GEOTECHNICAL DESIGN REPORT
PENOBSCOT RIVER BRIDGE
MAINEDOT WIN 16705.00
HOWLAND/ENFIELD, MAINE

PREPARED FOR:
T.Y. Lin International
Falmouth, Maine

PREPARED BY:
GZA GeoEnvironmental, Inc.
Portland, Maine

October 2013
File No. 09.0025796.00

GZA
GeoEnvironmental, Inc.

Engineers and Scientists

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MaineDOT WIN 16705.00

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Re: Geotechnical Design Report
Penobscot River Bridge, Route 6 and Route 155 over the Penobscot River
Howland-Enfield, Maine

Dear Norm:

GZA GeoEnvironmental, Inc. (GZA) is pleased to provide you with this Draft Geotechnical Design Report prepared for the Penobscot River Bridge project. Our services were provided in accordance with our executed contract dated August 5, 2013, and the attached Limitations included in **Appendix A**.

It has been a pleasure serving you on this project. If you have any questions regarding the report, or if we can provide further assistance, please do not hesitate to contact the undersigned.

Very truly yours,

GZA GEOENVIRONMENTAL, INC.

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1.0 INTRODUCTION

This report presents the results of GZA's geotechnical evaluation for the proposed replacement of Maine Department of Transportation (MaineDOT) Penobscot River Bridge, Route 6 and Route 155, over the Penobscot River. The bridge connects the towns of Howland and Enfield, Maine. Our services were provided in accordance with our executed contract dated August 5, 2013, and the attached Limitations included in **Appendix A**.

GZA is providing geotechnical engineering services as a Subconsultant to T.Y. Lin International, Inc., who is under contract with the State of Maine, MaineDOT for design of the proposed bridge.

1.1 OBJECTIVES AND SCOPE OF SERVICES

The objectives of our work were to evaluate subsurface conditions and to provide geotechnical engineering recommendations for the proposed Penobscot River Bridge replacement. To meet these objectives, GZA completed the following Scope of Services:

- Conducted a site visit to observe surficial conditions; and reviewed existing bridge plans, and mapped surficial and bedrock geology of the site;
- Coordinated and observed a subsurface exploration program consisting of seven test borings;
- Conducted a laboratory testing program to evaluate engineering properties of the site soils and bedrock;
- Conducted geotechnical engineering analyses to evaluate foundations for the new bridge;
- Developed geotechnical engineering recommendations including foundation alternatives and foundation design recommendations for the preferred foundation type; and
- Prepared this report summarizing our findings and design recommendations.

1.2 BACKGROUND

The Penobscot River Bridge carries State Routes 6 and 155 over the Penobscot River from Howland, west of the river, to Enfield, east of the river, as shown on the *Locus Plan*, **Figure 1**. The existing bridge was originally constructed in 1896 and was rebuilt in 1934. The substructures were widened and the superstructure replaced in 1941. The current structure consists of an approximately 900-foot long, five-span, steel through-truss with a concrete deck. The 1941 bridge replacement plans indicated the original masonry abutments were reused to support the new bridge. The original abutment footings, which were founded on steeply sloping bedrock, were underpinned. Concrete footing extensions were constructed at the ends of the abutments and were founded on bedrock. Based on the 1896 plans, the four piers are believed to be supported on stone masonry placed directly on native soil within a timber form/cofferdam. In 1941, the original stone piers were widened at both ends with concrete. Riprap is present in front of each abutment and around each of the piers. The existing bridge location is shown on the *Boring Location Plans*, **Figures 2A and 2B**,

MaineDOT intends to replace the existing bridge with a new, approximately 940-foot-long, four-span bridge with stub abutments and three solid-shaft piers. The alignment of the replacement bridge is approximately 100 feet downstream from the current bridge location. The proposed baseline and bridge limits are shown on **Figures 2A** and **2B**.

2.0 SUBSURFACE EXPLORATIONS

2.1 PREVIOUS MAINEDOT SUBSURFACE EXPLORATIONS

MaineDOT completed a subsurface exploration program in 2010 consisting of five test borings (BB-HEPR-101 through BB-HEPR-105) drilled through the existing bridge and approaches. Subsurface conditions encountered at the test borings located at the bridge abutments consisted of sand and gravel fill overlying a silt and sand deposit (Glaciomarine), silty sand and gravel (Glacial Till), and Bedrock. Subsurface conditions at the existing pier locations consisted of sand and silt deposits with varying amounts of gravel (Alluvium), overlying sandy silt, sand and gravel (Glacial Till), and Bedrock. Logs of each boring are included in **Appendix B**.

2.2 GZA SUBSURFACE EXPLORATION

GZA completed a subsurface investigation program consisting of seven (7) test borings (BB-HEPR-201 through BB-HEPR-207). Four of the borings were completed in the river using a barge-mounted skid rig. The two test borings at the proposed eastern abutment were drilled by winching the skid rig off the barge onto the riverbank. The remaining test boring, at the proposed western abutment location, was drilled using a truck-mounted drill rig. The as-drilled boring locations and ground surface elevations were surveyed by MaineDOT. For the river borings the barge deck elevations were surveyed and the distance from the top of the barge deck to ground surface at each boring location was measured by GZA and used to assess the riverbed elevation. The boring locations are shown on **Figures 2A** and **2B.**

Test borings were drilled to depths of approximately 11.5 to 67 feet below ground surface. Six (6) of the seven (7) borings were terminated after coring approximately 6 to 24 feet into bedrock. Maine Test Boring of Hermon, Maine provided drilling services and coordinated utility clearance for the project. Drilling was completed between August 19, 2013 and August 29, 2013. GZA personnel monitored the drilling work and prepared logs of each boring that are included in **Appendix B**.

Test borings were drilled using a combination of 3- and 4-inch spun casing and drive and wash drilling techniques. Standard penetration testing (SPT) and split-spoon sampling were performed at 5-foot typical intervals using a 24-inch-long, 1-3/8-inch inside diameter sampler. Bedrock cores were obtained using a 2-inch nominal diameter, NQ2, core barrel.

3.0 LABORATORY TESTING

MaineDOT completed a laboratory soil testing program in conjunction with their 2010 Geotechnical Data Report. The program included 20 gradation analyses, one gradation analysis including hydrometer, and one set of Atterberg Limits. Results of the testing are included in **Appendix C.**

GZA retained Thielsch Engineering's Geotechnical Laboratory in Cranston, Rhode Island to complete a soil and bedrock testing program to assess the gradation and engineering characteristics of the soil and strength of the bedrock. The testing program consisted of six gradation analysis/AASHTO Classification/Frost Classification assessments on soil, and two unconfined compression strength tests with strain measurements on bedrock core samples. Results of the testing are included in **Appendix C**.

4.0 SUBSURFACE CONDITIONS

4.1 SURFICIAL AND BEDROCK GEOLOGY

Surficial geologic units mapped in the Penobscot River Bridge area include Sand, Gravel and Silt Stream Alluvium, Marine Sand and Silty Clay (Presumpscot), and Silty Sand and Gravel Glacial Till overlying bedrock. Bedrock at the site is mapped as the Vassalboro Formation bedrock unit. The Vassalboro Formation is described as beds of fine to medium grained, feldspathic wacke, interbeds of dark gray phyllite, minor black carbonaceous phyllite and feldspathic coarse sand to granule conglomerate.

4.2 SUBSURFACE SOIL PROFILE

Four soil units were encountered in the test borings overlying bedrock: Fill, Sand and Silt, Glacial Till and Weathered/Decomposed Bedrock. The thicknesses and generalized description are presented in the following table, in descending order from existing ground surface. Detailed descriptions of the materials encountered at specific locations are provided in the boring logs in **Appendix B**.

Soil Unit	Approx. Encountered Thickness (ft)	Generalized Description
Fill	4 to 20	Medium dense to very dense, brown to gray, fine to medium SAND, some to little Gravel, some to trace Silt (SM). • MaineDOT Frost Classification = II • Encountered in borings BB-HEPR-201 and -207
Sand and Silt (Alluvium)	4.5 to 34	Medium dense to very dense, brown to gray, gravelly, fine to coarse SAND, little to trace Silt (SM, GM). • MaineDOT Frost Classification = 0, II • Encountered in borings BB-HEPR-202, -203, -204, -205, -206
Glacial Till	5 to 25	Very dense, gray, fine to medium SAND, with varying amounts of Gravel and Silt (SM, ML). • MaineDOT Frost Classification = II • Encountered in borings BB-HEPR-203, -204, -205
Weathered/ Decomposed Bedrock	7 to 11	Dense, brown and gray, GRAVEL, little fine to medium Sand, trace Silt. (SM). • MaineDOT Frost Classification = II • Encountered in borings BB-HEPR-206 and -207
Top of Bedrock Elevation	Encountered Top of Rock: Approx. El. 67 to El. 134	

4.2.1 Bedrock

Bedrock was cored in six of the seven test borings and was classified as Phyllite. The Phyllite was described as hard, fresh to slightly weathered, fine grained, and gray/white. The primary joint set was closely to moderately spaced, low to high angle, planar to undulating, rough to smooth, fresh to discolored, and tight to wide. A secondary joint set was occasionally noted and was closely to moderately spaced, horizontal to moderately dipping, planar to stepped, rough to smooth, discolored, and partially open to open. Occasional silt and sand deposits were noted on joint surfaces. The Rock Quality Designation (RQD) ranged from 0 to 87 percent, with an average RQD of 43 percent.

Two laboratory unconfined compressive strength and secant modulus tests were conducted on bedrock core samples. The test results are included in **Appendix C** and yielded an average unconfined compressive strength of 4.3 kips per square inch (ksi) and an average Young's modulus of 3,900 ksi.

4.2.2 Groundwater

Groundwater levels were not discernible during the recent abutment test borings. The test borings were drilled using drive-and-wash techniques, which introduce large volumes of water into the borehole during drilling. As a result, stabilized groundwater levels were not determined at either proposed abutment location. At the time of drilling the river level was approximately El. 128.

Groundwater was observed at the western abutment during the 2010 test boring program at approximately El. 138.5. Groundwater was not observed at the eastern abutment in 2010.

The groundwater observations were made at the times and under the conditions stated in the borings logs. Fluctuations in groundwater and river levels will occur due to variations in seasonal influences, precipitation amounts, and other factors. Consequently, water levels during and after construction are likely to vary from those encountered at the time the observations were made.

5.0 ENGINEERING EVALUATIONS

5.1 GENERAL

GZA conducted geotechnical engineering evaluations in accordance with 2012 AASHTO LRFD Bridge Design Specifications, 6th Edition (herein referred to as LRFD) and the Maine Department of Transportation Bridge Design Guide, 2003 Edition (MaineDOT BDG). The sections that follow describe the evaluations made and the geotechnical basis for evaluation of each element.

5.2 APPROACH EMBANKMENTS

Approach fills are proposed up to 28 feet above existing grades immediately behind the abutments. The maximum side slope angles are anticipated to be 2 horizontal to 1 vertical (2H:1V), or flatter, with loam and seed surface treatments. Steeper slopes may be utilized in conjunction with riprap scour protection along the riverbank. The new roadway alignment will maintain existing grade on the Enfield side and be approximately 4 feet above existing grade on the Howland side. The abutments will consist of stub abutments with new fill slopes beneath and in front of the abutments.

Subsurface conditions at both approaches include granular fill and dense glacial till overlying bedrock. These materials are expected to compress elastically as the new embankment fill is placed. Consequently, post-construction settlements are anticipated to be negligible.

The embankments will be constructed per MaineDOT standard specifications and details using engineered fill placed over fill and glacial till. In our experience, conventional earthfill embankments constructed over relatively dense overburden soils meet the minimum required safety factors for global stability.

5.3 EVALUATION OF FOUNDATION TYPES

5.3.1 Abutment Foundation Alternatives

Selection of the preferred foundation types is influenced by the nature and depth of overburden at the site. Both abutments will be constructed with a 14- to 20-foot-thick fill, alluvium and weathered rock layer beneath the bottom of the abutment. Both spread footings bearing on bedrock, and piles are feasible foundation types at the abutments. It is anticipated that a cofferdam and seal would be required to construct the footings since the bedrock level extends below the river level. The use of a higher-elevation stub abutment and piles will eliminate the need for a full cofferdam and seal system, therefore driven piles are the preferred foundation system at the abutments.

5.3.2 River Pier Foundation Alternatives

Dewatering of the river piers is anticipated to be achieved using cofferdams with tremie seals. The preliminary profile indicates the bottoms of seal elevations are at approximately El. 97, and the top of bedrock varies from as high as approximately El. 97 at Pier 1, to El. 67 to El. 70 at Piers 2 and 3. Since the bottom of tremie is roughly at the top of bedrock level at Pier 1, pile foundations would have no embedment below the tremie. Therefore, spread footing foundations bearing on sound, intact bedrock are the preferred foundation type at Pier 1.

At Piers 2 and 3 the pile embedments are anticipated to range from approximately 25 to 30 feet below the tremie base. Excavation to this depth is technically feasible. However, footings bearing on rock would be considerably more difficult to construct than pile foundations. Consequently, driven piles are considered the preferred foundation alternative at Piers 2 and 3.

5.4 SEISMIC DESIGN CONSIDERATIONS

The new abutments and most pier foundations (except for Pier 1) will bear on steel HP-section piles driven to bearing in dense soils or on bedrock.

The subsurface profile for seismic design includes the proposed approach fills (including backfill behind and beneath abutments) and existing glacial till and bedrock. Seismic site class was determined in general accordance with LRFD Table C3.10.3.1 using the average SPT N-value from the soil materials encountered in the borings. LRFD allows the assumption that rock within the upper 100 feet of the profile has an N-value equal to 100. However, the SPT N-value used to determine the site class was conservatively evaluated by including only the blow counts and thickness of soil above the rock, reducing the effective thickness of the profile and neglecting the bedrock in the upper 100 feet. On this basis, the SPT N-value fell between 15 and 50 blows per foot, and Abutments 1 and 2; and Piers 2 and 3 were assigned to Site Class D. Supporting calculations are provided in **Appendix E**.

The test boring data indicate that the fill, alluvium, and glacial till encountered at the site are sufficiently dense that the potential for liquefaction is very low.

Determination of the seismic Site Class for bedrock conditions is typically based on the shear wave velocity approach in accordance with LRFD Table C3.10.3.1-1. At Pier 1, and in the absence of site-specific shear wave velocity data, the Pier 1 should be assigned to Site Class B.

The United States Geological Survey software Seismic Design Parameters Version 2.10 was used to develop seismic parameters for design. Based on the bridge location, and the Site Classes B and D, the recommended AASHTO Response Spectrum for a 7 percent probability of exceedance in 75 years is as follows:

Site Class B - Fpga = 1.0, Fa = 1.0, Fv = 1.0
Data are based on a 0.05 deg grid spacing.

Period	Sa	
(sec)	(g)	
0.0	0.070	As, Site Class B
0.2	0.153	SDs, Site Class B
1.0	0.046	SD1, Site Class B

Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40
Data are based on a 0.05 deg grid spacing.

Period	Sa	
(sec)	(g)	
0.0	0.112	As, Site Class D
0.2	0.245	SDs, Site Class D
1.0	0.112	SD1, Site Class D

T.Y. Lin has indicated that based on a Site Class of D at the abutments, the bridge meets the criteria for design under Seismic Zone 1, per LRFD Section 3.10.6. Per LRFD Section 4.7.4, bridges in Seismic Zone 1 need not be analyzed for seismic loads, but the minimum requirements specified in LRFD Sections 4.7.4.4 and 3.10.9 apply.

5.5 SPREAD FOOTING DESIGN CONSIDERATIONS

Spread footing foundations bearing on sound, intact bedrock are the preferred foundation type at Pier 1.

5.5.1 LRFD Resistance Factors

LRFD factors should be applied to horizontal earth pressure (EH), vertical earth pressure (EV) and earth surcharge (ES) loads using the load factors for permanent loads (γ_p) provided in LRFD Table 3.4.1-2 for strength and extreme limit state design. The resistance factor for global stability of abutments and piers, ϕ, is 0.65.

Recommended LRFD resistance factors for strength limit state design of the spread footing foundation at Pier 1, from LRFD Table 10.5.5.2.2-1, are presented in the following table.

RESISTANCE FACTORS – STRENGTH LIMIT STATE		
Foundation Resistance Type	**Method/Condition**	**Resistance Factor (ϕ)**
Bearing	Footings on Rock	0.45
Sliding	Tremie Concrete on Rock[1]	0.80
Sliding	Cast-in-Place Concrete on Tremie Concrete[1]	0.80

[1] Sliding resistance factor for concrete on rock or concrete is taken as equal to footing on sand.

Resistance factors for service and extreme limit state design (vessel impact, ice, $Q_{500scour}$, debris, and earthquake) should be taken as 1.0, except for uplift resistance of piles. The resistance factor for pile uplift for these conditions, ϕ_{up}, should be taken as 0.80.

5.5.2 Footing Bearing Resistance on Intact Bedrock

Bedrock at Pier 1 underlies medium dense to dense alluvial material. Footings will be founded directly on intact bedrock. Therefore, foundation design is controlled by the engineering properties of the bedrock.

Samples of bedrock collected during GZA's recent subsurface investigation were submitted for laboratory testing for use in determining the in-situ bedrock Rock Mass Rating (RMR). Using bedrock data obtained in test borings BB-HEPR-102 and BB-HEPR-203, GZA developed engineering parameters for the bedrock mass for the proposed footing at Pier 1, which are summarized below:

- RQD = Ranged from 61 to 67
- Average Unconfined Compressive Strength ($\sigma_{u,r}$) = 4.3 ksi
- RMR = 49 (Fair Rock Quality)
- Semi-empirical rock quality constants, m=0.26, s=0.00021 (by interpolation)

It is anticipated that the highly fractured (RQD = 0 to 11) rock encountered near the bedrock surface in borings BB-HEPR-102 and BB-HEPR-203 will be removed during subgrade preparation. Consequently, these data were not included in the analyses.

The RMR-based approach was used to calculate the nominal and factored bearing resistance for spread footings bearing on intact bedrock. Footings designed to bear on intact bedrock should be designed for a recommended nominal bearing resistance, q_n, is 55 kips per square foot (ksf). At the strength limit state, the recommended maximum factored bearing resistance is 25 ksf. To limit settlement, the bearing resistance of 25 ksf is also recommended for service limit state design. Supporting calculations are provided in **Appendix E**.

Pier 1 may be founded on intact bedrock. LRFD Article 10.6.2.4.4 indicates that footings bearing on rock with an RMR-based rock quality of Fair to very good are generally anticipated to experience ½ inch or less of elastic settlement.

5.6 PILE DESIGN CONSIDERATIONS

Steel H-piles are proposed to support the new abutments and Piers 2 and 3. The pile material should consist of ASTM A572, Grade 50 steel.

The axial geotechnical resistance of piles was calculated using the Nordlund method in accordance with LRFD Section 10.7. Supporting calculations are provided in **Appendix E**. The results indicate that the piles will gain support through a combination of side friction in the fill and glacial till and end bearing in glacial till or on bedrock. It is likely that the piles will drive onto or slightly into bedrock to achieve the required end resistance. The side friction distribution was also used as an input in preliminary wave equation analyses to assess the pile drivability. Since the piles will gain support in primarily dense granular soil and on bedrock, there is no reduction for group interaction in axial compression.

By utilizing steel H-piles for support of the abutments and Piers 2 and 3, total and differential settlement will be limited to elastic compression of the piles and should be less than ½ inch.

The estimated pile cap bottom elevations are shown on the preliminary plans prepared by T.Y. Lin; the top of bedrock elevations based on the test borings and the estimated pile embedment lengths are summarized below.

Structure	Pile Cap /Tremie Seal Bottom Elevation (ft)	Estimated Top of Bedrock Elevation (ft)	Estimated Pile Length Below Cap or Tremie Seal (ft)
Abutment 1	139	125	14
Pier 1	97	97	NA – spread footing recommended
Pier 2	97	67	30
Pier 3	97	71	26
Abutment 2	149	130	19

The estimated pile lengths do not include the pile length embedded in the tremie seal or pile cap, batter, or additional length needed for installation or testing.

Piles should be designed at the strength limit state considering the structural resistance of the piles and a resistance factor of 0.50, per LRFD Section 10.7.3.2.3 for hard driving condition; the geotechnical resistance of the piles; and the potential loss of lateral support due to scour at the design flood event. In GZA's experience for end bearing piles on bedrock, the structural resistance or drivability resistance will control the geotechnical static resistance of the pile.

The pile driving criteria are expected to be established based on dynamic pile testing with signal matching analysis. The piles should be driven to a nominal capacity calculated by dividing the maximum factored pile load by a resistance factor of 0.65, per LRFD Table 10.5.5.2.3-1.

GZA considered the potential for downdrag loading for piles supporting the abutments. It is GZA's opinion that any settlement associated with filling at the abutments will occur prior to pile driving. Therefore, downdrag loading should not be included in the pile design.

It is understood that different pile sizes and layouts will be evaluated by T.Y. Lin in order to select the most efficient design. In order to support geotechnical aspects of the design development, GZA evaluated a range of pile sections suitable for support of the replacement bridge. Preliminary wave equation analyses were completed indicating that the factored geotechnical pile resistance for drivability is lower than the factored axial structural capacity, and therefore controls the design.

For the subsurface profiles at this site, GZA ran a selected number of wave equation analyses to assess the nominal geotechnical drivability resistance. The nominal geotechnical drivability resistance was established as the maximum axial capacity that could be achieved using the assumed pile hammer, while meeting MaineDOT preferred driving criteria: driving stresses not exceeding 0.9fy (or 45ksi), and penetration resistance between 3 to 15 blows per inch (preferably approximately 10). The results of those analyses indicate that the piles can reliably be installed to approximately 70 to 85 percent of the factored structural compressive resistance. The table that follows summarizes alternative pile sections for use on the project. Supporting calculations are provided in **Appendix E**.

Pile Section ASTM A572 Grade 50	Structural Resistance Axial Compression		Geotechnical Resistance for Drivability Axial Compression	
	Nominal / Extreme Limit State $P_{r.e.}$ (kips) ($\phi = 1.0$)	Strength Limit State Hard Driving $P_{r.s.}$ (kips) ($\phi_c = 0.50$)	Nominal / Extreme Limit State R_{ndr} (kips) ($\phi = 1.0$)	Strength Limit State Hard Driving R_{ndr} (kips) ($\phi = 0.65$)
HP12x53	775	388	435	282
HP14x73	1070	535	553	359
HP14x89	1305	653	700	455
HP14x117	1720	860	900	585

It is anticipated that scour depth will not extend below the base of the tremie seals. However, if final estimated scour depth is determined to be below the tremie base, the factored structural compressive resistance of the piles should be reduced to account for any unbraced length of pile. Detailed pile design recommendations will be presented under separate cover after the final pile sections and footing layouts are established by T.Y. Lin.

5.7 EVALUATION OF ABUTMENT FOUNDATIONS

5.7.1 Frost Protection

Fill soils are anticipated to be present at the abutments, either as existing fill or imported backfill. Based on the MaineDOT BDG, Section 5.2.1, the Freezing Index for the site is 1900, and with low-moisture content (<10%) soils, the estimated depth of frost penetration is 5.5 feet.

5.7.2 Design Soil Profiles

GZA developed design subsurface profiles for use in evaluating abutment foundations. The profiles are summarized in the following tables.

DESIGN SUBSURFACE PROFILE - ABUTMENT 1				
Strata Designation	Approx. Base El. (ft-NAVD 88)	Unit Weight (pcf)	Representative Φ' (°)	Description
Fill	134	125	32º	Brown, dense to very dense, SAND, some to little Gravel, little to trace Silt
Bedrock	--	--	--	Gray/white, hard, fresh, fine grained, PHYLLITE

DESIGN SUBSURFACE PROFILE - ABUTMENT 2				
Strata Designation	**Approx. Base El. (ft-NAVD 88)**	**Unit Weight (pcf)**	**Representative Φ' (°)**	**Description**
Fill (possible reworked material)	133	125	32º	Olive, medium dense, SAND, some Silt, little Gravel
Weathered/ Decomposed Rock	130	130	34º	Brown to gray, very dense, clayey silt, some GRAVEL, little fine to medium Sand
Bedrock	--	--	--	Gray/white, hard, fresh, fine grained, PHYLLITE

5.7.3 Lateral Earth Pressure

We understand that the proposed abutments and wing walls will be free to rotate at the top. Therefore, the walls should be designed to resist active earth pressures. Passive resistance in front of the abutment footings should be ignored for sliding and overturning evaluations.

Lateral earth pressure evaluations for abutments and wing walls are based on the MaineDOT BDG and are summarized below:

- Battered piles will be used to resist lateral loads. Passive resistance on pile caps should be neglected.
- Imported fill material will be used as backfill behind the walls. The material will be specified as either Granular Borrow Underwater Backfill Material or Granular Borrow (MaineDOT Standard Specifications Section 703.19). Therefore, Soil Type 4 was used to develop earth pressure coefficients in accordance with Table 3-3 of the MaineDOT BDG.
- Live load horizontal surcharge pressures were evaluated in accordance with Table 3-4 of the MaineDOT BDG and LRFD Section 3.11.6.4 (the more stringent applies). The walls for the subject project require a surcharge equivalent to a fill height of 2 feet be used for design. If approach slabs are utilized, a surcharge load should not be applied over the length of the slab.

5.7.4 Lateral Resistance – Abutment Piles

Lateral loads on abutments may be reacted by a combination of bending and the horizontal component of battered piles. Final design of the foundations may be performed to evaluate pile top deflections and bending stresses under the combined loads using L-Pile® or FB-Pier® software. Analyses should take into account pile orientation, including pile batter.

Combined axial and bending stresses should be evaluated to ensure piles are not overstressed. Due to the relatively short abutment pile lengths (approximately 14 to 19 feet), a check should be performed to assess if piles achieve adequate fixity.

Recommended geotechnical parameters for use in lateral pile analyses are provided in the table below. We recommend the analyses be completed assuming a fixed-head pile condition.

FBPIER GEOTECHNICAL PARAMETERS - ABUTMENT 1				
Soil Model	**El. Range (ft-NAVD 88)**	**Unit Weight (pcf)**	**Representative Φ' (°)**	**k (pounds per cubic inch)**
Reese Sand	Bottom of Cap to 134	125	32º	225

FBPIER GEOTECHNICAL PARAMETERS - ABUTMENT 2				
Soil Model	**El. Range (ft-NAVD 88)**	**Unit Weight (pcf)**	**Representative Φ' (°)**	**k (pounds per cubic inch)**
Reese Sand	Bottom of Cap to 130	125	32º	225
Reese Sand	El 130 to 122	130	34º	225

5.8 EVALUATION OF PIER FOUNDATIONS

5.8.1 Design Soil Profiles – Pier 2 and 3 Foundations

GZA evaluated subsurface conditions and developed a representative design soil profile for Pier 2 and 3 evaluations as summarized in the table that follows.

PIER 2 AND 3 PROFILE				
Strata Designation	**Approx. Base El. (ft-NAVD 88)**	**Unit Weight (pcf)**	**Representative Φ' (°) or s_u (psf) for layer**	**Description**
Alluvial Deposit	89	120	34º	Brown to gray, medium to very dense, fine to coarse SAND, some to little Gravel, trace Silt
Glacial Till	67	130	34º	Varying from gray, very dense, fine to medium sandy SILT, some to little Gravel to gray, very dense, fine to coarse SAND, little Silt, trace Gravel
Bedrock	--	--	--	Gray/white, hard, fresh, fine grained, PHYLLITE

5.8.2 Lateral Pile Resistance –Piers 2 and 3

Lateral loads on Piers 2 and 3 may be reacted a combination of bending and the horizontal component of battered piles. Final design of the foundations may be performed to evaluate pile top deflections and bending stresses under the combined loads using L-Pile® or FB-Pier® software. Analyses should take into account pile orientation, including pile batter.

Combined axial and bending stresses should be evaluated to ensure piles are not overstressed. A check should be performed to assess if piles achieve adequate fixity.

Recommended geotechnical parameters for use in lateral pile analyses are provided in the table below. We recommend the analyses be completed assuming a fixed-head pile condition.

FB-PIER GEOTECHNICAL PARAMETERS - PIERS 2 AND 3				
Soil Model	**El. Range (ft-NAVD 88)**	**Unit Weight (pcf)**	**Representative Φ' (°)**	**k (pounds per cubic inch)**
Reese Sand	Bottom of Seal to 89	120	34°	125
Reese Sand	67 to 89	130	34°	125

6.0 RECOMMENDATIONS

6.1 EMBANKMENT DESIGN CONSIDERATIONS

T.Y. Lin will be responsible for selection of scour countermeasures to be employed in front of the new abutments. If riprap slopes are selected, they should be constructed in accordance with MaineDOT Standard Detail 610(03), Plain Riprap Slope at Structures.

Embankment side slopes should be designed with MaineDOT typical slope angles of 2H:1V, and should be provided with loam and seed for permanent erosion protection.

6.2 RECOMMENDATIONS FOR FOUNDATIONS

6.2.1 Abutment and Wingwall Design

- Backfill behind new abutments and wingwalls should consist of MaineDOT 703.19 Granular Borrow for Underwater Backfill, MaineDOT BDG Type 4 soil. Recommended soil properties for Type 4 soils are as follows:
  - Internal Friction Angle of Soil = 32°
  - Soil Total Unit Weight = 125 pcf
  - Coefficient of Active Earth Pressure, K_a = 0.31, assuming $\beta=0°$
  - Coefficient of At-Rest Earth Pressure, K_o = 0.47, assuming $\beta=0°$
- Live load surcharge should be applied as a uniform lateral surcharge pressure using the equivalent fill height (H_{eq}) values developed in accordance with LRFD Section 3.11.6.4. A minimum H_{eq} of 2 feet is recommended.
- Foundation drainage should be provided in accordance with Section 5.4.1.4 of the MaineDOT BDG. We recommend the use of French drains on the uphill side of abutments. The drains should outlet through a series of 4-inch diameter weep holes, spaced approximately 10-feet center-to-center.

6.2.2 Abutment and Pier Pile Design

- The proposed bridge abutments and Piers 2 and 3 may be supported on HP12x53, HP14x73, HP14x89, or HP14x117, ASTM A572 steel (50 ksi yield stress) piles driven to the required nominal resistance, anticipated to be developed in skin friction in glacial till and end-bearing on or near the bedrock surface.
- Pile installation should be controlled using wave equation analysis and field logging of the pile installation with final penetration resistance based on dynamic pile testing with signal matching analysis.

- The piles should be driven to a nominal resistance calculated by dividing the maximum factored pile load by a resistance factor of 0.65.
- Preliminary wave equation analyses should be completed to assess drivability of the selected pile section/s. Criteria for acceptability of the drivability analyses are that the piles can be driven to the required nominal resistance without exceeding the allowable driving stress (0.9Fy = 45 ksi for Grade 50 steel). The final penetration resistance must be within the MaineDOT range of 3 to 15 blows per inch. However, in GZA's experience, the preferred range of final penetration resistance is 6 to 10 blows per inch.
- Splices should be made in accordance with MaineDOT Standard Specification Section 501.09 – Splicing Piles.
- To limit driving damage, the steel H-piles should be fitted with protective driving tips in accordance with MaineDOT Standard Specification Section 501.10 – Pile Tips.

6.2.3 Spread Footing Design – Pier 1

- Pier 1 should be supported on spread footing foundations bearing on tremie concrete bearing on sound, intact bedrock. Footings designed to bear on intact bedrock should be designed for a nominal bearing resistance, q_n, of 55 ksf. At the strength limit state, footings should be designed for a maximum factored bearing resistance of 25 ksf. A bearing resistance of 25 ksf should be used for service limit state design.
- It is anticipated that the footing excavation will be completed in-the-wet within a braced cofferdam. The bedrock surface cleaned of loose soil and rock and sounded to assess the surface variation prior to placement of tremie concrete. Bearing surface preparation should be completed in accordance with an appropriate Special Provision to the contract. A typical Special Provision Section 511 is included in **Appendix D** for reference.
- The top of bedrock at Pier 1 is approximately Elevation 97. After removal of loose soil and rock, the prepared surface will be lower. We recommend the design bearing level at this location be set at or below El. 92 for purposes of design. It is important to note that the top of rock is not known for the entire foundation area until it is exposed. We expect that the bedrock bearing surface will be encountered above and possibly below the estimated level. Some construction-phase engineering should be anticipated to address the encountered conditions.
- Concrete used for cofferdam seals should consist of Class S Concrete, while concrete used for footings should consist of Class A Concrete, in accordance with MaineDOT BDG guidelines and MaineDOT Standard Specifications Section 502.05 – Composition and Proportioning.
- For spread footing foundations bearing directly on bedrock, the lateral loads may be resisted by friction between the footing bottoms and the bedrock. The recommended base resistance against sliding is based on NAVFAC DM7.02-63, Table 1, which indicates the sliding resistance coefficient (tan δ) is 0.6 for cast-in-place concrete on clean, sound rock. Therefore, the nominal sliding resistance between footings and bedrock subgrades is equal to the vertical force multiplied by 0.7. The factored sliding resistance coefficient is 0.56 for Strength Limit State.
- Anchoring, doweling, benching or other means of improving sliding resistance are recommended at locations where the prepared bedrock surface is steeper than 4 horizontal to 1 vertical (4H:1V) in any direction. Based on available boring data the bedrock slope at Pier 1 is not expected to exceed 4H:1V.

- Spread footings founded on bedrock should be checked for eccentricity with LRFD Section 10.6.3.3. Eccentricity of the footing reaction at the strength limit state should be limited such that the resultant reaction on the base of the footing is no further than 0.45 B from the centerline of the footing, where B is the principal dimension of the footing perpendicular to the axis of rotation.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 FOUNDATION SUBGRADE PREPARATION

It is anticipated that the footing excavation for Pier 1 will be completed in-the-wet within a braced cofferdam. The bedrock surface should be cleaned of loose soil and rock and sounded to assess the surface variation prior to placement of tremie concrete. Bearing surface preparation should be completed in accordance with an appropriate Special Provision to the contract. A typical Special Provision Section 511 is included in **Appendix D** for reference.

GZA anticipates that bedrock bearing surface will be variable in terms of elevation, slope and localized weathering. All soil and loose, decomposed, highly weathered and fractured bedrock should be removed from the footing bearing surface prior to placement of concrete. It is possible that the depth and degree of weathering below the bedrock surface could be highly irregular due to the geologic setting.

7.2 PILE INSTALLATION CONTROL

We recommend that the pile installation be controlled using wave equation analysis and field logging of the pile installation and that final penetration resistance be based on dynamic pile testing with signal matching analysis. We recommend that two dynamic pile tests with signal matching be performed at each substructure, one on a plumb and one on a battered pile, at the end of initial drive and again at the beginning of restrike 24 hours later. If the results of the first restrike test indicate no loss of estimated capacity compared to the end of initial drive, a request for waiver of subsequent restrike tests could be made to MaineDOT.

7.3 TEMPORARY LATERAL SUPPORT AND DEWATERING

Excavation is required for the bridge foundations at the river pier locations. The river piers will be constructed within braced steel sheet pile cofferdams. After the sheet piling and wale systems are installed, the cofferdams will be excavated in the wet to base of tremie seal level, the foundation piling will be installed, and unreinforced concrete tremie seals will be poured. Once the seal concrete sets, the cofferdams may be unwatered to allow foundation construction to proceed in the dry. Unwatering of the cofferdams will be achieved by open pumping from the top of tremie seal level.

The contractor should be responsible for controlling groundwater, surface runoff, infiltration and water from all other sources by methods which preserve the undisturbed condition of the subgrade and permit foundation construction in-the-dry. Discharge of pumped groundwater and river water should comply with all local, State, and federal regulations.

7.4 REUSE OF ON-SITE MATERIALS

Based on the test boring results, the existing material at the abutments contains more than 10 percent passing the No. 200 sieve. Therefore, the excavated material does not meet the MaineDOT requirements for Granular Borrow and/or Granular Borrow for Underwater Backfill and is unsuitable for use as structural backfill.

If the contractor wishes to reuse excavated material as embankment fill or in other areas, we recommend that the proposed material be stockpiled and tested for grain size distribution. Stockpiled materials meeting the appropriate MaineDOT specifications may be reused on the project.

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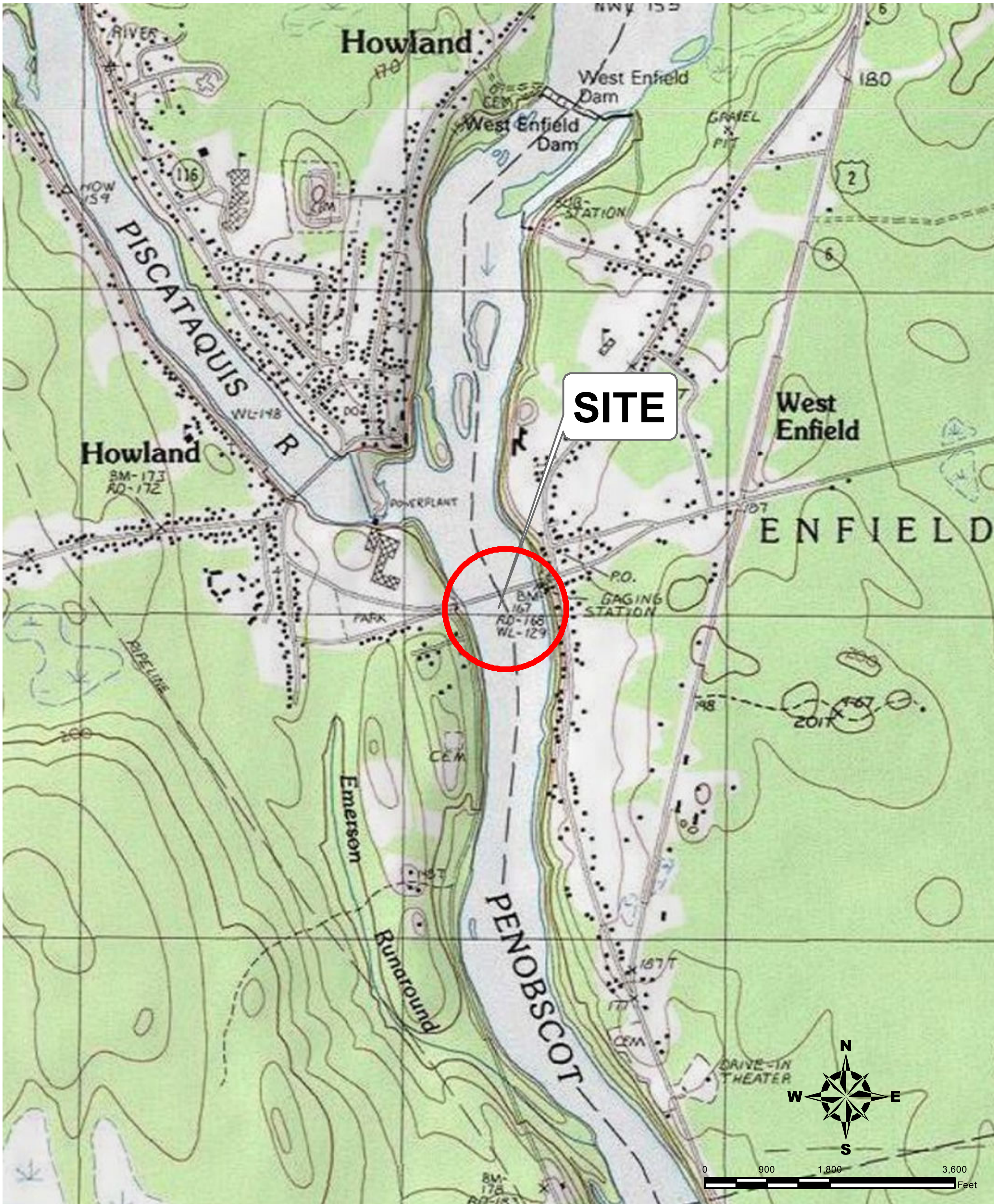
FIGURES



SOURCE : THIS MAP CONTAINS THE ESRI ARCGIS ONLINE USA TOPOGRAPHIC MAP SERVICE, PUBLISHED DECEMBER 12, 2009 BY ESRI ARCIMS SERVICES AND UPDATED AS NEEDED. THIS SERVICE USES UNIFORM NATIONALLY RECOGNIZED DATUM AND CARTOGRAPHY STANDARDS AND A VARIETY OF AVAILABLE SOURCES FROM SEVERAL DATA PROVIDERS

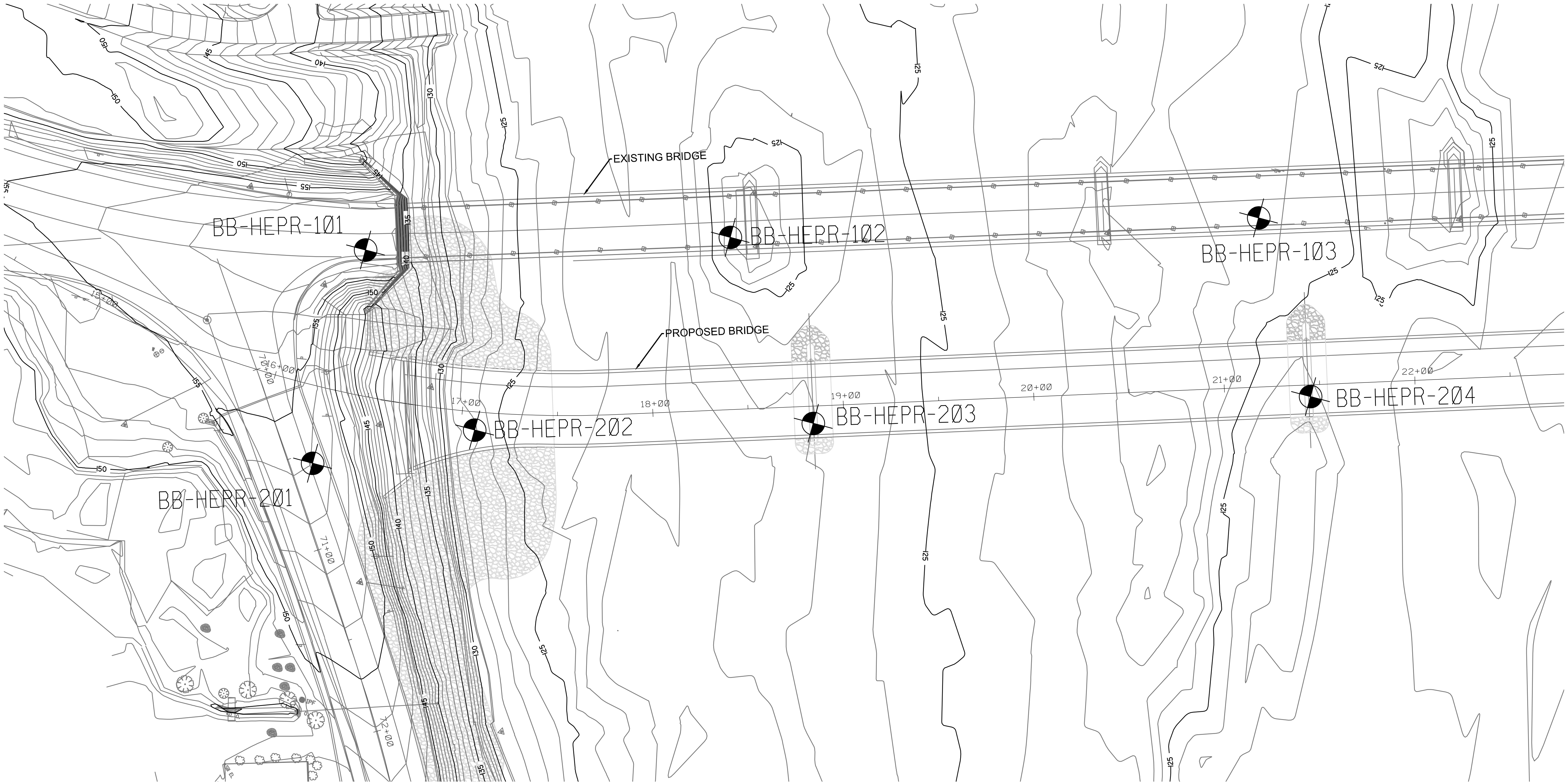
PENOBSCOT RIVER BRIDGE
MAINE DOT PIN 16705.00
HOWLAND/ENFIELD, MAINE

PREPARED BY:
GZA GeoEnvironmental, Inc.
Engineers and Scientists
www.gza.com

PREPARED FOR:
T.Y. LIN INTERNATIONAL, INC.

LOCUS PLAN

PROJ MGR: JRB	REVIEWED BY: RJM	CHECKED BY: CLS	FIGURE
DESIGNED BY: JRB	DRAWN BY: JAP	SCALE: 1 in = 2,000 ft	1
DATE: SEPTEMBER 2013	PROJECT NO. 09.0025796.00	REVISION NO.	



NOTES:

1) BASE MAP DEVELOPED FROM ELECTRONIC FILES PROVIDED BY T.Y. LIN INTERNATIONAL ON SEPTEMBER 10, 2012 (FILES INCLUDED CONTOURS.DGN, TOPO.DGN, ALIGNMENTS.DGN AND BRIDGE.DGN).

2) THE AS DRILLED LOCATIONS OF THE TEST BORINGS AND PROBES WERE SURVEYED BY MAINE DOT SURVEY CREW. COORDINATES WERE GIVEN BY MAINE DOT SURVEY.

3) BB–HEPR–200 SERIES BORINGS WERE PERFORMED BY MAINE TEST BORING OF HERMON, MAINE BETWEEN AUGUST 19, 2013 AND AUGUST 29, 2013 AND OBSERVED BY GZA PERSONNEL.

4) HB–HEPR–100 SERIES BORINGS WERE PERFORMED BY MAINE DOT IN 2010.

5) UNLESS SPECIFICALLY STATED BY WRITTEN AGREEMENT, THIS DRAWING IS THE SOLE PROPERTY OF GZA GEOENVIRONMENTAL, INC. (GZA). THE INFORMATION SHOWN ON THE DRAWING IS SOLELY FOR USE BY GZA'S CLIENT OR THE CLIENT'S DESIGNATED REPRESENTATIVE FOR THE SPECIFIC PROJECT AND LOCATION IDENTIFIED ON THE DRAWING. THE DRAWING SHALL NOT BE TRANSFERRED, REUSED, COPIED, OR ALTERED IN ANY MANNER FOR USE AT ANY OTHER LOCATION OR FOR ANY OTHER PURPOSE WITHOUT THE PRIOR WRITTEN CONSENT OF GZA. ANY TRANSFER, REUSE, OR MODIFICATION TO THE DRAWING BY THE CLIENT OR OTHERS, WITHOUT THE PRIOR WRITTEN EXPRESS CONSENT OF GZA, WILL BE AT THE USER'S SOLE RISK AND WITHOUT ANY RISK OR LIABILITY TO GZA.

LEGEND

BB–HEPR–207 — LOCATION AND DESIGNATION OF RT. 155 BRIDGE TEST BORING

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
PROJECT NO. 09.0025796.00
BRIDGE NO.2660 16705.00 BRIDGE PLANS

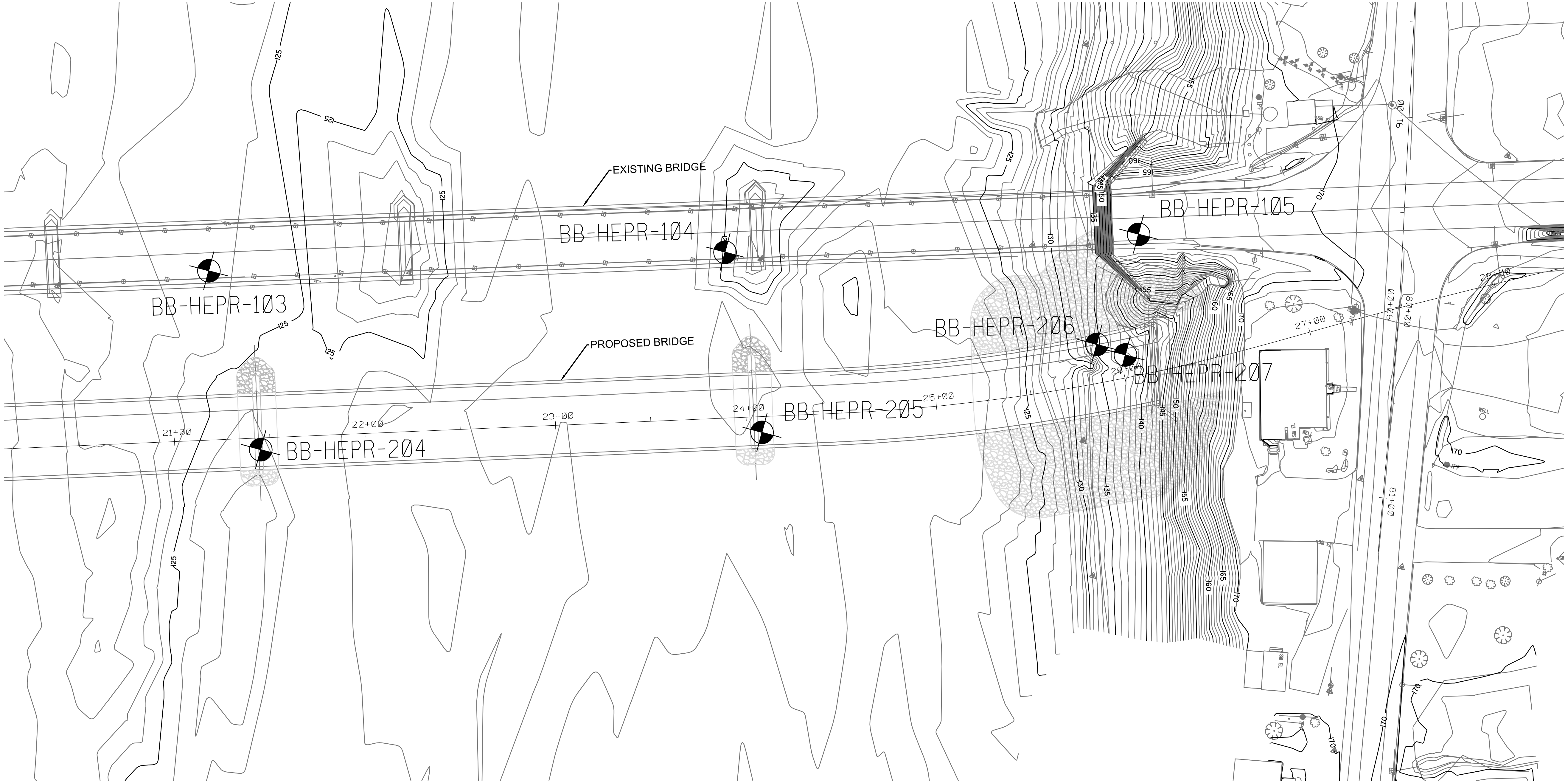
	BY	DATE
PROJ. MANAGER		
DESIGN–DETAILED		
CHECKED–REVIEWED		
DESIGN2–DETAILED2		
DESIGN3–DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

ROUTE 155 BRIDGE OVER THE
PENOBSCOT RIVER
HOWLAND–ENFIELD PENOBSCOT COUNTY

BORING LOCATION PLAN 1 OF 2

SHEET NUMBER
2A
OF 3



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LEGEND

BB-HEPR-207 — LOCATION AND DESIGNATION OF RT. 155 BRIDGE TEST BORING

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

PROJECT NO. 09.0025796.00

BRIDGE PLANS

BRIDGE NO.2660 16705.00

SIGNATURE

P.E. NUMBER

DATE

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		
CHECKED-REVIEWED		
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

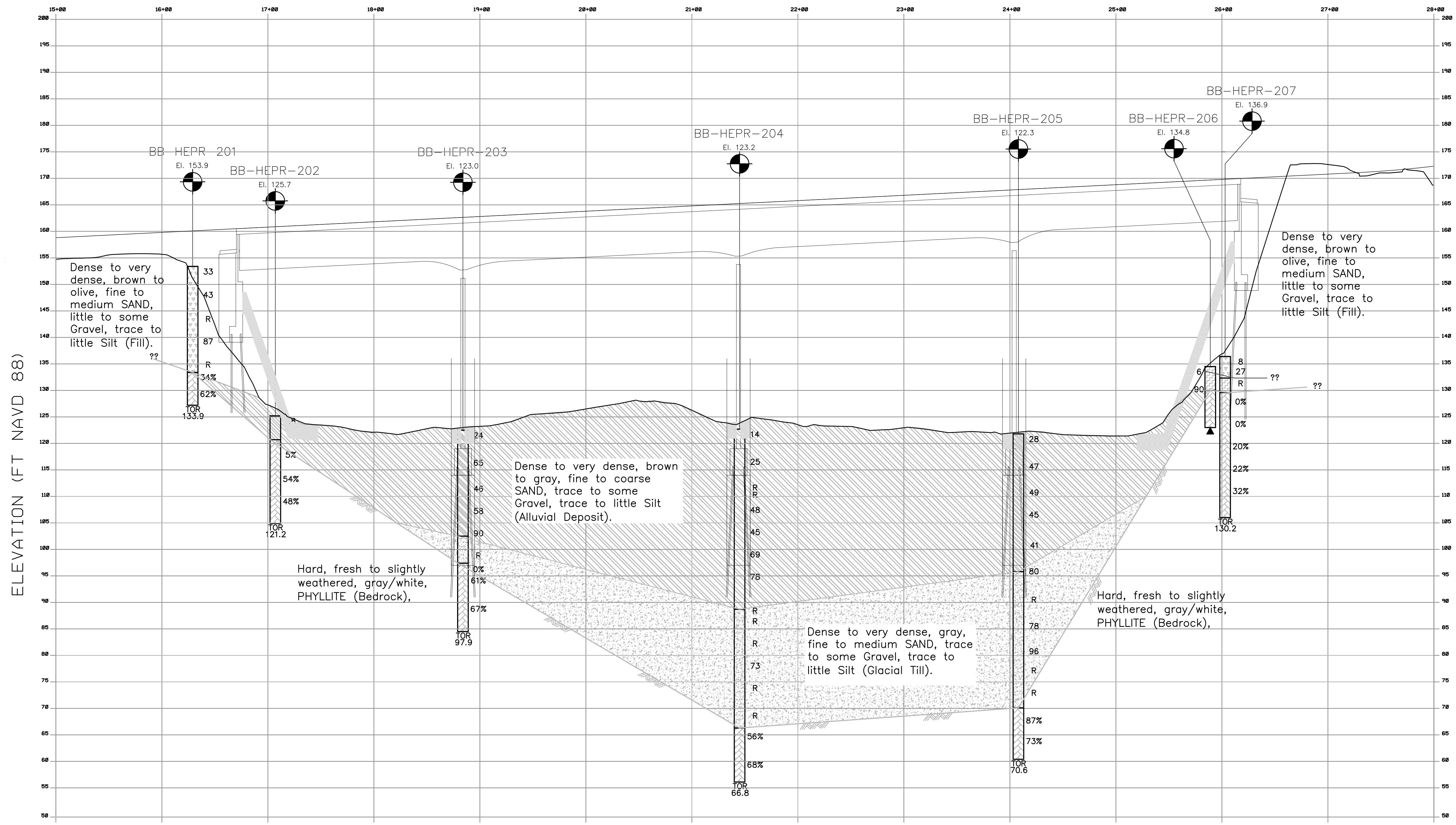
ROUTE 155 BRIDGE OVER THE PENOBSCOT RIVER
HOWLAND-ENFIELD PENOBSCOT COUNTY

BORING LOCATION PLAN
2 OF 2

SHEET NUMBER

2B

OF 3



LEGEND:

BB-HEPR-207 — BOREHOLE DESIGNATION
OFFSET 55'L — OFFSET FROM BASELINE

30 — ENERGY-CORRECTED STANDARD PENETRATION TEST (SPT) N-VALUE, N60
▽ — WATER LEVEL MEASURED DURING/AFTER DRILLING
R — REFUSAL (>50 BLOWS FOR 1" PENETRATION)
30% — RQD OF CORE RUN
TOR 130.2 — TOP OF CORED BEDROCK (FT-NAVD 1988)
▲ — DRILLING TERMINATED WITHOUT ROCK CORE

BORING STICK HATCH LEGEND:

FILL

VARIABLE RIVER BOTTOM SAND & SILT DEPOSIT

GLACIAL TILL

WEATHERED BEDROCK

BEDROCK

NOTES:

1) BASE PROFILE DEVELOPED FROM ELECTRONIC FILE DATED SEPTEMBER 4, 2013 PROVIDED BY T.Y. LIN INTERNATIONAL ENTITLED Profile–Geotech.dgn.

2) THIS GENERALIZED INTERPRETIVE SOIL PROFILE IS INTENDED TO CONVEY TRENDS IN SUBSURFACE CONDITIONS. THE BOUNDARIES BETWEEN STRATA ARE APPROXIMATE AND IDEALIZED, AND HAVE BEEN DEVELOPED BY INTERPRETATIONS OF WIDELY SPACED EXPLORATIONS AND SAMPLES. ACTUAL SOIL TRANSITIONS MAY VARY AND ARE PROBABLY MORE ERRATIC. FOR MORE SPECIFIC INFORMATION REFER TO THE EXPLORATION LOGS.

3) UNLESS SPECIFICALLY STATED BY WRITTEN AGREEMENT, THIS DRAWING IS THE SOLE PROPERTY OF GZA GEOENVIRONMENTAL, INC. (GZA). THE INFORMATION SHOWN ON THE DRAWING IS SOLELY FOR USE BY GZA'S CLIENT OR THE CLIENT'S DESIGNATED REPRESENTATIVE FOR THE SPECIFIC PROJECT AND LOCATION IDENTIFIED ON THE DRAWING. THE DRAWING SHALL NOT BE TRANSFERRED, REUSED, COPIED, OR ALTERED IN ANY MANNER FOR USE AT ANY OTHER LOCATION OR FOR ANY OTHER PURPOSE WITHOUT THE PRIOR WRITTEN CONSENT OF GZA. ANY TRANSFER, REUSE, OR MODIFICATION TO THE DRAWING BY THE CLIENT OR OTHERS, WITHOUT THE PRIOR WRITTEN EXPRESS CONSENT OF GZA, WILL BE AT THE USER'S SOLE RISK AND WITHOUT ANY RISK OR LIABILITY TO GZA.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
PROJECT NO. 09.0025796.00
WIN 16705.00
BRIDGE PLANS
BRIDGE NO.2660

SIGNATURE | P.E. NUMBER | DATE

PROJ. MANAGER | DESIGN–DETAILED | CHECKED–REVIEWED | DESIGN2–DETAILED2 | DESIGN3–DETAILED3 | REVISIONS 1 | REVISIONS 2 | REVISIONS 3 | REVISIONS 4 | FIELD CHANGES

ROUTE 155 BRIDGE OVER THE PENOBSCOT RIVER
HOWLAND–ENFIELD PENOBSCOT COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
3
OF 3

APPENDIX A

LIMITATIONS

LIMITATIONS

Explorations

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

2. The generalized soil profile described in the text is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized and have been developed by interpretations of widely spaced explorations and samples; actual soil transitions are probably more erratic. For specific information, refer to the boring logs.

3. Water level readings have been made in the drill holes at times and under conditions stated on the boring logs. These data have been reviewed and interpretations have been made in the text of this report. However, it must be noted that fluctuations in the level of the groundwater may occur due to variations in river level, rainfall, temperature, and other factors occurring since the time measurements were made.

Review

4. In the event that any changes in the nature, design, or location of the proposed structures are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions of this report modified or verified in writing by GZA GeoEnvironmental, Inc. It is recommended that this firm be provided the opportunity for a general review of final design and specifications in order that earthwork and foundation recommendations may be properly interpreted and implemented in the design and specifications.

Construction

5. It is recommended that this firm be retained to provide soil engineering services during construction of the excavation and foundation phases of the work. This is to observe compliance with the design concepts, specifications, and recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated prior to start of construction.

Use of Report

6. This soil and foundation engineering report has been prepared for this project by GZA GeoEnvironmental, Inc. If this report is used in preparing bids or cost estimates, the geotechnical assumptions should be reviewed by GZA.

7. This report has been prepared for this project by GZA GeoEnvironmental, Inc. for the exclusive use of the T.Y. Lin International for specific application to the Penobscot River Bridge project in Howland and Enfield, Maine in accordance with generally accepted soil and foundation engineering practices. No Warranty, express or implied, is made.

APPENDIX B

TEST BORING LOGS

Appendix B

Previous Test Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS (little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly-graded gravels, gravel sand mixtures, little or no fines
		GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	CLEAN SANDS (little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines
			SP	Poorly-graded sands, gravelly sand, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures.
FINE-GRAINED SOILS (more than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS (liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS (liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts
	HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.

Desired Soil Observations: (in this order)
Color (Munsell color chart)
Moisture (dry, damp, moist, wet, saturated)
Density/Consistency (from above right hand side)
Name (sand, silty sand, clay, etc., including portions - trace, little, etc.)
Gradation (well-graded, poorly-graded, uniform, etc.)
Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)
Structure (layering, fractures, cracks, etc.)
Bonding (well, moderately, loosely, etc., if applicable)
Cementation (weak, moderate, or strong, if applicable, ASTM D 2488)
Geologic Origin (till, marine clay, alluvium, etc.)
Unified Soil Classification Designation
Groundwater level

TERMS DESCRIBING DENSITY/CONSISTENCY

Coarse-grained soils (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) silty or clayey gravels; and (3) silty, clayey or gravelly sands. Consistency is rated according to standard penetration resistance.

Modified Burmister System

Descriptive Term	Portion of Total
trace	0% - 10%
little	11% - 20%
some	21% - 35%
adjective (e.g. sandy, clayey)	36% - 50%

Density of Cohesionless Soils	Standard Penetration Resistance N-Value (blows per foot)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

Fine-grained soils (more than half of material is smaller than No. 200 sieve): Includes (1) inorganic and organic silts and clays; (2) gravelly, sandy or silty clays; and (3) clayey silts. Consistency is rated according to shear strength as indicated.

Consistency of Cohesive soils	SPT N-Value blows per foot	Approximate Undrained Shear Strength (psf)	Field Guidelines
Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily Penetrates
Soft	2 - 4	250 - 500	Thumb easily penetrates
Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort
Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort
Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail
Hard	>30	over 4000	Indented by thumbnail with difficulty

Rock Quality Designation (RQD):

RQD = $\frac{\text{sum of the lengths of intact pieces of core* > 100 mm}}{\text{length of core advance}}$

*Minimum NQ rock core (1.88 in. OD of core)

Correlation of RQD to Rock Mass Quality

Rock Mass Quality	RQD
Very Poor	<25%
Poor	26% - 50%
Fair	51% - 75%
Good	76% - 90%
Excellent	91% - 100%

Desired Rock Observations: (in this order)
Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Lithology (igneous, sedimentary, metamorphic, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
- -dip (horiz - 0-5, low angle - 5-35, mod. dipping - 35-55, steep - 55-85, vertical - 85-90)
- -spacing (very close - <5 cm, close - 5-30 cm, mod. close 30-100 cm, wide - 1-3 m, very wide >3 m)
- -tightness (tight, open or healed)
- -infilling (grain size, color, etc.)

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock mass quality (very poor, poor, etc.)
ref: AASHTO Standard Specification for Highway Bridges 17th Ed. Table 4.4.8.1.2A
Recovery

Maine Department of Transportation
Geotechnical Section
Key to Soil and Rock Descriptions and Terms
Field Identification Information

Sample Container Labeling Requirements:
PIN
Bridge Name / Town
Boring Number
Sample Number
Sample Depth
Blow Counts
Sample Recovery
Date
Personnel Initials

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Penobscot River Bridge #2660 carries Route 155 over Penobscot River
Location: Howland-Enfield, Maine

Boring No.: BB-HEPR-101
PIN: 16705.00

Driller:	Northern Test Boring	**Elevation (ft.)**	157.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike Nadeau/Ty Whitworth	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/16/10; 10:30-16:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	N633293 E1761886	**Casing ID/OD:**	HW	**Water Level*:**	19.0 ft bgs.
Hammer Efficiency Factor:	0.678	**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, PP = Pocket Penetrometer
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/C = weight of rods or casing
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							SSA	157.10		PAVEMENT. 0.40	
	1D	24/19	1.00 - 3.00	15/13/13/12	26	29				Brown, damp, medium dense, fine to coarse SAND, some gravel, trace silt, (Fill).	G#237550 A-1-b, SW-SM WC=3.9%
5	2D	24/20	5.00 - 7.00	16/31/36/32	67	76					
								151.50		6.00 Black, damp, very dense, fine to coarse SAND, little gravel, trace silt, (Fill).	
								149.00		8.50	
10	3D	24/15	10.00 - 12.00	3/3/2/2	5	6	20			Light brown to orange, wet, loose, fine to medium SAND, trace gravel and coarse sand, little silt (Fill).	
							26				
							26				
							35				
							25				
15	4D	24/16	15.00 - 17.00	3/3/2/2	5	6	11			Similar to above.	G#207064 A-2-4, SM WC=21.8%
							16				
							27				
							30				
							31				
20	5D	24/12	20.00 - 22.00	11/9/8/10	17	19	52	137.50		20.00 Brown, wet, medium dense, fine to coarse SAND, little gravel, some silt, trace organics, with brick pieces, (Fill).	
							320				
							55				
							72	134.50		23.00	
							94				
25											

Remarks:

Auto Hammer #283

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Penobscot River Bridge #2660 carries Route 155 over Penobscot River Location: Howland-Enfield, Maine	Boring No.: BB-HEPR-101 PIN: 16705.00

Driller:	Northern Test Boring	**Elevation (ft.)**	157.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike Nadeau/Ty Whitworth	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/16/10; 10:30-16:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	N633293 E1761886	**Casing ID/OD:**	HW	**Water Level*:**	19.0 ft bgs.
Hammer Efficiency Factor:	0.678	**Hammer Type:**	Automatic ☒	Hydraulic ☐	Rope & Cathead ☐

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U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test, PP = Pocket Penetrometer
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/C = weight of rods or casing
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.
25	6D	9.6/7	25.00 - 25.80	23/50(3.6")	---		60			Light brown, wet, very dense, fine to coarse SAND, some gravel and silt, (Till). Cobble from 25.8-26.0 ft bgs. Roller Coned ahead to 30.3 ft bgs.	G#207066 A-2-4, SM WC=9.7%
							89				
							60	129.70		27.80 Very dense from 27.8-30.3 ft bgs., Weathered Rock?	
30	7D R1	3.6/3.6 36/23	30.00 - 30.30 30.30 - 33.30	50(3.6") RQD = 16%	---		NQ-2	127.20		30.30 Top of Intack Bedrock at Elev. 127.2 ft. Bedrock: Grey, fine-grained, moderately hard, slightly weathered, highly fractured, PHYLLITE, thin, steep bedding planes, joints very close, minor silt in-filling and iron staining. Rock Mass Quality is Very Poor [Vassalboro Formation] R1:Core Times (min:sec) 30.3-31.3 ft (6:00) 31.3-32.3 ft (5:15) 32.3-33.3 ft (6:45) 64% Recovery Core Blocked	
	R2	42/42	33.30 - 36.80	RQD = 0%						R2:CoreTimes (min:sec) 33.3-34.3 ft (4:20) 34.3-35.3 ft (3:16) 35.3-36.3 ft (4:30) 36.3-36.8 ft (5:00) 100% Recovery Core Blocked	
35											
	R3	36/36	36.80 - 39.80	RQD = 0%				117.70		R3:CoreTimes (min:sec) 36.8-37.8 ft (5:00) 37.8-38.8 ft (6:30) 38.8-39.8 ft (6:45) 100% Recovery Core Blocked 39.80 **Bottom of Exploration at 39.80 feet below ground surface.**	
40											
45											
50											

Remarks:

Auto Hammer #283

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Penobscot River Bridge #2660 carries Route 155 over Penobscot River
Location: Howland-Enfield, Maine

Boring No.: BB-HEPR-102
PIN: 16705.00

Driller:	MaineDOT	**Elevation (ft.)**	126.8	**Auger ID/OD:**	N/A
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Schonewald	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/16/10-8/17/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	N633348.9 E1762069	**Casing ID/OD:**	HW & NW	**Water Level*:**	River Boring
Hammer Efficiency Factor:	0.84	**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, PP = Pocket Penetrometer
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/C = weight of rods or casing
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	0.00 - 2.00	1/2/11/10	13	18				Brown-grey, medium dense, fine to coarse sandy GRAVEL, trace silt, (Alluvium).	G#207067 A-1-a, GP WC=10.2%
							67				
							78				
							76				
							129	122.80		4.00	
5							118				
	2D	24/11	6.00 - 8.00	22/14/12/11	26	36	72			Grey, dense, fine to medium SAND, some gravel, little to some silt, trace coarse sand, (Alluvium).	
							71				
							64				
							76				
10							82				
	3D	24/9	11.00 - 13.00	14/19/17/20	36	50	73			Grey, dense, fine to coarse SAND, some gravel, little to some silt, (Alluvium).	
							93				
							125				
							134				
15							168				
	4D	24/12	16.00 - 18.00	16/36/33/34	69	97	123			Grey, very dense, fine to coarse SAND, some gravel, little silt, (Alluvium).	G#207068 A-1-b, SW-SM WC=11.7%
							194				
							252				
							270				
20							432				
	5D	9.6/3	21.00 - 21.80	21/50(3.6")	---			104.80		Brownish-grey, very dense, fine to coarse sandy GRAVEL, trace to little silt, (Alluvium). Attempt to roller cone ahead, solid at 21.8 ft bgs. Telescoped NW Casing, drive and washout to 22.0 ft bgs.	
	R1	45.6/32	22.00 - 25.80	RQD = 11%			NQ-2	103.90		22.00	
										R1: Top 0.9 ft. Boulder underlain by Gravel. 22.90	
25										Top of Bedrock at Elev. 103.9 ft. R1:Bedrock: Grey, fine-grained, medium hard, fresh to slightly	

Remarks:

Bridge Deck to mudline, 33.1 ft. Bridge Deck Core 9 in, no asphalt.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Penobscot River Bridge #2660 carries Route 155 over Penobscot River
Location: Howland-Enfield, Maine

Boring No.: BB-HEPR-102
PIN: 16705.00

Driller:	MaineDOT	**Elevation (ft.)**	126.8	**Auger ID/OD:**	N/A
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Schonewald	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/16/10-8/17/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	N633348.9 E1762069	**Casing ID/OD:**	HW & NW	**Water Level*:**	River Boring
Hammer Efficiency Factor:	0.84	**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/C = weight of rods or casing
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.
25	R2	22.8/20	25.80 - 27.70	RQD = 0%						weathered, thinly bedded, calcereous muscovite PHYLLITE. Bedding high angle to vertical, highly undulating on small scale. Close, high angle, planar to undulating, smooth to rough, fresh to discolored, tight to open, breaks along bedding. Mud seam at 23.1 to 23.4 ft. Highly fractured at 24.8 ft. Rock Mass Quality is Very Poor. [Vassalboro Formation] R1:Core Times (min:sec) 22.0-23.0 ft (2:10) 23.0-24.0 ft (2:10) 24.0-25.0 ft (3:55) 25.0-25.8 ft (8:15) 70% Recovery R2:Bedrock: Same as R1, except fresh, highly fractured entire run. 80% Recovery. Core Times not recorded.	
	R3	60/60	27.70 - 32.70	RQD = 50%							
30											
								94.10		R3:Bedrock: Grey, fine-grained, medium hard, fresh, thinly bedded, calcareous muscovite PHYLLITE. Bedding high angle to vertical, highly undulating on small scale. Close to moderate, high angle upper and lower run and low angle middle run, undulating, smooth to rough, fresh, tight breaks along bedding. Highly fractured at 32.4 ft. Rock Mass Quality is Poor. [Vassalboro Formation] R3:Core Times (min:sec) 28.0-29.0 ft (2:40) 29.0-30.0 ft (2:30) 30.0-31.0 ft (2:25) 31.0-32.7 ft (2:45) 100% Recovery 32.70 **Bottom of Exploration at 32.70 feet below ground surface.**	
35											
40											
45											
50											

Remarks:

Bridge Deck to mudline, 33.1 ft. Bridge Deck Core 9 in, no asphalt.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Penobscot River Bridge #2660 carries Route 155 over Penobscot River
Location: Howland-Enfield, Maine

Boring No.: BB-HEPR-103
PIN: 16705.00

Driller:	Northern Test Boring	**Elevation (ft.)**	127.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike Nadeau/Ty Whitworth	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/17/10; 08:00-16:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	N633430.5 E1762334	**Casing ID/OD:**	HW	**Water Level*:**	River Boring
Hammer Efficiency Factor:	0.678	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
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RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
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.
0	1D	24/8	0.00 - 2.00	6/9/17/10	26	29	32			Grey-brown, saturated, medium dense, fine to coarse sandy GRAVEL, trace silt, (Alluvium).	G#207069 A-1-a, GW WC=9.9%
							105				
							204				
	2D	24/13	3.50 - 5.50	26/31/20/18	51	58	152			Grey, saturated, very dense, GRAVEL, some fine to coarse sand, trace silt, (Alluvium). Roller Coned ahead from 3.5-8.5 ft bgs.	G#207070 A-1-a, GW WC=7.7%
							125				
5							88				
							84				
							91				
	3D	24/5	8.50 - 10.50	10/5/3/6	8	9	58			Similar to above, except loose.	
							55				
10							67				
							97				
							88				
	4D	24/4	13.50 - 15.50	5/7/8/7	15	17	51			Similar to above, except medium dense.	
							62				
15							76				
							92				
							174	110.00		17.00	
	5D	24/15	18.50 - 20.50	12/21/32/33	53	60	70			Grey, wet, very dense, fine to coarse SAND, some gravel, trace silt, (Alluvium). Roller Coned ahead to 23.5 ft bgs.	G#207071 A-1-b, SW-SM WC=11.5%
							41				
20							38				
							61				
							143	105.00		22.00	
	6D	24/17	23.50 - 25.50	29/45/33/50	78	88	136			Similar to above but gravelly. Roller Coned ahead to 28.5 ft bgs.	
25							92				

Remarks:

Auto Hammer #283
36.2 ft from Bridge Deck to Ground.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Penobscot River Bridge #2660 carries Route 155 over Penobscot River	**Boring No.:**	BB-HEPR-103		
Location:	Howland-Enfield, Maine	**PIN:**	16705.00		
Driller:	Northern Test Boring	**Elevation (ft.)**	127.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike Nadeau/Ty Whitworth	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/17/10; 08:00-16:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	N633430.5 E1762334	**Casing ID/OD:**	HW	**Water Level*:**	River Boring
Hammer Efficiency Factor:	0.678	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
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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.
25							148				
							250				
							422	100.00		27.00	
	7D	18/16	28.50 - 30.00	22/33/50	83	94	350			Similar to 5D. Roller Coned ahead to 33.5 ft bgs.	G#207072 A-1-a, SW WC=10.1%
							131				
30							112				
							222				
							315				
	8D	16.8/14	33.50 - 34.90	31/39/56(4.8")	---		242			Similar to 5D. Roller Coned ahead to 38.5 ft bgs.	
							139				
35							225				
							232				
							OPEN HOLE	89.40		Open Hole, used 2 cups ACCU-VIS drilling mud. 37.60	
	9D	9.6/8	38.50 - 39.30	40/50(3.6")	---					Grey, wet, very dense, SILT, some fine to coarse sand, little gravel, (Till). Roller Coned ahead to 43.5 ft bgs.	
40											
	10D	15.6/12	43.50 - 44.80	23/43/50(3.6")	---					Similar to above. Roller Coned ahead to 49.0 ft bgs.	G#207073 A-4, SM WC=7.9%
45											
										Cobble from 48.5-48.9 ft bgs.	
50	11D	13.2/7	49.00 - 50.10	15/40/30 (1.2")	---					Similar to above. Roller Coned ahead to 50.1 ft bgs.	

Remarks:

Auto Hammer #283
36.2 ft from Bridge Deck to Ground.

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Boring No.: BB-HEPR-103

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Penobscot River Bridge #2660 carries Route 155 over Penobscot River
Location: Howland-Enfield, Maine

Boring No.: BB-HEPR-103
PIN: 16705.00

Driller:	Northern Test Boring	**Elevation (ft.)**	127.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike Nadeau/Ty Whitworth	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/17/10; 08:00-16:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	N633430.5 E1762334	**Casing ID/OD:**	HW	**Water Level*:**	River Boring
Hammer Efficiency Factor:	0.678	**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, PP = Pocket Penetrometer	WOH = weight of 140lb. hammer	Hammer Efficiency Factor = Annual Calibration Value	PI = Plasticity Index
MV = Unsuccessful Insitu Vane Shear Test attempt	WOR/C = weight of rods or casing	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.
50								76.90		50.10 — Top of Bedrock at Elev. 76.9 ft. Roller Coned ahead into Bedrock to 51.1 ft bgs.	
	R1	60/60	51.10 - 56.10	RQD = 93%			NQ-2			Bedrock: Grey, fine-grained, moderately hard, fresh, PHYLLITE, thin, steep bedding planes, joints close to moderatrely close except R2 is a weathered zone, otherwise minor silt in-filling, no iron staining. Rock Mass Quality is Good in R1 and R3, and Very Poor in R2. [Vassalboro Formation] R1:Core Times (min:sec) 51.1-52.1 ft (4:10) 52.1-53.1 ft (5:30) 53.1-54.1 ft (5:00) 54.1-55.1 ft (5:10) 55.1-56.1 ft (5:30) 100% Recovery	
55											
	R2	15.6/12	56.10 - 57.40	RQD = 0%						R2:Core Times (min:sec) 56.1-57.1 ft (5:45) 57.1-57.4 ft (4:00) 80% Recovery Core Blocked	
	R3	48/48	57.40 - 61.40	RQD = 90%						R3:Core Times not recorded. 100% Recovery	
60											
								65.60		61.40 — **Bottom of Exploration at 61.40 feet below ground surface.**	
65											
70											
75											

Remarks:

Auto Hammer #283
36.2 ft from Bridge Deck to Ground.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Penobscot River Bridge #2660 carries Route 155 over Penobscot River
Location: Howland-Enfield, Maine

Boring No.: BB-HEPR-104
PIN: 16705.00

Driller:	MaineDOT	**Elevation (ft.)**	123.4	**Auger ID/OD:**	N/A
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Schonewald	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/17/10-8/18/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	N633510 E1762593	**Casing ID/OD:**	HW & NW	**Water Level*:**	River Boring
Hammer Efficiency Factor:	0.84	**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, PP = Pocket Penetrometer
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/C = weight of rods or casing
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	0.00 - 2.00	6/4/8/14	12	17	16			Brown, medium dense, GRAVEL, some fine to coarse sand, trace silt, (River Alluvium).	G#207074 A-1-a, GP WC=10.1%
							52				
							62				
							94				
							131	118.90		4.50	
5							86				
							144				
	2D	16/4	7.00 - 8.33	38/47/50(4")	---		196			Grey, very dense, fine to coarse SAND, some gravel and silt, (Alluvium). [a]Washed Ahead of casing from 8.0-22.0 ft bgs.	G#207075 A-1-b, SM WC=8.3%
							[a]WA				
10											
	3D	24/11	12.00 - 14.00	15/19/31/32	50	70				Greyish brown, very dense, fine to coarse SAND, little gravel, trace silt, (Alluvium).	G#239827 A-1-b, SP-SM WC=15.1%
15											
	4D	24/4	17.00 - 19.00	20/19/19/20	38	53				Brown, very dense, gravelly, fine to coarse SAND, trace silt, (Alluvium).	
										Switch to NW Casing at 19.0 ft bgs.	
20											
	5D	24/13	22.00 - 24.00	22/23/32/21	55	77	45			Greyish brown, very dense, gravelly, fine to coarse SAND, trace silt, (Alluvium).	G#239828 A-1-a, SW-SM WC=10.3%
							81				
25							108				

Remarks:

Bridge Deck to mudline, 42.3 ft. Bridge Deck Core lost through deck.
Deck to waterline 38.2 ft.

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: Penobscot River Bridge #2660 carries Route 155 over Penobscot River
Location: Howland-Enfield, Maine

Boring No.: BB-HEPR-104
PIN: 16705.00

Driller:	MaineDOT	**Elevation (ft.)**	123.4	**Auger ID/OD:**	N/A
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Schonewald	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/17/10-8/18/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	N633510 E1762593	**Casing ID/OD:**	HW & NW	**Water Level*:**	River Boring
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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q_p = Unconfined Compressive Strength (ksf)
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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.
25							176				
							162				
	6D	8.4/5	27.00 - 27.70	40/50(2.4")	---		WASH			Greyish brown, very dense, gravelly, fine to coarse SAND, trace silt, (Alluvium).	
							67	95.40		Roller Cone refusal at 27.8 ft bgs. Attempt rock core. Dropped through boulder at 28.3 ft bgs. 28.00	
							126				
30							153				
							166				
	7D	24/17	32.00 - 34.00	44/40/48/48	88	123	aWA			Grey, very dense, SILT, some fine to coarse sand, trace gravel, (Till). aWashed Ahead of casing from 32.0-47.0 ft bgs.	G#239829 A-4, ML WC=11.6%
35											
	8D	9.6/1	37.00 - 37.80	59/56(3.6")	---					Same as above.	
40								83.40		40.00	
	9D	9.6/5	42.00 - 42.80	48/50(3.6")	---					Grey, very dense, fine to coarse sandy SILT, trace gravel, (Till) .	
45											
	10D	21.6/20	47.00 - 48.80	35/46/49/50(3.6)	95	133	aWA			Grey, very dense, fine to coarse sandy SILT, trace gravel, (Till). a Washed ahead to 52.0 ft bgs.	G#239830 A-4, SM WC=8.4%
50											

Remarks:

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Soil/Rock Exploration Log
US CUSTOMARY UNITS

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Location: Howland-Enfield, Maine

Boring No.: BB-HEPR-104
PIN: 16705.00

Driller:	MaineDOT	**Elevation (ft.)**	123.4	**Auger ID/OD:**	N/A
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Schonewald	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/17/10-8/18/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	N633510 E1762593	**Casing ID/OD:**	HW & NW	**Water Level*:**	River Boring
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
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WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
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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
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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										Casing and Roller Cone Refusal at 52.0 ft bgs.	
								71.40		52.00	
	R1	60/54	52.00 - 57.00	RQD = 62%			NQ-2			Top of Bedrock at Elev. 71.4 ft. R1:Bedrock: Grey, fine-grained, hard, fresh, thinly bedded calcereous muscovite PHYLLITE. Bedding high angle, highly undulating on smaller scale. Moderately spaced, high angle and one low angle, planar to undulating, rough, fresh, tight breaks along bedding. Upper foot more fractured, (close spacing). Rock Mass Quality is Fair. [Vassalboro Formation] R1:Core Times (min:sec) 52.0-53.0 ft (3:30) 53.0-54.0 ft (2:50) 54.0-55.0 ft (2:40) 55.0-56.0 ft (-:--) (Core Time not recorded) 56.0-57.0 ft (2:35) 90% Recovery	
55											
	R2	60/60	57.00 - 62.00	RQD = 52%						R2:Bedrock: Grey, fine-grained, hard, fresh, thinly bedded calcereous muscovite PHYLLITE. Bedding high angle, highly undulating on small scale. Close, high angle, undulating to stepped, rough, fresh, tight to open breaks along bedding. Fractured at 57.0 to 57.5 ft and 58.8 to 60.0 ft. Rock Mass Quality is Fair. [Vassalboro Formation] R2:Core Times (min:sec) 57.0-58.0 ft (3:00) 58.0-59.0 ft (3:30) 59.0-60.0 ft (4:30) 60.0-61.0 ft (4:00) 61.0-62.0 ft (3:40) 100% Recovery	
60											
								61.40		62.00	
										Bottom of Exploration at 62.00 feet below ground surface.	
65											
70											
75											

Remarks:

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Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Penobscot River Bridge #2660 carries Route 155 over Penobscot River
Location: Howland-Enfield, Maine

Boring No.: BB-HEPR-105
PIN: 16705.00

Driller:	Northern Test Boring	**Elevation (ft.)**	167.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike Nadeau/Ty Whitworth	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Schonewald	**Rig Type:**	Diedrich D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/19/10; 08:00-14:15	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	N633575.2 E1762800	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.678	**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.
0							SSA	166.60		ASPHALT. 0.80	
	1D	24/6	1.00 - 3.00	17/14/15/15	29	33				Brown, damp, dense, fine to coarse SAND, some gravel, trace silt, (Fill).	G#239831 A-1-b, SW-SM WC=2.7%
5	2D	24/2	5.00 - 7.00	8/8/9/8	17	19				Brown, dry, medium dense, gravelly fine to coarse SAND, trace silt, piece of gravel in tip of spoon, (Fill).	
10	3D	24/16	10.00 - 12.00	6/10/7/5	17	19	62			Brown, damp, medium dense, fine to coarse SAND, some gravel, trace silt, (Fill).	G#239832 A-1-b, SW-SM WC=7.5%
							81				
							136				
							126			Possible old asphalt layer at 13.0 ft based on wash water.	
							148	152.40		15.00	
15	4D/AB	24/12	15.00 - 17.00	8/5/6/7	11	12	38	152.20		4D/A (15.0-15.2) Brown, moist, CLAY-SILT mixed with fill. 15.20	G#239833 A-6, CL WC=24.1% LL=33 PL=21 PI=12
							45			4D/B (15.2-17.0) Brown, moist, stiff, CLAY-SILT, trace fine to medium sand, mottled, (Glaciomarine).	
							66				
							81	148.40		19.00	
							72			Driller notes material change at 19.0 ft bgs.	
20	5D	24/10	20.00 - 22.00	2/2/2/3	4	5	40			Dark brown and black, moist, loose, fine to medium SAND, little silt, (Glaciomarine).	G#239834 A-2-4, SM WC=18.5%
							44				
							53				
							60				
25							68				

Remarks:

Auto Hammer #283

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US CUSTOMARY UNITS

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Location: Howland-Enfield, Maine

Boring No.: BB-HEPR-105
PIN: 16705.00

Driller:	Northern Test Boring	**Elevation (ft.)**	167.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike Nadeau/Ty Whitworth	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Schonewald	**Rig Type:**	Diedrich D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/19/10; 08:00-14:15	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	N633575.2 E1762800	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.678	**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.
25	6D	24/7	25.00 - 27.00	3/4/4/5	8	9	45			Same as above.	
							59				
							82				
							99	138.80		28.60 Drilling behavior suggests material change at 28.6 ft, boney material.	
							194			Brown, dense, gravelly fine to coarse SAND, some silt, (Till).	
30	7D	24/7	30.00 - 32.00	11/15/17/18	32	36	74				
							104				
							208				
							338				
							286				
35	8D	24/12	35.00 - 37.00	39/21/38/36	59	67	aWA			Brown, very dense, gravelly, fine to coarse SAND, litle silt, (Till). aWashed Ahead to 39.8 ft bgs.	G#239835 A-1-b, SM WC=9.1%
	R1	60/54	39.80 - 44.80	RQD = 35%			NQ-2	127.60		39.80	
40										Top of Bedrock at Elev. 127.6 ft.	
										R1:Bedrock: Grey, fine-grained, hard, fresh to sliightly weathered, calcereous METASILTSTONE. Original bedding highly disturbed to not discernible. Quartz inclusions, close to moderately spaced, moderately dipping and low angle, undulating, rough, discolored, open fractures. Rock Mass Quality is Poor. [Vassalboro Formation]	
										R1:Core Times (min:sec) 39.8-40.8 ft (4:00) 40.8-41.8 ft (6:30) 41.58-42.8 ft (7:40) 42.8-43.8 ft (6:40) 43.8-44.8 ft (8:30) 90% Recovery	
	R2	42/38	44.80 - 48.30	RQD = 86%							
45											
										R2:Bedrock: Same as R1, except original bedding not discernible, more quartz inclusions, and fractures are fresh to discolored, two drill breaks at 47.15 and 47.65 ft. Rock Mass Quality is Good. 90% Recovery	
	R3	24/22	48.30 - 50.30	RQD = 71%							
										R3:Bedrock: Same as R2, one drill break at 49.6 ft. Rock Mass Quality	
50											

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Auto Hammer #283

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Logged By:	B. Schonewald	**Rig Type:**	Diedrich D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/19/10; 08:00-14:15	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	N633575.2 E1762800	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.678	**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.
50							V	117.10		is Fair.	
										50.30	
										Bottom of Exploration at 50.30 feet below ground surface.	
55											
60											
65											
70											
75											

Remarks:

Auto Hammer #283

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Appendix B

GZA Test Boring Logs

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Penobscot River Bridge, Route 155
Location: Howland, Maine

Boring No.: BB-HEPR-201
PIN: 16705.00

Driller:	Maine Test Boring	**Elevation (ft.)**	153.9'	**Auger ID/OD:**	N/A
Operator:	Brad Enos	**Datum:**	NAVD88	**Sampler:**	Split Spoon
Logged By:	Joshua Szmyt	**Rig Type:**	CME-45 (Skid)	**Hammer Wt./Fall:**	140/30"
Date Start/Finish:	8/29/13 - 8/29/13	**Drilling Method:**	Cased Washed	**Core Barrel:**	NQ2
Boring Location:	N 633178, E 1761888	**Casing ID/OD:**	3"/3-1/2"	**Water Level*:**	River Boring
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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RC = Roller Cone
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WOR = weight of rods
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T_v = Pocket Torvane Shear Strength (psf)
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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	24/12	0.0 - 2.0	12-14-19-24	33	33	WA	153.6		-ASPHALT- 0.3 Dense, light brown, fine to medium SAND, some Gravel, trace Silt (SM). -FILL-	
5	2D	24/16	5.0 - 7.0	12-23-20-16	43	43	42			Dense, brown, fine to coarse SAND, some Gravel, little Silt. (SM)	SM A-1-b
							40				
							40				
							66				
							70				
10	3D	1/0	10.0 - 10.1	50/1"	--		WA			No Recovery.	
							79				
							71				
15	4D	24/12	15.0 - 17.0	29-46-41-53	87	87	40			Very dense, olive, fine to medium SAND, little Gravel, little Silt. Iron staining 9" from top of sample. (SM)	
							63				
							153				
							94				
							161				
20	5D	2/0	20.0 - 20.2	105/2"	--			133.9		20.0 No Recovery Top of Bedrock at 20.0' bgs. Advanced roller bit to 21.5' bgs and set 3" ID casing.	
	R1	32/29	21.5 - 24.2	RQD = 34%			NQ			Hard, fresh, fine grained, gray/white PHYLLITE. Joints are closely spaced, high angle, planar, rough, fresh, tight, with Silt deposits. Rock Mass Quality = Very Poor Rock Core Times (min/ft): 5.0, 4.0, 3.5	
25	R2	24/22	24.2 - 26.2	RQD = 62%						Hard, fresh, fine-grained, gray/white PHYLLITE. Joints are moderately dipping, planar, rough, open.	

Remarks:

1. Encountered boulder at 9.8' bgs (approximately 2.6' thick).
2. Encountered boulder at 12.2' bgs (approximately 1.0' thick).

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: Penobscot River Bridge, Route 155		**Boring No.:** BB-HEPR-201	
Location: Howland, Maine		**PIN:** 16705.00	
Driller: Maine Test Boring	**Elevation (ft.):** 153.9'	**Auger ID/OD:** N/A	
Operator: Brad Enos	**Datum:** NAVD88	**Sampler:** Split Spoon	
Logged By: Joshua Szmyt	**Rig Type:** CME-45 (Skid)	**Hammer Wt./Fall:** 140/30"	
Date Start/Finish: 8/29/13 - 8/29/13	**Drilling Method:** Cased Washed	**Core Barrel:** NQ2	
Boring Location: N 633178, E 1761888	**Casing ID/OD:** 3"/3-1/2"	**Water Level*:** River Boring	
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.
25							↓			Rock Mass Quality = Fair Rock Core Times (min/ft): 4.25, 4.25	
								127.7		26.2 **Bottom of Exploration at 26.20 feet below ground surface.**	
30											
35											
40											
45											
50											

Remarks:

1. Encountered boulder at 9.8' bgs (approximately 2.6' thick).
2. Encountered boulder at 12.2' bgs (approximately 1.0' thick).

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:	Penobscot River Bridge, Route 155
Location:	Howland, Maine
Boring No.:	BB-HEPR-202
PIN:	16705.00

Driller:	Maine Test Boring	**Elevation (ft.)**	125.7' (mudline)	**Auger ID/OD:**	N/A
Operator:	Rich Leonard/Jay O'Leary	**Datum:**	NAVD88	**Sampler:**	Split Spoon
Logged By:	Joshua Szmyt	**Rig Type:**	CME-45 (Skid)	**Hammer Wt./Fall:**	140/30"
Date Start/Finish:	8/19/13 - 8/19/13	**Drilling Method:**	Cased Washed	**Core Barrel:**	NQ2
Boring Location:	N 633217, E 1751966	**Casing ID/OD:**	4"/4-1/2" & 3"/3-1/2"	**Water Level*:**	River Boring
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	24/7	0.0 - 2.0	6-17-11-11	28	28	66			Medium dense, gray, fine to coarse, sandy GRAVEL, trace Silt. -ALLUVIAL DEPOSIT- (GM)	
							81				
							131				
							108				
							388	121.2		4.5	
5	R1	57/51	5.5 - 10.3	RQD = 5%			NQ			Top of Bedrock at 4.5' bgs. Advanced roller bit to 5.5' bgs and set 3" ID casing. Hard, fresh, fine grained, gray/white PHYLLITE. Primary joints are closely spaced, high angle to moderately dipping, planar to undulating, rough to smooth, fresh to discolored, very tight to tight. Secondary joints are closely to very closely spaced, moderately dipping, planar to stepped, rough to smooth, discolored, partially open to open. Rock Mass Quality = Very Poor. Rock Core Times (min/ft): 6.5, 5.0, 5.25, 6.0, 6.0	
10	R2	60/56	10.3 - 15.3	RQD = 54%						Hard, fresh to slightly weathered, fine grained, gray/ white PHYLLITE. Joints are very closely to moderately spaced, low to high angle, undulating to planar, fresh to discolored, partially open to wide. Rock Mass Quality = Fair. Rock Core Times (min/ft): 5.5, 9.0, 6.75, 11.0, 8.0	q_p = 3.49 ksi
15	R3	60/51	15.3 - 20.3	RQD = 48%						Hard, fresh to slightly weathered, fine grained, gray/ white PHYLLITE. Joints are closely spaced, low to high angle, planar, rough to smooth, discolored, open. Rock Mass Quality = Poor. Rock Core Times (min/ft): 9.5, 7.25, 7.75, 9.25, 8.5	
20								105.4		20.3 **Bottom of Exploration at 20.30 feet below ground surface.**	
25											

Remarks:

1. Cobbles from 2.5' bgs to 4.5' bgs (El 123.2' to 121.2').
2. Encountered Bedrock at El 121.2' and advanced roller bit to El 120.2' before coring. Set 3"-ID casing before coring.
3. Depth of water = 2.3'.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Penobscot River Bridge, Route 155	**Location:**	Howland, Maine	**Boring No.:**	BB-HEPR-203
				PIN:	16705.00
Driller:	Maine Test Boring	**Elevation (ft.)**	123.0' (mudline)	**Auger ID/OD:**	N/A
Operator:	Rich Leonard/Jay O'Leary	**Datum:**	NAVD88	**Sampler:**	Split Spoon
Logged By:	Joshua Szmyt	**Rig Type:**	CME-45 (Skid)	**Hammer Wt./Fall:**	140/30"
Date Start/Finish:	8/20/13 - 8/22/13	**Drilling Method:**	Cased Washed	**Core Barrel:**	NQ2
Boring Location:	N 633266, E 1762136	**Casing ID/OD:**	4"/4-1/2" & 3"/3-1/2"	**Water Level*:**	River Boring
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	5-13-11-17	24	24	29			Medium dense, brown, gravelly, fine to coarse SAND, trace Silt. (SM)	
							38				
							49				
							36				
							39				
5							41	118.0		5.0	
	2D	24/5	6.0 - 8.0	19-17-48-30	65	65	34			Very dense, gray, fine to coarse SAND, some Gravel, little Silt. -ALLUVIAL DEPOSIT- (SW-SM)	
							54				
							68				
							83				
10							99				
	3D	24/11	11.0 - 13.0	9-27-19-24	46	46	41			Dense, gray, fine to medium SAND, little Gravel, trace Silt. (SM)	
							48				
							77				
							111				
15	4D	24/10	15.0 - 17.0	9-22-36-40	58	58	50			Very dense, gray, fine to coarse SAND, some Gravel, trace Silt. (SW-SM)	SW-SM A-1-b
							76				
							119				
							111				
							112				
20	5D	24/14	20.0 - 22.0	20-28-62-102	90	90	71	103.0		20.0 Very dense, gray, sandy GRAVEL, trace Silt. -GLACIAL TILL- (SM)	
							262				
							69				
							144				
							281				
25											

Remarks:

1. Rock in tip of sample 1D split spoon.
2. Encountered Cobbles from approximately 2' bgs to 5.5' bgs.
3. Increased drilling effort after sample 2D.
4. Increased drilling effort at 13' bgs.
5. Encountered casing refusal at 21.6' bgs. Advanced roller bit to 24.0' bgs and continued driving casing to 24.5' bgs.

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: Penobscot River Bridge, Route 155
Location: Howland, Maine

Boring No.: BB-HEPR-203
PIN: 16705.00

Driller:	Maine Test Boring	**Elevation (ft.)**	123.0' (mudline)	**Auger ID/OD:**	N/A
Operator:	Rich Leonard/Jay O'Leary	**Datum:**	NAVD88	**Sampler:**	Split Spoon
Logged By:	Joshua Szmyt	**Rig Type:**	CME-45 (Skid)	**Hammer Wt./Fall:**	140/30"
Date Start/Finish:	8/20/13 - 8/22/13	**Drilling Method:**	Cased Washed	**Core Barrel:**	NQ2
Boring Location:	N 633266, E 1762136	**Casing ID/OD:**	4"/4-1/2" & 3"/3-1/2"	**Water Level*:**	River Boring
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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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	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	1/0	25.0 - 25.1	100/1"	--		>100	97.9		No Recovery. Split spoon refusal at 25.1' bgs. —25.1— Top of Bedrock at 25.1' bgs. Advanced roller bit to 27.0' and set 3" ID casing.	
	R1	12/9	27.0 - 28.0	RQD = 0%			NQ			Hard, fresh, fine grained, gray/white, PHYLLITE. Joints are closely spaced, moderately dipping, fresh to discolored, planar, rough, open. Rock Mass Quality = Very Poor Rock Core Times (min/ft): 3.0	
	R2	60/60	28.0 - 33.0	RQD = 61%						Hard, fresh, fine grained, gray/white, PHYLLITE. Joints are closely to moderately spaced, high angle to vertical, undulating to planar, rough, fresh to discolored, tight. Rock Mass Quality = Fair Rock Core Times (min/ft): 3.25, 4.0, 3.5, 2.5, 2.75	
30											
	R3	60/60	33.0 - 38.0	RQD = 67%						Hard, fresh to discolored, fine grained, gray/white, PHYLLITE. Primary joints are moderately spaced, high angle, undulating to stepped, rough, discolored to fresh, open. Secondary joints are moderately spaced, horizontal to low angle, discolored, open. Rock Mass Quality = Fair Rock Core Times (min/ft): 3.5, 4.25, 3.75, 2.0, 2.0	
35											
								85.0		—38.0— **Bottom of Exploration at 38.00 feet below ground surface.**	
40											
45											
50											

Remarks:

1. Rock in tip of sample 1D split spoon.
2. Encountered Cobbles from approximately 2' bgs to 5.5' bgs.
3. Increased drilling effort after sample 2D.
4. Increased drilling effort at 13' bgs.
5. Encountered casing refusal at 21.6' bgs. Advanced roller bit to 24.0' bgs and continued driving casing to 24.5' bgs.

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: Penobscot River Bridge, Route 155 Location: Howland, Maine	Boring No.: BB-HEPR-204 PIN: 16705.00

Driller:	Maine Test Boring	**Elevation (ft.)**	123.2' (mudline)	**Auger ID/OD:**	N/A
Operator:	Rich Leonard/Brad Enos	**Datum:**	NAVD88	**Sampler:**	Split Spoon
Logged By:	Joshua Szmyt	**Rig Type:**	CME-45 (Skid)	**Hammer Wt./Fall:**	140/30"
Date Start/Finish:	8/22/13 - 8/26/13	**Drilling Method:**	Cased Washed	**Core Barrel:**	NQ2
Boring Location:	N 633347, E 1762385	**Casing ID/OD:**	4"/4-1/2" & 3"/3-1/2"	**Water Level*:**	River Boring
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	4-8-6-7	14	14	9			Medium dense, light brown, medium to coarse SAND, some Gravel, trace Silt. -ALLUVIAL DEPOSIT- (SM)	
							19				
							25				
							27				
	2D	24/11	4.0 - 6.0	16-10-15-16	25	25	16			Medium dense, light brown, gravelly, medium to coarse SAND, trace Silt. (SM)	
5							32				
							38				
							117				
							102	115.2		8.0	
	3D	6/0	9.0 - 9.5	132/6"	--		WA			No Recovery.	
10											
	4D	13/6	11.0 - 12.1	14/45/100/1"	--					Very dense, gray, fine to coarse SAND, little Gravel, trace Silt. (SM)	
	5D	24/5	14.0 - 16.0	20-24-24-23	48	48	54			Dense, gray, fine to coarse SAND, some Gravel, trace Silt. (SM)	
15							78				
							84				
							88				
							130				
	6D	24/12	19.0 - 21.0	14-21-24-53	45	45	59			Dense, gray, fine to coarse SAND, some Gravel, trace Silt. -ALLUVIAL DEPOSIT- (SM)	
20							115				
							113				
							151				
							150				
	7D	24/14	24.0 - 26.0	27-32-37-29	69	69	101			Very dense, gray, fine to coarse SAND, some Gravel, trace Silt. (SW-SM)	SW-SM A-1-b
25											

Remarks:

1. Rock in tip of split spoon for sample 2D.
2. Wash water change from brown to gray at 8' bgs.
3. Encountered Cobbles from 9' bgs to 11' bgs. Probably pushed Cobble with sample 3D. Advance roller bit from 9' bgs to 11' bgs before advancing casing to 11' bgs.
4. Encountered Boulder 12.1' bgs (0.8' thick). ler bit to 50.6' bgs and set 3" ID casing.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Penobscot River Bridge, Route 155	**Location:**	Howland, Maine	**Boring No.:**	BB-HEPR-204
				PIN:	16705.00
Driller:	Maine Test Boring	**Elevation (ft.)**	123.2' (mudline)	**Auger ID/OD:**	N/A
Operator:	Rich Leonard/Brad Enos	**Datum:**	NAVD88	**Sampler:**	Split Spoon
Logged By:	Joshua Szmyt	**Rig Type:**	CME-45 (Skid)	**Hammer Wt./Fall:**	140/30"
Date Start/Finish:	8/22/13 - 8/26/13	**Drilling Method:**	Cased Washed	**Core Barrel:**	NQ2
Boring Location:	N 633347, E 1762385	**Casing ID/OD:**	4"/4-1/2" & 3"/3-1/2"	**Water Level*:**	River Boring
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.
25							96				
							92				
							183				
							WA				
	8D	24/14	29.0 - 31.0	41-42-36-54	78	78				Very dense, gray, gravelly, fine to coarse SAND, trace Silt. (GM)	
30											
	9D	12/0	34.0 - 35.0	72/100/6"	--			89.2		34.0 No Recovery.	
35											
	10D	15/12	36.0 - 37.3	32/70/73/3"	--					Very dense, gray, fine to medium, sandy SILT, some Gravel. (ML)	
	11D	15/11	39.0 - 40.3	47/67/58/3"	--					Very dense, gray, fine to medium, sandy SILT, little Gravel. -GLACIAL TILL- (ML)	
40											
	12D	24/12	44.0 - 46.0	28-35-34-46	73	73		79.2		44.0 Very dense, gray, fine SAND, little Silt, trace Gravel. -GLACIAL TILL- (SM)	
45											
	13D	24/11	49.0 - 51.0	8-56-63-89	>100		73			Very dense, gray, fine to medium SAND, little Silt, trace Gravel. (SM)	
50											

Remarks:

1. Rock in tip of split spoon for sample 2D.
2. Wash water change from brown to gray at 8' bgs.
3. Encountered Cobbles from 9' bgs to 11' bgs. Probably pushed Cobble with sample 3D. Advance roller bit from 9' bgs to 11' bgs before advancing casing to 11' bgs.
4. Encountered Boulder 12.1' bgs (0.8' thick). ler bit to 50.6' bgs and set 3" ID casing.

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: Penobscot River Bridge, Route 155
Location: Howland, Maine

Boring No.: BB-HEPR-204
PIN: 16705.00

Driller:	Maine Test Boring	**Elevation (ft.)**	123.2' (mudline)	**Auger ID/OD:**	N/A
Operator:	Rich Leonard/Brad Enos	**Datum:**	NAVD88	**Sampler:**	Split Spoon
Logged By:	Joshua Szmyt	**Rig Type:**	CME-45 (Skid)	**Hammer Wt./Fall:**	140/30"
Date Start/Finish:	8/22/13 - 8/26/13	**Drilling Method:**	Cased Washed	**Core Barrel:**	NQ2
Boring Location:	N 633347, E 1762385	**Casing ID/OD:**	4"/4-1/2" & 3"/3-1/2"	**Water Level*:**	River Boring
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							177				
							WA				
	14D	24/13	54.0 - 56.0	43-47-61-69	>100		86			Very dense, gray, fine to coarse SAND, some Gravel, little Silt. (SM)	SM A-1-b
55							151				
	R1	60/60	56.6 - 61.6	RQD = 56%			NQ	66.8		56.4 Top of Bedrock at 56.4' bgs. Advanced roller bit to 56.6' bgs and set 3" ID casing. Hard, fresh, fine grained, white/gray PHYLLITE. Joints are closely to moderately spaced, high angle to vertical, undulating, smooth, fresh to discolored, tight to open. Rock Mass Quality = Fair Rock Core Times (min/ft): 1.25, 1.5, 3.5, 3.0, 2.25	
60											
	R2	60/58	61.6 - 66.6	RQD = 68%						Hard, fresh, fine grained, white/gray PHYLLITE. Joints are closely spaced, moderately dipping, undulating, rough, fresh to discolored, open to moderately wide. Rock Mass Quality = Fair Rock Core Times (min/ft): 2.0, 2.0, 3.5, 3.5, 4.75	q_p= 4.18 ksi
65											
								56.6		66.6 **Bottom of Exploration at 66.60 feet below ground surface.**	
70											
75											

Remarks:

1. Rock in tip of split spoon for sample 2D.
2. Wash water change from brown to gray at 8' bgs.
3. Encountered Cobbles from 9' bgs to 11' bgs. Probably pushed Cobble with sample 3D. Advance roller bit from 9' bgs to 11' bgs before advancing casing to 11' bgs.
4. Encountered Boulder 12.1' bgs (0.8' thick). ler bit to 50.6' bgs and set 3" ID casing.

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:	Penobscot River Bridge, Route 155
Location:	Howland, Maine
Boring No.:	BB-HEPR-205
PIN:	16705.00

Driller:	Maine Test Boring	**Elevation (ft.)**	122.3' (mudline)	**Auger ID/OD:**	N/A
Operator:	Brad Enos	**Datum:**	NAVD88	**Sampler:**	Split Spoon
Logged By:	Joshua Szmyt	**Rig Type:**	CME-45 (Skid)	**Hammer Wt./Fall:**	140/30"
Date Start/Finish:	8/26/13 - 8/26/13	**Drilling Method:**	Cased Washed	**Core Barrel:**	NQ2
Boring Location:	N 633424, E 1762636	**Casing ID/OD:**	4"/4-1/2" & 3"/3-1/2"	**Water Level*:**	River Boring
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.
0	1D	24/3	0.0 - 2.0	5-11-17-30	28	28				Medium dense, light brown, fine to medium, sandy GRAVEL, trace Silt. (GM)	
							32				
							40	120.3		2.0	
							132				
							261				
5	2D	24/15	5.0 - 7.0	16-20-27-28	47	47	69			Dense, light brown, fine to medium SAND, some Gravel, little Silt. (SM)	
							44				
							33				
							116				
							171				
10	3D	24/12	10.0 - 12.0	15-24-25-21	49	49	31			Dense, light brown, fine to coarse SAND, trace Gravel, trace Silt. -ALLUVIAL DEPOSIT- (SM)	
							76				
							107				
							138				
							161				
15							178				
	4D	24/11	16.0 - 18.0	13-22-23-18	45	45	WA			Dense, light brown, fine to coarse SAND, some Gravel, trace Silt. (SW-SM)	
20							148				
	5D	24/16	21.0 - 23.0	14-19-22-32	41	41	183			Dense, light brown, fine SAND, trace Gravel, trace Silt. (SM)	
							WA				
25											

Remarks:

1. Increased drilling effort at 33' bgs.

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* 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: Penobscot River Bridge, Route 155
Location: Howland, Maine

Boring No.: BB-HEPR-205
PIN: 16705.00

Driller:	Maine Test Boring	**Elevation (ft.)**	122.3' (mudline)	**Auger ID/OD:**	N/A
Operator:	Brad Enos	**Datum:**	NAVD88	**Sampler:**	Split Spoon
Logged By:	Joshua Szmyt	**Rig Type:**	CME-45 (Skid)	**Hammer Wt./Fall:**	140/30"
Date Start/Finish:	8/26/13 - 8/26/13	**Drilling Method:**	Cased Washed	**Core Barrel:**	NQ2
Boring Location:	N 633424, E 1762636	**Casing ID/OD:**	4"/4-1/2" & 3"/3-1/2"	**Water Level*:**	River Boring
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.
25											
	6D	24/9	26.0 - 28.0	43-49-31-29	80	80		95.7		Top 7": Dense, light brown, fine SAND, little Gravel, trace Silt. (SM)	
										26.6	
										Bottom 2": Dense, gray, fine SAND, little Gravel, little Silt. (SM)	
30											
	7D	24/7	31.0 - 33.0	86-72-62-73	>100					Very dense, gray, fine to coarse SAND, some Silt, little Gravel. -GLACIAL TILL- (SM)	
35											
	8D	24/10	36.0 - 38.0	29-38-40-54	78	78				Very dense, gray, silty, fine to medium SAND, trace Gravel. (SM)	
40											
	9D	24/14	41.0 - 43.0	50-42-54-63	96	96				Very dense, gray, silty, fine to medium SAND, little Gravel. (SM)	SM A-4
45											
	10D	24/2	46.0 - 48.0	45-62-63-83	>100					Very dense, gray, gravelly, fine to medium SAND, little Silt. (SM)	
50											

Remarks:

1. Increased drilling effort at 33' bgs.

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Penobscot River Bridge, Route 155 Location: Howland, Maine	Boring No.: BB-HEPR-205 PIN: 16705.00

Driller:	Maine Test Boring	**Elevation (ft.)**	122.3' (mudline)	**Auger ID/OD:**	N/A
Operator:	Brad Enos	**Datum:**	NAVD88	**Sampler:**	Split Spoon
Logged By:	Joshua Szmyt	**Rig Type:**	CME-45 (Skid)	**Hammer Wt./Fall:**	140/30"
Date Start/Finish:	8/26/13 - 8/26/13	**Drilling Method:**	Cased Washed	**Core Barrel:**	NQ2
Boring Location:	N 633424, E 1762636	**Casing ID/OD:**	4"/4-1/2" & 3"/3-1/2"	**Water Level*:**	River Boring
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.
50											
	11D	9/6	51.0 - 51.8	42/70/2"	--			70.6		Very dense, gray, silty, fine to medium SAND, little Gravel. (SM) 51.7	
	R1	60/52	52.0 - 57.0	RQD = 87%			NQ			Top of Bedrock at 51.7' bgs. Advanced roller bit to 52.0' bgs and set 3" ID casing. Hard, fresh, fine grained, gray/white PHYLLITE. Joints are very closely to closely spaced, moderately dipping, undulating, rough, fresh to discolored, tight to partially open. Rock Mass Quality = Good Rock Core Times (min/ft): 6.0, 6.5, 6.5, 5.5, 3.0	
55											
	R2	53/52	57.0 - 61.4	RQD = 73%						Hard, fresh, fine grained, gray/white PHYLLITE. Joints are very closely to moderately spaced, moderately dipping, undulating, rough, fresh to discolored, tight to moderately wide. Rock Mass Quality = Fair Rock Core Times (min/ft): 3.0, 3.5, 3.75, 4.0, 3.5	
60											
								60.9		61.4 **Bottom of Exploration at 61.40 feet below ground surface.**	
65											
70											
75											

Remarks:

1. Increased drilling effort at 33' bgs.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Penobscot River Bridge, Route 155
Location: Howland, Maine

Boring No.: BB-HEPR-206
PIN: 16705.00

Driller:	Maine Test Boring	**Elevation (ft.)**	134.8'	**Auger ID/OD:**	N/A
Operator:	Brad Enos	**Datum:**	NAVD88	**Sampler:**	Split Spoon
Logged By:	Joshua Szmyt	**Rig Type:**	CME-45 (Skid)	**Hammer Wt./Fall:**	140/30"
Date Start/Finish:	8/28/13 - 8/28/13	**Drilling Method:**	Cased Washed	**Core Barrel:**	NQ2
Boring Location:	N 633514, E 1762794	**Casing ID/OD:**	3"/3-1/2"	**Water Level*:**	NA
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
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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.
0	1D	24/9	0.0 - 2.0	1-2-4-5	6	6	4			Loose, olive/brown, fine to medium SAND, little Gravel, trace Silt, trace organics. -ALLUVIAL DEPOSIT- (SM)	
							10				
							19				
							44				
							WA	130.3		4.5	
5	2D	24/11	5.0 - 7.0	28-48-42-53	90	90				Dense, brown and gray, highly weathered/decomposed PHYLLITE.	
10											
								123.3		11.5	
										Bottom of Exploration at 11.50 feet below ground surface.	
15											
20											
25											

Remarks:

1. Drove casing with 140 lb. hammer.
2. Increased drilling effort at 4.5' bgs.
3. Increased drilling effort at 11.5' bgs and wash water turned from brown to gray, probable top of competent Bedrock.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Penobscot River Bridge, Route 155
Location: Howland, Maine

Boring No.: BB-HEPR-207
PIN: 16705.00

Driller:	Maine Test Boring	**Elevation (ft.)**	136.9'	**Auger ID/OD:**	N/A
Operator:	Brad Enos	**Datum:**	NAVD88	**Sampler:**	Split Spoon
Logged By:	Joshua Szmyt	**Rig Type:**	CME-45 (Skid)	**Hammer Wt./Fall:**	140/30"
Date Start/Finish:	8/28/13 - 8/28/13	**Drilling Method:**	Cased Washed	**Core Barrel:**	NQ2
Boring Location:	N 633512, E 1762810	**Casing ID/OD:**	3"/3-1/2"	**Water Level*:**	NA
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	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/0	0.0 - 2.0	1-4-4-5	8	8	WA			No Recovery.	
							34				
	2D	24/11	2.0 - 4.0	4-8-19-18	27	27	42			Medium dense, olive, damp, fine to medium SAND, some Silt, little Gravel. (SM) -FILL- (Possible reworked material)	
							--	132.9		4.0	
							WA				
5	3D	21/11	5.0 - 6.8	30-62-78-60/3"	>100					Very dense, brown to gray, fine to coarse SAND, some Gravel, little Silt. (Decomposed/highly weathered Bedrock) (SM)	SM A-1-b
	R1	60/12	6.8 - 11.8	RQD = 0%				130.1		6.8	
							NQ			Top of Bedrock at 6.8' bgs. Highly weathered PHYLLITE.	
10											
	R2	48/21	11.8 - 15.8	RQD = 0%						Hard, fresh, fine grained, gray/white PHYLLITE. Primary joints are very closely to closely spaced, high angle to vertical, undulating to planar, rough, fresh to discolored, tight. Secondary joints are very closely spaced, moderately dipping, undulating to planar, rough, discolored, tight. Rock Mass Quality = Very Poor Rock Core Times (min/ft): 7.25, 5.75, 4. 5, 4.0	
15	R3	60/60	15.8 - 20.8	RQD = 20%				122.1		14.8	
										Hard, fresh, fine grained, gray/white PHYLLITE. Joints are very closely to moderately spaced, moderately dipping to vertical, undulating to planar, fresh, open to wide, with Silt and Sand deposits. Rock Mass Quality = Very Poor Rock Core Times (min/ft): 2.5, 2.5, 2.75, 3.25, 3.5	
20	R4	60/51	20.8 - 25.8	RQD = 22%						R4 same as R3. Rock Mass Quality = Very Poor Rock Core Times (min/ft): 2.25, 1.5, 1.75, 2.5, 2.75	
25											

Remarks:

1. Drove casing with 140 lb. hammer.
2. Rock fragments (decomposed/highly weathered Bedrock) in tip of split spoon for sample 3D.
3. Wash water change from brown to gray during coring at approximately 14.8' bgs.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Penobscot River Bridge, Route 155
Location: Howland, Maine

Boring No.: BB-HEPR-207
PIN: 16705.00

Driller:	Maine Test Boring	**Elevation (ft.)**	136.9'	**Auger ID/OD:**	N/A
Operator:	Brad Enos	**Datum:**	NAVD88	**Sampler:**	Split Spoon
Logged By:	Joshua Szmyt	**Rig Type:**	CME-45 (Skid)	**Hammer Wt./Fall:**	140/30"
Date Start/Finish:	8/28/13 - 8/28/13	**Drilling Method:**	Cased Washed	**Core Barrel:**	NQ2
Boring Location:	N 633512, E 1762810	**Casing ID/OD:**	3"/3-1/2"	**Water Level*:**	NA
Hammer Efficiency Factor:	0.6	**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)
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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.
25	R5	55/50	25.8 - 30.3	RQD = 32%						Hard, fresh, fine grained, gray/white PHYLLITE. Joints are extremely closely to moderately spaced, moderately dipping to vertical, planar, rough, open to wide. Rock Mass Quality = Poor Rock Core Times (min/ft): 1.75, 2.25, 4.0, 7.5, 4.0	
30								106.6		30.3 **Bottom of Exploration at 30.30 feet below ground surface.**	
35											
40											
45											
50											

Remarks:

1. Drove casing with 140 lb. hammer.
2. Rock fragments (decomposed/highly weathered Bedrock) in tip of split spoon for sample 3D.
3. Wash water change from brown to gray during coring at approximately 14.8' bgs.

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.

APPENDIX C

LABORATORY TESTING RESULTS

Appendix C

Previous Laboratory Testing Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Howland-Enfield Project Number: 16705.00

Boring & Sample Identification Number	Northing (NAVD88)	Easting (NAVD88)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification Unified	Classification AASHTO	Classification Frost
BB-HEPR-101, 1D	633293	1761886	1.0-3.0	237550	1	3.9			SW-SM	A-1-b	0
BB-HEPR-101, 4D	633293	1761886	15.0-17.0	207064	1	21.8			SM	A-2-4	II
BB-HEPR-101, 6D	633293	1761886	25.0-25.8	207066	1	9.7			SM	A-2-4	II
BB-HEPR-102, 1D	633348.9	1762069	0.0-2.0	207067	1	10.2			GP	A-1-a	0
BB-HEPR-102, 4D	633348.9	1762069	16.0-18.0	207068	1	11.7			SW-SM	A-1-b	0
BB-HEPR-103, 1D	633430.5	1762334	0.0-2.0	207069	2	9.9			GW	A-1-a	0
BB-HEPR-103, 2D	633430.5	1762334	3.5-5.5	207070	2	7.7			GW	A-1-a	0
BB-HEPR-103, 5D	633430.5	1762334	18.5-20.5	207071	2	11.5			SW-SM	A-1-b	0
BB-HEPR-103, 7D	633430.5	1762334	28.5-30.0	207072	2	10.1			SW	A-1-a	0
BB-HEPR-103, 10D	633430.5	1762334	43.5-44.8	207073	2	7.9			SM	A-4	IV
BB-HEPR-104, 1D	633510	1762593	0.0-2.0	207074	3	10.1			GP	A-1-a	0
BB-HEPR-104, 2D	633510	1762593	7.0-8.33	207075	3	8.3			SM	A-1-b	II
BB-HEPR-104, 3D	633510	1762593	12.0-14.0	239827	3	15.1			SP-SM	A-1-b	0
BB-HEPR-104, 5D	633510	1762593	22.0-24.0	239828	3	10.3			SW-SM	A-1-a	0
BB-HEPR-104, 7D	633510	1762593	32.0-34.0	239829	3	11.6			ML	A-4	IV
BB-HEPR-104, 10D	633510	1762593	47.0-48.8	239830	3	8.4			SM	A-4	IV
BB-HEPR-105, 1D	633575.2	1762800	1.0-3.0	239831	4	2.7			SW-SM	A-1-b	0
BB-HEPR-105, 3D	633575.2	1762800	10.0-12.0	239832	4	7.5			SW-SM	A-1-b	0
BB-HEPR-105, 4D/B	633575.2	1762800	15.0-17.0	239833	4	24.1	33	12	CL	A-6	III
BB-HEPR-105, 5D	633575.2	1762800	20.0-22.0	239834	4	18.5			SM	A-2-4	II
BB-HEPR-105, 8D	633575.2	1762800	35.0-37.0	239835	4	9.1			SM	A-1-b	II

Classification of these soil samples is in accordance with AASHTO Classification System M-145-40. This classification is followed by the "Frost Susceptibility Rating" from zero (non-frost susceptible) to Class IV (highly frost susceptible). The "Frost Susceptibility Rating" is based upon the MaineDOT and Corps of Engineers Classification Systems.

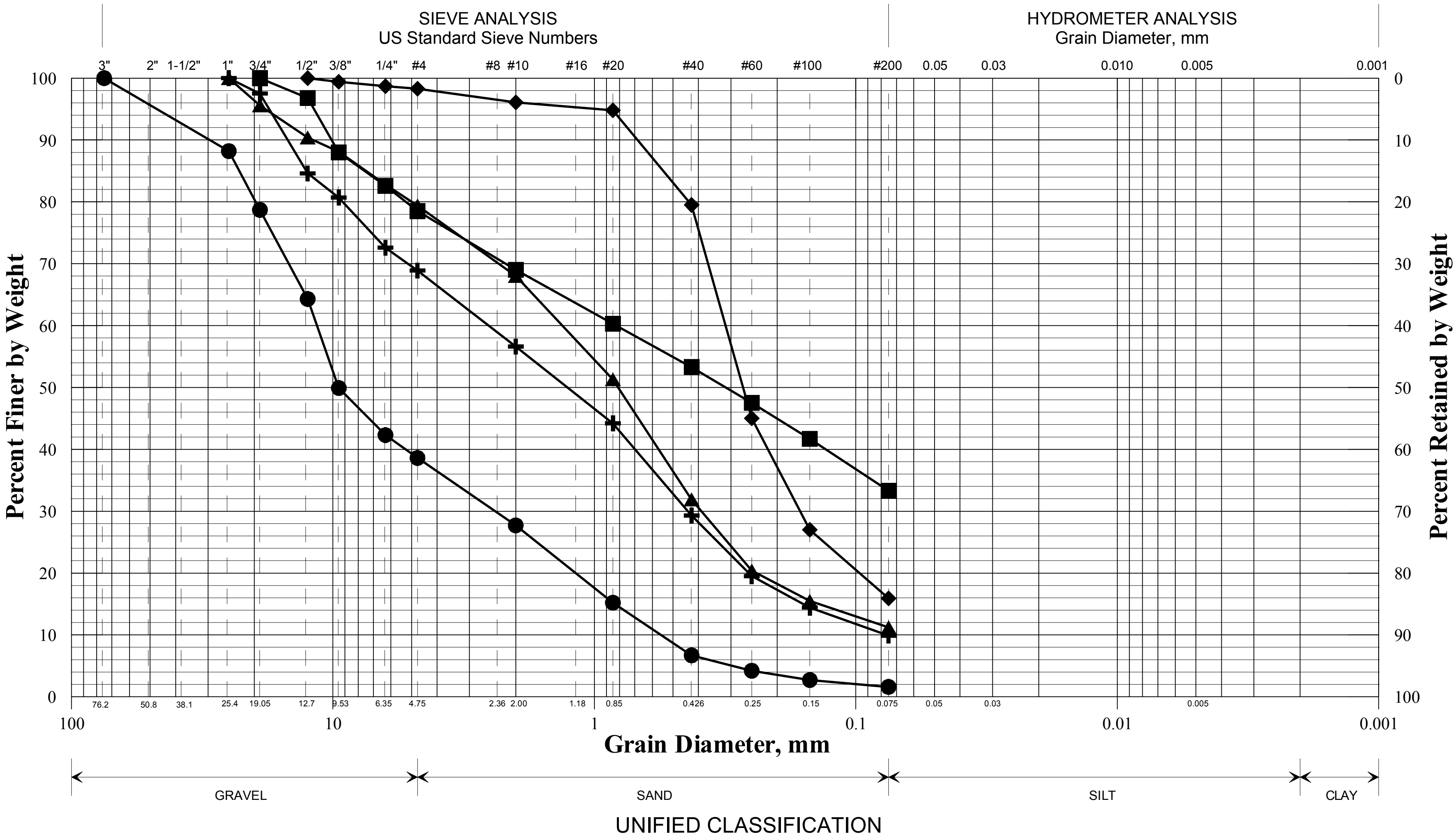
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

LL = Liquid limit as determined by AASHTO T 89-96 and/or ASTM D 4318-98

PI = Plasticity Index as determined by AASHTO 90-96 and/or ASTM D4318-98

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

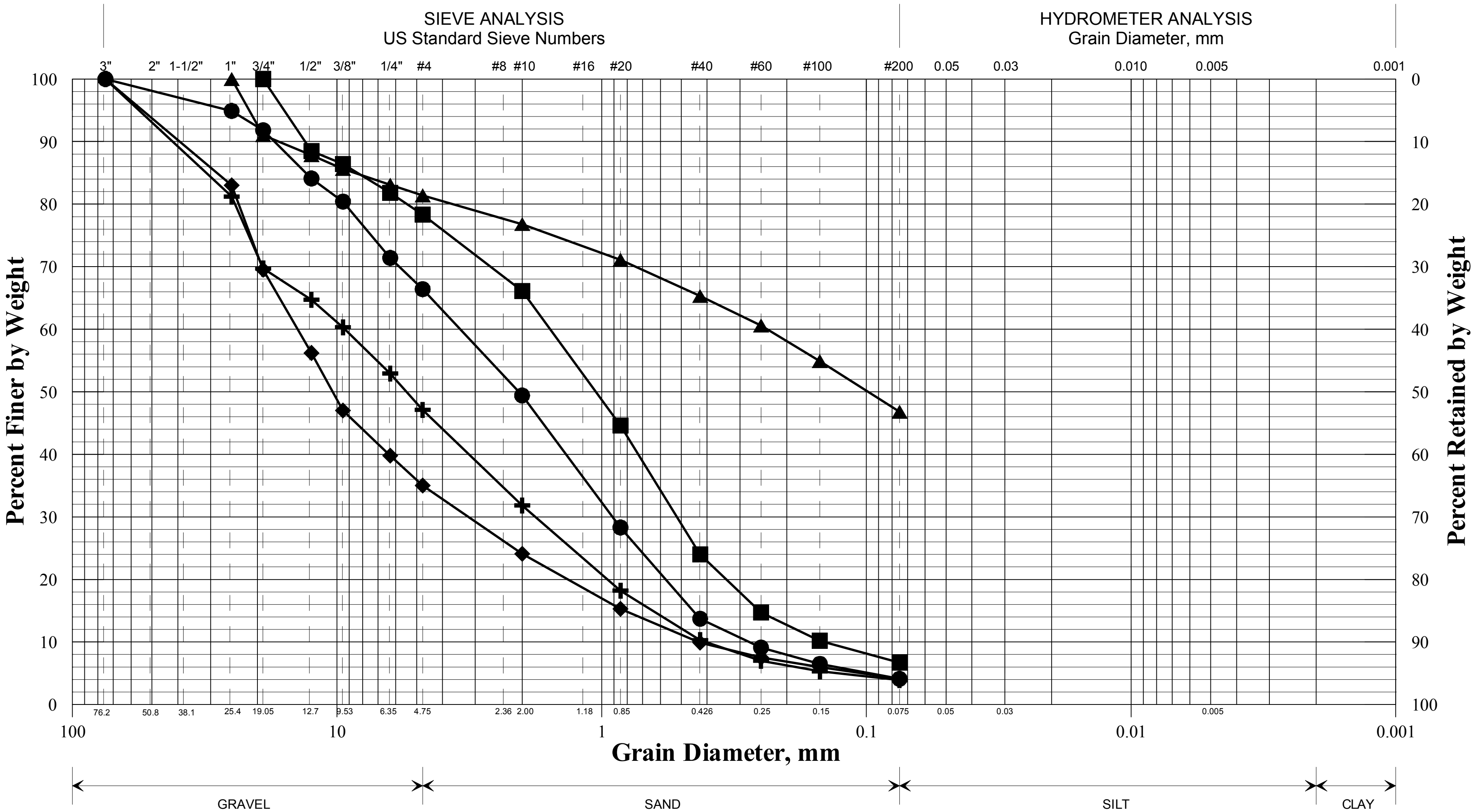


	Boring/Sample No.	Northing	Easting	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-HEPR-101/1D	633293	1761886	1.0-3.0	SAND, some gravel, trace silt.	3.9			
◆	BB-HEPR-101/4D	633293	1761886	15.0-17.0	SAND, little silt, trace gravel.	21.8			
■	BB-HEPR-101/6D	633293	1761886	25.0-25.8	SAND, some silt, some gravel.	9.7			
●	BB-HEPR-102/1D	633348.9	1762069	0.0-2.0	Sandy GRAVEL, trace silt.	10.2			
▲	BB-HEPR-102/4D	633348.9	1762069	16.0-18.0	SAND, some gravel, little silt.	11.7			
✖									

PIN
016705.00
Town
Howland,Enfield
Reported by/Date
WHITE, TERRY A 9/29/2010

SHEET 1

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

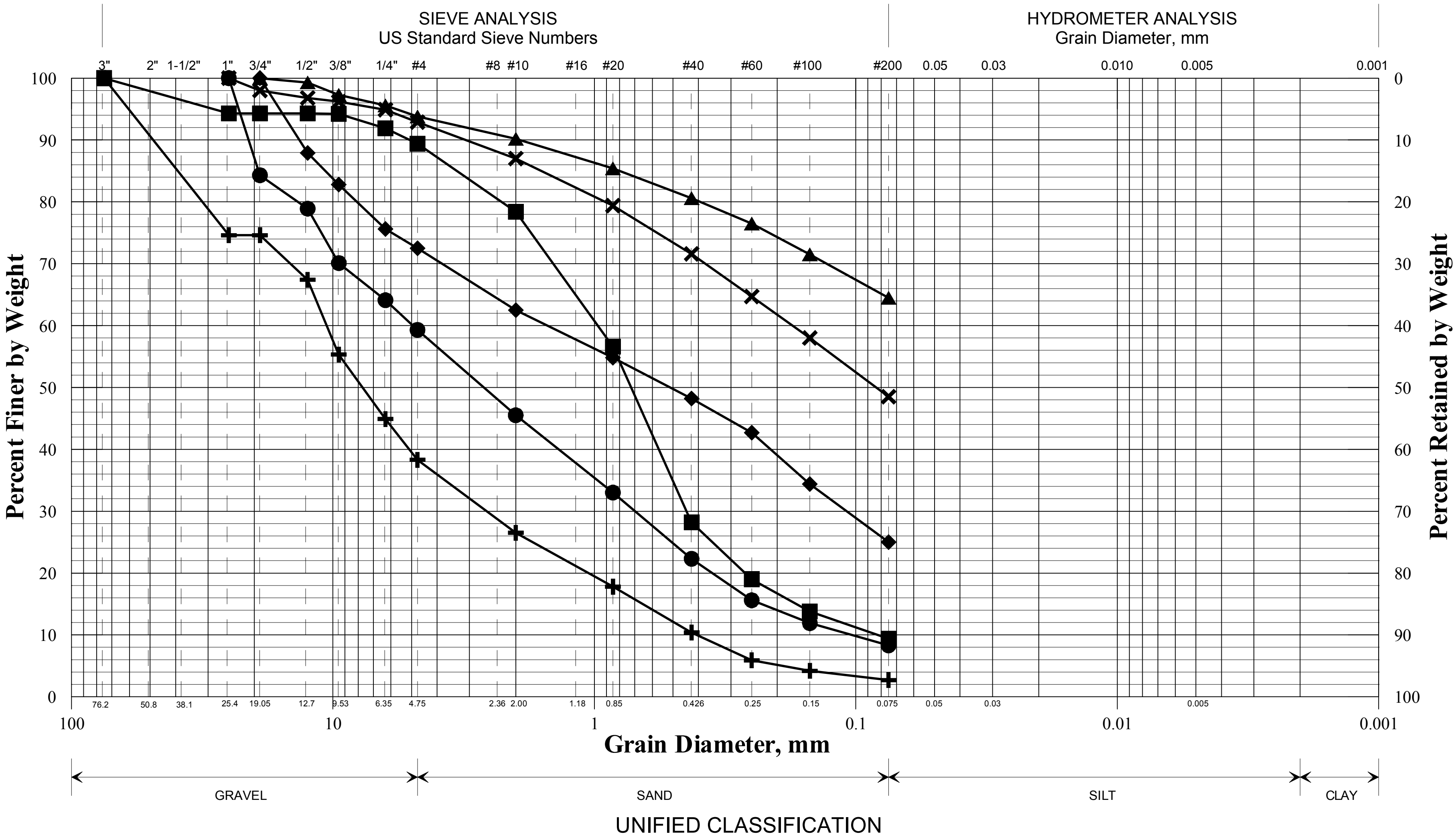


	Boring/Sample No.	Northing	Easting	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-HEPR-103/1D	633430.5	1762334	0.0-2.0	Sandy GRAVEL, trace silt.	9.9			
◆	BB-HEPR-103/2D	633430.5	1762334	3.5-5.5	GRAVEL, some sand, trace silt.	7.7			
■	BB-HEPR-103/5D	633430.5	1762334	18.5-20.5	SAND, some gravel, trace silt.	11.5			
●	BB-HEPR-103/7D	633430.5	1762334	28.5-30.0	SAND, some gravel, trace silt.	10.1			
▲	BB-HEPR-103/10D	633430.5	1762334	43.5-44.8	SILT, some sand, little gravel.	7.9			
✖									

PIN
016705.00
Town
Howland,Enfield
Reported by/Date
WHITE, TERRY A 9/29/2010

SHEET 2

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

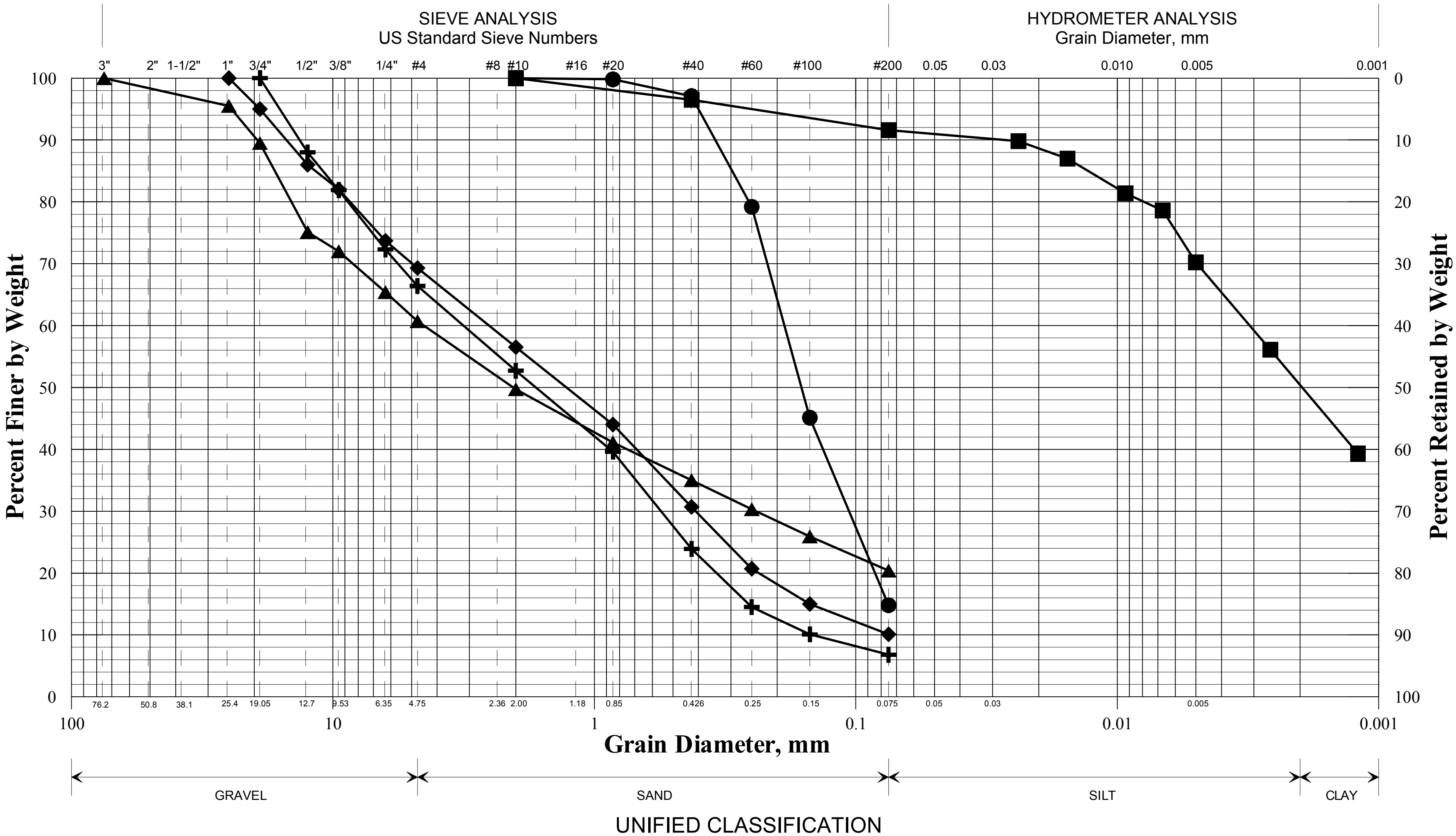


	Boring/Sample No.	Northing	Easting	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-HEPR-104/1D	633510	1762593	0.0-2.0	GRAVEL, some sand, trace silt.	10.1			
◆	BB-HEPR-104/2D	633510	1762593	7.0-8.3	SAND, some gravel, some silt.	8.3			
■	BB-HEPR-104/3D	633510	1762593	12.0-14.0	SAND, little gravel, trace silt.	15.1			
●	BB-HEPR-104/5D	633510	1762593	22.0-24.0	Gravelly SAND, trace silt.	10.3			
▲	BB-HEPR-104/7D	633510	1762593	32.0-34.0	SILT, some sand, trace gravel.	11.6			
✖	BB-HEPR-104/10D	633510	1762593	47.0-48.8	Sandy SILT, trace gravel.	8.4			

PIN
016705.00
Town
Howland,Enfield
Reported by/Date
WHITE, TERRY A 9/29/2010

State of Maine Department of Transportation

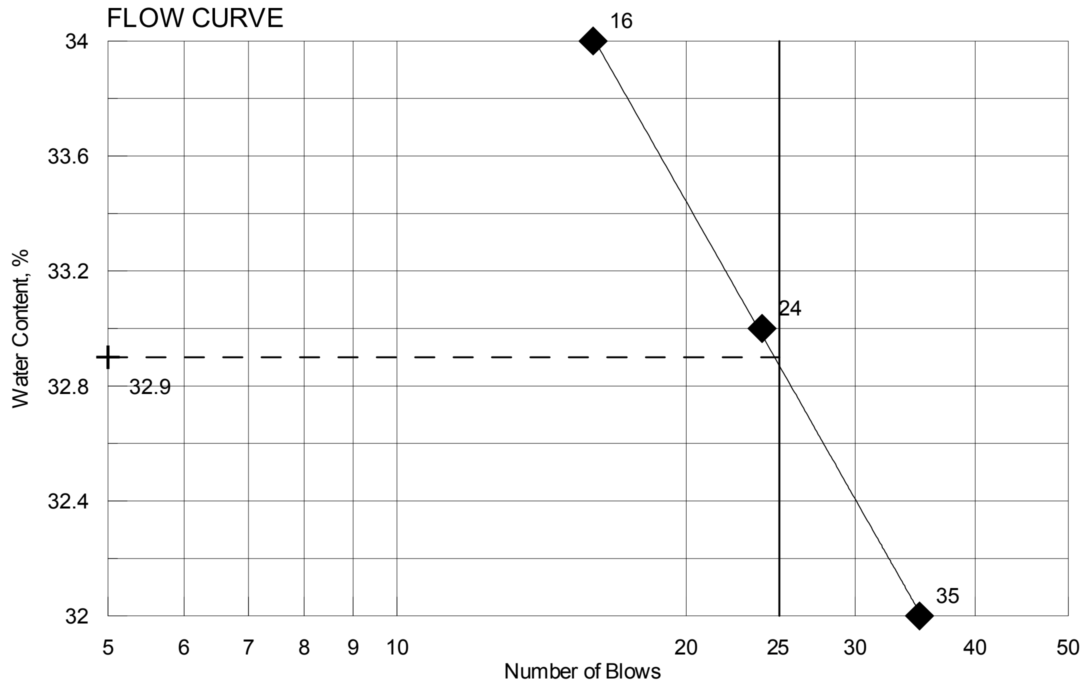
GRAIN SIZE DISTRIBUTION CURVE

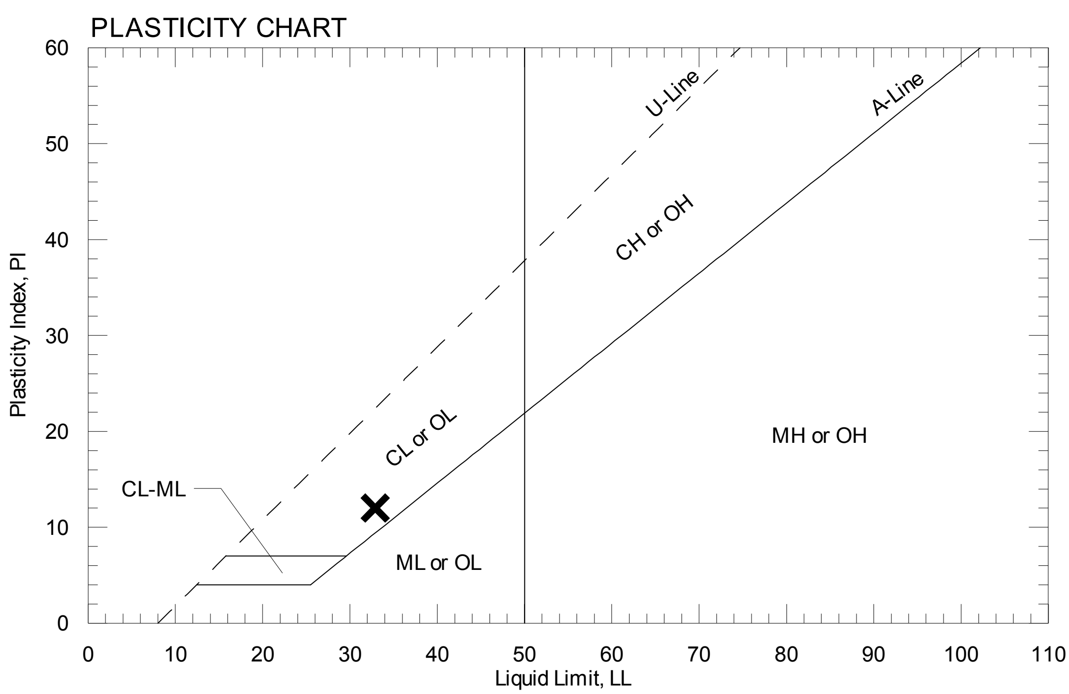


	Boring/Sample No.	Northing	Easting	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-HEPR-105/1D	633575.2	1762800	1.0-3.0	SAND, some gravel, trace silt.	2.7			
◆	BB-HEPR-105/3D	633575.2	1762800	10.0-12.0	SAND, some gravel, trace silt.	7.5			
■	BB-HEPR-105/4DB	633575.2	1762800	15.2-17.0	CLAY-SILT, trace sand.	24.1	33	21	12
●	BB-HEPR-105/5D	633575.2	1762800	20.0-22.0	SAND, little silt.	18.5			
▲	BB-HEPR-105/8D	633575.2	1762800	35.0-37.0	Gravelly SAND, little silt.	9.1			
✖									

PIN
016705.00
Town
Howland,Enfield
Reported by/Date
WHITE, TERRY A 10/20/2010

TOWN	Howland,Enfield	Reference No.	239833
PIN	016705.00	Water Content, %	24.1
Sampled	8/19/2010	Plastic Limit	21
Boring No./Sample No.	BB-HEPR-105/4DB	Liquid Limit	33
Station		Plasticity Index	12
Depth	15.2-17.0	Tested By	BBURR





Appendix C

GZA Laboratory Testing Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

THIELSCH ENGINEERING

Penobscot River Bridge

MDOT Project Number:

Town(s): Howland, ME

GZA Project Number: 09.0025796.00

Boring & Sample Identification Number	Station (Feet)	Sample No.	Depth (Feet)	Reference Number	Organic %	W.C.	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-HEPR-201		2D	5-7						SM	A-1-b	II
BB-HEPR-203		4D	15-17						SW-SM	A-1-b	0
BB-HEPR-204		7D	24-26						SW-SM	A-1-b	0
BB-HEPR-204		14D	54-56						SM	A-1-b	II
BB-HEPR-205		9D	41-43						SM	A-4	II
BB-HEPR-207		3D	5-6.8						SM	A-1-b	II

Classification of these soil samples is in accordance with AASHTO Classification System M-145-40. This classification is followed by the "Frost Susceptibility Rating" from zero (non-frost susceptible) to Class IV (highly frost susceptible). The "Frost Susceptibility Rating" is based upon the MDOT and Corps of Engineers Classification Systems.

GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

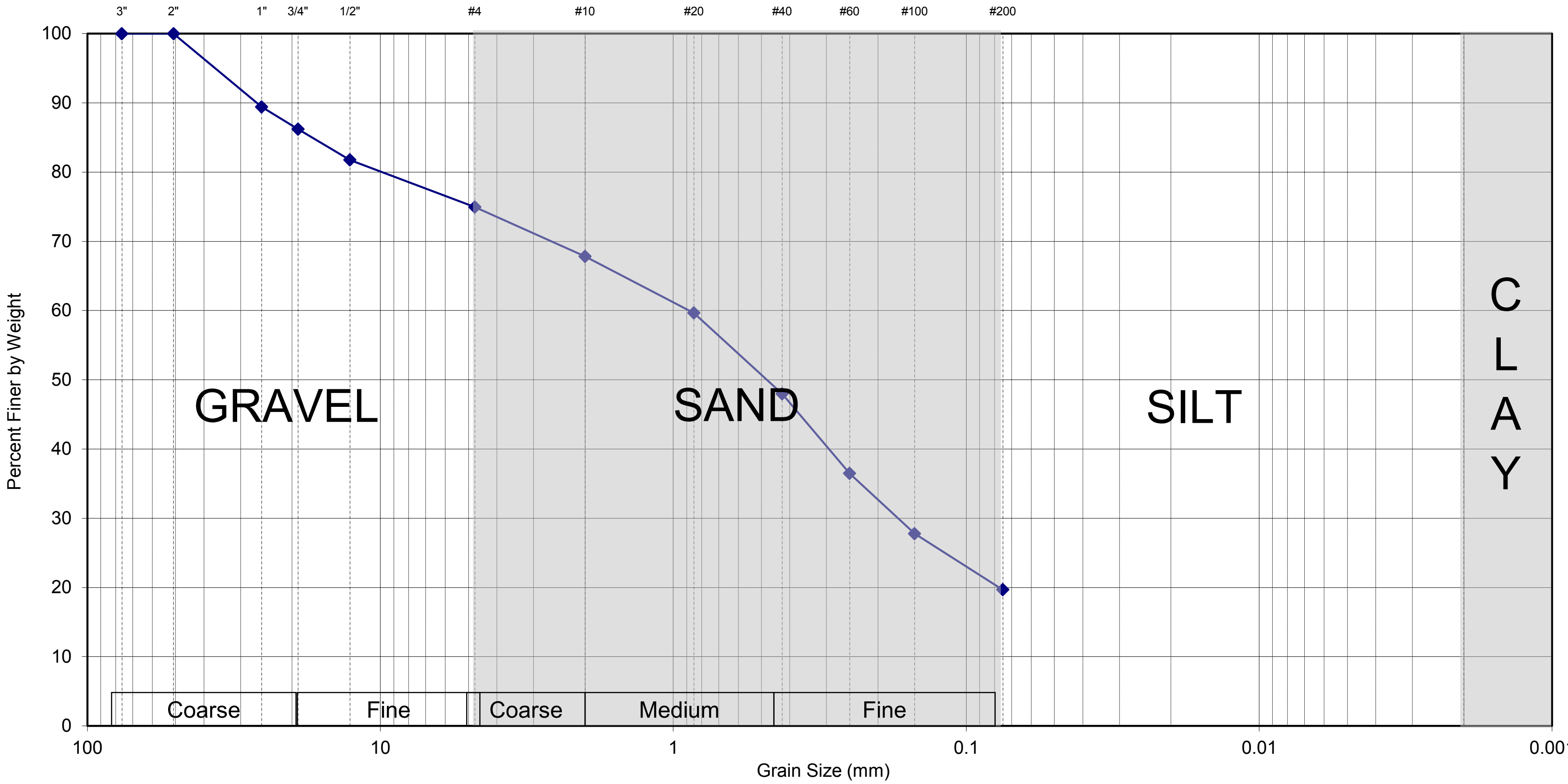
WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

LL = Liquid limit as determined by AASHTO T 89-96 and/or ASTM D 4318-98

PI = Plasticity Index as determined by AASHTO 90-96 and/or ASTM D4318-98

ASTM D422

U.S. STANDARD SIEVE AND HYDROMETER



Gravel	Sand	Fines
25.1%	55.3%	19.7%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
3	BB-HEPR-201	2D	5-7'	Brown f-c SAND, some Gravel, little Silt (SM)				

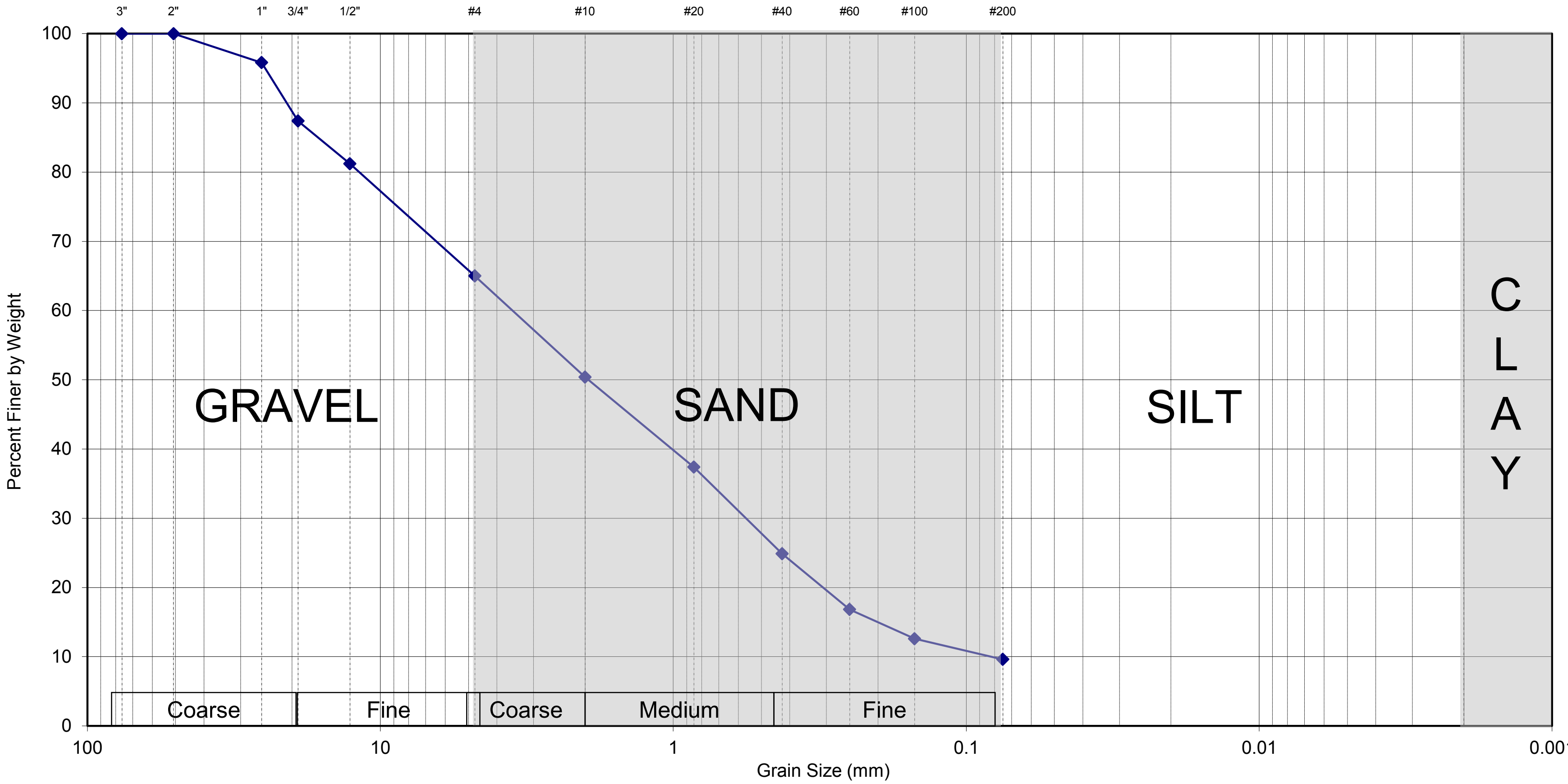
Sieve Size	% Passing
¾"	86.2
½"	81.7
#4	74.9
#10	67.8
#20	59.6
#40	48.0
#60	36.5
#100	27.8
#200	19.7

THIELSCH ENGINEERING
195 Frances Ave., Cranston, RI 02910
401-467-6454

CTS-74-13-0003
Penobscot River Bridge
Howland, ME
GZA File # 09.0025796.00

Tested by: AS Date: 9/6/13
Reviewed by: MBP Date: 9/6/13

ASTM D422



Gravel	Sand	Fines
35.0%	55.4%	9.6%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
4	BB-HEPR-203	4D	15-17'	Gray fine to coarse SAND, some Gravel, trace Silt (SW-SM)				

Sieve Size	% Passing
¾"	87.4
½"	81.2
#4	65.0
#10	50.4
#20	37.4
#40	24.8
#60	16.8
#100	12.6
#200	9.6

CTS-74-13-0003
Penobscot River Bridge
Howland, ME
GZA File # 09.0025796.00

Tested by:	AS	Date:	9/6/13
Reviewed by:	MBP	Date:	9/6/13

THIELSCH ENGINEERING
195 Frances Ave., Cranston, RI 02910
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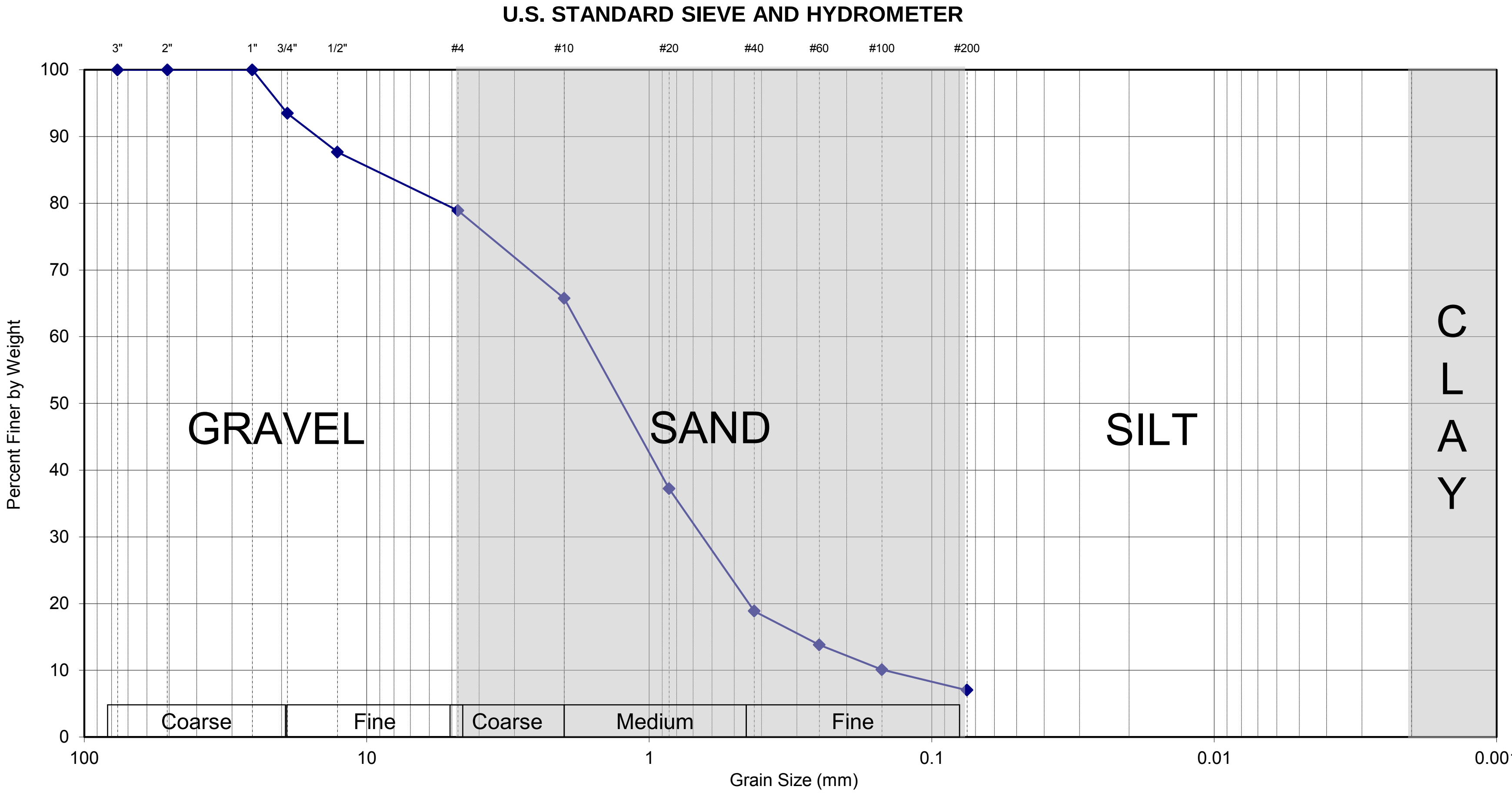
ASTM D422



Gravel	Sand	Fines
21.1%	71.9%	7.0%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
5	BB-HEPR-204	7D	24-26'	Brown fine to coarse SAND, some Gravel, trace Silt (SW-SM)				

Sieve Size	% Passing
¾"	93.5
½"	87.7
#4	78.9
#10	65.7
#20	37.2
#40	18.9
#60	13.8
#100	10.1
#200	7.0

THIELSCH ENGINEERING
195 Frances Ave., Cranston, RI 02910
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CTS-74-13-0003
Penobscot River Bridge
Howland, ME
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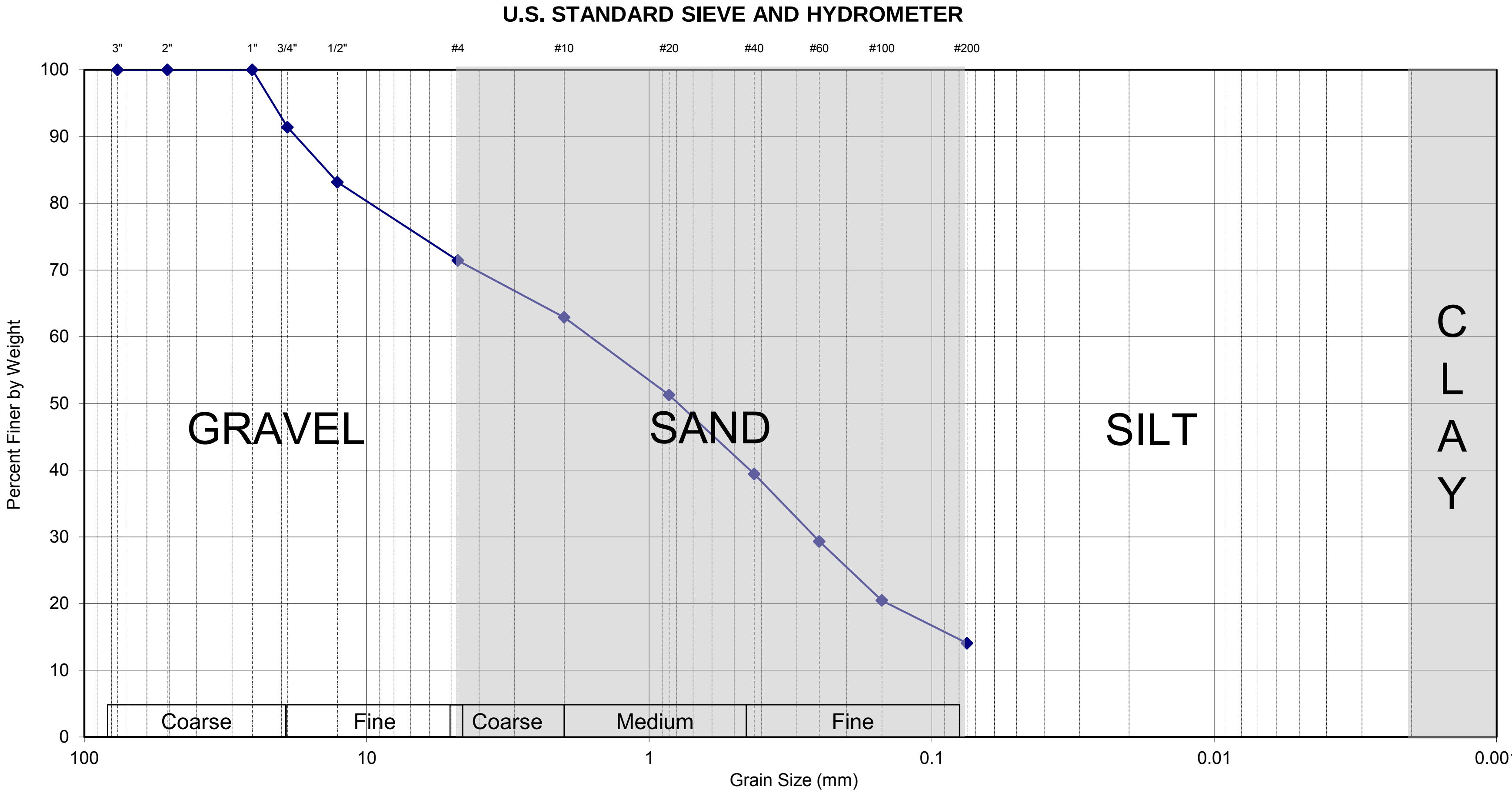
Tested by: AS Date: 9/6/13
Reviewed by: MBP Date: 9/6/13

ASTM D422



Gravel	Sand	Fines
28.6%	57.4%	14.1%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
6	BB-HEPR-204	14D	54-56'	Gray fine to coarse SAND, some Gravel, little Silt (SM)				

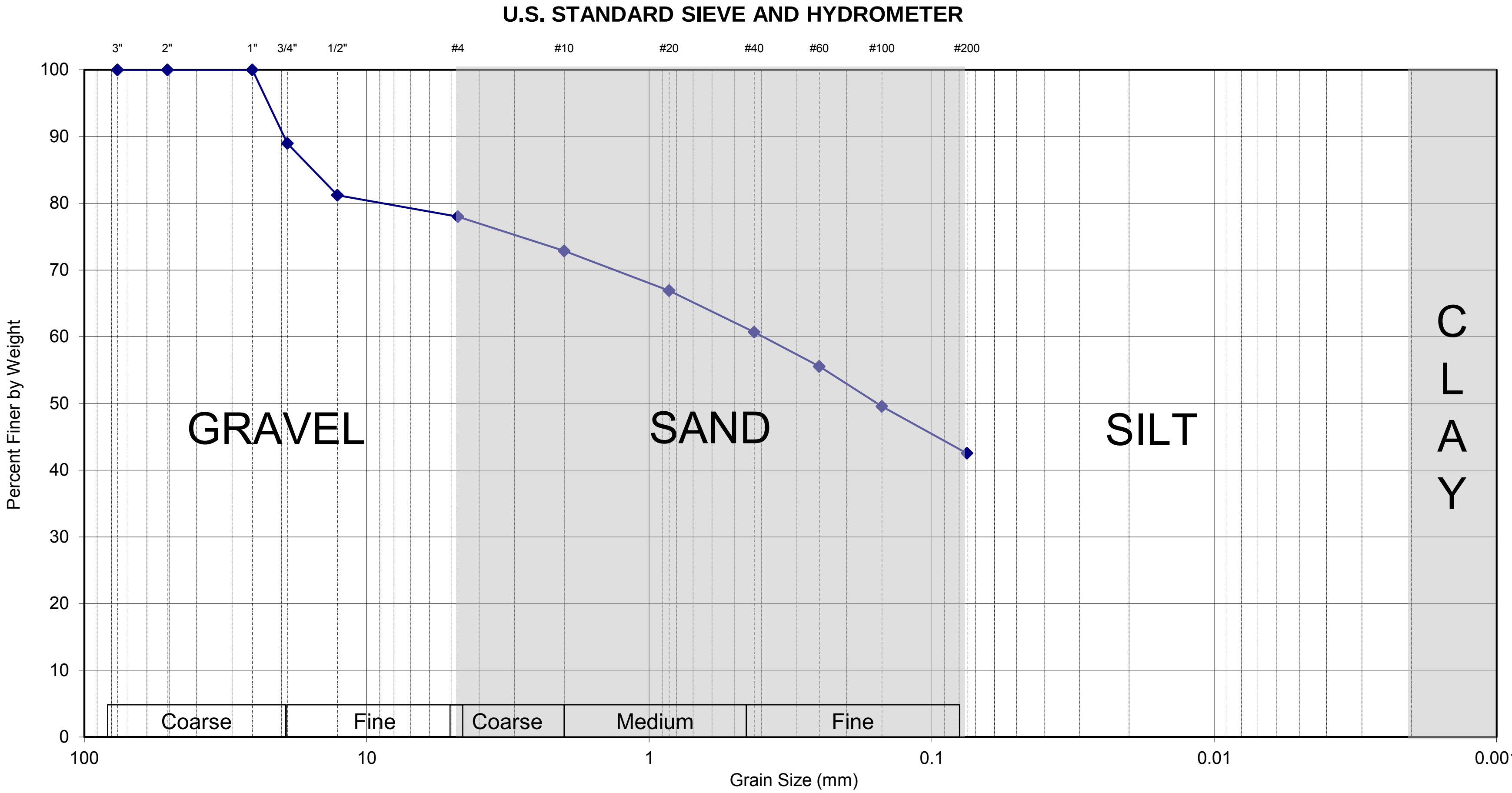
Sieve Size	% Passing
¾"	91.4
½"	83.2
#4	71.4
#10	62.9
#20	51.3
#40	39.4
#60	29.3
#100	20.4
#200	14.1

THIELSCH ENGINEERING
195 Frances Ave., Cranston, RI 02910
401-467-6454

CTS-74-13-0003
Penobscot River Bridge
Howland, ME
GZA File # 09.0025796.00

Tested by: AS Date: 9/6/13
Reviewed by: MBP Date: 9/6/13

ASTM D422



Gravel	Sand	Fines
22.0%	35.5%	42.5%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
7	BB-HEPR-205	9D	41-43'	Gray Silty Sand, some Gravel (SM)				

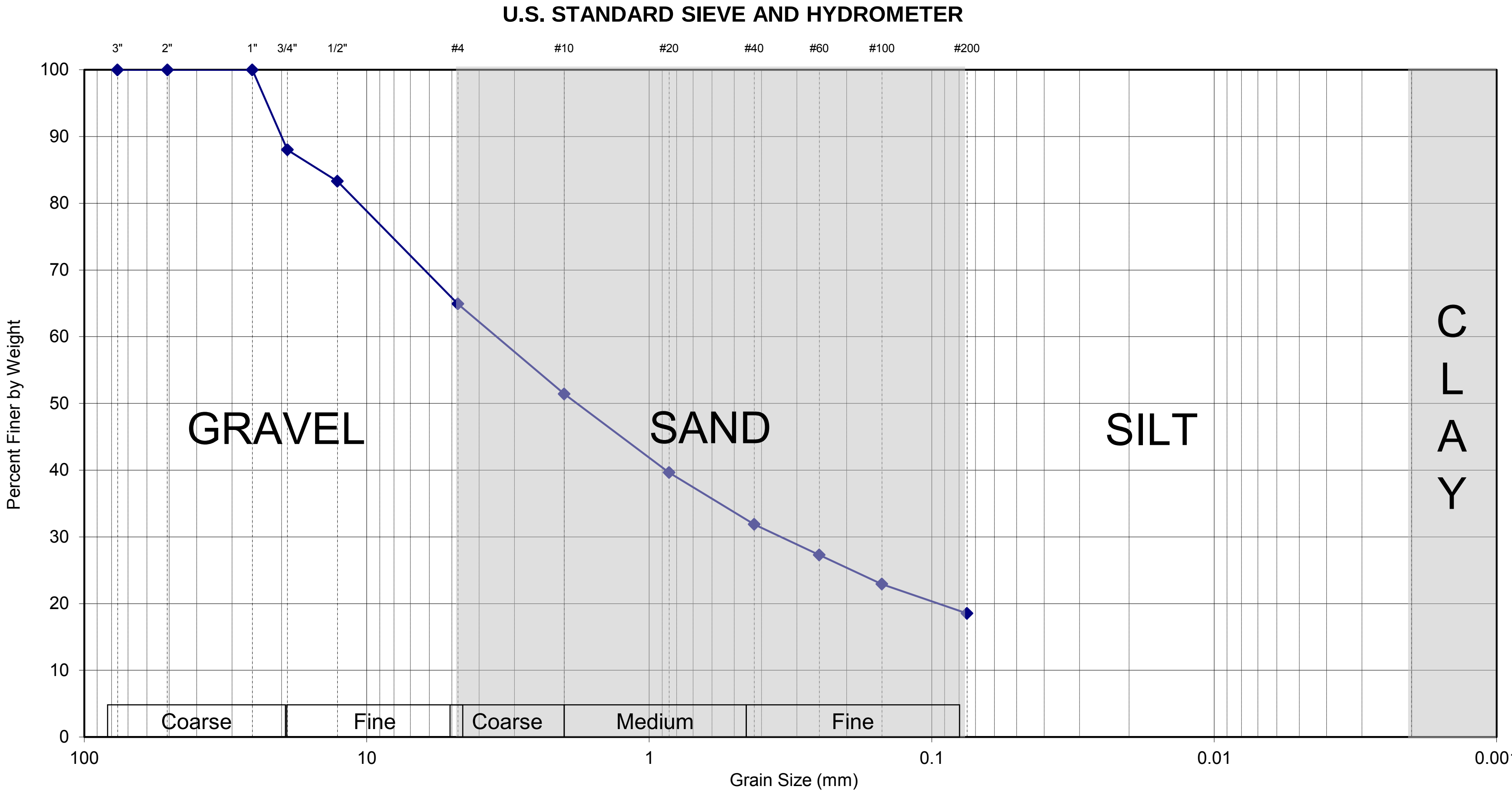
Sieve Size	% Passing
¾"	89.0
½"	81.2
#4	78.0
#10	72.8
#20	66.9
#40	60.7
#60	55.5
#100	49.5
#200	42.5

THIELSCH ENGINEERING
195 Frances Ave., Cranston, RI 02910
401-467-6454

CTS-74-13-0003
Penobscot River Bridge
Howland, ME
GZA File # 09.0025796.00

Tested by: AS Date: 9/6/13
Reviewed by: MBP Date: 9/6/13

ASTM D422



Gravel	Sand	Fines
35.1%	46.4%	18.5%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
8	BB-HEPR-207	3D	5-6.8'	Gray fine to coarse SAND, some Gravel, little Silt (SM)				

Sieve Size	% Passing
¾"	88.0
½"	83.3
#4	64.9
#10	51.4
#20	39.6
#40	31.9
#60	27.3
#100	22.9
#200	18.5

THIELSCH ENGINEERING
195 Frances Ave., Cranston, RI 02910
401-467-6454

CTS-74-13-0003
Penobscot River Bridge
Howland, ME
GZA File # 09.0025796.00

Tested by: AS Date: 9/6/13
Reviewed by: MBP Date: 9/6/13

LABORATORY TESTING DATA SHEET

Project Name Penobscot River Bridge — Location Howland, ME — Reviewed By Matthew Pulgis (signature)

Project No. 09.0025796.00 — Assigned By J. Baron

Project Manager J. Baron — Report Date 9/5/2013 — Date Reviewed 9/5/2013

Boring No.	Sample No.	Depth Ft.	Lab No.	Sample Data: Water Content %	Do in.	L in.	(1) Unit Wt. PCF	(2) Wet Density PCF	Bulk Gs.	Compression Tests: (3) Other Tests	(4) Strength PSI	(5) Strain %	(6) Conf. Stress	(7) E sec PSI EE+06	(8) Poisson's Ratio	στ PSI	Rock Formation or Description or Remarks
BB-HEPR-202	R2	10.4-10.8	1		1.980	4.630	171.2			U	3,488	0.09		3.45			Failed on foliation planes
BB-HEPR-204	R2	62.0-62.4	2		1.980	4.573	170.1			U	5,177	0.10		4.35			Failed on foliation planes

(1) Volume Determined By Measuring Dimensions
(2) Determined by Measuring Dimensions and Weight of Saturated Sample

(3) P=Petrographic PLD=Point Load (diametrica
PLA= Point Load (Axial) RST= Splitting Tensile
U= Unconfined Compressive Strength
(4) Taken at Peak Deviator Stress

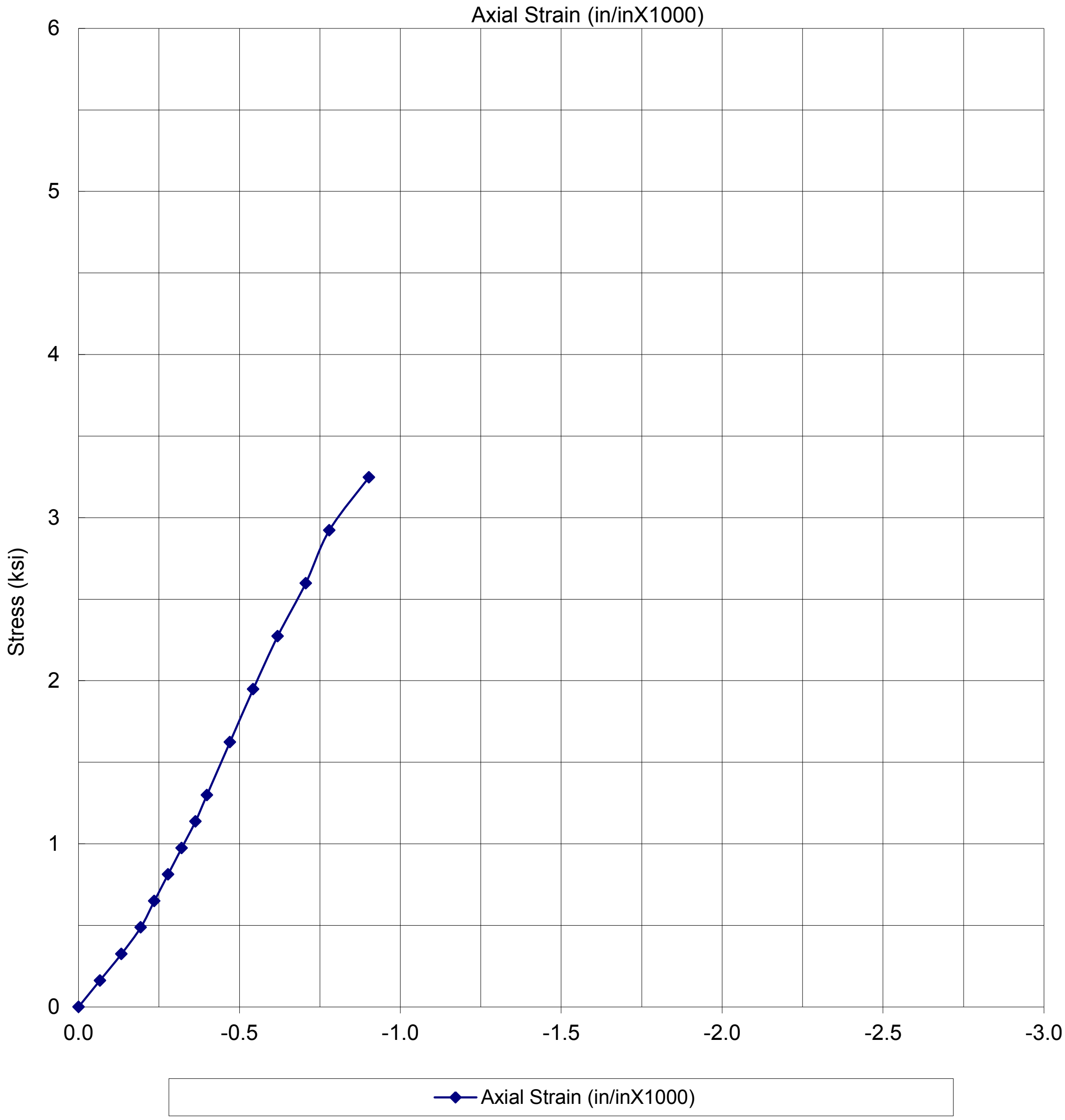
(5) Strain at Peak Deviator Stress
(6) Represents Confining Stress on Triaxial Tests
(7) Represents Secant Modulus at 50% of Total Failure Stress
(8) Represents Secant Poisson's Ratio at 50% of Total Failure Stress

THIELSCH ENGINEERING

195 Frances Ave.
Cranston, RI 02910

401-467-6454

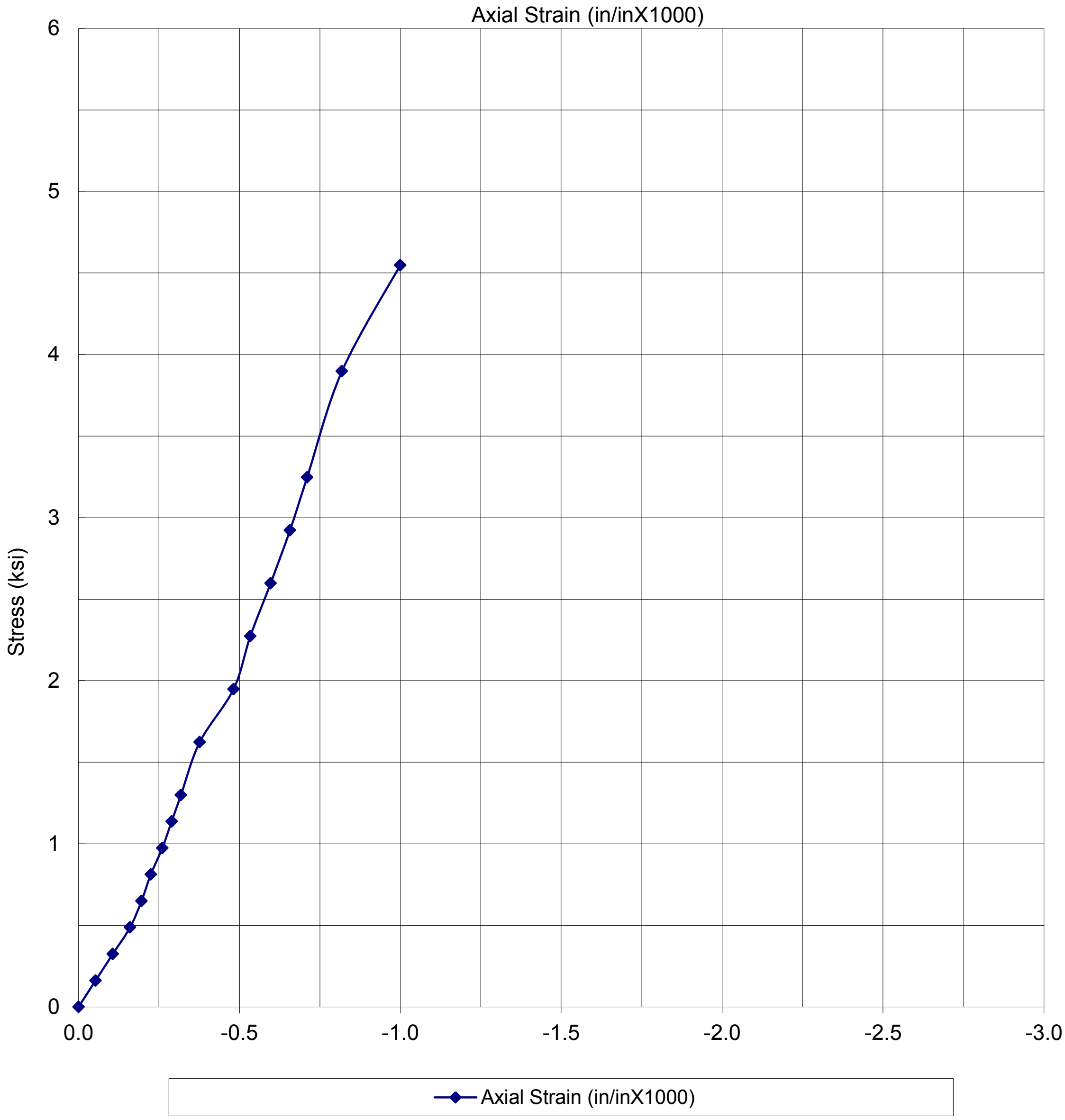
Penobscot River Bridge
Howland, ME



Rock Testing

Boring No.	BB-HEPR-202	File No.	09.002596.00
Sample No.	R2	Date:	9/5/2013
Depth:	10.4-10.8'	Test No.	U 1

Penobscot River Bridge
Howland, ME



Rock Testing

Boring No.	BB-HEPR-204	File No.	09.002596.00
Sample No.	R2	Date:	9/5/2013
Depth:	62.0-62.4'	Test No.	U 2

APPENDIX D

SPECIAL PROVISION

SPECIAL PROVISION
SECTION 511
COFFERDAMS

Section 511 is deleted in its entirety and replaced with the following:

511.01 Description This work shall consist of the complete design, construction, maintenance and removal of cofferdams and other related work, including dewatering and inspection, required to allow for the excavation of foundation units, to permit and protect the construction of bridge or other structural units and to protect adjacent Roadways, embankments or other structural units, in accordance with the Contract.

511.02 Materials As specified in the cofferdam Working Drawings.

511.03 Cofferdam Construction

A. Working Drawings. The Contractor shall submit Working Drawings, showing the materials to be used and the proposed method of construction of cofferdams to the Department. Construction shall not start on cofferdams until such Working Drawings have been submitted. Any review of or comment on, or any lack of review of or comment on, these Working Drawings by the Department shall not result in any liability upon the Department and it shall not relieve the Contractor of the responsibility for the satisfactory functioning of the cofferdam.

B. Construction. Construct cofferdams in conformance with the submitted Working Drawings. Cofferdams shall, in general, be carried below the elevation of the bottom of footings to adequate depths to ensure stability and adequate heights to seal off water. Cofferdams shall be braced to withstand pressure without buckling, secured in place to prevent tipping or movement and be as watertight as necessary for the safe and proper construction of the substructure Work inside them. With the exception of construction of a concrete foundation seal placed under water, the interior dimensions of cofferdams shall provide sufficient clearance for the construction and inspection of forms and to permit pumping outside of forms. The Contractor shall be responsible for the righting and resetting of cofferdams that have tilted or moved laterally, as required for construction.

During the placing and curing of seal concrete, maintain the water level inside the cofferdam at the same level as the water outside the cofferdam, to prevent flow through the concrete.

No timber or bracing shall be used in cofferdams in such a way as to remain in the substructure Work.

Cofferdams shall be constructed to protect fresh concrete against damage from the sudden rising of the water body, to prevent damage by erosion and to prevent damage to adjacent Roadways, embankments or other structural units.

Unless otherwise noted, cofferdams, including all sheeting and bracing involved, shall be removed after the completion of the substructure Work in a manner that prevents disturbance or injury to the finished Work.

Cofferdams shall be constructed, dewatered and removed in accordance with the requirements of Section 656 - Temporary Soil Erosion and Water Pollution Control and related Special Provisions.

C. Inspection of Seal Cofferdams. Seal cofferdam excavations shall initially be inspected and approved by the Contractor.

For each seal cofferdam excavation, the Contractor shall submit a written procedure to the Resident for sediment/overburden removal and excavation inspection. For cofferdams where seal concrete is to be placed on bedrock, the inspection procedure shall describe the Contractor's final cleaning and inspection process for attaining cleanliness of each cofferdam excavation. For cofferdams where seal concrete is not excavated to bedrock, the procedure shall describe the Contractor's final cleaning and inspection process for attaining the bottom of seal elevation shown on the Plans.

The Contractor shall notify the Resident at least 48 hours prior to when each seal cofferdam excavation will be ready for final inspection by the Department. The Contractor shall allow adequate time for each occurrence of cofferdam excavation inspection by the Department. The Contractor shall provide and maintain access and equipment, such as steel probes, for the Resident and/or the Department's Dive Team to independently inspect each cofferdam excavation.

No seal concrete placement shall begin until the Department has approved the cofferdam excavation.

<u>511.04 Pumping</u> Pumping from the interior of any cofferdam shall be done in such a manner as to prevent any current of water that would carry away or segregate the concrete.

Pumping to dewater a sealed cofferdam shall not commence until the seal concrete has set sufficiently to withstand the hydrostatic pressure and meets the following minimum curing time, after the completion of the installation of the seal concrete:

1. When the temperature of the water body outside the cofferdam is greater than 40°F, a minimum of 5 days.
2. When the temperature of the water body outside the cofferdam is less than 40°F, a minimum of 7 days.

Procedures for the removal of all water and materials from cofferdams shall be described in the Soil Erosion and Water Pollution Control Plan as required in Section 656 Temporary Soil Erosion and Water Pollution Control and related Special Provisions.

<u>511.05 Method of Measurement</u> Cofferdams will be measured as one lump sum unit, as indicated on the Plans or called for in the Contract.

511.06 Basis of Payment The accepted quantity of cofferdam will be paid for at the Contract lump sum price for the respective cofferdam items, which price shall be full compensation for design, construction, maintenance, inspection and removal.

When required, the elevation of the bottom of the footing of any substructure unit may be lowered, without change in the price to be paid for cofferdams. However, if the average elevation of more than 25% of the area of the excavation is more than 3 feet below the elevation shown on the Plans, and if requested by the Contractor, then the additional costs incurred that are included in the cofferdam Pay Item will be paid for in accordance with Section 109.7 - Equitable Adjustments to Compensation. The Contractor shall immediately notify the Department when these additional costs commence. Failure of the Contractor to provide this notification will result in undocumented additional work that will be non-reimbursable. The Department will evaluate this additional work to determine an appropriate time extension, if warranted.

All costs for sedimentation control practices, including, but not limited to, constructing, maintaining, and removing sedimentation control structures, and pumping or transporting water and other materials for sedimentation control will not be paid for directly, but will be considered incidental to the cofferdam Pay Item(s).

All costs for related temporary soil erosion and water pollution controls, including inspection and maintenance, will not be paid for directly, but will be considered incidental to the cofferdam Pay Item(s).

All costs associated with preparation of Working Drawings, design calculations, written procedure for sediment/overburden removal and excavation inspection, and the inspection of the seal cofferdam excavation shall be considered incidental to the cofferdam Pay Item(s). There shall be no additional payment for repeated inspection by the Department of the same cofferdam excavation.

All costs for cofferdams and related temporary soil erosion and water pollution controls, including inspection and maintenance, will be considered incidental to related Pay Items, when a specific Pay Item for cofferdams is not included in the Contract.

Seal concrete will be evaluated under Section 502.

Payment will be made under:

Pay Item	Pay Unit
511.07 Cofferdam	Lump Sum

APPENDIX E

CALCULATIONS

Penobscot River Bridge
SEISMIC SITE CLASS DETERMINATION

Performed By: J. Baron
Reviewed By: E. Lonstein

BB-HEPR-201 (G.S. El. 153.9)

Depth (ft)	Elevation (ft)	SPT N-value	di/Ni
1.0	153	33	0.03
6.0	148	43	0.12
10.0	144	100	0.04
16.0	138	87	0.07
100.0	54	100	0.84

BB-HEPR-202 (G.S. El. 125.7)

Depth (ft)	Elevation (ft)	SPT N-value	di/Ni
1.0	125	28	0.04
100.0	26	100	0.99

BB-HEPR-203 (G.S. El. 123)

Depth (ft)	Elevation (ft)	SPT N-value	di/Ni
1.2	122	24	0.05
7.0	116	65	0.09
11.0	112	46	0.09
16.0	107	58	0.09
21.0	102	90	0.06
25.0	98	100	0.04
100.0	23	100	0.75

BB-HEPR-204 (G.S. El. 123.2)

Depth (ft)	Elevation (ft)	SPT N-value	di/Ni
1.0	122	14	0.07
5.0	118	25	0.16
9.0	114	100	0.04
11.0	112	100	0.02
15.0	108	48	0.08
20.0	103	45	0.11
25.0	98	69	0.07
30.0	93	78	0.06
34.0	89	100	0.04
36.0	87	100	0.02
40.0	83	100	0.04
45.0	78	73	0.07
50.0	73	100	0.05
55.0	68	100	0.05
100.0	23	100	0.45

BB-HEPR-205 (G.S. El. 122.3)

Depth (ft)	Elevation (ft)	SPT N-value	di/Ni
1.0	121	28	0.04
6.0	116	47	0.11
11.0	111	49	0.10
17.0	105	45	0.13
22.0	100	41	0.12
27.0	95	80	0.06
32.0	90	100	0.05
37.0	85	78	0.06
42.0	80	96	0.05
47.0	75	100	0.05
51.0	71	100	0.04
100.0	22	100	0.49

BB-HEPR-206 (G.S. El. 123.2)

Depth (ft)	Elevation (ft)	SPT N-value	di/Ni
1.0	122	6	0.17
6.0	117	90	0.06
100.0	23	100	0.94

BB-HEPR-207 (G.S. El. 123.2)

Depth (ft)	Elevation (ft)	SPT N-value	di/Ni
1.0	122	8	0.13
3.0	120	27	0.07
5.0	118	100	0.02
100.0	23	100	0.95

$$N_{ch} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N_i}}$$

Abutments and Piers 2 and 3

Boring I.D.	N_{ch}	
BB-HEPR-201	91.3	
BB-HEPR-202	97.5	
BB-HEPR-203	86.4	**Pier 1**
BB-HEPR-204	74.6	
BB-HEPR-205	76.4	
BB-HEPR-206	86.0	
BB-HEPR-207	85.5	
Avg.	**85.2**	

Conclusions: Site Class D 15<N<50

Determination of N_{ch}:

Determine N_{ch} via the equation:

where: d_s = the total thickness of cohesionless soil layers in the top 100 feet.
d_i = the thickness of any layer between 0 and 100 feet.
N_i = the Standard Penetration Resistance (ASTM D 1586) not to exceed 100 blows/ft as directly measured in the field without corrections.

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Engineers and Scientists

Objective

Assess nominal and factored bearing resistance of a foundation on rock at Pier 1 location.

Methodology

Use data from test borings and evaluate the nominal bearing resistance as follows:

1. Bedrock Properties From Test Borings
2. Calculation Of Rock Mass Rating
3. Determine Rock Property Constants s and m
4. Calculate Nominal Bearing Resistance of Bedrock q_n

References

1. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 5th edition, 2010. (AASHTO LRFD)

2. Wyllie, Duncan C., "Foundations on Rock", Second edition, 1992.

1. Rock Properties

Bedrock properties were obtained from rock core specimens and logs completed for the Penobscot River Bridge Project. The following table presents the data for the Pier 1 test boring where spread footings on bedrock will be utilized.

Run	Depth	RQD(%)	Rock Type	Joint Spacing Desc.	Corr. Spacing (in)	Apeture Desc	Corr. Apeture (in)
B102R1	22-26	11	Phyllite	Close	2.5-8	Tight-Open	0.004-0.1
B102R2	26-28	0	Phyllite	Close	2.5-8	Tight-Open	0.004-0.1
B102R1	28-33	50	Phyllite	Close to Moderate	2.5-24	Tight	0.004-0.01
B203R1	27-28	0	Phyllite	Close	2.5-8	Open	0.02-0.1
B203R2	28-33	61	Phyllite	Close to Moderate	2.5-24	Tight	0.004-0.01
B203R3	33-38	67	Phyllite	Moderate	8-24	Open	0.02-0.1

Anticipate that 0 RQD rock will be excavated to prepare footing - typical RQDs = 50, 61, 67

2. Calculation of Rock Mass Rating (RMR)

From AASHTO LRFD Table 10.4.6.4-1, determine the RMR.

Parameter 1- Uniaxial Compressive Strength

Uniaxial compressive strength tests were performed on two core specimens at or in the vicinity of the Penobscot River Bridge.

Boring	Run	Depth	Rock Type	qp (ksi)
202	R2	10.4-10.8	phyllite	3.49
204	R2	62-62.4	phyllite	5.18
			Average	**4.33 ksi**

Representative unconfined compressive strength of intact rock. $\sigma_{u.r} := 4.33\text{ksi}$

$\sigma_{u.r} = 624 \cdot \text{ksf}$

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_1 := 4$ for $\sigma_{u.r}$=520 to 1080 ksf

Parameter 2- Drill Core Quality

Average RQD % = 50 - 67%

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_2 := 13$ for RQD = 50% to 75%

Parameter 3- Spacing of Joints

From Boring Logs, generally close to moderate = 2.5 in to 24 in

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_3 := 10$ for 2 in to 1 ft spacing

Parameter 4- Condition of Joints

From boring logs, aperture generally less than 0.01 inches and hard joint walls.

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_4 := 20$ for slightly rough surfaces, separation <0.05 in, hard joint wall rock

Parameter 5- Ground Water Conditions

Groundwater Conditions

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_5 := 4$ for Moderate water pressure. joint water pressure = 0.2 to 0.5 total vertical stress

Adjustment for joint orientation (Parameter 6)

The joint sets are generally high angle and generally rough and tight to open. Considering rock will remain embedded below bearing level and steep joints tend to compress less, joint orientation is considered favorable.

From AASHTO LRFD Table 10.4.6.4-2

Relative Rating $RR_6 := -2$ for foundations - fair conditions

Total RMR Rating

$$RMR := RR_1 + RR_2 + RR_3 + RR_4 + RR_5 + RR_6$$

$$RMR = 49$$

From AASHTO LRFD Table 10.4.6.4-3 RMR= 41-60 is indicative of Fair Rock Quality (Class No. 3)

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Engineers and Scientists

3. Determine Rock Property Constants s and m

From AASHTO LRFD Table 10.4.6.4-4 for Fair Quality Rock Mass

Categorized as rock type B (phyllite), RMR=49, using is and m values interpolated from the logarithmic trend of plotted values from AASHTO Table 10.4.6.4-4 (plots on sheet 10).

$m := .264$

$s := .00021$

4. Calculate Nominal and Factored Bearing Resistance of Bedrock q_n and q_R

From Wyllie "Foundations on Rock"

Eq. 5.4 Pg.138

$$q_n := C_{f1}\cdot\sqrt{s}\cdot\sigma_{u.r}\cdot\left[1 + \sqrt{m\cdot\left(s^{-\frac{1}{2}}\right) + 1}\right]$$

Where

$C_{f1} := 1.12$ From Wyllie Table 5.4 Pg. 138 Correction factor for foundation shape for rectangular foundation:

For L/B>5, use factor C_{fl}=1.05,
For L/B=2, use factor C_{fl}=1.12,
Estimate footing is roughly L/B=2, use 1.12.

$s = 0.00021$

$m = 0.26$

$\sigma_{u.r} = 4.33\cdot ksi$

Nominal Bearing Resistance

$$q_n := C_{f1}\cdot\sqrt{s}\cdot\sigma_{u.r}\cdot\left[1 + \sqrt{m\cdot\left(s^{-\frac{1}{2}}\right) + 1}\right]$$

$q_n = 54.5\cdot ksf$ Say 55 ksf

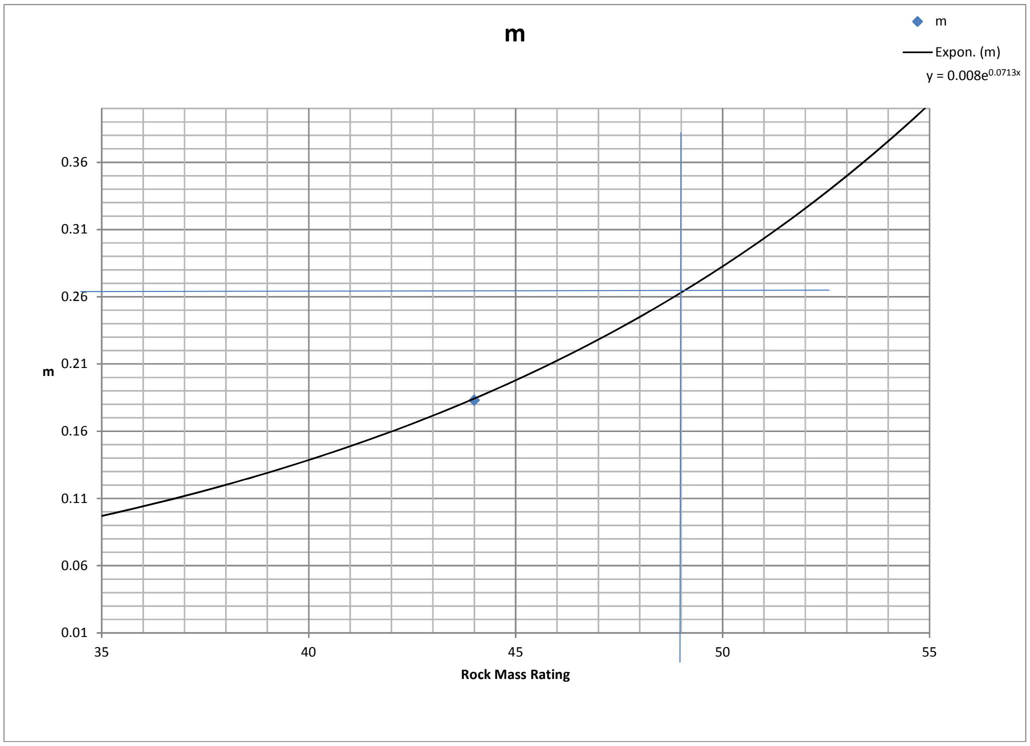
Factored Bearing Resistance

Bearing Resistance Factor is specified in Table 10.5.5.2.2-1

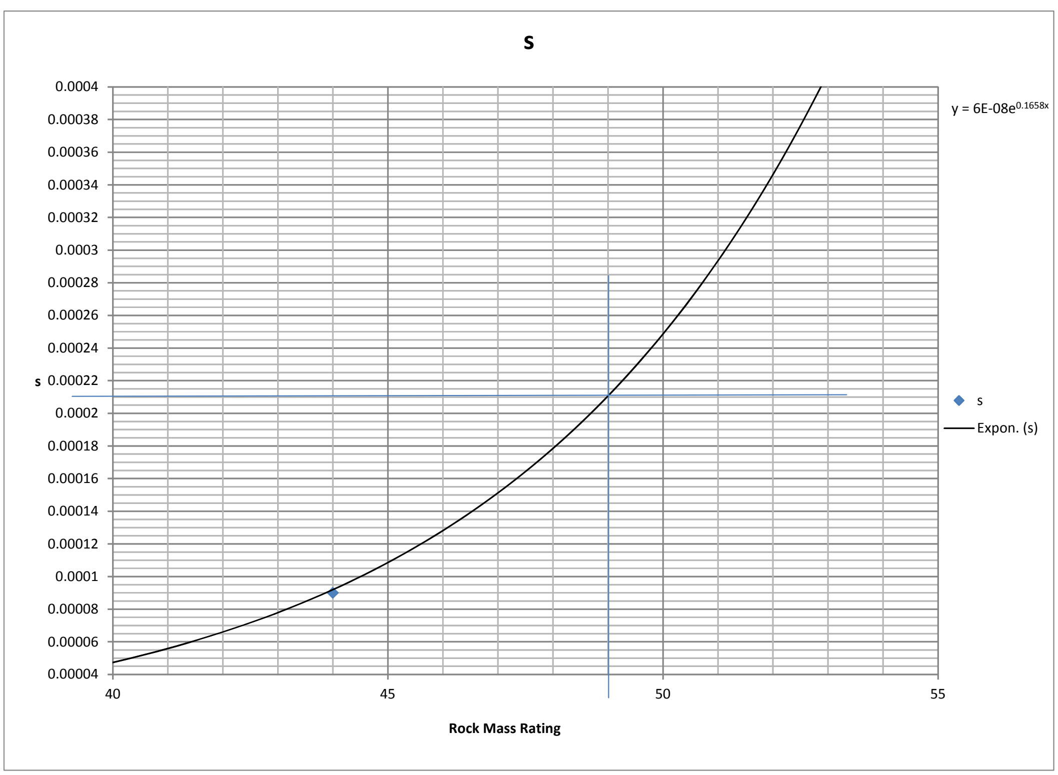
$\phi_b := 0.45$ Footing on rock

$q_R := \phi_b\cdot q_n$

$q_R = 24.5\cdot ksf$ Say 25 ksf



m
m
Expon. (m)
y = 0.008e^0.0713x
0.36
0.31
0.26
0.21
0.16
0.11
0.06
0.01
m
35
40
45
50
55
Rock Mass Rating



s
y = 6E-08e^0.1658x
0.0004
0.00038
0.00036
0.00034
0.00032
0.0003
0.00028
0.00026
0.00024
0.00022
0.0002
0.00018
0.00016
0.00014
0.00012
0.0001
0.00008
0.00006
0.00004
s
s
Expon. (s)
40
45
50
55
Rock Mass Rating

Reference:M:\FILES\GEOTECH\Design Calculations\Units v7.xmcd

10-22 AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

Table 10.4.6.4-1 Geomechanics Classification of Rock Masses.

	Parameter		Ranges of Values						
1	Strength of intact rock material	Point load strength index	>175 ksf	85–175 ksf	45–85 ksf	20–45 ksf	For this low range, uniaxial compressive test is preferred		
		Uniaxial compressive strength	>4320 ksf	2160–4320 ksf	1080–2160 ksf	520–1080 ksf	215–520 ksf	70–215 ksf	20–70 ksf
	Relative Rating		15	12	7	4	2	1	0

	Parameter	Ranges of Values				
2	Drill core quality RQD	90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%
	Relative Rating	20	17	13	8	3
3	Spacing of joints	>10 ft.	3–10 ft.	1–3 ft.	2 in.–1 ft.	<2 in.
	Relative Rating	30	25	20	10	5
4	Condition of joints	• Very rough surfaces • Not continuous • No separation • Hard joint wall rock	• Slightly rough surfaces • Separation <0.05 in. • Hard joint wall rock	• Slightly rough surfaces • Separation <0.05 in. • Soft joint wall rock	• Slicken-sided surfaces or • Gouge <0.2 in. thick or • Joints open 0.05–0.2 in. • Continuous joints	• Soft gouge >0.2 in. thick or • Joints open >0.2 in. • Continuous joints
	Relative Rating	25	20	12	6	0

	Parameter		Ranges of Values			
5	Ground water conditions (use one of the three evaluation criteria as appropriate to the method of exploration)	Inflow per 30 ft. tunnel length	None	<400 gal./hr.	400–2000 gal./hr.	>2000 gal./hr.
		Ratio = joint water pressure/ major principal stress	0	0.0–0.2	0.2–0.5	>0.5
		General Conditions	Completely Dry	Moist only (interstitial water)	Water under moderate pressure	Severe water problems
	Relative Rating		10	7	4	0

Table 10.4.6.4-2 Geomechanics Rating Adjustment for Joint Orientations.

Strike and Dip Orientations of Joints		Very Favorable	Favorable	Fair	Unfavorable	Very Unfavorable
Ratings	Tunnels	0	–2	–5	–10	–12
	Foundations	0	–2	–7	–15	–25
	Slopes	0	–5	–25	–50	–60

Table 10.4.6.4-3 Geomechanics Rock Mass Classes Determined From Total Ratings.

RMR Rating	100–81	80–61	60–41	40–21	<20
Class No.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

Table 10.4.6.4-4 Approximate relationship between rock-mass quality and material constants used in defining nonlinear strength (*Hoek and Brown, 1988*)

Rock Type

A = Carbonate rocks with well developed crystal cleavage—*dolomite, limestone and marble*
B = Lithified argrillaceous rocks—*mudstone, siltstone, shale and slate (normal to cleavage)*
C = Arenaceous rocks with strong crystals and poorly developed crystal cleavage—*sandstone and quartzite*
D = Fine grained polyminerallic igneous crystalline rocks—*andesite, dolerite, diabase and rhyolite*
E = Coarse grained polymінerallic igneous & metamorphic crystalline rocks—*amphibolite, gabbro gneiss, granite, norite, quartz-diorite*

Rock Quality	Constants	A	B	C	D	E
INTACT ROCK SAMPLES Laboratory size specimens free from discontinuities CSIR rating: *RMR* = 100	*m* *s*	7.00 1.00	10.00 1.00	15.00 1.00	17.00 1.00	25.00 1.00
VERY GOOD QUALITY ROCK MASS Tightly interlocking undisturbed rock with unweathered joints at 3–10 ft. CSIR rating: *RMR* = 85	*m* *s*	2.40 0.082	3.43 0.082	5.14 0.082	5.82 0.082	8.567 0.082
GOOD QUALITY ROCK MASS Fresh to slightly weathered rock, slightly disturbed with joints at 3–10 ft. CSIR rating: *RMR* = 65	*m* *s*	0.575 0.00293	0.821 0.00293	1.231 0.00293	1.395 0.00293	2.052 0.00293
FAIR QUALITY ROCK MASS Several sets of moderately weathered joints spaced at 1–3 ft. CSIR rating: *RMR* = 44	*m* *s*	0.128 0.00009	0.183 0.00009	0.275 0.00009	0.311 0.00009	0.458 0.00009
POOR QUALITY ROCK MASS Numerous weathered joints at 2 to 12 in.; some gouge. Clean compacted waste rock. CSIR rating: *RMR* = 23	*m* *s*	0.029 3×10^{-6}	0.041 3×10^{-6}	0.061 3×10^{-6}	0.069 3×10^{-6}	0.102 3×10^{-6}
VERY POOR QUALITY ROCK MASS Numerous heavily weathered joints spaced <2 in. with gouge. Waste rock with fines. CSIR rating: *RMR* = 3	*m* *s*	0.007 1×10^{-7}	0.010 1×10^{-7}	0.015 1×10^{-7}	0.017 1×10^{-7}	0.025 1×10^{-7}

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Engineers and Scientists

Objective

Evaluate pile foundations including the axial geotechnical resistance of the pile .

Methodology

Evaluate proposed pile section for governing axial compression resistance as follows. Pile properties are for full section - no corrosion allowance.

1. Nominal Compressive Resistance
2. Factored Structural Compressive Resistance - Strength Limit State
3. Factored Structural Compressive Resistance - Extreme/Service Limit State
4. Geotechnical Resistance (Static Analysis)
5. Geotechnical Resistance (Drivability Analysis)
6. Factored Geotechnical Resistance - Strength Limit State
7. Factored Geotechnical Resistance - Extreme/Service Limit State

References

1. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 4th edition, 2007 with 2008 and 2009 interim Revisions. (AASHTO LRFD)
2. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 6th edition, 2012 with June 2012 errata. (AASHTO LRFD)
3. Preliminary Design Plans, Fairfield BRO 1448(38) prepared by McFarland-Johnson, Inc. dated 7/31/12.
4. Test borings B-1 through B-4 drilled by NH Boring and observed by GZA, March 2012.

Soil Properties

Soil Profile was interpolated based on borings BB-HEPR-201 through -207
The subsurface profile consists of Fill at the abutments, Alluvial at the piers, overlying Glacial Till and Bedrock.
Bedrock was encountered approximately 14 to 19 feet below the bottom of pile cap elevation at the Abutments.
Bedrock was encountered approximately 43 to 49 feet below the bottom of pile cap elevation at the Piers.

Structural Properties

Young's Modulus of Steel $E_s := 29000 \cdot ksi$

Yield Strength of Steel $F_y := 50ksi$

Area of section

$$A_s := \begin{pmatrix} 15.5 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} in^2$$

For **HP 12 X 53**
HP 14 X 73
HP 14 X 89
HP 14 X 117

1. Nominal Compressive Resistance P_n

Nominal Compressive Resistance: $P_n := F_y \cdot A_s$

$$P_n = \begin{pmatrix} 775 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot \text{kip}$$

For **HP 12 X 53**
HP 14 X 73
HP 14 X 89
HP 14 X 117

2. Factored Structural Compressive Resistance - Strength Limit State:

Assuming that the bottom of the tremie elevation is below scour, buckling need not be considered.

Factor for piles in compression under severe driving conditions:

From Article 6.5.4.2 $\phi_c := 0.5$

Factored Compressive Resistance for Strength Limit State:

$P_{r.s} := \phi_c \cdot P_n$ AASHTO Eq. 6.9.2.1-1 pg. 6-81

$$P_{r.s} = \begin{pmatrix} 388 \\ 535 \\ 653 \\ 860 \end{pmatrix} \cdot \text{kip}$$

For **HP 12 X 53**
HP 14 X 73
HP 14 X 89
HP 14 X 117

3. Factored Structural Compressive Resistance - Service/Extreme Limit State:

Resistance Factors for Extreme Limit States:

From Article 10.5.5.1 and 10.5.5.3 $\phi := 1$

Factored Compressive Resistance for Service/Extreme Limit State:

$P_{r.e} := \phi \cdot P_n$ AASHTO Eq. 6.9.2.1-1 pg. 6-81

$$P_{r.e} = \begin{pmatrix} 775 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot \text{kip}$$

For **HP 12 X 53**
HP 14 X 73
HP 14 X 89
HP 14 X 117

4. Geotechnical Axial Resistance - Static Analysis

In GZA's experience for end bearing on rock, the structural resistance or drivability resistance will control this analysis.

5. Geotechnical Axial Resistance - Drivability Analysis

$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y$ AASHTO Eq. 10.7.8-1 Pg. 10-121

$f_y := F_y$ yield Strength of steel

$\phi_{da} := 1.0$ AASHTO Table 10.5.5.2.3-1, page 10-46, Refers to Article 6.5.4.2, Pg. 6-30

$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y$ $\sigma_{dr} = 45 \cdot ksi$ Driving Stress in pile cannot exceed 45 ksi

6. Factored Drivability Resistance - Strength Limit State:

Strength Limit State Factored Drivability Resistance:

$R_{ndr\_factored} := R_{ndr} \cdot \phi_{dyn}$

$\phi_{dyn} := 0.65$ AASHTO Table 10.5.5.2.3-1, Page 10-45 PDA, WEAP and CAPWAP used to establishing driving criteria, hard driving

$$R_{ndr} := \begin{pmatrix} 435 \\ 553 \\ 700 \\ 900 \end{pmatrix} \cdot kip$$

$R_{ndr\_factored} := R_{ndr} \cdot \phi_{dyn}$ $R_{ndr\_factored} = \begin{pmatrix} 283 \\ 359 \\ 455 \\ 585 \end{pmatrix} \cdot kip$

7. Factored Drivability Resistance - Service/Extreme Limit States:

Service and Extreme Limit State Factored Drivability Resistance:

Resistance Factors for Extreme Limit States:

From Article 10.5.5.1 and 10.5.5.3 $\phi_{serv\_ext} := 1$

$R_{ndr\_serv\_ext} := R_{ndr} \cdot \phi_{serv\_ext}$ $R_{ndr\_serv\_ext} = \begin{pmatrix} 435 \\ 553 \\ 700 \\ 900 \end{pmatrix} \cdot kip$

Penobscot River Bridge
Howland-Enfield, Maine
09.0025796.00

WEAP Analysis							
Hammer	**E_{rated} (ft-lbs)**	**Fuel Setting**	**Pile Size**	**Location**	**Pile Length (ft)**	**R_{ult} (kips) @ 45ksi**	**Blow Count @ 45ksi**
Delmag D22-02	48,500	4	12x53	Abut 1	14	435	14
Delmag D22-02	48,500	3	12x53	Abut 2	19	444	11
Delmag D22-02	48,500	1	12x53	Pier	29	500	15
Delmag D22-02	48,500	2	14x73	Abut 2	19	553	12
Delmag D30-02	66,200	4	14x89	Abut 2	19	700	13
Delmag D46-02	107,100	4	14x117	Abut 2	19	900	13

ABUT1BOX.OUT

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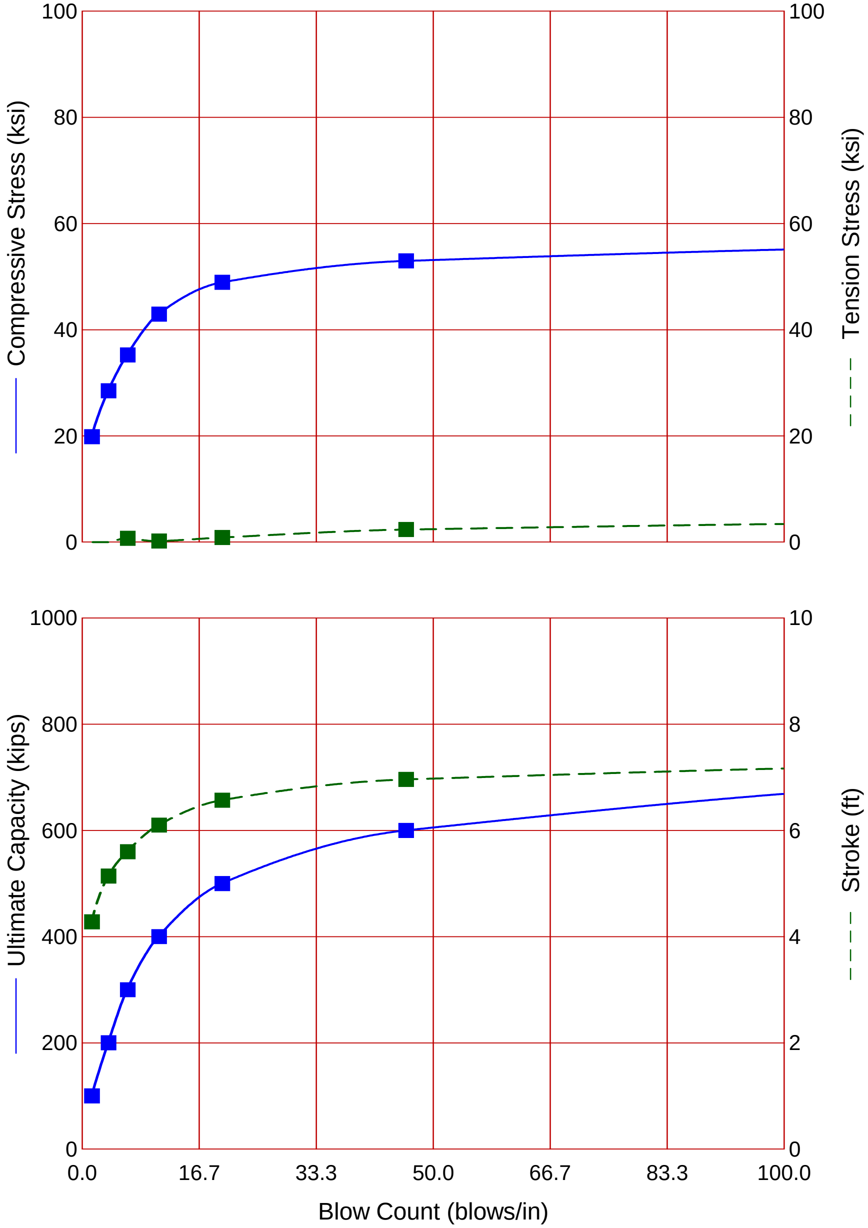
ÚÄÄÄÄÄÄÄÄ ULTIMATE STATIC PILE CAPACITY/Federal Highway Administration ÄÄÄÄÄÄÄÄ¿
3           Nordlund (1963, 1979) and Tomlinson (1979, 1980) methods            3
3                                                                              3
3 Project Name     : Howland               Client          : TYLin             3
3 File Name        : abut1                 Project Manager : JRB               3
3 Date             : 10/28/13              Computed by     : JRB               3
3                                                                              3
3 Depth of Top of Pile =   0.00 ft.         Pile length          =  14.00 ft.  3
3 Depth to Water Table =   6.00 ft.                                            3
3 Type of Pile         = H Pile                                                3
3   HP 12x53                                                                   3
3                                                                              3
3                          SKIN FRICTION CONTRIBUTION                          3
3                                                                              3
3 Layer     Soil        Thickness    Effective   Internal   N-SPT      Pile    3
3           Type                      Stress     Friction           Perimeter  3
3                          (ft)        (psf)       Angle               (ft)    3
3                                                                              3
3   1    Cohesionless     14.00       812.60      32.00      --        3.97    3
3                                                                              3
3                                                                              3
3 Layer     Soil      Undrained Shear  Adhesion   Pile    Sliding      Skin    3
3           Type         Strength                Taper   Friction  Resistance  3
3                          (psf)                          Angle      (Kips)    3
3                                                                              3
3   1    Cohesionless       --         --------   ----   24.10       16.30     3
3                                                                              3
3                                                                              3
3                                        Total Side Friction  :      16.30     3
3                                                                              3
3                        POINT RESISTANCE CONTRIBUTION                         3
3                                                                              3
3 Effective      Internal      SPT      Pile End      Bearing      End Bearing 3
3 Stress at      Friction      Value      Area        Capacity     Resistance  3
3 pile Tip        Angle                                Factor                  3
3   (psf)                               (ft*ft)          Nq           (Kips)   3
3                                                                              3
3  1250.80        43.00*      68.39      0.99         307.00         296.64    3
3                                                                              3
3                          Limiting End Bearing Resistance :         667.67    3
3                                                                              3
3                                                                              3
3                          Ultimate Static Pile Capacity   :         312.94    3
3                                                                              3
ÀÄÄÄÄÄ Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu ÄÄÄÄÄÙ
```

@ Abut1 :

$\frac{16.3}{400} = \sim 4\%$

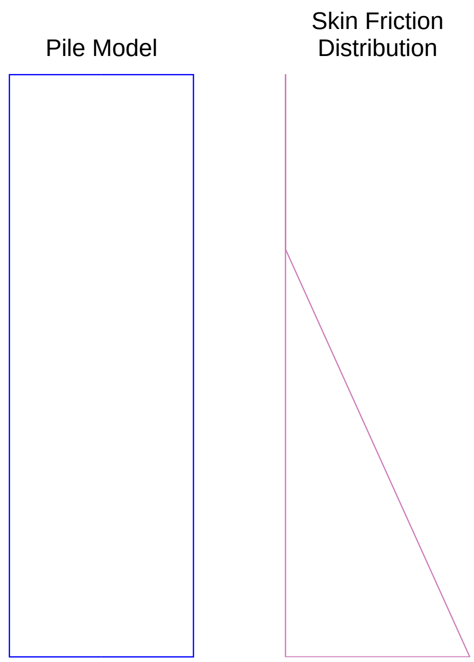
GZA Geo Environmental, Inc.
Route 155 Abutment 1

28-Oct-2013
GRLWEAP Version 2010



DELMAG D 22-02

Ram Weight	4.85	kips
Efficiency	0.800	
Pressure	729 (72%)	psi
Helmet Weight	1.90	kips
Hammer Cushion	60155	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.070	in
Skin Damping	0.100	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	20.00	ft
Pile Penetration	14.00	ft
Pile Top Area	15.50	in2



Res. Shaft = 4 %
(Proportional)

GZA Geo Environmental, Inc.
Route 155 Abutment 1

12x53 pile

28-Oct-2013
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
100.0	19.85	0.00	1.4	4.28	10.40
200.0	28.49	0.00	3.8	5.14	9.06
300.0	35.25	0.72	6.5	5.60	9.29
400.0	42.91	0.21	11.0	6.10	10.05
500.0	48.91	0.89	20.0	6.57	10.93
600.0	52.95	2.38	46.1	6.96	11.61
700.0	56.07	3.92	153.3	7.26	12.20
800.0	57.52	4.78	9999.0	7.40	12.44

435 kips at 45 ksi at 14 blows per inch

ABUT2BOX.OUT

ÚÄÄÄÄÄÄÄÄ ULTIMATE STATIC PILE CAPACITY/Federal Highway Administration ÄÄÄÄÄÄÄÄ¿

Nordlund (1963, 1979) and Tomlinson (1979, 1980) methods

Project Name : RT 155 Client : MDOT
File Name : Abut 2 Project Manager : CLS
Date : 10/28/13 Computed by : JRB

Depth of Top of Pile = 0.00 ft. Pile length = 19.00 ft.
Depth to Water Table = 16.00 ft.
Type of Pile = H Pile
HP 12x53

SKIN FRICTION CONTRIBUTION

Layer	Soil Type	Thickness (ft)	Effective Stress (psf)	Internal Friction Angle	N-SPT	Pile Perimeter (ft)
1	Cohesionless	16.00	1000.00	32.00	--	3.97
2	Cohesionless	3.00	2101.40	34.00	--	3.97

Layer	Soil Type	Undrained Shear Strength (psf)	Adhesion	Pile Taper	Sliding Friction Angle	Skin Resistance (Kips)
1	Cohesionless	--	--------	----	24.10	22.92
2	Cohesionless	--	--------	----	25.61	10.60

Total Side Friction : 33.52

POINT RESISTANCE CONTRIBUTION

Effective Stress at pile Tip (psf)	Internal Friction Angle	SPT Value	Pile End Area (ft*ft)	Bearing Capacity Factor Nq	End Bearing Resistance (Kips)
2202.80	42.71*	58.54	0.99	282.47	479.24

Limiting End Bearing Resistance : 644.65

Ultimate Static Pile Capacity : 512.76

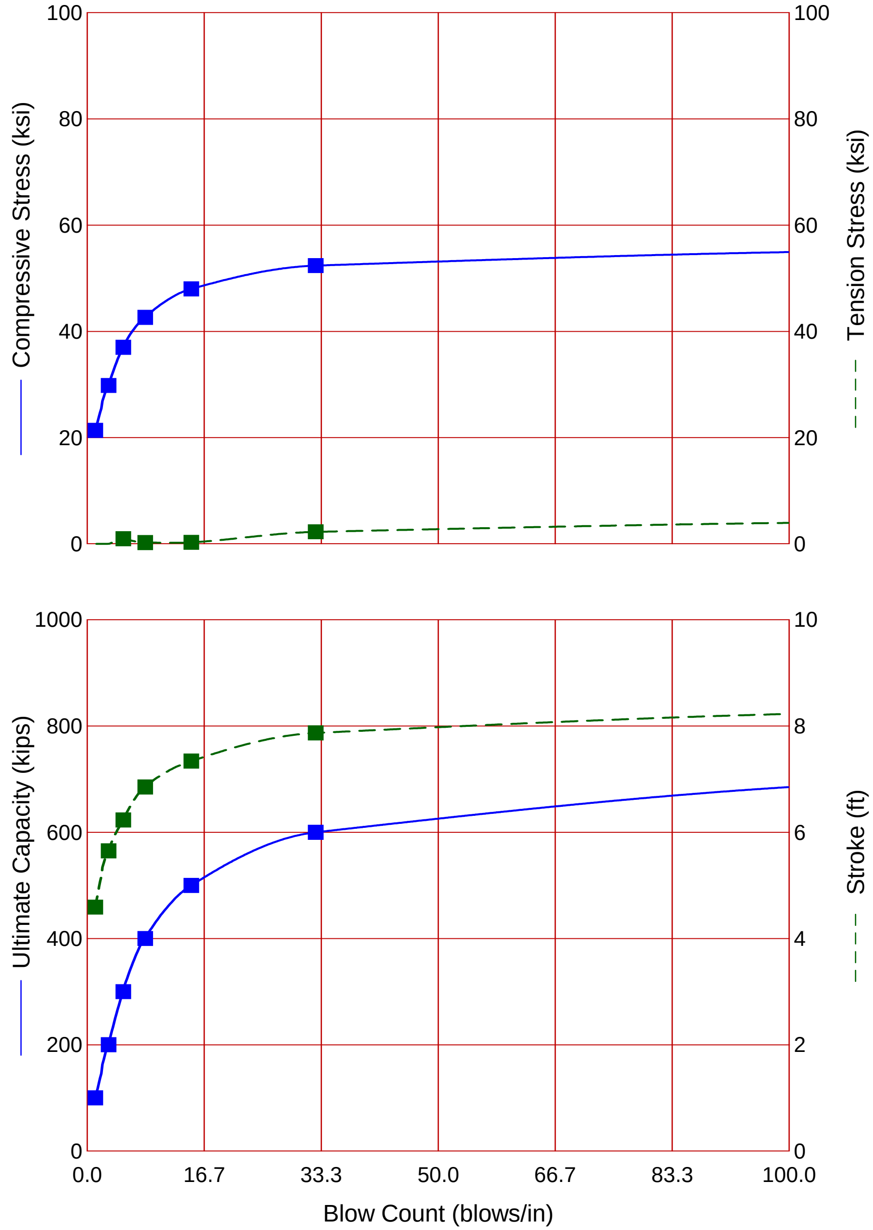
ÀÄÄÄÄÄ Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu ÄÄÄÄÄÙ

@ Abut 2:

$\frac{33.52}{400} = \sim 8\%$

GZA Geo Environmental, Inc.
Route 155 Abutment 2

28-Oct-2013
GRLWEAP Version 2010

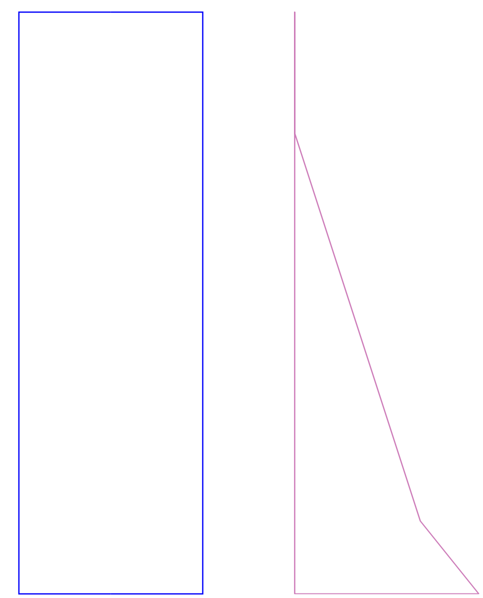


DELMAG D 22-02

Ram Weight	4.85	kips
Efficiency	0.800	
Pressure	810 (81%)	psi
Helmet Weight	1.90	kips
Hammer Cushion	60155	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.070	in
Skin Damping	0.100	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	24.00	ft
Pile Penetration	19.00	ft
Pile Top Area	15.50	in2

Pile Model

Skin Friction Distribution



Res. Shaft = 8 %
(Proportional)

GZA Geo Environmental, Inc. 28-Oct-2013
Route 155 Abutment 2 12x53 pile GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
100.0	21.36	0.00	1.2	4.59	12.92
200.0	29.80	0.00	3.1	5.65	11.24
300.0	37.00	0.96	5.2	6.23	11.64
400.0	42.61	0.26	8.3	6.85	12.57
500.0	47.99	0.30	14.8	7.34	13.30
600.0	52.38	2.27	32.6	7.87	14.17
700.0	55.37	4.26	133.4	8.29	14.90
800.0	56.78	4.68	9999.0	8.45	15.17

444 kips at 45 ksi at 11 blows per inch

PIERSBOX.OUT

ÚÄÄÄÄÄÄÄÄ ULTIMATE STATIC PILE CAPACITY/Federal Highway Administration ÄÄÄÄÄÄÄÄ¿

Nordlund (1963, 1979) and Tomlinson (1979, 1980) methods

Project Name	: RT 155	Client	: MDOT
File Name	: piers	Project Manager	: CLS
Date	: 10/28/13	Computed by	: JRB

Depth of Top of Pile = 0.00 ft. Pile length = 50.00 ft.
Depth to Water Table = 0.00 ft.
Type of Pile = H Pile
HP 12x53

SKIN FRICTION CONTRIBUTION

Layer	Soil Type	Thickness (ft)	Effective Stress (psf)	Internal Friction Angle	N-SPT	Pile Perimeter (ft)
1	Cohesionless	25.00	720.00	32.00	--	3.97
2	Cohesionless	25.00	2285.00	34.00	--	3.97

Layer	Soil Type	Undrained Shear Strength (psf)	Adhesion	Pile Taper	Sliding Friction Angle	Skin Resistance (Kips)
1	Cohesionless	--	--------	----	24.11	25.82
2	Cohesionless	--	--------	----	25.62	96.18

Total Side Friction : 121.99

POINT RESISTANCE CONTRIBUTION

Effective Stress at pile Tip (psf)	Internal Friction Angle	SPT Value	Pile End Area (ft*ft)	Bearing Capacity Factor Nq	End Bearing Resistance (Kips)
3130.00	41.72*	53.60	0.99	232.20	551.87

Limiting End Bearing Resistance : 558.94

Ultimate Static Pile Capacity : 673.87

ÀÄÄÄÄÄ Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu ÄÄÄÄÄÙ

For Embedment
From El. 97 - El. 70
Qs = 105

$\frac{105}{400} = \sim 26\%$

GZA
GeoEnvironmental, Inc.
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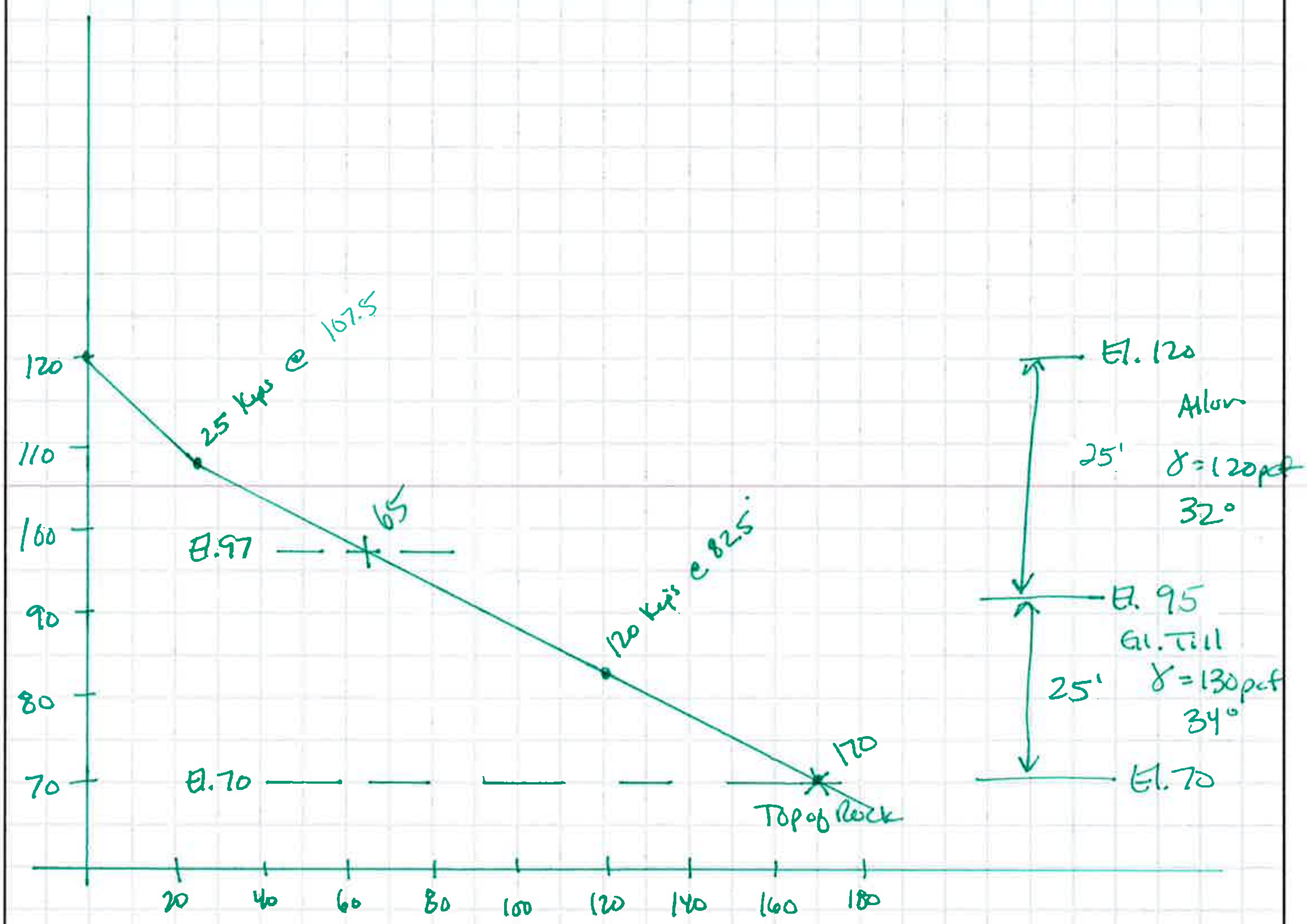
Fax (207) 879-0099
http://www.gza.com

Engineers and Scientists

JOB Howland
SHEET NO Side Friction Calc OF
Calculated By JRB Date 10/28/13
Checked By Date
Scale

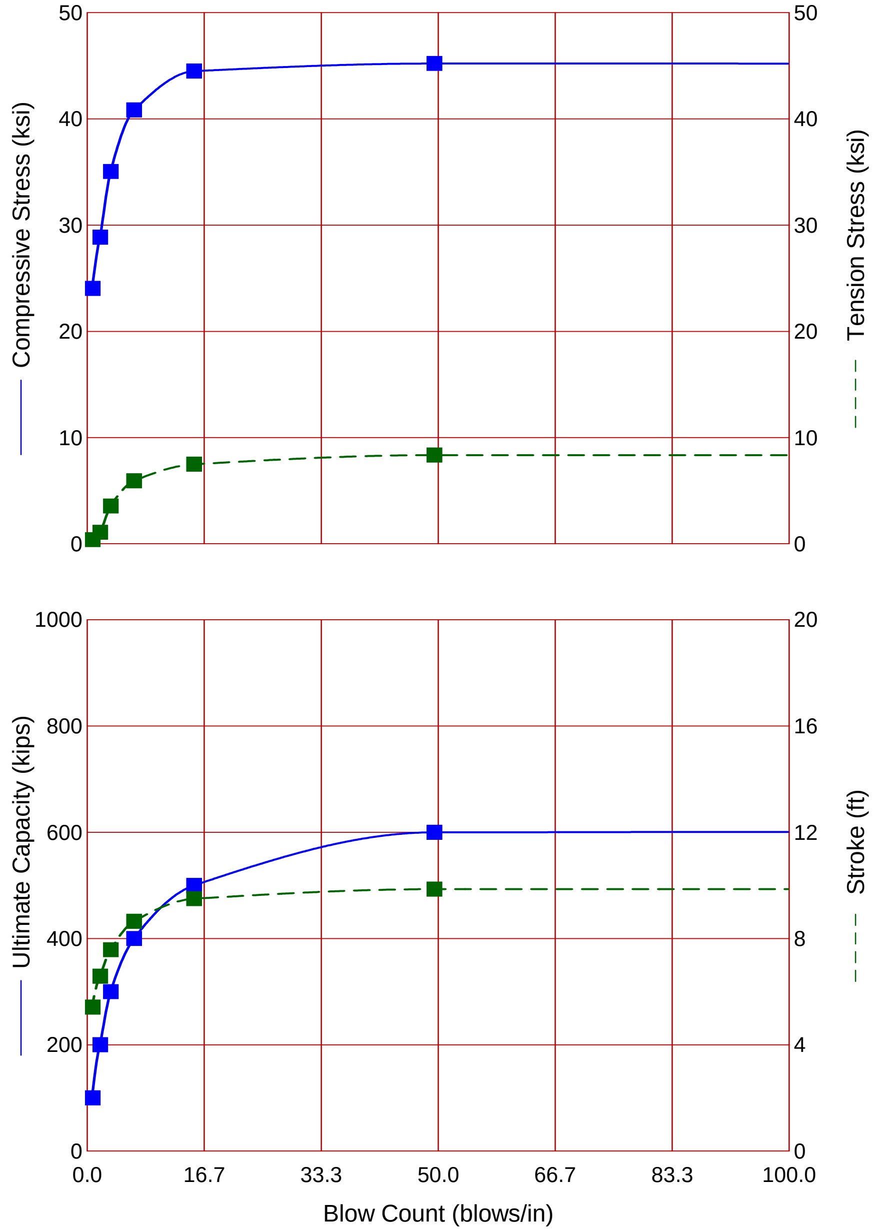


For Embedment
from El. 97-70

$$Q_s = 170 - 65 = 105$$

GZA Geo Environmental, Inc.
Route 155 Pier

28-Oct-2013
GRLWEAP Version 2010

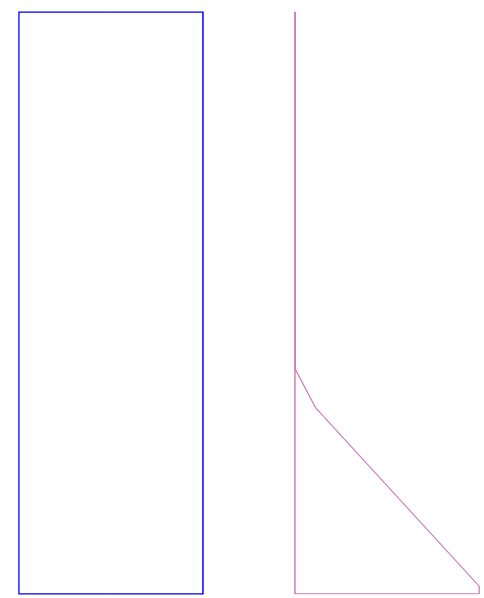


DELMAG D 22-02

Ram Weight	4.85	kips
Efficiency	0.800	
Pressure	1000 (100%)	psi
Helmet Weight	1.90	kips
Hammer Cushion	60155	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.070	in
Skin Damping	0.100	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	75.00	ft
Pile Penetration	29.00	ft
Pile Top Area	15.50	in2

Pile Model

Skin Friction Distribution



Res. Shaft = 25 %
(Proportional)

GZA Geo Environmental, Inc. 28-Oct-2013
Route 155 Pier 12x53 pile GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
100.0	24.04	0.40	0.8	5.42	19.49
200.0	28.86	1.10	1.9	6.58	18.90
300.0	35.04	3.57	3.4	7.58	20.94
400.0	40.83	5.93	6.7	8.65	23.52
500.0	44.49	7.50	15.2	9.50	25.86
600.0	45.21	8.36	49.5	9.86	26.86
700.0	44.74	8.76	9999.0	10.00	27.22

ABUT2_14.OUT

ÚÄÄÄÄÄÄÄÄ ULTIMATE STATIC PILE CAPACITY/Federal Highway Administration ÄÄÄÄÄÄÄÄ¿

Nordlund (1963, 1979) and Tomlinson (1979, 1980) methods

Project Name	:	RT 155	Client	:	MDOT
File Name	:	Abut 2	Project Manager	:	CLS
Date	:	10/28/13	Computed by	:	JRB

Depth of Top of Pile = 0.00 ft. Pile length = 19.00 ft.
Depth to Water Table = 16.00 ft.
Type of Pile = H Pile
HP 14x73

SKIN FRICTION CONTRIBUTION

Layer	Soil Type	Thickness (ft)	Effective Stress (psf)	Internal Friction Angle	N-SPT	Pile Perimeter (ft)
1	Cohesionless	16.00	1000.00	32.00	--	4.70
2	Cohesionless	3.00	2101.40	34.00	--	4.70

Layer	Soil Type	Undrained Shear Strength (psf)	Adhesion	Pile Taper	Sliding Friction Angle	Skin Resistance (Kips)
1	Cohesionless	--	--------	----	25.15	30.65
2	Cohesionless	--	--------	----	26.72	14.33

Total Side Friction : 44.98

POINT RESISTANCE CONTRIBUTION

Effective Stress at pile Tip (psf)	Internal Friction Angle	SPT Value	Pile End Area (ft*ft)	Bearing Capacity Factor Nq	End Bearing Resistance (Kips)
2202.80	42.71*	58.54	1.38	282.47	670.45

Limiting End Bearing Resistance : 901.85

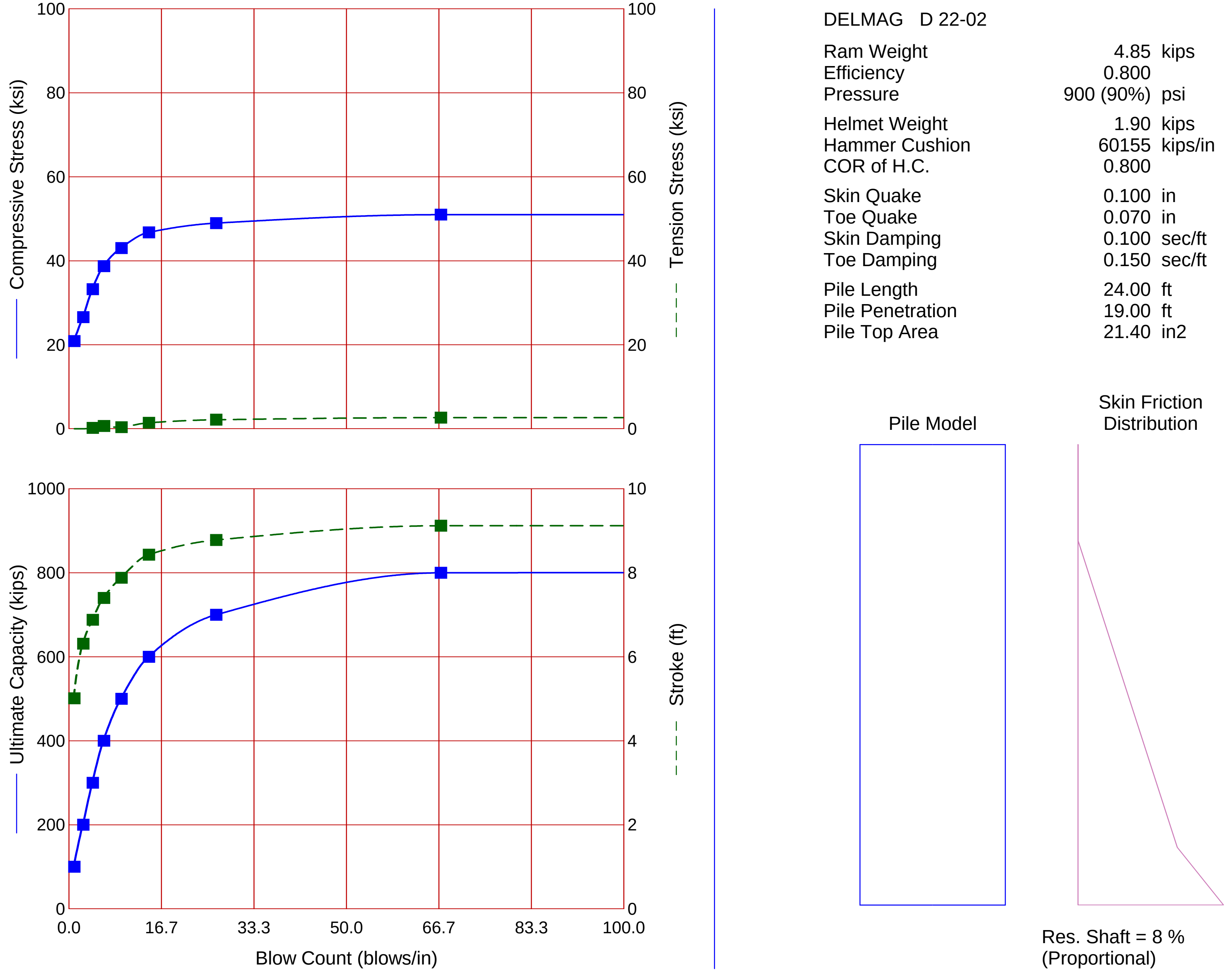
Ultimate Static Pile Capacity : 715.43

ÀÄÄÄÄÄ Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu ÄÄÄÄÄÙ

@ Abut 2 (14x73) : 44.98 / 550 = ~8%.

DELMAG D 22-02

Ram Weight	4.85	kips
Efficiency	0.800	
Pressure	900 (90%)	psi
Helmet Weight	1.90	kips
Hammer Cushion	60155	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.070	in
Skin Damping	0.100	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	24.00	ft
Pile Penetration	19.00	ft
Pile Top Area	21.40	in2



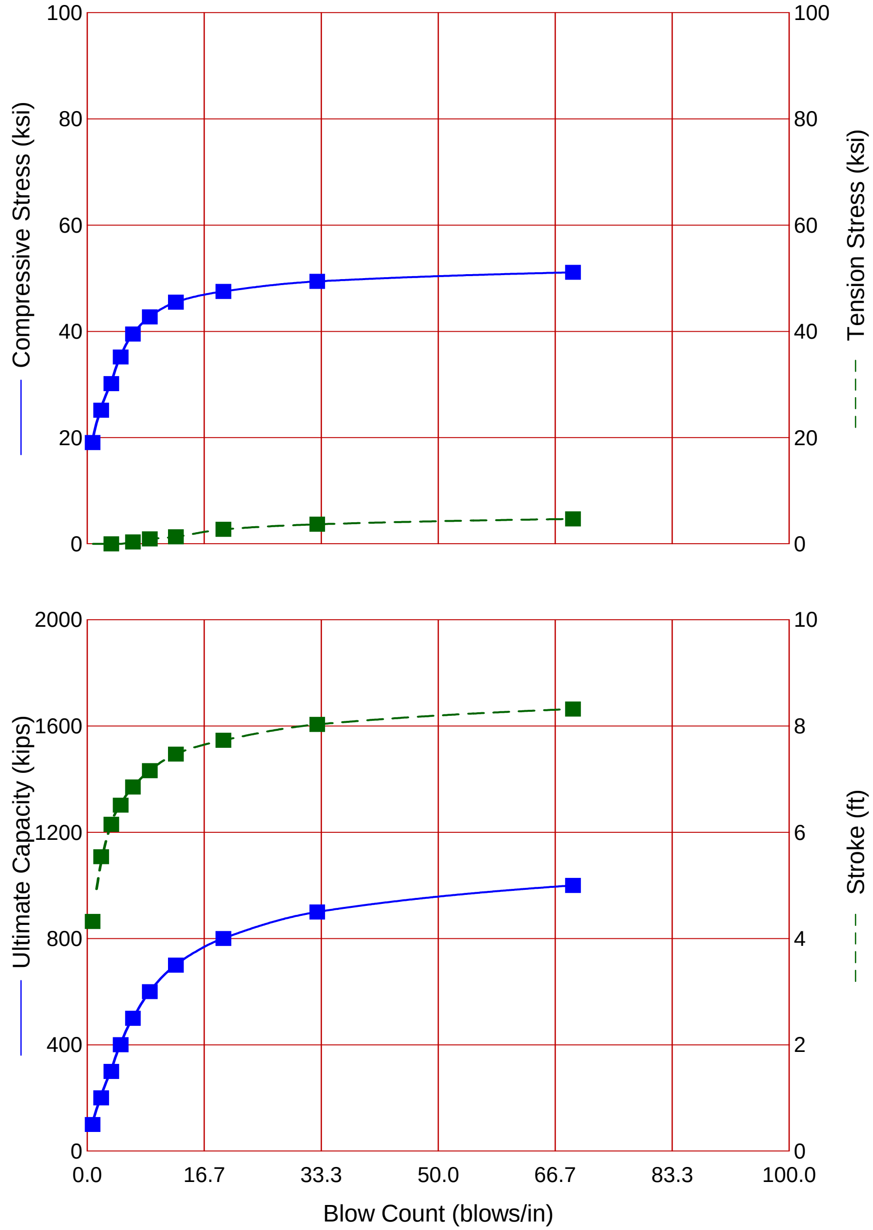
GZA Geo Environmental, Inc. | 28-Oct-2013
Route 155 Abutment 2 | 14x73 pile | GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
100.0	20.88	0.00	1.0	5.01	15.89
200.0	26.55	0.00	2.6	6.31	13.81
300.0	33.22	0.22	4.3	6.88	13.49
400.0	38.72	0.68	6.3	7.40	13.88
500.0	43.02	0.39	9.5	7.88	14.32
600.0	46.75	1.44	14.4	8.43	15.16
700.0	48.97	2.16	26.6	8.78	15.58
800.0	50.98	2.67	67.0	9.12	16.32
900.0	52.46	3.19	9999.0	9.39	16.89

553 kips at 45 ksi at 12 blows per inch

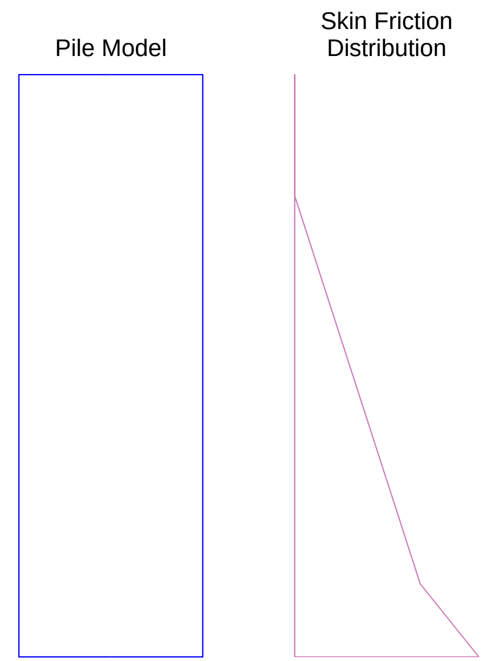
GZA Geo Environmental, Inc.
Route 155 Abutment 2

28-Oct-2013
GRLWEAP Version 2010



DELMAG D 30-32

Ram Weight	6.60	kips
Efficiency	0.800	
Pressure	1021 (72%)	psi
Helmet Weight	1.90	kips
Hammer Cushion	60155	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.070	in
Skin Damping	0.100	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	24.00	ft
Pile Penetration	19.00	ft
Pile Top Area	26.10	in2



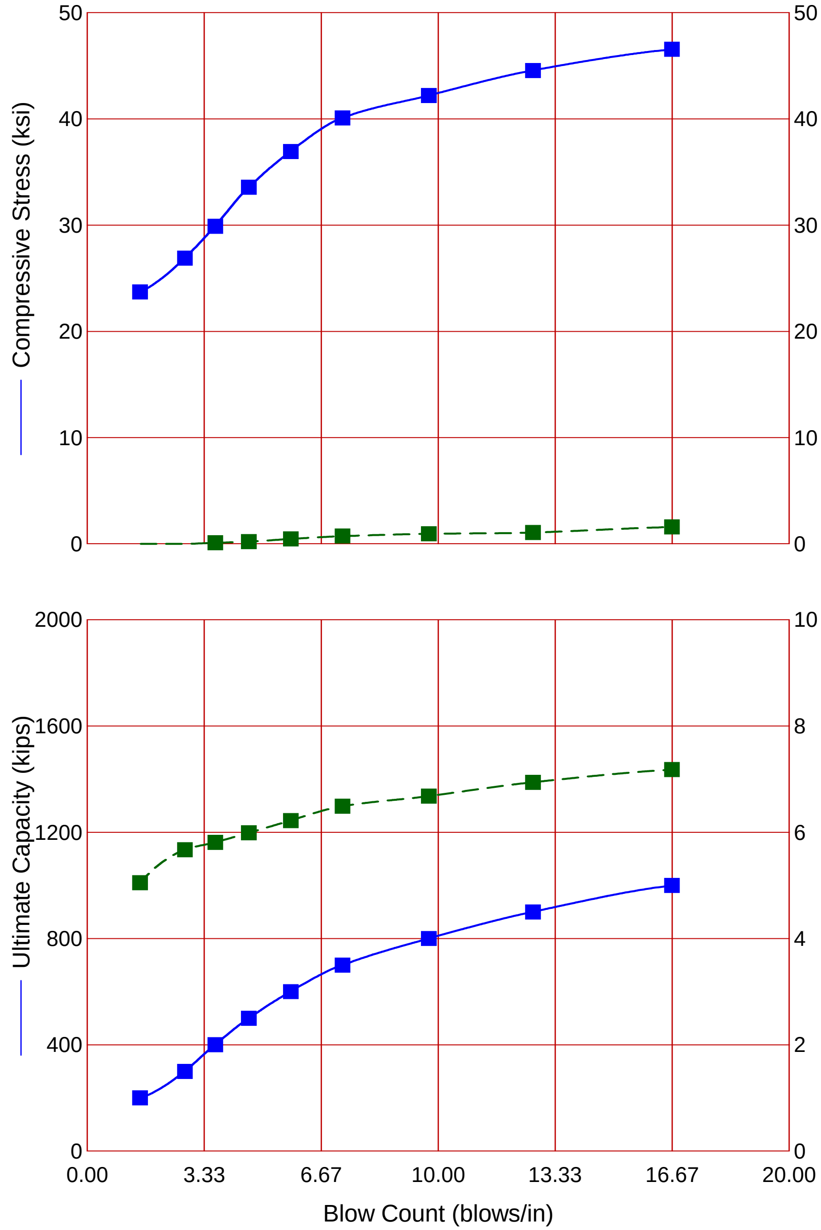
Res. Shaft = 8 %
(Proportional)

GZA Geo Environmental, Inc. 28-Oct-2013
Route 155 Abutment 2 14x89 pile GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
100.0	19.07	0.00	0.8	4.32	20.17
200.0	25.15	0.00	2.0	5.54	17.48
300.0	30.15	0.00	3.4	6.15	16.72
400.0	35.17	0.00	4.8	6.51	16.96
500.0	39.49	0.35	6.5	6.85	17.32
600.0	42.73	0.94	8.9	7.16	17.87
700.0	45.48	1.33	12.6	7.47	18.36
800.0	47.51	2.73	19.4	7.73	19.21
900.0	49.40	3.69	32.8	8.03	20.22
1000.0	51.11	4.70	69.2	8.32	21.19

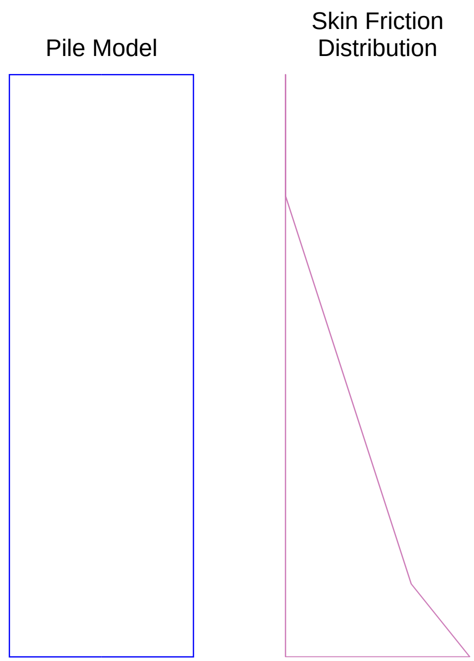
GZA Geo Environmental, Inc.
Route 155 Abutment 2

28-Oct-2013
GRLWEAP Version 2010



DELMAG D 46-02

Ram Weight	10.14	kips
Efficiency	0.800	
Pressure	802 (72%)	psi
Helmet Weight	1.90	kips
Hammer Cushion	60155	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.070	in
Skin Damping	0.100	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	24.00	ft
Pile Penetration	19.00	ft
Pile Top Area	34.40	in2



Res. Shaft = 8 %
(Proportional)

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
200.0	23.71	0.00	1.5	5.05	20.99
300.0	26.88	0.00	2.8	5.67	19.63
400.0	29.88	0.11	3.6	5.81	19.93
500.0	33.56	0.22	4.6	5.99	20.55
600.0	36.91	0.47	5.8	6.22	21.23
700.0	40.07	0.73	7.3	6.49	22.13
800.0	42.19	0.95	9.7	6.68	22.38
900.0	44.54	1.07	12.7	6.94	23.31
1000.0	46.54	1.59	16.7	7.18	24.26