CME 45 Skid	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/07/08 to 10/14/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E1005333, N326399 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	Not observed
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							55				
							58				
	11D	24/19	28.0 - 30.0	WOR-1-4-4	5	5	55			Gray-brown, saturated, loose, fine SAND, poorly graded, no structure, trace coarse and medium sand, little silt (SP)	G#150314 A-2-4(0)
							54				
30							60				
							90				
							91				
							110				
							119				
35							121				
							132				
							144				
	12D	24/24	38.0 - 40.0	22-37-50/3"			66			Tan, very dense, wet, medium and fine SAND, trace fine gravel, little silt, poorly graded -MARINE DEPOSIT-(SP)	G#150315 A-1-b(0), SP
							101				
40							57				
							57				
							67				
							59				
							65				
45							75				
							92				
							110				
	13D	24/17	48.0 - 50.0	3-4-10-16	14	14	100			Gray, medium dense, wet, medium and fine SAND, trace coarse sand, little silt, poorly graded -MARINE DEPOSIT-(SP)	G#150316 A-1-b(0)
							110				
50							142	-49.9		50.5 Note: Encountered cobbles based on drill action at 50.5 ft.	
							149				

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 29

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Proposed Replacement Bridge Over Presumpscot River and MCRR **Location:** Routes 26/100 Falmouth, Maine	**Boring No.:** BB-FPR-103 **PIN:** 15094.00

Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	0.6	**Auger ID/OD:**	-
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E. Beirne/B. Steinert	**Rig Type:**	CME 45 Skid	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/07/08 to 10/14/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E1005333, N326399 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	Not observed
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
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C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							150				
							151				
							173				
55							136				
							137				
							200				
	14D	24/12	58.0 - 60.0	4-9-10-14	19	19	172			Gray, medium dense, wet, SAND, trace clay, poorly graded -GLACIAL TILL-(SP)	
							159				
60							155				
							163				
							154				
							115				
							112				
65							164				
							226				
							175				
	15D	24/12	68.0 - 70.0	25-7-10-50/5"	17	17	166			Gray, medium dense, wet, sandy SILT -GLACIAL TILL-(ML)	
							139				
70							353			Note: Cobbles encountered from 70.9 to 71.4 ft (approximate).	
							138				
							163				
							208				
							Wash Ahead				
75							Wash Ahead				
										Gray, very dense, wet, SAND, little coarse to fine gravel, trace silt,	

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 30

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Proposed Replacement Bridge Over Presumpscot River and MCRR Location: Routes 26/100 Falmouth, Maine	Boring No.: BB-FPR-103 PIN: 15094.00

Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	0.6	**Auger ID/OD:**	-
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E. Beirne/B. Steinert	**Rig Type:**	CME 45 Skid	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/07/08 to 10/14/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E1005333, N326399 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	Not observed
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	16D	24/21	78.0 - 80.0	34-39-70-50/3"	109	109	100			poorly graded -GLACIAL TILL-(SP)	
							122				
80							180				
							197				
							148			Note: Clay cuttings observed in wash at 82.0 ft (approximate)	
							167				
85											
	R1	62/62	86.5 - 91.7	RQD = 35%			NQ	-85.9		86.5 Top of Bedrock at El.-85.9 Very soft to hard, slightly to highly weathered, dark gray SCHIST. Joints are extremely close to close, low to vertical angles, planar, stepped to undulating, smooth to rough, disintegrated to decomposed, tight to open, some silt infilling Rock Mass Quality=Poor -BERWICK FORMATION- R1:Core Times (min:sec): 86.5-87.5' (2:00), 87.5-88.5' (6:00), 88.5-89.5' (4:00), 89.5-90.5' (4:00), 90.5-91.7' (4:00)	
90											
	R2	60/60	91.7 - 96.7	RQD = 45%						Moderately hard to hard, slightly to highly weathered, dark gray SCHIST. Joints are extremely close to close, low to vertical angles, planar, stepped to undulating, smooth to rough, disintegrated to decomposed, tight to open, some silt infilling Rock Mass Quality=Poor -BERWICK FORMATION- R2:Core Times (min:sec): 91.7-92.7' (3:00), 92.7-93.7' (2:00), 93.7-94.7' (3:00), 94.7-95.7' (2:00), 95.7-96.7' (3:00)	
95											
								-96.1		96.7 **Bottom of Exploration at 96.7 feet Below Ground Surface.**	
100											

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 31

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Proposed Replacement Bridge Over Presumpscot River and MCRR Location: Routes 26/100 Falmouth, Maine	Boring No.: BB-FPR-104 PIN: 15094.00

Driller:	New Hampshire Boring, Inc.	**Elevation (ft.)**	29.4	**Auger ID/OD:**	-
Operator:	B. Thompson	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X ATV	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	11/03/08 to 11/05/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E100537, N326256 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	Not observed
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/11	0.0 - 2.0	1-2-3-4	5	5	Push			Brown to black, moist, loose, SAND, trace silt and fine gravel, poorly graded, no structure -FILL-(SP)	
								28.2		1.2 Brown, moist to wet, medium stiff to loose, sandy SILT to silty SAND, no structure -FILL-(SM/ML)	
	MD	24/0	2.0 - 4.0	7-6-4-5	10	10				No Recovery	
								25.4		4.0	
	3D	24/13	4.0 - 6.0	7-4-3-3	7	7				Dark brown to brown, wet, loose, silty SAND, poorly graded, no structure -ALLUVIAL DEPOSIT-(SM)	
5											
	4D	24/9	6.0 - 8.0	4-3-1-2	4	4				Dark brown to brown, wet, loose, silty SAND, poorly graded, no structure -ALLUVIAL DEPOSIT-(SM)	
10	5D	24/24	10.0 - 12.0	1-1-WOH-1	1	1				Brown, wet to saturated, very loose, silty SAND, no structure, occasional organics -ALLUVIAL DEPOSIT-(SM)	
								18.4		11.0 Brown to light gray, wet to saturated, very soft, silty CLAY, some fine to medium sand, no structure, occasional organics -ALLUVIAL DEPOSIT-(CL)	
										Note: Wash water indicated layer of sand and gravel from 14.5 to 15.0 ft.	
15	6D	24/16	15.0 - 17.0	3-5-4-7	9	9				Light gray, wet, medium stiff, SILT, some fine sand, no structure -ALLUVIAL DEPOSIT-(ML)	
								13.0		16.4 Brown to light gray, saturated SAND, little silt, poorly graded, no structure -ALLUVIAL DEPOSIT-(SP)	
								10.4		19.0 Note: Driller noted change from sand to gray clay at approximately 19.0 ft based on wash cuttings.	
20	7D	24/24	20.0 - 22.0	2-1-2-1	3	3	15			Gray, wet, soft, silty CLAY, trace fine sand, blocky to no structure -MARINE DEPOSIT-(CL)	
							9				
	V1A		22.5 - 23.0	Su=>880 psf			8			3x6 in. vane raw torque readings: V1A: >600 in-lbs	
							11				
							13				
25	V2A		25.5 - 26.0	Su=>880 psf			Wash Ahead			3x6 in. vane raw torque readings:	

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 32

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge Over Presumpscot River and MCRR	**Location:** Routes 26/100 Falmouth, Maine	**Boring No.:** BB-FPR-104	**PIN:** 15094.00

Driller:	New Hampshire Boring, Inc.	**Elevation (ft.)**	29.4	**Auger ID/OD:**	-
Operator:	B. Thompson	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X ATV	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	11/03/08 to 11/05/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E100537, N326256 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	Not observed
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
										V2A: >600 in-lbs	
								1.4		28.0 Note: Driller noted sand in wash beginning at approximately 28.0 ft.	
30	8D	24/19	30.0 - 32.0	7-7-8-7	15	15				Brown to gray-brown, wet to saturated, SAND, some silt, trace gravel, well graded, no structure -GLACIAL TILL-(SW)	
35	9D	24/14	35.0 - 37.0	9-10-12-15	22	22	26			Gray, wet, medium dense, SAND, little silt, little coarse to fine gravel, well graded, no structure to somewhat bonded -GLACIAL TILL-(SW)	
							27				
							114				
							116				
							119				
40	10D	24/16	40.0 - 42.0	14-14-22-23	36	36	39			Gray-brown, moist, dense, SAND, little silt, little coarse to fine gravel, well graded, bonded, contains weathered gravel -GLACIAL TILL-(SW)	
							40				
							60				
							52				
							49	-14.6		44.0 Note: Driller noted color and density change at 44.0 ft.	
45	11D	24/15	45.0 - 47.0	12-23-37-32	60	60	47			Gray, moist, very dense, silty SAND, little gravel, poorly graded, bonded -GLACIAL TILL-(SM)	
							52				
							51				
							68				
							70				
50	12D	24/15	50.0 - 52.0	19-16-18-18	34	34	36	-20.6		50.0 Gray, moist, dense, SAND, little silt, little coarse to fine gravel, poorly graded, bonded -GLACIAL TILL-(SP)	
							44				

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 33

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge Over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FPR-104
PIN: 15094.00

Driller:	New Hampshire Boring, Inc.	**Elevation (ft.)**	29.4	**Auger ID/OD:**	-
Operator:	B. Thompson	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X ATV	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	11/03/08 to 11/05/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E100537, N326256 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	Not observed
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
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V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							44				
							57				
							42				
55	13D	24/16	55.0 - 57.0	19-22-38-25	60	60	44			Gray, moist, very dense, SAND, some silt, little coarse to fine gravel, poorly graded, bonded -GLACIAL TILL-(SP)	
							43	-26.8		56.2 Gray, wet to saturated, SAND, trace silt, little coarse to fine gravel, well graded, no structure -GLACIAL TILL-(SW) Note: Driller noted sand layers up to 1 ft thick between 55.0 and 60.0 ft.	
							40				
							38				
							36				
60	14D	24/15	60.0 - 62.0	8-6-15-36	21	21	9			Gray, wet to saturated, medium dense, SAND, some coarse to fine gravel, little silt, well graded, no structure to bonded -GLACIAL TILL-(SW)	
							21				
							61				
							82				
							73				
65	15D	24/10	65.0 - 67.0	16-20-26-19	46	46	Wash Ahead			Gray, moist, dense, SAND, some silt, little gravel, poorly graded, bonded -GLACIAL TILL-(SP)	
70											
75	16D	24/18	75.0 - 77.0	23-33-51-48	84	84	Wash Ahead			Note: Cobble encountered from 74.5 to 75.0 ft. Gray, moist to wet, very dense, SAND, some silt, little coarse to fine gravel, well graded, bonded, last 0.5 ft coarse to fine layer -GLACIAL TILL-(SW)	

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 34

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge Over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FPR-104
PIN: 15094.00

Driller:	New Hampshire Boring, Inc.	**Elevation (ft.)**	29.4	**Auger ID/OD:**	-
Operator:	B. Thompson	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X ATV	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	11/03/08 to 11/05/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E100537, N326256 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	Not observed
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
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MV = Unsuccessful Insitu Vane Shear Test attempt
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
80											
85	17D	24/24	85.0 - 87.0	40-73-55-71	128	128				Gray, moist to wet, very dense, silty SAND, little coarse to fine gravel, bonded -GLACIAL TILL-(SM)	
90											
95	18D	24/16	95.0 - 97.0	18-37-47-48	84	84				Gray, wet, very dense, silty SAND, little coarse to fine gravel, bonded, occasional fine sand layers -GLACIAL TILL-(SM)	
100							Wash Ahead				

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 35

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Proposed Replacement Bridge Over Presumpscot River and MCRR	**Boring No.:**	BB-FPR-104		
Location:	Routes 26/100 Falmouth, Maine	**PIN:**	15094.00		
Driller:	New Hampshire Boring, Inc.	**Elevation (ft.)**	29.4	**Auger ID/OD:**	-
Operator:	B. Thompson	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X ATV	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	11/03/08 to 11/05/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E100537, N326256 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	Not observed
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
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MU = Unsuccessful Thin Wall Tube Sample attempt
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MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
105	19D	24/17	105.0 - 107.0	49-33-108-100/5"	141	141				Gray, wet, very dense, silty SAND, little coarse to fine gravel, bonded, occasional fine sand layers, cobble in tip of spoon -GLACIAL TILL-(SM)	
110								-80.6		110.0 Note: Driller noted clay in wash between 110.0 and 115.0 ft.	
115	20D	24/22	115.0 - 117.0	20-43-70-62	113	113				Gray, moist to saturated, very dense, clayey SAND, little fine gravel, bonded to laminated, medium to coarse sand layers in upper 18 in., alternating fine sand and clay layers for last 4 in. of spoon -GLACIAL TILL-(SC)	
120											
125	21D	24/20	125.0 - 127.0	44-90-95-100/4"	185	185	Wash Ahead	-96.8		Gray, moist to wet, hard, sandy SILT, trace fine gravel, bonded, fine sand layers throughout -GLACIAL TILL-(ML)	
								-97.6		126.2 Gray, moist, hard, silty CLAY, trace fine sand, no structure -GLACIAL TILL-(CL)	
										127.0 Top of Bedrock at El.-97.6	
	R1	60/56	129.0 - 134.0	RQD = 36%			NQ	-99.6		129.0 Hard to very hard, fresh, dark gray to white, fine-grained to aphanitic, GNEISS. Joints are low angle to moderately dipping, very close to moderately spaced, planar to undulating, smooth to rough, fresh to	
130											

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 36

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Proposed Replacement Bridge Over Presumpscot River and MCRR Location: Routes 26/100 Falmouth, Maine	Boring No.: BB-FPR-104 PIN: 15094.00

Driller:	New Hampshire Boring, Inc.	**Elevation (ft.)**	29.4	**Auger ID/OD:**	-
Operator:	B. Thompson	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X ATV	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	11/03/08 to 11/05/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E100537, N326256 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	Not observed
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
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R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
										discolored, tight to partly open, some silt infilling Rock Mass Quality=Poor -BERWICK FORMATION- R1:Core Times (min:sec): 129.0-130.0' (10:00), 130.0-131.0' (9:00), 131.0-132.0' (6:00), 132.0-133.0' (6:00), 133.0-134.0' (6:00)	
135	R2	60/60	134.0 - 139.0	RQD = 54%						Hard to very hard, fresh, white to gray, medium grained to aphanitic GNEISS. Joints are low angle to moderately dipping, very close to moderately spaced, planar and stepped, smooth to rough, fresh to discolored, tight to partly open, some silt infilling Rock Mass Quality=Fair -BERWICK FORMATION- R2:Core Times (min:sec): 134.0-135.0' (4:00), 135.0-136.0' (4:00), 136.0-137.0' (5:00), 137.0-138.0' (5:00), 138.0-139.0' (5:00)	
								-109.6		139.0 **Bottom of Exploration at 139.0 feet Below Ground Surface.**	
140											
145											
150											
155											

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 37

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Proposed Replacement Bridge Over Presumpscot River and MCRR	**Location:**	Routes 26/100 Falmouth, Maine	**Boring No.:**	BB-FRR-101
				PIN:	15094.00
Driver:	Maine Test Borings, Inc.	**Elevation (ft.)**	50.7	**Auger ID/OD:**	-
Operator:	D. McKeen	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E.Beirne/M.Snow/B.Steinert	**Rig Type:**	Mobile Drill B-47 Bombardier	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/07/08 to 10/14/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E1005192, N326956 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	Not observed
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/10	0.0 - 2.0	5-15-13-4	28	28	Push			Brown, moist to wet, medium dense, SAND, trace silt, little gravel, well graded, no structure, cobble at approximately 1.3-1.5 ft. -FILL-	
								48.7		2.0	
	2D	24/18	2.0 - 4.0	2-2-3-2	5	5				Gray-brown to gray, wet, medium stiff, silty CLAY, little fine sand, trace coarse to medium sand, no structure, small amount of sandy silt on top of clay, fine sand layers throughout -FILL-(Reworked Natural Soil)	
								46.7		4.0	
	3D	24/19	4.0 - 6.0	2-2-2-2	4	4				Gray-brown to gray, moist to wet, medium stiff, silty CLAY, little fine sand, no structure to somewhat laminated, occasional fine sand layers (CL)	
5							Open				
	4D V1A	24/22	6.0 - 8.0 6.5 - 7.0	push thru vane Su=>880 psf						Gray, wet, medium stiff, silty CLAY, no structure -MARINE DEPOSIT-(CL) 3x6 in. vane raw torque readings: V1A: >600 in-lbs Note: Unable to rotate vane 90 degrees.	
10	5D	24/24	10.0 - 12.0	WOR-WOR-WOH-WOH						Gray-brown, wet, medium stiff, silty CLAY, no structure -MARINE DEPOSIT-(CL)	
15	6D V2A	24/24	15.0 - 17.0 15.5 - 16.0	push thru vane Su=620/130 psf						Gray-brown to gray, wet, medium stiff, silty CLAY, no structure -MARINE DEPOSIT-(CL) 3x6 in. vane raw torque readings: V2A: 419/85 in-lbs	
20	7D V3A	24/24	20.0 - 22.0 20.5 - 21.0	push thru vane Su=630/120 psf						Gray, wet, medium stiff, silty CLAY, no structure -MARINE DEPOSIT-(CL) 3x6 in. vane raw torque readings: V3A: 430/80 in-lbs	
25	8D V4A	24/20	25.0 - 27.0 25.5 - 26.0	push thru vane Su=440/90 psf			Open			Gray, wet, soft, silty CLAY, no structure -MARINE DEPOSIT-(CL)	

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 38

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge Over Presumpscot River and MCRR	**Location:** Routes 26/100 Falmouth, Maine	**Boring No.:** BB-FRR-101	**PIN:** 15094.00

Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	50.7	**Auger ID/OD:**	-
Operator:	D. McKeen	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E.Beirne/M.Snow/B.Steinert	**Rig Type:**	Mobile Drill B-47 Bombardier	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/07/08 to 10/14/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E1005192, N326956 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	Not observed
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
										3x6 in. vane raw torque readings: V4A: 300/60 in-lbs	
30	9D V5A	24/21	30.0 - 32.0 30.5 - 31.0	push thru vane Su=520/130 psf						Gray, wet, medium stiff, silty CLAY, no structure -MARINE DEPOSIT-(CL) 3x6 in. vane raw torque readings: V5A: 350/40 in-lbs	
35	10D V6A	24/23	35.0 - 37.0 35.5 - 36.0	push thru vane Su=640/100 psf						Gray, wet, medium stiff, silty CLAY, no structure -MARINE DEPOSIT-(CL) 3x6 in. vane raw torque readings: V6A: 435/70 in-lbs	
40	11D V7A	24/24	40.0 - 42.0 40.5 - 41.0	push thru vane Su=170/110 psf						Gray to dark gray, wet, medium stiff, silty CLAY, no structure, organic odor, black organic streaks -MARINE DEPOSIT-(CL) 3x6 in. vane raw torque readings: V7A: 115/75 in-lbs Note: Vane pushed with hydraulic pressure	
45	12D V8A	24/24	45.0 - 47.0 45.5 - 46.0	push thru vane Su=560/40 psf						Gray, wet, medium stiff, silty CLAY, no structure, black organic streaks throughout, organic odor -MARINE DEPOSIT-(CL) 3x6 in. vane raw torque readings: V8A: 380/30 in-lbs	
50	13D V9A	24/24	50.0 - 52.0 50.5 - 51.0	push thru vane Su=520/50 psf			Open			Gray, wet, medium stiff, silty CLAY, no structure, micaceous -MARINE DEPOSIT-(CL) 3x6 in. vane raw torque readings: V9A: 350/35 in-lbs	

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 39

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Proposed Replacement Bridge Over Presumpscot River and MCRR			**Boring No.:**	BB-FRR-101
Location:	Routes 26/100 Falmouth, Maine			**PIN:**	15094.00
Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	50.7	**Auger ID/OD:**	-
Operator:	D. McKeen	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E.Beirne/M.Snow/B.Steinert	**Rig Type:**	Mobile Drill B-47 Bombardier	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/07/08 to 10/14/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E1005192, N326956 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	Not observed
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
55	14D ~~V10A~~	24/18	55.0 - 57.0 ~~55.5 - 56.0~~	push thru vane ~~Su=810/40 psf~~						Gray, wet, medium stiff, silty CLAY, trace medium to fine sand, no structure -MARINE DEPOSIT-(CL) 3x6 in. vane raw torque readings: V10A: 55/25 in-lbs	
60	15D ~~V11A~~	24/19	60.0 - 62.0 ~~60.5 - 61.0~~	push thru vane ~~Su=630/40 psf~~						Gray, wet, medium stiff, silty CLAY, trace medium to fine sand, no structure -MARINE DEPOSIT-(CL) 3x6 in. vane raw torque readings: V11A: 430/25 in-lbs	
65	16D ~~V12A~~	24/24	65.0 - 67.0 ~~65.5 - 66.0~~	push thru vane ~~Su=650/30 psf~~						Gray, wet, medium stiff, silty CLAY, trace fine sand, no structure -MARINE DEPOSIT- 3x6 in. vane raw torque readings: V12A: 445/20 in-lbs	
70	17D ~~V13A~~	24/18	70.0 - 72.0 ~~70.5 - 71.0~~	push thru vane ~~Su=880/40 psf~~						Gray, wet, medium stiff, silty CLAY, trace fine sand, no structure -MARINE DEPOSIT-(CL) 3x6 in. vane raw torque readings: V13A: >600/30 in-lbs	
75	18D ~~V14A~~	24/24	75.0 - 77.0 ~~75.5 - 76.0~~	push thru vane ~~Su=840/70 psf~~			Open			Gray, wet, medium stiff, silty CLAY, little fine sand, trace medium sand, no structure, sandy clay layer from 76.0 to 76.5 ft (approximate) -MARINE DEPOSIT-(CL) 3x6 in. vane raw torque readings: V14A: 570/50 in-lbs	

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 40

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	50.7	**Auger ID/OD:**	-
Operator:	D. McKeen	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E.Beirne/M.Snow/B.Steinert	**Rig Type:**	Mobile Drill B-47 Bombardier	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/07/08 to 10/14/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E1005192, N326956 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	Not observed
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
80	19D MV	24/24	80.0 - 82.0	3-WOH-1-7	1	1				Gray, wet, medium stiff, sandy CLAY, alternating between sand and clay layers, few pieces of gravel at top of spoon -MARINE DEPOSIT-(CL) Note: Attempted vane at 80.0 ft, unable to push.	
								-32.3		83.0	
85	20D	24/12	85.0 - 87.0	14-17-27-37	44	44				Note: Attempted vane shear test at 85.0 ft, unable to push vane. Gray to light gray, wet, dense, SAND, trace clay, poorly graded, no structure -MARINE DEPOSIT-(SP-SC)	
90	21D	24/13	90.0 - 92.0	14-19-27-37	46	46	Wash Ahead			Gray, wet, dense, SAND, trace silt, poorly graded, no structure -MARINE DEPOSIT-(SP)	
95	22D	24/16	95.0 - 97.0	20-22-25-34	47	47				Gray, wet, dense, SAND, trace silt, trace fine gravel, poorly graded, no structure -MARINE DEPOSIT-(SP)	
100	23D	24/12	100.0 - 102.0	11-15-19-21	34	34	Wash Ahead			Gray, wet, dense, SAND, trace silt, poorly graded, no structure -MARINE DEPOSIT-(SP)	

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 41

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge Over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FRR-101
PIN: 15094.00

Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	50.7	**Auger ID/OD:**	-
Operator:	D. McKeen	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E.Beirne/M.Snow/B.Steinert	**Rig Type:**	Mobile Drill B-47 Bombardier	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/07/08 to 10/14/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E1005192, N326956 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	Not observed
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
105	24D	24/4	105.0 - 107.0	14-16-18-16	34	34	50			Gray, wet, dense, SAND, trace silt, poorly graded, no structure -MARINE DEPOSIT-(SP)	
							25				
							21				
							19				
							23				
110	MD	24/0	110.0 - 112.0	11-12-12-11	24	24	20			No Recovery	
							14				
							17				
							21			Note: Encountered gravel or cobble at approximately 112.8 ft and an obstruction from 114.7 to 115.0 ft (approximate).	
							17				
115	26D	24/9	115.0 - 117.0	11-12-12-8	24	24	40			Gray, wet, medium dense, SAND, trace fine gravel, poorly graded, no structure -MARINE DEPOSIT-(SP)	
							68				
							75				
							44				
							80				
120	27D	24/8	120.0 - 122.0	9-7-11-11	18	18	77			Gray, wet, medium dense, SAND, trace fine gravel, poorly graded, no structure, large piece of gravel in tip of sampler -MARINE DEPOSIT-(SP)	
							87				
							72				
							74				
							84				
125	28D	24/3	125.0 - 127.0	6-6-7-6	13	13	30			Gray, wet, medium dense, SAND, little fine to coarse gravel, well graded, no structure -MARINE DEPOSIT-(SW)	
							28				
							66				
							93				
							105				
130										Gray, wet, loose, SAND, trace fine gravel, well graded, no structure	

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 42

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Proposed Replacement Bridge Over Presumpscot River and MCRR **Location:** Routes 26/100 Falmouth, Maine	**Boring No.:** BB-FRR-101 **PIN:** 15094.00

Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	50.7	**Auger ID/OD:**	-
Operator:	D. McKeen	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E.Beirne/M.Snow/B.Steinert	**Rig Type:**	Mobile Drill B-47 Bombardier	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/07/08 to 10/14/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E1005192, N326956 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	Not observed
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	29D	24/3	130.0 - 132.0	5-3-4-5	7	7	7			-MARINE DEPOSIT-(SW)	
							10				
							14				
							15				
	30D	24/12	134.0 - 136.0	11-11-13-20	24	24	Push			Gray, wet, medium dense, SAND, well graded, no structure -MARINE DEPOSIT-(SW)	
135											
							7				
							2				
	31D	24/14	139.0 - 141.0	11-18-23-35	41	41	18			Gray, wet, dense, SAND, little coarse to fine gravel, well graded, no structure, glacial till in tip of sampler -MARINE DEPOSIT-(SW)	
140							49				
								-90.2		140.9	
							46				
							48				
							4				
	32D	24/14	144.0 - 146.0	31-9-10-19	19	19	35			Gray, wet, medium dense, SAND, trace fine gravel, well graded, no structure -GLACIAL TILL-(SW)	
145							75				
							106				
							100				
							50				
	MD	24/0	149.0 - 151.0	35-35-34-25	69	69	58			No Recovery, pushing on gravel with sampler	
150							63				
							66				
							60				
							62				
								-103.3		154.0	
	34D	24/8	154.0 - 156.0	33-40-22-15	62	62	75			Gray to light gray-brown, wet, very dense, well graded, SAND, some gravel, trace silt, slightly bonded -GLACIAL TILL-(SW-GW)	
155							63				
							68				

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 43

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge Over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FRR-101
PIN: 15094.00

Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	50.7	**Auger ID/OD:**	-
Operator:	D. McKeen	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	E.Beirne/M.Snow/B.Steinert	**Rig Type:**	Mobile Drill B-47 Bombardier	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/07/08 to 10/14/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E1005192, N326956 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	Not observed
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							60			Note: Occasional cobbles encountered between 158.0 and 164.0 ft.	
							60				
							57				
160							55				
							90				
							105				
							130				
	35D	24/9	164.0 - 166.0	44-36-28-24	64	64	70			Gray, wet, very dense, well graded sandy GRAVEL, trace silt, some small portions of sample are bonded -GLACIAL TILL-(GW)	
165							107				
							140				
							200(0.7)				
								-118.1		168.8	
										Top of Bedrock at El.-118.1	
170	R1	60/48	170.1 - 175.1	RQD = 43%			NQ			Dark gray to gray, aphanitic to fine grained, metatmorphic, SCHIST, hard, fresh to slightly weathered. Joints dipping at low to moderate angles, very close to close, tight to open, pyrite observed on some joint surfaces. 2 in. quartz vein at approximately 171.9 ft, occasional calcite veins. Pitting observed from approximately 170.1 to 171.2 ft. Rock Mass Quality=Poor -BERWICK FORMATION- R1:Core Times (min:sec): 170.1-171.1' (4:00), 171.1-172.1' (5:00), 172.1-173.1' (5:00), 173.1-174.1' (4:00), 174.1-175.1' (5:00)	
175	R2	58/58	175.1 - 179.9	RQD = 67%			NQ			Dark gray to gray, aphanitic to fine grained, metamorphic, SCHIST, hard, fresh to slightly weathered. Primary joints dipping at low to moderate angles, very close to wide, tight to open, frequent thin calcite veins, calcite coating observed on one joint surface. One secondary high angle joint perpendicular to foliation. Highly fractured zone from approximately 176.2 to 176.5 ft. Rock Mass Quality=Fair -BERWICK FORMATION- R2:Core Times (min:sec): 175.1-176.1' (4:00), 176.1-177.1' (4:00), 177.1-178.1' (4:00), 178.1-179.1' (4:00), 179.1-179.9' (4:00)	
180								-129.2		179.9	
										Bottom of Exploration at 179.9 feet Below Ground Surface.	

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 44

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Proposed Replacement Bridge Over Presumpscot River and MCRR	**Boring No.:**	BB-FRR-102
Location:	Routes 26/100 Falmouth, Maine	**PIN:**	15094.00

Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	31.3	**Auger ID/OD:**	-
Operator:	B. Enos/D. McKeen	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	B. Steinert/E. Beirne	**Rig Type:**	Mobile Drill B-47 Bombardier	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/29/08 to 11/06/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E1005237, N326767 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	19.5 (11/7/08, 0830)
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							Push			Note: Split spoon refusal on probable existing pile cap at 2.0 ft below ground surface. Relocate borehole approximately 5 ft south. Advance probe auger to 4.0 ft below ground surface to confirm no obstruction and begin sampling.	
	1D	24/19	4.0 - 6.0	1-1-2-5	3	3				Dark brown to black, moist to wet, soft, SILT (ML), trace medium sand, little fine sand, highly organic	
5							60	26.3		-FILL-(Reworked Natural Soil) 5.0	
							90			Gray brown, moist, medium stiff, SILT (ML), little fine sand, slight mottling, trace organics -MARINE DEPOSIT-	
							130	24.3		7.0 Gray brown, moist, very stiff, lean CLAY -MARINE DEPOSIT-(CL)	
							132				
							150				
10	2D	24/24	10.0 - 12.0	6-9-11-16	20	20	Open			Olive gray, moist, very stiff, lean CLAY -MARINE DEPOSIT-(CL)	
15	3D	24/24	15.0 - 17.0	3-3-5-5	8	8				Olive gray, wet, medium stiff, lean CLAY -MARINE DEPOSIT-(CL)	
20	V1A		20.5 - 21.0	Su=>880 psf						3x6 in. vane raw torque readings: V1A: >600/120 in-lbs	
25							Open				

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 45

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge Over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FRR-102
PIN: 15094.00

Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	31.3	**Auger ID/OD:**	-
Operator:	B. Enos/D. McKeen	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	B. Steinert/E. Beirne	**Rig Type:**	Mobile Drill B-47 Bombardier	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/29/08 to 11/06/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E1005237, N326767 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	19.5 (11/7/08, 0830)
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
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MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
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V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
30	V2A		30.5 - 31.0	Su=590/150 psf						3x6 in. vane raw torque readings: V2A: 400/100 in-lbs	
	1U	24/24	32.0 - 34.0								C#CRC-2 WC=40.3% LL=32 PL=18 PI=14
35	V3A		34.5 - 35.0	Su=540/100 psf						3x6 in. vane raw torque readings: V3A: 365/70 in-lbs	
40	V4A		40.5 - 41.0	Su=350/110 psf						3x6 in. vane raw torque readings: V4A: 235/75 in-lbs	
										Note: Dropstones encountered at approximately 44.5 ft.	
45	4D MV	24/18	45.0 - 47.0 45.5 - 46.0	push thru vane						Gray, wet, medium stiff, lean CLAY -MARINE DEPOSIT-(CL) Note: Attempted vane at 45 ft, unable to push. Note: Coarse sand present in sample.	
	V5A		48.5 - 49.0	Su=610/60 psf						3x6 in. vane raw torque readings: V5A: 415/40 in-lbs	
	MU		49.5 - 51.5								
50							Open				
	MU		51.5 - 53.5								

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 46

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Proposed Replacement Bridge Over Presumpscot River and MCRR Location: Routes 26/100 Falmouth, Maine	Boring No.: BB-FRR-102 PIN: 15094.00

Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	31.3	**Auger ID/OD:**	-
Operator:	B. Enos/D. McKeen	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	B. Steinert/E. Beirne	**Rig Type:**	Mobile Drill B-47 Bombardier	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/29/08 to 11/06/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E1005237, N326767 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	19.5 (11/7/08, 0830)
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
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V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	2U	24/23	54.0 - 56.0								C#CRC-4 WC=33.1% LL=23 PL=14 PI=9
55											
	V6A		56.5 - 57.0	Su=870/120 psf						3x6 in. vane raw torque readings: V6A: 590/80 in-lbs	
								-27.2		58.5 Note: Unable to advance vane due to drop stones/sand layers.	
60	5D	24/12	60.0 - 62.0	10-11-14-21	25	25	1			Gray, wet, saturated, medium dense, SAND, trace silt, poorly graded, no structure -MARINE DEPOSIT-(SP)	
							21				
							44				
							45				
							36				
65							48				
							58				
							64				
							64				
							54				
70	6D	24/17	70.0 - 72.0	3-3-1-1	4	4	22			Gray, wet, saturated, very loose, SAND, trace silt, poorly graded, no structure, sandy CLAY with layers of poorly graded SAND below 71.0 ft -MARINE DEPOSIT-(SP)	
							19				
							25				
							53				
75							61				
							51				
							55				
							70				

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 47

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge Over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FRR-102
PIN: 15094.00

Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	31.3	**Auger ID/OD:**	-
Operator:	B. Enos/D. McKeen	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	B. Steinert/E. Beirne	**Rig Type:**	Mobile Drill B-47 Bombardier	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/29/08 to 11/06/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E1005237, N326767 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	19.5 (11/7/08, 0830)
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
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V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							71				
							76				
80	7D	24/14	80.0 - 82.0	5-5-8-8	13	13	44			Gray, wet, medium dense, SAND, trace fine gravel, well graded, no structure, coarser with depth -MARINE DEPOSIT-(SP)	
							45				
							88				
							85				
							88				
85							91				
							97				
							82				
							80				
							96				
90	MD	24/0	90.0 - 92.0	7-10-17-18	27	27	54			Note: Wash water indicates medium to fine SAND, becoming coarse at approximately 95.0 ft.	
							59				
							91				
							113				
							124				
95							119				
							157				
							220				
							287				
							300				
100	9D	24/14	100.0 - 102.0	15-24-32-32	56	56	Wash Ahead			Gray, wet, very dense, SAND, trace silt, poorly graded, no structure -MARINE DEPOSIT-(SP)	

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 48

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Proposed Replacement Bridge Over Presumpscot River and MCRR **Location:** Routes 26/100 Falmouth, Maine	**Boring No.:** BB-FRR-102 **PIN:** 15094.00

Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	31.3	**Auger ID/OD:**	-
Operator:	B. Enos/D. McKeen	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	B. Steinert/E. Beirne	**Rig Type:**	Mobile Drill B-47 Bombardier	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/29/08 to 11/06/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E1005237, N326767 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	19.5 (11/7/08, 0830)
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
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V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
105								-73.7		105.0	
110	10D	24/13	110.0 - 112.0	23-22-22-29	44	44				Gray, wet, dense, SAND, trace fine gravel, well graded, no structure -GLACIAL TILL-(SW)	
								-82.2		113.5 Note: Began to encounter gravel and cobbles at 113.5 ft.	
115											
										Note: Cobble from approximately 117.8 to 118.3 ft.	
120	11D	24/18	120.0 - 122.0	25-24-42-39	66	66				Gray, wet to saturated, very dense, SAND, little silt, poorly graded, no structure -GLACIAL TILL-(SP)	
125							Wash Ahead				
								-97.2		128.5 Note: Driller noted change in drill action at approximately 128.5 ft, probable weathered bedrock.	
130								-98.7		130.0 Top of Bedrock at El.-98.7	

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 49

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge Over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FRR-102
PIN: 15094.00

Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	31.3	**Auger ID/OD:**	-
Operator:	B. Enos/D. McKeen	**Datum:**	NAVD 88	**Sampler:**	Split Spoon - 1.375in. I.D.
Logged By:	B. Steinert/E. Beirne	**Rig Type:**	Mobile Drill B-47 Bombardier	**Hammer Wt./Fall:**	S-140/30 - C-300/16
Date Start/Finish:	10/29/08 to 11/06/08	**Drilling Method:**	Drive/Wash	**Core Barrel:**	NQ - 2.0 I.D.
Boring Location:	E1005237, N326767 (See Plan)	**Casing ID/OD:**	HW/NW - 4.0/3.0	**Water Level*:**	19.5 (11/7/08, 0830)
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
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WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	MD	2/0	130.0 - 130.2	50/3"							
	R1	62/61	131.3 - 136.5	RQD = 66%			NQ			Soft to hard, fresh to moderately weathered, dark gray, fine grained to aphanitic, GNEISS. Joints are moderately dipping to horizontal, very close to moderately spaced, planar and stepped, smooth to rough, fresh to disintegrated, tight to open. Highly fractured and weathered from approximately 131.3 to 132.3 ft. Primary joint set along micaceous zones. Rock Mass Quality=Fair -BERWICK FORMATION- R1:Core Times (min:sec): 131.3-132.3' (4:00), 132.3-133.3' (3:00), 133.3-134.3' (3:00), 134.3-135.3' (3:00, 135. 3-136.5' (3:00)	
135											
	R2	58/29	136.5 - 141.3	RQD = 50%						Hard, fresh, fine grained to aphanitic, GNEISS. Joints are high angle to moderately dipping, close to moderately spaced, planar and undulating, smooth to rough, fresh to disintegrated, very tight to open. Primary joint set along highly micaceous zones. Rock Mass Quality=Poor -BERWICK FORMATION- R2:Core Times (min:sec): 136.5-137.5' (2:00), 137.5-138.5' (2:00), 138.5-139.5' (2:00), 139.5-140.5' (2:00), 140.5-141.3' (1:00)	
140											
								-110.0		141.3 **Bottom of Exploration at 141.3 feet Below Ground Surface.**	
145											
150											
155											

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 50

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Proposed Replacement Bridge over Presumpscot River and MCRR	**Boring No.:**	BB-FPR-201
Location:	Routes 26/100 Falmouth, Maine	**PIN:**	15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	51.0	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 NW
Date Start/Finish:	4/28/09	**Drilling Method:**	NW Drive to 24.0 ft	**Core Barrel:**	--
Boring Location:	E1005421, N326022 (See Plan)	**Casing ID/OD:**	NW - 3.0 in. I.D.	**Water Level*:**	Dry (4/28/09, 1230)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/15	0.5 - 2.5	10/21/24/22	45	45	15	50.6		-BITUMINOUS CONCRETE- 0.4	
							29			Brown, moist to dry, dense, medium to fine SAND, some gravel, trace silt, well graded, increasing percentage of gravel with depth -FILL-(SW) (Base/Subbase)	
	2D	24/5	2.5 - 4.5	66/68/66/63	134	134	111			Brown, dry, very dense, coarse to fine SAND, well graded, pushing gravel -FILL-(SW) (Base/Subbase)	
							80				
	3D	24/17	4.5 - 6.5	14/10/7/8	17	17	1	46.5		Note: Washed ahead of casing from 4.5 to 10.5 ft. 4.5	
5							2			Gray-brown, moist, medium dense, silty coarse to fine SAND, little gravel, occasional organics -MARINE DEPOSIT-(SM)	
	4D	24/19	6.5 - 8.5	3/4/4/4	8	8	WOH			Gray-brown, moist, medium stiff, sandy SILT, little gravel, organics present at approximately 6.7 ft (ML)	
							3			Note: Accidentally overdrove casing by 1.0 ft, sampled inside the casing from 8.5 to 9.5 ft.	
	5D	24/16	8.5 - 10.5	32/56/44/25	100	100	46			Brown, wet, very dense, silty coarse to fine SAND, trace gravel (SM)	
							37				
10	6D	24/13	10.5 - 12.5	9/5/4/7	9	9	10	41.0 / 40.5		10.0 Gray, moist to dry, very dense, GRAVEL, poorly graded (GP) 10.5	
							16	39.7		Gray-brown, wet, loose, silty, coarse to fine SAND, trace gravel (SM) 11.3	
	7D	24/12	12.5 - 14.5	4/4/6/8	10	10	10	38.7		Olive-brown, moist, stiff, silty CLAY, trace medium to fine sand (CL) 12.3 Brown, wet, loose, fine SAND, trace medium sand, poorly graded (SP)	
							17				
	8D	24/6	14.5 - 16.5	8/17/22/25	39	39	8			Brown, wet, dense, coarse to fine SAND, some silt and gravel, well graded, iron oxidized layers (SW)	
15							83			Note: Cobble at bottom of casing, washed through before sample.	
	9D	24/15	16.5 - 18.5	17/15/20/13	35	35	31			Gray-brown to brown, mottled, wet, dense, coarse to fine SAND, little gravel, trace silt, well graded -MARINE DEPOSIT-(SW)	
							43				
	10D	24/11	18.5 - 20.5	5/6/6/8	12	12	13	32.5		18.5 Brown, wet, medium dense, SAND, little gravel, well graded -MARINE DEPOSIT-(SW)	
							10				
20	11D	24/16	20.5 - 22.5	6/7/8/9	15	15	18			Brown, wet, medium dense, SAND, little gravel, well graded (SW)	
							20	29.6		21.4 Brown to gray-brown, moist to wet, medium dense, fine SAND, little medium sand, little silt, trace coarse sand, poorly graded, one silt layer from approximately 22.1 to 22.2 ft (SP)	
	12D	24/19	22.5 - 24.5	5/9/12/13	21	21	21			Brown, wet, medium dense, fine SAND, trace silt, poorly graded, coarser with depth (SP)	
							46				
	13D	24/21	24.5 - 26.5	22/16/24/23	40	40		26.9		24.1 Gray-brown, moist, medium dense, silty coarse to fine SAND, little gravel, bonded -GLACIAL TILL-(SW-SM)	
25										Gray-brown, wet, dense, coarse to fine SAND, some silt, little gravel,	

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 51

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Proposed Replacement Bridge over Presumpscot River and MCRR **Location:** Routes 26/100 Falmouth, Maine	**Boring No.:** BB-FPR-201 **PIN:** 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	51.0	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 NW
Date Start/Finish:	4/28/09	**Drilling Method:**	NW Drive to 24.0 ft	**Core Barrel:**	--
Boring Location:	E1005421, N326022 (See Plan)	**Casing ID/OD:**	NW - 3.0 in. I.D.	**Water Level*:**	Dry (4/28/09, 1230)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
								24.5		frequent coarse to medium sand layers -GLACIAL TILL-(SW-SM)	
										26.5	
										Bottom of Exploration at 26.5 feet Below Ground Surface.	
										Note: No Refusal Encountered	
30											
35											
40											
45											
50											

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 52

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Proposed Replacement Bridge over Presumpscot River and MCRR	**Location:**	Routes 26/100 Falmouth, Maine	**Boring No.:**	BB-FPR-202
				PIN:	15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	32.9	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	B. Steinert	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 HW
Date Start/Finish:	4/15/09	**Drilling Method:**	HW Drive to 14.0 ft	**Core Barrel:**	--
Boring Location:	E1005456, N326125 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/6	0.0 - 2.0	1/1/2/1	3	3	Push			Dark brown, wet, very loose, medium to fine SAND, little coarse sand, little silt, roots -FILL-(SP-SM)	
	2D	24/4	2.0 - 4.0	2/1/2/3	3	3				Gray-brown, wet, very loose, silty medium to fine SAND, trace coarse sand, organic odor, trace organics (SP-SM)	
								28.9		4.0	
5	3D	24/24	4.0 - 6.0	2/3/2/1	5	5				Gray, wet, loose, silty fine SAND, little coarse to medium sand, to fine sandy SILT, trace coarse to medium sand, layered, poorly graded, organics throughout, dark brown organics in tip of spoon -ALLUVIAL DEPOSIT-(SP-SM)	
								26.9		6.0	
	4D	24/17	6.0 - 8.0	1/2/3/10	5	5		26.4		Dark brown, wet, medium stiff, organic SILT (OH) 6.5	
								25.4		Loose, gray-brown, wet, medium stiff, SILT, trace coarse to medium sand, dark brown organics throughout (ML) 7.5	
	5D	24/16	8.0 - 10.0	17/9/8/10	17	17	12	24.4		Loose, brown, wet, loose, medium to fine SAND, little silt, little coarse sand, little coarse gravel, well graded (SW) 8.5	
10							HW Drive	22.9		Gray-brown, wet, medium dense, fine SAND, trace medium sand, trace silt, poorly graded (SP) 10.0	
	6D	24/14	10.0 - 12.0	8/6/5/6	11	11		22.4		Gray-brown, wet, medium dense, clayey SAND (SC) 10.5	
	7D	24/14	12.0 - 14.0	7/6/22/14	28	28		19.9		Gray-brown, wet, medium dense, fine SAND, trace medium sand, trace silt, poorly graded -ALLUVIAL DEPOSIT-(SP) 13.0	
15	8D	24/12	14.0 - 16.0	7/10/8/10	18	18				Gray-brown, moist to wet, medium dense, medium to fine SAND, some silt, little coarse sand, little coarse to fine gravel, well graded, mottled -GLACIAL TILL-(SW-SM)	
								16.9		Gray, moist to wet, medium dense, medium to fine SAND, little coarse sand, little silt, little coarse to fine gravel, well graded -GLACIAL TILL-(SW) 16.0	
										Bottom of Exploration at 16.0 feet Below Ground Surface.	
										Note: No Refusal Encountered	
20											
25											

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 53

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Proposed Replacement Bridge over Presumpscot River and MCRR	**Location:**	Routes 26/100 Falmouth, Maine	**Boring No.:**	BB-FPR-203
				PIN:	15094.00
Driller:	Maine Test Borings	**Elevation (ft.)**	31.2	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 HW
Date Start/Finish:	4/14/09	**Drilling Method:**	HW Drive to 18.0 ft	**Core Barrel:**	--
Boring Location:	E1005495, N326168 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	3.9 (4/14/09, 1550)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/16	0.0 - 2.0	1/3/2/1	5	5	PUSH			Dark brown, moist, medium stiff, sandy SILT, with rootlets -TOPSOIL-(ML)	
								29.2		2.0	
	2D	24/13	2.0 - 4.0	2/4/14/38	18	18				Gray-brown, mottled, moist to wet, medium dense, fine SAND, little silt, poorly graded -ALLUVIAL DEPOSIT-(SP)	
							110				
	3D	24/12	4.0 - 6.0	9/17/13/10	30	30	48	26.7		4.5	
5							27			Gray-brown to black, mottled, wet, medium dense, medium to fine SAND, little coarse sand, little silt, trace fine gravel, well graded (SW)	
								25.2		6.0	
	4D	24/14	6.0 - 8.0	7/7/6/6	13	13	12			Brown to gray, mottled to approximately 7.2 ft, wet, medium dense, fine SAND, little medium sand, trace silt, poorly graded, one gray silt layer at approximately 6.2 ft (SP)	
							15				
	5D	24/13	8.0 - 10.0	6/3/2/4	5	5	9			Brown and gray, mottled, wet, loose, fine SAND, little silt, poorly graded, occasional silt layers from approximately 9.6 to 10.0 ft (SP)	
							8				
10	6D	24/12	10.0 - 12.0	3/2/2/4	4	4	9			Brown, wet, very loose, fine SAND, little medium sand, trace silt, poorly graded -ALLUVIAL DEPOSIT-(SP)	
							12				
	7D	24/14	12.0 - 14.0	6/4/3/3	7	7	14			Gray-brown, wet, loose, fine SAND, little medium sand, little silt, poorly graded (SP)	
							10				
	8D	24/17	14.0 - 16.0	3/2/3/3	5	5	10			Brown, wet, loose, fine SAND, some medium sand, trace silt, poorly graded (SP)	
15							16	16.0		15.2 Gray, wet, loose, coarse to fine SAND, some silt, little gravel, well graded -GLACIAL TILL-(SW-SM)	
	9D	24/5	16.0 - 18.0	9/16/19/22	35	35	32			Gray, wet, dense, coarse to fine SAND, some gravel, little silt, well graded (SW)	
							31				
	10D	24/16	18.0 - 20.0	12/12/12/12	24	24				Gray, moist to wet, medium dense, medium to fine SAND, some coarse sand, some gravel, little silt, well graded -GLACIAL TILL-(SW)	
20								11.2		20.0 **Bottom of Exploration at 20.0 feet Below Ground Surface.**	
										Note: No Refusal Encountered	
25											

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 54

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Boring No.: BB-FPR-203

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FPR-204
PIN: 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	28.1	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne	**Rig Type:**	Mobile B47 Bombardier	**Hammer Wt./Fall:**	140/30 SS - 300/16 HW
Date Start/Finish:	5/22/09	**Drilling Method:**	HW Drive to 16.0 ft	**Core Barrel:**	--
Boring Location:	E1005332, N326234 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/8	0.0 - 2.0	1/2/3/2	5	5	HW Push			Dark brown, moist, loose, silty coarse to fine SAND, trace fine gravel -FILL-(SM)	
	2D	24/4	2.0 - 4.0	3/3/3/9	6	6				Dark brown, moist, loose, silty coarse to fine SAND, trace fine gravel (SM)	
							27	24.5		3.6	
	3D	24/12	4.0 - 6.0	3/2/2/2	4	4	38			Olive-brown, moist, medium stiff, SILT, some gravel, little coarse to fine sand -FILL-(ML)	
5							32			Brown to gray-brown, wet, very loose, silty fine SAND, little medium sand, trace gravel and coarse sand (SM)	
	4D	24/5	6.0 - 8.0	1/3/2/8	5	5	34			Brown to dark brown, mottled, wet, loose, silty fine SAND, little coarse to fine gravel and medium sand, trace coarse sand (SM)	
							117	20.1		8.0	
	5D	24/11	8.0 - 10.0	18/25/17/8	42	42	9			Olive-brown, wet, hard, SILT, little medium to fine sand, gravel at top of spoon, mottled, occasional organics -ALLUVIAL DEPOSIT-(ML)	
							44				
10	6D	24/4	10.0 - 12.0	5/6/5/3	11	11	34			Olive-brown, wet, stiff, SILT, little medium to fine sand, trace gravel (ML)	
							31	16.6		11.5	
	7D	24/13	12.0 - 14.0	4/1/WOH/WOH	1	1	43			Brown to gray, saturated, very loose, silty fine SAND, trace medium sand, color change at approximately 13.0 ft to gray -ALLUVIAL DEPOSIT-(SM)	
							20				
	8D	24/13	14.0 - 16.0	2/1/4/1	5	5	48			Gray, saturated, loose, fine SAND, some silt, poorly graded (SP)	
15							39				
	9D	24/19	16.0 - 18.0	3/1/3/2	4	4	Open			Gray-brown, saturated, very loose, fine SAND, some silt, poorly graded, organics between 16.7 to 18.0 ft (SP)	
								10.1		18.0	
	10D	24/20	18.0 - 20.0	WOH/WOH/WOH/2						Gray, wet, very soft, silty CLAY, trace fine sand -MARINE DEPOSIT-(CL)	
20											
	11D	24/24	24.0 - 26.0	WOR/WO1P/WO1P/ WO1P						Gray, wet, very soft, silty CLAY, black organic streaking throughout (CL)	
25							Open				

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 55

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Proposed Replacement Bridge over Presumpscot River and MCRR **Location:** Routes 26/100 Falmouth, Maine	**Boring No.:** BB-FPR-204 **PIN:** 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	28.1	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne	**Rig Type:**	Mobile B47 Bombardier	**Hammer Wt./Fall:**	140/30 SS - 300/16 HW
Date Start/Finish:	5/22/09	**Drilling Method:**	HW Drive to 16.0 ft	**Core Barrel:**	--
Boring Location:	E1005332, N326234 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
30	12D	24/24	29.0 - 31.0	WOH/WOH/WOH/WOH						Gray, wet, very soft, silty CLAY, little medium to fine sand, sand seams below 30.5 ft (CL)	
35	13D	24/24	34.0 - 36.0	WOH/WOH/WOH/WOH						Gray, wet, very soft, silty CLAY, little fine sand, occasional fine sand partings -MARINE DEPOSIT- (CL)	
								-8.9		37.0 Note: Driller noted change in density of material from drill action at 37.0 ft.	
40	14D	24/13	39.0 - 41.0	19/16/19/19	35	35				Brown, wet, dense, coarse to fine SAND, little coarse to fine gravel, trace silt, well graded, somewhat bonded -GLACIAL TILL-(SW)	
								-12.9		41.0 **Bottom of Exploration at 41.0 feet Below Ground Surface.** Note: No Refusal Encountered	
45											
50											

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 56

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Proposed Replacement Bridge over Presumpscot River and MCRR Location: Routes 26/100 Falmouth, Maine	Boring No.: BB-FPR-205 PIN: 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	27.9	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	B.Steinert/E.Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 NW
Date Start/Finish:	4/16/09 to 4/27/09	**Drilling Method:**	HW Drive to 75.0 ft/NW Drive to 125.5 ft	**Core Barrel:**	NQ - 2.0 in. I.D.
Boring Location:	E1005412, N326283 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	12.5 (4/27/09, 1345)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/6	0.0 - 2.0	1/1/2/3	3	3	Push			Dark brown, moist, soft, fine sandy SILT with organics, roots -TOPSOIL-(OL)	
								26.9		1.0	
	2D	24/16	2.0 - 4.0	2/2/2/3	4	4				Tan to yellow-brown, moist, very loose, fine SAND, little silt, poorly graded, frequent dark brown organics -ALLUVIAL DEPOSIT-(SP)	
	3D	24/20	4.0 - 6.0	2/2/1/1	3	3				Tan to yellow-brown, moist to wet, very loose, fine SAND, little clay, occasional dark brown organics (SM)	
5											
	4D	24/24	6.0 - 8.0	2/1/1/1	2	2				Tan to yellow-brown, wet, very loose, fine SAND, little silt, trace clay, occasional dark brown organics (SM)	
								19.9		8.0	
	5D	24/15	8.0 - 10.0	WOR/WOR/1/1	1	1				Olive-gray, wet, very soft, SILT, little fine sand, occasional organics -ALLUVIAL DEPOSIT-(ML)	
10											
	6D	24/20	10.0 - 12.0	2/2/1/1	3	3		17.4		10.5 Gray-brown, wet, mottled, very soft, silty CLAY with organics -ALLUVIAL DEPOSIT-(CL)	
	7D	24/24	12.0 - 14.0	WOH/2/5/5	7	7				Gray-brown, wet, medium stiff, silty CLAY with organics (CL)	
								14.9		13.0 Gray-brown, wet, slightly oxidized, loose, silty fine SAND (SM)	
	8D	24/16	14.0 - 16.0	2/4/5/7	9	9		13.4		14.5 Gray-brown, wet, loose, medium to fine SAND, trace coarse sand, trace silt (SP)	
15											
	9D	24/11	16.0 - 18.0	4/6/3/2	9	9					
								10.9		17.0 Gray-brown, mottled, wet, medium stiff, silty CLAY -MARINE DEPOSIT-(CL)	
	10D	24/24	18.0 - 20.0	WOH/WOH/WOH/1						Gray, wet, very soft, silty CLAY	
20											
25											
	11D	24/24	25.0 - 27.0	push thru vane			Push			Gray, wet, medium stiff, silty CLAY -MARINE DEPOSIT-(CL)	
	~~V1A~~		~~25.6 - 26.0~~	~~Su=755/155 psf~~							

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 57

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FPR-205
PIN: 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	27.9	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	B.Steinert/E.Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 NW
Date Start/Finish:	4/16/09 to 4/27/09	**Drilling Method:**	HW Drive to 75.0 ft/NW Drive to 125.5 ft	**Core Barrel:**	NQ - 2.0 in. I.D.
Boring Location:	E1005412, N326283 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	12.5 (4/27/09, 1345)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	V1B		26.6 - 27.0	Su=775/155 psf						55x110 mm vane raw torque readings: V1A: 195/40 in-lbs V1B: 200/40 in-lbs	
30	1U	24/24	30.0 - 32.0								
	12D	24/24	32.0 - 34.0	push thru vane						Gray to gray-brown, wet, medium stiff, silty CLAY to SILT, little sand (CL-ML)	
	V2A		32.6 - 33.0	Su=875/195 psf						55x110 mm vane raw torque readings: V2A: 225/50 in-lbs	
	MV		33.6 - 34.0				5	-5.6		Note: Unable to push vane beyond 33.0 ft.	
							14			33.5	
35							17			Brown, wet, very loose, silty fine SAND -MARINE DEPOSIT-(SM)	
							30	-8.6		36.5	
							35			Note: Driller noted change in density at approximately 36.5 ft and small cobble at 38.8 ft.	
							105				
							110				
40	13D	24/15	40.0 - 42.0	16/14/12/9	26	26	10			Brown, wet, medium dense, coarse to fine SAND, some silt and gravel, well graded, variable amounts of silt throughout -GLACIAL TILL-(SW-SM)	
							11				
							13				
							12				
							9				
45	14D	24/17	45.0 - 47.0	17/28/25/26	53	53	24	-17.7		Brown, wet, medium dense, coarse to fine SAND, well graded (SW) 45.6	
							27			Gray, moist to wet, very dense, medium to fine SAND, some gravel, little silt, trace coarse sand, poorly graded -GLACIAL TILL-(SP)	
							58				
							120				
							115				
50	15D	24/15	50.0 - 52.0	32/36/32/32	68	68	34			Gray to light gray, wet, very dense, alternating layers of SAND, little gravel and SAND, some gravel, little silt, well graded -GLACIAL TILL-(SW)	
							22				

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 58

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FPR-205
PIN: 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	27.9	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	B.Steinert/E.Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 NW
Date Start/Finish:	4/16/09 to 4/27/09	**Drilling Method:**	HW Drive to 75.0 ft/NW Drive to 125.5 ft	**Core Barrel:**	NQ - 2.0 in. I.D.
Boring Location:	E1005412, N326283 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	12.5 (4/27/09, 1345)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							50				
							52				
							43				
55	16D	24/20	55.0 - 57.0	7/10/18/27	28	28	37			Gray, wet, medium dense, medium to fine SAND, some gravel, little coarse sand, little silt, one layer of poorly graded fine sand from approximately 56.7 to 56.9 ft -GLACIAL TILL-(SW)	
							67				
							107				
							136				
							206				
60	17D	24/19	60.0 - 62.0	23/35/30/37	65	65	119			Gray, wet, very dense, coarse to fine SAND, some gravel, little silt, well graded, one layer of coarse to medium sand from approximately 61.5 to 61.8 ft -GLACIAL TILL-(SW)	
							58				
							102				
							98				
							128				
65	18D	24/12	65.0 - 67.0	13/14/13/13	27	27	HW Washed Ahead			Gray, wet, medium dense, medium to fine SAND, some silt, little gravel, trace fine sand (SM)	
70	19D	13/10	70.0 - 71.1	19/26/50(1")	76	76				Gray, wet, very dense, coarse to fine SAND, some silt, little gravel -GLACIAL TILL-(SW-SM)	
							14				
							126				
							63				
							69				
75	20D	24/24	75.0 - 77.0	26/24/20/20	44	44	12			Gray, wet, dense, coarse to fine SAND, some silt, little gravel -GLACIAL TILL-(SM)	
							32				
							35				

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 59

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Proposed Replacement Bridge over Presumpscot River and MCRR **Location:** Routes 26/100 Falmouth, Maine	**Boring No.:** BB-FPR-205 **PIN:** 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	27.9	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	B.Steinert/E.Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 NW
Date Start/Finish:	4/16/09 to 4/27/09	**Drilling Method:**	HW Drive to 75.0 ft/NW Drive to 125.5 ft	**Core Barrel:**	NQ - 2.0 in. I.D.
Boring Location:	E1005412, N326283 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	12.5 (4/27/09, 1345)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
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MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							47				
							27				
80							6				
							16				
							26				
							145				
85	21D	24/24	84.5 - 86.5	45/14/53/100(5")	67	67	500			Gray, wet, very dense, coarse to fine SAND, trace fine gravel, well graded, occasional silty sand lenses (SW) Note: Hole open to 84.5 ft when sample was taken, after sample was retrieved, hole open to approximately 60 ft.	
							8				
							40			Note: Driller noted change in density at approximately 87.0 ft.	
							240				
							650				
90	22D	24/16	89.5 - 91.5	28/30/30/45	60	60	795/.7' 7/.3'			Gray, wet, very dense, medium to fine SAND, little coarse sand, little silt and gravel, well graded, bonded, one coarse to fine layer from approximately 90.0 to 90.2 ft -GLACIAL TILL-(SW)	
							15				
							20				
							22				
							12				
95	23D	24/19	94.5 - 96.5	15/33/58/100(4")	91	91	13			Gray, wet, very dense, medium to fine SAND, some gravel, little coarse sand, little silt, well graded, bonded -GLACIAL TILL-(SW)	
							13				
							18				
							15				
							13				
100	24D	24/17	99.5 - 101.5	27/56/78/84	134	134	12			Gray, wet, very dense, coarse to fine SAND, trace silt and gravel, well graded (SW)	
							18				
							27	-73.3		101.2 Gray, wet, very dense, medium to fine SAND, little coarse sand, little gravel and silt, bonded -GLACIAL TILL-(SP)	
							89			Note: Washed ahead of casing to 104.5 ft, caved to approximately 100.0 ft after pulling rods.	
							104				

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 60

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge over Presumpscot River and MCRR	**Location:** Routes 26/100 Falmouth, Maine	**Boring No.:** BB-FPR-205	**PIN:** 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	27.9	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	B.Steinert/E.Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 NW
Date Start/Finish:	4/16/09 to 4/27/09	**Drilling Method:**	HW Drive to 75.0 ft/NW Drive to 125.5 ft	**Core Barrel:**	NQ - 2.0 in. I.D.
Boring Location:	E1005412, N326283 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	12.5 (4/27/09, 1345)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	25D	24/19	104.5 - 106.5	44/93/93/94	186	186	137			Gray, wet, very dense, medium to fine SAND, some silt, little gravel, bonded, occasional coarse to fine sand lenses -GLACIAL TILL-(SM)	
105							46				
							45				
							36				
							21				
	26D	24/21	109.5 - 111.5	13/20/27/63	47	47	22			Gray, wet, dense, medium to fine SAND, some silt and clay, little coarse sand, trace gravel, bonded, silty clay layer from approximately 111.2 to 111.5 ft -GLACIAL TILL-(SM)	
110							12				
							19				
							21				
							21				
	27D	24/5	114.5 - 116.5	100(6")			28			Gray, wet, very dense, medium to fine SAND, some silt, little coarse sand, trace gravel, poorly graded, possibly pushing gravel ahead of spoon (SP-SM)	
115							26				
							22				
							18				
							21				
	28D	13/12	119.5 - 120.6	15/25/100(1")	125	125	21	-91.6		119.5 Gray, wet, hard, SILT, little clay, fine sand layers throughout (ML)	
120							100/.4' 100/.1'	-92.7		120.6 Note: NQ core through boulder from 120.6 to 121.7 ft.	
							Washed Ahead	-93.8		121.7	
125	29D	6/6	125.0 - 125.5	100(6")			Washed Ahead			Gray, wet, very dense, silty medium to fine SAND, some gravel, trace coarse sand -GLACIAL TILL-(SM)	
	R1	42/40	126.5 - 130.0	RQD = 60%			NQ	-98.6		126.5 Top of Bedrock at Elevation -98.6 ft. Dark gray, fine-grained to aphanitic, metamorphic SCHIST. Very hard, fresh to slightly weathered. Joints are low angle, very close to close, tight, some silt infilling, high angle to verticial undulating secondary joints. Calcite veins and stringers throughout. Rock Mass Quality=fair -BERWICK FORMATION- R1:Core Times (min:sec):	
130											

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 61

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge over Presumpscot River and MCRR
Location: Routes 26/100, Falmouth, Maine

Boring No.: BB-FPR-205
PIN: 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	27.9	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	B.Steinert/E.Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 NW
Date Start/Finish:	4/16/09 to 4/27/09	**Drilling Method:**	HW Drive to 75.0 ft/NW Drive to 125.5 ft	**Core Barrel:**	NQ - 2.0 in. I.D.
Boring Location:	E1005412, N326283 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	12.5 (4/27/09, 1345)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	R2	60/57	130.0 - 135.0	RQD = 58%						125.0-126.0' (3:00), 126.0-127.0' (2:00), 127.0-128.0' (3:00), 128.0-129.0' (6:00) Dark gray, fine-grained to aphanitic, metamorphic SCHIST. Very hard, fresh to slightly weathered. Joints are low angle, very close to close, tight, some silt infilling, high angle to verticial undulating secondary joints. Highly fractured zone from approximately 131.0 to 132.1 ft. Calcite veins and stringers throughout. Rock Mass Quality=fair -BERWICK FORMATION-	
								-105.9		R2: Core Times (min:sec): 130.0-131.0' (4:00), 131.0-132.0' (3:00), 132.0-133.0' (3:00), 133.0-134.0' (3:00), 134.0-135.0' (3:00)	
135								-107.1		133.8	
										Light gray, fine grained to aphanitic, metamorphic, slightly migmitized GNEISS. Very hard, fresh. Joints are moderately dipping, moderately close, displacement evident. Some shear features and grossular garnets, quartz and calcite veins throughout. Rock Mass Quality=fair -BERWICK FORMATION-	
										135.0	
140										**Bottom of Exploration at 135.0 feet Below Ground Surface.**	
145											
150											
155											

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 62

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Proposed Replacement Bridge over Presumpscot River and MCRR	**Boring No.:**	BB-FPR-206
Location:	Routes 26/100 Falmouth, Maine	**PIN:**	15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	26.4	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	B. Steinert	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 HW
Date Start/Finish:	4/15/09	**Drilling Method:**	HW Drive to 10.0 ft	**Core Barrel:**	--
Boring Location:	E1005467, N326292 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt

R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person

S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected

$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/16	0.0 - 2.0	WOH/1/2/1	3	3	HW Push			Dark brown to brown, moist, soft, SILT, trace fine sand, roots and organics -TOPSOIL-(OL/OH)	
								24.4		2.0	
	2D	24/20	2.0 - 4.0	1/1/1/WOH	2	2				Brown, wet, very soft, fine sandy SILT -ALLUVIAL DEPOSIT-(ML)	
	3D	24/20	4.0 - 6.0	WOH/1/1/WOH	2	2				Tan to yellow-brown, wet, very soft, SILT, trace fine SAND, mottled, occasional rootlet (ML)	
5											
	4D	24/24	6.0 - 8.0	2/1/1/1	2	2				Tan to yellow-brown, wet, very loose, fine SAND, poorly graded, trace silt, trace rootlets, slight mottling -ALLUVIAL DEPOSIT-(SP)	
								18.4		8.0	
	5D	24/24	8.0 - 10.0	1/1/1/1	2	2				Gray-brown, wet, very soft, clayey SILT, trace fine sand, mottled, frequent organics (CL-ML)	
10	6D V1A	24/18	10.0 - 12.0 10.5 - 10.9	push thru vane Su=620/175 psf			Open	15.4		Gray-brown, wet, very soft, clayey SILT, trace fine sand, mottled, dark brown organics throughout, desiccated (CL-ML) 55x110 mm vane raw torque readings: V1A: 160/45 in-lbs	
	MV		11.5 - 11.9							11.0	
	7D	24/12	12.0 - 14.0	4/6/7/11	13	13				V1B: Could not push vane for second reading Brown, wet, medium dense, fine SAND, little silt (SM)	
								12.9		13.5 Brown, coarse SAND	
	8D	24/16	14.0 - 16.0	6/4/4/3	8	8		11.9		14.5 Olive-gray to gray, wet, medium stiff, silty CLAY, slightly mottled (CL)	
15								10.9		15.5	
	9D	24/24	16.0 - 18.0	WOH/WOH/1/2	1	1				Gray, wet, very soft, silty CLAY, occasional black streaks/specs -MARINE DEPOSIT-(CL)	
20	10D V2A	24/24	20.0 - 22.0 20.5 - 20.9	push thru vane Su=1,515/115 psf				5.4		Gray, wet, very soft, silty CLAY (CL) 55x110 mm vane raw torque readings: V2A: 390/30 in-lbs	
	MV		21.5 - 21.9							21.0 V2B: Could not push vane for second reading Brown, wet, loose, silty fine SAND, interlayered with gray silty CLAY (SM/CL)	
25	11D	24/12	25.0 - 27.0	4/6/5/10	11	11	Open	1.4		25.0 Brown, wet, medium dense, medium to fine SAND, trace coarse sand, poorly graded	

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 63

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Boring No.: BB-FPR-206

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Proposed Replacement Bridge over Presumpscot River and MCRR Location: Routes 26/100 Falmouth, Maine	Boring No.: BB-FPR-206 PIN: 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	26.4	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	B. Steinert	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 HW
Date Start/Finish:	4/15/09	**Drilling Method:**	HW Drive to 10.0 ft	**Core Barrel:**	--
Boring Location:	E1005467, N326292 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
										-MARINE DEPOSIT-(SP)	
30	12D	24/8	30.0 - 32.0	8/12/13/12	25	25				Brown, moist, medium dense, medium to fine SAND, little coarse sand, trace gravel, poorly graded -MARINE DEPOSIT-(SP)	
35	MD	24/0	35.0 - 37.0	4/6/7/8	13	13				No Recovery	
								-10.6		37.0 **Bottom of Exploration at 37.0 feet Below Ground Surface.** Note: No Refusal Encountered	
40											
45											
50											

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 64

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge over Presumpscot River and MCRR
Location: Routes 26/100, Falmouth, Maine

Boring No.: BB-FPR-207
PIN: 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	23.7	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	B. Steinert	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 HW
Date Start/Finish:	4/16/09	**Drilling Method:**	HW Drive to 10 ft	**Core Barrel:**	--
Boring Location:	E1005390, N326305 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
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V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/16	0.0 - 2.0	WOH/1/1/1	2	2	Push	23.2		Dark brown to black, moist, very loose, fine SAND, little medium sand, trace coarse sand, little silt, poorly graded, organics throughout, rootlets -TOPSOIL-(SP)	
										0.5	
										Olive-gray, moist to wet, soft, clayey SILT, trace fine sand, occasional rootlets, slight mottling, blocky structure (CL-ML)	
	2D	24/16	2.0 - 4.0	2/2/3/2	5	5				Olive-gray, wet, medium stiff, clayey SILT, trace fine sand, slight mottling -ALLUVIAL DEPOSIT-(CL-ML)	
								19.7		4.0	
	3D	24/18	4.0 - 6.0	4/3/3/3	6	6				Brown, wet, medium stiff, fine sandy SILT, rootlets (ML)	
5								18.2		5.5	
	4D	24/16	6.0 - 8.0	4/4/2/2	6	6				Gray-brown to tan, wet, loose, fine SAND, little silt, poorly graded -ALLUVIAL DEPOSIT-(SP)	
	5D	24/16	8.0 - 10.0	1/2/1/1	3	3				Gray-brown to tan, wet, loose, fine SAND, little silt, poorly graded, occasional rootlets -ALLUVIAL DEPOSIT-(SP)	
10								13.7		10.0	
	6D	24/3	10.0 - 12.0	1/1/WOH/WOH	1	1	Open			Gray, wet, very soft, clayey SILT, trace fine sand (CL-ML)	
								11.7		12.0	
	7D	24/16	12.0 - 14.0	WOH/2/2/2	4	4				Gray, wet, very loose, fine SAND, little silt, poorly graded, occasional organics (SP)	
										Gray, wet, very loose, fine SAND, trace silt, poorly graded (SP)	
	8D	24/14	14.0 - 16.0	2/8/8/7	16	16		9.2		14.5	
15										Tan to rust-brown, wet, medium SAND, little coarse and fine sand, trace silt, poorly graded -ALLUVIAL DEPOSIT-(SP)	
	9D	24/15	16.0 - 18.0	2/1/1/WOH	2	2		7.2		16.5	
										Gray, wet, medium stiff, silty CLAY, frequent black streaks/specks -MARINE DEPOSIT-(CL)	
	10D	24/24	18.0 - 20.0	push thru vane						Gray, wet, medium stiff, silty CLAY, frequent black streaks/specks	
	V1A		18.6 - 19.0	Su=700/115 psf						55x110 mm vane raw torque readings:	
	V1B		19.6 - 20.0	Su=660/135 psf						V1A: 180/30 in-lbs V1B: 170/35 in-lbs	
20											
25											
	11D	24/24	25.0 - 27.0	push thru vane			Open			Gray, wet, medium stiff, silty CLAY -MARINE DEPOSIT- (CL)	
	V2A		25.6 - 26.0	Su=640/80 psf							

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 65

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FPR-207
PIN: 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	23.7	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	B. Steinert	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 HW
Date Start/Finish:	4/16/09	**Drilling Method:**	HW Drive to 10 ft	**Core Barrel:**	--
Boring Location:	E1005390, N326305 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	V2B		26.6 - 27.0	Su=660/270 psf						55x110 mm vane raw torque readings: V2A: 165/20 in-lbs V2B: 170/70 in-lbs	
30	12D	24/24	30.0 - 32.0	push thru vane						Gray, wet, medium stiff, silty CLAY -MARINE DEPOSIT-(CL) 55x110 mm vane raw torque readings: V3A: 170/90 in-lbs V3B: 185/40 in-lbs	
	V3A		30.6 - 31.0	Su=660/350 psf							
	V3B		31.6 - 32.0	Su=720/155 psf							
35	13D	24/24	35.0 - 37.0	WOR/WOR/WOR/ WOH						Gray, wet, medium stiff, silty CLAY, frequent black streaks/specks -MARINE DEPOSIT-(CL) Note: Trace fine sand observed in wash water from 37.0 to 40.0 ft.	
40	14D	24/16	40.0 - 42.0	2/13/15/17	28	28		-16.8		Olive-gray, wet, medium stiff, silty CLAY, trace fine sand (CL) 40.5 Tan to rust-brown, wet, medium dense, medium to fine SAND, little coarse sand, trace silt, fine gravel, occasional weathered gravel pieces, well graded -GLACIAL TILL-(SW)	
								-18.3		42.0 **Bottom of Exploration at 42.0 feet Below Ground Surface.** Note: No Refusal Encountered	
45											
50											

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 66

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Proposed Replacement Bridge over Presumpscot River and MCRR	**Location:**	Routes 26/100 Falmouth, Maine	**Boring No.:**	BB-FPR-208
				PIN:	15094.00
Driller:	Maine Test Borings	**Elevation (ft.)**	25.4	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne/B. Steinert	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 HW
Date Start/Finish:	5/7/09 to 5/14/09	**Drilling Method:**	HW Drive to 122.0 ft	**Core Barrel:**	NQ - 2.0 in. I.D.
Boring Location:	E1005336, N326640 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/5	0.0 - 2.0	1/WOH/1/3	1	1	Push			Dark brown to gray, wet, very soft, alternating SILT and SAND, poorly graded -TOPSOIL-(ML/SP)	
	2D	24/17	2.0 - 4.0	2/1/2/2	3	3		22.9		2.5 Brown, wet, soft, sandy SILT, organics throughout -ALLUVIAL DEPOSIT-(ML)	
	3D	24/20	4.0 - 6.0	WOH/WOH/WOH/ WOH						Brown, saturated, very soft, SILT, little medium to fine sand, occasional organics (ML)	
5											
	4D	24/6	6.0 - 8.0	WOH/1/2/2	3	3				Brown, saturated, very soft, SILT, little medium to fine sand, occasional organics (ML)	
								17.6		7.8	
	5D	24/14	8.0 - 10.0	1/1/1/1	2	2				Brown, saturated to wet, very loose, silty SAND -ALLUVIAL DEPOSIT-(SM)	
10	6D	24/20	10.0 - 12.0	1/2/1/1	3	3	20			Gray-brown, mottled, wet, soft, sandy SILT (ML)	
							17				
	7D	24/14	12.0 - 14.0	WOH/1/1/1	2	2	18			Gray-brown, mottled, wet, soft, sandy SILT (ML)	
							17	12.3		13.1 Gray, wet to saturated, very soft, sandy SILT, frequent organics (ML)	
	8D	24/15	14.0 - 16.0	3/2/3/2	5	5	18	11.4		14.0 Gray, wet to saturated, loose, SAND, little silt, poorly graded, occasional organics -ALLUVIAL DEPOSIT-(SP)	
15							14				
	9D	24/14	16.0 - 18.0	2/21/9/6	30	30	245			Gray, wet, medium dense, SAND, trace silt, poorly graded, layer of wood at approximately 16.7 to 17.7 ft (SP) Note: Large root encountered at approximately 16.7 ft, washed ahead of casing to 18.0 ft before driving, washed to 18.2 ft to try to advance beyond wood.	
							30				
	MD	22/0	18.2 - 20.0	6/3/3/1	6	6	19			No Recovery in two attempts, pushing on wood	
							17				
20	11D	24/8	20.0 - 22.0	4/4/4/5	8	8	31			Gray, wet to saturated, loose, fine SAND, trace medium sand and silt, poorly graded, occasional organic lenses less than 0.1 ft thick -ALLUVIAL DEPOSIT-(SP)	
							36				
	12D	24/10	22.0 - 24.0	5/5/7/10	12	12	32			Gray, wet, medium dense, fine SAND, little medium sand, poorly graded, one wood fragment at approximately 23.0 to 23.1 ft (SP)	
							43				
	13D	24/12	24.0 - 26.0	4/4/4/3	8	8	51			Gray, wet, loose, fine SAND, little medium sand, poorly graded, clay in tip (SP)	
25							38				

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 67

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge over Presumpscot River and MCRR
Location: Routes 26/100, Falmouth, Maine

Boring No.: BB-FPR-208
PIN: 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	25.4	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne/B. Steinert	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 HW
Date Start/Finish:	5/7/09 to 5/14/09	**Drilling Method:**	HW Drive to 122.0 ft	**Core Barrel:**	NQ - 2.0 in. I.D.
Boring Location:	E1005336, N326640 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	14D	24/24	26.0 - 28.0	WOR/WOR/WOH/ WOH			31	-0.6		26.0 Gray, wet, medium stiff, silty CLAY -MARINE DEPOSIT-(CL)	
							29				
							28				
							29				
30	15D	24/24	30.0 - 32.0	push thru vane			35			Gray, wet, medium stiff, silty CLAY -MARINE DEPOSIT-(CL) 55x110 mm vane raw torque readings: V1A: 11.5/1 ft-lbs V1B: 12/0.5 ft-lbs	
	V1A		30.5 - 30.9	Su=545/40 psf							
	V1B		31.5 - 31.9	Su=565/20 psf			29				
							28				
							23				
							18				
35	16D	24/24	35.0 - 37.0	WOR/WOR/WOR/ WO1P			25			Gray, wet, medium stiff, silty CLAY (CL)	
							24				
							24				
							23				
							21				
40	17D	24/24	40.0 - 42.0	push thru vane			25			Gray, wet, medium stiff, silty CLAY (CL) 55x110 mm vane raw torque readings: V2A: 9/0.5 ft-lbs Note: Unable to push vane past 41.4 ft.	
	V2A		40.5 - 40.9	Su=425/20 psf							
	MV		41.5 - 41.9				26				
							32				
							30				
							28				
45	18D	24/24	45.0 - 47.0	WOR/2/4/10	6	6	34			Gray, wet to saturated, medium stiff, silty CLAY, some sand, alternating clay and fine sand layers from 45.5 to 46.3 ft	
							55	-20.9		46.3 Gray, saturated, loose, fine SAND, trace medium sand and silt, poorly graded -MARINE DEPOSIT-(SP)	
							56				
							68				
							66				
50	19D	24/14	50.0 - 52.0	5/5/8/8	13	13	48			Gray, saturated, medium dense, fine SAND, trace medium sand and silt, poorly graded, two small silt lenses at approximately 50.8 ft -MARINE DEPOSIT-(SP)	
							45				

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 68

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FPR-208
PIN: 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	25.4	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne/B. Steinert	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 HW
Date Start/Finish:	5/7/09 to 5/14/09	**Drilling Method:**	HW Drive to 122.0 ft	**Core Barrel:**	NQ - 2.0 in. I.D.
Boring Location:	E1005336, N326640 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
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R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							52				
							102				
							102				
55	20D	24/11	55.0 - 57.0	10/9/4/6	13	13	98			Gray, wet, medium dense, coarse to fine SAND, little coarse to fine gravel, well graded, finer with depth (SW)	
							79	-31.2		56.6	
							102			Gray, wet, medium dense, silty, coarse to fine SAND -MARINE DEPOSIT-(SM)	
							97				
							77				
60	21D	24/20	60.0 - 62.0	2/WOH/WOH/WOH			51			Gray, saturated, very loose, silty, medium to fine SAND, trace coarse sand and fine gravel, occasional medium to fine sand layers throughout (SM)	
							49				
							46				
							66				
							90				
65	22D	24/12	65.0 - 67.0	3/4/6/18	10	10	93			Gray-green, wet, medium dense, medium to fine SAND, trace coarse sand and fine gravel, poorly graded, large piece of gravel in tip -MARINE DEPOSIT-(SP)	
							70				
							150				
							131				
							138				
70	23D	24/3	70.0 - 72.0	3/3/12/18	15	15	63			Gray, wet, medium dense, fine SAND, trace coarse to medium sand, trace fine gravel, poorly graded, no structure, likely pushing coarse gravel -MARINE DEPOSIT-(SP)	
							60				
							92				
							104			Note: Increased resistance at approximately 74 ft. Coarse sand, some fine gravel observed in wash water.	
							109				
75	MD	24/0	75.0 - 77.0	13/8/9/13	17	17	78			No Recovery after two attemps	
							100				
	25D	24/4	77.0 - 79.0	2/4/5/6	9	9	141			Gray, wet, loose, fine SAND, little coarse to medium sand, trace fine gravel, poorly graded, no structure (SP)	

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 69

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Proposed Replacement Bridge over Presumpscot River and MCRR **Location:** Routes 26/100 Falmouth, Maine	**Boring No.:** BB-FPR-208 **PIN:** 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	25.4	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne/B. Steinert	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 HW
Date Start/Finish:	5/7/09 to 5/14/09	**Drilling Method:**	HW Drive to 122.0 ft	**Core Barrel:**	NQ - 2.0 in. I.D.
Boring Location:	E1005336, N326640 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
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R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
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WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
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q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							175				
							193				
80	26D	24/10	80.0 - 82.0	16/19/17/15	36	36	220	-54.6		80.0 Gray, wet, dense, coarse to fine SAND, some clay, trace fine gravel, well graded, bonded -GLACIAL TILL- (SW)	
							215				
							253				
							250				
							213				
85	27D	24/7	85.0 - 87.0	64/22/15/18	37	37	94			Gray, wet, dense, medium SAND, little clay, little coarse and fine sand, trace coarse gravel, clods of well bonded soil -GLACIAL TILL- (SW)	
							125				
							195				
							210			Note: Approximately 3 ft of soil inside borehole after washing out.	
							225				
90	28D	24/16	90.0 - 92.0	35/52/73/87	125	125	96			Gray, moist, very dense, fine SAND, little medium sand, trace coarse sand, poorly graded -GLACIAL TILL-(SP)	
							82				
							95				
							160				
							346			Note: Approximately 15 ft of soil inside casing after advancing to 95.0 ft.	
95	29D	24/12	95.0 - 97.0	78/42/42/75	84	84	74			Gray, wet, very dense, sandy GRAVEL, little silt, well graded (GW)	
							42				
							71			Note: Encountered several cobbles and boulders when washing from 95.0 to 100.0 ft and 100.0 to 105 ft.	
							189				
							129				
100	30D	8/6	100.0 - 100.7	61/100(3")			80			Gray, wet, very dense, fine SAND, some medium sand, little coarse sand, little gravel, trace silt -GLACIAL TILL-(SW)	
							70				
							73				
							85				

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 70

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Proposed Replacement Bridge over Presumpscot River and MCRR Location: Routes 26/100 Falmouth, Maine	Boring No.: BB-FPR-208 PIN: 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	25.4	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne/B. Steinert	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 HW
Date Start/Finish:	5/7/09 to 5/14/09	**Drilling Method:**	HW Drive to 122.0 ft	**Core Barrel:**	NQ - 2.0 in. I.D.
Boring Location:	E1005336, N326640 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							141				
105	31D	24/12	105.0 - 107.0	80/175			105			Gray, moist, very dense, fine SAND, little coarse to medium sand, trace gravel and silt, well bonded (SW)	
							99				
							83				
							115				
							120				
110	32D	24/16	110.0 - 112.0	34/42/14/15	56	56	150			Gray, wet, very dense, fine SAND, little medium sand and silt, trace coarse sand and gravel, well bonded -GLACIAL TILL-(SW)	
							117				
							132				
							102				
							112			Note: Cobble at approximately 114.2 to 114. 5 ft, sampled open hole.	
115	33D	6/6	115.0 - 115.5	137(6")			138			Gray, wet, very dense, coarse to fine SAND, some gravel, trace silt, loosely bonded -GLACIAL TILL-(SW)	
							195				
							190				
							132			Note: Encountered several cobbles between 115.0 and 120.0 ft.	
							132				
120	34D	4/4	120.0 - 120.3	100(4")			200			Gray, wet, very dense, gravelly coarse to fine SAND, trace silt, bonded, several weathered pieces of gravel (SW)	
							201				
								-97.1		122.5 Top of Bedrock at El. -97.1	
125	R1	48/47	125.0 - 129.0	RQD = 35%			NQ			Dark gray, fine-graied to aphanitic, metamorphic SCHIST, moderately hard to hard, very slight to moderate weathering. Joints are horizontal. Rock Mass Quality=poor -BERWICK FORMATION- R1: Core Times (min:sec): 125.0-126.0' (3:00), 126.0-127.0' (3:00) 127.0-128.0' (2:00), 128.0-129.0' (3:00)	
	R2	60/60	129.0 - 134.0	RQD = 0%						Dark gray to gray, fine-grained to aphanitic, metamorphic SCHIST, soft to moderately hard, moderate to severe weathering. 130.0 to 133.0 ft - zones of moderate to severe weathering with two silt infilled joints. Few	
130											

Remarks:

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2. Hammer consisted of rope and cathead and safety hammer.

SHEET 71

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Proposed Replacement Bridge over Presumpscot River and MCRR Location: Routes 26/100 Falmouth, Maine	Boring No.: BB-FPR-208 PIN: 15094.00

Driller:	Maine Test Borings	Elevation (ft.)	25.4	Auger ID/OD:	--
Operator:	R. Leonard	Datum:	NAVD 88	Sampler:	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne/B. Steinert	Rig Type:	CME 550X	Hammer Wt./Fall:	140/30 SS - 300/30 HW
Date Start/Finish:	5/7/09 to 5/14/09	Drilling Method:	HW Drive to 122.0 ft	Core Barrel:	NQ - 2.0 in. I.D.
Boring Location:	E1005336, N326640 (See Plan)	Casing ID/OD:	HW - 4.0 in. I.D.	Water Level*:	None Observed
Hammer Efficiency Factor:	0.6	Hammer Type:	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
										secondary high angle joints. Frequent calcite veins. Rock Mass Quality=very poor -BERWICK FORMATION- R2: Core Times (min:sec): 129.0-130.0' (3:00), 130.0-131.0' (9:00), 131.0-132.0' (4:00), 132.0-133.0' (4:00), 133.0-134.0' (5:00)	
135	R3	60/58	134.0 - 139.0	RQD = 50%						Dark gray to gray, fine-grained to aphanitic, metamorphic SCHIST. Hard, slight to moderate weathering, joints are very close to close, horizontal to low angle, tight to partly open. Frequent thin calcite veins and stringers. Rock Mass Quality=poor -BERWICK FORMATION- R3: Core Times (min:sec): 134.0-135.0' (3:00), 135.0-136.0' (3:00), 136.0-137.0' (3:00), 137.0-138.0' (3:00), 138.0-139.0' (3:00)	
								-113.6		139.0 **Bottom of Exploration at 139.0 feet Below Ground Surface.**	
140											
145											
150											
155											

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 72

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Proposed Replacement Bridge over Presumpscot River and MCRR	**Location:**	Routes 26/100 Falmouth, Maine	**Boring No.:**	BB-FRR-201
				PIN:	15094.00
Driller:	Maine Test Borings	**Elevation (ft.)**	24.3	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 HW
Date Start/Finish:	5/15/09 to 5/21/09	**Drilling Method:**	HW Drive to 121.8 ft	**Core Barrel:**	NQ - 2.0 in. I.D.
Boring Location:	E1005290, N326795 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:

D = Split Spoon Sample	R = Rock Core Sample	S_u = Insitu Field Vane Shear Strength (psf)	$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
MD = Unsuccessful Split Spoon Sample attempt	SSA = Solid Stem Auger	T_v = Pocket Torvane Shear Strength (psf)	WC = water content, percent
U = Thin Wall Tube Sample	HSA = Hollow Stem Auger	q_p = Unconfined Compressive Strength (ksf)	LL = Liquid Limit
MU = Unsuccessful Thin Wall Tube Sample attempt	RC = Roller Cone	N-uncorrected = Raw field SPT N-value	PL = Plastic Limit
V = Insitu Vane Shear Test	WOH = weight of 140lb. hammer	Hammer Efficiency Factor = Annual Calibration Value	PI = Plasticity Index
MV = Unsuccessful Insitu Vane Shear Test attempt	WOR = weight of rods	N_{60} = SPT N-uncorrected corrected for hammer efficiency	G = Grain Size Analysis
	WO1P = Weight of one person	N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected	C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/16	0.0 - 2.0	WOH/2/1/1	3	3	Push			Dark brown, wet, soft, SILT, trace coarse to medium SAND, organics throughout -TOPSOIL-(ML)	
								23.3		1.0	
	2D	24/24	2.0 - 4.0	2/5/9/16	14	14				Brown, wet, soft, SILT, little medium to fine sand, some roots (ML) Brown, wet, stiff, some coarse to fine sand, no roots, somewhat mottled (ML)	
							19			Gray-brown, dry, stiff, SILT, little medium to fine sand, somewhat mottled, desiccated (ML)	
								20.3		4.0	
	3D	24/18	4.0 - 6.0	8/13/16/18	29	29	34			Gray-brown, moist, very stiff, silty CLAY, trace fine sand, blocky structure, occasional black organic staining on partings -MARINE DEPOSIT-(CL)	
5							16				
	4D	24/20	6.0 - 8.0	4/8/10/13	18	18	14			Olive-brown, moist, very stiff, silty CLAY, trace fine sand, blocky structure, somewhat mottled -MARINE DEPOSIT-(CL)	
							16				
							13				
							12				
10	5D MV	24/23	10.0 - 12.0 10.4 - 11.0	5/7/7/8	14	14	16			Olive-brown to gray-brown, mottled, moist to wet, stiff, silty CLAY, somewhat blocky structure -MARINE DEPOSIT-(CL)	
							13			Note: Attempted vane at 10.0 ft., unable to push.	
							12				
							9			Note: Driller noted slight change in density at approximately 14.5 ft.	
							6	9.8		14.5	
15	6D V1A	24/24	15.0 - 17.0 15.6 - 16.0	Push thru vane Su=425/20 psf			9			Gray, wet, soft, silty CLAY (CL) 55x110 mm vane raw torque readings: V1A: 9/0.5 ft-lbs	
	V1B		16.6 - 17.0	Su=445/40 psf			10			V1B: 9.5/1 ft-lbs	
							8				
							7				
							7				
20	7D	24/19	20.0 - 22.0	WOR/WOR/WOR/ WO1P			5			Gray to dark gray, wet, soft, silty CLAY, trace fine sand, black organic streaking throughout -MARINE DEPOSIT-(CL)	
							6				
							6				
							5				
							5				
25	8D V2A	24/4	25.0 - 27.0 25.6 - 26.0	push thru vane Su=370/20 psf			7			Gray, saturated, soft, silty CLAY, trace fine sand (CL) 55x110 mm vane raw torque readings:	

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 73

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FRR-201
PIN: 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	24.3	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 HW
Date Start/Finish:	5/15/09 to 5/21/09	**Drilling Method:**	HW Drive to 121.8 ft	**Core Barrel:**	NQ - 2.0 in. I.D.
Boring Location:	E1005290, N326795 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	V2B		26.6 - 27.0	Su=250/0 psf			6			V2A: 8/0.5 ft-lbs V2B: 5.5/0 ft-lbs	
							5				
							3				
							1				
30	9D	24/24	30.0 - 32.0	WOR/WOR/WOR/ WOR			4			Gray, saturated, soft, silty CLAY, trace fine sand, black organic streaking throughout -MARINE DEPOSIT-(CL)	
							7				
							7				
							7				
							5				
35	10D V3A	24/20	35.0 - 37.0 35.6 - 36.0	push thru vane Su=465/20 psf			8			Gray, saturated, soft, silty CLAY, trace fine sand, occasional black streaking, several drop stones (CL) 55x110 mm vane raw torque readings: V3A: 10/0.5 ft-lbs V3B: 6.5/0 ft-lbs	
	V3B		36.6 - 37.0	Su=310/0 psf			10				
							10				
							10				
							8				
40	11D	24/24	40.0 - 42.0	WOR/WOR/WOR/ WOR			8			Gray, wet to saturated, soft, silty CLAY, trace fine sand -MARINE DEPOSIT-(CL)	
							8				
							6				
							4				
							3				
45	MD V4A	24/0	45.0 - 47.0 45.6 - 46.0	push thru vane Su=310/0 psf			4			No Recovery, noted drop stone in spoon 55x110 mm vane raw torque readings: V4A: 6.5/0 ft-lbs Note: Unable to push vane beyond 46.1 ft.	
	MV		46.6 - 47.0				4				
							5				
							5				
							4				
50	13D	24/24	50.0 - 52.0	1/WOH/2/2	2	2	6			Gray, wet to saturated, soft, silty CLAY, little fine sand, fine sand layers (<0.1 to 0.3 ft thick) throughout -MARINE DEPOSIT-(CL)	
							9				

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 74

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Proposed Replacement Bridge over Presumpscot River and MCRR **Location:** Routes 26/100 Falmouth, Maine	**Boring No.:** BB-FRR-201 **PIN:** 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	24.3	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 HW
Date Start/Finish:	5/15/09 to 5/21/09	**Drilling Method:**	HW Drive to 121.8 ft	**Core Barrel:**	NQ - 2.0 in. I.D.
Boring Location:	E1005290, N326795 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	None Observed
Hammer Efficiency Factor: 0.6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							8	-28.5		Note: Driller noted change in density at approximately 52.8 ft. 52.8	
							8				
							8				
55	14D ~~MV~~	24/12	55.0 - 57.0 ~~55.6 - 56.0~~	6/8/11/11	17	17	8			Gray, wet, medium dense, fine SAND, trace medium sand and silt, poorly graded -MARINE DEPOSIT-(SP) Note: Unable to push vane past 55.0 ft.	
							10				
							13				
							14				
							21				
60	15D	24/15	60.0 - 62.0	5/7/8/22	15	15	32			Gray, wet, medium dense, fine SAND, trace medium sand and silt, poorly graded (SP)	
							40				
							41				
							54				
							72				
65	16D	24/14	65.0 - 67.0	4/3/10/9	13	13	66			Gray, wet to saturated, medium dense, silty fine SAND, little coarse to medium sand, trace coarse gravel, clay layer from approximately 65.4 to 65.9 ft -MARINE DEPOSIT-(SM)	
							72				
							85				
							110				
							155				
70	17D	24/6	70.0 - 72.0	8/10/11/11	21	21	60			Gray, wet, medium dense, coarse to fine SAND, little silt, well graded (SW)	
							78				
							119				
							114				
							107			Note: Hole caved to approximately 72 ft after pulling sample.	
75	18D	24/5	75.0 - 77.0	7/7/11/13	18	18	88			Gray, wet, medium dense, coarse to fine SAND, trace gravel, well graded -MARINE DEPOSIT-(SW)	
							80				
							77				

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 75

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Proposed Replacement Bridge over Presumpscot River and MCRR Location: Routes 26/100 Falmouth, Maine	Boring No.: BB-FRR-201 PIN: 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	24.3	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 HW
Date Start/Finish:	5/15/09 to 5/21/09	**Drilling Method:**	HW Drive to 121.8 ft	**Core Barrel:**	NQ - 2.0 in. I.D.
Boring Location:	E1005290, N326795 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							78				
							96				
80	19D	24/12	80.0 - 82.0	12/14/16/24	30	30	66			Gray, wet, medium dense, coarse to fine SAND, little fine gravel, well graded (SW)	
							63				
							62				
							56				
							61				
85	20D	24/5	85.0 - 87.0	10/16/10/10	26	26	115			Gray, wet, medium dense, coarse to fine SAND, little coarse to fine gravel, trace silt, well graded -MARINE DEPOSIT-(SW)	
							123				
							148				
							154	-64.7		89.0	
							172				
90	21D	24/5	90.0 - 92.0	22/30/12/6	42	42	76			Gray, wet, dense, coarse to fine SAND, some gravel, trace silt, well graded, loosely bonded -GLACIAL TILL- (SW) Note: Encountered cobbles between 92.0 and 94.5 ft.	
							62				
							91				
							98				
							107				
95	22D	24/3	95.0 - 97.0	19/8/15/24	23	23	79			Gray, wet, medium dense, fine SAND, little coarse gravel and silt, trace coarse to medium sand, poorly graded (SP)	
							80				
							95				
							80				
							118				
100	23D	24/12	100.0 - 102.0	18/14/13/16	27	27	113			Gray, wet, medium dense, coarse to fine SAND, some coarse to fine gravel, trace silt, well graded -GLACIAL TILL-(SW) Note: Encountered cobble at approximately 103.3 ft. Note: Washed through obstruction (cobble/boulder) at approximately 103.7 ft.	
							98				
							99				
							236/0.7' 12/0.3'				

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 76

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FRR-201
PIN: 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	24.3	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 HW
Date Start/Finish:	5/15/09 to 5/21/09	**Drilling Method:**	HW Drive to 121.8 ft	**Core Barrel:**	NQ - 2.0 in. I.D.
Boring Location:	E1005290, N326795 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							90				
105	24D	24/5	105.0 - 107.0	123/35/15/8	50	50	71			Gray, wet, dense, coarse to fine SAND, some coarse to fine gravel, trace silt, well graded (SW)	
							89				
							74				
							225/0.5' 33/0.5'			Note: Washed through obstruction (cobble/boulder) at approximately 108.5 ft.	
							135				
110	25D	24/11	110.0 - 112.0	38/70/49/31	119	119	104			Gray, wet, very dense, medium to fine SAND, little gravel, trace coarse sand and silt, tip of spoon contained gravel in silt matrix -GLACIAL TILL- (SW)	
							101				
							76				
							95				
							114				
115	26D	24/4	115.0 - 117.0	135(6")			111			Gray, wet, hard, sandy CLAY, trace fine gravel, no structure, possible wash (CL)	
							95				
							97				
							103				
							115				
120	27D	24/6	120.0 - 122.0	67/100(3")			122			Gray, wet, very dense, fine SAND, little medium sand and silt, trace gravel and coarse sand, somewhat bonded -GLACIAL TILL-(SP)	
							138/0.8'				
	R1	54/54	124.5 - 129.0	RQD = 52%			NQ	-100.2		124.5	
125							NQ			Top of Bedrock at El. -100.2 Gray, fine-grained to aphanitic, metamorphic GNEISS. Very hard to hard, fresh to slightly weathered. Joints are horizontal to moderately dipping, tight to open, occasional silt infilling. Rock Mass Quality=fair -BERWICK FORMATION- R1: Core Times (min:sec): 124.5-125.5' (6:00), 125.5-126.5' (6:00), 126.5-127.5' (5:00), 127.5-128.5' (5:00), 128.5-129.0' (3:00)	
	R2	60/60	129.0 - 134.0	RQD = 37%						Gray, fine-grained to aphanitic, metamorphic GNEISS. Very hard to hard, fresh to slightly weathered. Joints are horizontal to moderately dipping, very close to close, tight to partly open, some sand and silt	
130											

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 77

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FRR-201
PIN: 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	24.3	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 HW
Date Start/Finish:	5/15/09 to 5/21/09	**Drilling Method:**	HW Drive to 121.8 ft	**Core Barrel:**	NQ - 2.0 in. I.D.
Boring Location:	E1005290, N326795 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
										infilling. Rock Mass Quality=poor -BERWICK FORMATION- R2: Core Times (min:sec): 129.0-130.0' (5:00), 130.0-131.0' (6:00), 131.0-132.0' (4:00), 132.0-133.0' (4:00), 133.0-134.0' (4:00)	
								-109.7		134.0 **Bottom of Exploration at 134.0 feet Below Ground Surface.**	
135											
140											
145											
150											
155											

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 78

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Boring No.: BB-FRR-201

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Proposed Replacement Bridge over Presumpscot River and MCRR	**Location:**	Routes 26/100 Falmouth, Maine	**Boring No.:**	BB-FRR-202
				PIN:	15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	53.2	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/16 HW
Date Start/Finish:	4/28/09 to 5/6/09	**Drilling Method:**	HW Drive to 10.0 ft/NW Drive to 168.5 ft	**Core Barrel:**	NQ - 2.0 in. I.D.
Boring Location:	E1005206, N326994 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	27.9 (5/6/09, 1720)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/20	0.0 - 2.0	3/5/8/5	13	13	7			Brown, moist, medium dense, SAND, some silt and gravel, well graded -FILL-(SW)	
							21	51.4		1.8	
	2D	24/17	2.0 - 4.0	3/4/6/8	10	10	23	50.4		Brown to gray-brown, moist, SILT, some sand, frequent organics -MARINE DEPOSIT-(ML)	
							29			2.8 Olive-brown, moist, silty CLAY, trace sand, occasional organics, somewhat blocky -MARINE DEPOSIT-(CL)	
	3D	24/14	4.0 - 6.0	3/2/3/2	5	5	Open				
5	MV		4.6 - 5.0							Gray-brown, mottled, wet, medium stiff, sandy CLAY, frequent fine sand layers (CL) Note: Unable to push vane from 4.0 to 5.0 ft.	
	4D	24/7	6.0 - 8.0	WOR/3/1/1	4	4				Gray-brown, wet, soft, silty CLAY with sand (CL)	
	MV		6.6 - 7.0							Note: Unable to push vane beyond 6.5 ft.	
	5D	24/15	8.0 - 10.0	push thru vane						Gray, wet, soft, silty CLAY, trace fine sand (CL)	
	V1A		8.6 - 9.0	Su=485/80 psf						55x110 mm vane raw torque readings: V1A: 125/20 in-lbs	
	V1B		9.6 - 10.0	Su=425/60 psf						V1B: 110/15 in-lbs	
10											
15	6D	24/24	15.0 - 17.0	WOR/WOR/WOR/ WOR						Gray, wet, soft, silty CLAY -MARINE DEPOSIT-(CL)	
										Note: No recovery for 7D on first attempt.	
20	7D	24/10	20.0 - 22.0	push thru vane						Gray, wet, soft, silty CLAY (CL)	WC=42.8%
	V2A		20.6 - 21.0	Su=350/20 psf						55x110 mm vane raw torque readings: V2A: 90/5 in-lbs	LL=35 PL=19
	V2B		21.6 - 22.0	Su=330/20 psf						V2B: 85/5 in-lbs	PI=16
25	MU	24/0	25.0 - 27.0				Open				

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. Bentonite drilling mud used during drilling through marine clay.

SHEET 79

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FRR-202
PIN: 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	53.2	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/16 HW
Date Start/Finish:	4/28/09 to 5/6/09	**Drilling Method:**	HW Drive to 10.0 ft/NW Drive to 168.5 ft	**Core Barrel:**	NQ - 2.0 in. I.D.
Boring Location:	E1005206, N326994 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	27.9 (5/6/09, 1720)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	MU	24/0	27.0 - 29.0								
	MU	24/0	29.0 - 31.0								
30										Note: No recovery on first sample attempt.	
	8D	24/14	31.0 - 33.0	push thru vane						Gray, saturated, soft, silty CLAY (CL)	
	V3A		31.6 - 32.0	Su=330/20 psf						55x110 mm vane raw torque readings:	
	V3B		32.6 - 33.0	Su=270/20 psf						V3A: 7/0.5 ft-lbs V3B: 6/0.5 ft-lbs	
35	9D	24/24	35.0 - 37.0	push thru vane						Gray, saturated, very soft to soft, silty CLAY -MARINE DEPOSIT-(CL)	WC=42.5% LL=34 PL=19 PI=15
	V4A		35.6 - 36.0	Su=425/0 psf						55x110 mm vane raw torque readings:	
	V4B		36.5 - 36.9	Su=235/20 psf						V4A: 9/0 ft-lbs V4B: 5/0.5 ft-lbs	
40	10D	24/24	40.0 - 42.0	push thru vane						Gray, saturated, soft, silty CLAY (CL)	
	V5A		40.6 - 41.0	Su=425/20 psf						55x110 mm vane raw torque readings:	
	V5B		41.6 - 42.0	Su=445/0 psf						V5A: 9/0.5 ft-lbs V5B: 9.5/0 ft-lbs	
45	1U	24/24	45.0 - 47.0								C#CRC-1 WC=39.3% LL=31 PL=18 PI=13
	11D	24/24	47.0 - 49.0	push thru vane						Gray, wet, soft, silty CLAY, trace fine sand, black organic staining from approximately 48.5 to 49.0 ft (CL)	
	V6A		47.6 - 48.0	Su=465/0 psf						55x110 mm vane raw torque readings:	
	V6B		48.6 - 49.0	Su=425/20 psf						V6A: 10/0 ft-lbs V6B: 9/0.5 ft-lbs	
50							Open				

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. Bentonite drilling mud used during drilling through marine clay.

SHEET 80

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Proposed Replacement Bridge over Presumpscot River and MCRR
Location:	Routes 26/100 Falmouth, Maine
Boring No.:	BB-FRR-202
PIN:	15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	53.2	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/16 HW
Date Start/Finish:	4/28/09 to 5/6/09	**Drilling Method:**	HW Drive to 10.0 ft/NW Drive to 168.5 ft	**Core Barrel:**	NQ - 2.0 in. I.D.
Boring Location:	E1005206, N326994 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	27.9 (5/6/09, 1720)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
55	12D	24/24	55.0 - 57.0	push thru vane						Gray, saturated to wet, medium stiff, silty CLAY, black organic staining throughout -MARINE DEPOSIT-(CL) 55x110 mm vane raw torque readings: V7A: 12/0.5 ft-lbs V7B: 14/0.5 ft-lbs	WC=40.3% LL=30 PL=17 PI=13
	V7A		55.6 - 56.0	Su=565/20 psf							
	V7B		56.6 - 57.0	Su=660/20 psf							
60	13D	24/24	60.0 - 62.0	push thru bane						Gray, saturated, medium stiff, silty CLAY, trace fine sand, black organic staining (CL) 55x110 mm vane raw torque readings: V8A: 13.5/2 ft-lbs V8B: 11/2 ft-lbs	
	V8A		60.6 - 61.0	Su=620/95 psf							
	V8B		61.6 - 62.0	Su=505/95 psf							
65	14D	24/24	65.0 - 67.0	push thru vane						Gray, saturated, medium stiff, silty CLAY, trace fine sand (CL) 55x110 mm vane raw torque readings: V9A: 17/2 ft-lbs V9B: 21/0.5 ft-lbs	WC=30.8% LL=23 PL=15 PI=8
	V9A		65.6 - 66.0	Su=795/95 psf							
	V9B		66.6 - 67.0	Su=970/20 psf							
70	2U	24/14	70.0 - 72.0							Note: Attempted undisturbed piston tube sample from 70.0 to 72.0 ft, recovered 14 in., discarded tube after successful 3U (72 to 74).	
	3U	24/22	72.0 - 74.0								C#CRC-2 WC=32.2% LL=24 PL=15 PI=9
	15D	24/24	74.0 - 76.0	push thru vane						Gray, saturated, medium stiff, silty CLAY, trace fine sand (CL) 55x110 mm vane raw torque readings: V10A: 11/1 ft-lbs V10B: 18/2 ft-lbs	
75	V10A		74.6 - 75.0	Su=505/40 psf							
	V10B		75.6 - 76.0	Su=835/95 psf			Open				

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. Bentonite drilling mud used during drilling through marine clay.

SHEET 81

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Proposed Replacement Bridge over Presumpscot River and MCRR Location: Routes 26/100 Falmouth, Maine	Boring No.: BB-FRR-202 PIN: 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	53.2	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/16 HW
Date Start/Finish:	4/28/09 to 5/6/09	**Drilling Method:**	HW Drive to 10.0 ft/NW Drive to 168.5 ft	**Core Barrel:**	NQ - 2.0 in. I.D.
Boring Location:	E1005206, N326994 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	27.9 (5/6/09, 1720)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
80	16D	24/6	80.0 - 82.0	WOR/WOR/WOR/1						Gray, wet to saturated, medium stiff, silty CLAY, some fine sand, chunks of cemented sand	
								-29.8		83.0 Note: Driller noted change in density of material at approximately 83.0 ft based on drill action.	
85	17D	24/8	85.0 - 87.0	11/12/16/17	28	28				Gray, wet, medium dense, fine SAND, trace silt, poorly graded -MARINE DEPOSIT-(SP)	
90	18D	24/7	90.0 - 92.0	16/15/19/20	34	34				Gray, wet, dense, fine SAND, trace medium sand, trace silt, poorly graded (SP)	
95	19D	24/10	95.0 - 97.0	9/16/16/19	32	32				Gray, wet, dense, fine SAND, little medium sand, little silt, poorly graded, occasional gray silt lenses (SP)	
100	20D	24/5	100.0 - 102.0	8/12/13/12	25	25	Open			Gray, wet, medium dense, medium to fine SAND, little coarse sand, trace silt, poorly graded -MARINE DEPOSIT-(SP)	
							7				
							12				

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. Bentonite drilling mud used during drilling through marine clay.

SHEET 82

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FRR-202
PIN: 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	53.2	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/16 HW
Date Start/Finish:	4/28/09 to 5/6/09	**Drilling Method:**	HW Drive to 10.0 ft/NW Drive to 168.5 ft	**Core Barrel:**	NQ - 2.0 in. I.D.
Boring Location:	E1005206, N326994 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	27.9 (5/6/09, 1720)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
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MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							24				
105	21D	24/9	105.0 - 107.0	14/20/22/23	42	42	37			Gray, wet, dense, coarse to fine SAND, trace fine gravel and silt, well graded (SW)	
							47				
							47				
							32				
	22D	24/8	109.0 - 111.0	10/13/19/21	32	32	80			Gray, wet, dense, medium to fine SAND, trace coarse sand, trace silt, poorly graded (SP)	
110							58				
							60				
							59				
							63				
	23D	24/4	114.5 - 116.5	10/14/17/22	31	31	61			Gray, wet, dense, medium to fine SAND, trace coarse sand, trace silt and gravel, poorly graded -MARINE DEPOSIT-(SP)	
115							70				
							95				
							100				
							76				
	24D	24/7	119.5 - 121.5	7/15/23/27	38	38	72			Gray, wet, dense, medium to fine SAND, trace clay, trace silt, poorly graded (SP)	
120							92				
							134				
							170				
							161				
	25D	24/7	124.5 - 126.5	15/21/24/28	45	45	105			Gray, wet, dense, medium to fine SAND, little coarse sand, little gravel, well graded -MARINE DEPOSIT-(SW)	
125							122				
							118				
							125				
							148				
	MD	24/0	129.5 - 131.5	18/24/37/45	61	61	Wash Ahead			Note: No recovery on first attempt or second attempt, possibly pushing gravel.	
130											

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. Bentonite drilling mud used during drilling through marine clay.

SHEET 83

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FRR-202
PIN: 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	53.2	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/16 HW
Date Start/Finish:	4/28/09 to 5/6/09	**Drilling Method:**	HW Drive to 10.0 ft/NW Drive to 168.5 ft	**Core Barrel:**	NQ - 2.0 in. I.D.
Boring Location:	E1005206, N326994 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	27.9 (5/6/09, 1720)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	27D	24/6	134.5 - 136.5	14/23/27/31	50	50	40			Gray, wet, dense, coarse to fine SAND, trace fine gravel, well graded -MARINE DEPOSIT-(SW)	
135							54				
							71				
							77				
							82				
	28D	24/10	139.5 - 141.5	18/28/45/48	73	73	62			Gray, wet, very dense, coarse to fine SAND, trace fine gravel, well graded (SW)	
140							80				
							150				
							215				
							234				
	29D	24/15	144.5 - 146.5	26/41/55/38	96	96	21			Gray, wet, very dense, medium to fine SAND, little coarse sand and fine gravel, alternating layers of well graded and poorly graded sand -MARINE DEPOSIT-(SP/SW)	
145							8				
							55	-92.8		146.0 Gray, wet, very dense, SAND, little coarse to fine gravel, trace silt, well graded, bonded -GLACIAL TILL-(SW) Note: Driller noted increase in density at approximately 146.5 ft due to drill action.	
							114				
							98				
	30D	24/6	149.5 - 151.5	34/30/64/100	94	94	Open			Gray, wet, very dense, sandy GRAVEL, trace silt, well graded, slightly bonded (GW)	
150							Open				
							50				
							62				
							89			Note: Lost all drill fluid while washing to 154.5 ft. Note: Sand came up to approximately 145.0 ft in casing overnight.	
	31D	24/7	154.5 - 156.5	6/12/18/36	30	30	38			Gray, wet, medium dense, SAND, trace coarse to fine gravel, poorly graded, no structure -GLACIAL TILL-(SP)	
155							75				
							466				

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. Bentonite drilling mud used during drilling through marine clay.

SHEET 84

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FRR-202
PIN: 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	53.2	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/16 HW
Date Start/Finish:	4/28/09 to 5/6/09	**Drilling Method:**	HW Drive to 10.0 ft/NW Drive to 168.5 ft	**Core Barrel:**	NQ - 2.0 in. I.D.
Boring Location:	E1005206, N326994 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	27.9 (5/6/09, 1720)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							158			
							123		Note: 9 ft of sand inside casing after advancing casing to 159.5 ft.	
							83			
160	MD	24/0	160.5 - 162.5	17/14/18/71	32	32	117		No Recovery, rock in tip	
							195			
							259			
							342			
	MD	24/0	164.5 - 166.5	130/100(1")			300		No Recovery	
165							112	-111.9	165.1 — Note: Cobbles and boulders encountered between 165.1 and 169.5 ft.	
							92			
							202			
	R1	54/48	169.5 - 174.0	RQD = 57%					Note: Possible boulder, gravel seam at approximately 170.1 to 170.4 ft.	
170							NQ	-117.2	170.4 — see below	
	R2	54/54	174.0 - 178.5	RQD = 73%						
175							NQ			
								-125.3		
180									178.5 — Bottom of Exploration at 178.5 feet Below Ground Surface.	

170.4: Top of Bedrock at El. -117.2 ft.
Gray, fine grained to aphanitic, metamorphic SCHIST. Hard to very hard, fresh. Joints are low angle to steeply dipping, close, tight to partly open, some silt infilling.
Rock Mass Quality=fair
-BERWICK FORMATION-
R1:Core Times (min:sec):
169.5-170.5' (2:00), 170.5-171.5' (3:00), 171.5-172.5' (3:00), 172.5-173.5' (3:00), 173.5-174.0' (1:00)
Note: Approximately 4.5 ft of soil inside borehole after removing drill rods.
Gray, fine grained to aphanitic, metamorphic SCHIST. Hard to very hard, fresh to slightly weathered. Joints are low angle to steeply dipping, very close to moderately close, tight to partly open, some silt infilling, frequent calcite/ quartz veings.
Rock Mass Quality=fair
-BERWICK FORMATION-
R2:Core Times (min:sec):
174.0-175.0' (2:00), 175.0-176.0' (3:00), 176.0-177.0' (3:00), 177.0-178.0' (3:00), 178.0-178.5' (2:00)

Bottom of Exploration at 178.5 feet Below Ground Surface.

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. Bentonite drilling mud used during drilling through marine clay.

SHEET 85

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FRR-203
PIN: 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	63.3	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 HW
Date Start/Finish:	4/6/09 to 4/7/09	**Drilling Method:**	HW Drive to 35.0 ft	**Core Barrel:**	--
Boring Location:	E1005180, N327094 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	5.2 (4/7/09, 1400)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/18	0.0 - 2.0	4/7/6/4	13	13	Open			Brown to black, dry to moist, medium dense, fine SAND, little medium sand, little silt, trace coarse sand, little coarse gravel, poorly graded -FILL-(SP)	
								61.9		1.4	
	2D	24/14	2.0 - 4.0	3/5/10/10	15	15				Brown, moist, stiff, SILT, little fine sand, trace medium sand -MARINE DEPOSIT-(ML) Brown to olive-brown, moist to wet, stiff, SILT, some fine sand, little medium sand (ML)	
								60.1		3.2	
	3D	24/21	4.0 - 6.0	10/7/9/11	16	16		58.7		Brown, moist, medium dense, fine SAND, little medium sand, trace silt, poorly graded -MARINE DEPOSIT-(SP)	
5										4.6	
	4D	24/16	6.0 - 8.0	9/9/6/6	15	15	30			Brown, moist to wet, very stiff to medium dense, sandy SILT, little medium sand; to silty SAND, mottled -MARINE DEPOSIT-(ML/SM) Brown, wet, medium dense, fine SAND, little silt, poorly graded, mottled (SP)	
							19				
	5D	24/15	8.0 - 10.0	7/4/4/5	8	8	21			Brown, wet, loose, silty SAND, poorly graded, mottled, soft clay in tip of sampler (SP-SM)	
							23				
10	6D	24/17	10.0 - 12.0	1/1/1/1	2	2	5	52.8		10.5 Gray, wet, medium stiff, silty CLAY, little fine sand, occasional sand layers throughout -MARINE DEPOSIT-(CL)	
							6				
							6				
							4				
							3				
15	7D V1A	24/24	15.0 - 17.0 ~~15.6 - 16.0~~	push thru vane ~~Su=640/120 psf~~			WOH			Gray, wet, medium stiff, silty CLAY, little sand, sand layer from 16.2 to 16. 5 ft (CL) 65x130 mm vane raw torque readings: V1A: 270/50 in-lbs V1B: 220/30 in-lbs	
	V1B		16.6 - 17.0	Su=520/70 psf			3				
							WOH				
							WOC				
							4				
20	1U	24/24	20.0 - 22.0				2				C#CRC-8 WC=53.2% LL=48 PL=23 PI=25
							2				
	8D V2A	24/24	22.0 - 24.0 ~~22.6 - 23.0~~	push thru vane ~~Su=405/25 psf~~			WOH			Gray, wet, soft, silty CLAY (CL) 65x130 mm vane raw torque readings: V2A: 170/10 in-lbs V2B: 170/10 in-lbs	
	V2B		23.6 - 24.0	Su=405/25 psf			WOC				
							WOH				
25							WOC				

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. Bentonite drilling mud used during drilling through marine clay.

SHEET 86

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Proposed Replacement Bridge over Presumpscot River and MCRR	**Location:**	Routes 26/100 Falmouth, Maine	**Boring No.:**	BB-FRR-203
				PIN:	15094.00
Driller:	Maine Test Borings	**Elevation (ft.)**	63.3	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 HW
Date Start/Finish:	4/6/09 to 4/7/09	**Drilling Method:**	HW Drive to 35.0 ft	**Core Barrel:**	--
Boring Location:	E1005180, N327094 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	5.2 (4/7/09, 1400)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							WOH				
30	9D	24/24	30.0 - 32.0	WOR/WOR/WOR/ WOR			WOC			Gray, wet, soft, silty CLAY -MARINE DEPOSIT-(CL)	
35	10D	24/24	35.5 - 37.5	push thru vane			Open			Gray, wet, soft to medium stiff, silty CLAY (CL) 65x135 mm vane raw torque readings: V3A: 160/10 in-lbs V3B: 250/10 in-lbs	
	V3A		36.1 - 36.5	Su=380/25 psf							
	V3B		37.1 - 37.5	Su=595/25 psf							
40	2U	24/24	40.0 - 42.0								C#CRC-3 WC=47.9% LL=37 PL=21 PI=16
	11D	24/24	42.0 - 44.0	push thru vane						Gray, wet, medium stiff, silty CLAY (CL) 65x130 mm vane raw torque readings: V4A: 230/10 in-lbs V4B: 350/10 in-lbs	
	V4A		42.6 - 43.0	Su=545/25 psf							
	V4B		43.6 - 44.0	Su=830/25 psf							
45	12D	24/24	45.0 - 47.0	WOR/WOR/WOR/ WOR						Gray, wet, medium stiff, silty CLAY, trace fine sand -MARINE DEPOSIT-(CL)	
50							Open				
	13D	24/24	51.0 - 53.0	push thru vane						Gray, wet, soft to medium stiff, silty CLAY (CL) 55x110 mm vane raw torque readings:	
	V5A		51.6 - 52.0	Su=505/40 psf							

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. Bentonite drilling mud used during drilling through marine clay.

SHEET 87

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Proposed Replacement Bridge over Presumpscot River and MCRR Location: Routes 26/100 Falmouth, Maine	Boring No.: BB-FRR-203 PIN: 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	63.3	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 HW
Date Start/Finish:	4/6/09 to 4/7/09	**Drilling Method:**	HW Drive to 35.0 ft	**Core Barrel:**	--
Boring Location:	E1005180, N327094 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	5.2 (4/7/09, 1400)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	V5B		52.6 - 53.0	Su=445/60 psf						V5A: 130/10 in-lbs V5B: 115/15 in-lbs -MARINE DEPOSIT-(CL)	
55	14D	24/24	55.0 - 57.0	push thru vane						Gray to dark gray, wet, medium stiff, silty CLAY (CL) 55x110 mm vane raw torque readings: V6A: 150/5 in-lbs V6B: 180/5 in-lbs	
	V6A		55.6 - 56.0	Su=580/20 psf							
	V6B		56.6 - 57.0	Su=700/20 psf							
60	15D	24/24	60.0 - 62.0	WOR/WOR/WOR/ WOR						Gray, wet, medium stiff, silty CLAY, little sand, sandy clay layer from approximately 60.6 to 61.1 ft (CL)	
65	16D	24/18	65.0 - 67.0	push thru vane						Gray, wet, medium stiff, silty CLAY (CL) 55x110 mm vane raw torque readings: V7A: 220/5 in-lbs V7B: 165/5 in-lbs	
	V7A		65.6 - 66.0	Su=855/20 psf							
	V7B		66.6 - 67.0	Su=640/20 psf							
70	17D	24/21	70.0 - 72.0	WOR/WOR/WOR/ WOR						Gray, wet, medium stiff, silty CLAY -MARINE DEPOSIT-(CL)	
75	18D	24/24	75.0 - 77.0	push thru vane			Open			Gray, wet, medium stiff to stiff, silty CLAY, trace fine sand (CL) 55x110 mm vane raw torque readings: V8A: 275/5 in-lbs V8B: 240/10 in-lbs	
	V8A		75.6 - 76.0	Su=1,065/20 psf							
	V8B		76.6 - 77.0	Su=930/40 psf							

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. Bentonite drilling mud used during drilling through marine clay.

SHEET 88

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge over Presumpscot River and MCRR
Location: Routes 26/100, Falmouth, Maine

Boring No.: BB-FRR-203
PIN: 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	63.3	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 HW
Date Start/Finish:	4/6/09 to 4/7/09	**Drilling Method:**	HW Drive to 35.0 ft	**Core Barrel:**	--
Boring Location:	E1005180, N327094 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	5.2 (4/7/09, 1400)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
80	19D	24/20	80.0 - 82.0	WOR/WOR/WOR/ WOR						Gray, wet, medium stiff to stiff, silty CLAY, little sand, occasional fine sand seams -MARINE DEPOSIT-(CL)	
								-21.2		84.5	
85	20D	24/15	85.0 - 87.0	8/4/12/16	16	16				Note: Driller noted change in density from drill action. Gray, wet, medium dense, fine SAND, some clay, little medium sand, trace coarse sand, poorly graded, one sandy clay layer from approximately 85.5 to 85.8 ft -MARINE DEPOSIT-(SP)	
90	21D	24/12	90.0 - 92.0	16/17/19/38	36	36				Gray, wet, dense, fine SAND, little medium sand, trace silt, poorly graded (SP)	
								-28.7		92.0	
										Bottom of Exploration at 92.0 feet Below Ground Surface. Note: No Refusal Encountered	
95											
100											

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. Bentonite drilling mud used during drilling through marine clay.

SHEET 89

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FRR-204
PIN: 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	64.4	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 HW
Date Start/Finish:	4/8/09	**Drilling Method:**	HW Drive to 15.0 ft	**Core Barrel:**	--
Boring Location:	E1005173, N327147 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	5.1 (4/8/09, 1335)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/16	0.3 - 2.3	4/5/4/4	9	9	Open	64.1		-BITUMINOUS CONCRETE- 0.3	
								63.1		Dark brown to black, moist, loose, medium to fine SAND, some coarse sand, little silt -FILL-(SW) 1.3	
	2D	24/16	2.3 - 4.3	4/8/6/11	14	14		62.1		Brown, moist, loose, silty medium to fine SAND, trace coarse sand, trace coarse gravel (SP) 2.3	
								60.4		Brown, moist, medium stiff, laminated, SILT, trace fine sand (ML) 4.0	
	3D	24/12	4.3 - 6.3	9/12/11/13	23	23				Brown, moist, medium dense, fine SAND, trace medium sand -MARINE DEPOSIT-(SP)	
5											
	4D	24/20	6.3 - 8.3	5/6/7/7	13	13				Brown, wet, medium dense, medium to fine SAND, little coarse sand (SP)	
	5D	24/24	8.3 - 10.3	4/5/6/4	11	11				Brown, wet, medium dense, medium to fine SAND, trace coarse sand (SP)	
10								54.4		10.0	
	6D	24/24	10.3 - 12.3	3/3/2/2	5	5	11	53.8		Brown, wet, medium stiff, fine-sandy SILT -MARINE DEPOSIT-(ML) 10.6	
							10			Brown, wet, medium stiff, silty CLAY -MARINE DEPOSIT-(CL)	
							12				
							10				
							12				
15	7D	24/24	15.0 - 17.0	1/1/WOH/WOH	1	1	Open			Gray, wet, very soft, silty CLAY, occasional fine sand layers from 15.0 to 16.0 ft (CL)	
20	8D	24/24	20.0 - 22.0	WOR/WOR/WOH/ WOH						Gray, wet, very soft, silty CLAY (CL)	
25	9D	24/24	25.0 - 27.0	WOR/WOR/WOR/ WOR			Open			Gray, wet, very soft, silty CLAY -MARINE DEPOSIT-(CL)	

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 90

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Proposed Replacement Bridge over Presumpscot River and MCRR **Location:** Routes 26/100 Falmouth, Maine	**Boring No.:** BB-FRR-204 **PIN:** 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	64.4	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 HW
Date Start/Finish:	4/8/09	**Drilling Method:**	HW Drive to 15.0 ft	**Core Barrel:**	--
Boring Location:	E1005173, N327147 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	5.1 (4/8/09, 1335)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
30	10D	24/12	30.0 - 32.0	WOR/WOR/WOR/ WOR						Gray, wet, very soft, silty CLAY, occasional black streaks (CL)	
35	11D	24/12	35.0 - 37.0	WOR/WOR/WOR/ WOR						Gray, wet, very soft, silty CLAY, occasional black streaks (CL)	
40	12D	24/16	40.0 - 42.0	WOR/WOR/WOR/ WOR						Gray, wet, very soft, silty CLAY (CL)	
45	13D	24/18	45.0 - 47.0	WOR/WOR/WOR/ WOR						Gray, wet, very soft, silty CLAY, frequent black streaks/specks -MARINE DEPOSIT-(CL)	
50	14D	24/12	50.0 - 52.0	WOR/WOR/WOR/ WOR			Open			Gray, wet, very soft, silty CLAY, frequent black streaks/specks -MARINE DEPOSIT-(CL)	

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 91

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge over Presumpscot River and MCRR
Location: Routes 26/100 Falmouth, Maine

Boring No.: BB-FRR-204
PIN: 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	64.4	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 HW
Date Start/Finish:	4/8/09	**Drilling Method:**	HW Drive to 15.0 ft	**Core Barrel:**	--
Boring Location:	E1005173, N327147 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	5.1 (4/8/09, 1335)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
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R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
55	15D	24/20	55.0 - 57.0	WOR/WOR/WOR/ WOR						Gray, wet, very soft, silty CLAY, frequent black streaks/specks (CL)	
60	16D	24/24	60.0 - 62.0	WOR/WOR/WOR/ WOR						Gray, wet, very soft, silty CLAY, frequent black streaking throughout (CL)	
65	17D	24/24	65.0 - 67.0	WOR/WOR/WOR/ WOR						Gray, wet, very soft, silty CLAY to SILT (CL/ML)	
70	18D	24/21	70.0 - 72.0	WOR/WOR/WOR/ WOR						Gray, wet, very soft, silty CLAY -MARINE DEPOSIT-(CL)	
75	19D	24/21	75.0 - 77.0	WOR/WOR/WOR/ WOR			Open			Gray, wet, very soft, silty CLAY -MARINE DEPOSIT-(CL)	

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 92

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Boring No.: BB-FRR-204

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Proposed Replacement Bridge over Presumpscot River and MCRR
Location: Routes 26/100, Falmouth, Maine

Boring No.: BB-FRR-204
PIN: 15094.00

Driller:	Maine Test Borings	**Elevation (ft.)**	64.4	**Auger ID/OD:**	--
Operator:	R. Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. I.D.
Logged By:	E. Beirne	**Rig Type:**	CME 550X	**Hammer Wt./Fall:**	140/30 SS - 300/30 HW
Date Start/Finish:	4/8/09	**Drilling Method:**	HW Drive to 15.0 ft	**Core Barrel:**	--
Boring Location:	E1005173, N327147 (See Plan)	**Casing ID/OD:**	HW - 4.0 in. I.D.	**Water Level*:**	5.1 (4/8/09, 1335)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
80	20D	24/24	80.0 - 82.0	WOR/WOR/WOR/9						Gray, wet, very soft, silty CLAY, little fine sand (CL)	
								-17.1		81.5 Gray, wet, very soft, silty CLAY, some fine sand, trace medium sand -MARINE DEPOSIT-(CL)	
85	21D	24/11	85.0 - 87.0	26/31/29/27	60	60		-20.6		85.0 Gray, wet, very dense, fine SAND, little medium sand, trace silt, poorly graded -MARINE DEPOSIT-(SP)	
90	22D	24/8	90.0 - 92.0	12/15/15/15	30	30				Gray, wet, medium dense, medium to fine SAND, trace silt, trace coarse sand, poorly graded (SP)	
								-27.6		92.0 **Bottom of Exploration at 92.0 feet Below Ground Surface.** Note: No Refusal Encountered	
95											
100											

Remarks:

1. As-drilled coordinates of test boring determined by MaineDOT and provided in NAD83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.

SHEET 93

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

HALEY & ALDRICH

OBSERVATION WELL INSTALLATION REPORT

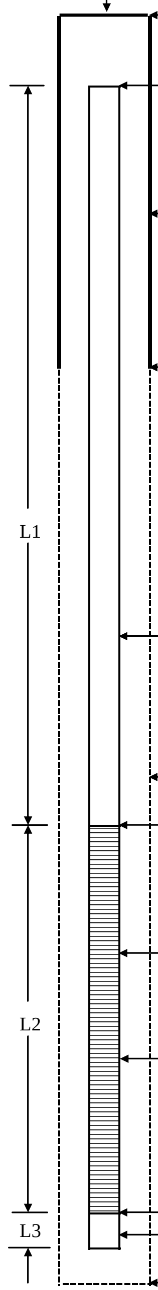
Well No. OW-1
Boring No. BB-FPR-101

PROJECT	Proposed Replacement Bridge over Presumpscot River and MCRR	**H&A FILE NO.**	35524-000
LOCATION	Routes 26/100, Falmouth, Maine	**PROJECT MGR.**	W. Chadbourne
CLIENT	Maine Department of Transportation	**FIELD REP.**	E. Beirne
CONTRACTOR	Maine Test Borings, Inc.	**DATE INSTALLED**	10/22/2008
DRILLER	B. Enos	**WATER LEVEL**	

Ground El. 24.7 ft
El. Datum NAVD 88
Location E1005263.6, N326663.1 (See Plan)
☑ **Guard Pipe**
☐ **Roadway Box**

SOIL/ROCK CONDITIONS	BOREHOLE BACKFILL
TOPSOIL	BENTONITE CHIPS
2.3	
-ALLUVIAL DEPOSIT- (CLAY)	2.5
15.0	
-ALLUVIAL DEPOSIT- (SAND)	FILTER SAND
23.0	
	26.0
-MARINE DEPOSIT- (CLAY)	DRILL CUTTINGS
	37.9
40.5	
-MARINE DEPOSIT- (SAND)	CAVE
92.3	
-GLACIAL TILL-	
125.1	125.1

(Bottom of Exploration)
(Numbers refer to depth from ground surface in feet)



Type of protective cover	Steel Lock/Cap	
Height of top of guard pipe above ground surface	2.9	ft
Height of top of riser pipe above ground surface	2.8	ft
Type of protective casing:	Steel Guardpipe	
Length	5.2	ft
Inside Diameter	3.5	in
Depth of bottom of guard pipe	2.3	ft

Type of Seals	Top of Seal (ft)	Thickness (ft)
Bentonite Seal	0.0	2.5

Type of riser pipe:	Schedule 40 PVC	
Inside diameter of riser pipe	2.0	in
Type of backfill around riser	Filter Sand/Bentonite Chips	
Diameter of borehole	4.0	in
Depth to top of well screen	3.5	ft
Type of screen	Slotted Schedule 40 PVC	
Screen gauge or size of openings	0.10	in
Diameter of screen	2.0	in
Type of backfill around screen	Filter Sand	
Depth of bottom of well screen	23.3	ft
Bottom of Silt trap	23.5	ft
Depth of bottom of borehole	125.1	ft

(Not to Scale)

6.3 ft + 19.8 ft + 0.2 ft = 26.3 ft
Riser Pay Length (L1) + Length of screen (L2) + Length of silt trap (L3) = Pay length

COMMENTS:

Form 2007

GROUNDWATER MONITORING REPORT

HALEY & ALDRICH

OW/PZ NUMBER
BB-FPR-101

PROJECT	Proposed Replacement Bridge over Presumpscot River and MCRR	**H&A FILE NO.**	35524-000
LOCATION	Routes 26/100, Falmouth, Maine	**PROJECT MGR.**	W. Chadbourne
CLIENT	Maine Department of Transportation	**FIELD REP.**	E. Beirne
CONTRACTOR	Maine Test Borings, Inc.	**DATE**	10/22/2008
ELEVATION SUBTRAHEND	24.7 (NAVD 88)		

Date	Time	Elapsed Time (days)	Depth of Water from Top of Riser	Elevation of Water	Remarks	Read By
10/30/2008	1205	8	5.7	21.8		ECB
10/31/2008	1026	9	5.9	21.6		ECB
11/5/2008	1408	14	6.3	21.2		ECB
11/6/2008	1005	15	6.4	21.1		ECB
11/7/2008	837	16	5.8	21.7	after rain storm 11/6 PM	ECB
11/26/2008	1311	35	3.0	24.5	after rain storm on 11/25, bailed out	BCS
11/26/2008	1323	35	17.2	10.3	reading after well bailed out	BCS
11/26/2008	1325	35	15.3	12.2		BCS
11/26/2008	1329	35	12.3	15.2		BCS
11/26/2008	1341	35	8.4	19.1		BCS
11/26/2008	1347	35	7.1	20.4		BCS
11/26/2008	1350	35	6.6	20.9		BCS
2/20/2009	1045	121	6.4	21.1		BCS

HALEY& ALDRICH

OBSERVATION WELL INSTALLATION REPORT

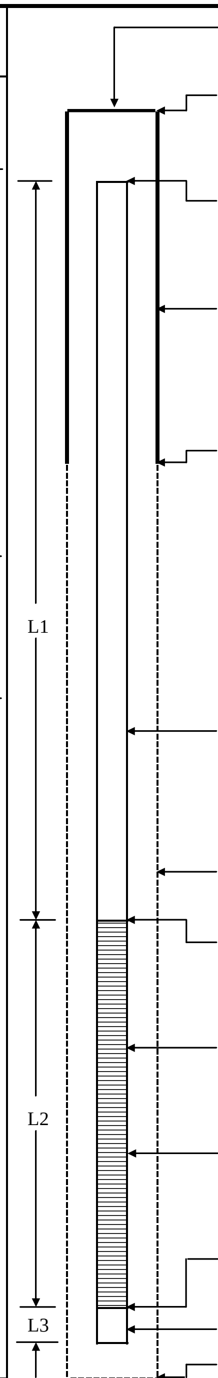
Well No. **OW-2**
Boring No. **BB-FPR-102**

PROJECT	Proposed Replacement Bridge over Presumpscot River and MCRR	**H&A FILE NO.**	35524-000
LOCATION	Routes 26/100, Falmouth, Maine	**PROJECT MGR.**	W. Chadbourne
CLIENT	Maine Department of Transportation	**FIELD REP.**	B. Steinert
CONTRACTOR	Maine Test Borings, Inc.	**DATE INSTALLED**	10/28/2008
DRILLER	B. Enos	**WATER LEVEL**	

Ground El. 26.3 ft
El. Datum NAVD 88
Location E1005296.7, N326554.8 (See Plan)
☑ **Guard Pipe**
☐ **Roadway Box**

SOIL/ROCK CONDITIONS	**BOREHOLE BACKFILL**
-TOPSOIL-	BENTONITE CHIPS
2.7	
	5.0
-ALLUVIAL DEPOSIT- SAND/SILT	FILTER SAND
27.5	
	29.0
-MARINE DEPOSIT- (CLAY)	DRILL CUTTINGS
	37.0
45.2	
-MARINE DEPOSIT- (SAND)	CAVE
92.5	
GLACIAL TILL	
130.0	
WEATHERED BEDROCK	
141.5	141.5

(Bottom of Exploration)
(Numbers refer to depth from ground surface in feet)



Type of protective cover/lock	Steel Lock/Cap	
Height of top of guard pipe above ground surface	3.0	ft
Height of top of riser pipe above ground surface	3.0	ft
Type of protective casing:	Steel Guardpipe	
Length	5.25	ft
Inside Diameter	3.5	in
Depth of bottom of guard pipe	2.25	ft

Type of Seals	**Top of Seal (ft)**	**Thickness (ft)**
Bentonite Seal	0.0	5.0

Type of riser pipe:	Schedule 40 PVC	
Inside diameter of riser pipe	3.0	in
Type of backfill around riser	Filter Sand/Bentonite Chips	
Diameter of borehole	4.0	in
Depth to top of well screen	7.0	ft
Type of screen	Slotted Schedule 40 PVC	
Screen gauge or size of openings	0.010	in
Diameter of screen	3.0	in
Type of backfill around screen	Filter Sand	
Depth of bottom of well screen	27.0	ft
Bottom of Silt trap	27.0	ft
Depth of bottom of borehole	141.5	ft

(Not to Scale)

10 ft + 20 ft + 0 ft = 30 ft
Riser Pay Length (L1) + Length of screen (L2) + Length of silt trap (L3) = Pay length

COMMENTS:

Form 2007

GROUNDWATER MONITORING REPORT

HALEY & ALDRICH

OW/PZ NUMBER

BB-FPR-102

Page 1 **of** 1

PROJECT	Proposed Replacement Bridge over Presumpscot River and MCRR	**H&A FILE NO.**	35524-000
LOCATION	Routes 26/100, Falmouth, Maine	**PROJECT MGR.**	W. Chadbourne
CLIENT	Maine Department of Transportation	**FIELD REP.**	B. Steinert
CONTRACTOR	Maine Test Borings, Inc.	**DATE**	10/28/2008
ELEVATION SUBTRAHEND	26.3 (NAVD 88)		

Date	Time	Elapsed Time (days)	Depth of Water from Top of Riser	Elevation of Water	Remarks	Read By
10/30/2008	1213	2	12.3	17.0		ECB
10/31/2008	1030	3	12.5	16.8		ECB
11/5/2008	1413	8	12.9	16.4		ECB
11/6/2008	1008	9	12.9	16.4		ECB
11/7/2008	833	10	12.4	16.9	after rain storm 11/6 PM	ECB
11/26/2008	1313	29	5.3	24.0	after rain storm on 11/25, bailed out	BCS
11/26/2008	1338	29	5.6	23.7	after bailed out, approximate river level	BCS
11/26/2008	1342	29	5.5	23.8	approximate river level	BCS
11/26/2008	1346	29	5.5	23.8	approximate river level	BCS
11/26/2008	1348	29	5.5	23.8	approximate river level	BCS
2/18/2009	1135	113	13.7	15.6	approximate river level	ECB
2/20/2009	1034	115	13.3	16.0	approximate river level	BCS

GeoTesting
express
a subsidiary of Geocomp Corporation

1145 Massachusetts Avenue
Boxborough, MA 01719
978 635 0424 Tel
978 635 0266 Fax

Transmittal

TO:

Mr. Bryan Steinert
Haley & Aldrich, Inc.
75 Washington Avenue, Suite 203
Portland, ME 04101-2617

DATE: 11/24/08	GTX NO: 8629
RE: Presumpscot River Bridge project	
H&A Project No. 35524-000	

COPIES	DATE	DESCRIPTION
1	11/24/08	**November 2008 Laboratory Test Reports**

REMARKS:

SIGNED: Joe
Joe Tomei – Laboratory Manager

APPROVED BY: Mark Dobday
Mark Dobday – Laboratory Manager

GeoTesting express
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Boston
Atlanta
New York

www.geocomp.com/geotesting

November 24, 2008

Mr. Bryan Steinert
Haley & Aldrich, Inc.
75 Washington Avenue, Suite 203
Portland, ME 04101-2617

Re: Presumpscot River Bridge Project (GTX-8629)

Dear Mr. Steinert:

Enclosed are the test results you requested for the above referenced project. GeoTesting Express, Inc. (GTX) received four Shelby tube soil samples from you between October 30 and November 12, 2008. These samples were labeled as follows:

BB-FPR-101, U1 (38.5-40.5 ft)
BB-FPR-102, U1 (31.5-33.5 ft)
BB-FRR-102, U1 (32-34 ft)
BB-FRR-102, U2 (54-56 ft)

GTX performed the following tests on each of these samples:

Moisture Content (ASTM D 2216)
Atterberg Limits (ASTM D 4318)
CRS Consolidation (ASTM D 4186)
X-Ray Report

As requested, the x-ray tests were performed first and reports were sent to H&A. After review of the x-ray reports, H&A provided GTX locations within the tubes to cut specimens for testing. Copies of your test requests and test locations are attached.

The results presented in this report apply only to the items tested. This report shall not be reproduced except in full, without written approval from GeoTesting Express. The remainder of these samples will be retained for a period of sixty (60) days and will then be discarded unless otherwise notified by you. Please call me if you have any questions or require additional information. Thank you for allowing GeoTesting Express the opportunity of providing you with testing services. We look forward to working with you again in the future.

Respectfully yours,

Joe Tomei
Laboratory Manager

GeoTesting Express, Inc. | 1145 Massachusetts Ave. | Boxborough, MA 01719 | Toll Free 800 434 1062 | Fax 978 635 0266

GeoTesting
express
a subsidiary of Geocomp Corporation

1145 Massachusetts Avenue
Boxborough, MA 01719
978 635 0424 Tel
978 635 0266 Fax

Geotechnical Test Report

November 24, 2008

GTX-8629
Presumpscot River Bridge Project

Falmouth/Portland, ME

Prepared for:

HALEY & ALDRICH

GeoTesting express
a subsidiary of Geocomp Corporation

Client: Haley & Aldrich, Inc.			
Project: Presumpscot River Bridge			
Location: Falmouth/Portland ME		Project No:	GTX-8629
Boring ID: BB-FPR-101	Sample Type: tube	Tested By:	ap
Sample ID:U1	Test Date: 11/24/08	Checked By:	jdt
Depth : 38.5-40.5 ft	Sample Id: 67203		
Test Comment: ---			
Sample Description: Moist, dark gray clay			
Sample Comment: ---			

Moisture Content of Soil - ASTM D 2216-05

Boring ID	Sample ID	Depth	Description	Moisture Content,%
BB-FPR-101	U1	38.5-40.5 ft	Moist, dark gray clay	31.2

Notes: Temperature of Drying : 110º Celsius

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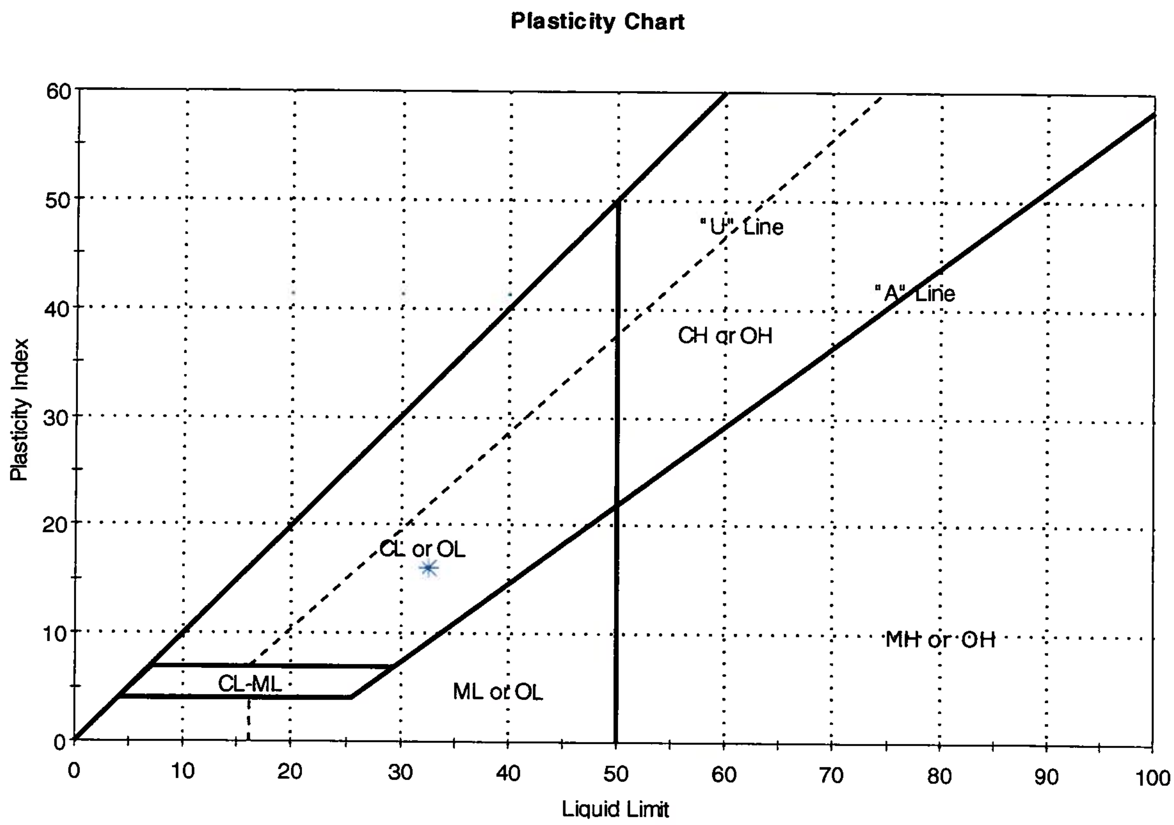
Client:	Haley & Aldrich, Inc.		
Project:	Presumpscot River Bridge		
Location:	Falmouth/Portland ME	Project No:	GTX-8629
Boring ID: BB-FPR-101	Sample Type: tube	Tested By:	ap
Sample ID:U1	Test Date: 11/17/08	Checked By:	jdt
Depth : 38.5-40.5 ft	Test Id: 141016		
Test Comment:	---		
Sample Description:	Moist, dark gray clay		
Sample Comment:	---		

Atterberg Limits - ASTM D 4318-05



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
*	U1	B-FPR-10	38.5-40.5 ft	31	33	17	16	1	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

Dilentancy: SLOW

Toughness: LOW

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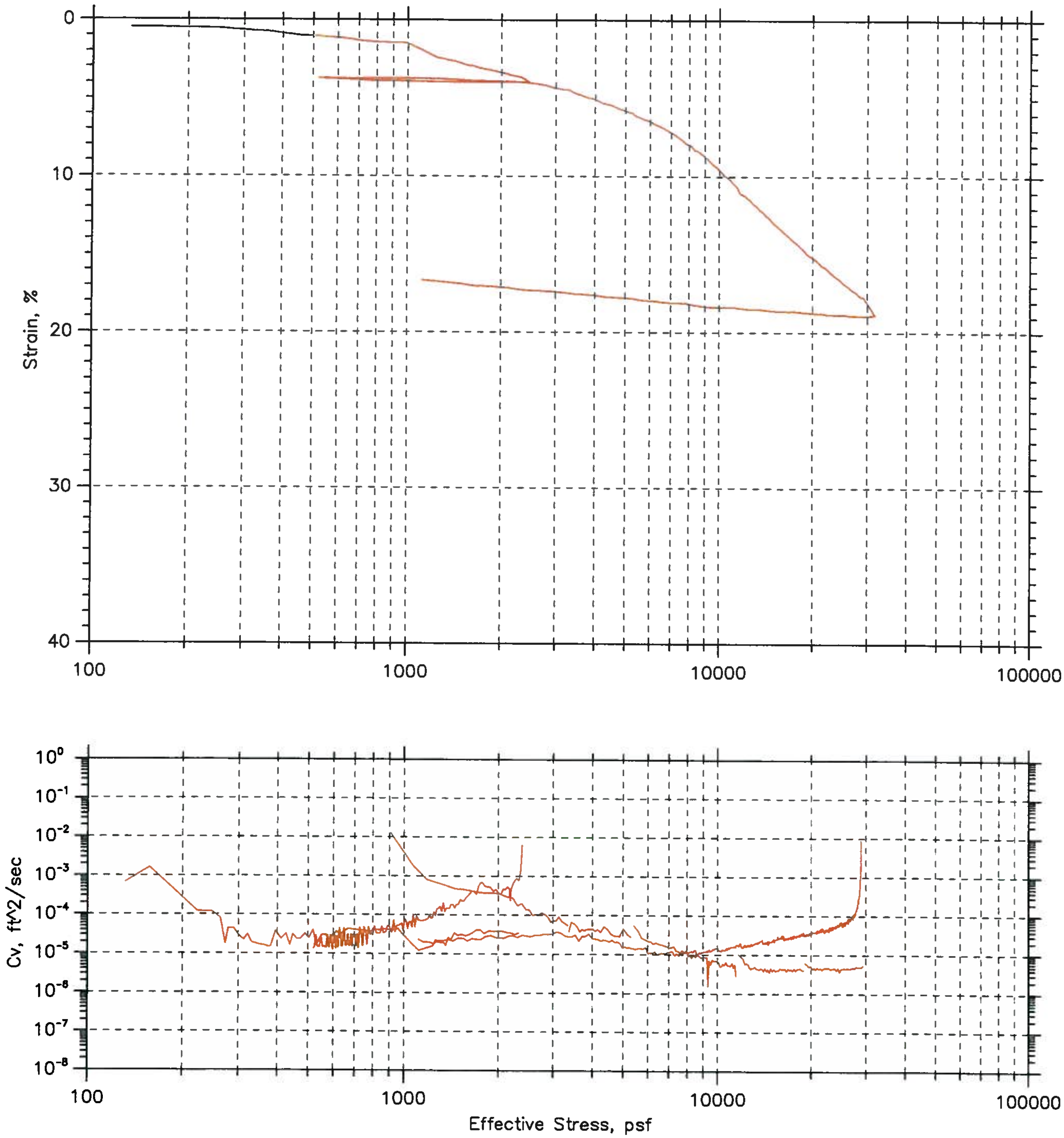
Constant Rate of Consolidation
Constant Strain Rate by ASTM D4186
Summary Report



Project: Presumpscot River	Location: Falmouth/Portland, ME	Project No.: GTX-8629
Boring No.: BB-FPR-101	Tested By: md	Checked By: jdt
Sample No.: U-1	Test Date: 11/16/08	Depth: 38.5-40.5
Test No.: CRC-3	Sample Type: tube	Elevation: ---
Description: Moist, dark gray clay		
Remarks: System S		

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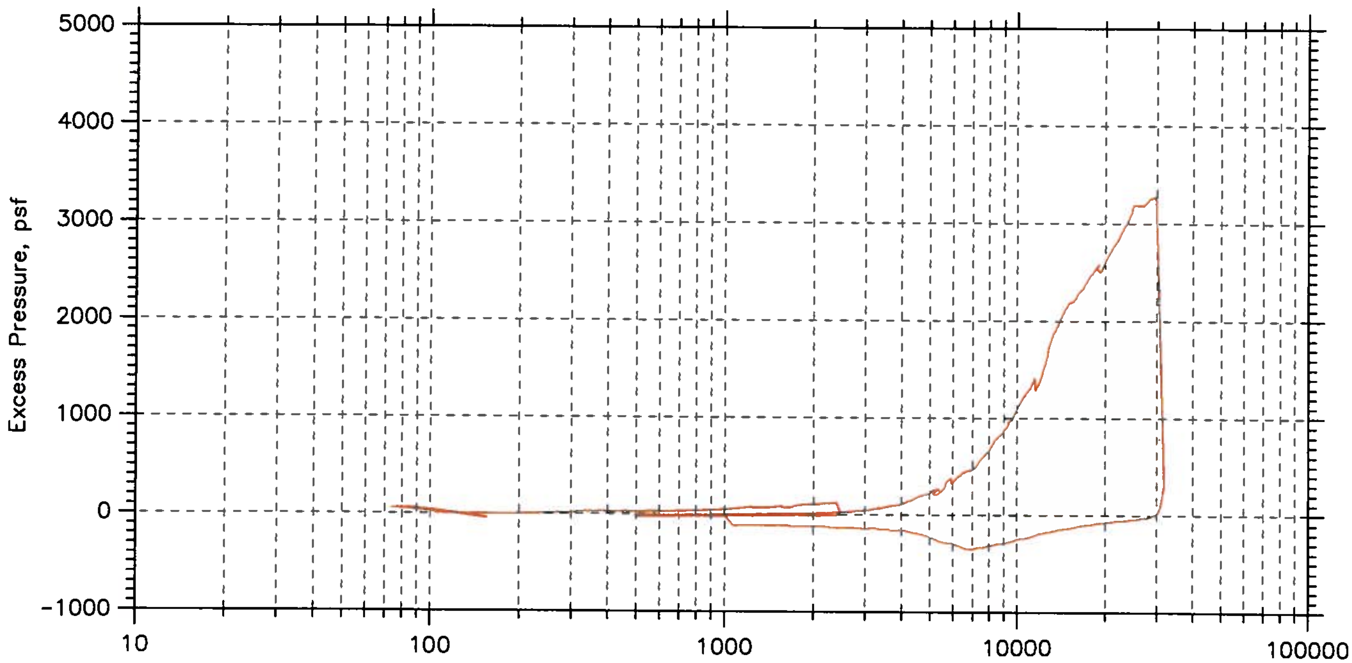
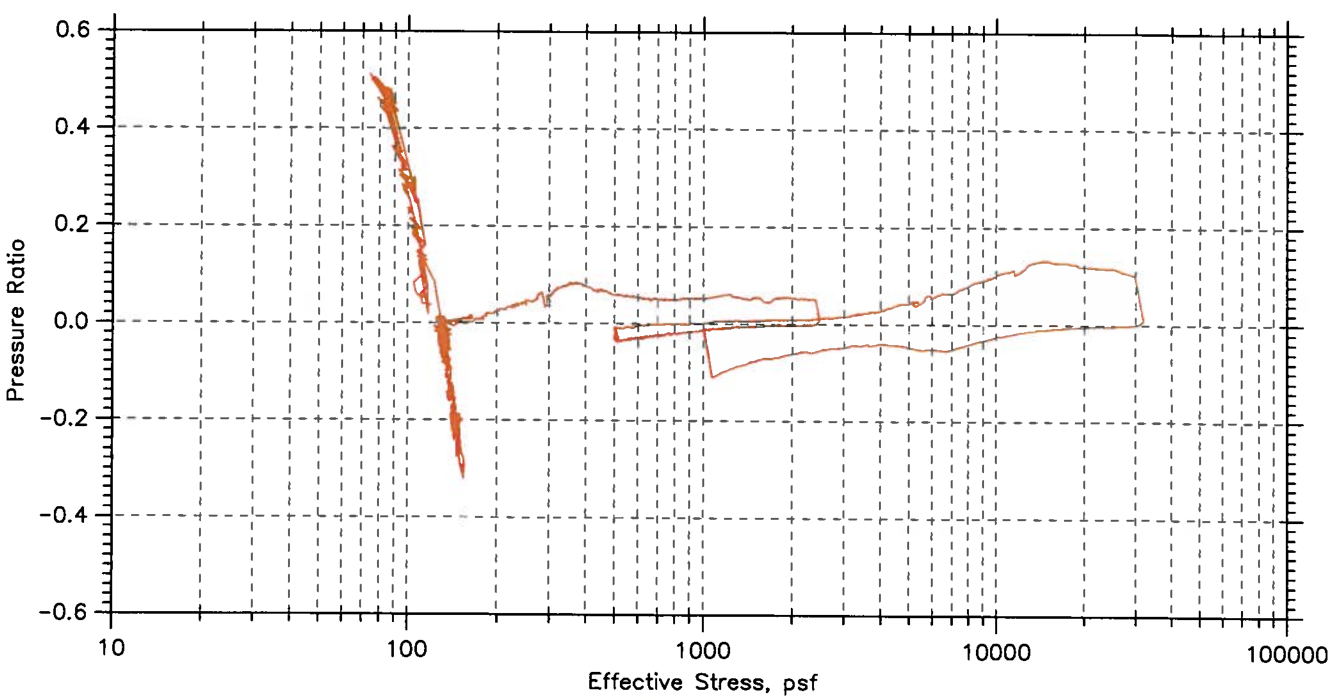
Constant Rate of Consolidation

Constant Strain Rate by ASTM D4186

Pressure Curves





Project: Presumpscot River	Location: Falmouth/Portland, ME	Project No.: GTX-8629
Boring No.: BB-FPR-101	Tested By: md	Checked By: jdt
Sample No.: U-1	Test Date: 11/16/08	Depth: 38.5-40.5
Test No.: CRC-3	Sample Type: tube	Elevation: ---
Description: Moist, dark gray clay		
Remarks: System S		

CRC TEST DATA

Project: Presumpscot River
Boring No.: BB-FPR-101
Sample No.: U-1
Test No.: CRC-3

Location: Falmouth/Portland, ME
Tested By: md
Test Date: 11/16/08
Sample Type: tube

Project No.: GTX-8629
Checked By: jdt
Depth: 38.5-40.5
Elevation: ---

Soil Description: Moist, dark gray clay
Remarks: System S

Estimated Specific Gravity: 2.89
Initial Void Ratio: 1.05
Final Void Ratio: 0.72

Liquid Limit: 33
Plastic Limit: 17
Plasticity Index: 16

Initial Height: 1.00 in
Specimen Diameter: 2.50 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	3455	RING		301
Wt. Container + Wet Soil, gm	134.35	362.68	352.36	149.17
Wt. Container + Dry Soil, gm	101.92	324.3	324.3	121.17
Wt. Container, gm	8.29	211.06	211.06	8.17
Wt. Dry Soil, gm	93.63	113.24	113.24	113
Water Content, %	34.64	33.89	24.78	24.78
Void Ratio	---	1.05	0.72	---
Degree of Saturation, %	---	93.10	100.00	---
Dry Unit Weight, pcf	---	87.884	105.08	---

Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

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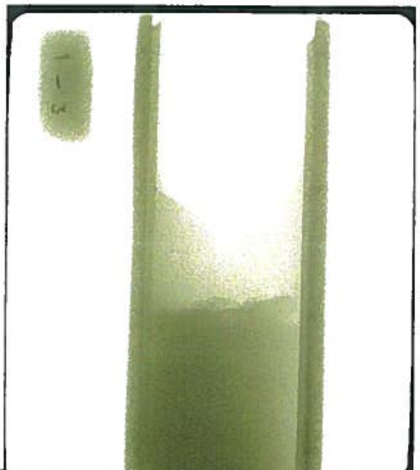
Client:	Haley & Aldrich, Inc.
Project Name:	Presumpscot River Bridge
Project Location:	Falmouth & Portland Maine
GTX #:	8629
Test Date:	11/03/08
Tested By:	md
Checked By:	jdt
Boring ID:	BB-FPR-101
Sample ID:	U-1
Depth, ft:	38.5-40.5

X-Ray of Soil Sample by ASTM D 4452



Top of Tube

Middle of Tube

Bottom of Tube

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Client: Haley & Aldrich, Inc.			
Project: Presumpscot River Bridge			
Location: Falmouth/Portland ME		Project No:	GTX-8629
Boring ID: BB-FRR-102	Sample Type: tube	Tested By:	ap
Sample ID:U1	Test Date: 11/24/08	Checked By:	jdt
Depth : 32.0-34.0 ft	Sample Id: 67202		
Test Comment: ---			
Sample Description: Wet, dark gray clay			
Sample Comment: ---			

Moisture Content of Soil - ASTM D 2216-05

Boring ID	Sample ID	Depth	Description	Moisture Content,%
BB-FRR-102	U1	32.0-34.0 ft	Wet, dark gray clay	40.3

Notes: Temperature of Drying : 110º Celsius

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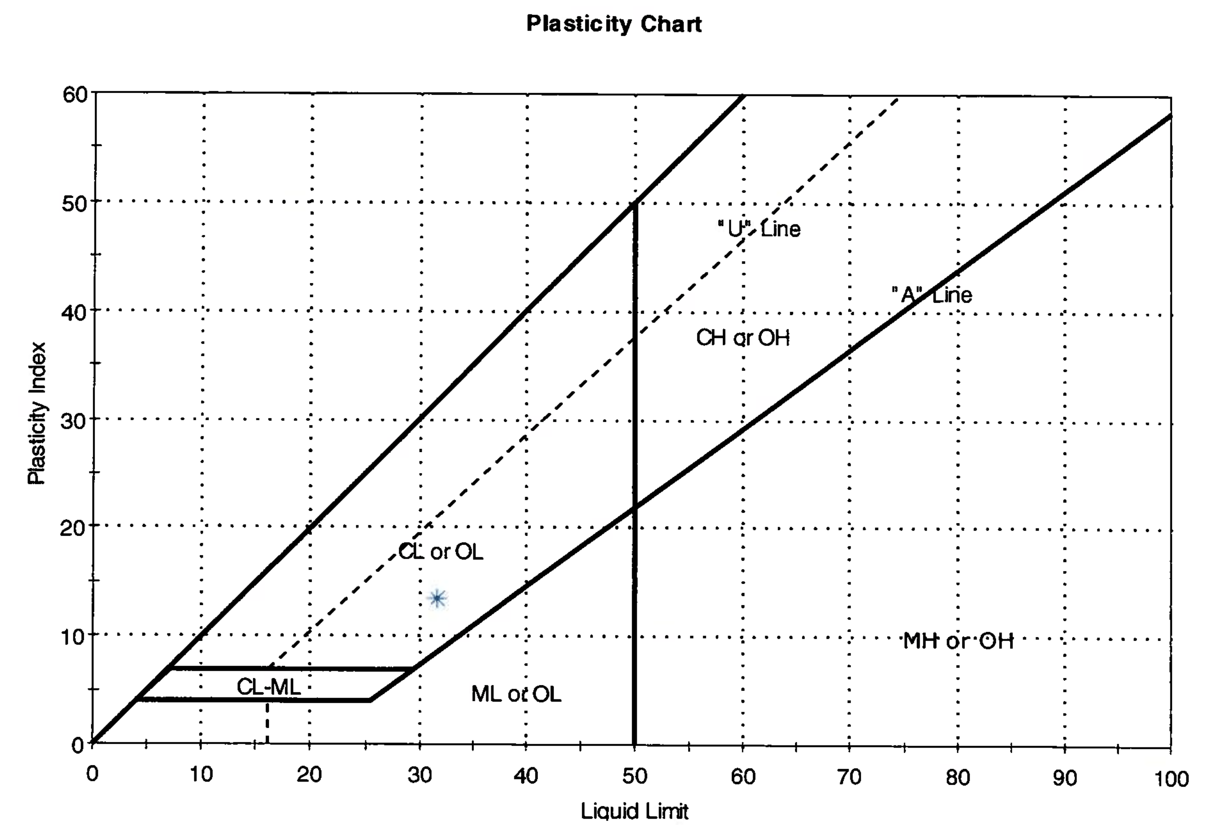
Client:	Haley & Aldrich, Inc.				
Project:	Presumpscot River Bridge				
Location:	Falmouth/Portland ME			Project No:	GTX-8629
Boring ID:	BB-FRR-102	Sample Type:	tube	Tested By:	ap
Sample ID:	U1	Test Date:	11/19/08	Checked By:	jdt
Depth :	32.0-34.0 ft	Test Id:	141015		
Test Comment:	---				
Sample Description:	Wet, dark gray clay				
Sample Comment:	---				

Atterberg Limits - ASTM D 4318-05



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
*	U1	B-FRR-102	32.0-34.0 ft	40	32	18	14	2	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

Dilentancy: SLOW

Toughness: LOW

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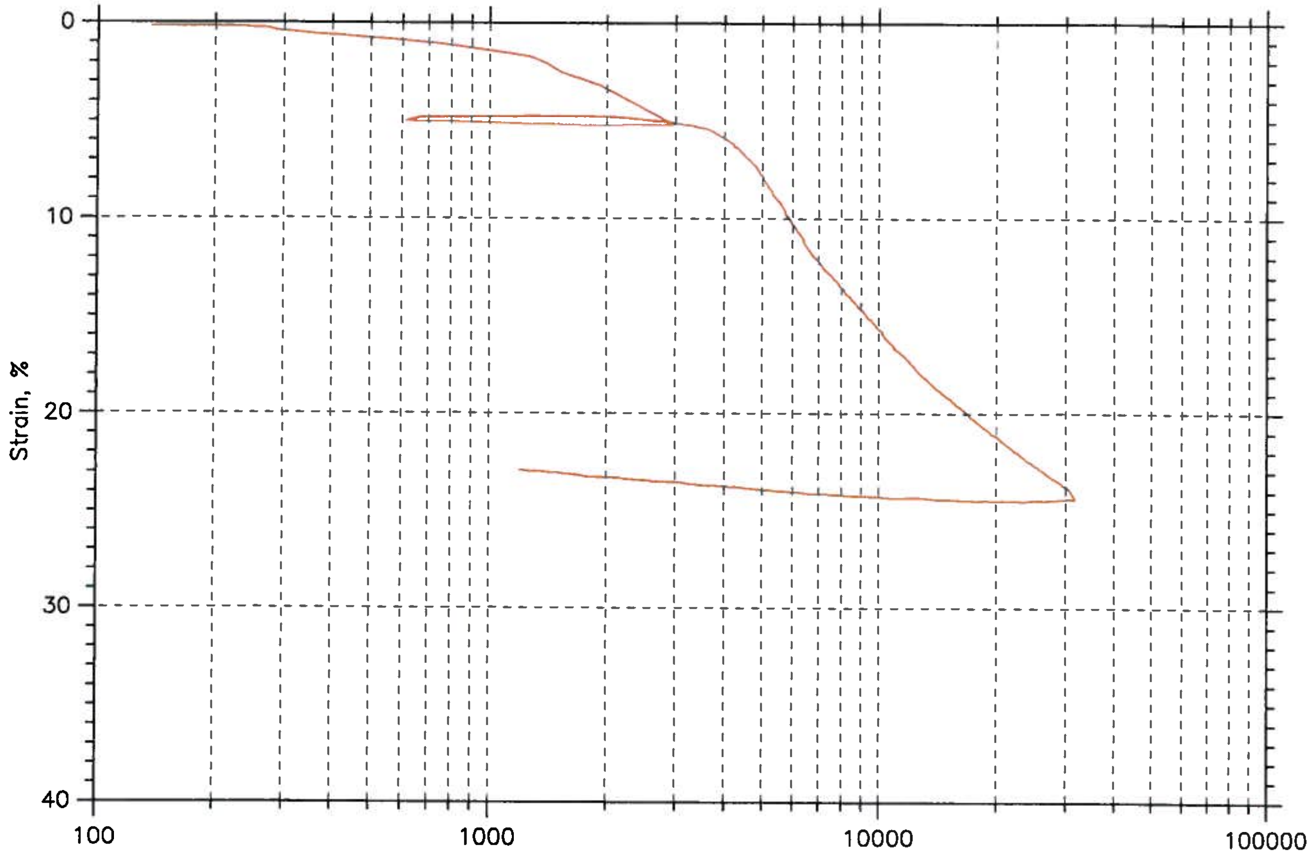
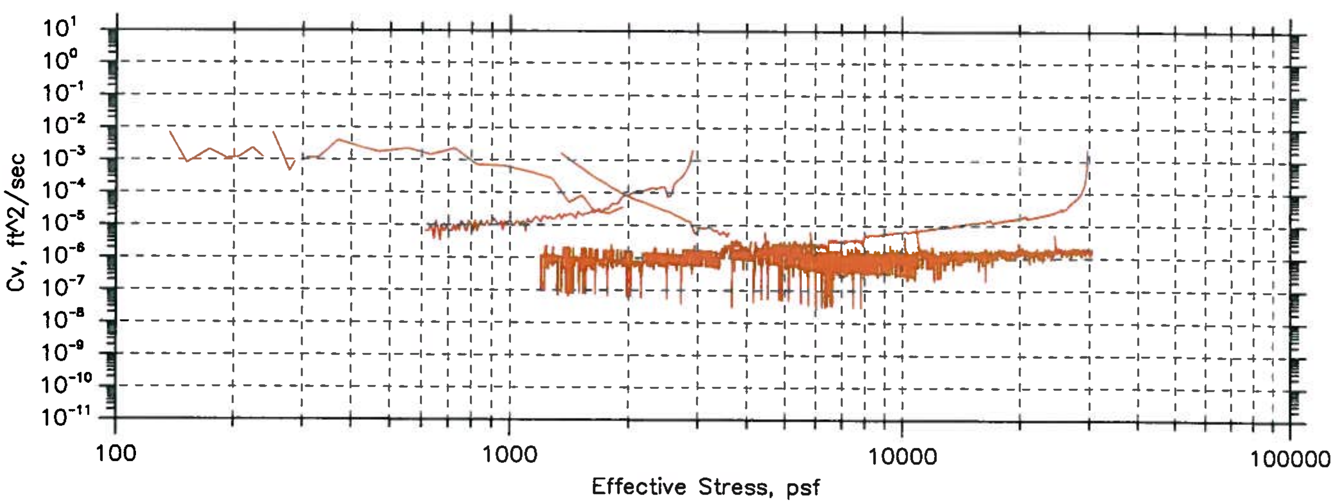
Constant Rate of Consolidation
Constant Strain Rate by ASTM D4186
Summary Report





Project: Presumpscot River	Location: Falmouth/Portland, ME	Project No.: GTX-8629
Boring No.: BB-FRR-102	Tested By: md	Checked By: jdt
Sample No.: U-1	Test Date: 11/16/08	Depth: 32-34 ft
Test No.: CRC-2	Sample Type: tube	Elevation: ---
Description: Wet, dark gray clay		
Remarks: System K		

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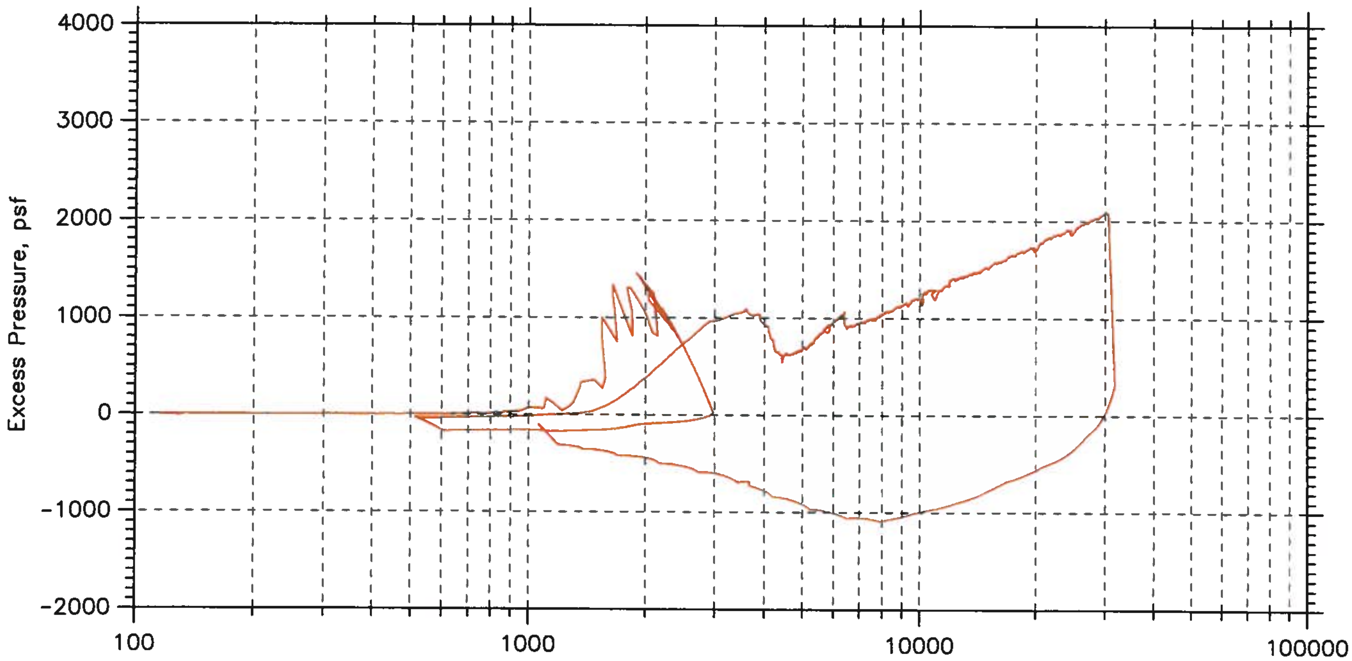
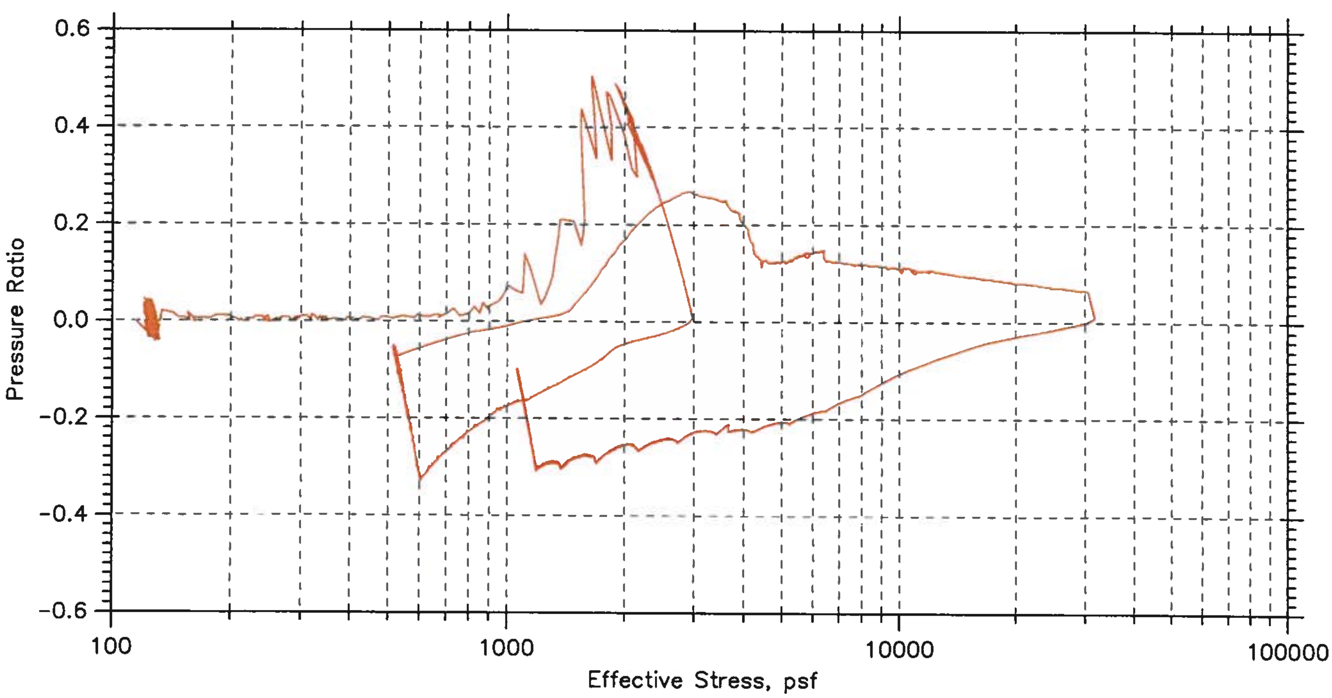
Constant Rate of Consolidation

Constant Strain Rate by ASTM D4186

Pressure Curves





Project: Presumpscot River	Location: Falmouth/Portland, ME	Project No.: GTX-8629
Boring No.: BB-FRR-102	Tested By: md	Checked By: jdt
Sample No.: U-1	Test Date: 11/16/08	Depth: 32-34 ft
Test No.: CRC-2	Sample Type: tube	Elevation: ---
Description: Wet, dark gray clay		
Remarks: System K		

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CRC TEST DATA

Project: Presumpscot River
Boring No.: BB-FRR-102
Sample No.: U-1
Test No.: CRC-2

Location: Falmouth/Portland, ME
Tested By: md
Test Date: 11/16/08
Sample Type: tube

Project No.: GTX-8629
Checked By: jdt
Depth: 32-34 ft
Elevation: ---

Soil Description: Wet, dark gray clay
Remarks: System K

Estimated Specific Gravity: 2.90
Initial Void Ratio: 1.39
Final Void Ratio: 0.84

Liquid Limit: 32
Plastic Limit: 18
Plasticity Index: 14

Initial Height: 1.00 in
Specimen Diameter: 2.50 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	3137	RING		smack
Wt. Container + Wet Soil, gm	106.41	360.1	342.33	133.45
Wt. Container + Dry Soil, gm	77.23	314	314	105.21
Wt. Container, gm	8.31	216.54	216.54	8.08
Wt. Dry Soil, gm	68.92	97.455	97.455	97.13
Water Content, %	42.34	47.31	29.07	29.07
Void Ratio	---	1.39	0.84	---
Degree of Saturation, %	---	98.49	100.00	---
Dry Unit Weight, pcf	---	75.633	98.186	---

Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

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Client: Haley & Aldrich, Inc.			
Project: Presumpscot River Bridge			
Location: Falmouth/Portland ME		Project No:	GTX-8629
Boring ID: BB-FPR-102	Sample Type: tube	Tested By:	ap
Sample ID:U1	Test Date: 11/24/08	Checked By:	jdt
Depth : 31.5-33.5 ft	Sample Id: 67204		
Test Comment: ---			
Sample Description: Moist, gray clay			
Sample Comment: ---			

Moisture Content of Soil - ASTM D 2216-05

Boring ID	Sample ID	Depth	Description	Moisture Content,%
BB-FPR-102	U1	31.5-33.5 ft	Moist, gray clay	36.6

Notes: Temperature of Drying : 110° Celsius

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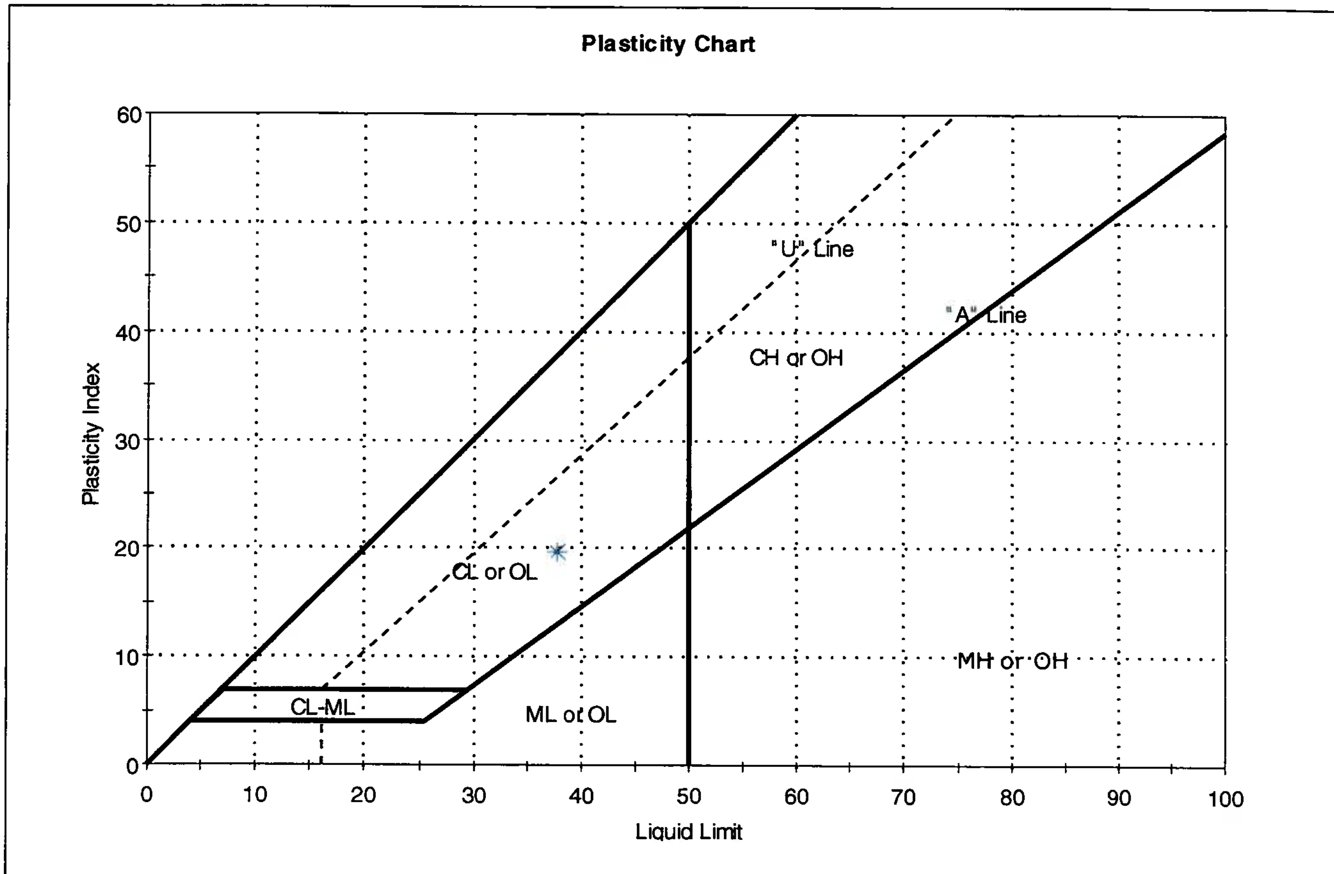
Client:	Haley & Aldrich, Inc.		
Project:	Presumpscot River Bridge		
Location:	Falmouth/Portland ME	Project No:	GTX-8629
Boring ID: BB-FPR-102	Sample Type: tube	Tested By:	ap
Sample ID:U1	Test Date: 11/17/08	Checked By:	jdt
Depth : 31.5-33.5 ft	Test Id: 141017		
Test Comment:	---		
Sample Description:	Moist, gray clay		
Sample Comment:	---		

Atterberg Limits - ASTM D 4318-05



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
*	U1	B-FPR-102	31.5-33.5 ft	37	38	18	20	1	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

Dilentancy: SLOW

Toughness: LOW

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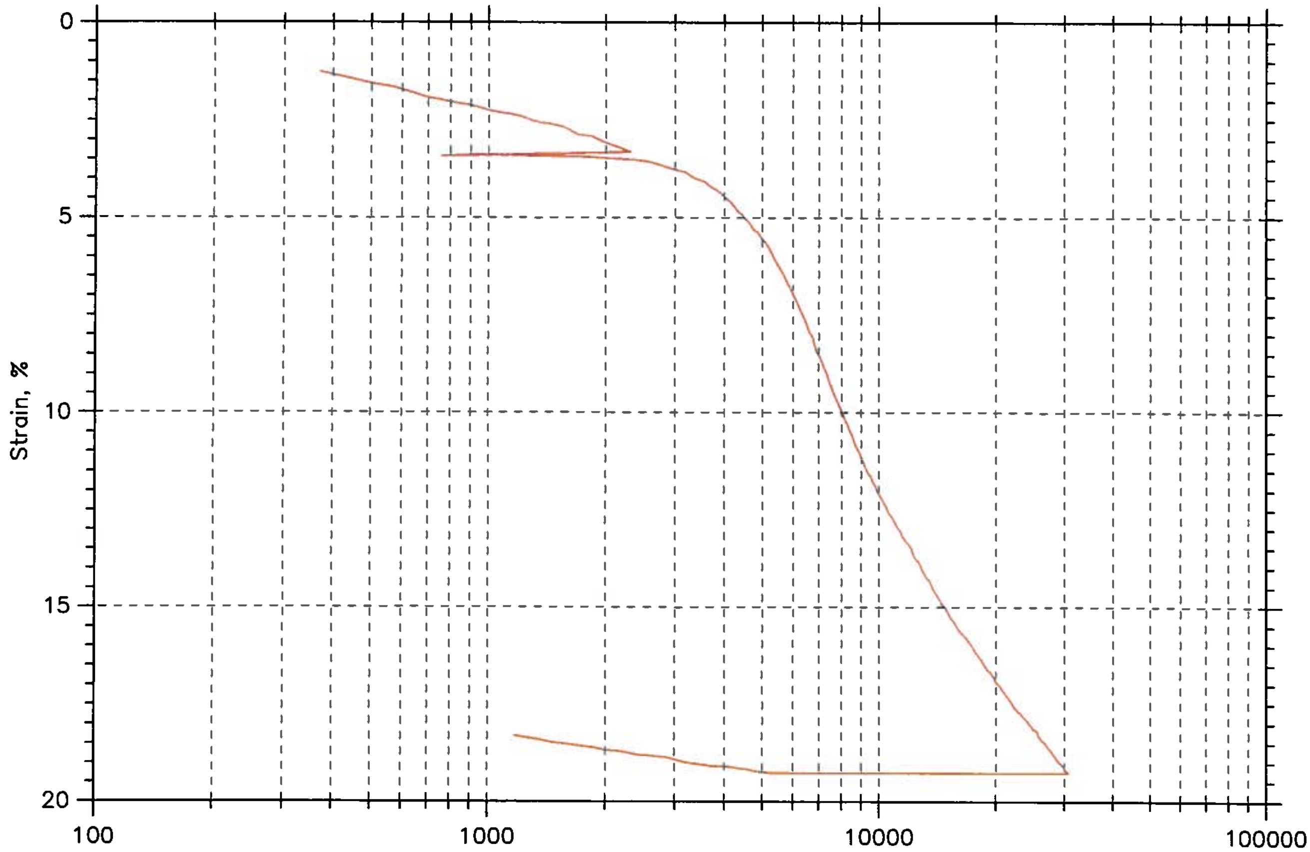
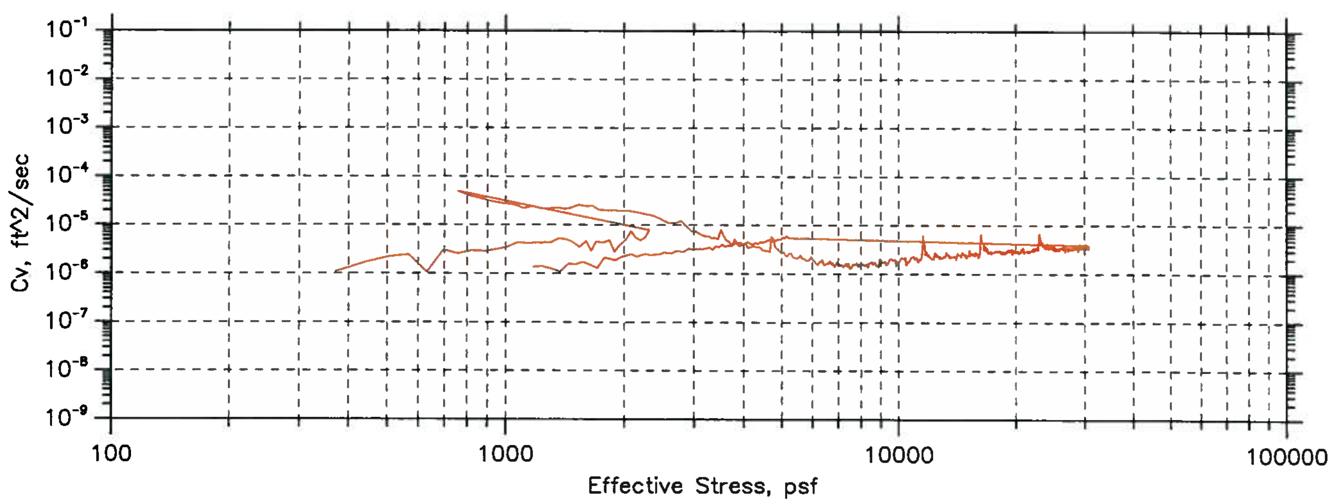
Constant Rate of Consolidation

Constant Strain Rate by ASTM D4186

Summary Report





Project: Presumpscot River	Location: Falmouth/Portland, ME	Project No.: GTX-8629
Boring No.: BB-FPR-102	Tested By: md	Checked By: jdt
Sample No.: U-1	Test Date: 11/11/08	Depth: 31.5-33.5
Test No.: CRC-1	Sample Type: tube	Elevation: ---
Description: Moist, dark gray clay		
Remarks: System E		

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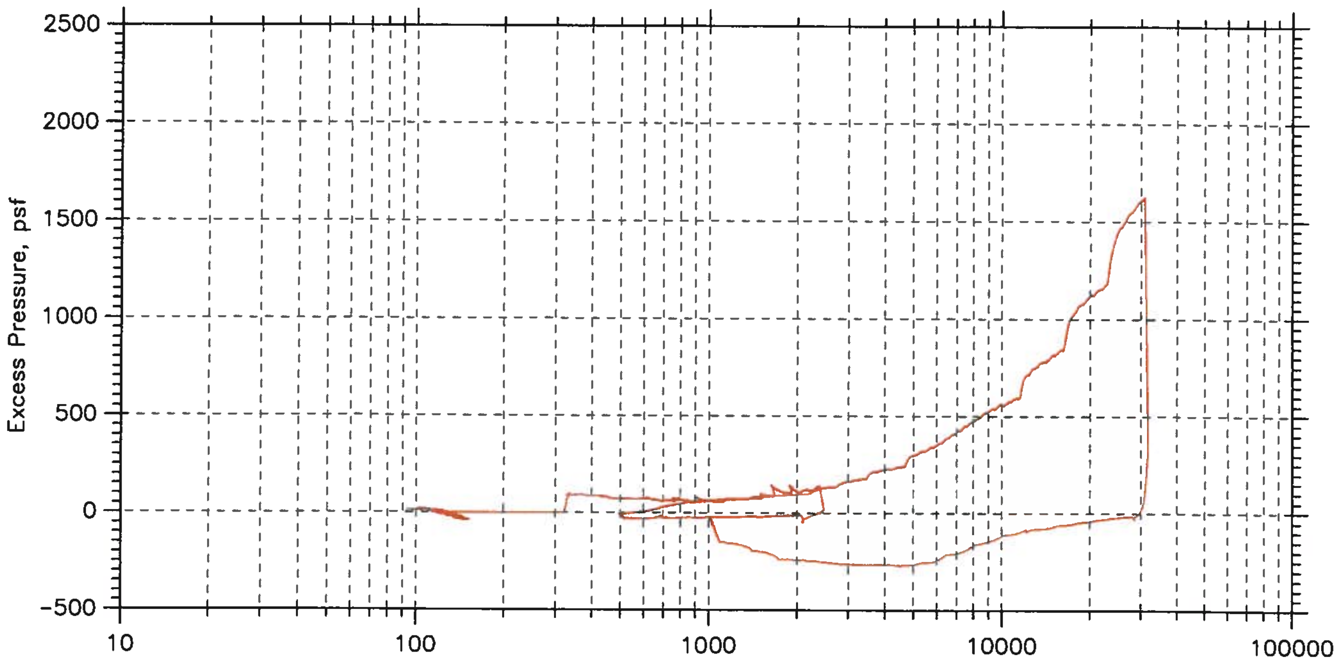
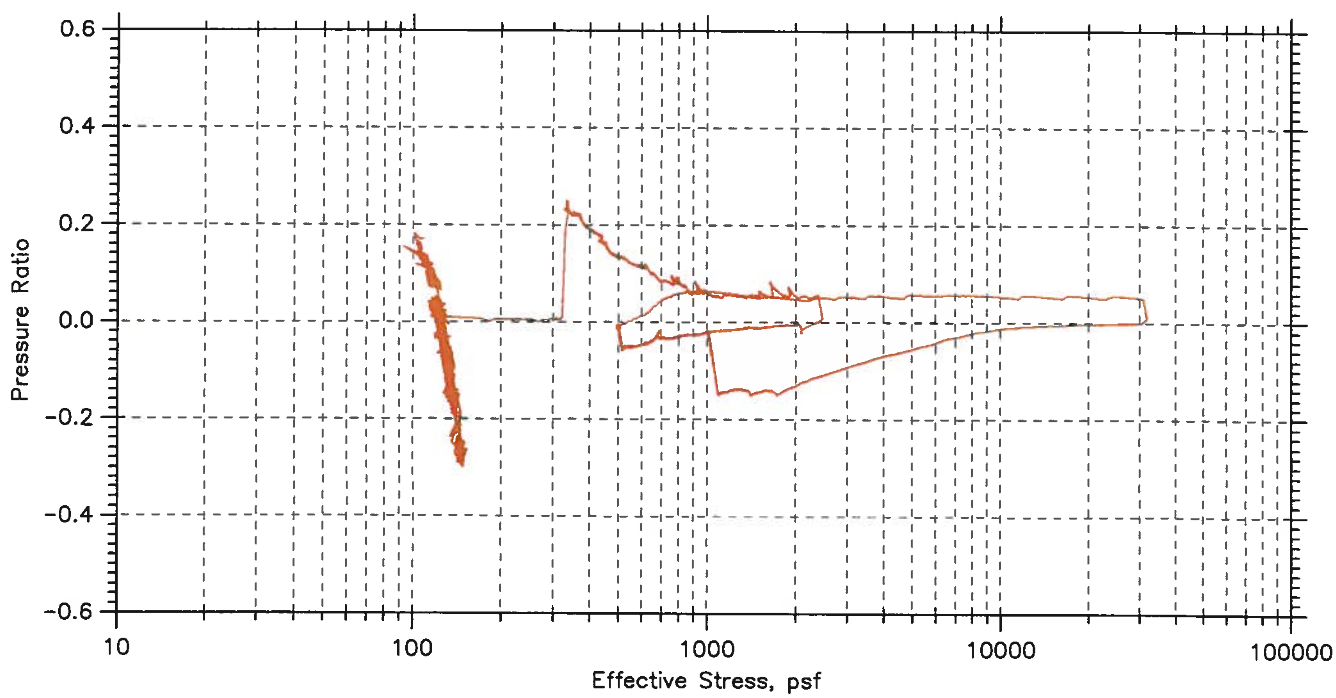
Constant Rate of Consolidation

Constant Strain Rate by ASTM D4186

Pressure Curves





Project: Presumpscot River	Location: Falmouth/Portland, ME	Project No.: GTX-8629
Boring No.: BB-FPR-102	Tested By: md	Checked By: jdt
Sample No.: U-1	Test Date: 11/11/08	Depth: 31.5-33.5
Test No.: CRC-1	Sample Type: tube	Elevation: ---
Description: Moist, dark gray clay		
Remarks: System E		

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CRC TEST DATA

Project: Presumpscot River
Boring No.: BB-FPR-102
Sample No.: U-1
Test No.: CRC-1

Location: Falmouth/Portland, ME
Tested By: md
Test Date: 11/11/08
Sample Type: tube

Project No.: GTX-8629
Checked By: jdt
Depth: 31.5-33.5
Elevation: ---

Soil Description: Moist, dark gray clay
Remarks: System E

Estimated Specific Gravity: 2.95
Initial Void Ratio: 1.21
Final Void Ratio: 0.79

Liquid Limit: 38
Plastic Limit: 18
Plasticity Index: 20

Initial Height: 1.00 in
Specimen Diameter: 2.50 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	Bear	RING		12
Wt. Container + Wet Soil, gm	144.83	365.18	352.47	143.87
Wt. Container + Dry Soil, gm	109.17	323.68	323.68	115.15
Wt. Container, gm	8.3	216.52	216.52	8.24
Wt. Dry Soil, gm	100.87	107.16	107.16	106.91
Water Content, %	35.35	38.72	26.86	26.86
Void Ratio	---	1.21	0.79	---
Degree of Saturation, %	---	94.18	100.00	---
Dry Unit Weight, pcf	---	83.167	102.66	---

Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

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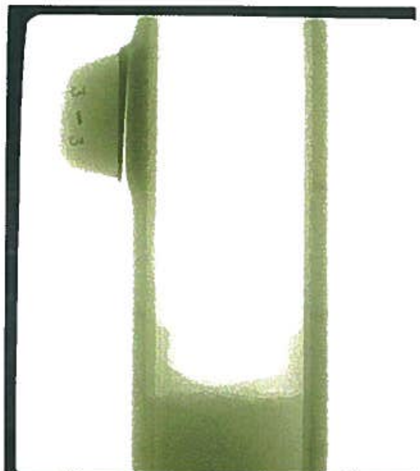
Client:	Haley & Aldrich, Inc.
Project Name:	Presumpscot River Bridge
Project Location:	Falmouth & Portland Maine
GTX #:	8629
Test Date:	11/03/08
Tested By:	md
Checked By:	jdt
Boring ID:	BB-FPR 102
Sample ID:	U-1
Depth, ft:	31.5-33.5

X-Ray of Soil Sample by ASTM D 4452



Top of Tube

Middle of Tube

Bottom of Tube

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Client:	Haley & Aldrich, Inc.				
Project:	Presumpscot River Bridge				
Location:	Falmouth/Portland ME			Project No:	GTX-8629
Boring ID:	BB-FRR-102	Sample Type:	tube	Tested By:	ap
Sample ID:	U2	Test Date:	11/24/08	Checked By:	jdt
Depth :	54.0-56.0 ft	Sample Id:	67685		
Test Comment:	---				
Sample Description:	Wet, dark gray clay				
Sample Comment:	---				

Moisture Content of Soil - ASTM D 2216-05

Boring ID	Sample ID	Depth	Description	Moisture Content,%
BB-FRR-102	U2	54.0-56.0 ft	Wet, dark gray clay	33.1

Notes: Temperature of Drying : 110° Celsius

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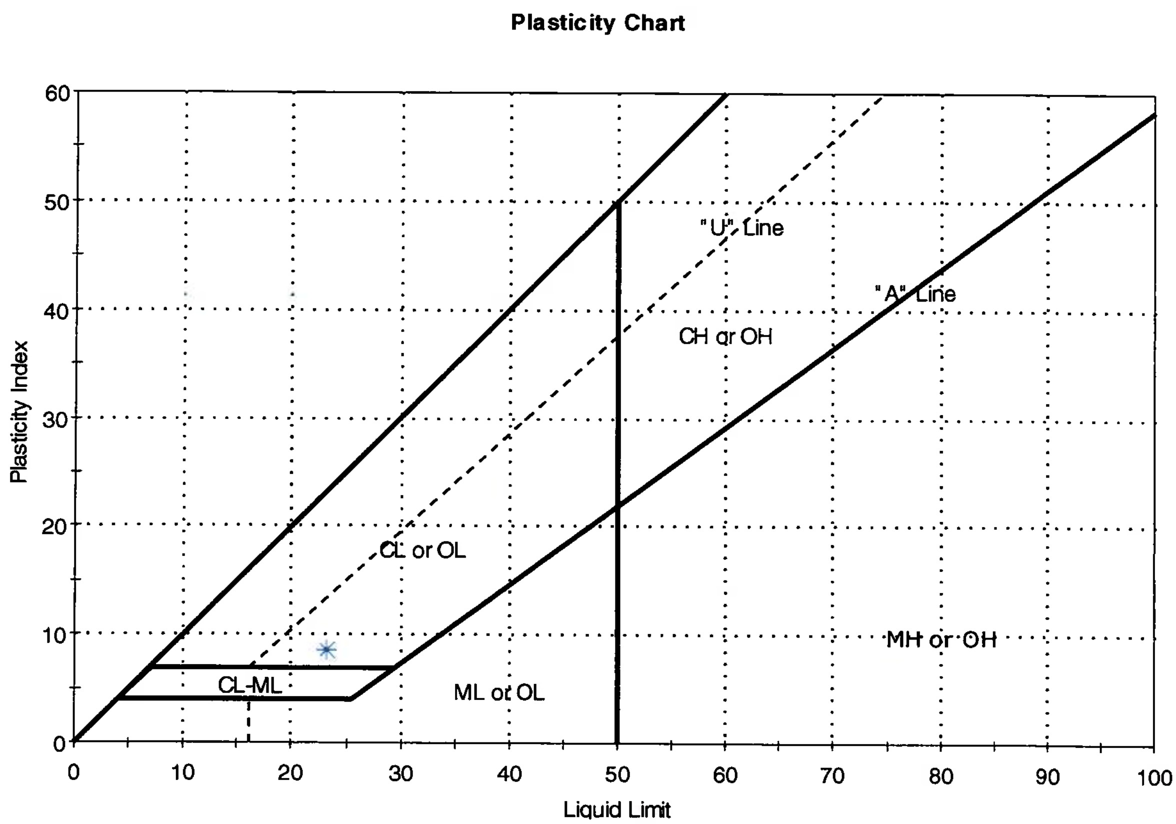
Client:	Haley & Aldrich, Inc.				
Project:	Presumpscot River Bridge				
Location:	Falmouth/Portland ME			Project No:	GTX-8629
Boring ID:	BB-FRR-102	Sample Type:	tube	Tested By:	ap
Sample ID:	U2	Test Date:	11/21/08	Checked By:	jdt
Depth :	54.0-56.0 ft	Test Id:	142055		
Test Comment:	---				
Sample Description:	Wet, dark gray clay				
Sample Comment:	---				

Atterberg Limits - ASTM D 4318-05



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
*	U2	B-FRR-10	54.0-56.0 ft	33	23	14	9	2	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

Dilentancy: SLOW

Toughness: LOW

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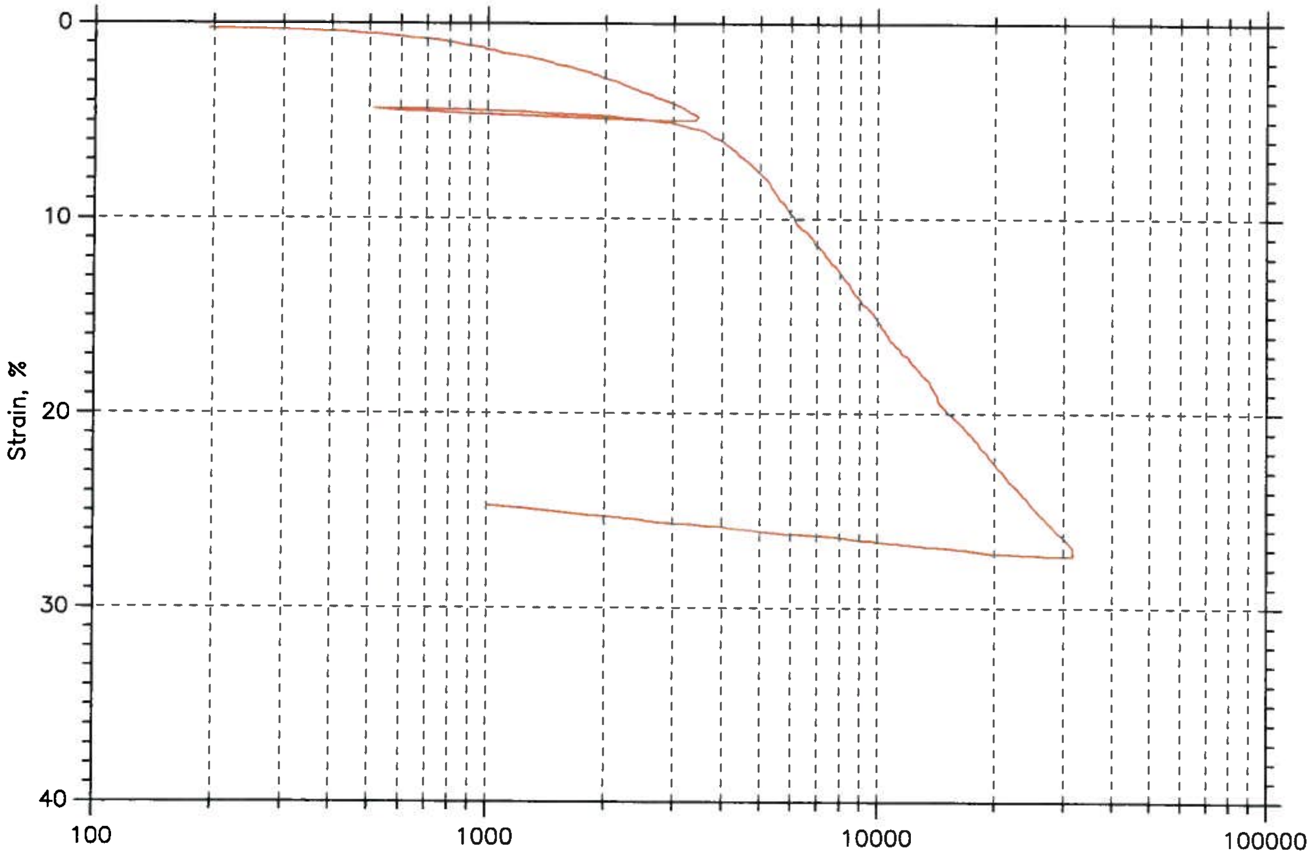
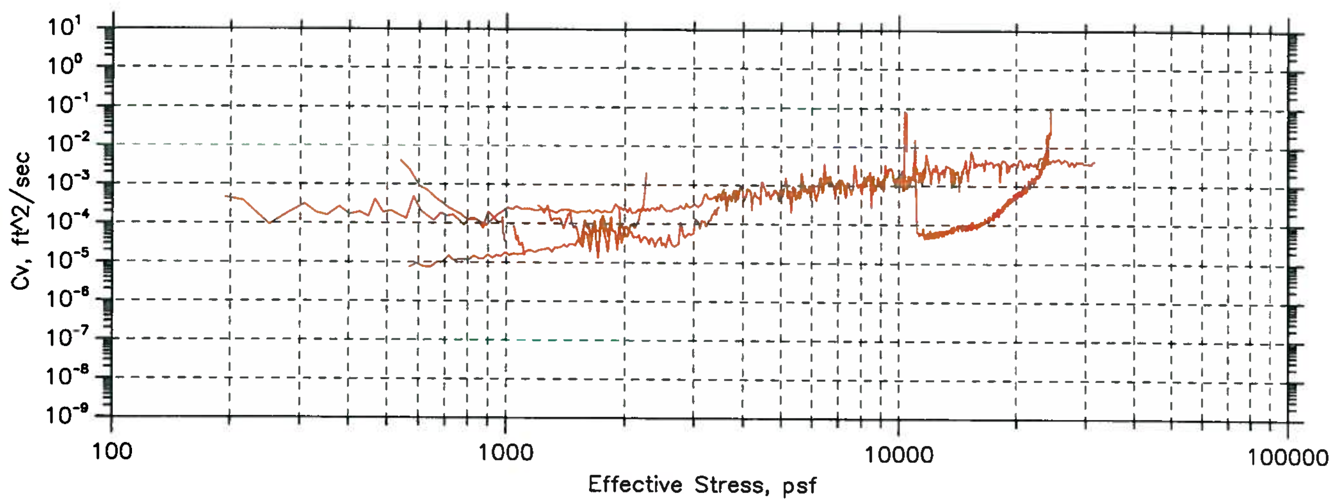
Constant Rate of Consolidation

Constant Strain Rate by ASTM D4186

Summary Report





Project: Presumpscot River	Location: Falmouth/Portland, ME	Project No.: GTX-8629
Boring No.: BB-FRR-102	Tested By: md	Checked By: jdt
Sample No.: U-2	Test Date: 11/18/08	Depth: 54-56 ft
Test No.: CRC-4	Sample Type: tube	Elevation: ---
Description: Wet, dark gray clay		
Remarks: System E		

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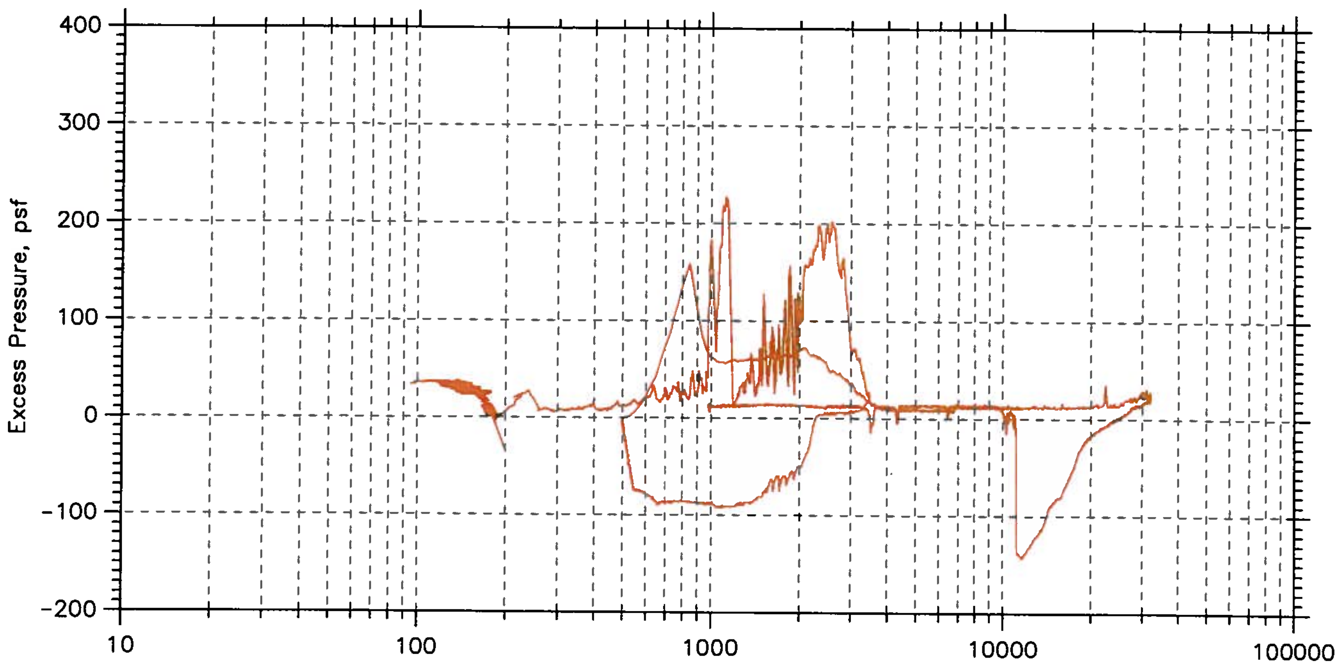
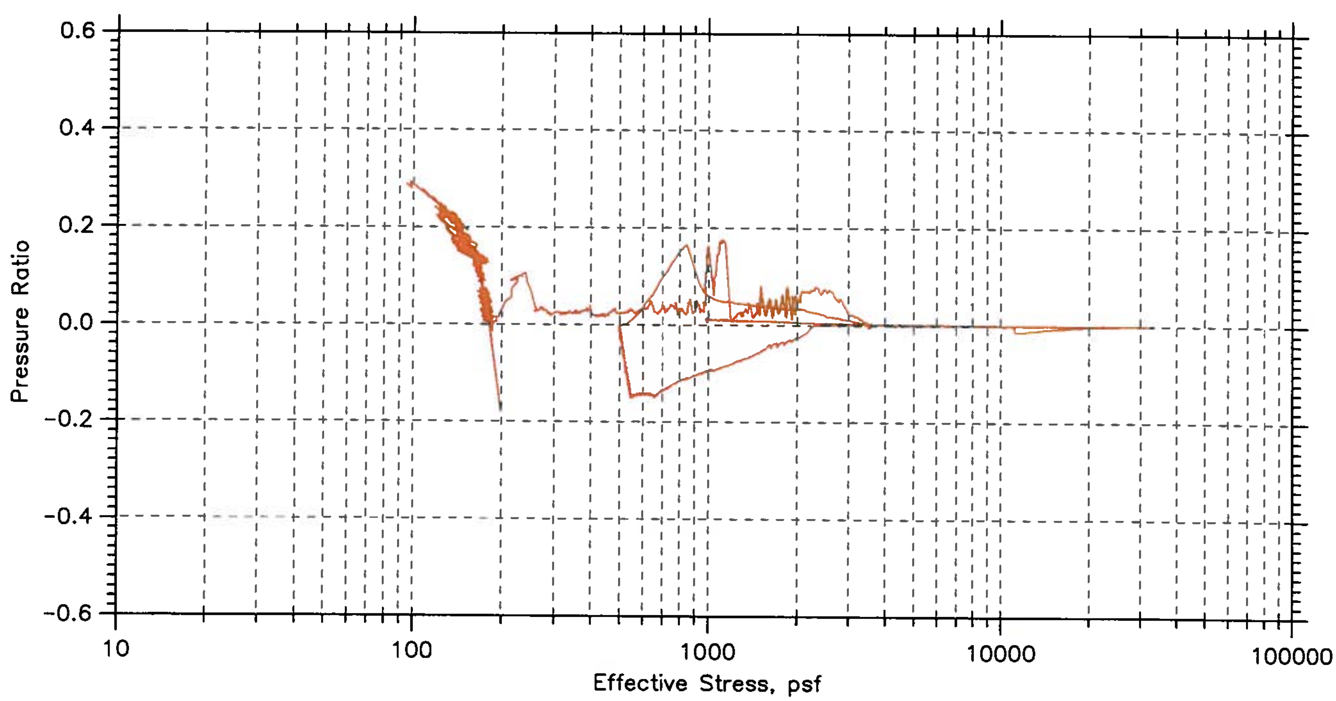
Constant Rate of Consolidation

Constant Strain Rate by ASTM D4186

Pressure Curves





Project: Presumpscot River	Location: Falmouth/Portland, ME	Project No.: GTX-8629
Boring No.: BB-FRR-102	Tested By: md	Checked By: jdt
Sample No.: U-2	Test Date: 11/18/08	Depth: 54-56 ft
Test No.: CRC-4	Sample Type: tube	Elevation: ---
Description: Wet, dark gray clay		
Remarks: System E		

Fri, 21-NOV-2008 14:50:47

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CRC TEST DATA

Project: Presumpscot River
Boring No.: BB-FRR-102
Sample No.: U-2
Test No.: CRC-4

Location: Falmouth/Portland, ME
Tested By: md
Test Date: 11/18/08
Sample Type: tube

Project No.: GTX-8629
Checked By: jdt
Depth: 54-56 ft
Elevation: ---

Soil Description: Wet, dark gray clay
Remarks: System E

Estimated Specific Gravity: 2.95
Initial Void Ratio: 1.25
Final Void Ratio: 0.70

Liquid Limit: 23
Plastic Limit: 14
Plasticity Index: 9

Initial Height: 1.00 in
Specimen Diameter: 2.50 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	3214	RING		balloons
Wt. Container + Wet Soil, gm	106.13	258.84	239.66	130.68
Wt. Container + Dry Soil, gm	81.01	214.64	214.64	107.29
Wt. Container, gm	8.18	109.05	109.05	8.56
Wt. Dry Soil, gm	72.83	105.59	105.59	98.73
Water Content, %	34.49	41.85	23.69	23.69
Void Ratio	---	1.25	0.70	---
Degree of Saturation, %	---	99.04	100.00	---
Dry Unit Weight, pcf	---	81.95	108.37	---

Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

GeoTesting express
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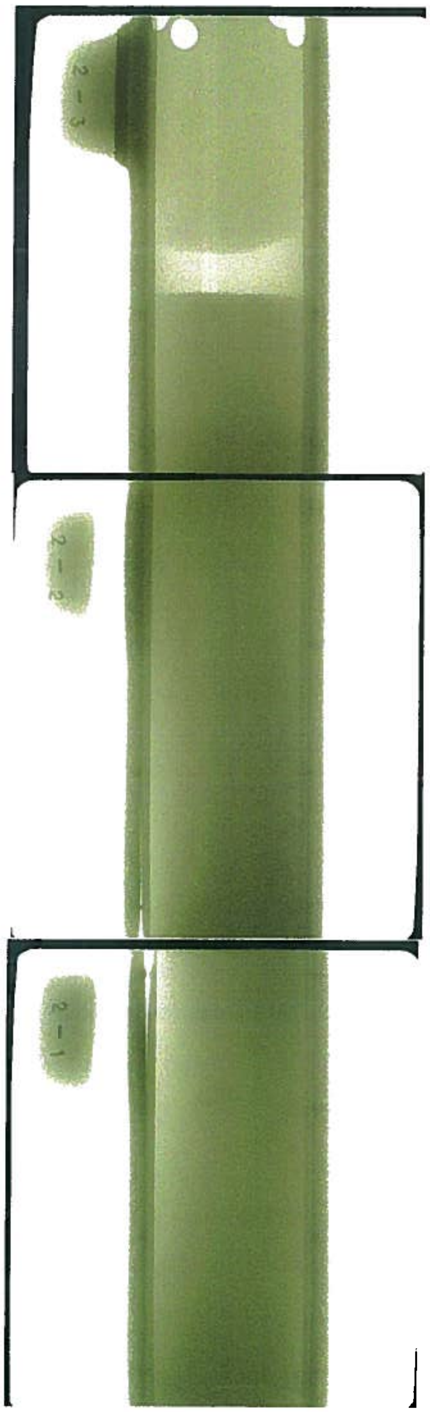
Client:	Haley & Aldrich, Inc.
Project Name:	Presumpscot River Bridge
Project Location:	Falmouth & Portland Maine
GTX #:	8629
Test Date:	11/03/08
Tested By:	md
Checked By:	jdt
Boring ID:	BB-FRR-102
Sample ID:	U-1
Depth, ft:	32-34

X-Ray of Soil Sample by ASTM D 4452



Top of Tube

Middle of Tube

Bottom of Tube

GeoTesting express
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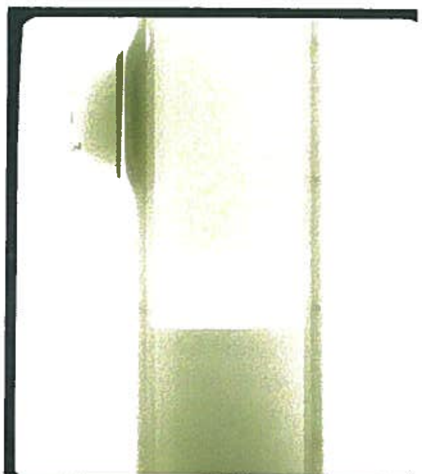
Client:	Haley & Aldrich, Inc.
Project Name:	Presumpscot River Bridge
Project Location:	Falmouth & Portland Maine
GTX #:	8629
Test Date:	11/14/08
Tested By:	md
Checked By:	jdt
Boring ID:	BB-FRR-102
Sample ID:	U2
Depth, ft:	54-56.0

X-Ray of Soil Sample by ASTM D 4452



Top of Tube

Middle of Tube

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Boxborough, MA 01719
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BY
MAY 14 2009
HALEY & ALDRICH
PORTLAND, MAINE

Transmittal

TO:

Mr. Bryan Steinert
Haley & Aldrich, Inc.
75 Washington Avenue, Suite 203
Portland, ME 04101-2617

DATE: 5/7/09	GTX NO: 8629
RE: Presumpscot River Bridge Project	

COPIES	DATE	DESCRIPTION
1	5/7/09	**April 2009 Laboratory Test Reports**

REMARKS:

SIGNED:
Joe Tomei – Laboratory Manager

APPROVED BY:
Mark Dobday – Laboratory Manager

GeoTesting
express
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Boston
Atlanta
New York

www.geocomp.com/geotesting

May 7, 2009

Mr. Bryan Steinert
Haley & Aldrich, Inc.
75 Washington Avenue, Suite 203
Portland, ME 04101-2617

Re: Presumpscot River Bridge Project (GTX-8629)

Dear Mr. Steinert:

Enclosed are the test results you requested for the above referenced project. GeoTesting Express, Inc. (GTX) received four soil samples from you on April 2, 2009. These samples were labeled as follows:

BB-FPR-103, D10 (18.0-20.0)
BB-FPR-103, D11 (28.0-30.0)
BB-FPR-103, D12 (38.0-40.0)
BB-FPR-103, D13 (48.0-50.0)

GTX performed the following test on each of these samples:

Grain Size Analysis (ASTM D 422) – sieve portion only

A copy of your test request is attached.

The results presented in this report apply only to the items tested. This report shall not be reproduced except in full, without written approval from GeoTesting Express. The remainder of these samples will be retained for a period of sixty (60) days and will then be discarded unless otherwise notified by you. Please call me if you have any questions or require additional information. Thank you for allowing GeoTesting Express the opportunity of providing you with testing services. We look forward to working with you again in the future.

Respectfully yours,

Joe Tomei
Laboratory Manager

GeoTesting
express
a subsidiary of Geocomp Corporation

1145 Massachusetts Avenue
Boxborough, MA 01719
978 635 0424 Tel
978 635 0266 Fax

Geotechnical Test Report

May 7, 2009

GTX-8629
Presumpscot River Bridge Project

Falmouth/Portland, ME

Prepared for:

HALEY & ALDRICH

GeoTesting express

a subsidiary of Geocomp Corporation

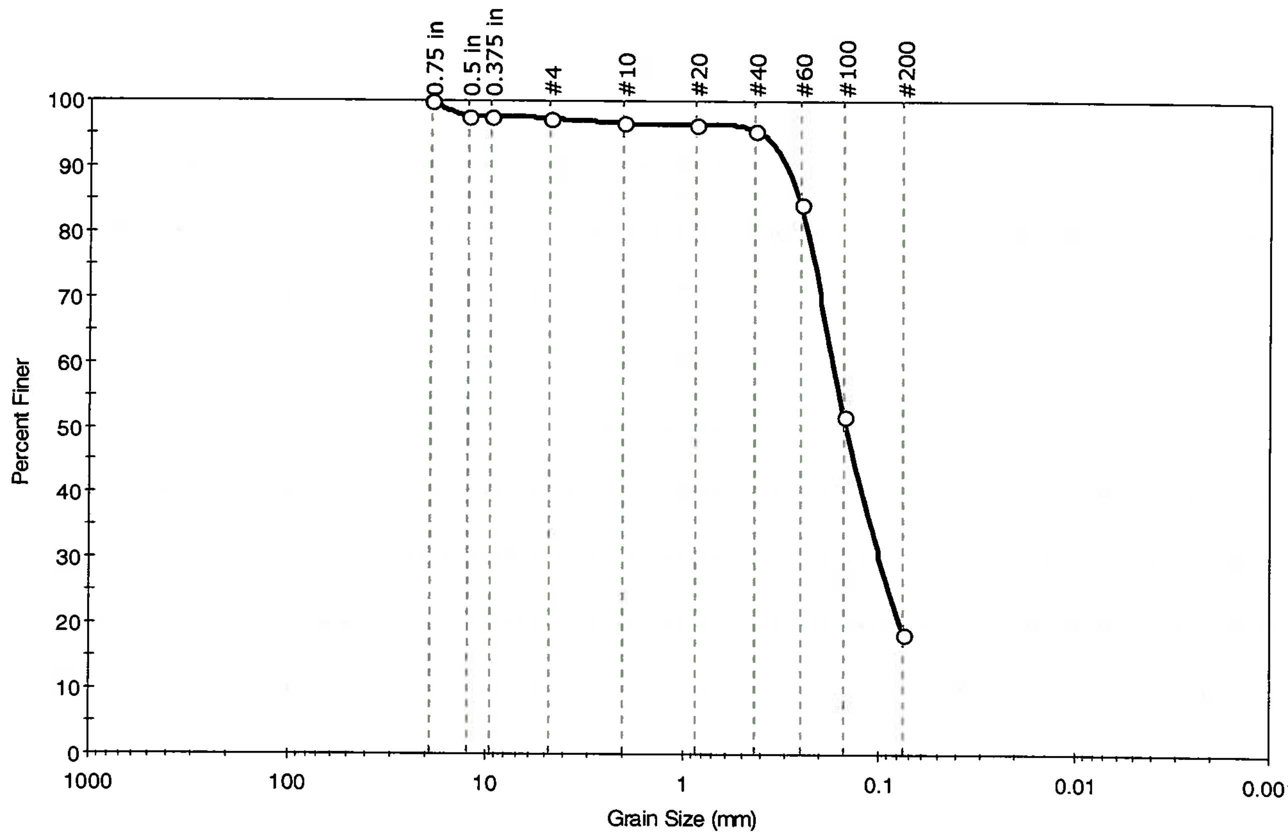
Client:	Haley & Aldrich, Inc.		
Project:	Presumpscot River Bridge		
Location:	Falmouth/Portland ME	Project No:	GTX-8629
Boring ID: BB-FPR-103	Sample Type: jar	Tested By:	jbr
Sample ID:D-10	Test Date: 04/07/09	Checked By:	jdt
Depth : 18.0-20.0 ft	Test Id: 150313		

Test Comment: ---

Sample Description: Moist, olive gray silty sand

Sample Comment: ---

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	2.7	78.8	18.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	98		
0.375 in	9.50	98		
#4	4.75	97		
#10	2.00	97		
#20	0.85	96		
#40	0.42	96		
#60	0.25	84		
#100	0.15	52		
#200	0.075	18		

Coefficients

D_{85} =0.2572 mm D_{30} =0.0953 mm

D_{60} =0.1705 mm D_{15} =N/A

D_{50} =0.1445 mm D_{10} =N/A

C_u =N/A C_c =N/A

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

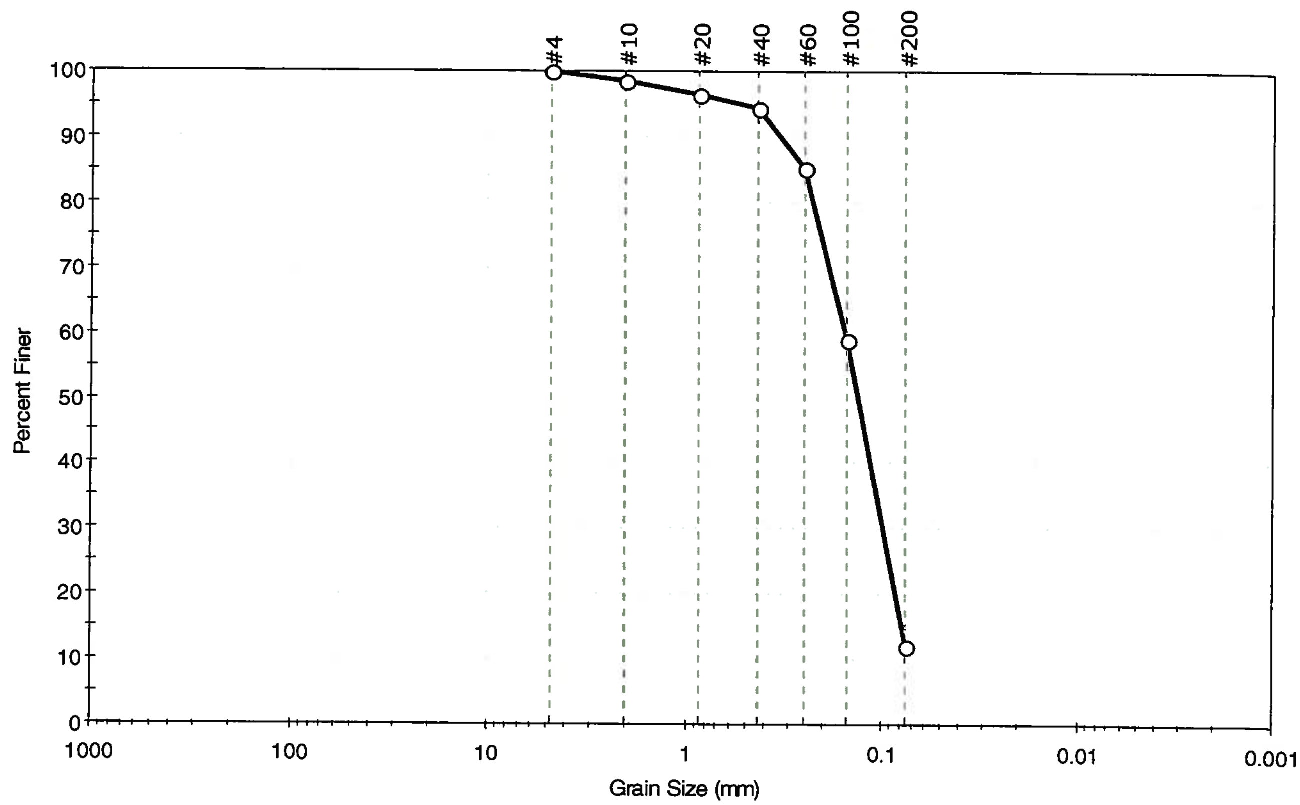
Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

GeoTesting express
a subsidiary of Geocomp Corporation

Client: Haley & Aldrich, Inc.			
Project: Presumpscot River Bridge			
Location: Falmouth/Portland ME		Project No:	GTX-8629
Boring ID: BB-FPR-103	Sample Type: jar	Tested By:	jbr
Sample ID:D-11	Test Date: 04/07/09	Checked By:	jdt
Depth : 28.0-30.0 ft	Test Id: 150314		
Test Comment: ---			
Sample Description: Moist, olive silty sand			
Sample Comment: ---			

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	87.8	12.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	99		
#20	0.85	96		
#40	0.42	94		
#60	0.25	85		
#100	0.15	59		
#200	0.075	12		

Coefficients

D_{85} =0.2494 mm — D_{30} =0.0977 mm
D_{60} =0.1532 mm — D_{15} =0.0782 mm
D_{50} =0.1314 mm — D_{10} =0.0726 mm
C_u =N/A — C_c =N/A

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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GeoTesting express
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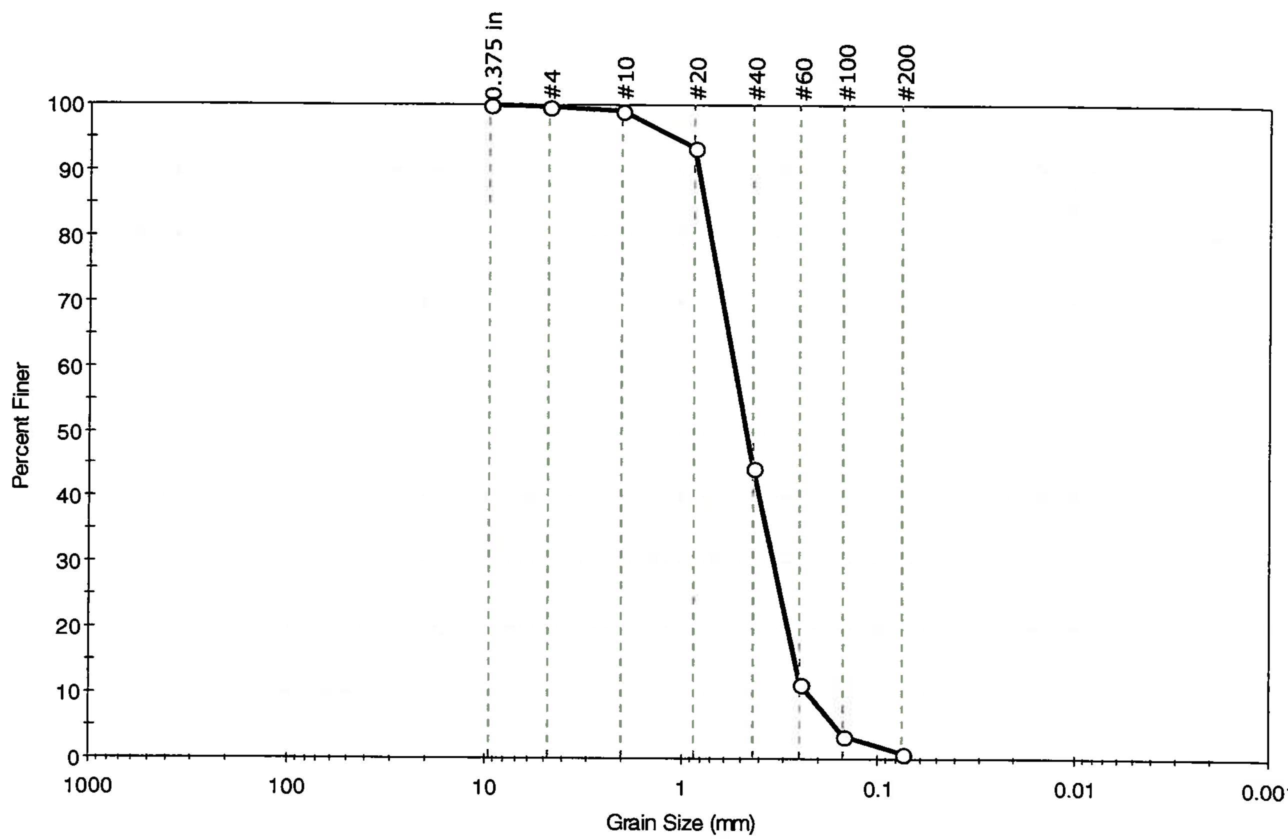
Client: Haley & Aldrich, Inc.			
Project: Presumpscot River Bridge			
Location: Falmouth/Portland ME		Project No:	GTX-8629
Boring ID: BB-FPR-103	Sample Type: jar	Tested By:	jbr
Sample ID:D-12	Test Date: 04/07/09	Checked By:	jdt
Depth : 38.0-40.0 ft	Test Id: 150315		
Test Comment: ---			
Sample Description: Moist, olive brown sand			
Sample Comment: ---			

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.2	98.9	0.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	100		
#10	2.00	99		
#20	0.85	94		
#40	0.42	44		
#60	0.25	12		
#100	0.15	3		
#200	0.075	1		

Coefficients

D_{85} =0.7533 mm D_{30} =0.3365 mm
D_{60} =0.5293 mm D_{15} =0.2641 mm
D_{50} =0.4596 mm D_{10} =0.2262 mm
C_u =2.340 C_c =0.946

Classification

ASTM Poorly graded sand (SP)

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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GeoTesting express
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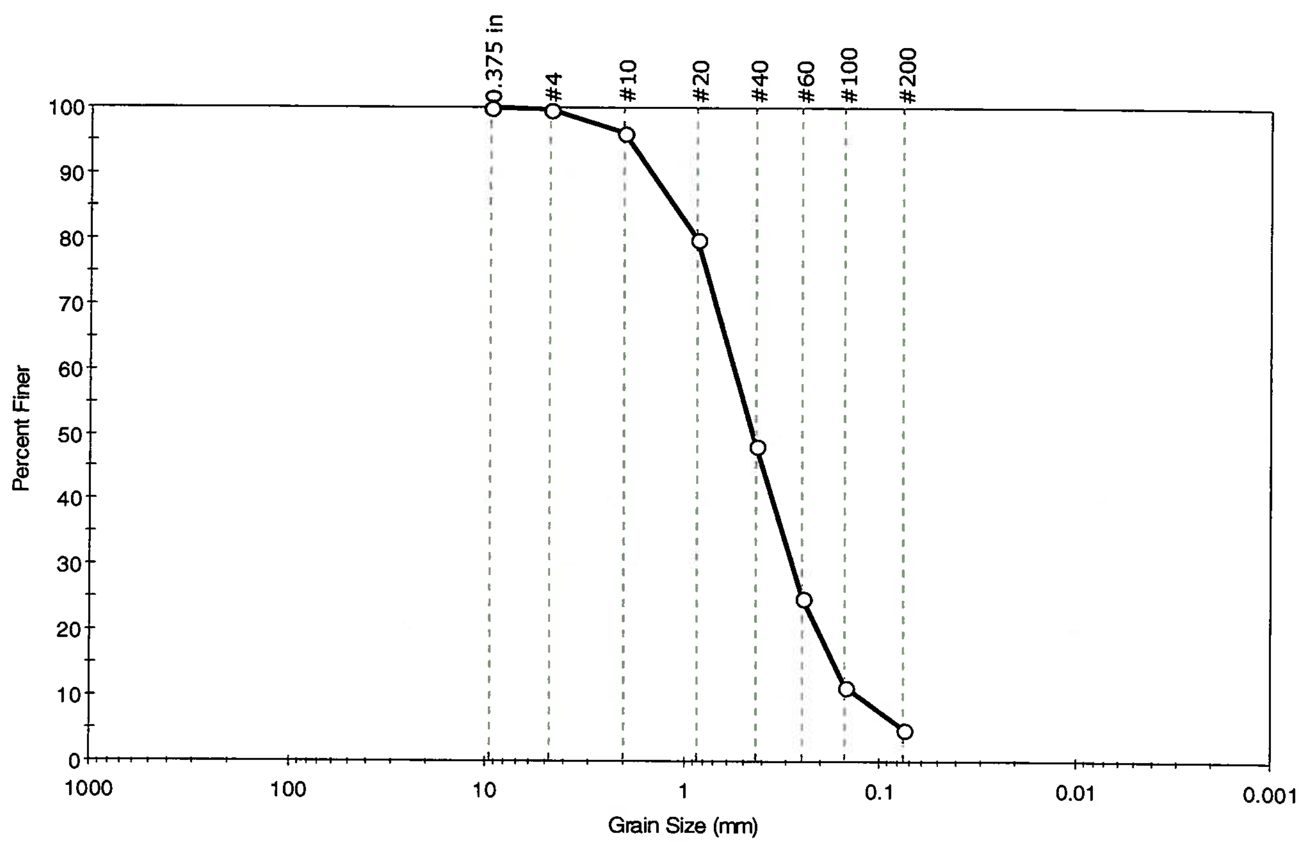
Client: Haley & Aldrich, Inc.			
Project: Presumpscot River Bridge			
Location: Falmouth/Portland ME		Project No:	GTX-8629
Boring ID: BB-FPR-103	Sample Type: jar	Tested By:	jbr
Sample ID: D-13	Test Date: 04/02/09	Checked By:	jdt
Depth : 48.0-50.0 ft	Test Id: 150316		

Test Comment: ---
Sample Description: Moist, olive brown sand with silt
Sample Comment: ---

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	0.4	94.6	5.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	100		
#10	2.00	96		
#20	0.85	80		
#40	0.42	48		
#60	0.25	25		
#100	0.15	12		
#200	0.075	5		

Coefficients

D_{85} = 1.1080 mm D_{30} = 0.2792 mm
D_{60} = 0.5488 mm D_{15} = 0.1703 mm
D_{50} = 0.4411 mm D_{10} = 0.1263 mm
C_u = 4.345 C_c = 1.125

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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JUN - 5 2009
HALEY & ALDRICH
PORTLAND, MAINE

Transmittal

TO:
Mr. Bryan Steinert
Haley & Aldrich, Inc.
75 Washington Avenue, Suite 203
Portland, ME 04101-2617

DATE: 5/29/09	GTX NO: 8629
RE: Presumpscot River Bridge Project	

COPIES	DATE	DESCRIPTION
1	5/29/09	**May 2009 Laboratory Test Reports**

REMARKS:

SIGNED: [signature]
Joe Tomei – Laboratory Manager

APPROVED BY: [signature]
Mark Dobday – Laboratory Manager

Boston
Atlanta
New York

www.geocomp.com/geotesting

May 29, 2009

Mr. Bryan Steinert
Haley & Aldrich, Inc.
75 Washington Avenue, Suite 203
Portland, ME 04101-2617

Re: Presumpscot River Bridge Project (GTX-8629)

Dear Mr. Steinert:

Enclosed are the test results you requested for the above referenced project. GeoTesting Express, Inc. (GTX) received eight soil samples from you on May 4, 2009. These samples were labeled as follows:

BB-FRR-202, U1 (45-47 ft)
BB-FRR-202, U2 (72-74 ft)
BB-FRR-203, U1 (20-22 ft)
BB-FRR-203, U2 (40-42 ft)
BB-FRR-202, D7 (20-22 ft)
BB-FRR-202, D9 (35-37 ft)
BB-FRR-202, D12 (55-57 ft)
BB-FRR-202, D14 (65-67 ft)

GTX performed the following tests on these samples:

7 Moisture Content tests (ASTM D 2216)
7 Atterberg Limits tests (ASTM D 4318)
3 CRS Consolidation tests (ASTM D 4186)
4 X-Ray of Soil

As requested, the x-ray tests were performed on the tube sample first and the reports were sent to H&A. After review of the x-ray reports, H&A provided GTX locations within the tubes to cut specimens for testing. Copies of your test requests are attached.

The results presented in this report apply only to the items tested. This report shall not be reproduced except in full, without written approval from GeoTesting Express. The remainder of these samples will be retained for a period of sixty (60) days and will then be discarded unless otherwise notified by you. Please call me if you have any questions or require additional information. Thank you for allowing GeoTesting Express the opportunity of providing you with testing services. We look forward to working with you again in the future.

Respectfully yours,

Joe Tomei
Laboratory Manager

GeoTesting
express
a subsidiary of Geocomp Corporation

1145 Massachusetts Avenue
Boxborough, MA 01719
978 635 0424 Tel
978 635 0266 Fax

Geotechnical Test Report

May 29, 2009

GTX-8629 Presumpscot River Bridge Project

Falmouth/Portland, ME

Prepared for:

HALEY & ALDRICH

GeoTesting express
a subsidiary of Geocomp Corporation

Client:	Haley & Aldrich, Inc.				
Project:	Presumpscot River Bridge				
Location:	Falmouth/Portland ME			Project No:	GTX-8629
Boring ID:	---	Sample Type:	---	Tested By:	mmd
Sample ID:	---	Test Date:	05/29/09	Checked By:	jdt
Depth :	---	Sample Id:	---		

Moisture Content of Soil - ASTM D 2216-05

Boring ID	Sample ID	Depth	Description	Moisture Content,%
BB-FRR-202	D7	20.0-22.0 ft	Moist, olive gray clay	42.8
BB-FRR-202	D9	35.0-37.0 ft	Moist, olive gray clay	42.5
BB-FRR-202	D12	55.0-57.0 ft	Moist, olive gray clay	40.3
BB-FRR-202	D14	65.0-67.0 ft	Moist, olive gray silty clay	30.8

Notes: Temperature of Drying : 110º Celsius

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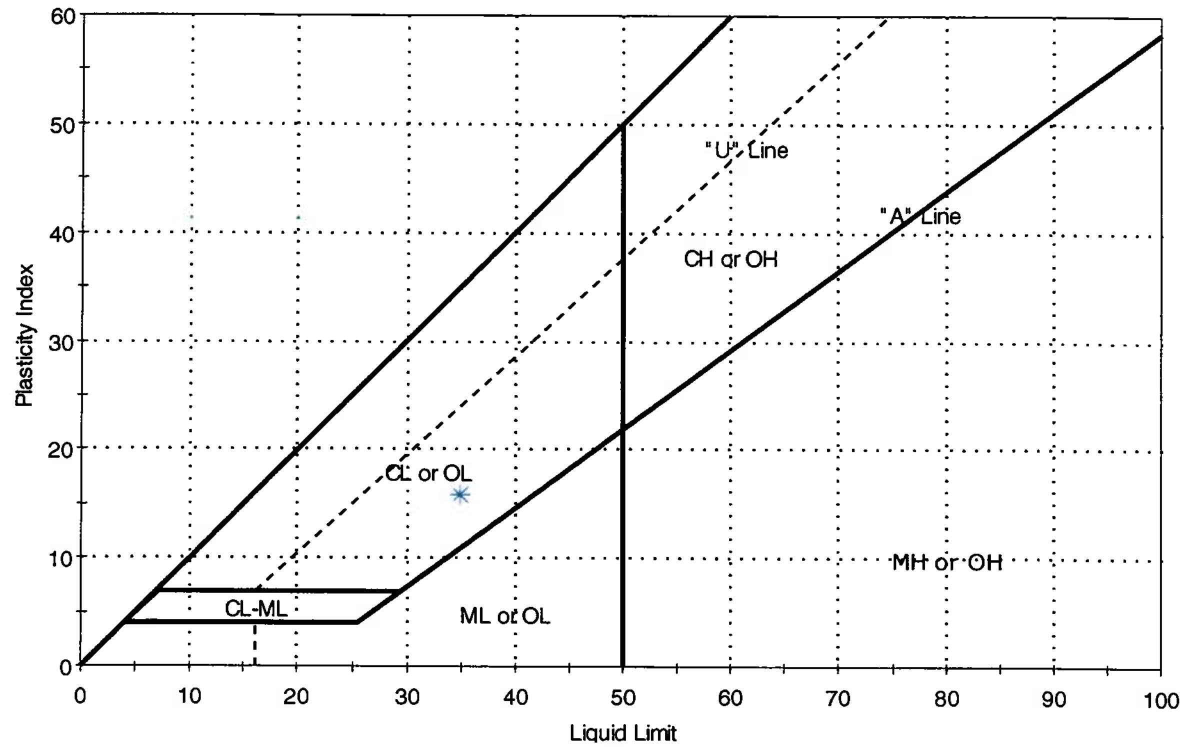
Client:	Haley & Aldrich, Inc.		
Project:	Presumpscot River Bridge		
Location:	Falmouth/Portland ME	Project No:	GTX-8629
Boring ID: BB-FRR-202	Sample Type: jar	Tested By:	cam
Sample ID:D7	Test Date: 05/06/09	Checked By:	jdt
Depth : 20.0-22.0 ft	Test Id: 151971		
Test Comment:	---		
Sample Description:	Moist, olive gray clay		
Sample Comment:	---		

Atterberg Limits - ASTM D 4318-05

Plasticity Chart



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
*	D7	B-FRR-202	20.0-22.0 ft	43	35	19	16	1	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

Dilentancy: RAPID

Toughness: LOW

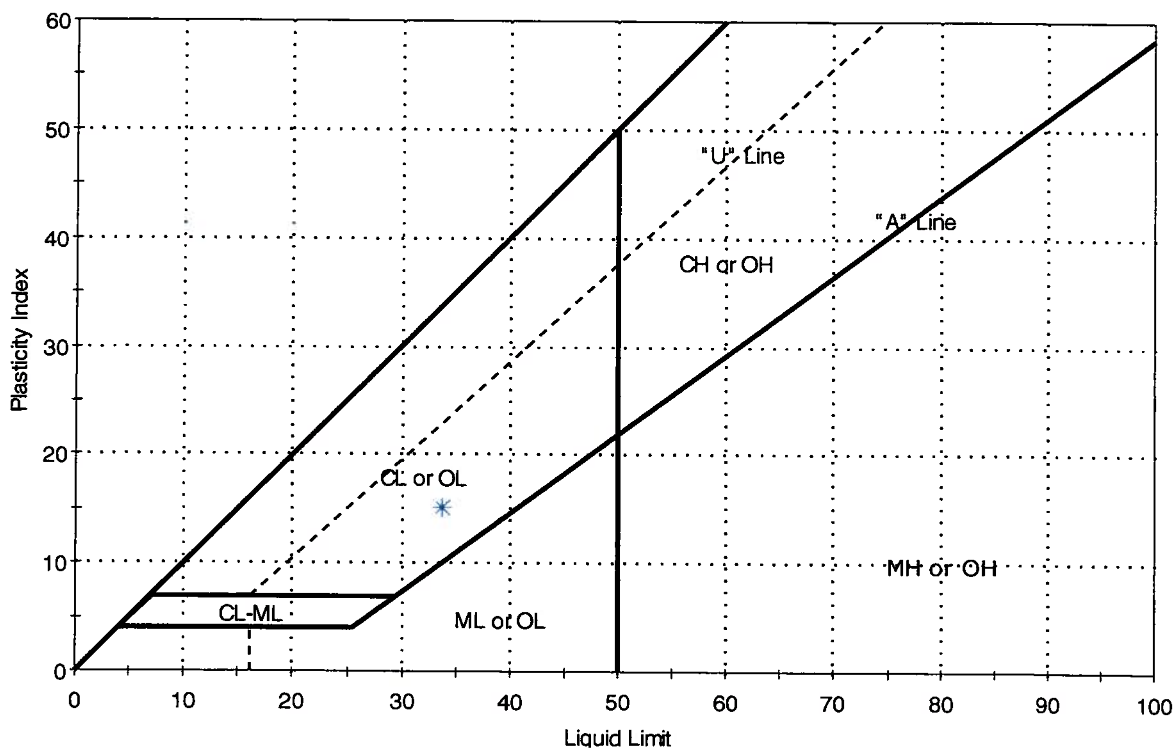
GeoTesting express
a subsidiary of Geocomp Corporation

Client: Haley & Aldrich, Inc.			
Project: Presumpscot River Bridge			
Location: Falmouth/Portland ME		Project No:	GTX-8629
Boring ID: BB-FRR-202	Sample Type: jar	Tested By:	cam
Sample ID:D9	Test Date: 05/06/09	Checked By:	jdt
Depth : 35.0-37.0 ft	Test Id: 151972		

Test Comment: ---
Sample Description: Moist, olive gray clay
Sample Comment: ---

Atterberg Limits - ASTM D 4318-05

Plasticity Chart



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
*	D9	B-FRR-20	35.0-37.0 ft	42	34	19	15	2	

Sample Prepared using the WET method

Dry Strength: VERY HIGH
Dilentancy: RAPID
Toughness: LOW

GeoTesting express
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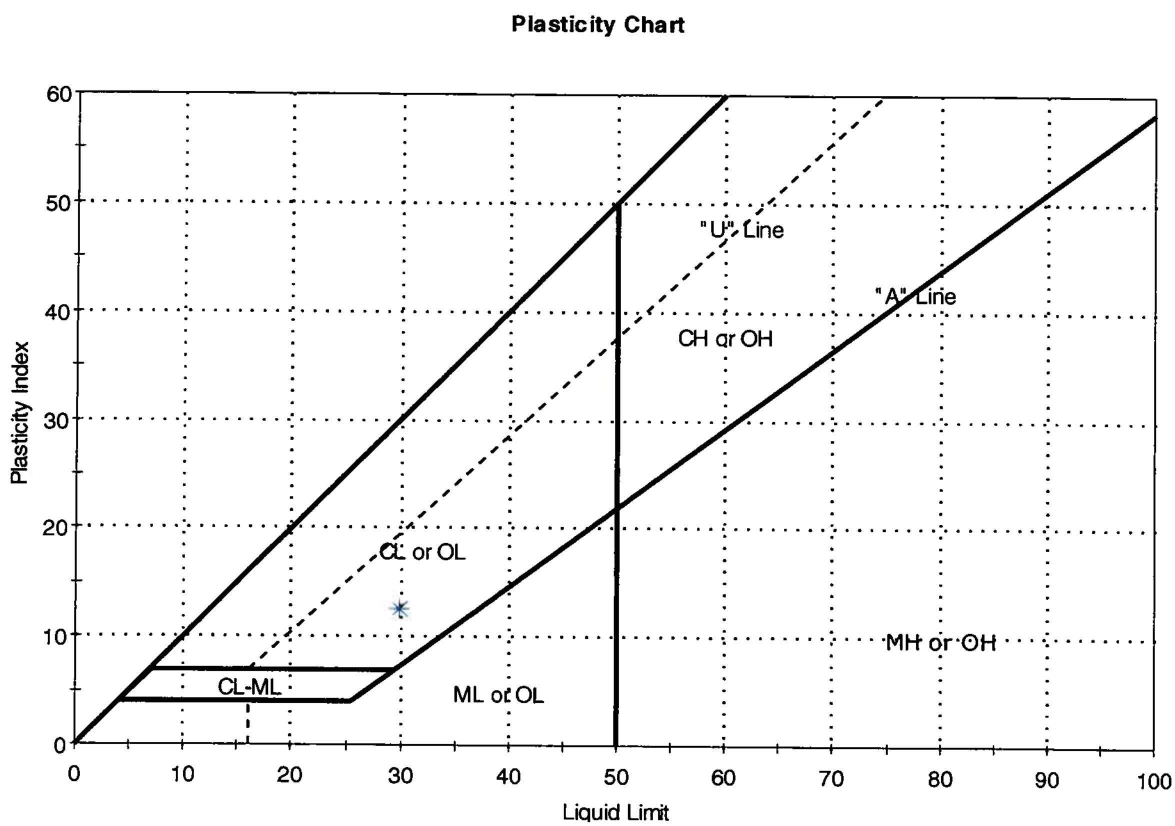
Client:	Haley & Aldrich, Inc.		
Project:	Presumpscot River Bridge		
Location:	Falmouth/Portland ME	Project No:	GTX-8629
Boring ID: BB-FRR-202	Sample Type: jar	Tested By:	cam
Sample ID:D12	Test Date: 05/06/09	Checked By:	jdt
Depth : 55.0-57.0 ft	Test Id: 151973		
Test Comment:	---		
Sample Description:	Moist, olive gray clay		
Sample Comment:	---		

Atterberg Limits - ASTM D 4318-05



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
*	D12	B-FRR-20	55.0-57.0 ft	40	30	17	13	2	

Sample Prepared using the WET method

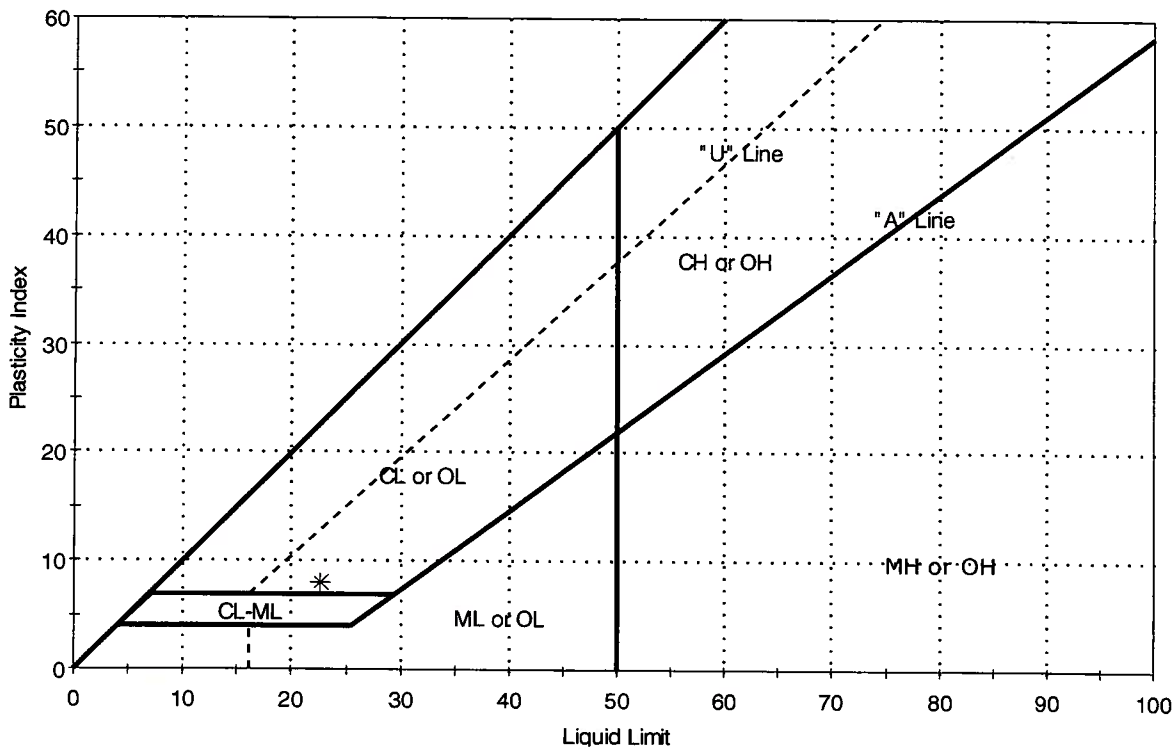
Dry Strength: VERY HIGH
Dilentancy: RAPID
Toughness: LOW

GeoTesting express

a subsidiary of Geocomp Corporation

Client:	Haley & Aldrich, Inc.				
Project:	Presumpscot River Bridge				
Location:	Falmouth/Portland ME			Project No:	GTX-8629
Boring ID:	BB-FRR-202	Sample Type:	jar	Tested By:	cam
Sample ID:	D14	Test Date:	05/06/09	Checked By:	jdt
Depth :	65.0-67.0 ft	Test Id:	151974		
Test Comment:	---				
Sample Description:	Moist, olive gray silty clay				
Sample Comment:	---				

Atterberg Limits - ASTM D 4318-05



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
*	D14	B-FRR-20	65.0-67.0 ft	31	23	15	8	2	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

Dilentancy: SLOW

Toughness: LOW

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GeoTesting express
a subsidiary of Geocomp Corporation

Client:	Haley & Aldrich, Inc.				
Project:	Presumpscot River Bridge				
Location:	Falmouth/Portland ME			Project No:	GTX-8629
Boring ID:	---	Sample Type:	---	Tested By:	mmd
Sample ID:	---	Test Date:	05/19/09	Checked By:	jdt
Depth :	---	Sample Id:	---		

Moisture Content of Soil - ASTM D 2216-05

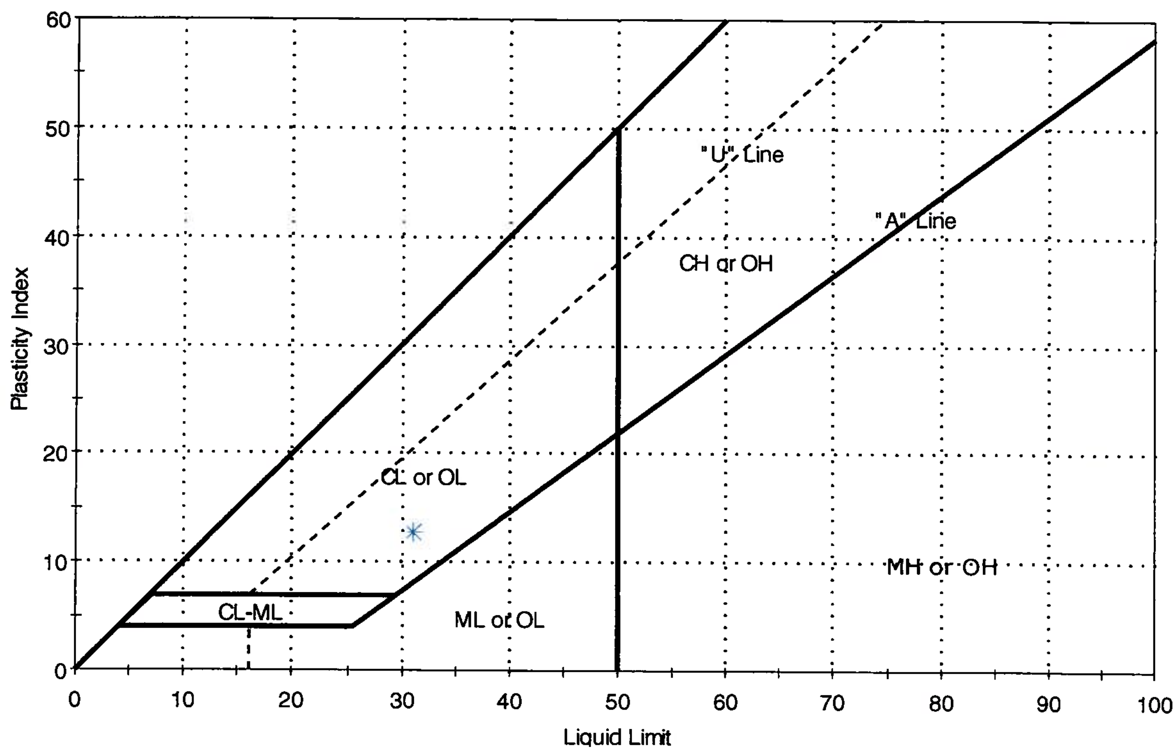
Boring ID	Sample ID	Depth	Description	Moisture Content,%
BB-FRR-202	U1	45.0-47.0 ft	Wet, greenish gray clay	39.3
BB-FRR-202	U2	72.0-74.0 ft	Moist, dark gray clay	32.2
BB-FRR-203	U2	40.0-42.0 ft	Wet, greenish gray clay	47.9

Notes: Temperature of Drying : 110° Celsius

GeoTesting express
a subsidiary of Geocomp Corporation

Client:	Haley & Aldrich, Inc.		
Project:	Presumpscot River Bridge		
Location:	Falmouth/Portland ME	Project No:	GTX-8629
Boring ID: BB-FRR-202	Sample Type: tube	Tested By:	cam
Sample ID:U1	Test Date: 05/13/09	Checked By:	jdt
Depth : 45.0-47.0 ft	Test Id: 152239		
Test Comment:	---		
Sample Description:	Wet, greenish gray clay		
Sample Comment:	---		

Atterberg Limits - ASTM D 4318-05



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
*	U1	B-FRR-20	45.0-47.0 ft	39	31	18	13	2	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

Dilentancy: RAPID

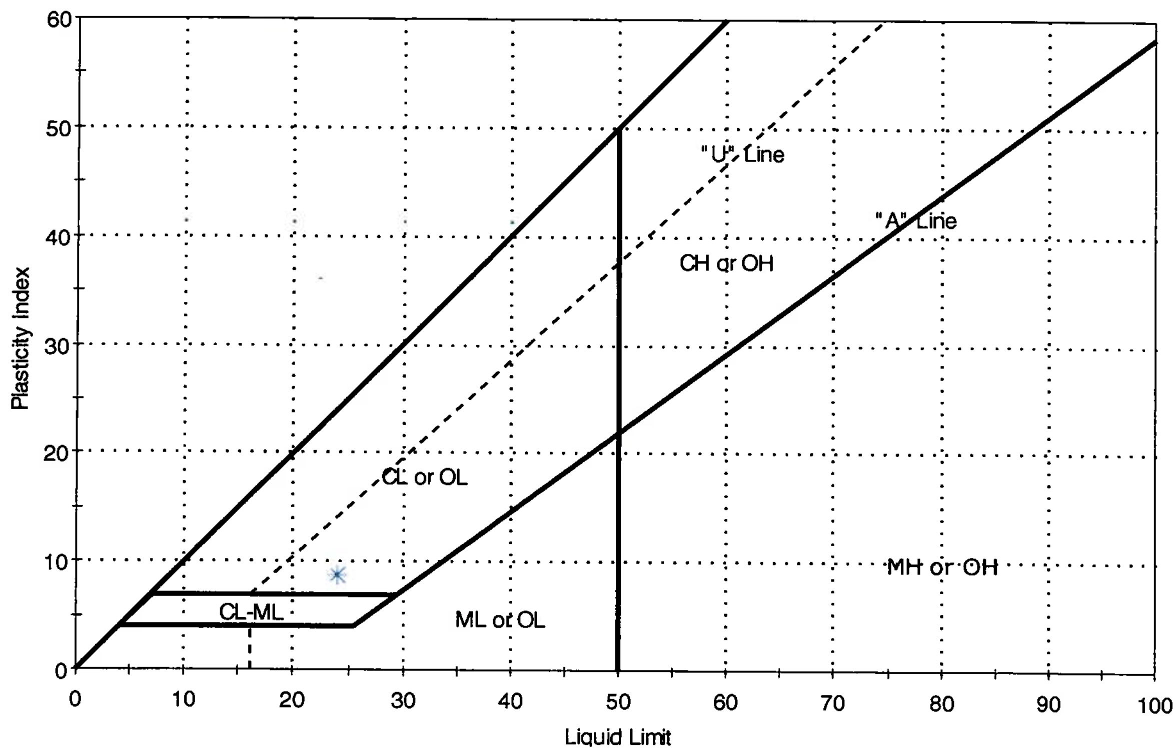
Toughness: LOW

GeoTesting express
a subsidiary of Geocomp Corporation

Client: Haley & Aldrich, Inc.			
Project: Presumpscot River Bridge			
Location: Falmouth/Portland ME		Project No:	GTX-8629
Boring ID: BB-FRR-202	Sample Type: tube	Tested By:	cam
Sample ID: U2	Test Date: 05/13/09	Checked By:	jdt
Depth : 72.0-74.0 ft	Test Id: 152240		
Test Comment: ---			
Sample Description: Moist, dark gray clay			
Sample Comment: ---			

Atterberg Limits - ASTM D 4318-05

Plasticity Chart



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
*	U2	B-FRR-20	72.0-74.0 ft	32	24	15	9	2	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

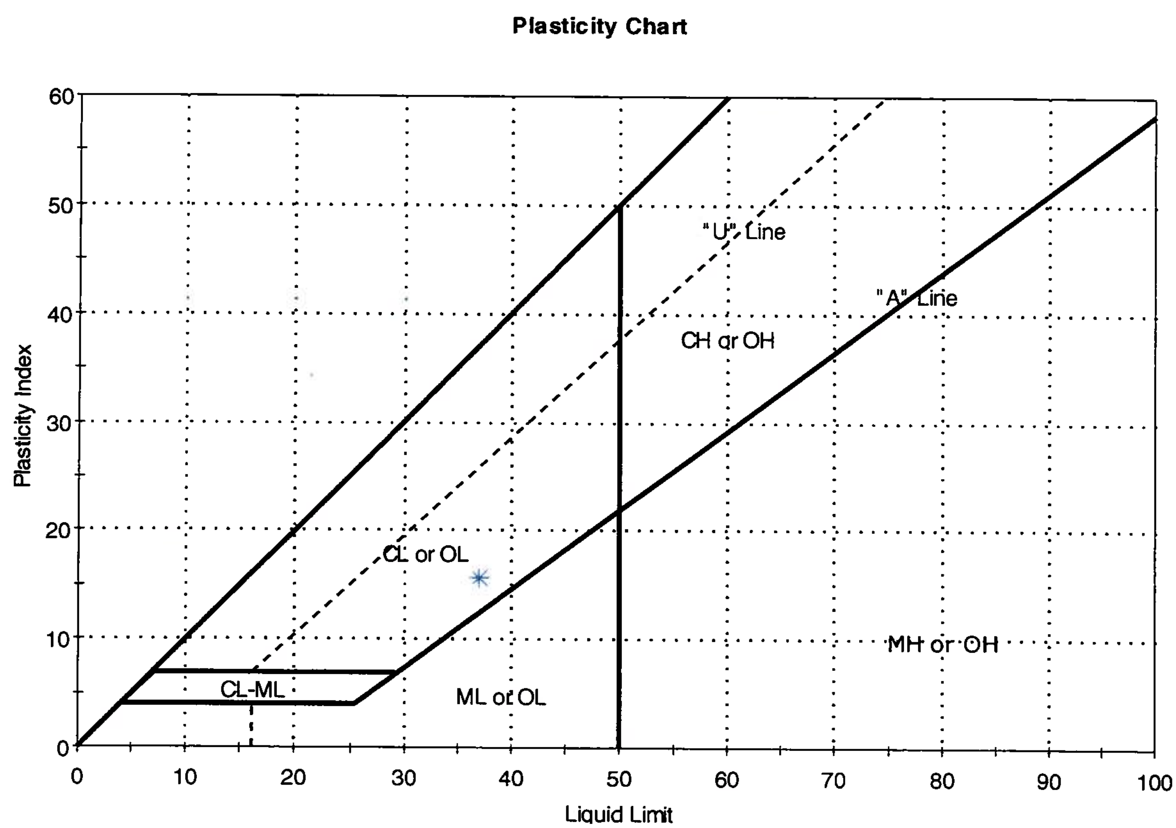
Dilentancy: RAPID

Toughness: LOW

GeoTesting express
a subsidiary of Geocomp Corporation

Client:	Haley & Aldrich, Inc.				
Project:	Presumpscot River Bridge				
Location:	Falmouth/Portland ME			Project No:	GTX-8629
Boring ID:	BB-FRR-203	Sample Type:	tube	Tested By:	cam
Sample ID:	U2	Test Date:	05/14/09	Checked By:	jdt
Depth :	40.0-42.0 ft	Test Id:	152241		
Test Comment:	---				
Sample Description:	Wet, greenish gray clay				
Sample Comment:	---				

Atterberg Limits - ASTM D 4318-05



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
*	U2	B-FRR-20	40.0-42.0 ft	48	37	21	16	2	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

Dilentancy: SLOW

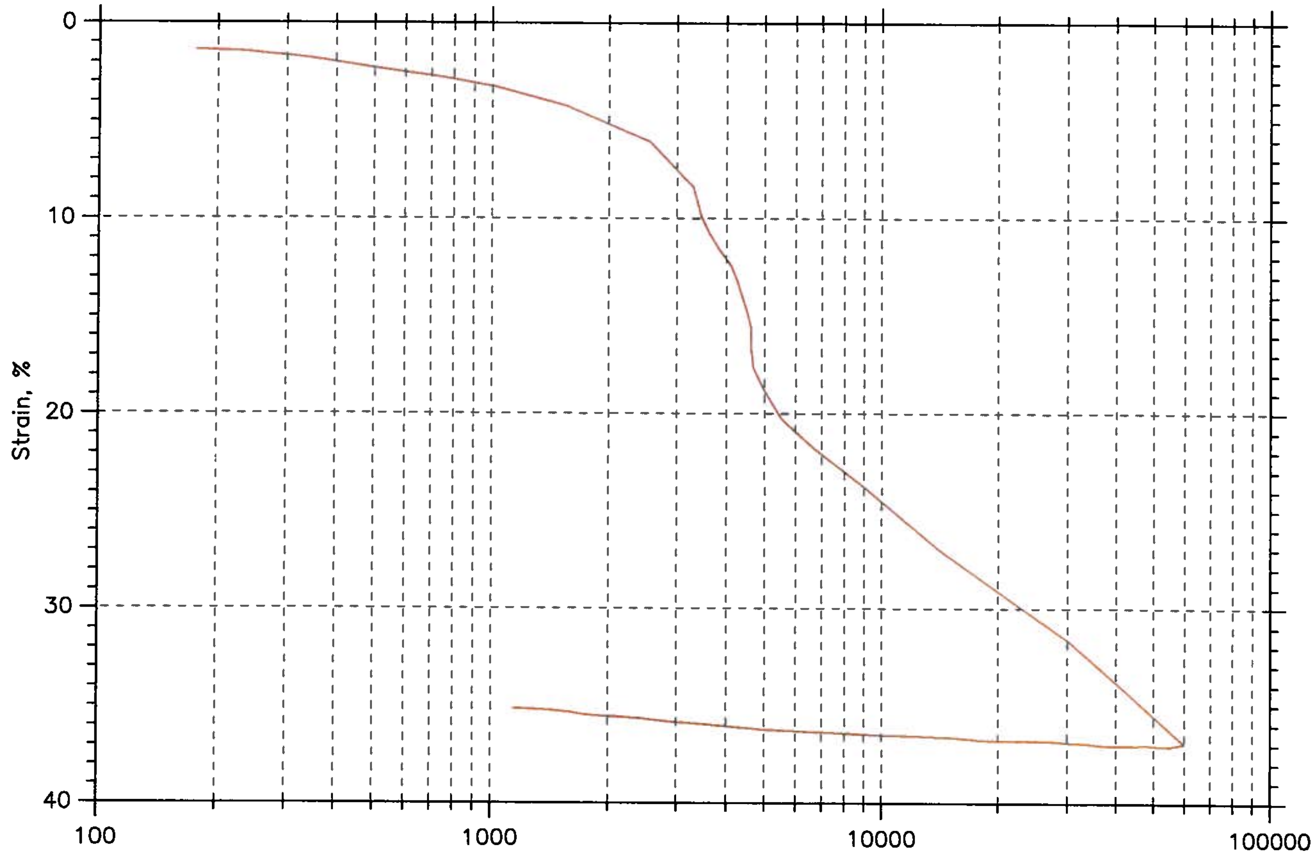
Toughness: LOW

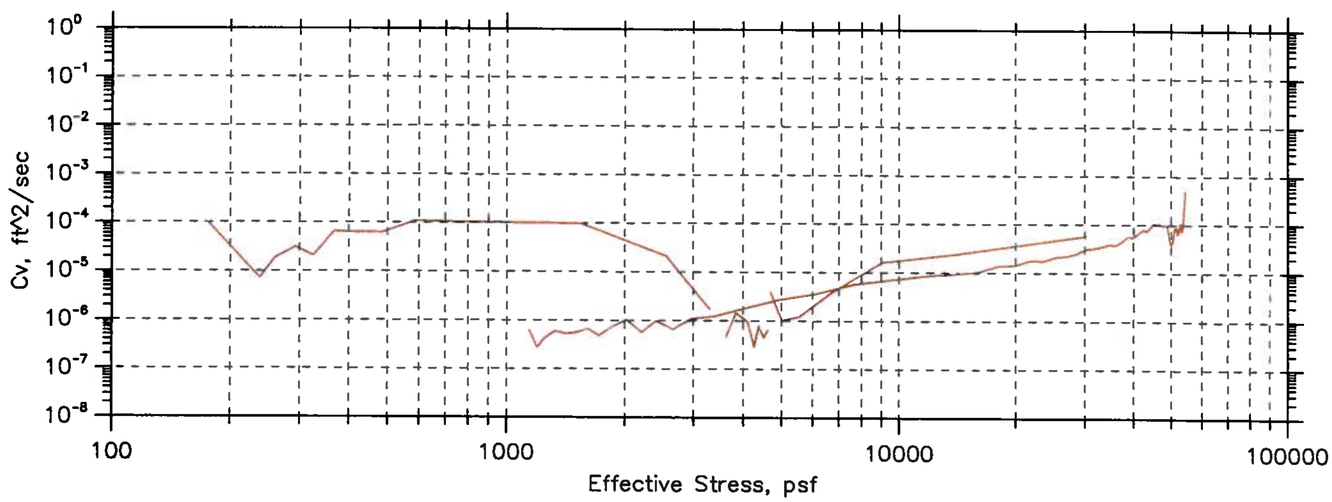
GeoTesting express
a subsidiary of Geocomp Corporation

Constant Rate of Consolidation

Constant Strain Rate by ASTM D4186

Summary Report





Project: Presumpscot River	Location: Falmouth/Portland	Project No.: GTX-8629
Boring No.: BB-FRR-202	Tested By: md	Checked By: jdt
Sample No.: U-1	Test Date: 05/14/09	Depth: 45-47 ft
Test No.: CRC-1	Sample Type: tube	Elevation: ---
Description: Wet, greenish gray clay		
Remarks: System E		

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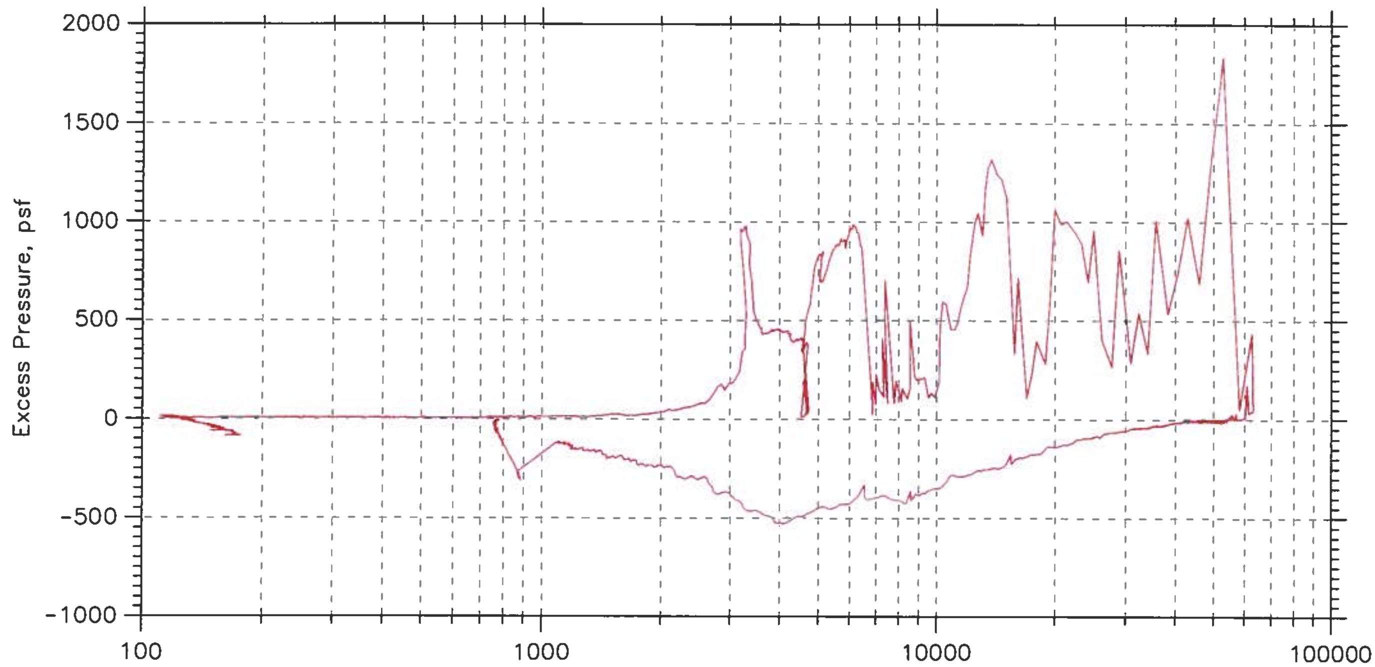
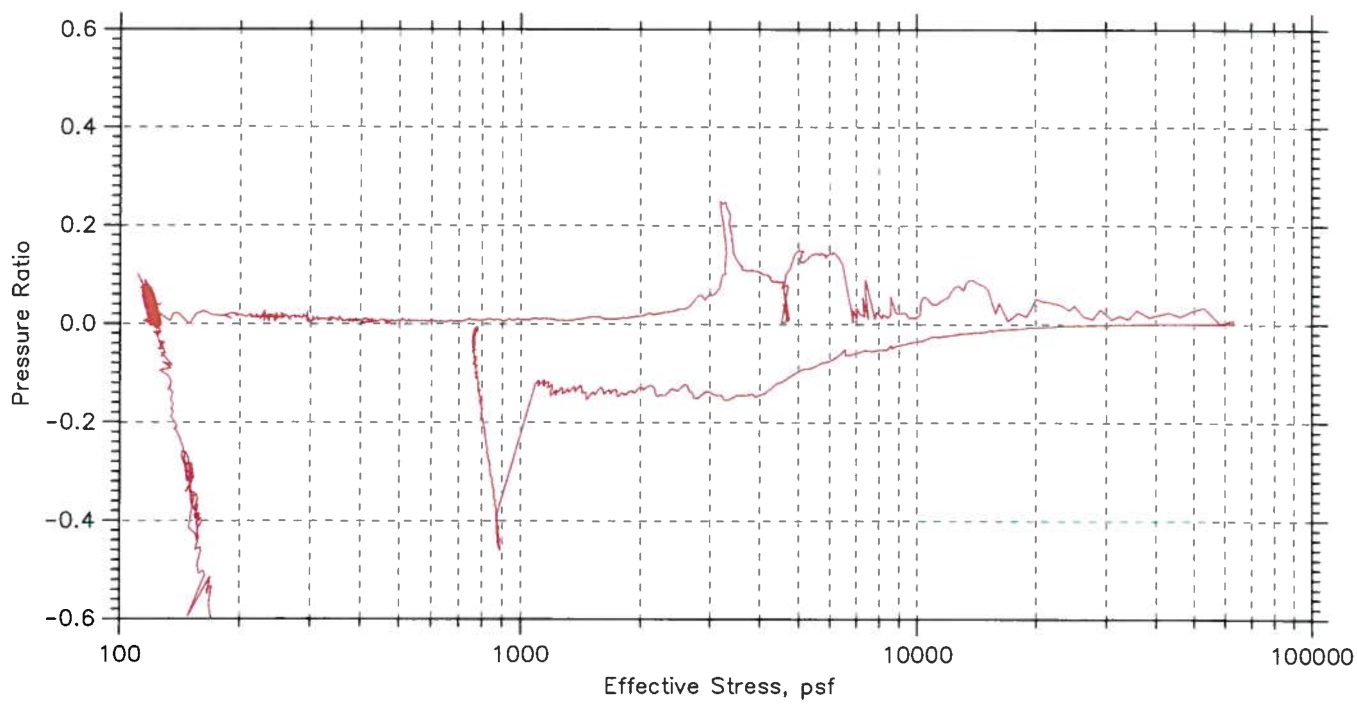
Constant Rate of Consolidation

Constant Strain Rate by ASTM D4186

Pressure Curves





Project: Presumpscot River	Location: Falmouth/Portland	Project No.: GTX-8629
Boring No.: BB-FRR-202	Tested By: md	Checked By: jdt
Sample No.: U-1	Test Date: 05/14/09	Depth: 45-47 ft
Test No.: CRC-1	Sample Type: tube	Elevation: ---
Description: Wet, greenish gray clay		
Remarks: System E		

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CRC TEST DATA

Project: Presumpscot River
Boring No.: BB-FRR-202
Sample No.: U-1
Test No.: CRC-1

Location: Falmouth/Portland
Tested By: md
Test Date: 05/14/09
Sample Type: tube

Project No.: GTX-8629
Checked By: jdt
Depth: 45-47 ft
Elevation: ---

Soil Description: Wet, greenish gray clay
Remarks: System E

Estimated Specific Gravity: 2.99
Initial Void Ratio: 1.42
Final Void Ratio: 0.56

Liquid Limit: 31
Plastic Limit: 18
Plasticity Index: 13

Initial Height: 1.00 in
Specimen Diameter: 2.50 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	3987	RING		3766
Wt. Container + Wet Soil, gm	117.99	258.5	230.17	123.93
Wt. Container + Dry Soil, gm	86.29	211.53	211.53	105.66
Wt. Container, gm	8.47	112.16	112.16	8.26
Wt. Dry Soil, gm	77.82	99.37	99.37	97.4
Water Content, %	40.74	47.27	18.76	18.76
Void Ratio	---	1.42	0.56	---
Degree of Saturation, %	---	99.39	100.00	---
Dry Unit Weight, pcf	---	77.12	119.71	---

Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

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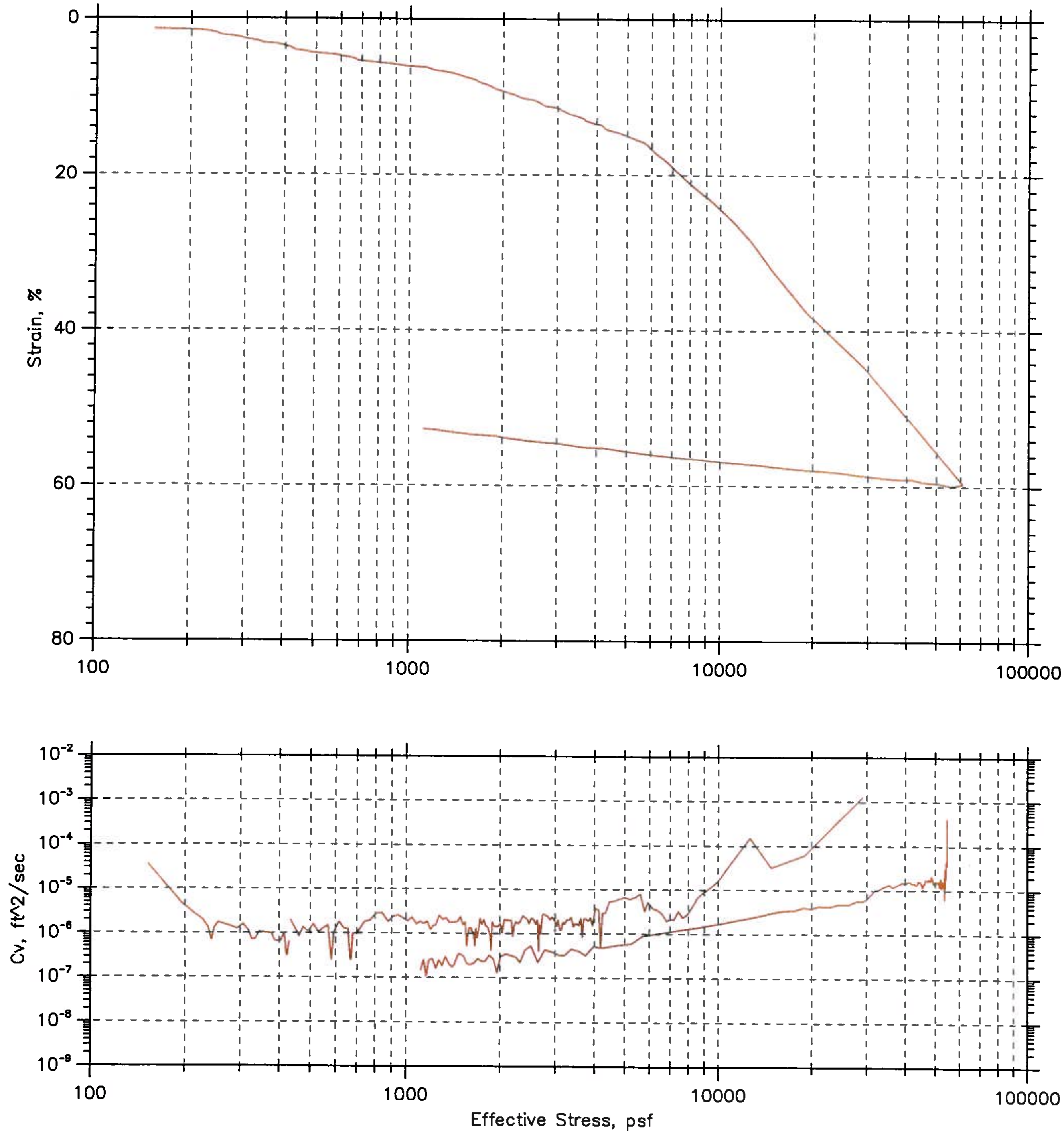
Constant Rate of Consolidation
Constant Strain Rate by ASTM D4186
Summary Report



Project: Presumpscot River	Location: Falmouth/Portland ME	Project No.: GTX-8629
Boring No.: BB-FRR-202	Tested By: md	Checked By: jdt
Sample No.: U-2	Test Date: 05/11/09	Depth: 72-74 ft
Test No.: CRC-2	Sample Type: tube	Elevation: ---
Description: Moist, dark gray clay		
Remarks: System S		

Tue, 19-MAY-2009 14:54:02

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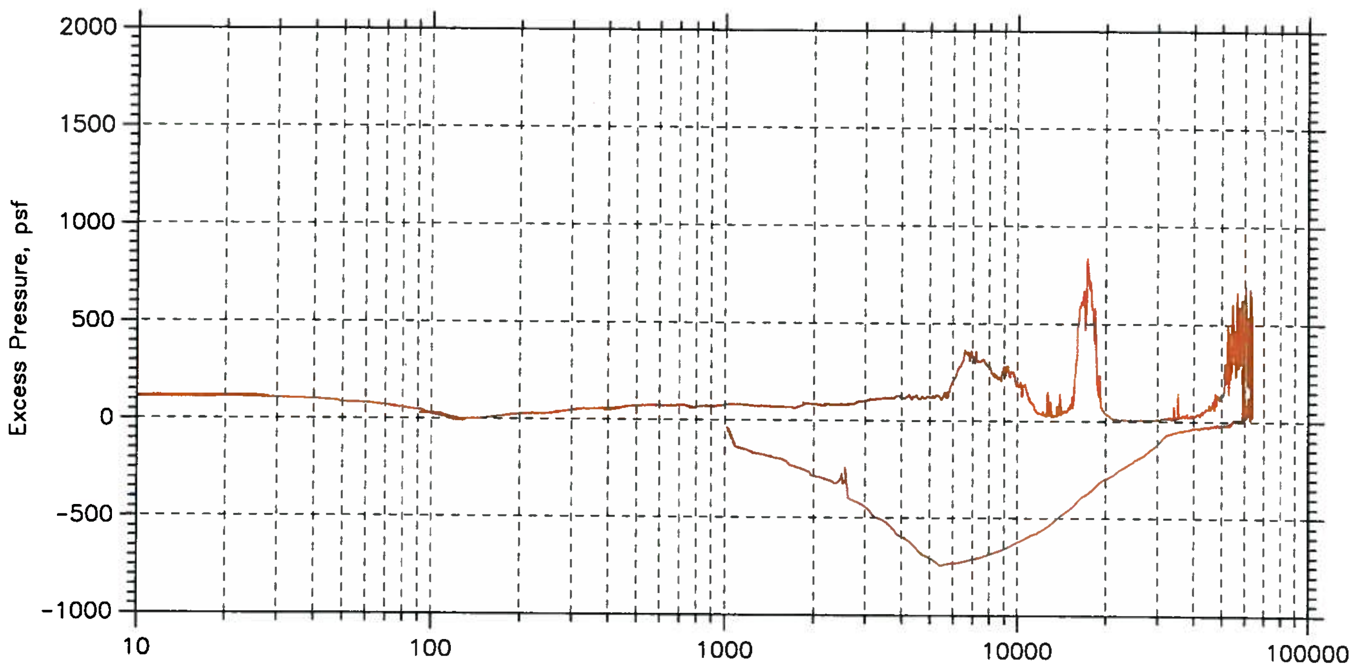
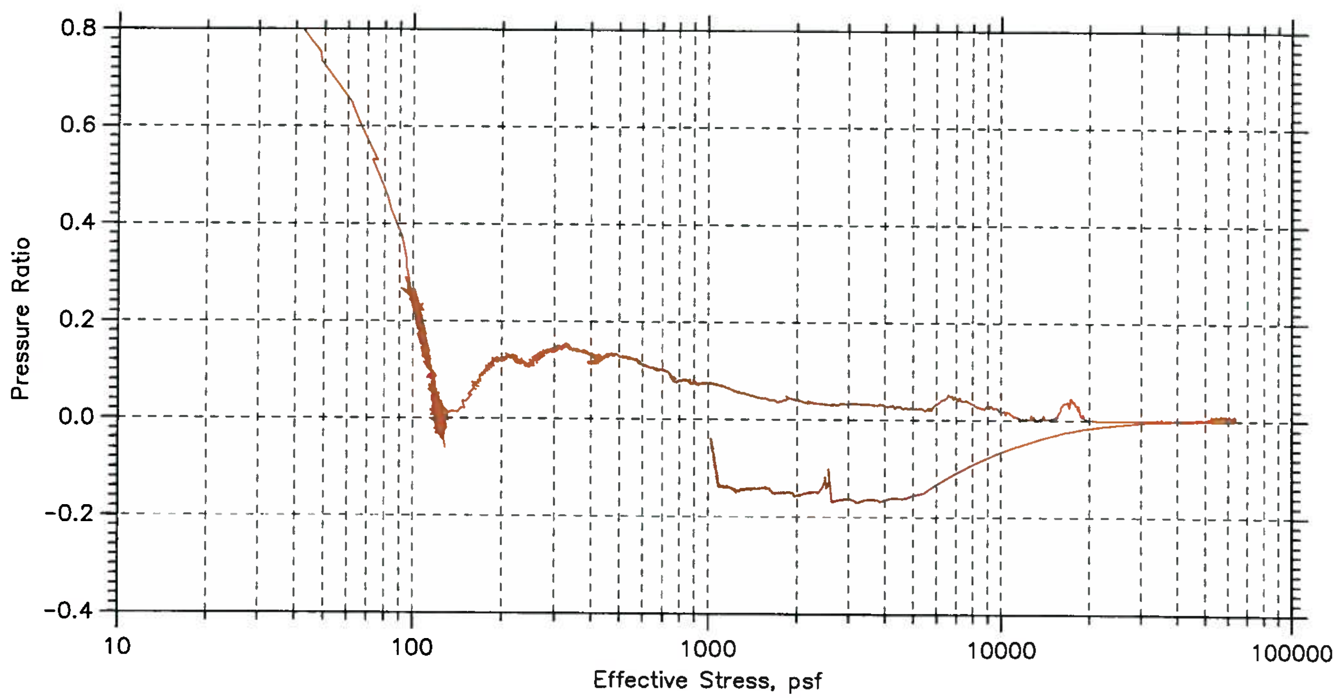
Constant Rate of Consolidation

Constant Strain Rate by ASTM D4186

Pressure Curves





Project: Presumpscot River	Location: Falmouth/Portland ME	Project No.: GTX-8629
Boring No.: BB-FRR-202	Tested By: md	Checked By: jdt
Sample No.: U-2	Test Date: 05/11/09	Depth: 72-74 ft
Test No.: CRC-2	Sample Type: tube	Elevation: ---
Description: Moist, dark gray clay		
Remarks: System S		

Tue, 19-MAY-2009 14:54:03

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CRC TEST DATA

Project: Presumpscot River	Location: Falmouth/Portland ME	Project No.: GTX-8629
Boring No.: BB-FRR-202	Tested By: md	Checked By: jdt
Sample No.: U-2	Test Date: 05/11/09	Depth: 72-74 ft
Test No.: CRC-2	Sample Type: tube	Elevation: ---

Soil Description: Moist, dark gray clay
Remarks: System S

Estimated Specific Gravity: 3.00	Liquid Limit: 24	Initial Height: 1.00 in
Initial Void Ratio: 2.05	Plastic Limit: 15	Specimen Diameter: 2.50 in
Final Void Ratio: 0.44	Plasticity Index: 9	

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	4228	RING		3894
Wt. Container + Wet Soil, gm	201.68	247.62	202.92	128.28
Wt. Container + Dry Soil, gm	150.5	191.27	191.27	112.9
Wt. Container, gm	8.26	112	112	8.3
Wt. Dry Soil, gm	142.24	79.265	79.265	104.6
Water Content, %	35.98	71.10	14.70	14.70
Void Ratio	---	2.05	0.44	---
Degree of Saturation, %	---	104.29	100.00	---
Dry Unit Weight, pcf	---	61.516	130.01	---

Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

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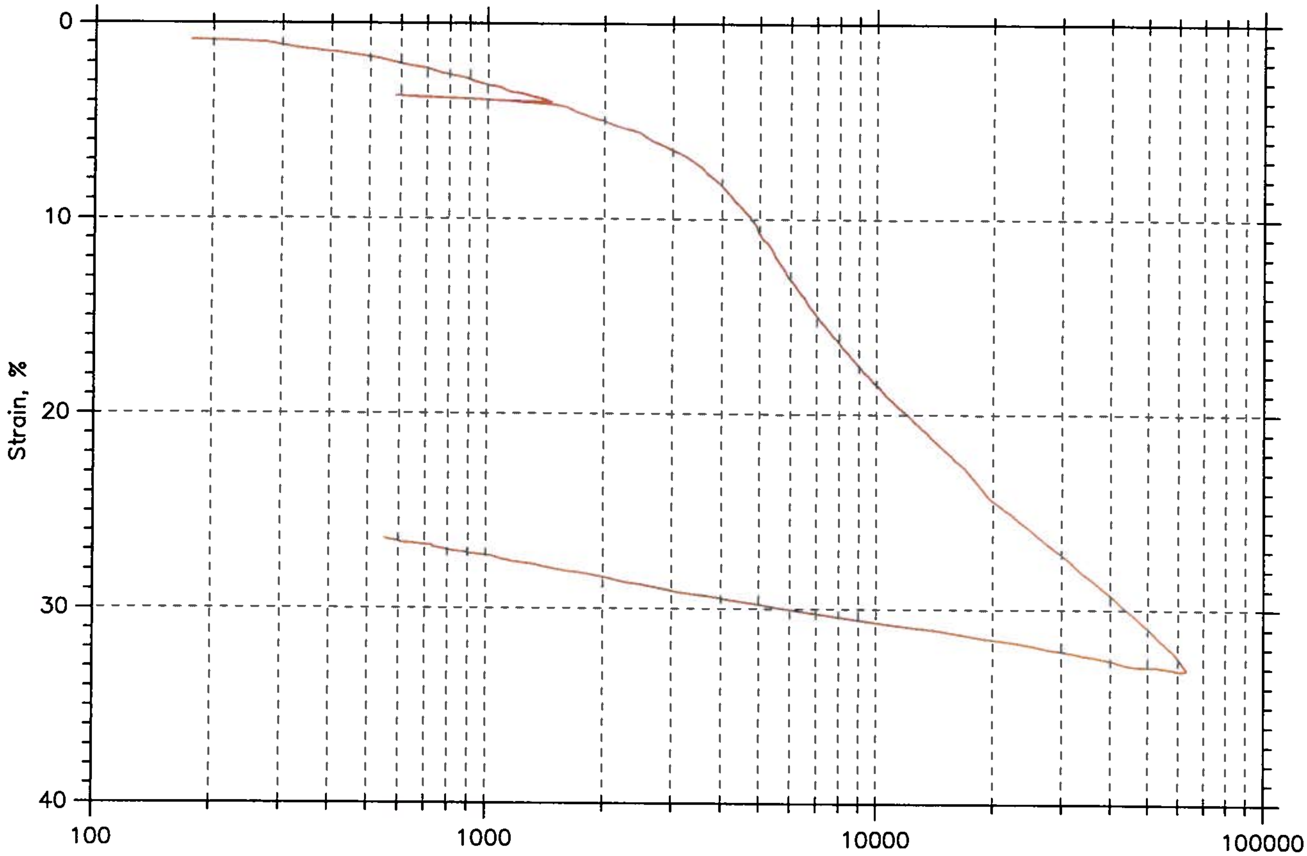
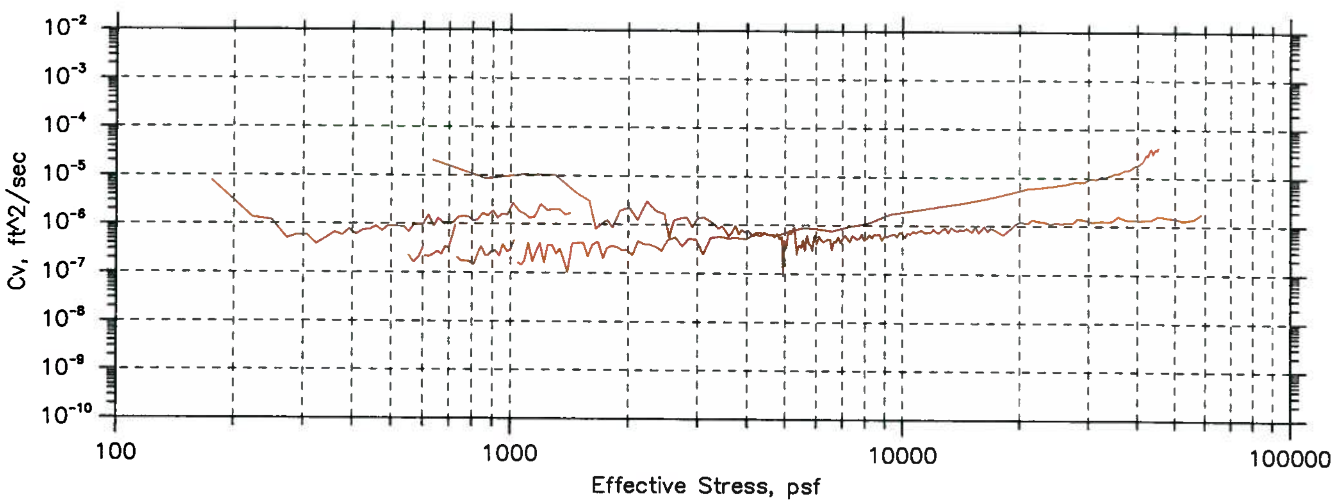
Constant Rate of Consolidation

Constant Strain Rate by ASTM D4186

Summary Report





Project: Presumpscot River	Location: Falmouth/Portland ME	Project No.: GTX-8629
Boring No.: BB-FRR-203	Tested By: md	Checked By: jdt
Sample No.: U-2	Test Date: 05/11/09	Depth: 40-42 ft
Test No.: CRC-3	Sample Type: tube	Elevation: ---
Description: Wet, greenish gray clay		
Remarks: System Q		

Tue, 19-MAY-2009 14:37:21

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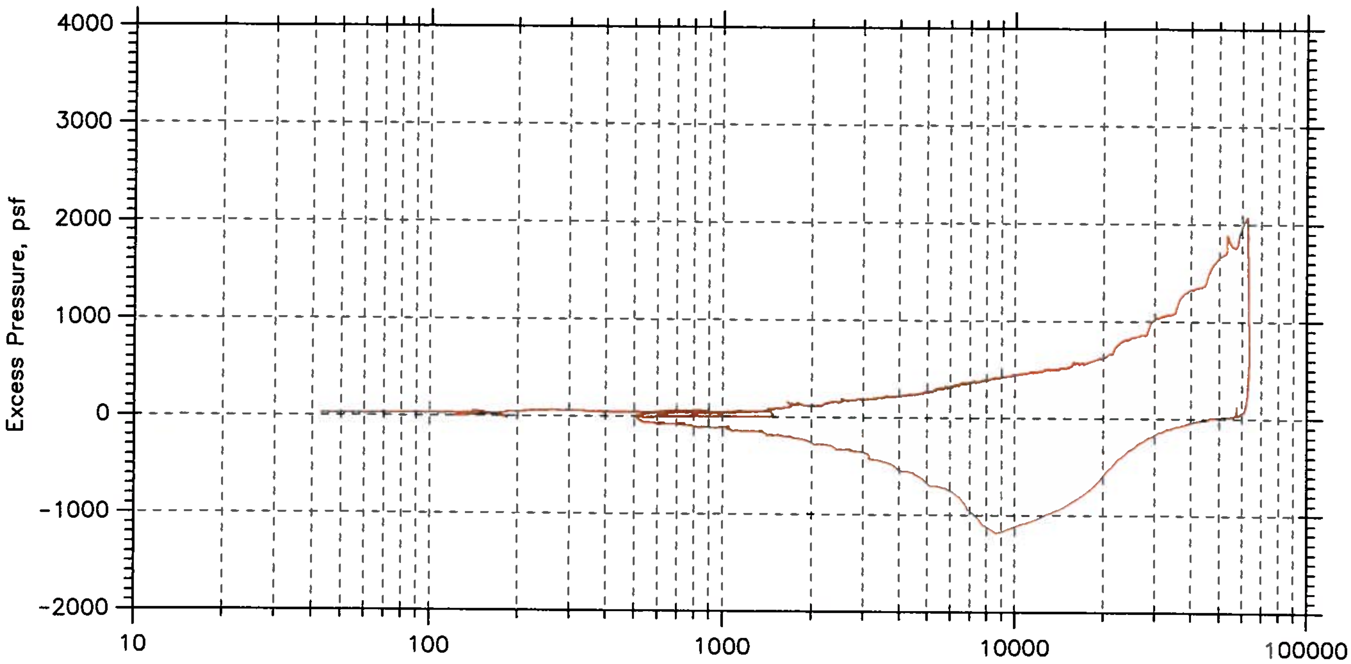
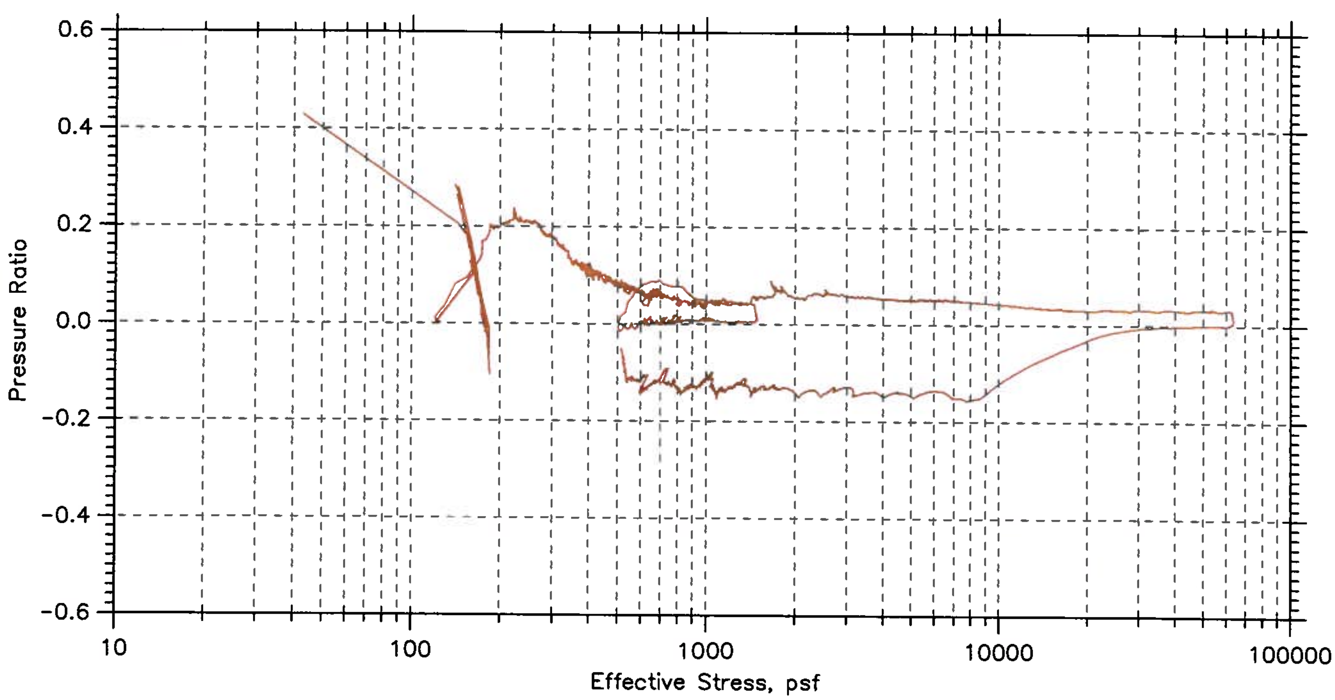
Constant Rate of Consolidation

Constant Strain Rate by ASTM D4186

Pressure Curves





Project: Presumpscot River	Location: Falmouth/Portland ME	Project No.: GTX-8629
Boring No.: BB-FRR-203	Tested By: md	Checked By: jdt
Sample No.: U-2	Test Date: 05/11/09	Depth: 40-42 ft
Test No.: CRC-3	Sample Type: tube	Elevation: ---
Description: Wet, greenish gray clay		
Remarks: System Q		

Tue, 19-MAY-2009 14:37:22

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CRC TEST DATA

Project: Presumpscot River
Boring No.: BB-FRR-203
Sample No.: U-2
Test No.: CRC-3

Location: Falmouth/Portland ME
Tested By: md
Test Date: 05/11/09
Sample Type: tube

Project No.: GTX-8629
Checked By: jdt
Depth: 40-42 ft
Elevation: ---

Soil Description: Wet, greenish gray clay
Remarks: System Q

Estimated Specific Gravity: 2.89
Initial Void Ratio: 1.52
Final Void Ratio: 0.84

Liquid Limit: 37
Plastic Limit: 21
Plasticity Index: 16

Initial Height: 1.00 in
Specimen Diameter: 2.50 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	3436	RING		3084
Wt. Container + Wet Soil, gm	177.22	348.9	330.57	41.77
Wt. Container + Dry Soil, gm	123.35	303.62	303.62	34.22
Wt. Container, gm	8.2	211.19	211.19	8.33
Wt. Dry Soil, gm	115.15	92.427	92.427	25.89
Water Content, %	46.78	48.99	29.16	29.16
Void Ratio	---	1.52	0.84	---
Degree of Saturation, %	---	93.38	100.00	---
Dry Unit Weight, pcf	---	71.731	97.963	---

Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

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Client:	Haley & Aldrich, Inc.
Project Name:	Presumpscot River Bridge
Project Location:	Falmouth & Portland Maine
GTX #:	8629
Test Date:	05/06/09
Tested By:	ema
Checked By:	jdt
Boring ID:	BB-FRR-202
Sample ID:	U1
Depth, ft:	45-47 ft

X-Ray of Soil Sample by ASTM D 4452

Top of Tube

Middle of Tube

Bottom of Tube

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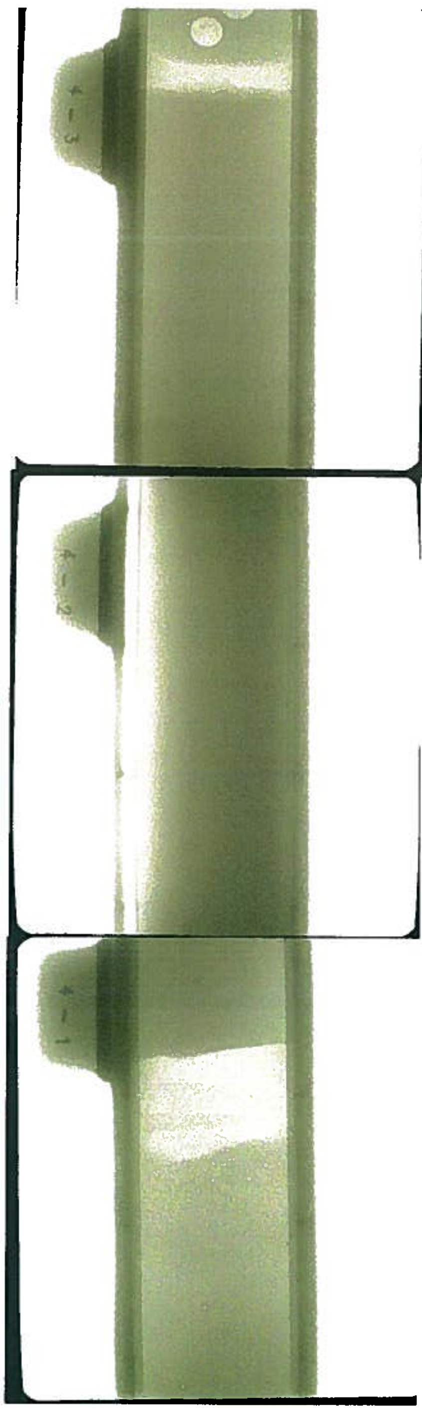
Client:	Haley & Aldrich, Inc.
Project Name:	Presumpscot River Bridge
Project Location:	Falmouth & Portland Maine
GTX #:	8629
Test Date:	05/06/09
Tested By:	ema
Checked By:	jdt
Boring ID:	BB-FRR-202
Sample ID:	U2
Depth, ft:	72-74 ft

X-Ray of Soil Sample by ASTM D 4452



Top of Tube

Middle of Tube

Bottom of Tube

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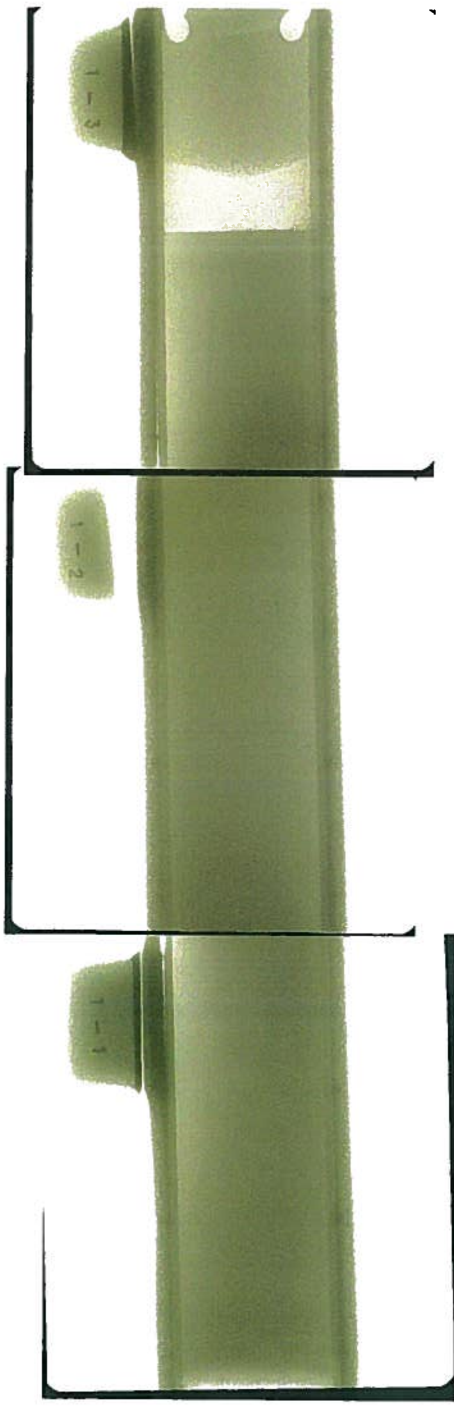
Client:	Haley & Aldrich, Inc.
Project Name:	Presumpscot River Bridge
Project Location:	Falmouth & Portland Maine
GTX #:	8629
Test Date:	05/06/09
Tested By:	ema
Checked By:	jdt
Boring ID:	BB-FRR-203
Sample ID:	U1
Depth, ft:	20-22 ft

X-Ray of Soil Sample by ASTM D 4452



Top of Tube

Middle of Tube

Bottom of Tube

GeoTesting express

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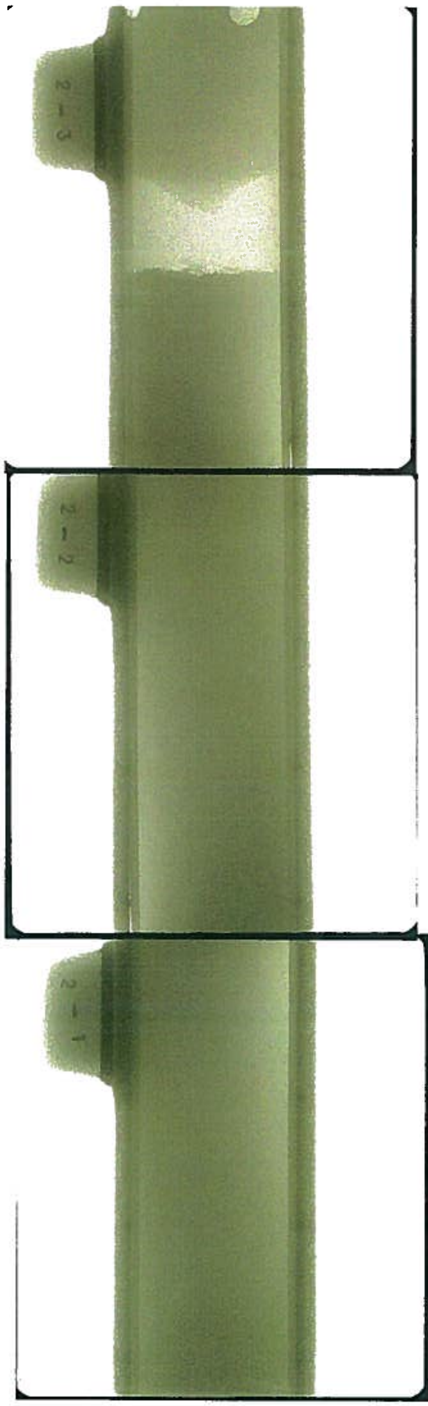
Client:	Haley & Aldrich, Inc.
Project Name:	Presumpscot River Bridge
Project Location:	Falmouth & Portland Maine
GTX #:	8629
Test Date:	05/06/09
Tested By:	ema
Checked By:	jdt
Boring ID:	BB-FRR-203
Sample ID:	U2
Depth, ft:	40-42 ft

X-Ray of Soil Sample by ASTM D 4452



Top of Tube

Middle of Tube

Bottom of Tube

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1145 Massachusetts Avenue
Boxborough, MA 01719
978 635 0424 Tel
978 635 0266 Fax

RECEIVED
BY
JUL - 8 2009
HALEY & ALDRICH
PORTLAND, MAINE

Transmittal

TO:
Mr. Bryan Steinert
Haley & Aldrich, Inc.
75 Washington Avenue, Suite 203
Portland, ME 04101-2617

DATE: 6/30/09	GTX NO: 8629
RE: Presumpscot River Bridge Project	

COPIES	DATE	DESCRIPTION
1	6/30/09	**June 2009 Laboratory Test Reports**

REMARKS:

SIGNED:
Joe Tomei – Laboratory Manager

APPROVED BY:
Mark Dobday – Laboratory Manager

GeoTesting
express
a subsidiary of Geocomp Corporation

Boston
Atlanta
New York

www.geocomp.com/geotesting

June 30, 2009

Mr. Bryan Steinert
Haley & Aldrich, Inc.
75 Washington Avenue, Suite 203
Portland, ME 04101-2617

Re: Presumpscot River Bridge Project (GTX-8629)

Dear Mr. Steinert:

Enclosed are the test results you requested for the above referenced project. GeoTesting Express, Inc. (GTX) received the following sample from you on May 4, 2009. This sample was labeled as follows:

BB-FRR-203, U1 (20-22 ft)

GTX performed the following tests on this sample:

Moisture Content test (ASTM D 2216)
Atterberg Limits test (ASTM D 4318)
CRS Consolidation test (ASTM D 4186)
X-Ray of Soil

As requested, the x-ray test was performed on the tube sample first and the reports were sent to H&A. After review of the x-ray reports, H&A provided GTX locations within the tubes to cut specimens for testing. Copies of your test requests are attached.

The results presented in this report apply only to the items tested. This report shall not be reproduced except in full, without written approval from GeoTesting Express. The remainder of these samples will be retained for a period of sixty (60) days and will then be discarded unless otherwise notified by you. Please call me if you have any questions or require additional information. Thank you for allowing GeoTesting Express the opportunity of providing you with testing services. We look forward to working with you again in the future.

Respectfully yours,

Joe Tomei
Laboratory Manager

GeoTesting express
a subsidiary of Geocomp Corporation

1145 Massachusetts Avenue
Boxborough, MA 01719
978 635 0424 Tel
978 635 0266 Fax

Geotechnical Test Report

June 30, 2009

GTX-8629 Presumpscot River Bridge Project

Falmouth/Portland, ME

Prepared for:

HALEY & ALDRICH

GeoTesting express
a subsidiary of Geocomp Corporation

Client:	Haley & Aldrich, Inc.				
Project:	Presumpscot River Bridge				
Location:	Falmouth/Portland ME			Project No:	GTX-8629
Boring ID:	BB-FRR-203	Sample Type:	tube	Tested By:	mmd
Sample ID:	U1	Test Date:	06/18/09	Checked By:	jdt
Depth :	20.0-22.0 ft	Sample Id:	72084		
Test Comment:	---				
Sample Description:	Wet, gray clay				
Sample Comment:	---				

Moisture Content of Soil - ASTM D 2216-05

Boring ID	Sample ID	Depth	Description	Moisture Content,%
BB-FRR-203	U1	20.0-22.0 ft	Wet, gray clay	53.2

Notes: Temperature of Drying : 110º Celsius

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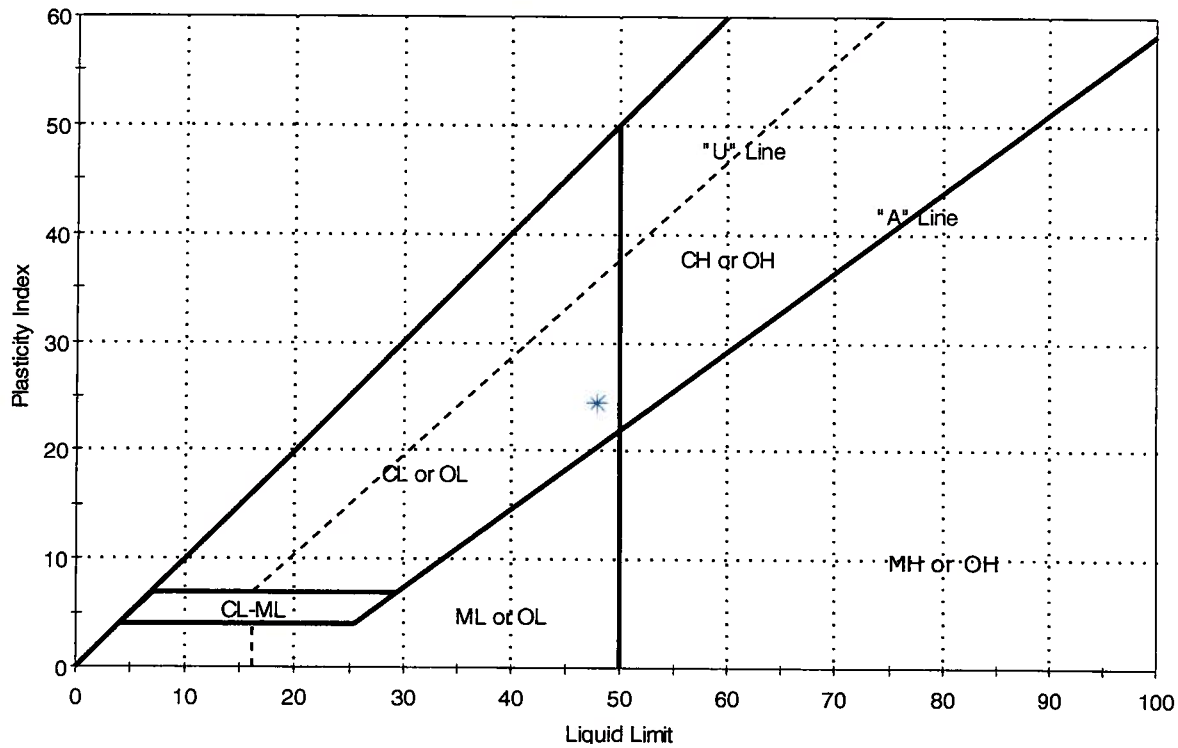
Client:	Haley & Aldrich, Inc.				
Project:	Presumpscot River Bridge				
Location:	Falmouth/Portland ME			Project No:	GTX-8629
Boring ID:	BB-FRR-203	Sample Type:	tube	Tested By:	cam
Sample ID:	U1	Test Date:	06/12/09	Checked By:	jdt
Depth :	20.0-22.0 ft	Test Id:	154318		
Test Comment:	---				
Sample Description:	Wet, gray clay				
Sample Comment:	---				

Atterberg Limits - ASTM D 4318-05



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
*	U1	B-FRR-20	20.0-22.0 ft	53	48	23	25	1	

Sample Prepared using the WET method

Dry Strength: VERY HIGH
Dilentancy: SLOW
Toughness: LOW

printed 6/18/2009 7:37:30 AM

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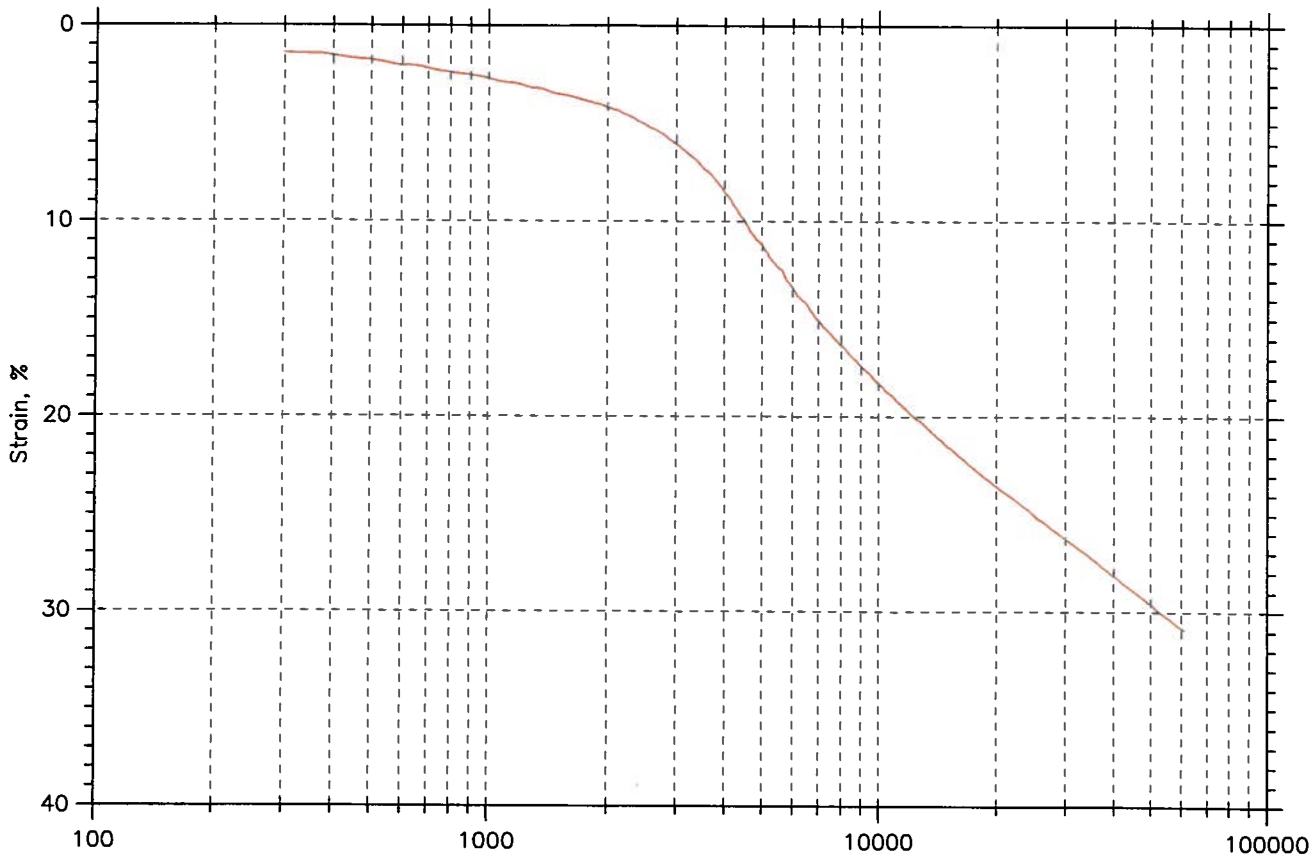
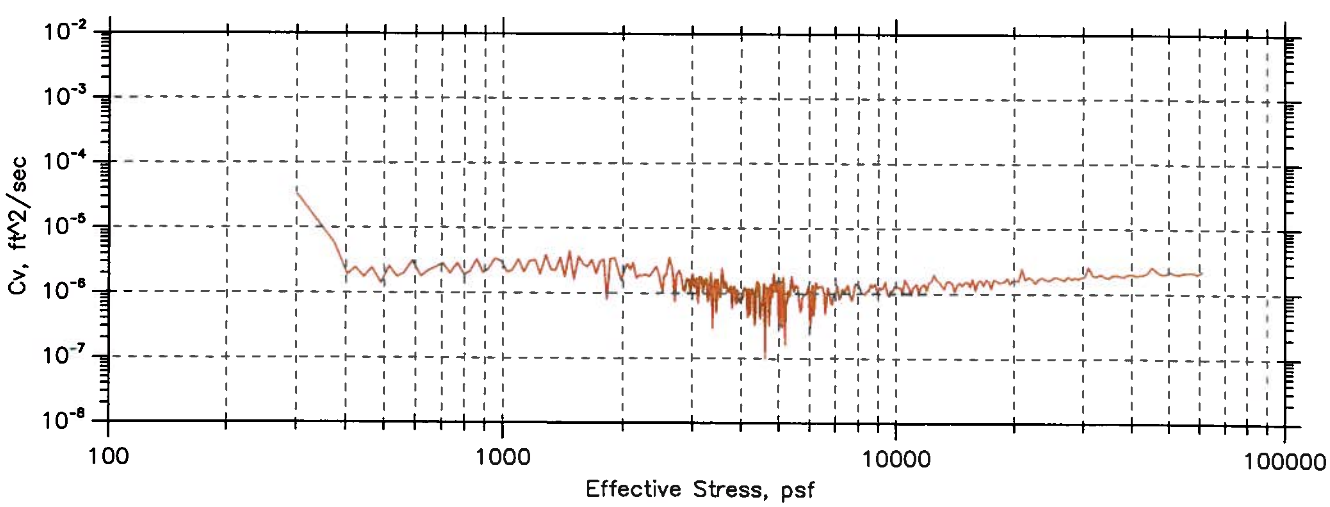
Constant Rate of Consolidation

Constant Strain Rate by ASTM D4186

Summary Report





Project: Presumpscot River	Location: Falmouth/Portland	Project No.: GTX-8629
Boring No.: BB-FRR-203	Tested By: njh	Checked By: jdt
Sample No.: U-1	Test Date: 06/11/09	Depth: 20-22 ft
Test No.: CRC-8	Sample Type: tube	Elevation: ---
Description: Wet, gray clay		
Remarks: System E		

Wed, 17-JUN-2009 13:46:20

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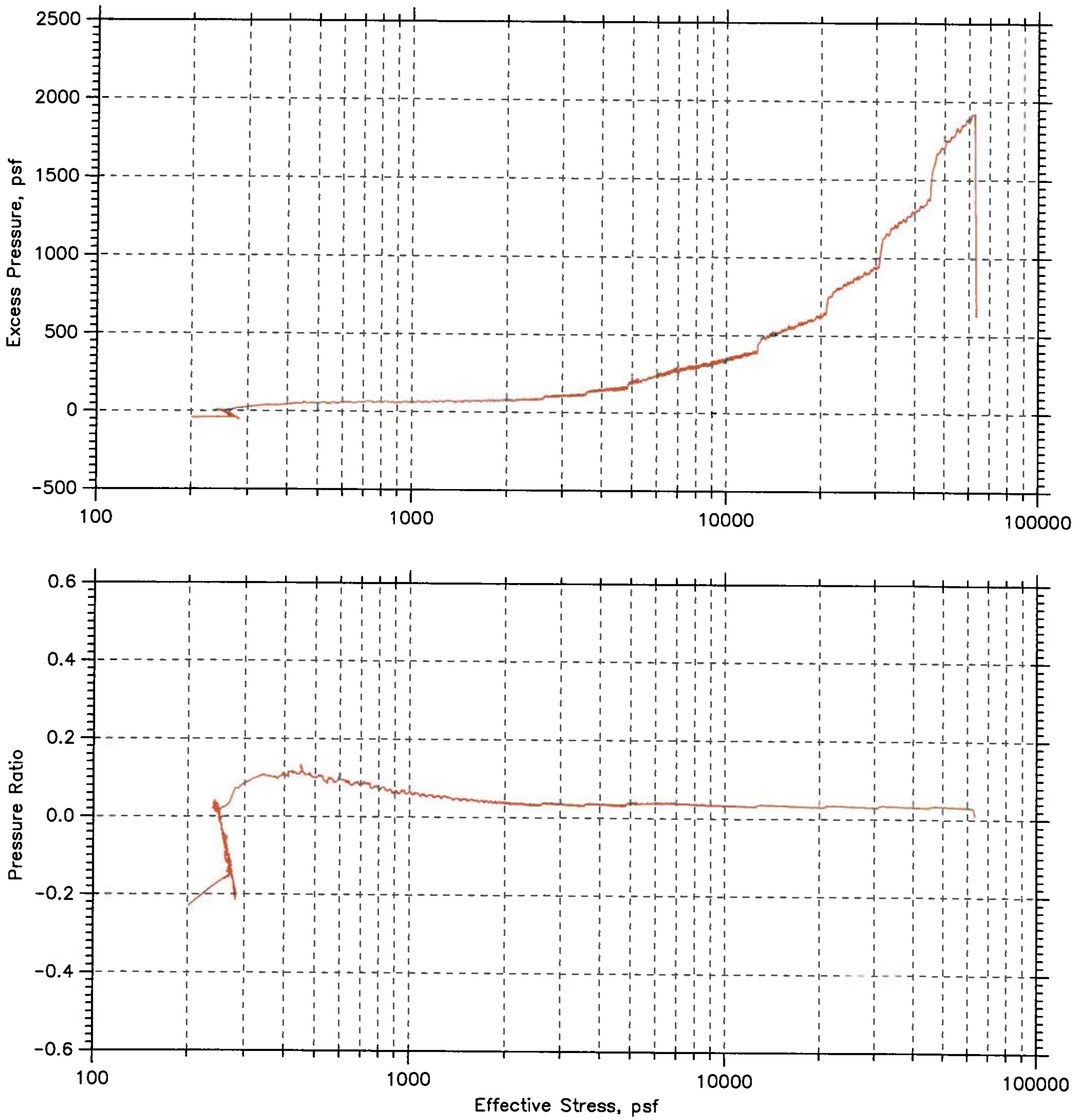
Constant Rate of Consolidation

Constant Strain Rate by ASTM D4186

Pressure Curves



Project: Presumpscot River	Location: Falmouth/Portland	Project No.: GTX-8629
Boring No.: BB-FRR-203	Tested By: njh	Checked By: jdt
Sample No.: U-1	Test Date: 06/11/09	Depth: 20-22 ft
Test No.: CRC-8	Sample Type: tube	Elevation: ---
Description: Wet, gray clay		
Remarks: System E		

Wed, 17-JUN-2009 13:46:21

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CRC TEST DATA

Project: Presumpscot River	Location: Falmouth/Portland	Project No.: GTX-8629
Boring No.: BB-FRR-203	Tested By: njh	Checked By: jdt
Sample No.: U-1	Test Date: 06/11/09	Depth: 20-22 ft
Test No.: CRC-8	Sample Type: tube	Elevation: ---

Soil Description: Wet, gray clay
Remarks: System E

Estimated Specific Gravity: 3.61	Liquid Limit: 48	Initial Height: 1.00 in
Initial Void Ratio: 2.41	Plastic Limit: 23	Specimen Diameter: 2.50 in
Final Void Ratio: 1.23	Plasticity Index: 25	

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	4482	RING		4177
Wt. Container + Wet Soil, gm	162.75	249.81	226.26	126
Wt. Container + Dry Soil, gm	109.06	197.13	197.13	96.12
Wt. Container, gm	8.08	112	112	8.79
Wt. Dry Soil, gm	100.98	85.132	85.132	87.33
Water Content, %	53.17	61.88	34.22	34.22
Void Ratio	---	2.41	1.23	---
Degree of Saturation, %	---	92.68	100.00	---
Dry Unit Weight, pcf	---	66.07	100.8	---

Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

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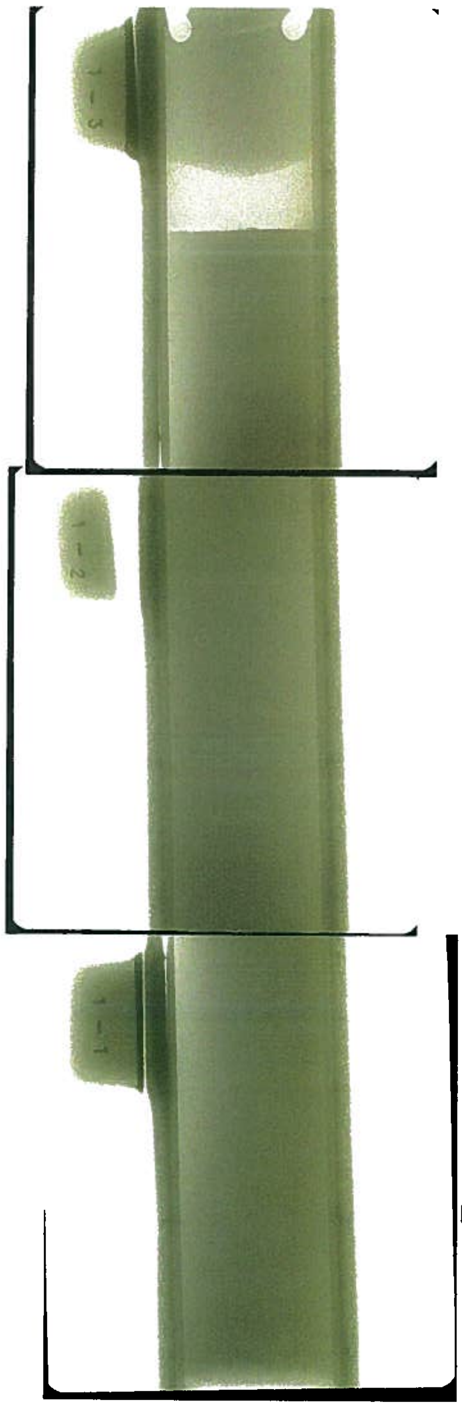
a subsidiary of Geocomp Corporation

Client:	Haley & Aldrich, Inc.
Project Name:	Presumpscot River Bridge
Project Location:	Falmouth & Portland Maine
GTX #:	8629
Test Date:	05/06/09
Tested By:	ema
Checked By:	jdt
Boring ID:	BB-FRR-203
Sample ID:	U1
Depth, ft:	20-22 ft

X-Ray of Soil Sample by ASTM D 4452



Top of Tube

Middle of Tube

Bottom of Tube

GeoTesting express
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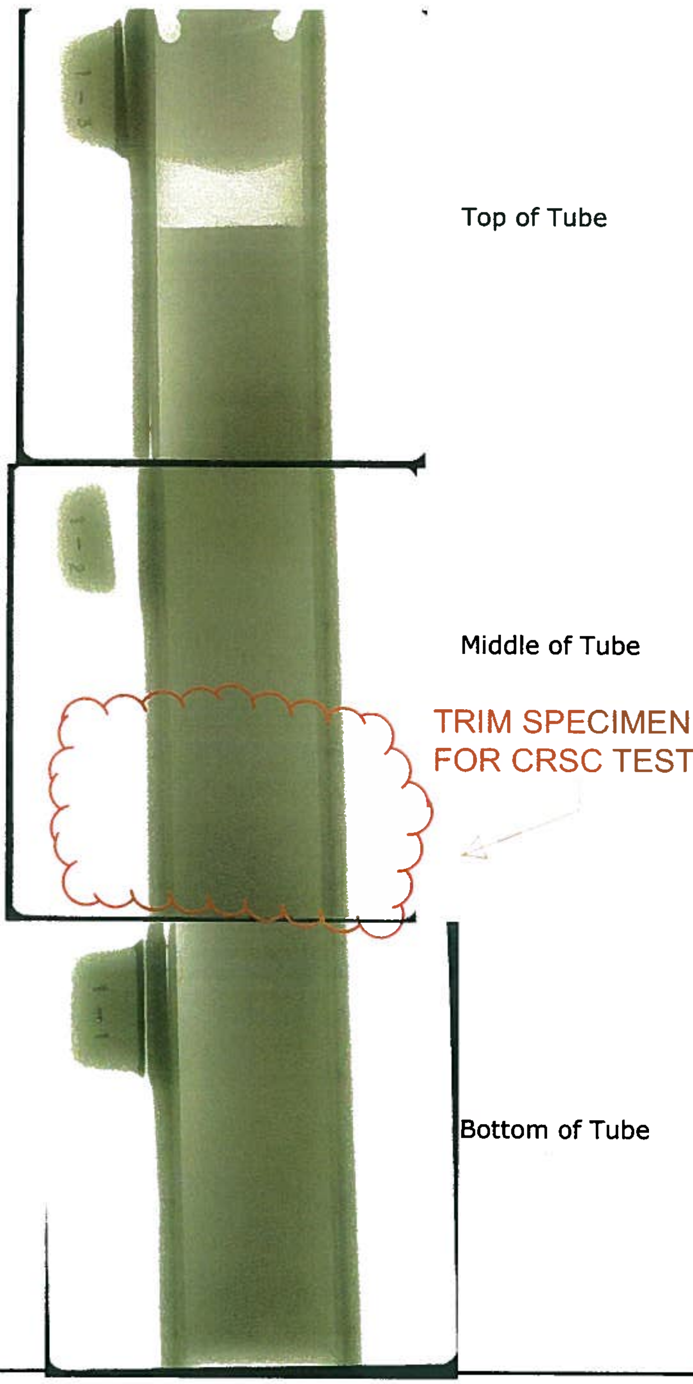
Client:	Haley & Aldrich, Inc.
Project Name:	Presumpscot River Bridge
Project Location:	Falmouth & Portland Maine
GTX #:	8629
Test Date:	05/06/09
Tested By:	ema
Checked By:	jdt
Boring ID:	BB-FRR-203
Sample ID:	U1
Depth, ft:	20-22 ft

X-Ray of Soil Sample by ASTM D 4452



GeoTesting
express

WARRANTY and LIABILITY

GeoTesting Express (GTX) warrants that all tests it performs are run in general accordance with the specified test procedures and accepted industry practice. GTX will correct or repeat any test that does not comply with this warranty. GTX has no specific knowledge as to conditioning, origin, sampling procedure or intended use of the material.

GTX may report engineering parameters that require us to interpret the test data. Such parameters are determined using accepted engineering procedures. However, GTX does not warrant that these parameters accurately reflect the true engineering properties of the *in situ* material. Responsibility for interpretation and use of the test data and these parameters for engineering and/or construction purposes rests solely with the user and not with GTX or any of its employees.

GTX's liability will be limited to correcting or repeating a test which fails our warranty. GTX's liability for damages to the Purchaser of testing services for any cause whatsoever shall be limited to the amount GTX received for the testing services. GTX will not be liable for any damages, or for any lost benefits or other consequential damages resulting from the use of these test results, even if GTX has been advised of the possibility of such damages. GTX will not be responsible for any liability of the Purchaser to any third party.

Commonly Used Symbols

A	pore pressure parameter for $\Delta\sigma_1 - \Delta\sigma_3$
B	pore pressure parameter for $\Delta\sigma_3$
CIU	isotropically consolidated undrained triaxial shear test
CR	compression ratio for one dimensional consolidation
C_c	coefficient of curvature, $(D_{30})^2 / (D_{10} \times D_{60})$
C_u	coefficient of uniformity, D_{60}/D_{10}
C_c	compression index for one dimensional consolidation
C_α	coefficient of secondary compression
c_v	coefficient of consolidation
c	cohesion intercept for total stresses
c'	cohesion intercept for effective stresses
D	diameter of specimen
D_{10}	diameter at which 10% of soil is finer
D_{15}	diameter at which 15% of soil is finer
D_{30}	diameter at which 30% of soil is finer
D_{50}	diameter at which 50% of soil is finer
D_{60}	diameter at which 60% of soil is finer
D_{85}	diameter at which 85% of soil is finer
d_{50}	displacement for 50% consolidation
d_{90}	displacement for 90% consolidation
d_{100}	displacement for 100% consolidation
E	Young's modulus
e	void ratio
e_c	void ratio after consolidation
e_o	initial void ratio
G	shear modulus
G_s	specific gravity of soil particles
H	height of specimen
PI	plasticity index
i	gradient
K_o	lateral stress ratio for one dimensional strain
k	permeability
LI	Liquidity Index
m_v	coefficient of volume change
n	porosity
PI	plasticity index
P_c	preconsolidation pressure
p	$(\sigma_1 + \sigma_3) / 2$, $(\sigma_v + \sigma_h) / 2$
p'	$(\sigma'_1 + \sigma'_3) / 2$, $(\sigma'_v + \sigma'_h) / 2$
p'_c	p' at consolidation
Q	quantity of flow
q	$(\sigma_1 - \sigma_3) / 2$
q_f	q at failure
q_o, q_i	initial q
q_c	q at consolidation
S	degree of saturation
SL	shrinkage limit
s_u	undrained shear strength
T	time factor for consolidation
T	temperature
t	time
U, UC	unconfined compression test
UU, Q	unconsolidated undrained triaxial test
u_a	pore gas pressure
u_e	excess pore water pressure
u, u_w	pore water pressure
V	total volume
V_g	volume of gas
V_s	volume of solids
V_v	volume of voids
V_w	volume of water
V_o	initial volume
v	velocity
W	total weight
W_s	weight of solids
W_w	weight of water
w	water content
w_c	water content at consolidation
w_f	final water content
w_l	liquid limit
w_n	natural water content
w_p	plastic limit
w_s	shrinkage limit
w_o, w_i	initial water content
α	slope of q_f versus p_f
α'	slope of q_f versus p_f'
γ_t	total unit weight
γ_d	dry unit weight
γ_s	unit weight of solids
γ_w	unit weight of water
ε	strain
ε_{vol}	volume strain
ε_h, ε_v	horizontal strain, vertical strain
μ	Poisson's ratio, also viscosity
σ	normal stress
σ'	effective normal stress
σ_c, σ'_c	consolidation stress in isotropic stress system
σ_h, σ'_h	horizontal normal stress
σ_v, σ'_v	vertical normal stress
σ_1	major principal stress
σ_2	intermediate principal stress
σ_3	minor principal stress
τ	shear stress
φ	friction angle based on total stresses
φ'	friction angle based on effective stresses
φ'_r	residual friction angle
φ_{ult}	φ for ultimate strength

SECTION 3

SECTION 3 – TABLE OF CONTENTS

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HALEY& ALDRICH

MEMORANDUM

7 November 2008
File No. 35524-000

TO: T.Y. Lin International
Rick Hebert, P.E.

C: MaineDOT
Leanne Timberlake, Laura Krusinski

FROM: Haley & Aldrich, Inc.
Bryan C. Steinert, Wayne A. Chadbourne, P.E.
BCS

SUBJECT: Preliminary Cost Estimate
EPS Geofoam Embankment Alternative
Proposed Route 100/26 Bridge Replacement Project
MaineDOT PIN 15094.00
Falmouth, Maine

As discussed in the project meeting on 29 October, we have completed a preliminary-level cost estimate for the lightweight-fill embankment alternative consisting solely of EPS geofoam (geofoam) between the Presumpscot River and the Maine Central Railroad (MCRR) tracks. For the purposes of our evaluation, we assumed that the proposed alignment would be located approximately 50 ft east of the current bridge structure. The total length of the embankment would be approximately 360 ft. The northern and southern limits of the embankment would consist of abutments for the river and MCRR bridge structures (see attached Sketch No. 1 for approximately embankment location). The width and maximum height of the embankment was assumed to be 50 ft.

We have assumed a 30-in. thick pavement structure would be underlain by a 6-in. thick reinforced concrete distribution slab, a 12-in. thick leveling course (sand) and an HDPE (30 to 60-mil thick) membrane liner. The core of the embankment would consist of geofoam with a total unit weight of approximately 2 pcf. A nominal 5-ft deep over-excavation would be required throughout the footprint of the embankment to compensate for the weight of the pavement section, distribution slab and leveling course materials. The vertical sides of the embankment would consist of either a MSE wall with geosynthetic reinforcing or tie rods, or a soldier pile wall with precast concrete lagging/ panels and tie rods. We anticipate that the vertical walls could be soil-supported (we will confirm once the consolidation test data is available). The attached Sketch No. 2 shows the vertical-sided embankment that we considered for this assessment. We believe the schematic is technically feasible to design and construct.

We have estimated an order-of-magnitude cost for the embankment alternative described above using unit pricing information from recent geofoam projects (University of Southern Maine Wishcamper Center, MaineDOT Payne Road Bridge Replacement, Maine Turnpike Kennebunk Rest Areas). A summary of the principal cost elements for this embankment alternative are summarized below.

Over-excavate & dispose of 5 ft of soil w/in embankment footprint	$100,000
Geofoam, HDPE liner, distribution slab, leveling sand (in-place)	$4,250,000
Vertical MSE or soldier pile/lagging walls (soil-supported)	$2,400,000
30-in. thick pavement section	$250,000
Total	$7,000,000

Please note that the estimate provided above does not include the cost of the abutments required for the MCRR Bridge (south abutment) or the Presumpscot River Bridge (north abutment). Furthermore, if the vertical walls need to be supported on pile foundations (to eliminate post-construction ground surface settlement) we anticipate that the total project cost would be increased by approximately $1,000,000.

One issue that could affect the feasibility of the embankment option is the design flood level in the Presumpscot River. We have contacted Kevin Ducharme (TY Lin) to obtain this information.

Attachments:
Sketch No. 1 – Plan showing approximate limits of geofoam embankment
Sketch No. 2 – Schematic cross-section showing geofoam embankment and reinforcement

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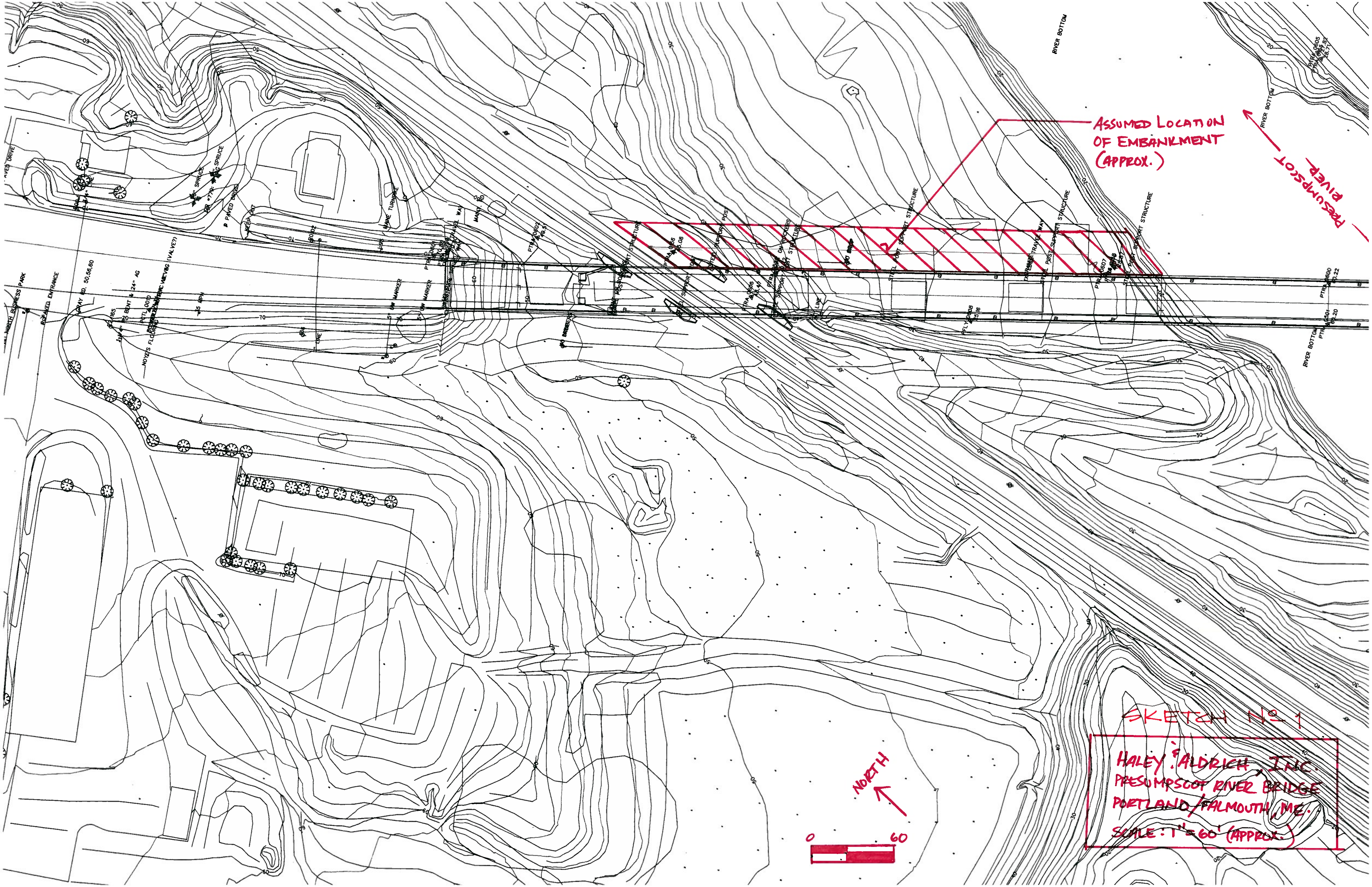
ASSUMED LOCATION
OF EMBANKMENT
(APPROX.)
PRESUMPSCOT
RIVER
RIVER BOTTOM
NORTH
0
60
SKETCH No. 1
HALEY & ALDRICH, INC.
PRESUMPSCOT RIVER BRIDGE
PORTLAND/FALMOUTH, ME.
SCALE: 1"=60' (APPROX.)

HALEY & ALDRICH

CALCULATIONS

Client	MaineDOT	File No.	35524-000
Project	Rt 26 Bridge	Sheet	1 of 1
Subject	EPS Embankment Schematic	Date	11-7-08
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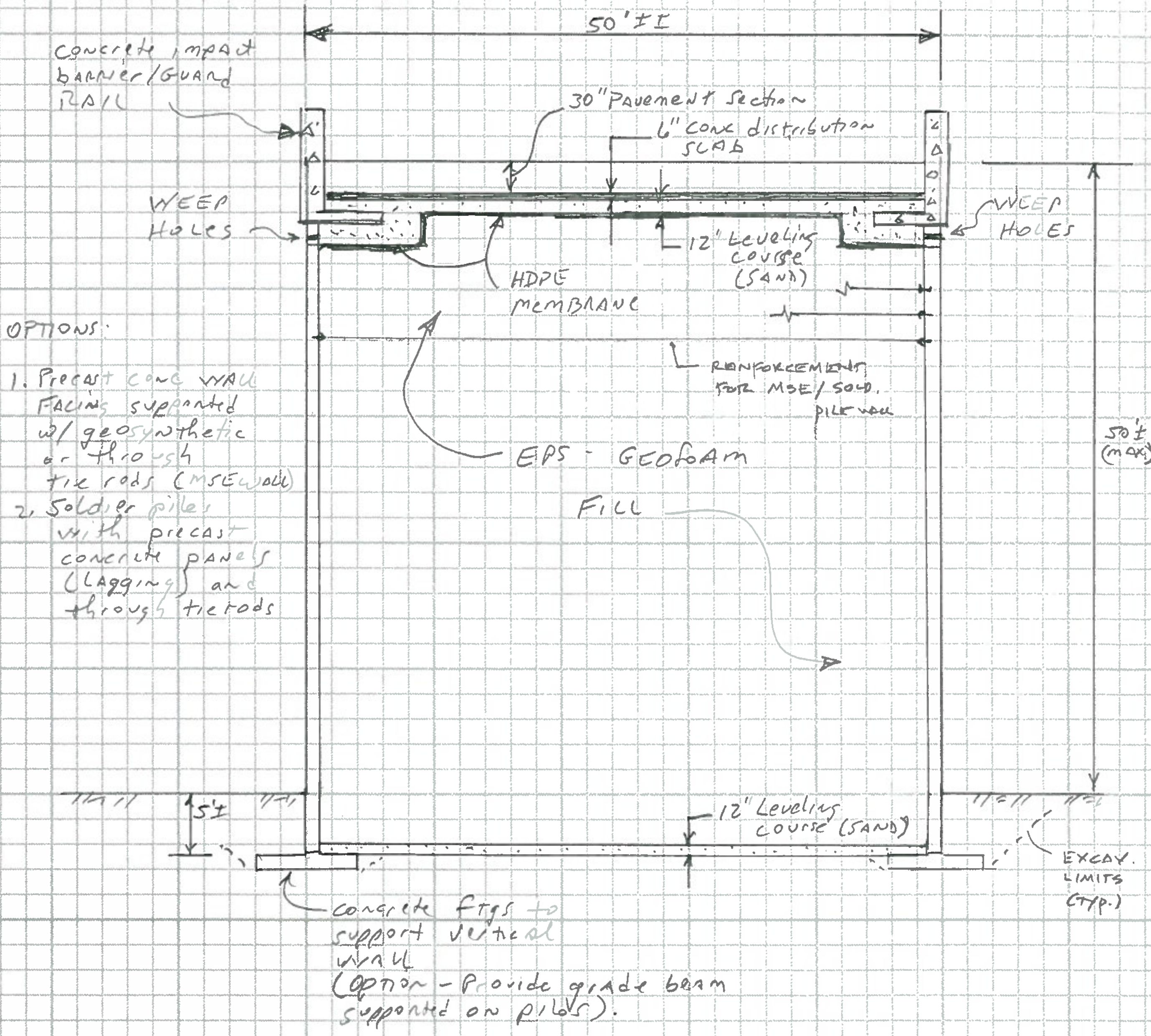
		Checked By	WAC



SCHEMATIC SECTION

NTS

SKETCH № 2

Form No. HA-080-Metric

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HALEY& ALDRICH

MEMORANDUM

14 November 2008
File No. 35524-000

TO:	T.Y. Lin International Rick Hebert, P.E.
C:	MaineDOT Leanne Timberlake, P.E., Laura Krusinski, P.E.
FROM:	Haley & Aldrich, Inc. Bryan C. Steinert, Wayne A. Chadbourne, P.E. BCS
SUBJECT:	Preliminary Cost Estimate Earthfill Embankment Alternative Proposed Route 100/26 Bridge Replacement Project MaineDOT PIN 15094.00 Falmouth, Maine

Per your email request on 11 November 2008, we have completed a preliminary-level cost estimate for the standard-weight earthfill embankment alternative located between the Presumpscot River and the Maine Central Railroad (MCRR) tracks. Please note that this memorandum provides a cost estimate only and does not address the technical feasibility of the earthfill embankment alternative. As discussed with you during our 13 November meeting, the technical feasibility of this alternative with regards to settlement, impacts to the existing bridge structure/MCRR tracks, and global embankment stability will be evaluated when the results of consolidation tests on marine clay samples are available.

For the purposes of our evaluation, we assumed that the proposed alignment would be located approximately 50 ft east of the current bridge structure. The total length of the embankment would be approximately 360 ft. The northern and southern limits of the embankment would consist of abutments for the river and MCRR bridge structures (see attached Sketch No. 1 for approximately embankment location). The width and maximum height of the embankment was assumed to be 50 ft.

We have assumed a 30-in. thick pavement structure would be underlain by approximately 50 ft of compacted common fill. The vertical sides of the embankment would consist of a MSE wall with geosynthetic reinforcing or tie rods. For the purposes of this cost estimate, we have assumed that the vertical walls would be soil-supported (we will confirm once the consolidation test data is available).

We have estimated an order-of-magnitude cost for this embankment alternative using weighted pay item averages taken from MaineDOT contracts awarded from 7 October 2005 to 6 October 2008. A summary of the principal cost elements for this embankment alternative are summarized below.

30-in. thick pavement section	$100,000
Vertical MSE walls (soil-supported)	$2,400,000
Common Borrow	$600,000
Total	$3,100,000

Please note that the estimate provided above does not include the cost of the abutments required for the MCRR Bridge (south abutment) or the Presumpscot River Bridge (north abutment). Furthermore, if the vertical walls need to be supported on pile foundations (to eliminate post-construction ground surface settlement) we anticipate that the total project cost would be increased by approximately $1,000,000.

We look forward to our continued association on this project. Please do not hesitate to contact us if you have any questions about the information provided in this memorandum.

Attachments:
Sketch No. 1 – Plan showing approximate limits of earthfill embankment
Sketch No. 2 - Schematic cross-section showing earthfill embankment

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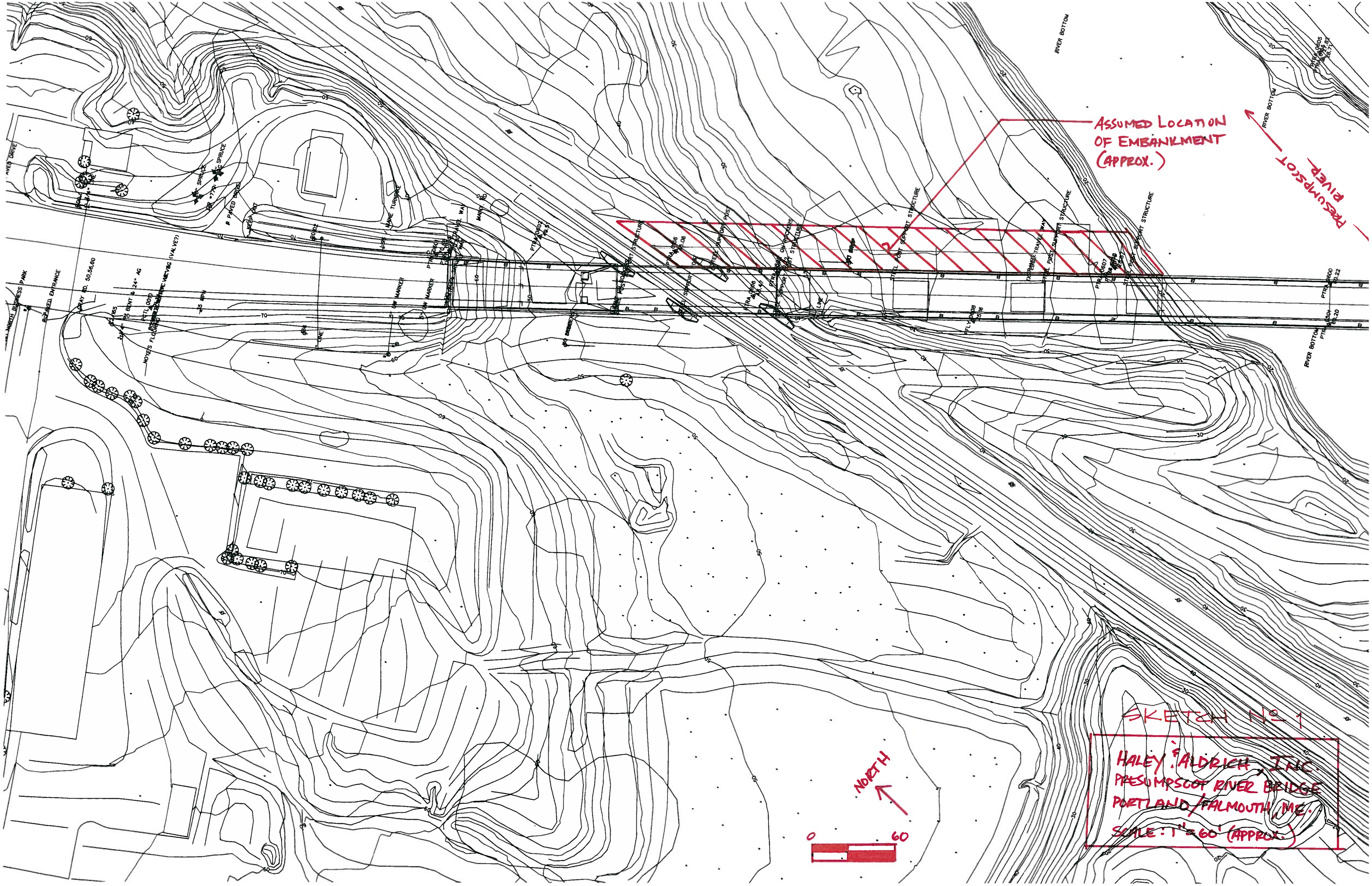

ASSUMED LOCATION OF EMBANKMENT (APPROX.)
PRESUMPSCOT RIVER
RIVER BOTTOM
STEEL POST SUPPORT STRUCTURE
NORTH
0
60
SKETCH No 1
HALEY & ALDRICH, INC.
PRESUMPSCOT RIVER BRIDGE
PORTLAND/FALMOUTH, ME.
SCALE: 1" = 60' (APPROX.)

HALEY& ALDRICH

Calculations

File No. 35524-000

Sheet 1 of 1

Client MAINE DOT

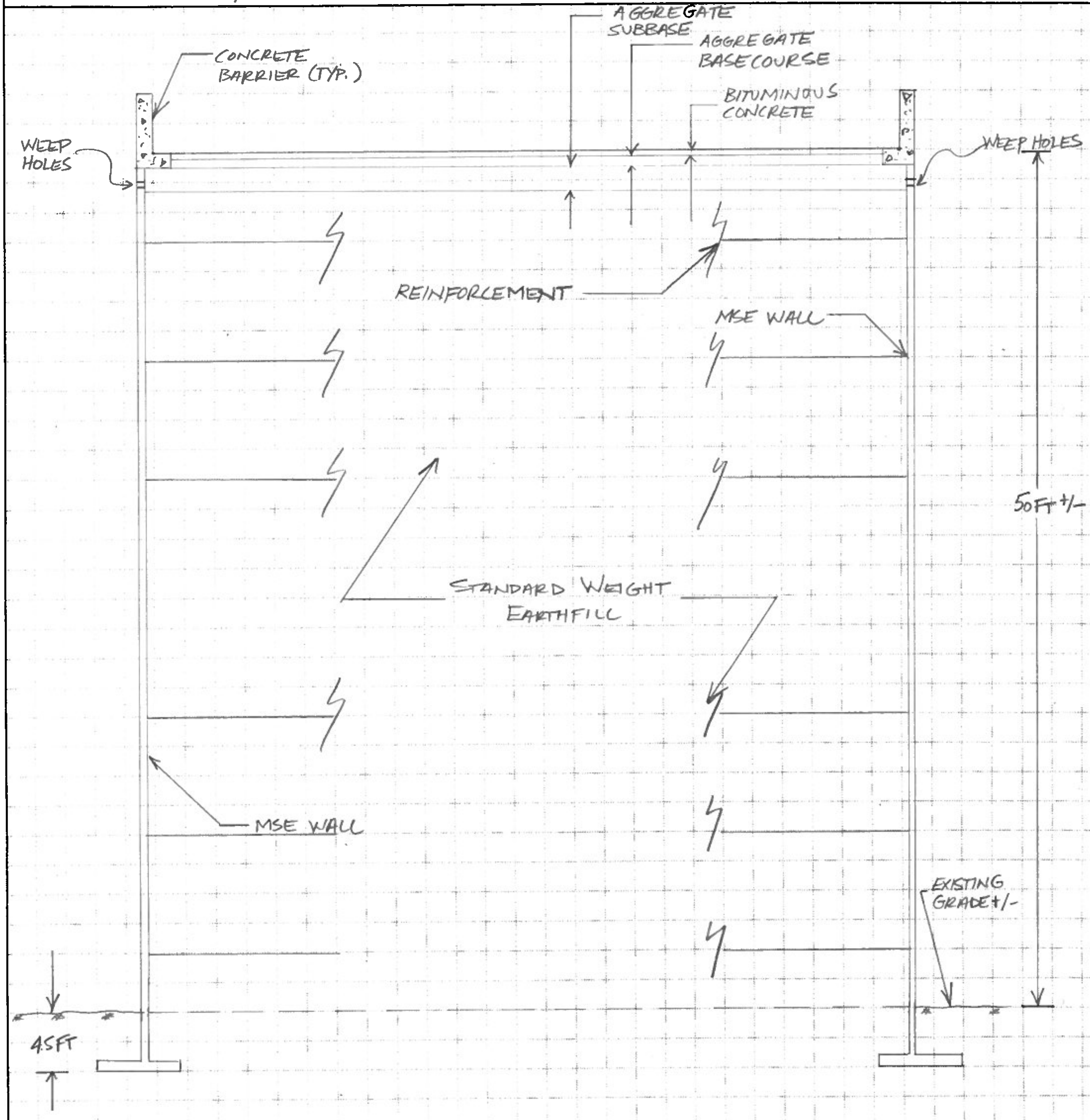
Date 11/13/2008

Project PRESUMPSCOT RIVER BRIDGE

Computed By BCS

Subject PRELIMINARY EVALUATION OF EARTHFILL EMBANKMENT

Checked By



SKETCH NO. 2

CAL02AFH.FRP 20 November 1997

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HALEY& ALDRICH

MEMORANDUM

25 November 2008
File No. 35524-000

TO: TY Lin International
Rick Hebert, P.E.

C: Maine Department of Transportation
Leanne Timberlake, P.E., Laura Krusinski, P.E.,

FROM: Haley & Aldrich, Inc.
Bryan C. Steinert, Wayne A. Chadbourne, P.E. BCS

SUBJECT: Earthfill Embankment Alternative Evaluation
Proposed Route 100/26 Bridge Replacement Project
PIN 15094.00
Falmouth, Maine

As discussed in our 29 October 2008 meeting, we have completed a preliminary-level technical and cost evaluation of various embankment alternatives to support a proposed roadway between the Presumpscot River and the Maine Central Railroad (MCRR) tracks. If feasible, the embankment would be constructed in lieu of a bridge structure between the river and MCRR tracks. For the purpose of our evaluation, we assumed that the embankment would be aligned parallel and offset approximately 20 ft (clear) east of the existing bridge structure. The northern and southern limits of the embankment would consist of abutments for the river and MCRR bridge structures (see attached Sketch No. 1 for approximate embankment alignment and location). The total length of the embankment was assumed to be 360 ft and the height of the embankment was assumed to range from approximately 50 ft near the river to approximately 25 ft adjacent to the MCRR tracks.

General Subsurface Conditions

Subsurface conditions used in the evaluations were generally as encountered in the recently-completed test borings (BB-FRR-102, BB-FPR-101, BB-FPR-102, BB-FPR-103 and BB-FPR-104; see attached Sketch No. 1). For the purpose of our evaluation, we assumed the following generalized subsurface conditions (in order of increasing depth below ground surface): 25 ft of loose to medium dense fine sand (alluvial deposit), 25 ft of stiff to soft silty clay and clayey silt (marine clay), 45 ft of medium dense to very dense medium to fine sand (marine sand), 40 ft of dense to very dense silty fine sand with trace gravel (glacial till), overlying bedrock. Recently completed laboratory consolidation testing of representative samples of the marine clay within the embankment footprint (adjacent to the river) indicate that the deposit is over-consolidated by approximately 2,000 to 2,500 pounds per square foot (psf). We believe that this amount of over-consolidation is likely attributed to post-glacial erosion of the Presumpscot River channel.

Embankment Alternatives

We considered the following four embankment alternatives:

1. An embankment constructed entirely of normal-weight earthfill with an approximate unit weight of 120 pounds per cubic foot (pcf).

2. An embankment constructed entirely of EPS geofoam (geofoam) lightweight fill with an approximate unit weight of 2 pcf.

3. A composite fill embankment constructed using a combination of geofoam and normal-weight earthfill. The section consisted of 18 ft of geofoam and 32 ft of normal-weight earthfill, and was designed to limit induced settlement of the existing, adjacent bridge piers to approximately ¼ in. (considered to be maximum permissible movement based on current condition of bridge superstructure). We also considered another composite section consisting of 30 ft of geofoam and 20 ft of normal-weight fill for the reasons described later in this memorandum.

4. A composite fill embankment constructed using a combination of expanded shale with approximate unit weight of 60 pcf and normal-weight earthfill. The section consisted of 35 ft of expanded shale and 15 ft of normal-weight earthfill, and was designed to limit induced settlement of the existing adjacent bridge piers to approximately ¼ in.

All of the embankment alternatives included vertical-sided, MSE walls to contain the embankment side slopes.

Engineering Evaluations

The following preliminary-level engineering evaluations were conducted for each of the embankment alternatives outlined above.

- Analyses to determine ground surface settlement due to consolidation of the marine clay and densification of the loose alluvial deposit were conducted. Settlements were estimated at the center and edge of the embankments as well as at the end of the embankment at the river bridge abutment location. We also estimated settlement at the adjacent existing bridge pier locations.

- Stability analyses were conducted at the location of the proposed river bridge abutment (parallel to bridge alignment) to assess both global stability and lateral squeeze. A "design" undrained shear strength of 800 psf was assigned to the marine clay for this evaluation based on the results of the in-situ vane shear tests conducted in test borings drilled within the embankment footprint (i.e., BB-FPR-101, BB-FPR-102 and BB-FPR-103).

- A liquefaction susceptibility assessment was conducted for the loose, saturated alluvial deposit. A summary of the liquefaction susceptibility assessment will be provided under separate cover.

- An evaluation of the potential for flotation of the embankment fills during the design flood event (i.e., a 500-year flood) was conducted.

- A preliminary assessment of the order-of-magnitude construction cost was conducted. The cost estimate for each embankment alternative was used to provide a relative comparison with the cost of constructing a bridge along this portion of the alignment. A per square foot (sf) cost of each embankment alternative was estimated by dividing the total embankment cost by 15,840 sf (44-ft wide and 360-ft long equivalent bridge structure). The estimates do not include the cost of the bridge abutments at both ends of the embankment. Also we assumed that the MSE walls used to form the vertical sides of the embankments were supported on soil-supported strip footings (pile-supported MSE wall foundations are estimated to cost an additional $1MM or approximately $63 per sf of equivalent bridge structure).

Refer to the attached Table I for a summary of the results of the various engineering evaluations that were conducted.

General Conclusions/Comments

We offer the following comments relative to each of the embankment alternatives that were considered (also refer to Table I):

- The normal-weight earthfill embankment is not considered to be technically feasible because the weight of the embankment would cause excessive ground surface settlement beneath the embankment and would cause the existing, pile-supported bridge piers to settle up to 0.75 in. Also this embankment would not be stable against deep-seated rotational failure of the clay under both static and seismic loading conditions. Some of these issues could be mitigated using staged construction, surcharging and ground improvement techniques (e.g., prefabricated vertical wick drains) but it would likely take several years to accomplish the mitigation measures, and add significant cost to the project.

- The lightweight fill embankment alternative constructed using geofoam is considered to be technically feasible because the ground surface settlement and embankment stability issues are acceptable. However, there would be a potential flotation issue for the geofoam associated with 500-year flood levels at the site. The most significant issue is that the cost of constructing the geofoam embankment is greater than the cost of an equivalent bridge structure (based on information provided by TY Lin).

- Composite embankment alternatives constructed using geofoam/normal-weight earthfill or expanded shale/normal-weight earthfill could be designed to mitigate the ground surface settlement beneath and adjacent to the embankment. However, our stability analyses indicate that these embankments would not be stable against deep-seated rotational failure of the clay under both static and earthquake loading conditions. In order to mitigate the global stability issue, additional volumes of geofoam or expanded shale would be needed, but its use would drive up the total cost of the embankment, and would make these alternatives, in our opinion, cost prohibitive.

  In order to achieve an acceptable factor of safety, the geofoam/earthfill composite embankment would need to be constructed of 30 ft of geofoam and 20 ft of normal-weight earthfill, with an estimated cost of $5.4 MM to $5.6 MM ($340 to $355 per sf; see Table I). For the expanded shale/earthfill composite embankment, more than 50 ft (greater than the maximum height of the embankment) of expanded shale would be needed to achieve an acceptable factor of safety (i.e., over-excavation and replacement of in-situ soils with expanded shale would be needed). It is our opinion that this would not be feasible from a cost standpoint.

- In our opinion, the composite geofoam/earthfill embankment alternative (with 30 ft of geofoam and 20 ft of normal-weight earthfill) is the only technically feasible embankment option.

- TY Lin should confirm that their construction estimate for the bridge structure (including substructure costs) between the river and the MCRR tracks is less than the cost estimate for the composite geofoam/earthfill embankment alternative (with 30 ft of geofoam).

Closure

We trust these comments and recommendations are suitable for your present needs. Please do not hesitate to contact us if you have any questions about this memorandum or the engineering evaluations.

Attachments:
Table I – Embankment Alternative Evaluation
Sketch No. 1 – Plan showing approximate limits of embankment and approximate test boring locations

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TABLE I
EMBANKMENT ALTERNATIVE EVALUATION
PROPOSED ROUTE 100/26 BRIDGE REPLACEMENT PROJECT
FALMOUTH, MAINE

MAINEDOT PIN NO.: 15094.000
HALEY & ALDRICH FILE NO.: 35524-000

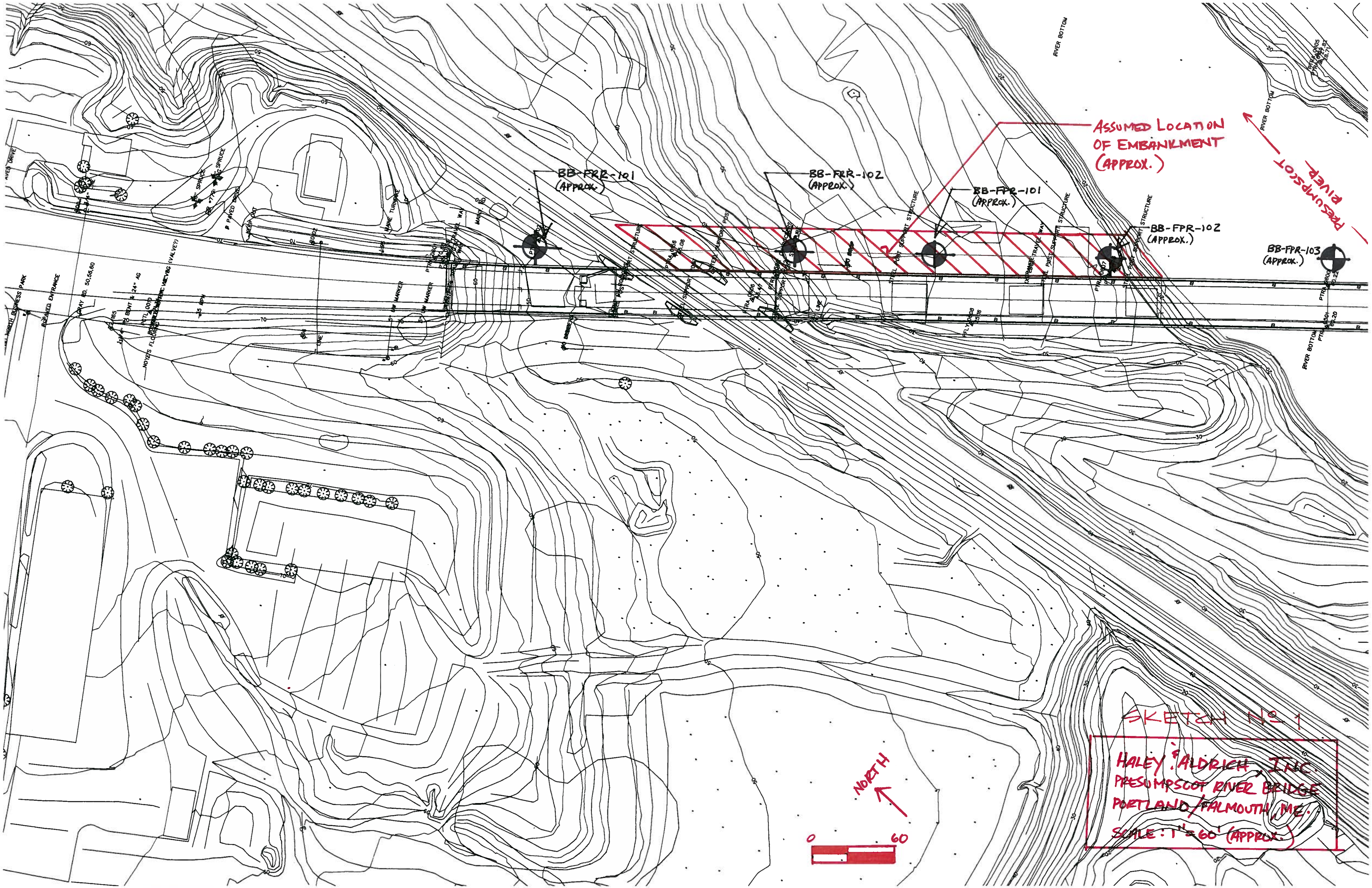
Technical/Cost Consideration	Embankment Alternative[1,4]				
	Normal-Weight Earthfill	Geofoam Fill	Composite Fill[5] (Geofoam/ Earthfill)	Composite Fill[7] (Geofoam/ Earthfill)	Composite Fill[6] (Expanded Shale/Earthfill)
Embankment Settlement					
Center	12 to 15 in.	1 to 2 in.	5 to 7 in.	3 to 5 in.	5 to 7 in.
Edge	5 to 7 in.	1 in.	1 to 3 in.	1 to 2 in.	1 to 3 in.
End-Center	2 to 3 in.	1 in.	1 to 3 in.	1 to 2 in.	1 to 3 in.
End-Edge	1 to 2 in.	1/2 in.	1 to 2 in.	1 to 1-1/2 in.	1 to 2 in.
Adjacent Bridge					
Pier Settlement	1/2 to 3/4 in.	Negligible	~1/4 in.	<1/4 in.	~1/4 in.
Stability					
Global	FS = 0.7	FS > 6.0	FS = 0.8	FS = 1.3	FS = 0.8
Lateral Squeeze	FS < 1.0	Negligible	Marginal	Marginal	Marginal
Flotation	No	Yes	No	No	No
Cost[2, 3]					
Total	\$3 - \$3.5 MM	\$6.5 - \$ 7 MM	\$4.5 - \$4.7 MM	\$5.4 - \$5.6 MM	\$3.9 - \$4.2 MM
Unit Cost (per sf of deck)	\$190 - \$220	\$410 - \$440	\$285 - \$300	\$340 - \$355	\$245 - \$265

Notes:
[1] - Embankment -260-ft long at 50 ft high and 100-ft long at 37.5 ft high between MCRR tracks and Presumpscot River.
[2] - Cost does not include bridge abutments at either end of the embankment, and assumes MSE walls are soil-supported. If pile support is required add ~\$1 MM to total cost or ~\$63/sf.
[3] - Square foot of equivalent bridge deck based on 44 ft wide and 360 ft long (15,840 sf) embankment.
[4] - Unit Weights: earthfill = 120 pcf, geofoam = 2 pcf, expanded shale = 60 pcf.
[5] - Assumes 18 ft geofoam and 32 ft normal-weight earth fill.
[6] - Assumes 35 ft expanded shale and 15 ft normal-weight earth fill.
[7] - Assumes 30 ft geofoam and 20 ft normal-weight earth fill.

Legend:

- (white) Result of technical/cost evaluation is considered to be acceptable
- (yellow) Result of technical/cost evaluation is considered to be marginal
- (red) Result of technical/cost evaluation is considered to be unacceptable

HALEY & ALDRICH



ASSUMED LOCATION OF EMBANKMENT (APPROX.)
BB-FRR-101 (APPROX.)
BB-FRR-102 (APPROX.)
BB-FPR-101 (APPROX.)
BB-FPR-102 (APPROX.)
BB-FPR-103 (APPROX.)
PRESUMPSCOT RIVER
RIVER BOTTOM
STEEL POST SUPPORT STRUCTURE
NORTH
0
60
SKETCH No. 1
HALEY & ALDRICH, INC.
PRESUMPSCOT RIVER BRIDGE
PORTLAND/FALMOUTH, ME.
SCALE: 1"=60' (APPROX.)

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HALEY& ALDRICH

MEMORANDUM

1 December 2008
File No. 35524-000

TO:	T.Y. Lin International Rick Hebert, P.E.
C:	MaineDOT Leanne Timberlake, P.E., Laura Krusinski, P.E.
FROM:	Haley & Aldrich, Inc. Bryan C. Steinert, Wayne A. Chadbourne, P.E. BCS
SUBJECT:	Liquefaction Susceptibility Evaluation Proposed Route 100/26 Bridge Replacement Project PIN 15094.00 Falmouth, Maine

As discussed during our 13 November 2008 meeting, we have evaluated the liquefaction susceptibility of the granular soil deposits at the subject site. Our analyses were based on the subsurface conditions encountered in the preliminary phase test borings drilled along the east side of the existing Route 100/26 bridge structure (BB-FRR and BB-FPR series; see attached site plan for boring locations). The liquefaction evaluations discussed herein have been conducted in general accordance with the requirements of AASHTO LRFD Bridge Design Specifications (LRFD Specifications), Appendix A10, "Seismic Analysis and Design of Foundations."

Liquefaction Analyses

The liquefaction susceptibility of the granular soils at the site was determined by comparing the equivalent uniform cyclic stress ratio (CSR) imposed by the design earthquake to the cyclic resistance ratio (CRR) of the in-situ soils at each sample location. Liquefaction of the in-situ granular soils would occur when the CRR is less than or equal to the CSR. In the instance where the CRR equals the CSR the factor of safety against liquefaction (FS_{liq}) is equal to 1.0. In Appendix A10 of the LRFD Specifications it is suggested that a FS_{liq} value of 1.5 or greater is desirable to establish "a reasonable margin of safety against liquefaction in the case of important bridge sites."

CRR is a function of clean sand-corrected blow counts, $N1_{60\text{-}CS}$, following the simplified methodology originally developed by Seed et al. (1985), and most recently updated by Idriss and Boulanger (2008). The CRR vs. $N1_{60\text{-}CS}$ correlation is based on an earthquake magnitude (M) equal to 7.5 and is applicable for "clean" sands (i.e., no fine contents).

Therefore, correction factors developed by Idriss and Boulanger were used to account for the design earthquake magnitude for this site (assumed M = 6.5; typically the maximum considered in the northeast) and the actual fines content of the granular soils at the site (i.e., percent passing the No. 200 sieve).

Values of peak ground acceleration were developed in accordance with the site classification procedure presented in Chapter 20 of the American Society of Civil Engineers (ASCE) Standard 7-05 "Minimum Design Loads for Buildings and Other Structures." This procedure is based on the 2003 version of the United States Geological Survey (USGS) probabilistic database.

Based on the corrected SPT blow count (granular soils) and undrained shear strength (cohesive soils) data obtained from our preliminary phase test borings, the southern portion of the alignment (BB-FPR-103, BB-FPR-104) should be classified as Site Class "D" and the northern portion of the alignment (BB-FRR series and BB-FPR-101, BB-FPR-102) should be classified as Site Class "E" (nomenclature per ASCE Standard 7-05). The approximate demarcation between the two areas is shown on the attached site plan.

Results of Analyses and Conclusions

Corrected SPT blow count data, calculated values of CRR and the resulting values of FS_{liq} for each sample in the near-surface alluvial soils have been graphically summarized on the attached figures. Figure 1 shows the data and results for the borings located in the southern portion of the alignment (Site Class "D"), and Figure 2 shows the data and results for the borings located in the northern portion of the alignment (Site Class "E"). The corrected SPT blow count data in the marine sand deposit underlying the alluvial soils are not shown on the plots but were analyzed and were determined to have acceptable factor of safety values (i.e., not susceptible to liquefaction).

The results of our liquefaction analyses show that corrected SPT blow counts measured within the near-surface alluvial deposit in the southern portion of the alignment (in borings BB-FPR-103, BB-FPR-104) resulted in acceptable FS_{liq} values generally greater than 1.5. One soil sample in BB-FPR-104 had a "marginal" FS_{liq} value slightly between 1.0 and 1.5 (i.e., 1.25). It is our opinion that the soils in the southern portion of the alignment are not susceptible to liquefaction-inducted instability or settlement.

The corrected SPT blow counts measured within the 20 to 25-ft thick, near-surface alluvial deposit in the northern portion of the alignment (encountered in test borings BB-FPR-101 and BB-FPR-102; note that alluvial soils were not encountered in the BB-FRR series of borings) result in FS_{liq} values generally less than 1.5, with many below 1.0. Because of this, it is our opinion that the following issues should be properly evaluated as part of the final foundation design:

- Downdrag loading on pile foundations located near test borings BB-FPR-101 and BB-FPR-102

- Reduction on lateral pile capacity during the design earthquake event as it pertains to resistance of lateral bridge design loads near test borings BB-FPR-101 and BB-FPR-102

- Lateral spreading of the near-surface alluvial deposit into the Presumpscot River (on the north bank of the river) as it pertains to increased lateral forces on the piles (from the liquefied soil mass) and potential scour adjacent to foundation elements near test borings BB-FPR-101 and BB-FPR-102

We recommend that additional design phase explorations be considered once the final plan alignment of the replacement bridge is established and the structure locations are determined. The purpose of these additional borings would be to determine the plan and vertical extent of the liquefiable soils along the replacement bridge alignment..

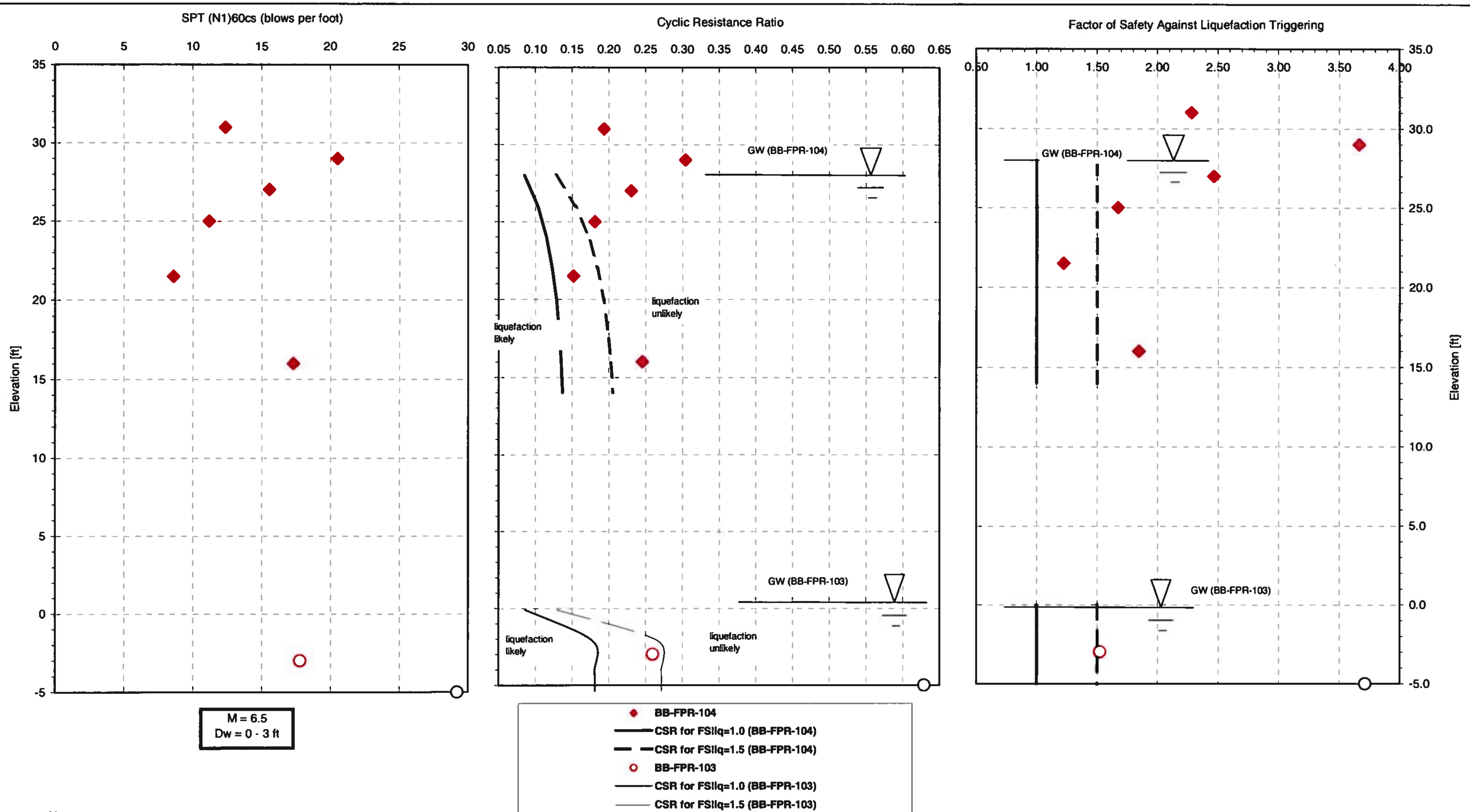
Closure

We trust these comments and recommendations are suitable for your present needs as you prepare the draft PDR. Please do not hesitate to contact us if you have any questions about this memorandum or the engineering evaluations summarized herein.

Attachments:
Sketch No. 1 – Plan showing approximate test boring locations
Figure 1 – Liquefaction Triggering Assessment – Southern Portion of Alignment
Figure 2 – Liquefaction Triggering Assessment – Northern Portion of Alignment

G:\PROJECTS\35524 - Presumpscot River Bridge\2008-1201-bcs&wac-liquefaction-f.doc


HALEY & ALDRICH, INC.
PROPOSED REPLACEMENT BRIDGE OVER PRESUMPSCOT RIVER AND MCRR
MAINE DEPARTMENT OF TRANSPORTATION PIN NO. 15094.00
ROUTES 100 / 26 - FALMOUTH, MAINE
NORTH
100
NORTH ABUTMENT
BB-FRR-101
LEGEND:
BB-FRR-101
DESIGNATION AND APPROXIMATE LOCATION OF PROPOSED TEST BORINGS IN THE VICINITY OF MAINE CENTRAL RAILROAD
BB-FPR-101
DESIGNATION AND APPROXIMATE LOCATION OF PROPOSED TEST BORINGS FOR THE PRESUMPSCOT RIVER CROSSING AND APPROACH EMBANKMENT
BB-FRR-102
Gray Rd
PRESUMPSCOT RIVER
BB-FPR-101
SITE CLASS E
APPROXIMATE DEMARCATION BETWEEN SITE CLASS "D" AND SITE CLASS "E"
BB-FPR-102
SITE CLASS D
MAINE CENTRAL RAILROAD
BB-FPR-103
SITE CLASS E
BB-FPR-104
SOUTH ABUTMENT
APPROXIMATE DEMARCATION BETWEEN SITE CLASS "D" AND SITE CLASS "E"
SITE CLASS D
2008 Maine GeoLibrary
© 2008 Tele Atlas
©2008 Google
43°43'45.80" N 70°17'44.00" W
2003
Eye alt 418 m



Notes:

1 - $N1_{60(CS)}$ = SPT N-value corrected for overburden, drilling and sampling methods, and fines content for use in evaluating liquefaction resistance.
2 - Cyclic Stress Ratio (CSR) = $CSR_{M=M,\sigma'vc=1\ atm}$, corresponds to cyclic shear stress induced by design earthquake (based on design earthquake parameters).
3 - Cyclic Resistance Ratio (CRR) = $CRR_{M=M,\sigma'vc=1\ atm}$ (corrected for magnitude and overburden), corresponds to resistance of soil layer to cyclic shear stress (based on Standard Penetration Test results and fines content).
4 - Factor of safety against liquefaction triggering = CRR / CSR.
5 - Dw = Depth from ground surface to design water table.
6 - Considered earthquake magnitude 6.5 with PGA = 0.132 g (assumes IBC 2006, Site Class D, Design Spectrum).
7 - Liquefaction analyses used procedures by Idriss and Boulanger (2008).
8 - All borings were drilled as cased borings with rope/cathead and safety hammer.
9 - Some data points from borings are not shown on the charts because they are not within the range of values shown; these data points are not relevant to liquefaction evaluation.

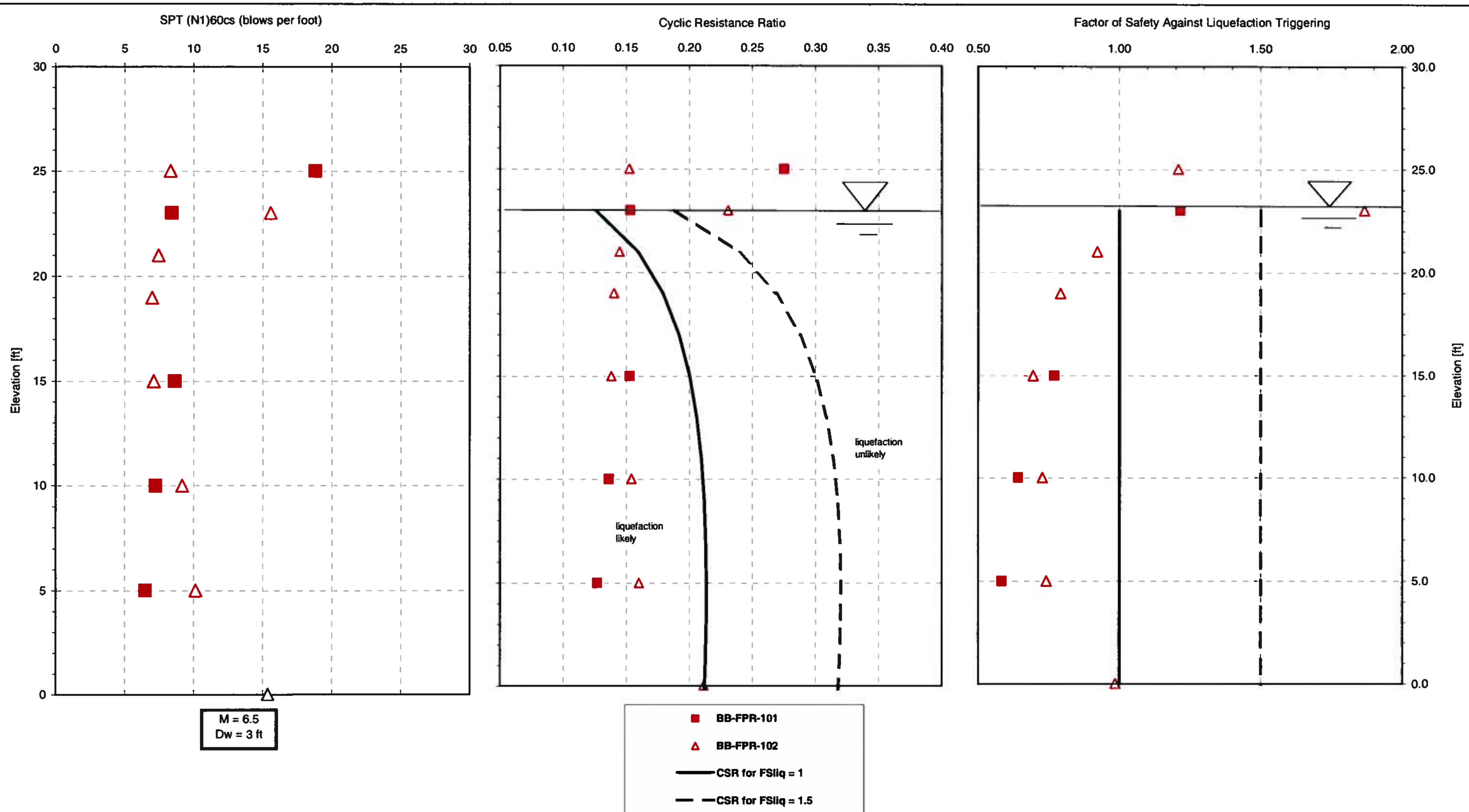
HALEY & ALDRICH

PROPOSED ROUTE 100/26 BRIDGE REPLACEMENT
MAINEDOT PIN NO.: 15094.00
FALMOUTH, MAINE

LIQUEFACTION TRIGGERING ASSESSMENT, SOUTHERN PORTION OF ALIGNMENT (IBC 2006, Site Class D)

NOT TO SCALE
DECEMBER 2008

FIGURE 1



Notes:

1 - $N1_{60(CS)}$ = SPT N-value corrected for overburden, drilling and sampling methods, and fines content for use in evaluating liquefaction resistance.
2 - Cyclic Stress Ratio (CSR) = $CSR_{M=M,\sigma'vc=1\,atm}$, corresponds to cyclic shear stress induced by design earthquake (based on design earthquake parameters).
3 - Cyclic Resistance Ratio (CRR) = $CRR_{M=M,\sigma'vc=1\,atm}$ (corrected for magnitude and overburden), corresponds to resistance of soil layer to cyclic shear stress (based on Standard Penetration Test results and fines content).
4 - Factor of safety against liquefaction triggering = CRR / CSR.
5 - Dw = Depth from ground surface to design water table.
6 - Considered earthquake magnitude 6.5 with PGA = 0.194 g (assumes IBC 2006, Site Class E, Design Spectrum).
7 - Liquefaction analyses used procedures by Idriss and Boulanger (2008).
8 - All borings were drilled as cased borings with rope/cathead and safety hammer.
9 - Some data points from borings are not shown on the charts because they are not within the range of values shown; these data points are not relevant to liquefaction evaluation.

HALEY & ALDRICH

PROPOSED ROUTE 100/26 BRIDGE REPLACEMENT
MAINEDOT PIN NO.: 15094.00
FALMOUTH, MAINE

LIQUEFACTION TRIGGERING ASSESSMENT, NORTHERN PORTION OF ALIGNMENT (IBC 2006, Site Class E)

NOT TO SCALE
DECEMBER 2008

FIGURE 2

G:\PROJECTS\35524 - Presumpscot River Bridge\2008-1114-HAI-Liquefaction IBC2006_FINAL.xls]BB-FPR-101,102 Chart

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HaleyAldrich.com

HALEY& ALDRICH

MEMORANDUM

26 December 2008
File No. 35524-000

TO: TY Lin International
Rick Hebert, P.E.

C: Maine Department of Transportation
Leanne Timberlake, P.E., Laura Krusinski, P.E.

FROM: Haley & Aldrich, Inc.
Bryan C. Steinert, Wayne A. Chadbourne, P.E., James W. Weaver, P.E.
BCS

SUBJECT: North Bridge Abutment and Approach Embankment Evaluation
Proposed Route 100/26 Bridge Replacement Project
PIN 15094.00
Falmouth, Maine

As discussed during our 22 December 2008 meeting, we have completed preliminary-level technical and cost evaluations for various bridge abutment and approach embankment alternatives for the north end of the project (north bridge abutment and approach embankment). For the purpose of our evaluations, we have assumed that the abutment and approach embankment will be located on an alignment parallel and to the east of the current bridge as shown on the attached Sketch No. 1 prepared by TY Lin.

General Subsurface Conditions

The subsurface conditions used in the evaluations were developed based on the conditions encountered in recently completed test borings BB-FRR-101 and BB-FRR-102, drilled in the vicinity of the existing north abutment (see Sketch No. 1). For purposes of our evaluation, we assumed the following generalized subsurface conditions (in order of increasing depth below ground surface): a variable thickness of medium dense sand (alluvial deposit and existing embankment fill), 70 ft of medium stiff marine clay (undrained shear strength ranging from about 500 to 1,000 psf), 60 ft of medium dense to very dense medium to fine sand (marine sand), 30 ft of dense to very dense silty fine sand with gravel (glacial till), overlying bedrock.

It is likely that the thickness of the marine units (clay and sand) will decrease, and the upper surfaces of the glacial till and bedrock will rise to the north of boring BB-FRR-101. Therefore, the subsurface conditions assumed for these evaluations are considered to be conservative. In addition, we do not have any compressibility data for the marine clay encountered at the north end of the site. However, we believe the deposit will be slightly overconsolidated relative to the existing ground surface elevation. Additional test borings will be required during final design in order to define the soil profile north of test boring BB-FRR-101 and to obtain undisturbed samples of marine clay for laboratory consolidation testing.

Engineering Evaluations

Based on our previous evaluations it appears that global stability of the approach embankments will control the location and design of the north approach embankment and bridge abutment.

Global stability evaluations were performed modeling the existing ground surface level along the centerline of the proposed approach embankment utilizing the subsurface soil profile described above. The calculated factor of safety for the existing site condition is on the order of 1.2. It is our opinion that this calculated factor of safety is conservative due to the assumptions made relative to marine clay thickness and strength properties. However, it does serve as a baseline to assess proposed abutment and approach embankment alternatives.

Subsequent global stability evaluations were conducted modeling three proposed bridge abutment location alternatives as shown below (see attached Sketch No. 1).

- Alternative No. 1 = Station 118+65
- Alternative No. 2 = Station 119+05
- Alternative No. 3 = Station 119+45

Alternative No. 1 was evaluated using normal-weight earthfill to construct the approach embankment resulting in a calculated factory of safety approximately equal to 0.8. Modeling Alternative No. 1 using lightweight fill (EPS, geofoam) behind the abutment and within the approach embankment resulted in a calculated factor of safety equal to the existing site condition (FS = 1.2). We believe this result is reasonable since we can create a "no net stress increase" condition by constructing the approach embankment primarily of geofoam (2 pounds per cubic foot) and by over-excavating and replacing approximately 5 ft of existing material with geofoam to compensate for the weight of the roadway pavement section.

Similar evaluations were performed for Alternative Nos. 2 and 3. Calculated factors of safety for Alternatives Nos. 2 and 3 using normal-weight earthfill behind the abutments and to construct the approach embankments are approximately 0.9 and 1.0, respectively. Therefore, it is our opinion that that using normal-weight earthfill exclusively is not practicable. Calculated factors of safety for Alternatives Nos. 2 and 3 using geofoam in lieu of normal-weight earthfill are approximately 1.2; equal to both Alternative No. 1 and the existing site condition.

Based on the results of the preliminary-level global stability evaluations summarized herein, it is our opinion that that lightweight fill (geofoam) will be needed behind the proposed bridge abutment and to construct the proposed bridge approach embankment. We have conservatively estimated that geofoam would need to extend approximately 220 ft north along the proposed alignment from Alternative No. 1 (Station 118 + 65) to

Station 120 + 85. It is considered likely that the quantity of geofoam actually required will be less than that described herein.

All three alternatives are considered to be technically feasible. Selecting the most practicable alternative will likely be based on cost implications to the project. Alternative No. 1 will require the greatest quantity of geofoam and the least amount of bridge superstructure. Conversely, Alternative No. 3 will minimize the quantity of geofoam but will maximize the amount of bridge superstructure.

Cost Implications

A preliminary-level estimate of the cost implications for the three alternatives described above was completed using the following assumptions and is summarized below:

- Geofoam will be used as lightweight fill at a cost of $130 per cubic yard (cy).
- Earthfill will be provided for pavement section and protection of the geofoam at $25 per cy.
- The side walls of the approach embankment and end wall (north abutment) will consist of vertical Mechanically Stabilized Earth (MSE) walls at a cost of $70 per square foot (sf).
- The approach embankments will be 50 ft wide and the bridge deck will be 44 ft wide.
- Volume calculations for earthfill and geofoam, and square footage estimates for the MSE walls are based on the centerline profile along the proposed road/bridge alignment as shown on Sketch 1 (attached).

North Abutment Location Alternative	Material Quantity				Cost ($)
	Geofoam (cy)	Earthfill (cy)	MSE Wall (sf)	Additional Bridge Deck (sf)	
1	4,500	2,700	7,200	0	$1,156,500
2	2,700	1,950	4,050	1,760	$683,250
3	1,525	1,450	2,450	3,520	$406,000

The embankment cost savings realized by moving the abutment north from Alternative No. 1 are:

- Alternative No. 1 (Station 118 + 65): no savings
- Alternative No. 2 (Station 119 + 05): $1,156,500 - $683,250 = $473,250
- Alternative No. 3 (Station 119 + 45): $1,156,500 - $406,000 = $750,500

However, the embankment cost savings will be offset by the increased cost of the bridge superstructure. Therefore the equivalent bridge superstructure cost to break even is estimated by dividing the embankment cost savings by the increased in bridge superstructure:

- Alternative No. 1: NA
- Alternative No. 2: $473,250 / 1,760 sf = $269/ sf
- Alternative No. 3: $750,500 / 3,520 sf = $213/ sf

As a result, if the actual cost of the bridge superstructure (reported by TY Lin as approximately $300/ sf) is greater than the embankment savings, it would be more cost effective to leave the abutment at its currently proposed location (Alternative No. 1, Station 118+65).

The cost of a lightweight fill (geofoam) approach embankment will be more costly than an approach embankment using normal-weight earthfill. Assuming a similar embankment geometry (50 ft wide embankment with vertical MSE side and end walls), the premium cost of a geofoam embankment will be the differential cost of the geofoam ($130/cy) and the earthfill ($25/cy) multiplied by the volume of geofoam, or approximately $500,000 to $600,000.

Closure

We trust these comments and recommendations are suitable for your present needs. Please do not hesitate to contact us if you have any questions about this memorandum or engineering evaluations.

We are currently completing a similar evaluation for the south bridge abutment and approach. A memorandum summarizing the results of that evaluation will be provided at a later date.

Attachment:
Sketch 1 - North Bridge Abutment and Approach Embankment Alternatives

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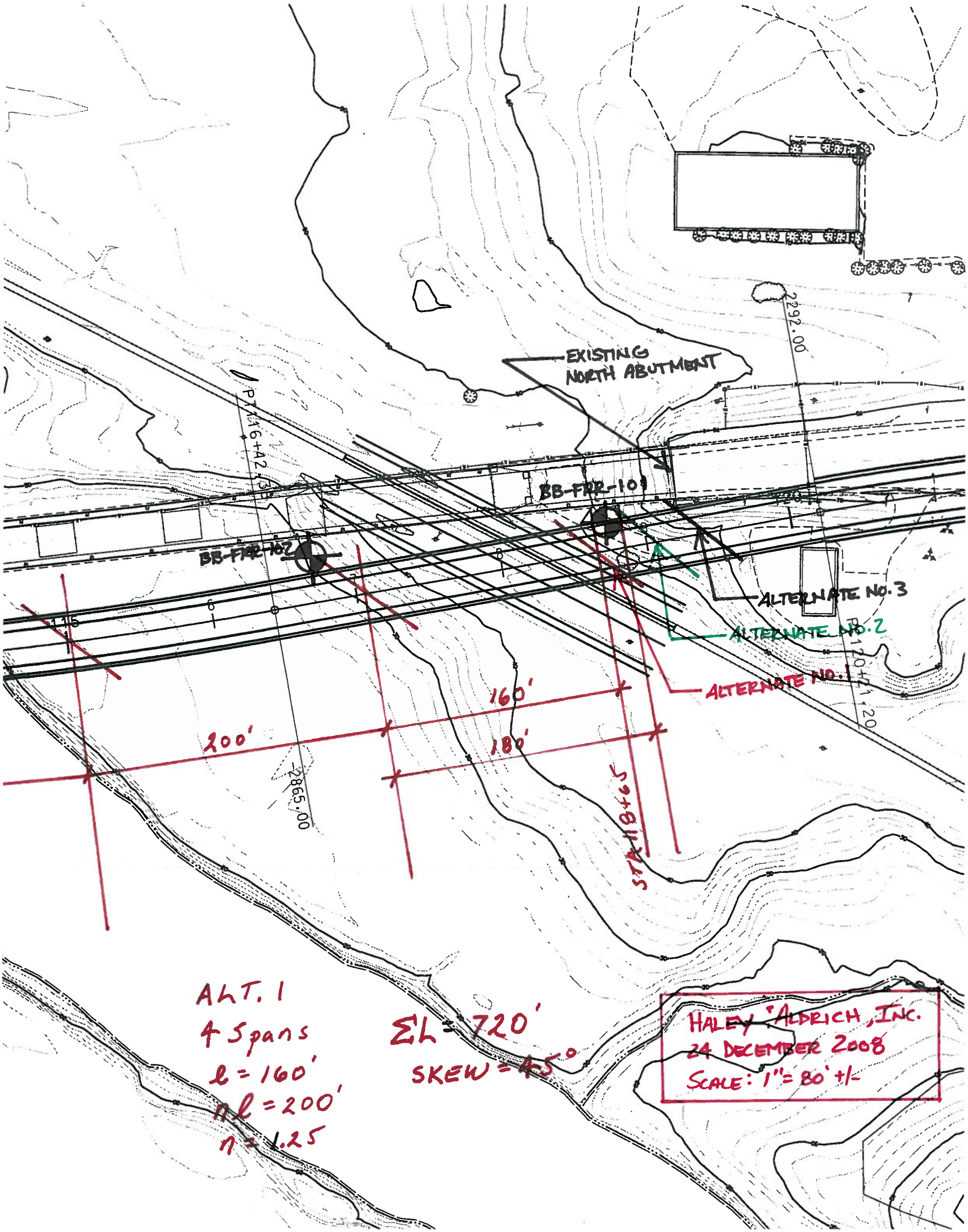

EXISTING
NORTH ABUTMENT
2292.00
BB-FRR-103
BB-FRR-102
ALTERNATE No. 3
ALTERNATE No. 2
ALTERNATE No. 1
160'
200'
180'
2865.00
STA 118+65
ALT. 1
4 Spans
l = 160'
n l = 200'
n = 1.25
EL = 720'
SKEW = 45°
HALEY & ALDRICH, INC.
24 DECEMBER 2008
SCALE: 1" = 80' +/-

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HALEY & ALDRICH

MEMORANDUM

14 January 2009
File No. 35524-000

TO: Maine Department of Transportation
Laura Krusinski, P.E., Leanne Timberlake, P.E.

C: TY Lin International
Rick Hebert, P.E.

FROM: Haley & Aldrich, Inc.
Bryan C. Steinert, Wayne A. Chadbourne, P.E.

SUBJECT: Proposed Design Phase Subsurface Exploration Program
Proposed Route 100/26 Bridge Replacement Project
PIN 15094.00
Falmouth, Maine

Based on the geotechnical evaluations conducted to date, we have identified the need to conduct additional subsurface explorations to support final bridge design. Therefore, in accordance with Table 2-10 of the Bridge Design Guide, we have developed a design phase subsurface exploration program for inclusion in TY Lin's preliminary bridge data report (PDR). We have summarized the proposed program herein.

A preliminary level field investigation was conducted in October and November 2008 in order to identify general subsurface conditions adjacent to the existing bridge alignment since a preferred alignment had not been determined. Subsequent engineering evaluations were conducted to assess how the subsurface conditions will affect the overall design and construction of the replacement bridge. The preliminary-level evaluations were summarized in several memoranda that will be included in the PDR.

Engineering issues related to approach embankment/abutment stability, embankment settlement and liquefaction potential of the alluvial soils were identified during the preliminary evaluations. As a result, it has been determined that additional explorations will be required for final design in order to refine and update the preliminary analyses. In addition, based on conversations with TY Lin it is our understanding that a preferred alignment for the proposed replacement bridge and approach roadway has been developed and will be included in the PDR. The proposed alignment consists of two abutments and three piers and is located up to as much as 70 ft east (approximate) of the existing bridge structure (see attached Figures 1 through 3). Therefore, we are proposing to drill borings at specific substructure locations.

In total, we are proposing to drill twelve additional test borings in order to provide subsurface information along the preferred alignment and at specific substructure locations as discussed below. A summary of the proposed test borings is provided in Table I (attached) and the approximate test boring locations are shown on Figure 1 (attached).

- One test boring at each of the following substructure locations: South Abutment, Pier 2, Pier 3 and the North Abutment). We do not currently envision drilling a test boring in the Presumpscot River (Pier 1). Based on the results of the preliminary phase investigation, we anticipate that these test borings would be drilled to depths ranging from approximately 140 to 180 ft below ground surface (BGS) and would be terminated a minimum of 10 ft into bedrock (per AASHTO LRFD recommendations). Undisturbed samples of marine clay would be obtained and in-situ vane shear testing would be performed within the marine clay deposit in the test boring drilled at the North Abutment.

- Two test borings along the preferred north approach embankment alignment. We anticipate that the test borings would penetrate through the marine clay deposit to an approximate depth of 90 ft BGS. Currently, there is no subsurface information available north of the proposed abutment.

- Three test borings along the preferred south approach embankment alignment. We anticipate that the test borings would be drilled a minimum of 10 ft into naturally deposited glacial till soils.

- Three test borings between the proposed south abutment and the Presumpscot River in order to provide information on the nature and extent of alluvial soils to provide additional information needed to more accurately assess abutment/embankment stability and riverbank stabilization measures.

We do not anticipate installing any additional groundwater observation wells. All test borings drilled in areas were alluvial soils are likely to be encountered will be continuously sampled in order to determine the nature and extent of the deposit as it relates to embankment/abutment stability (south of Presumpscot River) and liquefaction (south of MCRR).

CLOSURE

Please do not hesitate to contact us if you have any questions about this memorandum or engineering evaluations.

Attachment:
Table I: Proposed Design Phase Subsurface Exploration Program
Figure 1: Proposed Design Phase Exploration Plan (3 sheets)

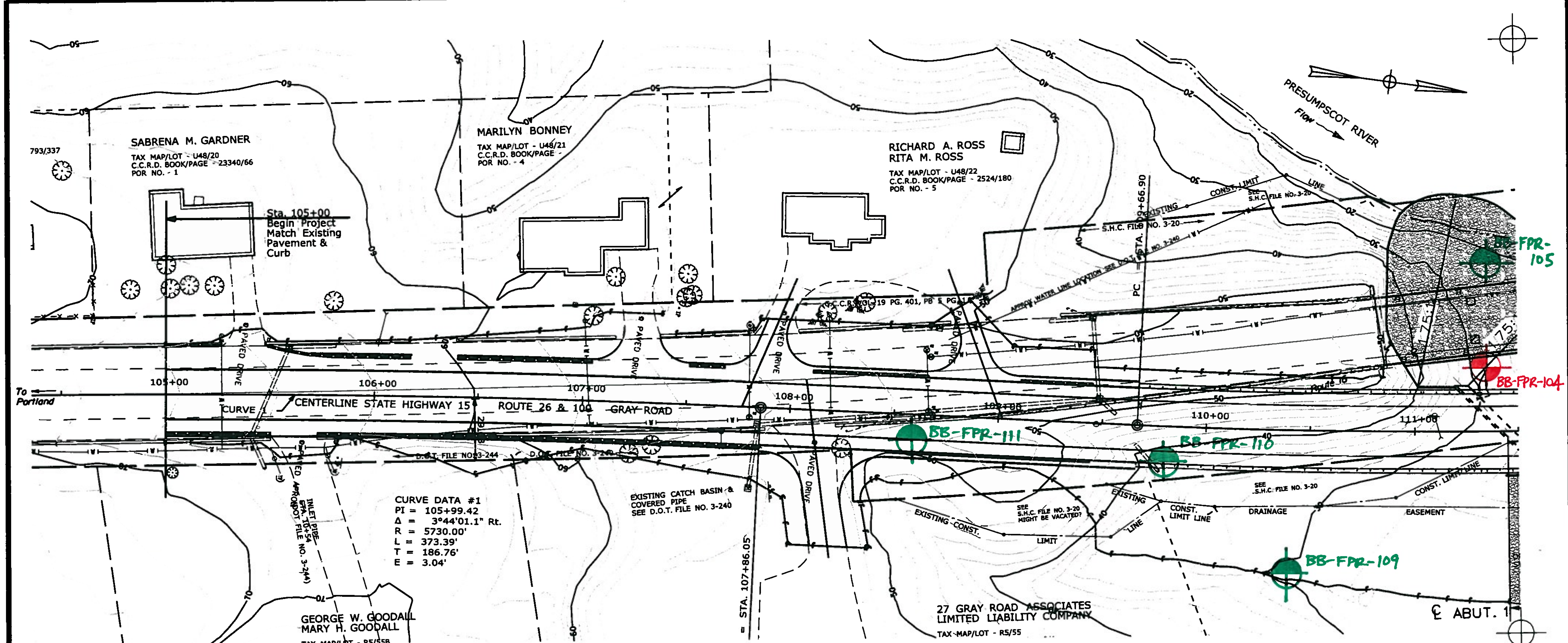
G:\PROJECTS\35524 - Presumpscot River Bridge\Design Phase Explorations\2009_0114_HAI_Design Phase Explorations_FINAL.doc

TABLE I
PROPOSED DESIGN PHASE SUBSURFACE EXPLORATION PROGRAM
PROPOSED BRIDGE REPLACEMENT OVER PRESUMPSCOT RIVER AND MCRR
ROUTES 100/26 - FALMOUTH, MAINE

MAINEDOT PIN NO.: 15094.00
HALEY & ALDRICH FILE NO.: 35524-000

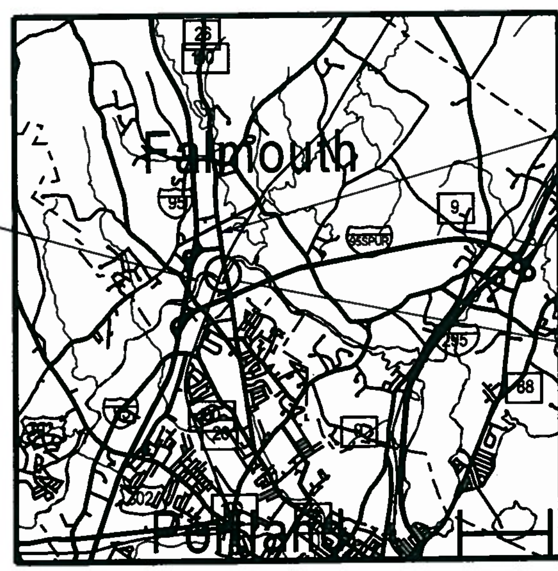
Test Boring Number	Approximate Test Boring Location (Station)	Anticipated Boring Length in Soil (lf)	Anticipated Boring Length in Rock (lf)	Anticipated Number of In-Situ Vanes	Anticipated Number of Tube Samples	OW/PZ Installed?	Purpose
BB-FRR-201	Pier 3 (STA 117+25)	130	10	0	0	No	substructure
BB-FRR-202	North Abutment (STA 118+50)	170	10	8	3	No	substructure, stability, settlement
BB-FRR-203	North Approach Embankment (STA 120+00)	90	0	0	0	No	stability, settlement
BB-FRR-204	North Approach Embankment (STA 121+25)	90	0	0	0	No	stability, settlement
BB-FPR-201	South Approach Embankment (STA 108+50)	20	0	0	0	No	stability
BB-FPR-202	South Approach Embankment (STA 109+75)	20	0	0	0	No	stability
BB-FPR-203	South Approach Embankment (STA 110+50)	30	0	2	0	No	stability
BB-FPR-204	South Abutment (STA 111+50)	30	0	0	0	No	liquefaction, stability
BB-FPR-205	South Abutment (STA 111+50)	130	10	0	0	No	substructure,liquefaction, stability
BB-FPR-206	South Approach Embankment (STA 111+50)	30	0	2	0	No	liquefaction, stability
BB-FPR-207	South Abutment (STA 112+00)	30	0	0	0	No	liquefaction, stability
BB-FPR-208	Pier 2 (STA 115+25)	130	10	0	0	No	substructure, liquefaction

HALEY & ALDRICH



PLAN

Scale of Feet: 25 0 25 50



Presumpscot River

RTE 100 / RTE 26

PROJECT

LOCATION

Scale of Miles: 1 0 1 2

LEGEND:

BB-FPR-104 PRELIMINARY DESIGN EXPLORATION (APPROX.)

BB-FPR-105 PROPOSED FINAL DESIGN EXPLORATION (APPROX)

SPECIFICATIONS

DESIGN: Load and Resistance Factor Design per AASHTO LRFD Bridge Design Specifications 2007 and interim specifications through 2008.

TRAFFIC DATA

Current (2008) AADT	13820
Future (2028) AADT	16570
DHV - % of AADT	10%
Design Hour Volume	1657
Heavy Trucks (% of AADT)	5%
Heavy Trucks (% of DHV)	5%
Directional Distribution (% of DHV)	51%
18 Kip Equivalent P 2.0	356
18 Kip Equivalent P 2.5	340
Design Speed	40 mph

DESIGN LOADING

LIVE LOAD: HL - 93 Modified for Strength I (Truck only increased 25%)

MATERIALS

CONCRETE:

Curbs, End Posts	Class LP
Seals	Class S
All Other	Class A

REINFORCING STEEL: ASTM A615 / A615M Grade 60

STRUCTURAL STEEL
All Material (unless otherwise noted) ---- ASTM A709, Grade 50W (unpainted)
High Strength Bolts ---- ASTM A325, Type 3

BASIC DESIGN STRESSES

CONCRETE:	f'c = 4,350 psi
REINFORCING STEEL:	fy = 60, 000 psi
STRUCTURAL STEEL	
ASTM A709, Grade 50W	Fy = 30, 000 psi
ASTM A325	Fn = 120, 000 psi

UTILITIES

Pan Am Railways
Portland Water District

MAINTENANCE OF TRAFFIC

Maintain traffic over existing bridge during construction.

SCOPE OF WORK

Construct replacement bridge and approaches on new parallel alignment east of existing bridge. Remove existing bridge following shift of traffic onto new bridge.

APPROXIMATE COST

Estimated Construction Cost ---- $X,XXX,XXX

HALEY & ALDRICH, INC.
8 JANUARY 2009

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1509(400)X

PIN 15094.00

BRIDGE NO 2702

BRIDGE PLANS

RR CROSSING BRIDGE
PRESUMPSCOT RIVER
FALMOUTH CUMBERLAND COUNTY

PRELIMINARY PLAN

SHEET NUMBER
1
OF 7

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Division: $wkgroup$
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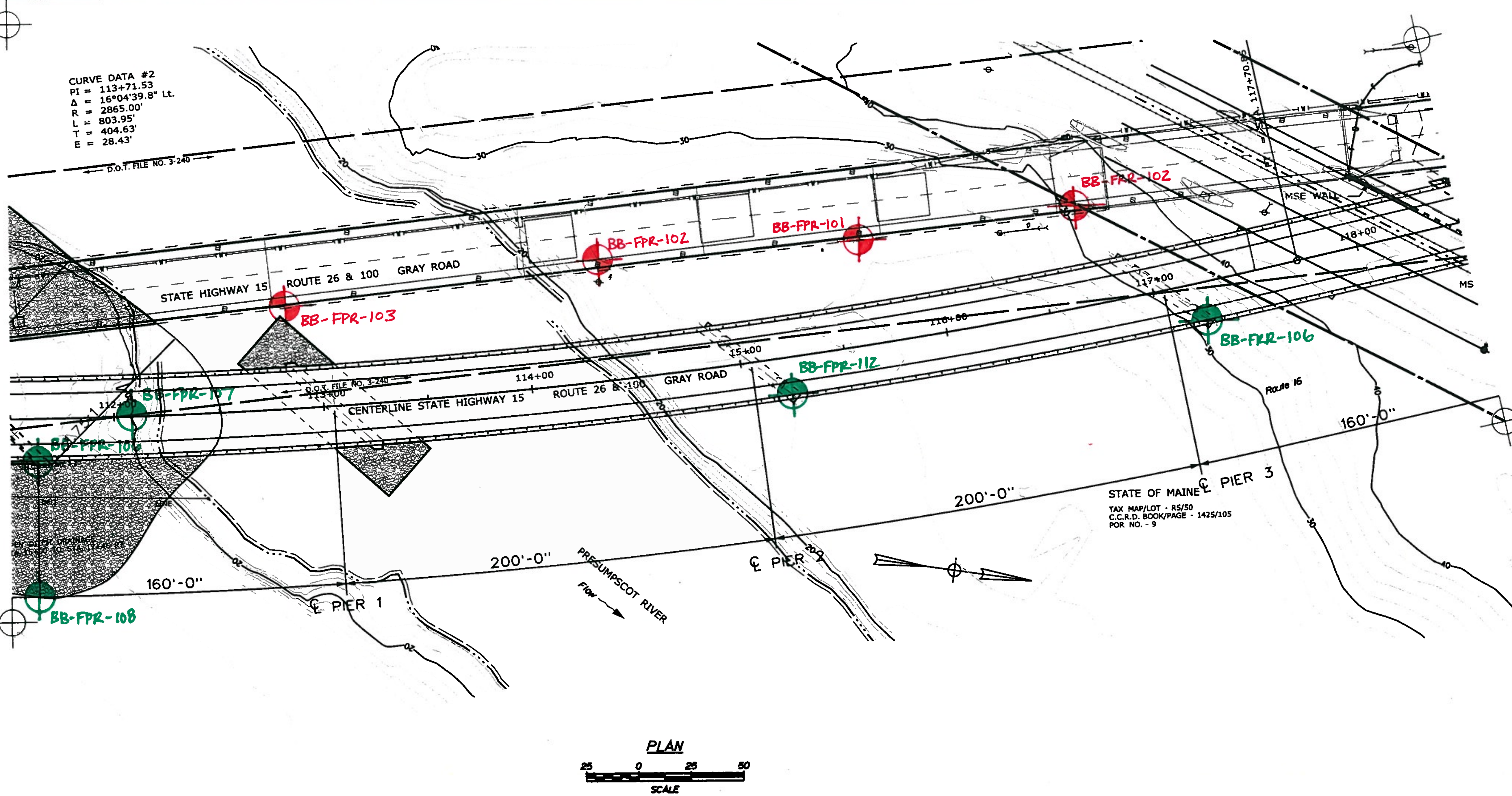
Date: $date$



LEGEND:

BB-FPR-101 PRELIMINARY DESIGN EXPLORATION (APPROX.)

BB-FPR-106 PROPOSED FINAL DESIGN EXPLORATION (APPROX.)

HALEY & ALDRICH, INC.
8 JANUARY 2009

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1509(400)X

BRIDGE NO. 2702

PIN 15094.00

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	JKO / DWB2	1/09
CHECKED-REVIEWED	RGH / DWB	1/09
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

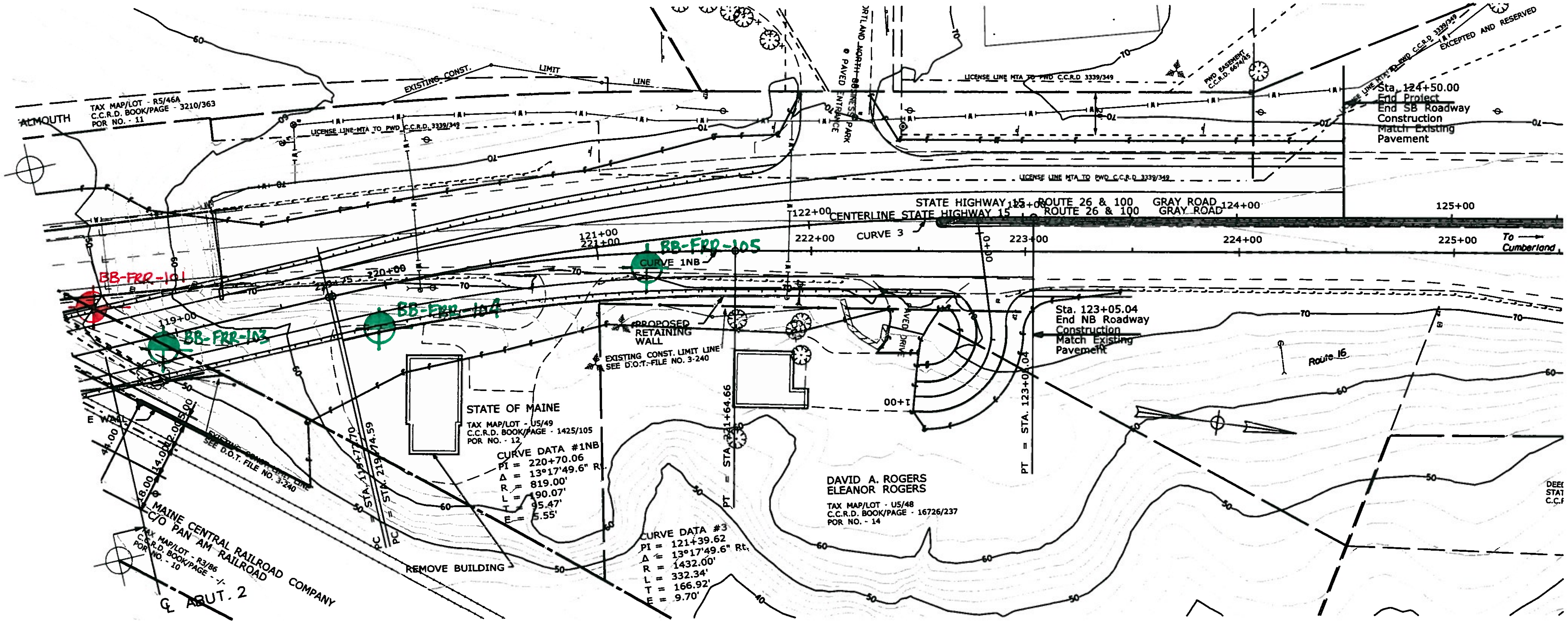
P.E. NUMBER

DATE

RR CROSSING BRIDGE
PRESUMPSCOT RIVER
FALMOUTH CUMBERLAND COUNTY

PRELIMINARY PLAN

SHEET NUMBER
2
OF 8



PLAN

25 0 25 50

SCALE

LEGEND:

BB-FRR-101 PRELIMINARY DESIGN EXPLORATION (APPROX.)

BB-FRR-103 PROPOSED FINAL DESIGN EXPLORATION (APPROX.)

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1509(400)X

PIN 15094.00

BRIDGE NO. 2702

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	JKO	1/09
CHECKED-REVIEWED	RSH	1/09
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

RR CROSSING BRIDGE
PRESUMPSCOT RIVER
FALMOUTH CUMBERLAND COUNTY

PRELIMINARY PLAN

SHEET NUMBER
3
OF 8

HALEY & ALDRICH, INC.
8 JANUARY 2009

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Fax: 207.775.7666
HaleyAldrich.com

HALEY& ALDRICH

MEMORANDUM

16 January 2009
File No. 35524-000

TO:	TY Lin International Rick Hebert, P.E.
C:	Maine Department of Transportation Leanne Timberlake, P.E., Laura Krusinski, P.E.
FROM:	Haley & Aldrich, Inc. Bryan C. Steinert, Wayne A. Chadbourne, P.E., James W. Weaver, P.E. BCS
SUBJECT:	Preliminary South Bridge Abutment and Approach Embankment Evaluation Proposed Route 100/26 Bridge Replacement Project PIN 15094.00 Falmouth, Maine

As discussed during our 22 December 2008 meeting, we have completed preliminary-level technical and cost evaluations for various bridge abutment and approach embankment alternatives for the south end of the project (south bridge abutment and approach embankment). For the purpose of our evaluations, we have assumed that the abutment and approach embankment will be located on an alignment parallel and to the east of the current bridge as shown on the attached Sketch No. 1 prepared by TY Lin.

General Subsurface Conditions

The subsurface conditions used in the evaluations were developed based on the conditions encountered in the recently completed test boring BB-FPR-104, drilled in the vicinity of the existing south abutment (see Sketch No. 1). For the purposes of our evaluation, we assumed the following generalized subsurface conditions (in order of increasing depth below ground surface): 4 ft of loose to medium dense sand (existing embankment fill), 15 ft of interbedded layers of sand, silt and clay (alluvial deposit), 10 ft of medium stiff marine clay (undrained shear strengths of approximately 900 psf) and roughly 100 ft of dense to very dense glacial till overlying bedrock.

It is likely that the thickness of the surficial alluvial deposit decreases in thickness to the south, as the ground surface elevation rises above the Presumpscot River (river) flood levels. In addition, it appears that the thickness of the marine clay unit also decreases from north to south. However, based on the limited subsurface information available, it is not known at this time specifically how the soil conditions along the proposed alignment change. Therefore, it is our opinion that the subsurface conditions assumed for these evaluations are considered to be conservative. Additional test borings will be required during final design in order to define the soil profile north, south and east of test boring BB-FPR-104 in order to accurately model the bridge abutment and approach embankment alternatives. We have provided a memorandum summarizing this program, dated 14 January 2009.

Engineering Evaluations

Based on our previous evaluations it appears that global stability of the approach embankments will control the location and design of the south approach embankment and bridge abutment.

Global stability evaluations were performed modeling the existing ground surface level along the centerline of the proposed approach embankment utilizing the subsurface soil profile described above. The calculated factor of safety for the existing site condition is on the order of 1.4. However, it should be noted that the existing river bank is susceptible to shallow slope failures and surficial sloughing with existing calculated factors of safety below 1.0. Historically, this type of failure has been common along the river and evidence is visible in the vicinity of the project site. It is our opinion that the calculated factors of safety are conservative, except in the vicinity of the river bank, due to the assumptions made relative to the presence and thickness of the alluvial and marine clay deposits as well as the strength properties assigned to the alluvial soils. However, it does serve as a baseline to assess proposed abutment and approach embankment alternatives.

Subsequent global stability evaluations were conducted modeling two proposed bridge abutment location alternatives as shown below (see attached Sketch No. 1).

- Alternative No. 1 = Station 111+45
- Alternative No. 2 = Station 111+00

Alternative No. 1 was evaluated using normal-weight earthfill to construct the approach embankment resulting in an unacceptable calculated factory of safety equal to 1.0 (we consider acceptable factors of safety to be greater than 1.3). This is primarily due to the raise in grade relative to existing ground surface levels (up to 35 ft) that will be required to construct the south abutment and approach embankment. As a result, we looked at the following two embankment construction options for Alternative No. 1:

- Alternative No. 1A: using lightweight fill (EPS, geofoam) extending 60 ft behind the abutment and within the approach embankment, with a rockfill toe berm in front of the pile-supported stub abutment, and a riprap slope (on top of the toe berm) extending into the river.
- Alternative No. 1B: using geofoam extending 70 ft behind the abutment and within the approach embankment, with a wrapped face reinforced soil mass behind the geofoam cell, a full-height, vertical-sided MSE wall in front of the stub abutment and a riprap slope extending into the river for scour protection.

Calculated factors of safety for Alternative Nos. 1A and 1B are approximately 1.3 and 1.4, respectively.

Similar evaluations were performed for Alternative No. 2. Using normal-weight earthfill to construct the approach embankment resulted in an unacceptable calculated factor of safety equal to approximately 1.2. Therefore, we looked at the following options for Alternative No. 2:

- Alternative No. 2A: using geofoam extending 25 ft behind the abutment, with a rockfill toe berm in front of the pile-supported stub abutment and a riprap slope (on top of the toe berm) extending down into the river.
- Alternative No. 2B: using geofoam extending 25 ft behind the abutment, with a wrapped face reinforced soil mass behind the geofoam cell, a full-height, vertical sided MSE wall in front of the pile-supported stub abutment, and a riprap slope extending into the river for scour protection.

Calculated factors of safety for Alternative Nos. 2A and 2B are approximately 1.3 and 1.4, respectively.

Therefore, it is our opinion that using a combination of lightweight (geofoam) and normal-weight (soil) fill is practicable for both Alternative Nos. 1 and 2. However, it should be noted that for Alternative No. 1, the global stability of the abutment and approach embankment is sensitive to the stability of the alluvial soils between the abutment and the river. Additional measures would be needed to improve the properties of the alluvial soils in this area (i.e., ground improvement) to ensure stability during the design life of the bridge.

Both alternatives are considered to be technically feasible. Selecting the most practicable alternative will likely be based on cost implications to the project. Alternative No. 1 will require the greatest quantity of geofoam, the least amount of bridge superstructure and ground improvement between the abutment and river. Conversely, Alternative No. 2 would minimize the quantity of geofoam but will maximize the amount of rockfill and bridge superstructure.

A preliminary-level assessment of potential impacts (due to the weight of the proposed toe berm) to the existing bridge abutment and pier west of the proposed south abutment were made. Due to construction phasing requirements, the portion of the new toe berm under the existing bridge would not be constructed until after the bridge has been demolished. Therefore, the existing abutment and pier should not be affected by construction of the toe berm east of the existing bridge. Furthermore, we believe the marine clay beneath the proposed south abutment is sufficiently preconsolidated such that post-construction consolidation will not cause significant downdrag on the proposed abutment piles.

Cost Implications

A preliminary-level estimate of the cost implications for Alternative No. 1 and Alternative No. 2 described above was completed using the following assumptions and is summarized below:

- Geofoam will be used as lightweight fill at a cost of $130 per cubic yard (cy).
- Earthfill will be provided for pavement section and protection of the geofoam at $25 per cy.
- The end wall (Alternative Nos. 1B and 2B) will consist of vertical mechanically stabilized earth (MSE) walls at a cost of $70 per square foot (sf).
- Wrapped faced reinforced soil mass behind the geofoam cells (Alternative Nos. 1B and 2B) $40 per sf.
- An allowance for ground improvement at a cost of $100,000.
- The approach embankments will be 50 ft wide and the bridge deck will be 44 ft wide.
- Volume calculations for earthfill and geofoam, and square footage estimates for the MSE walls are based on the centerline profile along the proposed road/bridge alignment as shown on Sketch 1 (attached).

South Abutment Location Alternative	Material Quantity						Cost ($)
	Geofoam (cy)	Toe Berm (cy)	MSE Wall (sf)	Reinforced Soil (sf)	Ground Improvement ($)	Additional Bridge Deck (sf)	
1A	2,800	2,500	0	0	$100,000	0	$536,500
1B	3,300	550	1,100	1,400	0	0	$590,700
2A	875	2,900	0	0	$100,000	1,760	$814,250
2B	875	770	750	1,050	0	1,760	$782,450

Please note that costs for earthfill volume required to construct the embankment side slopes and the embankment behind the wrapped reinforced soil mass.

Based on the information summarized above, it is more cost effective to construct the abutment at Alternative No. 1 primarily due to the increased cost of the bridge superstructure associated with Alternative No. 2. Furthermore, it is more cost effective to construct a pile-supported stub abutment on a geofoam fill embankment with a rockfill toe berm and ground improvement (1A) as compared to constructing a vertical-sided MSE wall (1B).

Therefore, we recommend that Alternative No. 1A, as described above, be used as the basis for the preliminary design report (PDR). Design phase studies will be conducted to develop design details, refine stability analyses, and determine the most practicable ground improvement method for improving the engineering properties of the alluvial soils between the abutment and river.

The cost of a lightweight fill (geofoam) approach embankment will be more costly than an approach embankment using normal-weight earthfill. Assuming a similar embankment geometry (50 ft wide embankment with vertical MSE side and end walls), the premium cost of a geofoam embankment will be the differential cost of the geofoam ($130/cy) and the earthfill ($25/cy) multiplied by the volume of geofoam, or approximately $300,000.

Closure

We trust these comments and recommendations are suitable for your present needs. Please do not hesitate to contact us if you have any questions about this memorandum or engineering evaluations.

Attachment:
Sketch 1 - South Bridge Abutment and Approach Embankment Alternatives

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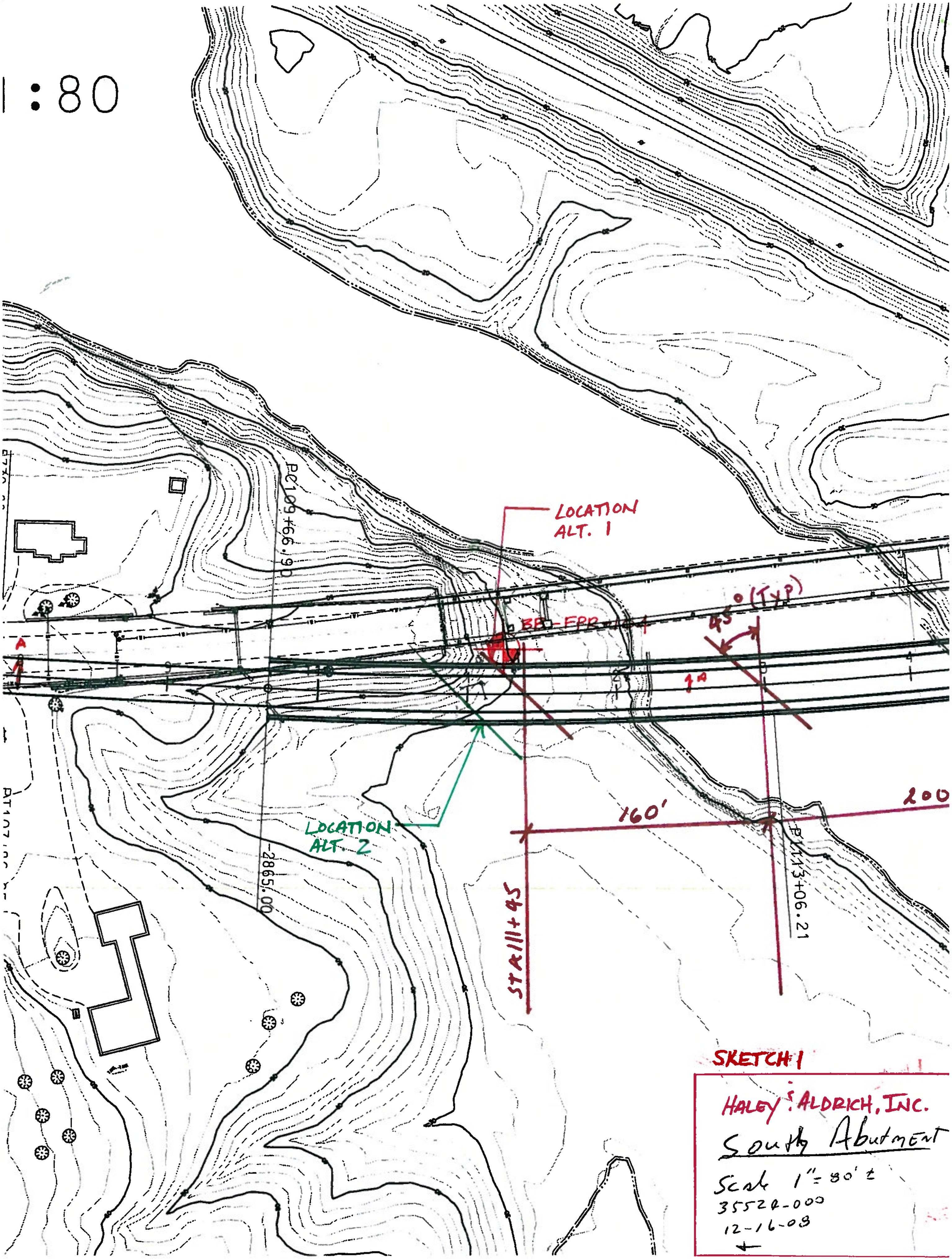
1:80


LOCATION
ALT. 1
45° (TYP)
PC109+66.90
LOCATION
ALT. 2
2865.00
160'
200
STA 111+45
PT113+06.21
SKETCH 1
HALEY & ALDRICH, INC.
South Abutment
Scale 1" = 80' ±
35520-000
12-16-08

Haley & Aldrich
75 Washington Avenue
Suite 203
Portland, ME 04101-2617

Tel: 207.482.4600
Fax: 207.775.7666
HaleyAldrich.com

HALEY& ALDRICH

MEMORANDUM

16 January 2009
File No. 35524-000

TO:	TY Lin International Rick Hebert, P.E.
C:	Maine Department of Transportation Leanne Timberlake, P.E., Laura Krusinski, P.E.
FROM:	Haley & Aldrich, Inc. Bryan C. Steinert, Wayne A. Chadbourne, P.E. BCS
SUBJECT:	Preliminary Pile Foundation Evaluation Proposed Route 100/26 Bridge Replacement Project PIN 15094.00 Falmouth, Maine

As discussed during our 13 November 2008 meeting, we have completed a preliminary-level technical evaluation for various driven pile alternatives for support of the bridge abutments and piers.

SUBSURFACE CONDITIONS

Based on the results of our recently completed preliminary phase test boring program, the subsurface conditions along the existing bridge structure alignment are moderately variable. The following generalized subsurface conditions are presented below, in order of increasing depth below existing ground surface.

Alluvial Deposit: Interbedded layers of sand, silt, clay and occasional organics were encountered at the ground surface in BB-FPR series test borings (see attached figure). The deposit was very loose to medium dense and ranged in thickness from approximately 4 ft to 25 ft.

Marine Clay Deposit: A marine clay deposit was encountered in each test boring. The clay was encountered near the ground surface in the FRR series test borings and ranged in thickness from approximately 50 to 80 ft (increasing from south to north). The marine clay was encountered beneath the alluvial deposit in the FPR series test borings and ranged in thickness from approximately 10 to 20 ft. The marine clay was generally soft to medium stiff with undrained shear strengths ranging from 400 to 900 pound per square foot (psf).

Marine Sand Deposit: A deposit of poorly graded fine to medium sand with silt was encountered in each test boring with the exception of BB-FPR-104. The marine sand was encountered directly beneath the marine clay layer and ranged in thickness from approximately 30 to 60 ft. The marine sand was generally loose to very dense.

Glacial Till Deposit: A heterogeneous mixture of clay, silt, sand and gravel was encountered in each test boring directly beneath the marine sand layer with the exception of test boring BB-FPR-104 where it was overlain by marine clay. The deposit ranged in thickness from approximately 26 ft in the vicinity of the railroad to approximately 35 ft north of the Presumpscot River (river) to approximately 100 ft, south of the river. The glacial till was typically medium dense to very dense in nature.

Bedrock: Bedrock was encountered in each test boring, typically at depths ranging from approximately 85 to 170 ft below ground surface.

ENGINEERING EVALUATIONS

Preliminary-level pile evaluations were conducted for the subject project. Pile evaluations were conducted in accordance with AASHTO LRFD methodology. Specific evaluations are discussed below.

Axial Compression Pile Capacity

Based on conversations with you, we have evaluated the potential for using either displacement piles, which would develop the majority of the load carrying capacity through end bearing resistance in the glacial till deposit, or non-displacement piles, which would be driven through overburden soils to practicable refusal in dense glacial till or in/on bedrock. Specifically, we have evaluated the following pile types:

- HP14x73 and HP14x117 steel H-piles (H-piles)
- 12-¾-in. diameter (0.375-in. wall thickness) and 16-in. diameter (½-in. wall thickness) concrete filled steel pipe piles driven with closed end (pipe piles)
- 16-in. square precast prestressed concrete (PPC) piles

It is our opinion that friction piles, which generate the majority of their load carrying capacity in side resistance between the pile material and soil, are technically feasible. However, their geotechnical design capacities would be significantly less than an end bearing piles. As a result, a greater number of friction piles would be required to resist the same load. Therefore, it is our opinion that friction piles are not cost competitive with end bearing piles.

Each of the pile types listed above is technically feasible. Since the displacement piles (pipe piles and PPC piles) will develop the majority of their load carrying capacity through end bearing resistance in the glacial till deposit, they will generally be shorter than H-piles.

The pipe piles would be cost competitive with the H-pile, however, additional time would be required during installation for concrete placement and closed-ended pipe piles are more susceptible to damage when driving into dense glacial till and/or bedrock as compared to H-piles.

PPC piles are also considered to be cost competitive with both H-piles and pipe piles. However, there are several constructability issues associated with driving PPC piles including the need for high capacity cranes (piles weigh 2 to 3 times more than H-piles or pipe piles) and large pile hammers. Furthermore, PPC piles are more difficult and costly to splice if they are driven to depths greater than anticipated.

Therefore it is our opinion that H-piles driven to practicable refusal into dense glacial till or in/on bedrock be considered for preliminary-level design. A summary of approximate pile tip elevations along the proposed alignment is provided below for both an HP14x73 with a factored geotechnical resistance of approximately 165 tons and an HP14x117 pile with a factored geotechnical resistance of approximately 215 tons.

Abutment / Pier Location	Applicable Test Boring	Approximate Pile Tip Elevation[1]	
		HP14x73	HP14x117
South of Presumpscot River	BB-FPR-104	-70	-100
Presumpscot River	BB-FPR-103	-90	-95
Between Presumpscot River & Railroad	BB-FPR-101,102 and BB-FRR-102	-90	-100
North of Railroad	BB-FRR-101	-110	-120

[1] – Elevations reference NAVD 88.

Pile Group Analyses

The preferred bridge alignment alternative, as provided by TY Lin, consists of two abutments and three piers. Pier 2 is located between the Presumpscot River and MCRR, generally within the area where liquefaction susceptible alluvial soils were encountered during the preliminary phase test boring program.

Pile group analyses were performed to evaluate pile cap deflections. Group analyses were conducted using the computer program FB-MultiPier. A nine pile group (3 rows of 3 piles) consisting of either plumb steel HP14x89 or HP14x117 section piles was modeled with the soil profile encountered in test boring BB-FPR-102 and the service limit state loads provided by TY Lin. Lateral deflections at the pile heads for both HP14x89 and HP14x117 section piles were in excess of 6 in. Based on conversations with TY Lin, it is our understanding that movement at the pile cap should be limited to no greater than 1.5 in., preferably 1.0 in. To reduce lateral deflections to tolerable limits, we modeled a composite pile section consisting of a 125-ft long HP14x117 pile with a concrete filled 24-in. diameter pipe pile (½-in. thick wall) extending from the bottom of pile cap to a depth of 60 ft (i.e., below the depth of fixity). The composite pile section reduced deflections at the pile heads to less than 1 in. Effects of loading due to potential lateral spreading of liquefied soils will be evaluated during the design phase of the project.

As a result, for preliminary design we recommend that you carry the composite pile section as described above to support Pier 2. Based on our conversations with pile driving contractors, we recommend that you carry a cost of $225/l.f. for the installation, cleaning out and concreting of the pipe piles in addition to the cost of furnishing and installing the HP14x117 pile.

Additional Factors Affecting Pile Compression Capacity

The alluvial soil deposit encountered in recently completed test borings drilled between the Presumpscot River and Maine Central Railroad (MCRR) is susceptible to liquefaction during the design earthquake event as discussed in our 1 December 2008 memorandum. It is our opinion that liquefaction induced downdrag loads on piles installed to support piers in this would not result in a reduction in design capacity due primarily to the limited strata thickness. However, it is possible that the presence of liquefaction susceptible soils in this area could preclude the use of batter piles used to resist lateral loads.

Liquefaction susceptible soils were not encountered in test borings drilled in the vicinity of the north and south abutments.

Significant raises in site grade will be required to construct the proposed north bridge approach embankment and abutment. Recommendations made for the north approach embankment and abutment; presented in our 26 December 2008 memorandum, describe the use of lightweight fill (geofoam) to reduce/eliminate consolidation settlement and global stability issues associated with a thick deposit of marine clay (no net stress increase on the underlying marine clay). Because of this, downdrag loading on piles driven to support the proposed north abutment will be negligible.

Similarly, large fills will be required to construct the south bridge approach embankment. However, the entire hybrid embankment (lightweight and normal-weight fill) will be constructed prior to pile installation. Due to the limited thickness and compressibility characteristics of the marine clay present south of the Presumpscot River, it is our opinion that the majority of consolidation settlement will occur prior to pile installation resulting in negligible downdrag loading on the south abutment piles.

CLOSURE

It is our understanding, based on conversations with you, that multiple bridge alternatives are still being considered at this time. Therefore, we recommend that additional test borings be drilled during final design, once the location and number of abutments and piers has been finalized in order to accurately assess pile capacity and anticipated pile lengths.

We trust these comments and recommendations are suitable for your present needs. Please do not hesitate to contact us if you have any questions about this memorandum or engineering evaluations.

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HALEY & ALDRICH

MEMORANDUM

10 July 2009
File No. 35524-010

TO: Maine Department of Transportation
Laura Krusinski, P.E.

C: T.Y. Lin International
Rick Hebert, P.E.

FROM: Haley & Aldrich, Inc.
Bryan Steinert, P.E., Wayne Chadbourne, P.E., James Weaver, P.E.

SUBJECT: North Abutment and Wingwall Alternative Evaluation
Proposed Replacement Bridge over Presumpscot River and MCRR
Routes 100/26 – Falmouth, Maine
PIN 15094.00

This memorandum summarizes the results of our evaluation of potential wall systems for the north abutment and wingwalls associated with the replacement of the Route 100/26 bridge structure over the Presumpscot River and Maine Central Railroad (MCRR) in Falmouth, Maine.

As detailed in our memorandum dated 26 December 2008 and subsequent correspondence, the north approach embankment will be constructed using lightweight fill (EPS), referenced herein as geofoam, due to the presence of soft, compressible and low strength marine silt and clay deposits beneath the roadway alignment. The maximum height of the approach embankment is approximately 30 feet above existing site grades at the east corner of the north abutment (Station 118+76) and is approximately 7 feet high at near Station 121+00. The proposed approach embankment will match existing site grades in the vicinity of Station 123+00.

In order to control project costs and reduce the amount of geofoam, it was decided that a vertical-sided approach embankment would be provided along the eastern edge of the new roadway alignment. The geofoam is self-supporting but requires a facing for protection. The geofoam also needs protection from traffic loads as well as from petroleum products in the event a spill occurs. Therefore, a nominal 5-foot thick layer of normal-weight earth fill will be provided over the geofoam. The pavement section for the new roadway will be provided within the 5-foot thick earthfill cover layer. The 5-foot thick cover layer needs to be supported at its edge with an earth retaining structure. Refer to the attached Sketch 1 for a schematic section of the wall system.

The combination of an earth retaining structure on top of a geofoam facing system has created a demand for a unique wall system. The upper portion of the wall will have to resist lateral earth pressures from the retained normal-weight earth fill as well as traffic loads and impact loads applied to the guard rail; whereas the lower section will not be subjected to significant lateral pressures, primarily due to the self-supporting nature of the geofoam cell. A relatively thin (4 to 6 in.) concrete distribution slab will be constructed over the geofoam cell, within the limits of the travel lanes, to distribute traffic loads. In addition, a high-density polyethylene (HDPE) liner will need to be provided to protect the geofoam from petroleum (or other products that can degrade the geofoam) spills.

A similar wall system was designed and constructed for the Utah Department of Transportation (UtahDOT) for the I-15 Corridor Reconstruction project in the late 1990's. Refer to the attached contract drawing sheets for typical wall sections and details showing the wall system components. As shown, the wall system consists of a nominal 6-inch thick prestressed, precast concrete wall panel supported on a cast-in-place concrete footing. A void is present between the inside of the wall panel and the geofoam blocks. The top of the wall panel is restrained using a reinforced concrete distribution slab. A nominal 2 to 3 feet of granular fill was placed over the distribution slab. The pavement surface consisted of reinforced concrete. A traffic barrier with a reinforced concrete moment slab is supported on the top of the wall panel. The maximum height of the wall panel is approximately 8 m (26 feet).

A variety of wall systems were considered for the subject project and are described as follows (concept sketches for each option are shown on the attached Sketch 2:

Option 1 – Prestressed, Precast Concrete Panel Wall – A concept similar to the UtahDOT panel wall was developed and considered. As shown on Sketch 2 the system would consist of a full height wall panel supported on a reinforced concrete wall footing, restrained near the top with a reinforced concrete distribution slab. The upper portion of the panel wall would be designed to resist lateral earth and traffic loads. The wall system would be essentially vertical (plumb).

Option 2 – Prestressed, Precast Concrete Panel Wall With Conventional MSE Wall on Top – The lower portion of the wall system would be the same as Option 1, however, the top 5 feet of the wall would consist of a conventional MSE wall system designed to resist the lateral earth and traffic loads. The wall panel would be designed to support the MSE facing blocks.

Option 3 – Prefabricated Concrete Block Gravity Wall – This wall system would consist of large precast hollow-core concrete blocks (typical dimensions 8 feet long, 3 feet high and 3.25 feet wide) supported on a reinforced concrete wall footing. The layers of blocks would be connected by placing reinforcing steel and concrete in the hollow cores. The face would be sloped back at a nominal 9V:1H slope as shown on Sketch 2. The upper courses of blocks would be designed to resist lateral earth and traffic loads using geotextile reinforcement embedded within the 5 foot soil layer.

Option 4 – Soldier Pile and Precast Concrete Lagging – This wall system would consist of vertical steel H-section (soldier) piles driven from the ground surface, through the underlying marine deposits to competent granular soils at nominal 8 foot on-center spacing. Precast reinforced concrete panels would be inserted in the pile webs to create a concrete facing. The upper portion of the wall would be restrained using reinforcing strips embedded within the 5 foot soil layer.

We also considered a number of other options including the proprietary "T-Wall Retaining Wall System" and "Reinforced Earth" system but eliminated both of them from consideration do to the need to penetrate the geofoam embankment with reinforcing strips or structural elements. It has been our experience with geofoam embankments that any penetrations that breach the HDPE liner are costly and time consuming. Only walls that could be designed and constructed as free-standing systems were considered for this project. We also considered a variety of sheeting systems (interlocked steel, vinyl and FRP sheets) that could be designed as free-standing elements through the geofoam portion of the embankment. However, we eliminated them from consideration due to cost, little or no project experience/case studies, etc.

It is our opinion that the four options described above and shown on Sketch 2 are technically feasible and constructible. All of the options would have to be designed by the project team (Haley & Aldrich, TY Lin) as compared with a vendor-type design. Cost estimates to design and construct the various wall systems were developed using information provided by manufacturers (prestressed, precast concrete wall panels), wall system suppliers (MSE and concrete gravity block walls), system designers and contractors. A summary of our cost evaluation for the four options is summarized herein.

Option 1 - Prestressed, Precast Concrete Panel Wall

- Wall panels, delivered to the site - $22/Square Foot (SF)
- Reinforced concrete distribution slab - $10/SF
- Reinforced concrete wall footing - $3/SF
- Erection and support of wall panels - $3/SF
- Design support - $2/SF
- Total Cost - $40/SF of wall

Option 2 - Prestressed, Precast Concrete Panel Wall with Conventional MSE Wall on Top

- Wall panels, delivered to the site - $22/SF
- MSE Wall, including reinforcing and backfill - $35/SF
- Reinforced concrete wall footing - $3/SF
- Design support - $2/SF
- Total Cost - $32/SF of wall

Option 3 – Prefabricated Concrete Block Gravity Wall

- Top 5 foot of wall, including reinforcing and backfill - $30/SF
  Middle of wall, including grouted reinforcing between blocks - $35/SF
  Bottom 15 feet of wall, including grouted reinforcing and extra block reinforcement - $40/SF
- Reinforced concrete wall footing - $3/SF
- Design support - $2/SF
- Total Cost - $40/SF of wall

Option 4 – Soldier Pile and Precast Concrete Lagging

- Purchase and install steel H-section piles - $68/SF
- Precast concrete lagging, delivered to the site $15/SF
- Design support - $2/SF
- Total Cost - $85/SF of wall

Based on our engineering evaluations related to the technical aspects of the wall system, anticipated construction costs, contractor familiarity with wall construction and constructability issues, it is our opinion that **Option 3 - Prefabricated Concrete Block Gravity Wall** is the most practicable wall system for the project. Based on our 7 July phone conversation and subsequent correspondence with you, we will not proceed with advancing the design until we receive your comments and instructions.

We trust this information is suitable for your present needs. Please do not hesitate to contact us if you have any questions regarding the wall system for the north approach embankment.

Attachments:

Sketch 1 - Typical Section (1 page)
Sketch 2 - Wall System Alternative Typical Sections (1 page)
UtahDOT - Panel Wall Contract Drawings (3 sheets)

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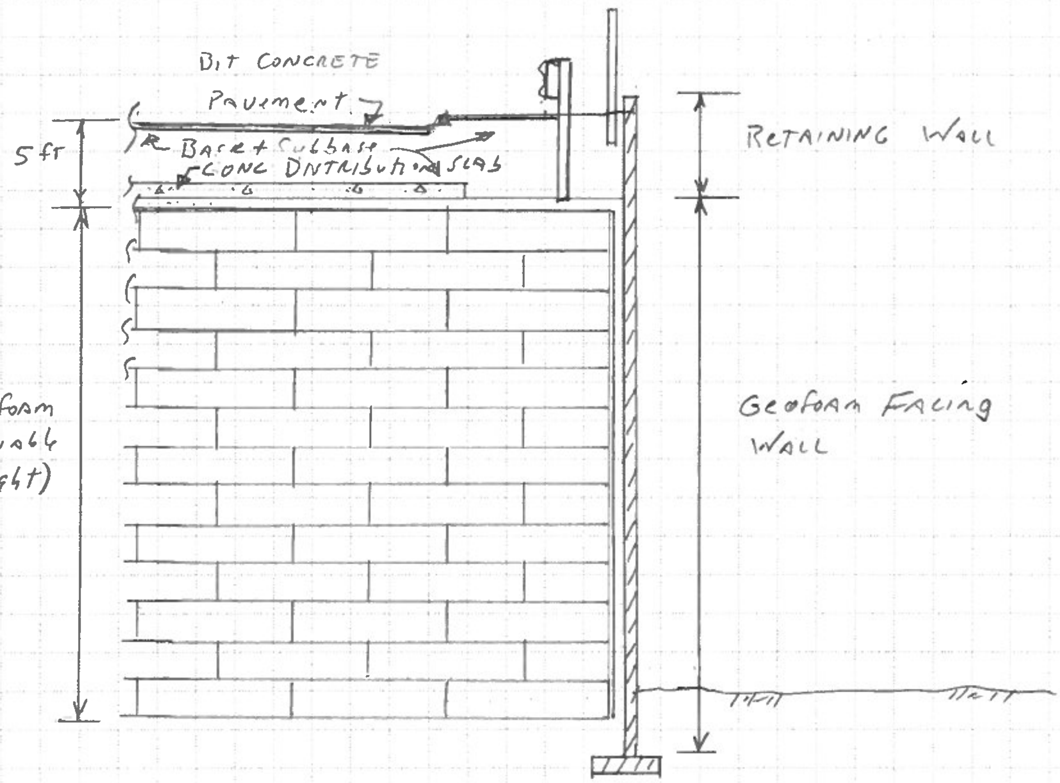
HALEY & ALDRICH

CALCULATIONS

Client	Maine DOT	File No.	35524-010
Project	Presumpscot River Bridge	Sheet	1 of 1
Subject	North Approach - Abutment + Wingwall Options Eval	Date	7-9-09
		Computed By	JW
		Checked By	BCS



TYPICAL SECTION

N.T.S.

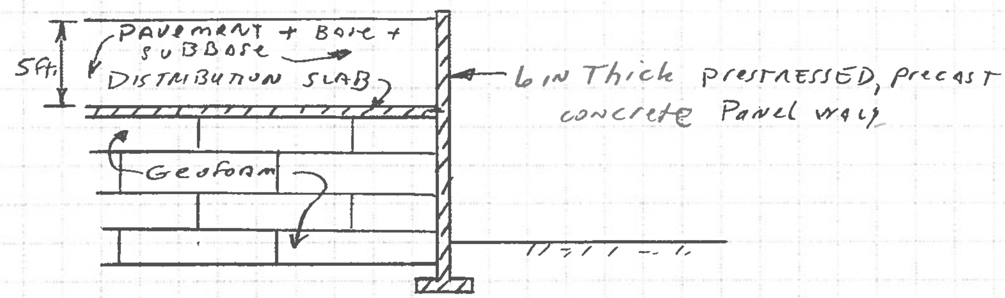
SKETCH 1

HALEY & ALDRICH

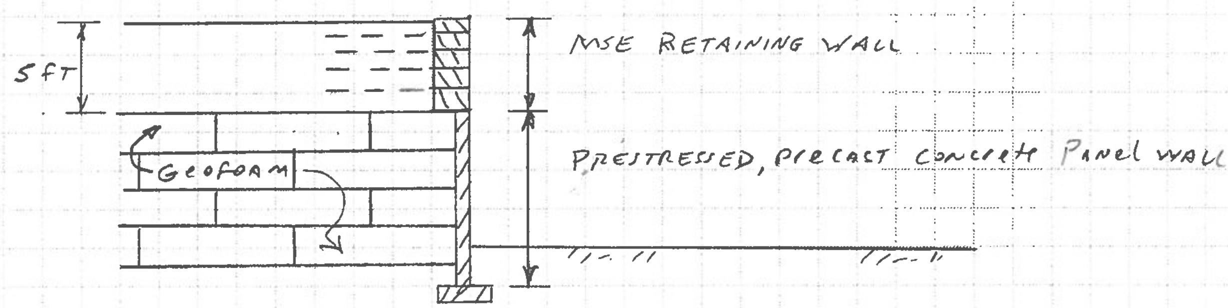
CALCULATIONS

Client	Maine DOT	File No.	35524-010
Project	Presumpscot River Bridge	Sheet	1 of 1
Subject	North Approach - Abutment + Wingwall Options Eval	Date	7-10-09
		Computed By	JW
		Checked By	BCS

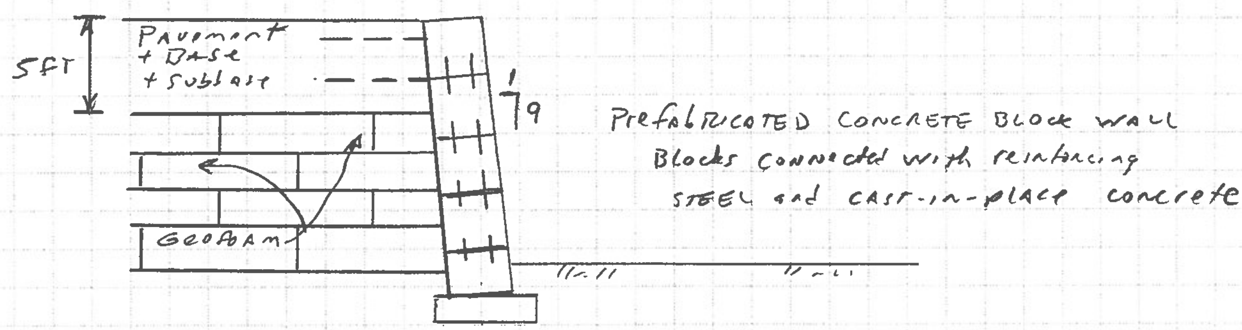
OPTION 1 - Prestressed, Precast Concrete Panel Wall



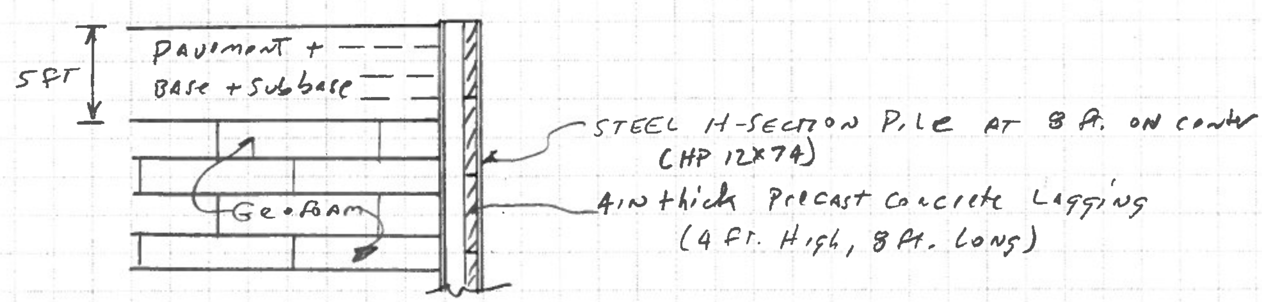
OPTION 2 - Prestressed, Precast Concrete Panel Wall with MSE Retaining Wall



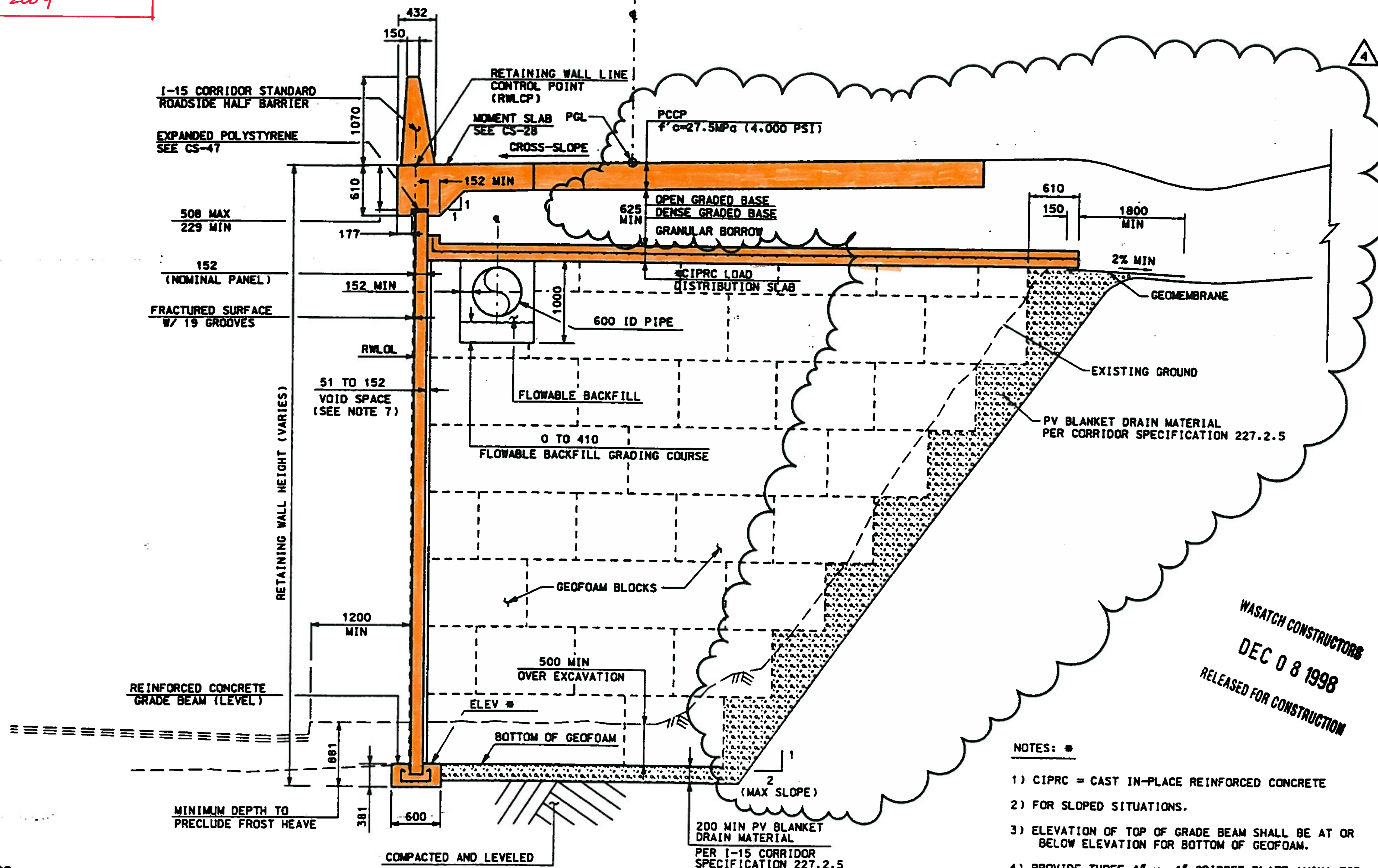
OPTION 3 - Prefabricated Concrete Block Gravity Wall



OPTION 4 - Soldier Pile and Concrete Lagging



HALEY & ALDRICH, INC.
10 JULY 2009



TYPICAL SECTION GEOFOAM (EPS) WALL

NTS

NOTES: *

1) CIPRC = CAST IN-PLACE REINFORCED CONCRETE
2) FOR SLOPED SITUATIONS.
3) ELEVATION OF TOP OF GRADE BEAM SHALL BE AT OR BELOW ELEVATION FOR BOTTOM OF GEOFOAM.
4) PROVIDE THREE 4" x 4" GRIPPER PLATE (MIN) FOR EACH 1200 x 2400 SECTION OF RIGID FOAM MATERIAL.
5) ALL DIMENSIONS ARE IN MM UNLESS OTHERWISE NOTED.
6) FOR DETAILS UNDER APPROACH AND BRIDGES SEE CS-49.
7) FOR TOP BLOCK OF GEOFOAM BELOW LOAD DISTRIBUTION SLAB 0 TO 152 VOID SPACE.
8) FOR EXISTING LOAD DISTRIBUTION SLABS, ATTACH GEOMEMBRANE TO THE TOP OF THE LOAD DISTRIBUTION SLAB WITH CIM1000.

WASATCH CONSTRUCTORS
DEC 0 8 1998
RELEASED FOR CONSTRUCTION

LICENSED PROFESSIONAL ENGINEER
No. 96-324447-2202
JOHN M. TERRY
11/24/98
STATE OF UTAH

APPROVED FOR CONSTRUCTION

NO.	DATE	DESCRIPTION
1	11/11/97	RELEASE FOR GEOFOAM WALL ONLY
2	08/28/98	APPROVED CONSTRUCTION AT BRIDGES
3	07/20/98	REVISED EPS DETAILS

UTAH DEPARTMENT OF TRANSPORTATION

SVERDRUP/DE LEUW

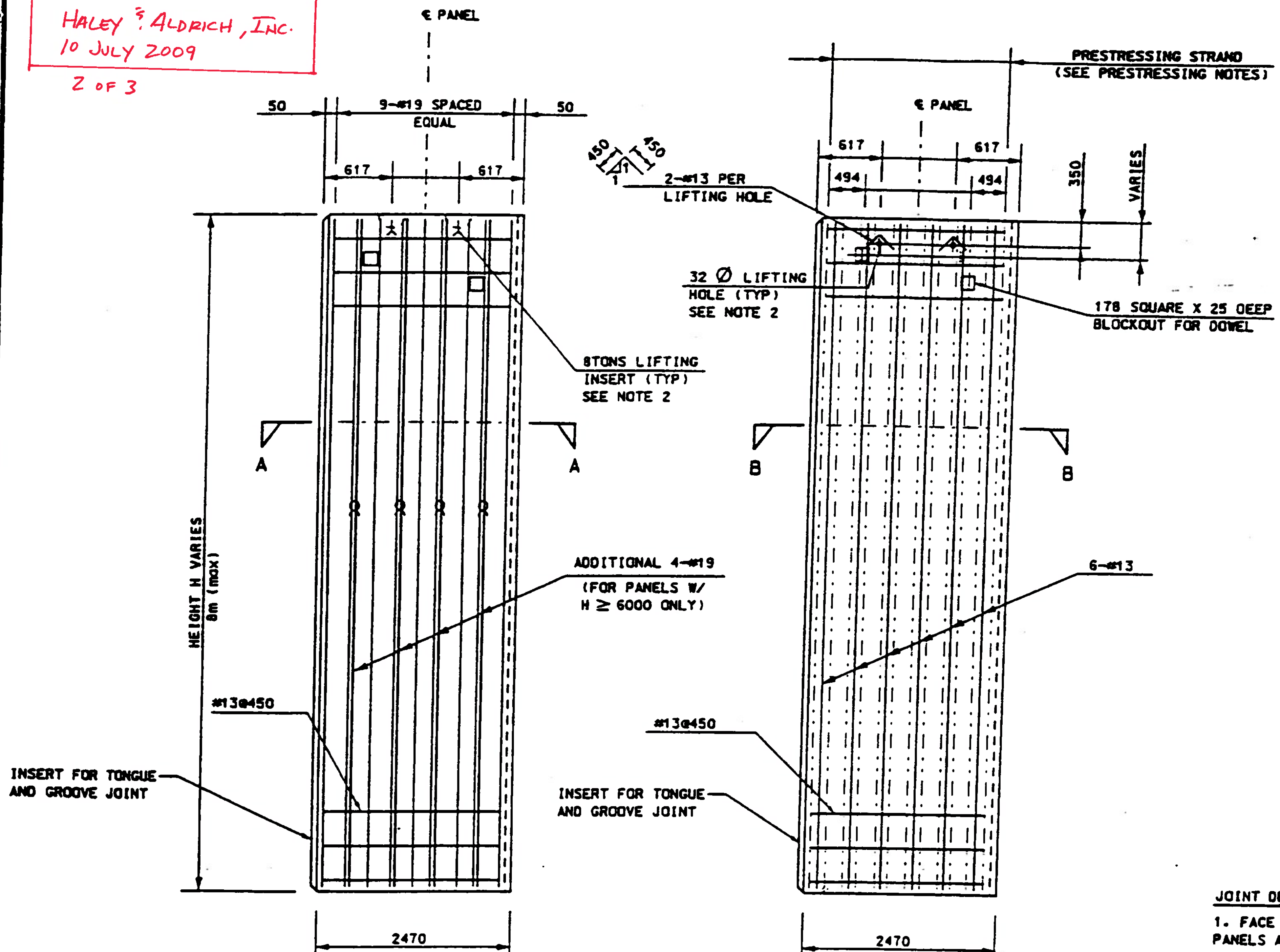
I-15 CORRIDOR RECONSTRUCTION
TYPICAL GEOFOAM SECTION
CORRIDOR STANDARD PLAN

PROJECT NUMBER #SP-15-7(135)296

SALT LAKE
COUNTY

DWG. NO. CS-51

HALEY & ALDRICH, INC.
10 JULY 2009
2 OF 3

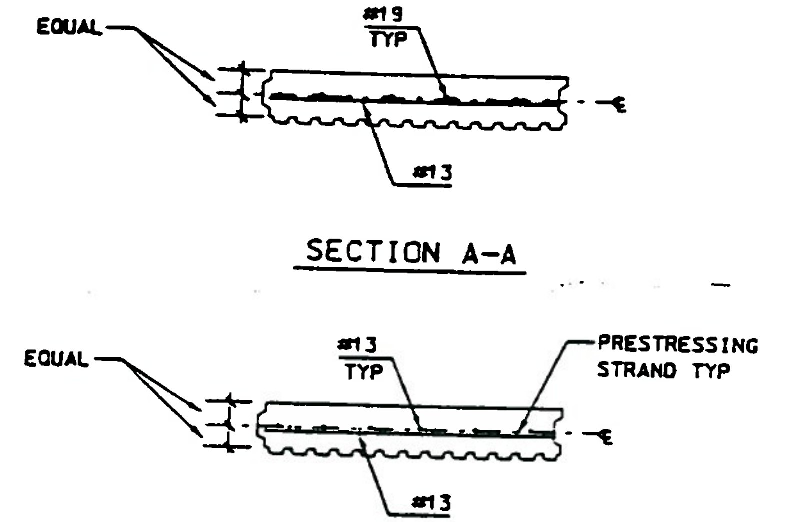


PRECAST REINFORCED CONCRETE OPTION

PRECAST PRESTRESSED CONCRETE OPTION

PRECAST WALL PANEL

NTS



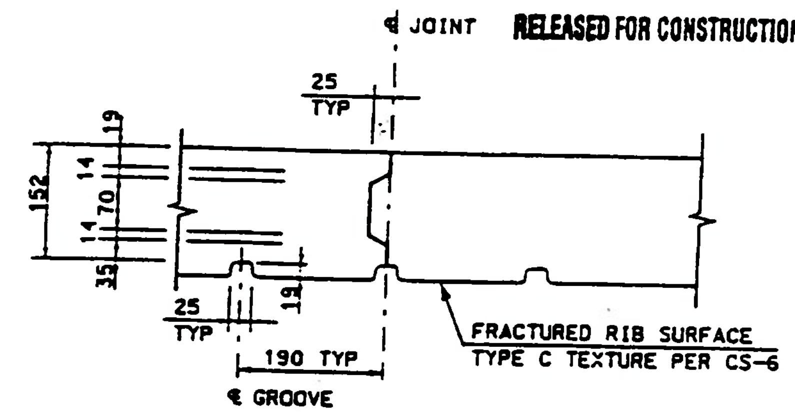
SECTION A-A

SECTION B-B

WASATCH CONSTRUCTORS

MAR 20 1998

RELEASED FOR CONSTRUCTION



TYPICAL WALL PANEL JOINT DETAIL

NTS

NOTES:

1. J BARS @ BOTTOM OF PANEL NOT SHOWN FOR CLARITY. SEE SECTION B & SHEET CS 46
2. LIFTING INSERTS OR LIFTING HOLES MAY BE UTILIZED FOR EITHER PANEL AT CONTRACTORS OPTION
3. CONTRACTOR TO PROVIDE DOWEL LOCATIONS
4. ALL DIMENSION ARE IN MM UNLESS OTHERWISE NOTED.

JOINT DETAIL NOTES:

1. FACE OF PANELS TO MATCH FACE OF MSE WALL PANELS AT INTERFACES

PRESTRESSING NOTES:

CONCRETE STRENGTH : $f'c$ = 34 MPa AT 28 DAYS
$f'c$ = 28 MPa AT TIME OF PRESTRESSING

PRESTRESSING STEEL : GRADE 270 LOW RELAXATION STRAND
Pf= FORCE REQUIRED AT CENTER OF SPAN AFTER ALL LOSSES
= 761 KN PER PANEL

2 DELETED NOT APPROVED FOR CONSTRUCTION NOTE

LICENSED PROFESSIONAL ENGINEER
No. 98-324447-2202
JOHN M. TERRY
3/12/98
STATE OF UTAH

NO.	DATE	DESCRIPTION
1	11/11/97	RELEASE FOR GEOFOAM WALL ONLY.
2	2/23/98	APPROVED FOR CONSTRUCTION AT BRIDGES

APPROVED FOR CONSTRUCTION

UTAH DEPARTMENT OF TRANSPORTATION

SVERDRUP/DE LEUW

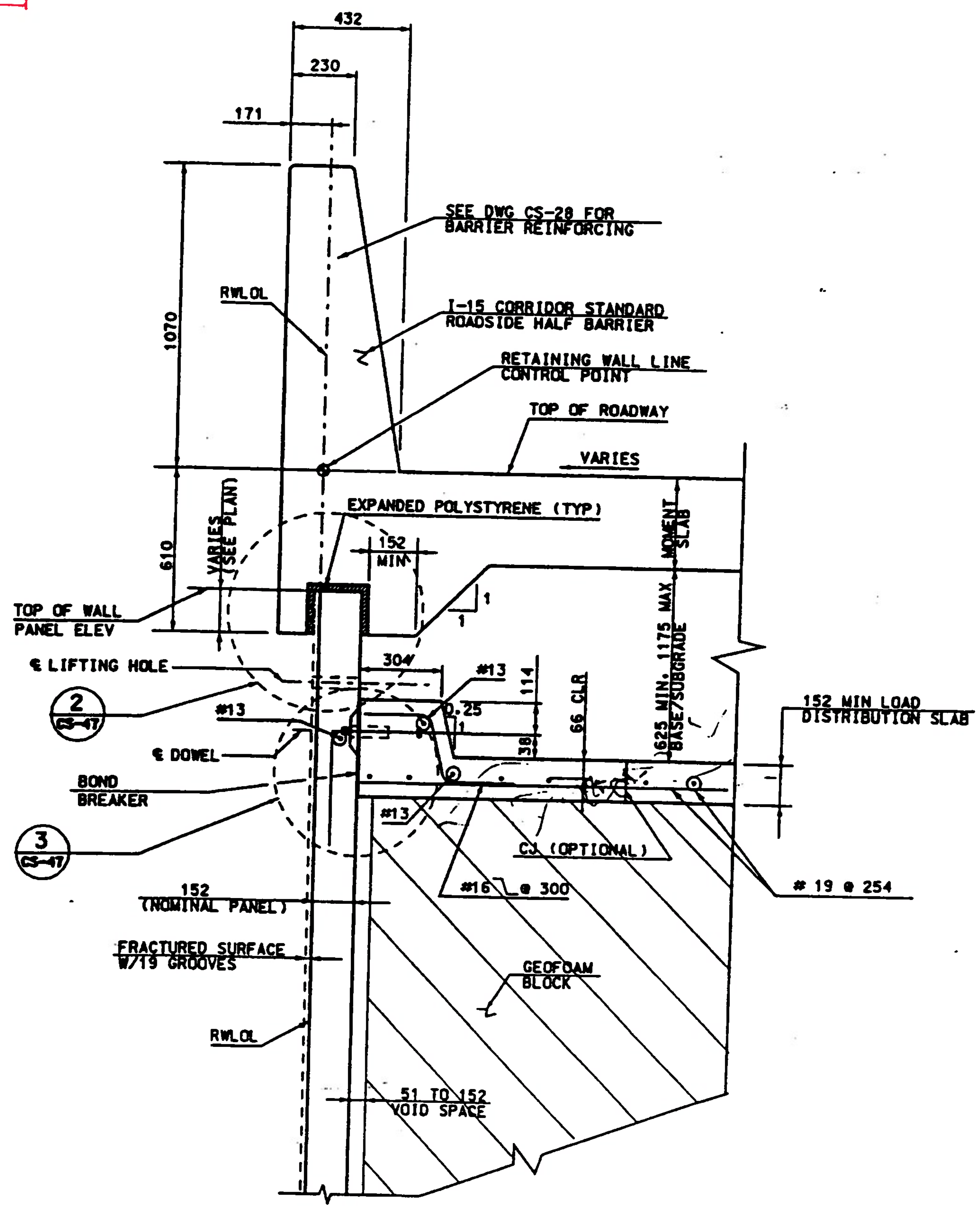
I-15 CORRIDOR RECONSTRUCTION
GEOFOAM WALL PANEL DETAILS
CORRIDOR STANDARD PLAN

PROJECT NUMBER #SP-15-7(135)296

SALT LAKE
COUNTY
DWG. NO.
CS-44

SHT. ___ OF ___

HALEY & ALDRICH, INC.
10 JULY 2009
3 OF 3



LOAD DISTRIBUTION SLAB RESTRAINT

SECTION A
NTS

LICENSED PROFESSIONAL ENGINEER
No. 96-324447-2202
JOHN M. TERRY
7/14/99
STATE OF UTAH

WASATCH CONSTRUCTORS
APR 13 1998
RELEASED FOR CONSTRUCTION

NOTES:

1. LOAD DISTRIBUTION SLAB DESIGNED FOR HS 20 LOADING.
2. WHEEL LOADS ARE NOT PERMITTED WITHIN 1500 OF FREE EDGE OF LOAD DISTRIBUTION SLAB PRIOR TO PLACING PCCP SLAB.
3. LIFTING HOLES TO BE DRY PACKED.
4. ALL DIMENSION ARE IN MM UNLESS OTHERWISE NOTED.
5. SEE CS-28 FOR MOMENT SLAB REINFORCING.
6. FOR DETAILS UNDER APPROACH SLABS AND BRIDGES SEE CS-49
7. TRANSVERSE CONSTRUCTION JOINTS ARE ALLOWED IN THE LOAD DISTRIBUTION SLAB AT THE OPTION OF THE CONTRACTOR.
8. ALL CRACKS OVER 0.5 MM SHALL BE SEALED WITH A HIGH MOLECULAR WEIGHT METHYL METHACRYLATE SEALANT (TRANSPO T-70 OR EQUAL.
9. CONCRETE SHALL BE CLASS AA(AE) EXCEPT AS MODIFIED FOR f'c=27.5MPa (4,000 PSI). NO CONSTRUCTION LOADS SHALL BE PERMITTED ON THE LOAD DISTRIBUTION SLAB UNTIL AFTER 7 DAYS OF CURING.

3

APPROVED FOR CONSTRUCTION

NO.	DATE	DESCRIPTION
1	11/11/97	RELEASE FOR GEOFOAM WALL ONLY.
2	2/25/98	APPROVED FOR CONSTRUCTION AT BRIDGES
3	4/10/98	ADD NOTES

UTAH DEPARTMENT OF TRANSPORTATION
SVERDRUP/DE LEUW

I-15 CORRIDOR RECONSTRUCTION
GEOFOAM WALL RESTRAINT DETAILS
CORRIDOR STANDARD PLAN
PROJECT NUMBER #SP-15-7(135)296
SALT LAKE
COUNTY
DWG. NO.
CS-48

Haley & Aldrich
75 Washington Avenue
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Portland, ME 04101-2617

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HaleyAldrich.com

HALEY& ALDRICH

MEMORANDUM

14 August 2009
File No. 35524-010

TO: TY Lin International
Rick Hebert, P.E., Josh Olund, Ph.D.

C: Maine Department of Transportation
Laura Krusinski, P.E.

FROM: Haley & Aldrich, Inc.
Bryan Steinert, P.E., Wayne Chadbourne, P.E., James Weaver, P.E.
BCS

SUBJECT: Geofoam Panel Wall Design Recommendations – North Approach
Proposed Route 100/26 Bridge Replacement
Falmouth, Maine
PIN 15094.00

Reinforced concrete walls will be required to cover and protect the geofoam-cored, north approach embankment and to contain an approximately 5-ft thick layer of granular fill on top of the approach embankment (i.e., pavement section). The walls selected for the project and discussed herein are vertical, precast concrete panel walls (panel walls). The project will also include conventional mechanically stabilized earth (MSE) walls where conventional earthfill is used to construct the approach embankments (i.e, north of approximate Sta. 120+75). The panel walls will be designed and detailed on the contract drawings whereas the MSE walls will be vendor-designed.

This memorandum presents our recommendations for the design of the panel walls. Panel wall heights will range from approximately 10 to 30 ft (height measured above finish grades). The basic configuration of the walls consist of precast concrete panels (nominal width of 8 ft) supported on a continuous "keyed" wall footing. The wall will be restrained using a reinforced concrete distribution/moment slab (concrete slab) constructed on top of the geofoam embankment, at a depth of approximately 5 ft below finish roadway grades. Standard weight earth fill will be placed above the concrete slab. The connection between the panel wall and the concrete slab will need to be designed to resist lateral loads (i.e., from static soil, surcharge (traffic), seismic soil and guardrail impact forces), along with vertical soil and surcharge loads. Along a portion of the approach alignment the concrete slab will span between parallel panel walls across the full embankment width. In areas where the geofoam does not cross the entire roadway width, the concrete slab will also not span the full roadway width. In either case the concrete slab will serve as both a distribution slab to protect the underlying geofoam from traffic loading, and as a restraint for the panel wall (using friction along the soil/concrete interface above and below the slab).

Cross sections and typical details of the geofoam embankment, panel walls, wall footings, etc. are being developed and delivered to TY Lin under separate cover.

Wall and Foundation Design Recommendations

- The panel wall foundations should consist of continuous wall footings designed to bear at a minimum depth of 4.5 ft below finish grade. Wall footings should be at least 3 ft wide and designed for a maximum allowable bearing pressure of 2,000 pounds per square foot (psf).
- The precast panels should be designed using a tongue and groove pattern along the vertical edges.
- The portion of the panels extending above the concrete distribution/restraint slab should be designed for lateral earth pressures using an equivalent fluid unit weight of 36 pounds per cubic foot (pcf) which assumes an active earth pressure coefficient of 0.3 and a soil unit weight of 120 pcf. This recommendation assumes the granular soil above the concrete slab will be drained (roadway underdrain system) and no unbalanced hydrostatic pressures will develop behind the wall.
- The wall should also be designed for a live load surcharge equivalent to 2 ft of earthfill (equivalent to an area load of 250 psf; in accordance with AASHTO LRFD Section A.11.1). A uniform horizontal load of 125 psf should be applied to the wall above the concrete slab to account for the live load surcharge.
- In accordance with AASHTO LRFD Section A.11.1, a uniform horizontal load of 55 psf should be applied to the wall above the concrete slab to account for seismic soil loading.
- The portion of the wall adjacent to the geofoam should be designed for a uniform horizontal load of 80 psf to account for lateral elastic strains (Poisson effect) from the weight of the pavement section and earthfill placed above the concrete distribution slab.
- It is expected that elastic strain of the geofoam embankment from the weight of the pavement section and earthfill placed above the concrete distribution slab will be on the order of ¼ to ¾ in. depending on the thickness of the underlying geofoam. The maximum strain is expected to occur near the middle of the embankment with little or no strain occurring at the edge of the embankment adjacent to the panel walls. The connection between the concrete distribution slab and the panel wall will have to be designed for a vertical load associated with the weight of the overlying granular fill and live load surcharge. At this time we recommend that the connection and the panel wall be designed for a vertical load of approximately 6,000 pounds per linear foot. Please note that we are currently conducting finite element analyses to further (and more accurately) evaluate how much the overburden soils will load the concrete slab (and the wall).
- The concrete distribution slab spanning between the panel walls (east and west sides of the roadway alignment) should be designed to resist the tension forces that will develop due to the horizontal and vertical loads as described above.
- In areas where the concrete distribution slab will not span between panel walls (geofoam does not extend across the entire roadway width) the concrete slab should be designed to resist tension loads from the panel wall (resisting horizontal earthfill, traffic surcharge, earthquake, guardrail impact and Poisson effect loads) by friction at the interface between the concrete slab and the granular soils above and below the slab. We recommend that an interface friction angle equal to 30 degrees be used to determine the frictional resistance. Once TY Lin determines the design tension force needed in the concrete slab, we will then determine the minimum length of the slab using the above friction value. Once the minimal length of the concrete slab has been determined

the interface frictional resistance between geofoam blocks will be checked assuming an interface friction angle of 20 degrees.

We have summarized the design lateral loading diagrams on the attached sketch. Please note that all loads shown on the sketch are unfactored. Appropriate load factors should be applied when determining the final structural design of the panels and concrete slab (and the design tension force in the concrete slab).

Other Design and Coordination Issues

Other issues that will be resolved as the panel wall design and details are developed include:

- Drainage of the granular soils located above the concrete distribution slab. At this time we anticipate that we will extend the proposed roadway underdrain system up to the back of the north approach abutment substructure. (Haley & Aldrich lead)
- Develop a drainage detail for the soils located between the abutment panel wall (wall 5) and the stub abutment substructure. (Haley & Aldrich lead)
- Develop details for a cast-in-place (or other) joint at the interface between the abutment panel wall (wall 5) and the panel walls along the east and west sides of the roadway (walls 4 and 3, respectively). (TY Lin lead)
- Develop details for the connection between the temporary steel sheeting and the panel walls on the west side of abutment No. 2. (TY Lin lead)
- Develop details for connection of the concrete distribution slab and the temporary steel sheeting on the west side of abutment No. 2. (TY Lin lead)
- Develop structural details for the concrete slab and vertical panel wall. (TY Lin lead)
- Evaluate and determine guardrail impact force per the requirements of AASHTO LRFD Section 11.10.10.2. (TY Lin lead)

It is anticipated that there will be continued interaction and refinement of the panel wall design and details, and that some of these recommendations will be modified. Please do not hesitate to contact us if you have any questions about these comments and recommendations.

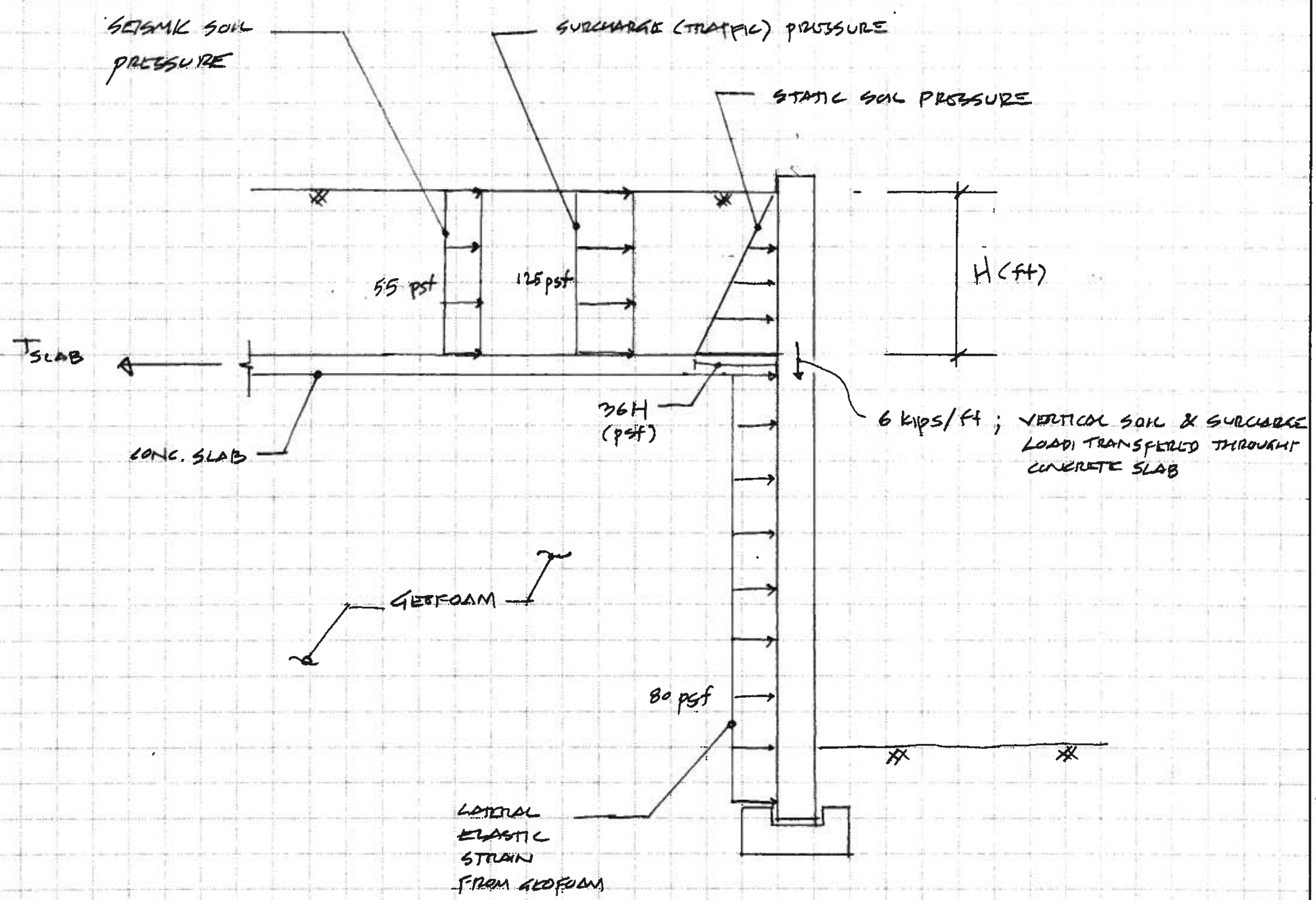
Attachments:
Sketch – Recommended Lateral Loading Diagram – North Approach

G:\PROJECTS\35524 - Presumpscot River Bridge\010\2009-0814-jwwwac-latloadnorthabut-f.doc

HALEY& ALDRICH

Calculations

Client	MEDOT / TYLIN	File No.	35524-010
Project	100/26 BRIDGE, FALMOUTH	Sheet	1 of 1
Subject	RECOMMENDED LATERAL LOADING DIAGRAM - N. ABUT	Date	14 AUG 07
		Computed By	MAC
		Checked By	BCS



NOTE: 1. ALL LOADS SHOWN ARE UNFACTORED. APPROPRIATE LOAD FACTORS SHOULD BE APPLIED WHEN DETERMINING THE FINAL STRUCTURAL DESIGN

2. IMPACT GUARDRAIL LOAD NOT SHOWN BUT SHOULD BE INCLUDED

3. 6 kips/ft VERTICAL LOAD ON WALL SHOULD BE CONSIDERED PRELIMINARY ; LOAD WILL BE REVISED UPON COMPLETION OF FINITE ELEMENT ANALYSES.

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HALEY& ALDRICH

MEMORANDUM

8 September 2009
File No. 35524-010

TO: TY Lin International
Rick Hebert, P.E., Josh Olund, Ph.D.

C: Maine Department of Transportation
Laura Krusinski, P.E.

FROM: Haley & Aldrich, Inc.
Bryan C. Steinert, P.E., James W. Weaver, P.E., Wayne A. Chadbourne, P.E.
BCS

SUBJECT: Geofoam Embankment Vertical Deformation
Proposed Replacement Bridge over Presumpscot River and MCRR
Routes 26/100, Falmouth, Maine
PIN 15094.00

This memorandum provides our recommendation for the grade of geofoam to be used to construct the north and south approach embankments for the subject project and also summarizes estimates of anticipated vertical embankment deformation as it relates to total and post-construction ground surface settlement. This was discussed with you at our team coordination meeting on 19 August 2009.

The total vertical deformation of the approach embankments will consist of 1.) recompression of marine clay foundation soils, 2.) elastic compression of the geofoam blocks and 3.) long-term (creep) compression of the geofoam blocks. We have completed an evaluation of vertical embankment deformation as discussed herein. This memorandum focuses on the elastic and long-term compression of the geofoam blocks at the north approach. Please note that by using lightweight geofoam fill (approximately 2 pcf) in the embankment design and since the contractor will be required to over-excavate approximately 3 ft of fill within the embankment footprint prior to geofoam placement, the anticipated amount of settlement due to recompression of the marine clay at the north approach will be limited to less than ½ in.

Expanded Polystyrene (EPS) Geofoam

The American Society for Testing and Materials (ASTM) standard D6817-07 defines seven different grades of Rigid Cellular Polystyrene (RCPS) geofoam based on physical property requirements: EPS12, EPS15, EPS19, EPS22, EPS29, EPS39 and EPS46. A summary of the physical properties of select grades of geofoam are provided below in accordance with ASTM.

Geofoam Grade	Minimum Density (pcf)	Compressive Resistance at 1 Percent Deformation (psi)	Elastic Modulus (psi)
EPS19	1.15	5.8	580
EPS22	1.35	7.3	730
EPS29	1.80	10.9	1,090
EPS39	2.40	15.0	1,500

Similarly, the Federal Highway Administration (FHWA) and the American Association of State Highway Transportation Officials (AASHTO) also define several different grades of geofoam based on physical properties: EPS40, EPS50, EPS70 and EPS100. A summary of the physical properties of different grades of geofoam are provided below in accordance with FHWA/AASHTO material designations.

Geofoam Grade	Minimum Density (pcf)	Compressive Resistance at 1 Percent Deformation (psi)	Elastic Modulus (psi)
EPS40	1.00	5.8	580
EPS50	1.25	7.2	725
EPS70	1.50	10.2	1,015
EPS100	2.00	14.5	1,450

The summary of physical properties for various grades of geofoam provided above indicate that there is no direct correlation between ASTM and FHWA/AASHTO material designations based on physical properties. Therefore, as discussed herein, geofoam material designations will reference ASTM D6817-07.

I-15 Reconstruction – Salt Lake City, Utah

Reports published documenting the instrumentation and performances of multiple geofoam fill embankments for the reconstruction of Interstate 15 (I-15) in Salt Lake City, Utah were reviewed as they relate to vertical deformations. It is our understanding that the I-15 reconstruction project is the only source of documented long-term performance of geofoam fill embankments based on instrumentation data.

Based on our review, the I-15 geofoam embankments were designed to limit elastic compression of the geofoam blocks to approximately 1 percent of the embankment height (measured during construction) and post-construction (creep) deformations to magnitudes corresponding to 1 percent strain (elastic) based on using a product which met the requirements of EPS19. Instrumentation installed to monitor the performance of the embankments confirmed elastic strains on the order of 1 percent. In addition, the I-15 project also considered limiting the "working" stress applied to the geofoam to 40 percent of the average compressive strength at 10 percent strain.

Based on the magnitude of predicted and measured creep deformations from the I-15 recommendations, additional guidance was sought from FHWA on the design of geofoam fill embankments for the subject project. Specifically, Mr. Silas Nichols, P.E. was contacted to discuss FHWA experience with long-term (creep) deformations of geofoam fill embankments. Taking into consideration the results of I-15 monitoring and on-going research by FHWA, Mr. Nichols suggested that creep related deformations would be negligible provided the embankment is designed to limit elastic strain in the geofoam to less than 1 percent.

Engineering Evaluations

As discussed above, the geofoam blocks used to construct the north and south approach embankments will experience elastic compression under the weight of overlying embankment fill, pavement section and related surcharge loads. Furthermore, creep deformation can be neglected if the elastic strain within the geofoam mass does not exceed 1 percent. In both cases, the magnitude of vertical deformation (elastic and creep) is related to the elastic modulus (stiffness) of a particular grade of geofoam.

For the purposes of this evaluation, a uniform vertical load equal to 1,000 pounds per square foot (psf) was applied to the top of the geofoam. This load includes the dead load of approximately 6 ft of normal weight earthfill overlying the geofoam and an assumed live load surcharge equal to 250 psf. Based on the applied load, elastic stress and strain were calculated for grade EPS19, EPS22, EPS29 and EPS39 geofoam in accordance with the methodology outlined in NCHRP Report 529 (Reference 3). The calculated elastic strain for each grade of geofoam is summarized below.

Geofoam Grade	Elastic Strain (percent)
EPS19	0.95 to 1.20
EPS22	0.75 to 0.95
EPS29	0.51 to 0.64
EPS39	0.37 to 0.46

The range of calculated elastic strain within the geofoam provided for multiple geofoam grades, as summarized above, indicate that creep deformation (on the order of 3 to 4 in.) would be anticipated if EPS19 grade geofoam (and possibly EPS22 grade geofoam) was used (because elastic strains are greater than or equal to 1 percent). Therefore, based on the documented performance and design recommendations from the I-15 project, our discussions with Mr. Nichols, and the calculated elastic strains shown above, we recommend that a material with the minimum physical properties of EPS29 be used to construct the north and south approach embankments for the subject project in order to minimize post-construction creep deformation.

Please recall that recommendations were provided based on the performance of the I-15 embankments that suggested limiting the "working" stress applied to the geofoam to 40 percent of the average compressive strength at 10 percent strain. The 1,000 psf loading represents only 24 percent of the reported 10.9 psi compressive strength (see ASTM D6817-07).

Elastic compression of the geofoam blocks was calculated along the length of the north approach embankment based on the physical properties of EPS29. We are currently determining the geofoam limits for the south approach and will provide estimates of elastic compression along the south approach in a separate memorandum. A summary of the anticipated marine clay recompression, elastic geofoam compression and long term (creep) compression of the geofoam along the north approach is provided below.

Station (ft)	Approximate Geofoam Thickness (ft)	Recompression of Marine Clay Soils (in.)	Elastic Compression (in.)	Long Term (Creep) Compression (in.)
118+74 to 118+90	16	< ½	< 1	negligible
118+90 to 119+00	27	< ½	1-¾ to 2-¼	negligible
119+00 to 119+50	20	< ½	1-¼ to 1-¾	negligible
119+50 to 120+25	9	< ½	¾ to 1	negligible
120+25 to 120+75	2	< ½	0 to ¼	negligible

The elastic compression of the geofoam blocks will generally occur during embankment construction, prior to roadway paving (i.e., construction of the concrete distribution slab and placement of embankment fill and pavement base/subbase materials. Since the elastic compression of the geofoam will occur prior to paving, we do not anticipate elastic deformations of the geofoam will impact roadway/ pavement performance. Based on the measured compressibility characteristics and stress history of the marine clay deposit beneath the north approach, we anticipate that post-construction embankment settlement due to recompression of the marine clay will be less than ½ in. We do not anticipate that post-construction creep-related deformations will occur within the geofoam embankment.

It should be noted that the thickness of the geofoam also varies transverse to the project baseline; therefore there may be some differential deformation within the embankment (deformations will take place during embankment construction). Furthermore, the geofoam in front of and below the pile supported stub abutment (Abutment 2), as well as the geofoam shielded by the bridge approach slab will not be subjected to the full design loading condition and will likely deform less than the values shown in the above table.

We trust these comments and recommendations are suitable for your present needs. Please do not hesitate to contact us if you have any questions regarding this matter.

References:

1. American Society of Testing and Materials Designation D 6817-07, "Standard Specification for Rigid Cellular Polystyrene Geofoam.
2. Farnsworth, et. al, (2008). "Rapid Construction and Settlement Behavior of Embankment Systems of Soft Foundation Soils, ASCE Journal of Geotechnical and Geoenvironmental Engineering, March, 2008.
3. National Cooperative Highway Research Program (NCHRP) Report 529, "Guideline and Recommended Standard for Geofoam Applications in Highway Embankments," Transportation Research Board, Washington D.C., 2004.
4. National Cooperative Highway Research Program (NCHRP) Web Document 65 (Project 94-11), "Geofoam Applications in the Design and Construction of Highway Embankments," Transportation Research Board, Washington D.C., July 2004.
5. Nichols, Silas, Federal Highway Administration. Telephone communication, September 2009.