MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

ENGLISHMAN RIVER BRIDGE
JOHNSON COVE ROAD OVER ENGLISHMAN RIVER
ROQUE BLUFFS, MAINE

Prepared by:
Brandon Slaven
Assistant Geotechnical Engineer

Reviewed by:
Laura Krusinski, P.E.
Senior Geotechnical Engineer

Washington County
WIN 17881.00

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Table of Contents

GEOTECHNICAL DESIGN SUMMARY **1**

1.0 INTRODUCTION **4**

2.0 GEOLOGIC SETTING **5**

3.0 SUBSURFACE INVESTIGATION **5**

4.0 LABORATORY TESTING **6**

5.0 SUBSURFACE CONDITIONS **6**

5.1 FILL 6
5.2 NATIVE SANDS AND GRAVEL 7
5.3 GLACIOMARINE SILTY CLAY 7
5.4 GLACIAL TILL 8
5.5 BEDROCK 9
5.6 GROUNDWATER 9

6.0 FOUNDATION ALTERNATIVES **10**

7.0 GEOTECHNICAL DESIGN PARAMETERS **10**

7.1 INTEGRAL ABUTMENT H-PILES 10
7.1.1 STRENGTH LIMIT STATE DESIGN 11
7.1.2 SERVICE AND EXTREME LIMIT STATE DESIGN 13
7.1.3 LATERAL PILE RESISTANCE/BEHAVIOR 15
7.1.4 DRIVEN PILE RESISTANCE AND PILE QUALITY CONTROL 16
7.2 INTEGRAL ABUTMENT DESIGN 17
7.3 IN-LINE WINGWALLS 18
7.4 SETTLEMENT 19
7.5 FROST PROTECTION 19
7.6 SCOUR AND RIPRAP 20
7.7 SEISMIC DESIGN CONSIDERATIONS 20
7.8 CONSTRUCTION CONSIDERATIONS 21

8.0 CLOSURE **21**

Tables

Table 1 – Summary of Atterberg Limits Tests - Glaciomarine
Table 2 – Summary of Atterberg Limits Test - Glacial Till
Table 3 – Summary of Approximate Bedrock Depths, Elevations and RQD
Table 4 – Estimated Pile Lengths for Integral Abutments No. 1 and No. 2
Table 5 – Factored Axial Resistances for Abutment H-Piles at the Strength Limit State
Table 6 – Factored Axial Resistances for Abutment H-Piles at the Service and Extreme Limit States
Table 7 – Soil Parameters for Generation of Soil-Resistance (p-y) Curves at Abutment No. 1
Table 8 – Soil Parameters for Generation of Soil-Resistance (p-y) Curves at Abutment No. 2
Table 9 – Equivalent Height of Soil for Estimating Live Load Surcharge on Abutments

Sheets

Sheet 1 – Location Map
Sheet 2 – Boring Location Plan
Sheet 3 – Interpretive Subsurface Profile
Sheet 4 – Boring Logs

Appendices

Appendix A – Boring Logs
Appendix B – Laboratory Test Results
Appendix C – Calculations

GEOTECHNICAL DESIGN SUMMARY

The purpose of this Geotechnical Design Report is to present subsurface information and provide geotechnical design requirements for the replacement of Englishman River Bridge which carries Johnson Cove Road over Englishman River in Roque Bluffs, Maine. The proposed replacement structure will consist of a single span superstructure founded on H-pile supported integral abutments. Due to the number of feasible superstructure options, this project will be advertised as a "Detail-Build" project to achieve the lowest market-cost superstructure. The following design recommendations are discussed in detail in this report:

Integral Abutment H-piles – H-piles for support of the integral abutments should be end bearing and driven to the required resistance on or within bedrock. H-piles should be 50 ksi, Grade A572 steel and fitted with pile tips. The H-piles shall be designed for all relevant strength, service, and extreme limit state load groups. The structural resistance checks should include checking axial, lateral and flexural resistance. The resistance of the piles should be evaluated for compliance with the interaction equation for combined axial load and flexure. It is recommended that final design include lateral pile resistance analyses using L-PILE or FB-Multipier software. Recommended geotechnical parameters for generation of soil-resistance (p-y) curves in lateral pile analyses are provided in Section 7.1.3 of this report.

The contractor is required to perform a wave equation analysis. The first pile driven at each abutment shall be dynamically tested to confirm nominal pile resistance and verify the stopping criteria developed by the contractor in the wave equation analysis. Minimum 24-hour restrike tests will be required due to poor quality bedrock. With this level of quality control, the pile should be driven to a nominal resistance equal to the factored axial pile load divided by a resistance factor, φ_{dyn}, of 0.65. Additional tests may be required as part of the pile field quality control program if pile behavior indicates a pile is refusing on a boulder or cobble above bedrock, is not seated firmly within bedrock, or if piles "walk" out of position.

Integral Abutment Design – Integral abutments shall be designed for all relevant strength, service and extreme limit states and load combinations. Calculation of passive earth pressures for integral abutment design should assume a Coulomb passive earth pressure coefficient, K_p, of 6.73. If the ratio of the calculated lateral abutment movement to abutment height (y/H) is less than 0.005, the designer may consider using the Rankine passive earth pressure coefficient of 3.25. For purposes of the integral abutment backwall reinforcing steel design, use a maximum load factor (γ_{EH}) of 1.50 to calculate factored passive earth pressures.

The abutment design shall include a drainage system behind the abutment to intercept any groundwater. The approach slab should be positively connected to the integral abutment. Additional lateral earth pressure due to construction surcharge or live load surcharge is required if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination of the surcharge load is permitted.

In-line Wingwalls – In-line, "butterfly" wingwalls will be used in conjunction with the integral abutments. The walls shall be designed to resist lateral earth pressures, vehicular loads, collision loads, creep and temperature shrinkage deformations, and the additional bending stresses resulting from the wingwall being cantilevered off the abutment. The design of the wingwalls shall, at a minimum, consider a load case where the wingwall is subjected to passive earth pressure to account for the bridge moving laterally and pushing the wingwall into the fill. There are no bearing resistance considerations or special foundation support needed for wingwalls that are cantilevered off the abutment.

Settlement – A 3-foot raise in the roadway grade is planned. The glaciomarine clayey silts encountered in the test borings were determined to be overconsolidated to heavily overconsolidated and of limited thickness. The fine grained glacial till sublayers encountered were also determined to be heavily overconsolidated. Therefore, recompression settlement in these soil deposits due to the raise in grade is anticipated to be negligible. Furthermore, the fill soils, native sands, and glacial till deposits encountered at the borings are loose to dense in consistency. Elastic settlements in these cohesionless soils are anticipated to occur relatively quickly during construction. Post construction settlement will be minimal.

Frost Protection – Integral abutments shall be embedded a minimum of 4 feet for frost protection. Foundations placed on or in granular soils should be founded a minimum of 5.5 feet below finished exterior grade for frost protection.

Scour and Riprap – For scour protection and protection of pile supported integral abutments the bridge approach slopes and slopes at abutments shall be armored at a minimum with 4 feet of heavy riprap. The toe of the heavy riprap section shall be constructed 3 feet below the streambed elevation and final ground surface. The top of the riprap should be located at a minimum elevation of 10.8 feet. The riprap shall be underlain by Class 1 nonwoven erosion control geotextile and 1 foot thick layer of bedding material.

Seismic Design Considerations – Englishman River Bridge is in Seismic Zone 1 and the response acceleration coefficient is less than 0.15g, therefore no consideration for seismic forces is required except that superstructure connections and minimum support length requirements shall be satisfied.

Construction Considerations – Construction of the abutments will require pile driving and soil excavation. Cofferdams and/or temporary lateral earth support systems may be required to permit construction of driven pile foundations.

The new integral abutments will be constructed in front of the existing abutments. The existing bridge abutments are founded on steel H-piles. The plans will call for the existing substructures to be removed to below the streambed. There is a potential that existing steel H-piles that are not removed in their entirety will obstruct pile driving operations at the proposed abutments. Work by the Contractor to excavate portions of the pre-existing abutments to allow pile driving operations will be considered incidental to the contract pay item for piles in place.

The site is subject to extreme tidal fluctuations. The Contractor should protect excavations and cut slopes from wave action and tidal fluctuations.

Exposed soils may become saturated and water seepage may be encountered during construction. There may be localized sloughing and instability in some excavations and cut slopes. The contractor should control groundwater, surface water infiltration and soil erosion. Water should be controlled by pumping from sumps.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and provide geotechnical design requirements for the replacement of Englishman River Bridge which carries Johnson Cove Road over Englishman River in Roque Bluffs, Maine. This report presents the subsurface information obtained at the site during the subsurface investigation and geotechnical design parameters for design of the new bridge substructures.

The existing Englishman River Bridge was constructed in 1949. The three-span painted steel girder bridge has span lengths of 25, 45, and 25 feet. The existing abutments are steel H-pile frames with steel cross beams and gravel borrow backfill, armored with heavy riprap. The existing piers consist of steel H-pile bents with cross bracing. There have been several pile repairs resulting in the north (west) abutment crossbeam and beam girder ends and all H-pile members of the pier bents being encased in concrete.

The bridge is in overall poor to serious condition according to the 2014 Maine Department of Transportation (MaineDOT) Bridge Inspection Report. Heavy section loss is noted in the exposed H-piles, crossbracing of the pier bents, and bottom flanges of the girders. The substructure is rated as 3 for “serious”. The concrete deck is rated as 4 for “poor” due to a number of areas with spalling and delamination. The superstructure is also rated as “serious” with moderate to heavy deterioration. The bridge has a Sufficiency Rating of 19.6.

The replacement structure will be a 86 or 87-foot single-span, integral abutment bridge. The integral abutments will be founded on H-piles driven to bedrock. Several feasible superstructure replacement options were identified during preliminary design and are detailed in the Preliminary Design Report (PDR). These are butted prestressed concrete boxes, New England Bulb Tees (NEBT’s), NEXT beams, Hillman composite hybrid beams, galvanized steel I-girders and concrete box beams. Due to the number of feasible bridge replacement options available this project will be advertised as a “Detail-Build” project to assure lowest market-cost superstructure. In lieu of standard plans, the contract documents will specify design codes, design loads, substructure type, minimum bottom chord elevation, and span length, among other requirements. The new integral abutments will be located approximately 5 feet in front of the existing abutments to account for the decreased span length.

The new Englishman River Bridge will be located on nearly the same horizontal alignment as the existing bridge. The vertical alignment of the replacement bridge will require approximately 513 feet of approach work and an approximate 3-foot raise in roadway grade at the bridge approaches. The new bridge will accommodate two (2) 11-foot lanes with 3-foot shoulders. A temporary one-lane bridge will be constructed on the upstream side to maintain alternating traffic during construction.

2.0 GEOLOGIC SETTING

Englishman River Bridge in Roque Bluffs crosses the Englishman River as shown on Sheet 1 – Location Map. The Englishman River is tidal at the bridge site.

The Maine Geological Survey (MGS) Surficial Geology Map of the Roque Bluffs Quadrangle, Maine, Open-file No. 75-5 (1975), indicates the surficial soils in the vicinity of the bridge project consist of end moraine deposits and glacial till with nearby contacts to glaciomarine deposits. End moraine deposits in coastal areas are composed of till, sand and gravel, and were deposited by glacial ice and/or meltwater streams in the marginal zone of the Late Wisconsinan glacier. Glacial till is a heterogeneous mixture of sand, silt, clay, and stones commonly deposited in a blanket that conforms to the bedrock surface. Glaciomarine deposits, known locally as the Presumpscot Formation, generally consist of clay and silt that washed out of the Late Wisconsinan glacier and accumulated on the ocean floor when the relative sea level was higher than at present.

The Bedrock Geologic Map of Maine, MGS (1985), cites the bedrock at the proposed bridge site as basaltic volcanic rock of the Eastport Formation.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling four (4) test borings terminating with bedrock cores. Borings BB-RBER-101 and BB-RBER-104 were located to explore subsurface conditions for a possible new bridge alignment approximately 25 feet upstream of the existing structure. Test borings BB-RBER-102 and BB-RBER-103 were located to determine subsurface conditions along the current bridge alignment and were drilled behind the existing south and north abutments, respectively. The test boring locations are shown on Sheet 2 – Boring Location Plan.

Test borings BB-RBER-103 and BB-RBER-102 were drilled on July 26, 2011 and September 12 and 13, 2011 by the MaineDOT Drill Crew. Test borings BB-RBER-101 and BB-RBER-104 were drilled on September 13 and 14, 2011 by Northern Test Boring (NTB) of Gorham, Maine. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs.

All borings were performed using cased wash boring and solid stem auger techniques. Soil samples were typically obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the split spoon sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. Both the MaineDOT and NTB drill rigs are equipped with automatic hammers to drive the split spoon. Both hammers were calibrated per ASTM D4633 “Standard Test Method for Energy Measurement for Dynamic Penetrometers” in March of 2010. All N-values discussed in this report are corrected values computed by applying the corresponding average energy transfer

factor of 0.84 (for the MaineDOT hammer) and 0.783 (for the NTB hammer) to the raw field N-values. The hammer efficiency factors (0.84 and 0.783), the raw field N-values, and the corrected N-values are shown on the boring logs.

The bedrock was cored in the four (4) borings using an NQ-2" core barrel and the Rock Quality Designation (RQD) of the cores was calculated. A Northeast Transportation Technician Certification Program (NETTCP) Certified Subsurface Inspector logged the subsurface conditions encountered. The MaineDOT geotechnical engineer selected the boring locations and drilling methods, designated type and depth of sampling techniques, reviewed draft boring logs and identified field and laboratory testing requirements. The elevations and coordinates of the as-drilled borings were located after completion of the drilling program by MaineDOT Survey.

4.0 LABORATORY TESTING

A laboratory testing program was conducted on selected soil samples recovered from test borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the project site.

Soil laboratory testing consisted of eighteen (18) standard grain size analyses with natural water content, three (3) grain size analyses with hydrometer and natural moisture content, and three (3) Atterberg Limits tests. The results of soil laboratory tests are included as Appendix B – Laboratory Test Results. Laboratory test information is also shown on the boring logs provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the borings generally consisted of granular fill, native sands and gravels, glaciomarine deposits, and glacial till all underlain by igneous volcanic bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs. A generalized subsurface profile is shown on Sheet 3 – Interpretive Subsurface Profile. The following paragraphs discuss the subsurface conditions encountered in the borings in detail:

5.1 Fill

A layer of granular fill was encountered in all four test borings. The fill layer is approximately 4.0 to 13.0 feet thick at the boring locations. Samples of the fill unit were:

- Brown, damp to wet, sandy gravel, trace silt;
- Brown or grey-brown, damp to wet, gravelly sand, with trace silt, trace shell fragments, and
- Brown, damp, sand, some gravel, trace silt.

SPT N-values in the fills ranged from 5 to 83 blows per foot (bpf) indicating that the fill soils are loose to very dense in consistency.

Three (3) grain size analysis of the fill soils resulted in the soil being classified as A-1-a under the AASHTO Soil Classification System and SW, SW-SM or GW under the Unified Soil Classification System (USCS). The natural water content of the samples tested ranged from approximately 2 to 8 percent.

5.2 Native Sands and Gravel

A layer of native sands and gravels was encountered below the fill unit in all the test borings. The deposit was approximately 8.5 to 17.5 feet thick at the boring locations. The granular soils generally consisted of:

- Grey, wet, gravelly sand, trace silt;
- Grey, wet, sand, some to trace gravel, trace silt;
- Grey-brown, wet, sand, some gravel, trace to little silt, trace shells;
- Brown, moist to wet, gravelly sand, trace silt;
- Brown, moist, sandy gravel, trace silt, and
- Brown, wet, sand, some gravel, trace silt.

SPT N-values in the fills ranged from 6 to 55 bpf indicating the deposit is loose to very dense in consistency.

Six (6) grain size analyses of the native granular soils resulted in the soil being classified as A-1-a, A-1-b, or A-3 under the AASHTO Soil Classification System and SP, SW, SP-SW, SW-SM, or GW-GM under the USCS. The moisture contents of the tested samples ranged from approximately 2 to 25 percent.

5.3 Glaciomarine Silty Clay

A layer of glaciomarine silty clay was encountered beneath upper sands and gravels on the north side of the project in borings BB-RBER-103 and BB-RBER-104. The glaciomarine deposit encountered was approximately 5.0 to 5.5 feet thick at the boring locations and consisted of grey, wet, silty clay, trace gravel, trace sand.

SPT N-values in the deposit ranged from 10 to 11 bpf indicating the silty clay is stiff in consistency.

Two (2) grain size analyses with hydrometer conducted on the silty clay resulted in the soil being classified as A-4 under the AASHTO Soil Classification System and CL under the USCS. Atterberg Limits tests were conducted on two (2) samples of the glaciomarine soils. The moisture contents of the tested samples ranged from approximately 22 to 23 percent and

plastic limits ranged from 14 to 19. Table 1 below summarizes the results of Atterberg Limits tests conducted on samples of the glaciomarine soil:

Sample No.	Visual Soil Description	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-RBER-103, 5D	Silty CLAY	21.6	33	19	14	0.19
BB-RBER-104, 3D	Silty CLAY	22.9	31	14	17	0.52

Table 1 – Summary of Atterberg Limits Tests **-** Glaciomarine

Interpretation of these results indicate that since the measured water content is between the liquid limit and the plastic limit the silty clay is overconsolidated to heavily overconsolidated.

5.4 Glacial Till

A layer of glacial till was encountered in all the borings. The thickness of the glacial till deposit ranged from was approximately 18 feet in boring BB-RBER-102 to approximately 35.3 in boring BB-RBER-104. The glacial till encountered consisted of:

- Grey or brown, wet, sandy silt, little to trace gravel;
- Grey, wet, silt, some sand, some to trace gravel;
- Grey, wet, silty sand, little to trace gravel;
- Grey, wet, sand, some silt, little gravel, and
- Grey, wet, silty clay, trace fine sand.

SPT N-values in the coarse-grained glacial till ranged from 20 to 34 bpf indicating the coarse-grained glacial till deposits are medium dense to dense in consistency. SPT N-values in the fine-grained glacial till deposits ranged from 10 to 29 bpf indicating those subunits are stiff to very stiff in consistency.

Three (3) grain size analyses of samples from the coarse-grained glacial till deposit resulted in the soil being classified as A-2-4 and A-4 under the AASHTO Soil Classification System and SM under the USCS. The natural water contents of the samples tested ranged from approximately 9 to 20 percent.

Six (6) grain size analyses of samples from the fine-grained glacial till deposits indicated those soils are classified as A-4 or A-A-6 under the AASHTO Soil Classification System and CL, CL-ML, SC-SM, or SM under the USCS. The moisture contents of the tested samples ranged from approximately 10 to 27 percent. One (1) Atterberg Limits test was conducted on a sample of the fine-grained glacial till. Table 2 summarizes the results of Atterberg Limits test conducted on the fine-grained sample:

Sample No.	Visual Soil Description	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-RBER-104, 7D	Silty CLAY	27.2	35	18	17	0.54

Table 2 – Summary of Atterberg Limits Test **-** Glacial Till

Interpretation of the results indicate that since the measured water content is between the liquid limit and the plastic limit the fine-grained glacial till layers are overconsolidated to heavily overconsolidated.

5.5 Bedrock

Bedrock was encountered and cored in all of the borings. Table 3 summarizes approximate depths to bedrock, corresponding approximate top of bedrock elevations and RQD.

Boring	Station	Offset (feet)	Approximate Depth to Bedrock (feet)	Approximate Elevation of Bedrock Surface (feet)	RQD (R1, R2) (%)
BB-RBER-104	12+61.7	50 ft Lt	53.8	-41.9	67, 75
BB-RBER-103	12+91.4	1.5 ft Rt	50.2	-36.2	0, 63
BB-RBER-102	14+28.6	0.9 ft Rt	49.0	-34.7	38, 18
BB-RBER-101	14+55.3	30.4 ft Lt	49.6	-38.9	46, 70

Table 3 – Summary of Approximate Bedrock Depths, Elevations and RQD

The bedrock at the site is identified as dark gray, aphanitic to fine grained, basalt, soft, fresh to slight weathered, joints at low angles to steep dipping, very close to close, tight to open, with quartz or minor silt infilling. The RQD of the bedrock was determined to range from 0 to 75 percent correlating to a rock mass quality of very poor to fair. Detailed bedrock descriptions and the RQD of each core run are provided on Sheet 4 – Boring logs and in Appendix A – Boring Logs.

5.6 Groundwater

Groundwater was measured at depths ranging from approximately 8 to 8.5 feet bgs in the borings. The water levels measured upon completion of drilling are indicated on the boring logs found in Appendix A. Note that water was introduced into the boreholes during the drilling operations. Therefore, the water levels indicated on the boring logs may not represent

stabilized groundwater conditions. Groundwater levels will fluctuate with changes in water levels in the tidal river, seasonal changes, precipitation, runoff, and construction activities.

6.0 FOUNDATION ALTERNATIVES

During preliminary design, pile-supported integral abutments, with piles driven to bedrock, were identified as the most cost effective and preferred superstructure type. Several superstructure replacement options were also identified as feasible during preliminary design. These were butted prestressed concrete boxes, New England Bulb Tees (NEBT's), NEXT beams, Hillman composite hybrid beams, galvanized steel I-girders and concrete box beams. Due to the number of feasible superstructure replacement options available this project will be advertised as a "Detail-Build" project to assure lowest market-cost superstructure. In lieu of standard plans, the contract documents will specify design codes, design loads, substructure type, minimum bottom chord elevation, and span length, among other requirements.

7.0 GEOTECHNICAL DESIGN PARAMETERS

The following sections provide geotechnical design considerations and recommendations for H-pile supported integral bridge abutments, which have been selected for the substructures for the Englishman River Bridge replacement project.

7.1 Integral Abutment H-Piles

Abutments No. 1 and No. 2 will be integral abutments founded on a single row of H-piles. The piles should be end bearing and driven to the required resistance on bedrock or within bedrock. Piles may be HP 12x53, 12x74, 14x73, 14x89, or 14x117 depending on the factored design axial loads. H-piles should be 50 ksi, Grade A572 steel. The piles should be oriented for weak axis bending. Abutment No. 1 and Abutment No. 2 piles should be fitted with pile tips conforming to MaineDOT Standard Specification 711.10, to protect the tips, improve penetration and improve friction at the pile tip.

Based on interpolation between bedrock surface elevations encountered in the borings, pile lengths at the proposed abutments may be estimated based on Table 4:

Location	Estimated Bottom Elevation of Proposed Abutment (feet)	Interpolated Top of Bedrock Elevation at Proposed Centerline (feet)	Estimated Pile Lengths[1] (feet)
Abutment No. 1	5.0	-36.0	41
Abutment No. 2	5.0	-35.0	40

Table 4 – Estimated Pile Lengths for Integral Abutments No. 1 and No. 2

The estimated pile lengths do not take into account embedment in the abutment, locations where bedrock may be deeper or shallower than that encountered in the test borings, damaged pile, the additional five (5) feet of pile required for dynamic testing instrumentation (per ASTM D4945) or additional pile length needed to accommodate the contractor's leads and driving equipment.

7.1.1 Strength Limit State Design

The design of pile foundations bearing on or within bedrock at the strength limit state shall consider:

- Compressive axial geotechnical resistance of individual piles bearing on bedrock
- drivability resistance of individual piles driven to bedrock
- structural resistance of individual piles in axial compression
- structural resistance of individual piles in combined axial loading and flexure

The pile groups should be designed to resist all lateral earth loads, vehicular loads, dead and live loads, and lateral forces transferred through the pile caps.

Per AASHTO LRFD Bridge Design Specifications 7th Edition (LRFD) Article 6.5.4.2, at the strength limit state, the axial resistance factor ϕ_c = 0.60 (good driving conditions) shall be applied to the structural compressive resistance of the pile. Since the H-piles will be subjected to lateral loading, the piles shall also be checked for resistance against combined axial compression and flexure as prescribed in LRFD Articles 6.9.2.2 and 6.15.2. This design axial load may govern the design. Per LRFD Article 6.5.4.2, at the strength limit state, the axial resistance factor ϕ_c = 0.70 and the flexural resistance factor ϕ_f = 1.0 shall be applied to the combined axial and flexural resistance of the pile in the interaction equation

[1] Pile lengths do not include embedment into the pile cap.

(LRFD Eq. 6.9.2.2-1 or -2). Integral abutment H-piles shall also be analyzed for determination of unbraced lengths and fixity using L-Pile or FB-MultiPier software.

LRFD specifies that the pile group, after scour due to the design flood, provide adequate foundation resistance using the resistance factors given in this section. It is noted that scour depths were not calculated by the designer for the 100-year flood. Difficulty in determining tidal wave heights with the limited available hydrologic data at the site and the cost of obtaining the necessary data was considered prohibitive. Further discussion can be found in the Preliminary Design Report dated March 24, 2014. Local experience and site observations have determined that special scour countermeasures will be required (refer to Section 7.6 Scour and Riprap).

Structural Resistance. The nominal axial compressive structural resistance (P_n) for piles loaded in compression shall be as specified in LRFD Article 6.9.4.1. Preliminary estimates of the structural axial resistance of five (5) H-pile sections were calculated using a resistance factor, ϕ_c = 0.60 (for good driving conditions) and an effective length factor (K) of 1.2. These factored axial compressive structural resistance factors are presented in Table 5. Supporting calculations are provided in Appendix C – Calculations. The unbraced pile length (L) and effective length factor (K) used in these evaluations have been assumed. It is the responsibility of the structural engineer to recalculate the nominal axial structural compressive resistance based on actual unbraced lengths and effective length factors determined from L-Pile or FB-MultiPier analyses for the pile section chosen for design.

Geotechnical Resistance. The nominal axial geotechnical resistance in the strength limit state was calculated using the guidance in LRFD Article 10.7.3.2.3 which states the nominal bearing resistance of piles driven to point bearing on hard rock shall not exceed the structural pile resistances obtained from LRFD Article 6.9.4.1 with a resistance factor ϕ_c, of 0.50, for severe driving conditions applied. The resulting limiting factored geotechnical compressive resistances for piles driven to rock are provided in Table 5.

Drivability Analyses. Drivability analyses were performed to determine the pile resistance that might be achieved considering available diesel hammers. The maximum driving stresses in the pile, assuming the use of 50 ksi steel, shall be less than 45 ksi. The drivability resistances were calculated using the resistance factor, $\varphi_{dyn,}$ of 0.65, for a single pile in axial compression when a dynamic test is performed as specified in LRFD Table 10.5.5.2.3-1. This factored drivability resistance is provided in Table 5.

A summary of the calculated factored axial compressive structural, geotechnical and drivability resistances of five (5) H-piles sections for each abutment for the strength limit state is provided in Table 5. Supporting calculations are provided in Appendix C – Calculations.

Pile Section	Strength Limit State Factored Axial Pile Resistance			
	Structural Resistance[2] ϕ_c=0.60 (kips)	Controlling Geotechnical Resistance[3] (kips)	Drivability Resistance φ_{dyn} = 0.65 (kips)	Governing Axial Pile Resistance (kips)
HP 12 x 53[4]	464	387	314	314
HP 12 x 74	653	544	374	374
HP 14 x 73[4]	641	534	372	372
HP 14 x 89	782	652	398	398
HP 14 x 117	1031	859	432 (510)[5]	432 (510)[5]

Table 5 – Factored Axial Resistances for Abutment H-Piles at the Strength Limit State

LRFD Article 10.7.3.2.3 states that the nominal axial compressive resistance of piles driven to hard rock is typically controlled by the structural resistance with a resistance factor for severe driving conditions applied. However, for the site conditions, the estimated factored axial pile resistances from the drivability analyses for the H-pile sections are less than the controlling factored axial structural resistance per LRFD Article 10.7.3.2.3. Therefore, drivability controls, and the recommended governing resistances for pile design are the drivability resistances provided in the rightmost column "Governing Axial Pile Resistance (kips)" in Table 5, above. The maximum applied factored axial pile load should not exceed the governing factored pile resistance shown in Table 5 above.

7.1.2 Service and Extreme Limit State Design

The design of H-piles at the service limit state shall consider tolerable transverse and longitudinal movement of the piles, overall global stability of the pile group, and pile group movements/stability considering changes in soil conditions due to scour during the design storm event. For the service limit state, resistance factors of ϕ = 1.0 should be used in accordance with LRFD Article 10.5.5.1. The exception is the overall global stability of the foundation that should be investigated at the Service I load combination and a resistance factor, ϕ, of 0.65.

Extreme limit state design checks for the H-piles shall include pile axial compressive resistance, overall global of the pile group, pile failure by uplift in tension, and structural

[2]Structural resistances were calculated for approximated unbraced pile segments assuming L=1 foot and K=1.2.
[3]Based on guidance in LRFD Article 10.7.3.2.3., *Piles Driven to Hard Rock*
[4]Does not consider resistance factors of slender elements. 12x53 and 14x73 H-pile sections require additional reductions based upon structural performance considerations.
[5]Drivability resistance based on a Delmag D19-42. Drivability resistance with a Delmag D36-32 shown in parentheses.

failure. The extreme event load combinations are those related to seismic forces, ice loads, debris loads, and certain hydraulic events. Extreme limit state design shall also check that the nominal pile foundation resistance remaining after scour due to the check flood can support the extreme limit state loads. Resistance factors for extreme limit states, per LRFD Article 10.5.5.3 shall be taken as ϕ = 1.0, with the exception of uplift of piles, for which the resistance factor, φ_{up}, shall be 0.80 or less per LRFD Article 10.5.5.3.2.

The nominal axial geotechnical pile resistance in the service and extreme limit state was calculated using the guidance in LRFD Article 10.7.3.2.3. The calculated factored axial structural, geotechnical and drivability resistances of five (5) H-pile sections for the service and extreme limit states are provided in Table 6. Supporting documentation is provided in Appendix C – Calculations.

Pile Section	Service and Extreme Limit State Factored Axial Pile Resistance			
	Structural Resistance (normal conditions)[6] ϕ_c=1.0 (kips)	Controlling Geotechnical Resistance ϕ_c=1.0[7] (kips)	Drivability Resistance φ = 1.0 (kips)	Governing Axial Pile Resistance (kips)
HP 12 x 53[8]	775	774	483	483
HP 12 x 74	1090	1088	576	576
HP 14 x 73[8]	1070	1069	573	573
HP 14 x 89	1305	1303	613	613
HP 14 x 117	1720	1718	665 (785)[9]	665 (785)[9]

Table 6 – Factored Axial Resistances for Abutment H-Piles at the Service and Extreme Limit States

LRFD Article 10.7.3.2.3 states that the nominal axial compressive resistance of piles driven to hard rock is typically controlled by the structural resistance with a resistance factor for severe driving conditions applied. However, for the site conditions, the estimated factored axial pile resistances from the drivability analyses for the H-pile sections are less than the controlling factored axial structural resistance per LRFD Article 10.7.3.2.3 and the nominal structural resistances. Local experience also supports the estimated factored resistances from the drivability analyses. Therefore, drivability controls and the recommended governing resistances for pile design are the resistances provided in the rightmost column "Governing

[6] Normal conditions consider no soil loss due to scour. Nominal structural resistances were calculated for an approximate unbraced pile segment. Controlling value shown here is using a resistance factor, ϕ = 1.0.
[7] Based on guidance in LRFD Article 10.7.3.2.3., *Piles Driven to Hard Rock.*
[8] Does not consider resistance factors of slender elements. 12x53 and 14x73 H-pile sections require additional reductions based upon structural performance considerations
[9] Drivability resistance based on a Delmag D19-42. Drivability resistance with a Delmag D36-32 shown in parentheses.

Axial Pile Resistance (kips)" in Table 6 above. The maximum applied factored axial pile load for the extreme and service limits states should not exceed the governing factored pile resistance shown in Table 6.

LRFD specifies the pile foundations have adequate resistance after soil loss due to the Q_{500} flood to resist extreme limit state loads. The factored structural resistance considering soil loss from the Q_{500} scour event and a resulting unbraced length has not been determined where scour depths were not calculated for the 500-year event. Therefore, special scour countermeasures will be required.

7.1.3 Lateral Pile Resistance/Behavior

In accordance with LRFD Article 6.15.1, the structural analysis of pile groups subjected to lateral loads shall include explicit consideration of soil-structure interaction effects as specified in LRFD Article 10.7.3.9. Assumptions regarding a fixed or pinned condition at the pile tip should be also confirmed with soil-structure interaction analyses.

A series of lateral pile resistance analyses should be performed to evaluate pile behavior at both abutments using L-Pile or FB-MultiPier software. The structural engineer shall utilize lateral pile analyses to determine maximum factored lateral and axial loads permissible based on allowable displacement and pile stress criteria.

Recommended geotechnical parameters for generation of soil-resistance (p-y) curves in lateral pile analyses are provided in Table 7 and Table 8. In general, the models developed emulate the soil at the site by using the soil layers (referenced in Table 7 and Table 8 by elevations) and using appropriate structural parameters and pile-head boundary conditions for the pile section being analyzed.

Soil Layer	Approx. Elevation of Soil Layer (feet)	Water Table Condition	Effective Unit Weight lbs/in^3 (lbs/ft^3)	k_s (lb/in^3)	Cohesion psi (psf)	E_{50}	Internal Angle of Friction
Sand and Gravel Fill (medium dense)	14 to 5	Above	0.723 (125)	90	-	-	32°
Sands (dense)	5 to -3	Below	.0365 (63)	125	-	-	32°
Silty Clay (stiff)	-3 to -8	Below	0.0318 (55)	500	10.42 (1500)	0.005	-
Glacial Till, Silt (stiff)	-8 to -25	Below	0.033 (58)	500	10.42 (1500)	0.005	-
Glacial Till (dense)	-25 to -36	Below	0.038 (65)	125	-	-	34°

Table 7 – Soil Parameters for Generation of Soil-Resistance (p-y) Curves at Abutment No. 1

Soil Layer	Approx. Elevation of Soil Layer (feet)	Water Table Condition	Effective Unit Weight lbs/in^3 (lbs/ft^3)	k_s (lb/in^3)	Cohesion lb/in^2 (lb/ft^2)	E_{50}	Internal Angle of Friction
Sand Fill (medium dense to dense)	5 to 14	Above	0.723 (125)	90	-	-	32°
Sand Fill (medium dense)	1 to 5	Below	.0365 (63)	60	-	-	32°
Sands (loose to dense)	1 to -16	Below	.0365 (63)	40	-	-	32°
Glacial Till (medium dense)	-16 to -35	Below	0.035 (63)	60	-	-	34°

Table 8 – Soil Parameters for Generation of Soil-Resistance (p-y) Curves at Abutment No. 2

7.1.4 Driven Pile Resistance and Pile Quality Control

The contract plans shall require the contractor to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test with signal matching at each abutment. The first pile driven at each abutment should be dynamically tested to confirm nominal pile resistance and verify the stopping criteria developed by the contractor in the wave equation analysis. Minimum 24-hour restrike tests will be required for test piles. Additional dynamic

tests may be required as part of the pile field quality control program should pile behavior vary radically between adjacent piles, should pile behavior indicate a pile is refusing on a boulder or in a cobble layer above bedrock, or if the pile should "walk" out of position.

With this level of quality control, the ultimate resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor, ϕ_{dyn}, of 0.65. The maximum factored axial pile load should be shown on the plans.

Piles should be driven to an acceptable penetration resistance as determined by the contractor based on the results of a wave equation analysis and as approved by the Resident. Driving stresses in the pile determined in the drivability analysis shall be less than 45 ksi, in accordance with LRFD Article 10.7.8. A hammer should be selected which provides the required pile resistance when the penetration resistance for the final 3 to 6 inches is 3 to 15 blows per inch (bpi). If an abrupt increase in driving resistance is encountered, the driving could be terminated when the penetration is less than 0.5-inch in 10 consecutive blows.

7.2 Integral Abutment Design

Integral abutment sections shall be designed for all relevant strength, service and extreme limit states and load combinations specified in LRFD Articles 3.4.1 and 11.5.5. Stub abutments shall be designed to resist all lateral earth loads, vehicular loads, dead and live loads, and lateral forces transferred through the integral superstructure. The design of the integral abutment at the strength limit state shall consider reinforced-concrete structural design. Strength limit state design shall also consider changes in foundation conditions and foundation resistance after scour due to the design (Q_{100}) flood.

A resistance factor (ϕ) of 1.0 shall be used to assess abutment design at the service limit state, including: settlement, excessive horizontal movement, and movement resulting after scour due to the design (Q_{100}) flood. The overall stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, ϕ, of 0.65.

Extreme limit state design of integral abutment supported on H-piles shall include pile structural resistance, pile geotechnical resistance, pile resistance in combined axial and flexure, and overall stability. Resistance factors for extreme limit state shall be taken as 1.0. Extreme limit state design shall also check that the nominal foundation resistance remaining after scour due to the check (Q_{500}) flood can support the extreme limit state loads with a resistance factor of 1.0.

The designer may assume Soil Type 4 (MaineDOT Bridge Design Guide (BDG) Section 3.6.1) for abutment backfill material soil properties. The backfill properties are as follows: $\phi = 32°$, $\gamma = 125$ pcf and a soil-concrete interface friction angle of 20 degrees.

Integral abutment sections shall be designed to withstand a lateral earth load equal to the passive pressure state. Calculation of passive earth pressures should assume a Coulomb

passive earth pressure coefficient, K_p, of 6.73. Developing full passive pressure assumes that the ratio of lateral abutment movement to abutment height (y/H) exceeds 0.005. If the calculated displacements are significantly less than that required to develop full passive pressure the designer may consider using the Rankine passive earth pressure coefficient of 3.25. A load factor for passive earth pressure is not specified in LRFD. For purposes of the integral abutment backwall reinforcing steel design, use a maximum load factor (γ_{EH}) of 1.50 to calculate factored passive earth pressures.

Additional lateral earth pressure due to live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for abutments if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination of the surcharge load is permitted per LRFD Article 3.11.6.5. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 9 below:

Abutment Height (feet)	h_{eq} (feet)
5	4.0
10	3.0
≥20	2.0

Table 9 – Equivalent Height of Soil for Estimating Live Load Surcharge on Abutments

The abutment design shall include a drainage system behind the abutment to intercept any groundwater. We recommend weep holes be constructed approximately 6 inches above high water. Drainage behind the structure shall be in accordance with MaineDOT BDG Section 5.4.1.9.

Backfill within 10 feet of the abutments and side slope fill shall conform to Granular Borrow for Underwater Backfill – MaineDOT Specification 703.19. This gradation specifies 7 percent or less of the material passing the No. 200 sieve. This material is specified in order to reduce the amount of fines and to minimize frost action behind the structure.

Slopes in front of the pile supported integral abutments should be set back from the riverbank and should be constructed with riprap and erosion control geotextile. The slopes should not exceed 1.75H:1V unless project specific slope stability analyses are performed.

7.3 In-line Wingwalls

In-line, “butterfly” wingwalls will be used in conjunction with the integral abutments. The wingwalls shall be designed for all relevant strength, service limit and extreme states and load combinations specified in LRFD Articles 3.4.1, 11.5.5 and 11.6. The walls shall be designed to resist lateral earth pressures, vehicular loads, collision loads, and creep and temperature and shrinkage deformations. The design of “butterfly” wingwalls shall account for the additional bending stresses resulting from the wingwall being cantilevered off the abutment.

The design of the wingwalls shall at a minimum consider a load case where the wingwall is subjected to passive earth pressure to account for the bridge moving laterally and pushing the wingwall into the fill. Calculation of passive earth pressures may assume a Rankine passive earth pressure coefficient, K_p, of 3.25 assuming small wingwall movements. A load factor for passive earth pressure is not specified in LRFD; use a maximum load factor (γ_{EH}) of 1.50 to calculate factored passive earth pressures. The wingwalls shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) per LRFD Article 3.11.6.4.

There are no bearing resistance considerations or special foundation support needed for wingwalls that are cantilevered off the abutment.

7.4 Settlement

A 3-foot raise in the roadway grade is planned at the bridge. The glaciomarine clayey silts encountered in the test borings were determined to be overconsolidated to heavily overconsolidated and of limited thickness (less than 5.5 feet). The fine grained glacial till sublayers encountered were also determined to be heavily overconsolidated. Therefore, recompression settlement due in these soil deposits due to the raise in grade is anticipated to be negligible. Furthermore, the fill soils, native sands, and glacial till deposits encountered at the borings are loose to dense in consistency. Elastic settlements in these cohesionless soils due to the proposed raise in grade are anticipated to occur relatively quickly during construction. Post construction settlement will be minimal.

Any settlement of the bridge abutments should be due to axial compression of the foundation piles and is anticipated to be minimal.

7.5 Frost Protection

Pile-supported integral abutments shall be embedded a minimum of 4.0 feet for frost protection per Figure 5-2 of the MaineDOT BDG.

Foundations placed on fill side slopes should be designed with an appropriate embedment for frost protection. According to BDG Figure 5-1, Maine Design Freezing Index Map, Roque Bluffs has a design freezing index of approximately 1100 F-degree days. An assumed water content of 10% was used for coarse grained soils. These components correlate to a frost depth of approximately 5.8 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Roque Bluffs was assigned a design freezing index of approximately 1102 F-degree days, for Eastport, the closest location in the Modberg database. An assumed water content of 10% was used for coarse grained fill soils above the water table. These components correlate to a frost depth of approximately 5.3 feet. Based on an average of

these results, we recommend foundations be designed with an embedment of approximately 5.5 feet for frost protection. See Appendix C – Calculations for supporting documentation.

Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection.

7.6 Scour and Riprap

For scour protection of the pile supported abutments, the PDR indicates the bridge approach slopes and the abutment slopes will be armored with a minimum of 4.0 feet of heavy riprap. The top of the riprap should be located at a minimum elevation of 2 feet above Mean Higher High Water (MHHW) considering a 2-foot increase in sea level, or Elev. 10.8. Refer to MaineDOT BDG Section 2.3.11 for additional information regarding scour design. For this project, the top of heavy riprap along the bridge approach slopes should be extended to meet the roadway shoulder in order to prevent erosion and loss of the roadway approach embankments during a storm surge.

Heavy riprap shall conform MaineDOT Standard Specification number 703.28 – Heavy Riprap and be placed at a maximum slope of 2H:1V along side slopes and 1.75H:1V in front of the abutments. The toe of the heavy riprap section shall be constructed 3 feet below the streambed elevation and final ground. The riprap section shall be underlain by a 1 foot thick layer of bedding material conforming to item number 703.19 of the Standard Specification and Class 1 nonwoven erosion control geotextile per Standard Details 610(02) through 610(04).

7.7 Seismic Design Considerations

The following parameters were determined for the site from the USGS Seismic Parameters CD provided with the LRFD manual and LRFD Articles 3.10.3.1 and 3.10.6:

- Peak Ground Acceleration coefficient (PGA) = 0.068g
- Site Class D (based on an average N-value for the upper 100 ft of the soil profile greater than 15 bpf and less than 50 bpf, using steps in LRFD Table C3.10.3.1-1)
- Acceleration coefficient (As) = 0.109g
- Design spectral acceleration coefficient at 0.2-second period (S_{DS}) = 0.222g
- Design spectral acceleration coefficient at 1.0-second period, (S_{D1})= 0.092g
- Seismic Zone 1, based on a $S_{D1} \leq 0.15$g

See Appendix C – Calculations for supporting documentation.

In conformance with LRFD 5.10.11.2, no consideration for seismic forces is required except that superstructure connections and minimum support length requirements shall be satisfied per LRFD Articles 3.10.9.2 and 4.7.4.4, respectively.

7.8 Construction Considerations

Construction of the abutments will require pile driving and soil excavation. Cofferdams and/or temporary lateral earth support systems may be required to permit construction of driven pile foundations at the proposed abutments.

The new integral abutments will be constructed in front of the existing abutments. The existing bridge abutments are founded on steel H-piles. The plans will call for the existing substructures to be removed to below the streambed. There is a potential that steel H-pile that are not removed in their entirety will obstruct pile driving operations at the proposed abutments. Work by the Contractor to excavate portions of the pre-existing abutments to allow pile driving operations will be considered incidental to the contract pay item for piles in place.

The site is subject to extreme tidal fluctuations. The Contractor should protect excavations and cut slopes from wave action and tidal fluctuations.

Exposed soils may become saturated and water seepage may be encountered during construction. There may be localized sloughing and instability in some excavations and cut slopes. The contractor should control groundwater, surface water infiltration and soil erosion. Water should be controlled by pumping from sumps.

8.0 CLOSURE

This report has been prepared for use by the MaineDOT Bridge Program for the specific application of the proposed replacement of Englishman River Bridge in Roque Bluffs, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied.

In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. Further, the analyses and recommendations are based in part upon limited soil explorations at discrete locations completed at the site. If variations from the conditions encountered during the investigation appear evident during construction, it may also become necessary to re-evaluate the recommendations made in this report.

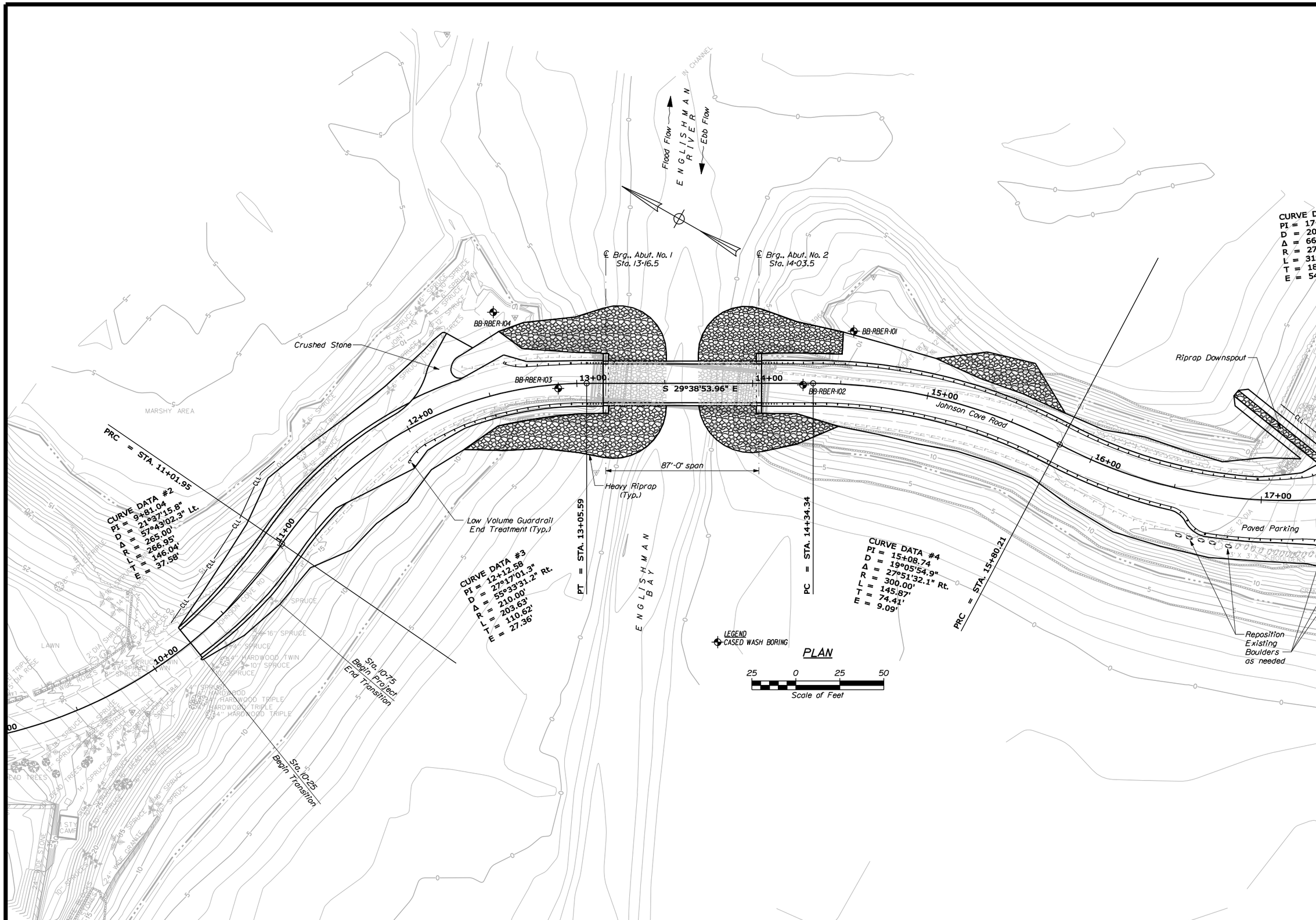
We also recommend that we be provided the opportunity for a general review of the final design and specifications in order that the earthwork and foundation recommendations may be properly interpreted and implemented in the design.

Sheets

Location Map
Englishman River Bridge #3964 carries
Johnson Cove Road over Englishman River
Roque Bluffs, Maine
Washington County
PIN. 17881.00 Federal No. BR-1788(100)X
USGS 7.5' Series Topographic
Machias and Roque Bluffs Quadrangles
DeLorme Map 26 Grid D3

Map Scale 1:24000

The Maine Department of Transportation provides this publication for information only. Reliance upon this information is at user risk. It is subject to revision and may be incomplete depending upon changing conditions. The Department assumes no liability if injuries or damages result from this information. This map is not intended to support emergency dispatch. Road names used on this map may not match official road names.



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1788(100)X

PIN 17881.00

BRIDGE NO. 3964

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	M.WRIGHT	
DESIGN-DETAILED		
CHECKED-REVIEWED		
DESIGN2-DETAILED2	B.SLAVEN	
DESIGN3-DETAILED3	T.WHITE	MAY 2015
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

ENGLISHMAN RIVER BRIDGE
ENGLISHMAN RIVER
ROQUE BLUFFS WASHINGTON COUNTY

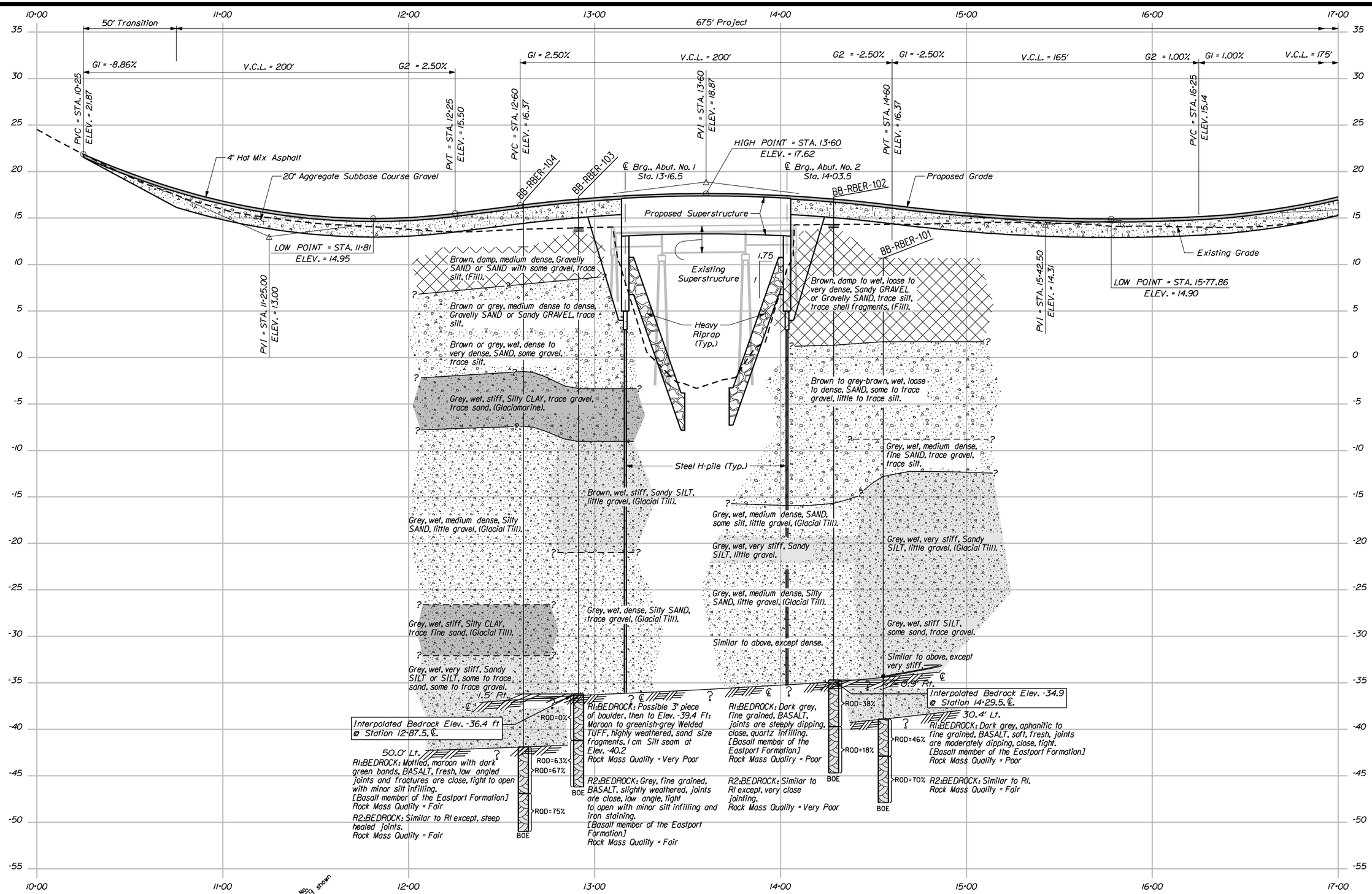
BORING LOCATION PLAN

SHEET NUMBER

2

OF 4

Filename: ...\00\geotech\msta\006_BLP1.dgn Division: GEOTECH Username: terry.white Date:5/29/2015



Note: 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.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1788(100)X

PIN 17881.00

BRIDGE NO. 3964

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	M.WIGHT		
DESIGN-DETAILED			
CHECKED-REVIEWED			
DESIGN2-DETAILED2	B.SLAVEN	T.WHITE	MAY 2015
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

ENGLISHMAN RIVER BRIDGE
ENGLISHMAN RIVER
ROQUE BLUFFS WASHINGTON COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

3

OF 4

Filename: ...\00\geotech\msta\007_ISP1.dgn Division: GEOTECH Username: terry.white Date:5/29/2015

Filename: ...\msta\008_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:5/29/2015

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Englishman River Bridge #3964 carries Johnson Cove Road over	Location: Roque Bluffs, Maine			Boring No.:	BB-RBER-104
				WIN:	17881.00
Driller:	Northern Test Boring	Elevation (ft.)	11.9	Auger ID/OD:	Hollow Stem 2 57/64 - 6 57/64
Operator:	Mike/Derreck	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	Diedrich D50 Track	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	9/13/11-9/14/11	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	12+61.7, 50.0 ft Lt.	Casing ID/OD:	HW	Water Level*:	8.5 ft bgs.
Hammer Efficiency Factor: 0.783		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

Sample Information

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.)
0	S1		0.00 - 4.00				HSA	11.50
								7.90
5	1D	24/10	5.00 - 7.00	14/7/6/5	13	17		
10	2D	24/16	10.00 - 12.00	14/11/16/18	27	35		
								-1.60
15	3D	24/18	15.00 - 17.00	3/4/4/6	8	10	16	
							30	
							43	
							91	-6.60
							123	
20	4D	24/14	20.00 - 22.00	7/5/11/10	16	21	OPEN HOLE	
25	5D	24/19	25.00 - 27.00	6/7/15/11	22	29		
30	6D	24/20	30.00 - 32.00	8/6/8/6	14	18		
35	MD	24/0	35.00 - 37.00	6/7/8/11	15	20		
								-26.10
40	7D	24/24	40.00 - 42.00	4/4/4/7	8	10		
								-32.10
45	8D	24/24	45.00 - 47.00	6/6/6/8	12	16		
50	9D	24/13	50.00 - 52.00	8/10/12/20	22	29		
	R1	60/60	53.80 - 58.80	RQD = 67%			NQ-2	-41.90
	R2	49.2/47	58.80 - 62.90	RQD = 75%				
								-51.00

Depth (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0.40	Cobbles from 0.0-0.4 ft bgs. Sample S1 from auger flights.	
4.00	Brown, damp, fine to coarse SAND, some gravel, trace silt, (Fill).	
	Brown, moist, medium dense, Gravelly SAND, trace silt.	G#261833 A-1-a, SP WC=2.1%
13.50	Brown, moist, dense, Sandy GRAVEL, trace silt.	G#261834 A-1-a, GW-GM WC=9.7%
18.50	Grey, wet, stiff, Silty CLAY, trace fine sand, (Glaciomarine). Dropped in HW Casing at 15.0 ft bgs.	G#261835 A-6, CL WC=22.9% LL=31 PL=14 PI=17
	Grey, wet, medium dense, Silty fine to coarse SAND, little gravel, (Glacial Till). Roller Coned ahead to 25.0 ft bgs.	
	Similar to above. Roller Coned ahead to 30.0 ft bgs.	G#261836 A-4, SM WC=8.8%
	Similar to above. Roller Coned ahead to 35.0 ft bgs.	
38.00	Failed sample attempt, similar to above in wash water. Roller Coned ahead to 40.0 ft bgs.	
44.00	Grey, wet, stiff, Silty CLAY, trace fine sand, (Glacial Till).	G#261837 A-6, CL WC=27.2% LL=35 PL=18 PI=17
	Grey, wet, very stiff, Sandy SILT, trace gravel, (Glacial Till).	
53.80	Grey, wet, very stiff, SILT, some sand, some gravel.	G#261838 A-4, SM WC=15.2%
62.90	Top of Bedrock at Elev. -41.9 ft. R1:Bedrock: Mottled maroon with dark green bands, BASALT, fresh, low angle joints and fractures are close, tight to open with minor silt infilling. Basalt member of the Eastport Formation. Rock Mass Quality = Fair R1:Core Times (min:sec) 53.8-54.8 ft (4:00) 54.8-55.8 ft (3:00) 55.8-56.8 ft (2:45) 2" soft layer of rock 56.8-57.8 ft (3:30) 57.8-58.8 ft (4:30) 100% Recovery R2:Bedrock: Similar to above except steep healed joints. Rock Mass Quality = Fair R2:Core Times (min:sec) 58.8-59.8 ft (4:30) 59.8-60.8 ft (4:00) 60.8-61.8 ft (4:15) 61.8-62.8 ft (4:30) 62.8-62.9 ft (2:00) 96% Recovery Core Blocked	
	Bottom of Exploration at 62.90 feet below ground surface.	

Remarks:

Auto Hammer #185

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: Englishman River Bridge #3964 carries Johnson Cove Road over	Location: Roque Bluffs, Maine			Boring No.:	BB-RBER-103
				WIN:	17881.00
Driller:	MaineDOT	Elevation (ft.)	14.0	Auger ID/OD:	5" Solid Stem
Operator:	Giguere/Giles/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	7/26/11; 07:30-16:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	12+91.4, 1.5 ft Rt.	Casing ID/OD:	HW & NW	Water Level*:	None Observed
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

Sample Information

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.)
0							SSA	13.58
	1D	20.4/13	1.00 - 2.70	5/7/4/40(2.4")	11	15		
5	2D	24/16	4.50 - 6.50	18/23/36/18	59	83	SPUN	
								5.50
10	3D	21.6/9	9.00 - 10.80	6/11/11/50(3.6")	22	31		
15	4D	24/16	14.50 - 16.50	6/16/23/45	39	55		
								-3.00
20	5D	24/14	19.00 - 21.00	2/4/4/16	8	11		
								-8.50
25	6D	24/20	24.00 - 26.00	4/5/5/7	10	14		
30	7D	24/20	29.00 - 31.00	3/4/6/7	10	14		
35	8D	24/2	34.00 - 36.00	5/5/5/6	10	14		
40	9D	24/18	39.00 - 41.00	9/12/12/15	24	34		
45	10D	24/14	44.00 - 46.00	10/15/8/13	23	32		
50	11D	14.4/12	49.00 - 50.20	9/16/50(2.4")	---			
	R1	60/55	50.20 - 55.20	RQD = 0%			NQ-2	-36.20
55	R2	60/60	55.20 - 60.20	RQD = 63%				
60								-46.20

Depth (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0.42	5" PAVEMENT.	
	Brown, damp, medium dense, Gravelly SAND, (Fill).	
8.50	Spun Casing ahead to 50.2 ft bgs. Grey-brown, wet, very dense, Gravelly SAND, trace silt.	
	Grey, wet, dense, Gravelly SAND, trace silt.	
17.00	Grey, wet, very dense, SAND, some gravel, trace silt. Changed to NW Casing at 15.0 ft bgs.	G#261828 A-1-a, SP WC=12.7%
22.50	Grey, wet, stiff, Silty CLAY, trace gravel, trace sand, (Glaciomarine).	G#261829 A-6, CL WC=21.6% LL=33 PL=19 PI=14
	Brown, wet, stiff, Sandy SILT, little gravel, (Glacial Till).	G#261830 A-4, SM WC=13.3%
	Grey, wet, stiff, Sandy SILT, little gravel.	G#261831 A-4, SM WC=10.5%
	Similar to above.	
	Grey, wet, dense, Silty SAND, trace gravel, (Glacial Till).	G#261832 A-4, SM WC=11.5%
	Similar to above.	
50.20	Similar to above.	
60.20	Top of Bedrock at Elev. -36.2 ft. R1:Bedrock: Possible 3" boulder fragment at top, then to Elev -39.4 ft.: Maroon to greenish-grey, Welded TUFF, highly weathered, with one zone of sand-sized fragments. Lower 22" is greenish gray and mottled. 1 cm silt seam at Elev. -40.2 ft. Rock Mass Quality = Very Poor R1:Core Times (min:sec) 50.2-51.2 ft (3:00) 51.2-52.2 ft (3:00) 52.2-53.2 ft (3:30) 53.2-54.2 ft (4:10) 54.2-55.2 ft (5:05) 92% Recovery R2:Bedrock: Grey, fine grained, BASALT, slight weathering, joints are low angled, close, tight to open with minor silt infilling and iron staining, Basalt member of the Eastport Formation. Rock Mass Quality = Fair R2:Core Times (min:sec) 55.2-56.2 ft (3:40) 56.2-57.2 ft (4:00) 57.2-58.2 ft (3:50) 58.2-59.2 ft (5:35) 59.2-60.2 ft (4:20) 100% Recovery	
	Bottom of Exploration at 60.20 feet below ground surface.	

Remarks:

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: Englishman River Bridge #3964 carries Johnson Cove Road over	Location: Roque Bluffs, Maine			Boring No.:	BB-RBER-102
				WIN:	17881.00
Driller:	MaineDOT	Elevation (ft.)	14.3	Auger ID/OD:	5" Solid Stem
Operator:	Giguere/Giles/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	9/12/11-9/13/11	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	14+28.6, 0.9 ft Rt.	Casing ID/OD:	HW & NW	Water Level*:	None Observed
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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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/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

Sample Information

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.)
0							SSA	13.97
	1D	24/12	1.00 - 3.00	5/7/5/7	12	17		
5	2D	24/14	5.00 - 7.00	13/15/19/20	34	48	55	
							95	
							78	
							59	
							50	
10	3D	24/7	10.00 - 12.00	7/7/9/13	16	22	55	
							75	
							89	1.30
							139	
							125	
15	4D	24/12	15.00 - 17.00	11/15/11/11	26	36	145	
							127	
							71	
							61	
							49	
20	5D	24/9	20.00 - 22.00	3/1/3/6	4	6	31	
							37	
							50	
							63	
							41	
25	6D	24/5	25.00 - 27.00	3/4/6/5	10	14	47	
							32	
							26	
							37	
							22	
30	7D	24/9	30.00 - 32.00	11/5/9/2	14	20	67	-16.20
							55	
							61	
							79	
							85	
35	8D	24/17	35.00 - 37.00	16/7/6/6	13	18	48	
							53	
							37	
							268	
							61	
40	9D	24/5	40.00 - 42.00	5/9/8/7	17	24	73	
							68	
							67	
							55	
							61	
45	10D	24/12	45.00 - 47.00	9/13/9/10	22	31	57	
							61	
							62	
							89	-34.70
	R1	60/60	49.00 - 54.00	RQD = 38%			NQ-2	
	R2	60/60	54.00 - 59.00	RQD = 18%				
								-44.70

Depth (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0.33	4" PAVEMENT.	
	Brown, damp, medium dense, Gravelly SAND, trace silt, (Fill).	G#261773 A-1-a, SW WC=1.7%
	Brown, wet, dense, Gravelly SAND, trace silt. Start of HW Casing at 5.0 ft bgs.	G#261774 A-1-a, SW-SM WC=7.9%
13.00	Similar to above, except medium dense, shell fragments (Reworked Native Soils, Fill).	
	Grey-brown, wet, dense, SAND, some fine gravel, trace silt.	
	Grey-brown, wet, loose, SAND, some fine gravel, little silt, shells.	G#261775 A-1-b, SW-SM WC=18.0%
30.50	Brown, wet, medium dense, fine to coarse SAND, some gravel, trace silt. Changed to NW Casing at 25.0 ft bgs.	
	Grey, wet, medium dense, SAND, some silt, little gravel, (Glacial Till).	G#261826 A-2-4, SM WC=19.5%
	Grey, wet, very stiff, Sandy SILT, little gravel.	G#261827 A-4, SM WC=11.2%
	Grey, wet, medium dense, Silty SAND, little gravel.	
49.00	Similar to above. Except dense.	
59.00	Top of Bedrock at Elev. -34.7 ft. R1:Bedrock: Dark Grey, fine grained, BASALT, joints are steeply dipping, close, with Quartz infilling. Basalt member of the Eastport Formation. Rock Mass Quality = Poor R1:Core Times (min:sec) 49.0-50.0 ft (3:30) 50.0-51.0 ft (3:35) 51.0-52.0 ft (3:00) 52.0-53.0 ft (3:50) 53.0-54.0 ft (3:50) 100% Recovery R2:Bedrock: Similar to above except very close jointing. Rock Mass Quality = Very Poor R2:Core Times (min:sec) 54.0-55.0 ft (3:00) 55.0-56.0 ft (3:45) 56.0-57.0 ft (3:40) 57.0-58.0 ft (5:10) 58.0-59.0 ft (5:30) 100% Recovery	
	Bottom of Exploration at 59.00 feet below ground surface.	

Remarks:

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: Englishman River Bridge #3964 carries Johnson Cove Road over	Location: Roque Bluffs, Maine			Boring No.:	BB-RBER-101
				WIN:	17881.00
Driller:	Northern Test Boring	Elevation (ft.)	10.7	Auger ID/OD:	Hollow Stem 2 57/64 - 6 57/64
Operator:	Mike/Derreck	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	Diedrich D50 Track	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	9/12/11-9/13/11	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	14+55.3, 30.4 ft Lt.	Casing ID/OD:	HW	Water Level*:	8.0 ft bgs.
Hammer Efficiency Factor: 0.783		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

Sample Information

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.)
0	1D	24/5	0.00 - 2.00	1/2/2/2	4	5	HSA	10.50
								8.70
5	2D	24/16	5.00 - 7.00	15/18/24/23	42	55		
								1.70
10	3D	24/13	10.00 - 12.00	8/11/12/17	23	30		
15	4D	24/15	15.00 - 17.00	5/7/8/13	15	20		
								-8.80
20	5D	24/17	20.00 - 22.00	5/7/7/6	14	18		
							62	
							70	
							72	-12.80
							80	
25	6D	24/18	25.00 - 27.00	7/7/8/8	15	20	OPEN HOLE	
30	7D	24/20	30.00 - 32.00	8/10/11/13	21	27		
35	8D	24/14	35.00 - 37.00	7/8/8/9	16	21		
40	9D	24/24	40.00 - 42.00	4/4/5/5	9	12		
45	10D	24/15	45.00 - 47.00	5/6/10/17	16	21		
	R1	48/48	49.60 - 53.60	RQD = 46%			NQ-2	-38.90
	R2	60/60	53.60 - 58.60	RQD = 70%				
								-47.90

Depth (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0.20	Topsoil, (Sod).	
2.00	Brown, damp, loose, fine to coarse SAND, trace silt, trace gravel.	
9.00	Brown, damp, very dense, Sandy GRAVEL, trace silt, (Fill).	G#261768 A-1-a, GW WC=3.5%
	Brown, wet, medium dense, Gravelly SAND, trace silt.	G#261772 A-1-a, SW WC=12.65%
19.50	Similar to above.	
23.50	Grey, wet, medium dense, fine SAND, trace gravel, trace silt. Dropped in HW Casing at 21.0 ft bgs.	G#261769 A-3, SP-SM WC=24.8%
	Grey, wet, very stiff, Sandy SILT, little gravel, (Glacial Till). Roller Coned ahead to 30.0 ft bgs.	G#261770 A-4, SC-SM WC=10.4%
	Similar to above. Roller Coned ahead to 35.0 ft bgs.	
	Similar to above. Roller Coned ahead to 40.0 ft bgs.	
	Grey, wet, stiff, SILT, some sand, trace gravel. Roller Coned ahead to 45.0 ft bgs.	G#261771 A-4, CL-ML WC=17.5%
49.60	Similar to above, except very stiff. Roller Coned ahead to 49.6 ft bgs.	
58.60	Top of Bedrock at Elev. -38.9 ft. R1:Bedrock: Dark Grey, aphanitic to fine grained, BASALT, soft, fresh, joints are moderately dipping, close, tight. Basalt member of the Eastport Formation. Rock Mass Quality = Poor R1:Core Times (min:sec) 49.6-50.6 ft (6:00) 50.6-51.6 ft (8:00) 51.6-52.6 ft (6:00) 52.6-53.6 ft (4:00) 100% Recovery R2:Bedrock: Similar to above. Rock Mass Quality = Fair R2:Core Times (min:sec) 53.6-54.6 ft (6:30) 54.6-55.6 ft (5:00) 55.6-56.6 ft (5:30) 56.6-57.6 ft (5:00) 57.6-58.6 ft (5:30) 100% Recovery	
	Bottom of Exploration at 58.60 feet below ground surface.	

Remarks:

Auto Hammer #185

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.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1788(100)X

BRIDGE PLANS

PIN 17881.00

BRIDGE NO. 3964

	BY	DATE
PROJ. MANAGER	M.WIGHT	
DESIGN-DETAILED		
CHECKED-REVIEWED	T.WHITE	MAY 2015
DESIGN2-DETAILED2	B.SLAVEN	
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

ENGLISHMAN RIVER BRIDGE
ENGLISHMAN RIVER
ROQUE BLUFFS WASHINGTON COUNTY

BORING LOGS

SHEET NUMBER

4

OF 4

Appendix A

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 thumbnai
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	Blow Counts
Bridge Name / Town	Sample Recovery
Boring Number	Date
Sample Number	Personnel Initials
Sample Depth	

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Englishman River Bridge #3964 carries Johnson Cove Road over Englishman River
Location: Roque Bluffs, Maine

Boring No.: BB-RBER-101
PIN: 17881.00

Driller:	Northern Test Boring	**Elevation (ft.)**	10.7	**Auger ID/OD:**	Hollow Stem 2¼-6¼"
Operator:	Mike/Derreck	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/12/11-9/13/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+55.3, 30.4 ft Lt.	**Casing ID/OD:**	HW	**Water Level*:**	8.0 ft bgs.
Hammer Efficiency Factor:	0.783	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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MD = Unsuccessful Split Spoon Sample attempt
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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
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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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/5	0.00 - 2.00	1/2/2/2	4	5	HSA	10.50	Topsoil, (Sod). 0.20	
									Brown, damp, loose, fine to coarse SAND, trace silt, trace gravel.	
								8.70	2.00	
5	2D	24/16	5.00 - 7.00	15/18/24/23	42	55			Brown, damp, very dense, Sandy GRAVEL, trace silt, (Fill).	G#261768 A-1-a, GW WC=3.5%
								1.70	9.00	
10	3D	24/13	10.00 - 12.00	8/11/12/17	23	30			Brown, wet, medium dense, Gravelly SAND, trace silt.	G#261772 A-1-a, SW WC=12.65%
15	4D	24/15	15.00 - 17.00	5/7/8/13	15	20			Similar to above.	
								-8.80	19.50	
20	5D	24/17	20.00 - 22.00	5/7/7/6	14	18			Grey, wet, medium dense, fine SAND, trace gravel, trace silt.	G#261769 A-3, SP-SM WC=24.8%
							62		Dropped in HW Casing at 21.0 ft bgs.	
							70			
							72	-12.80	23.50	
25							80			

Remarks:

Auto Hammer #185

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Boring No.: BB-RBER-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Englishman River Bridge #3964 carries Johnson Cove Road over Englishman River
Location: Roque Bluffs, Maine

Boring No.: BB-RBER-101
PIN: 17881.00

Driller:	Northern Test Boring	**Elevation (ft.)**	10.7	**Auger ID/OD:**	Hollow Stem 2¼-6¼"
Operator:	Mike/Derreck	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/12/11-9/13/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+55.3, 30.4 ft Lt.	**Casing ID/OD:**	HW	**Water Level*:**	8.0 ft bgs.
Hammer Efficiency Factor:	0.783	**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

Sample Information

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	24/18	25.00 - 27.00	7/7/8/8	15	20	OPEN HOLE			Grey, wet, very stiff, Sandy SILT, little gravel, (Glacial Till). Roller Coned ahead to 30.0 ft bgs.	G#261770 A-4, SC-SM WC=10.4%
30	7D	24/20	30.00 - 32.00	8/10/11/13	21	27				Similar to above. Roller Coned ahead to 35.0 ft bgs.	
35	8D	24/14	35.00 - 37.00	7/8/8/9	16	21				Similar to above. Roller Coned ahead to 40.0 ft bgs.	
40	9D	24/24	40.00 - 42.00	4/4/5/5	9	12				Grey, wet, stiff, SILT, some sand, trace gravel, Roller Coned ahead to 45.0 ft bgs.	G#261771 A-4, CL-ML WC=17.5%
45	10D	24/15	45.00 - 47.00	5/6/10/17	16	21				Similar to above, except very stiff. Roller Coned ahead to 49.6 ft bgs.	
50	R1	48/48	49.60 - 53.60	RQD = 46%			NQ-2	-38.90		49.60	

Remarks:

Auto Hammer #185

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Englishman River Bridge #3964 carries Johnson Cove Road over Englishman River
Location: Roque Bluffs, Maine

Boring No.: BB-RBER-101
PIN: 17881.00

Driller:	Northern Test Boring	**Elevation (ft.)**	10.7	**Auger ID/OD:**	Hollow Stem 2¼-6¼"
Operator:	Mike/Derreck	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/12/11-9/13/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+55.3, 30.4 ft Lt.	**Casing ID/OD:**	HW	**Water Level*:**	8.0 ft bgs.
Hammer Efficiency Factor:	0.783	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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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/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.
50										Top of Bedrock at Elev. -38.9 ft.	
										R1:Bedrock: Dark grey, aphanitic to fine grained, BASALT, soft, fresh, joints are moderately dipping, close, tight. Basalt member of the Eastport Formation.	
										Rock Mass Quality = Poor	
										R1:Core Times (min:sec)	
	R2	60/60	53.60 - 58.60	RQD = 70%						49.6-50.6 ft (6:00)	
										50.6-51.6 ft (8:00)	
										51.6-52.6 ft (6:00)	
										52.6-53.6 ft (4:00) 100% Recovery	
55										R2:Bedrock: Similar to above.	
										Rock Mass Quality = Fair	
										R2:Core Times (min:sec)	
										53.6-54.6 ft (6:30)	
										54.6-55.6 ft (5:00)	
										55.6-56.6 ft (5:30)	
										56.6-57.6 ft (5:00)	
								-47.90		57.6-58.6 ft (5:30) 100% Recovery	
										58.60	
60										**Bottom of Exploration at 58.60 feet below ground surface.**	
65											
70											
75											

Remarks:

Auto Hammer #185

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Englishman River Bridge #3964 carries Johnson Cove Road over Englishman River
Location: Roque Bluffs, Maine

Boring No.: BB-RBER-102
PIN: 17881.00

Driller:	MaineDOT	**Elevation (ft.)**	14.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/12/11-9/13/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+28.6, 0.9 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	13.97	4" PAVEMENT. 0.33	
	1D	24/12	1.00 - 3.00	5/7/5/7	12	17			Brown, damp, medium dense, Gravelly SAND, trace silt, (Fill).	G#261773 A-1-a, SW WC=1.7%
5	2D	24/14	5.00 - 7.00	13/15/19/20	34	48	55		Brown, wet, dense, Gravelly SAND, trace silt. Start of HW Casing at 5.0 ft bgs.	G#261774 A-1-a, SW-SM WC=7.9%
							95			
							78			
							59			
							50			
10	3D	24/7	10.00 - 12.00	7/7/9/13	16	22	55		Similar to above, except medium dense, shell fragments (Reworked Native Soils, Fill).	
							75			
							89			
							139	1.30	13.00	
							125			
15	4D	24/12	15.00 - 17.00	11/15/11/11	26	36	145		Grey-brown, wet, dense, SAND, some fine gravel, trace silt.	
							127			
							71			
							61			
							49			
20	5D	24/9	20.00 - 22.00	3/1/3/6	4	6	31		Grey-brown, wet, loose, SAND, some fine gravel, little silt, shells.	G#261775 A-1-b, SW-SM WC=18.0%
							37			
							50			
							63			
25							41			

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Englishman River Bridge #3964 carries Johnson Cove Road over Englishman River
Location: Roque Bluffs, Maine

Boring No.: BB-RBER-102
PIN: 17881.00

Driller:	MaineDOT	**Elevation (ft.)**	14.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/12/11-9/13/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+28.6, 0.9 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/5	25.00 - 27.00	3/4/6/5	10	14	47			Brown, wet, medium dense, fine to coarse SAND, some gravel, trace silt. Changed to NW Casing at 25.0 ft bgs.	
							32				
							26				
							37				
							22				
30	7D	24/9	30.00 - 32.00	11/5/9/2	14	20	67	-16.20		30.50 Grey, wet, medium dense, SAND, some silt, little gravel, (Glacial Till).	G#261826 A-2-4, SM WC=19.5%
							55				
							61				
							79				
							85				
35	8D	24/17	35.00 - 37.00	16/7/6/6	13	18	48			Grey, wet, very stiff, Sandy SILT, little gravel.	G#261827 A-4, SM WC=11.2%
							53				
							37				
							268				
							61				
40	9D	24/5	40.00 - 42.00	5/9/8/7	17	24	73			Grey, wet, medium dense, Silty SAND, little gravel.	
							68				
							67				
							55				
							61				
45	10D	24/12	45.00 - 47.00	9/13/9/10	22	31	57			Similar to above. Except dense.	
							61				
							62				
							89				
50	R1	60/60	49.00 - 54.00	RQD = 38%			NQ-2	-34.70		49.00 Top of Bedrock at Elev. -34.7 ft.	

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Englishman River Bridge #3964 carries Johnson Cove Road over Englishman River
Location: Roque Bluffs, Maine

Boring No.: BB-RBER-102
PIN: 17881.00

Driller:	MaineDOT	**Elevation (ft.)**	14.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/12/11-9/13/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+28.6, 0.9 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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

Sample Information

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										R1:Bedrock: Dark grey, fine grained, BASALT, joints are steeply dipping, close, with Quartz infilling. Basalt member of the Eastport Formation. Rock Mass Quality = Poor R1:Core Times (min:sec) 49.0-50.0 ft (3:30) 50.0-51.0 ft (3:35) 51.0-52.0 ft (3:00) 52.0-53.0 ft (3:50) 53.0-54.0 ft (3:50) 100% Recovery	
	R2	60/60	54.00 - 59.00	RQD = 18%							
55										R2:Bedrock: Similar to above except very close jointing. Rock Mass Quality = Very Poor R2:Core Times (min:sec) 54.0-55.0 ft (3:00) 55.0-56.0 ft (3:45) 56.0-57.0 ft (3:40) 57.0-58.0 ft (5:10) 58.0-59.0 ft (5:30) 100% Recovery	
								-44.70		59.00 **Bottom of Exploration at 59.00 feet below ground surface.**	
60											
65											
70											
75											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Englishman River Bridge #3964 carries Johnson Cove Road over Englishman River
Location: Roque Bluffs, Maine

Boring No.: BB-RBER-103
PIN: 17881.00

Driller:	MaineDOT	**Elevation (ft.)**	14.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/26/11; 07:30-16:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+91.4, 1.5 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	13.58	5" PAVEMENT. 0.42	
	1D	20.4/13	1.00 - 2.70	5/7/4/40(2.4")	11	15			Brown, damp, medium dense, Gravelly SAND, (Fill).	
	2D	24/16	4.50 - 6.50	18/23/36/18	59	83	SPUN		Spun Casing ahead to 50.2 ft bgs. Grey-brown, wet, very dense, Gravelly SAND, trace silt.	
5								5.50	8.50	
	3D	21.6/9	9.00 - 10.80	6/11/11/50(3.6")	22	31			Grey, wet, dense, Gravelly SAND, trace silt.	
10										
	4D	24/16	14.50 - 16.50	6/16/23/45	39	55			Grey, wet, very dense, SAND, some gravel, trace silt. Changed to NW Casing at 15.0 ft bgs.	G#261828 A-1-a, SP WC=12.7%
15								-3.00	17.00	
	5D	24/14	19.00 - 21.00	2/4/4/16	8	11			Grey, wet, stiff, Silty CLAY, trace gravel, trace sand, (Glaciomarine).	G#261829 A-6, CL WC=21.6% LL=33 PL=19 PI=14
20								-8.50	22.50	
	6D	24/20	24.00 - 26.00	4/5/5/7	10	14			Brown, wet, stiff, Sandy SILT, little gravel, (Glacial Till).	G#261830 A-4, SM
25										

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Boring No.: BB-RBER-103

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Englishman River Bridge #3964 carries Johnson Cove Road over Englishman River
Location: Roque Bluffs, Maine

Boring No.: BB-RBER-103
PIN: 17881.00

Driller:	MaineDOT	**Elevation (ft.)**	14.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/26/11; 07:30-16:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+91.4, 1.5 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
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.
25											WC=13.3%
	7D	24/20	29.00 - 31.00	3/4/6/7	10	14				Grey, wet, stiff, Sandy SILT, little gravel.	G#261831 A-4, SM WC=10.5%
30											
	8D	24/2	34.00 - 36.00	5/5/5/6	10	14				Similar to above.	
35											
	9D	24/18	39.00 - 41.00	9/12/12/15	24	34				Grey, wet, dense, Silty SAND, trace gravel, (Glacial Till).	G#261832 A-4, SM WC=11.5%
40											
	10D	24/14	44.00 - 46.00	10/15/8/13	23	32				Similar to above.	
45											
	11D	14.4/12	49.00 - 50.20	9/16/50(2.4")	---					Similar to above.	
50											

Remarks:

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: Englishman River Bridge #3964 carries Johnson Cove Road over Englishman River
Location: Roque Bluffs, Maine

Boring No.: BB-RBER-103
PIN: 17881.00

Driller:	MaineDOT	**Elevation (ft.)**	14.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/26/11; 07:30-16:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+91.4, 1.5 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
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

Sample Information

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	R1	60/55	50.20 - 55.20	RQD = 0%			NQ-2	-36.20		50.20 Top of Bedrock at Elev. -36.2 ft. R1:Bedrock: Possible 3" boulder fragment at top, then to Elev -39.4 ft.: Maroon to greenish-grey, Welded TUFF, highly weathered, with one zone of sand-sized fragments. Lower 22" is greenish gray and mottled. 1 cm silt seam at Elev. -40.2 ft. Rock Mass Quality = Very Poor R1:Core Times (min:sec) 50.2-51.2 ft (3:00) 51.2-52.2 ft (3:00) 52.2-53.2 ft (3:30) 53.2-54.2 ft (4:10) 54.2-55.2 ft (5:05) 92% Recovery	
55	R2	60/60	55.20 - 60.20	RQD = 63%						R2:Bedrock: Grey, fine grained, BASALT, slight weathering, joints are low angled, close, tight to open with minor silt infilling and iron staining, Basalt member of the Eastport Formation. Rock Mass Quality = Fair R2:Core Times (min:sec) 55.2-56.2 ft (3:40) 56.2-57.2 ft (4:00) 57.2-58.2 ft (3:50) 58.2-59.2 ft (5:35) 59.2-60.2 ft (4:20) 100% Recovery	
60								-46.20		60.20 **Bottom of Exploration at 60.20 feet below ground surface.**	
65											
70											
75											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Englishman River Bridge #3964 carries Johnson Cove Road over Englishman River
Location: Roque Bluffs, Maine

Boring No.: BB-RBER-104
PIN: 17881.00

Driller:	Northern Test Boring	**Elevation (ft.)**	11.9	**Auger ID/OD:**	Hollow Stem 2¼-6¼"
Operator:	Mike/Derreck	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/13/11-9/14/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+61.7, 50.0 ft Lt.	**Casing ID/OD:**	HW	**Water Level*:**	8.5 ft bgs.
Hammer Efficiency Factor:	0.783	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
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R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	S1		0.00 - 4.00				HSA	11.50	Cobbles from 0.0-0.4 ft bgs. Sample S1 from auger flights.	
									0.40 Brown, damp, fine to coarse SAND, some gravel, trace silt, (Fill).	
								7.90	4.00	
5	1D	24/10	5.00 - 7.00	14/7/6/5	13	17			Brown, moist, medium dense, Gravelly SAND, trace silt.	G#261833 A-1-a, SP WC=2.1%
10	2D	24/16	10.00 - 12.00	14/11/16/18	27	35			Brown, moist, dense, Sandy GRAVEL, trace silt.	G#261834 A-1-a, GW-GM WC=9.7%
								-1.60	13.50	
15	3D	24/18	15.00 - 17.00	3/4/4/6	8	10	16		Grey, wet, stiff, Silty CLAY, trace fine sand, (Glaciomarine). Dropped in HW Casing at 15.0 ft bgs.	G#261835 A-6, CL WC=22.9% LL=31 PL=14 PI=17
							30			
							43			
							91	-6.60	18.50	
							123			
20	4D	24/14	20.00 - 22.00	7/5/11/10	16	21	OPEN HOLE		Grey, wet, medium dense, Silty fine to coarse SAND, little gravel, (Glacial Till). Roller Coned ahead to 25.0 ft bgs.	
25										

Remarks:

Auto Hammer #185

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

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Boring No.: BB-RBER-104

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Englishman River Bridge #3964 carries Johnson Cove Road over Englishman River
Location: Roque Bluffs, Maine

Boring No.: BB-RBER-104
PIN: 17881.00

Driller:	Northern Test Boring	**Elevation (ft.)**	11.9	**Auger ID/OD:**	Hollow Stem 2¼-6¼"
Operator:	Mike/Derreck	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/13/11-9/14/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+61.7, 50.0 ft Lt.	**Casing ID/OD:**	HW	**Water Level*:**	8.5 ft bgs.
Hammer Efficiency Factor:	0.783	**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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/19	25.00 - 27.00	6/7/15/11	22	29			Similar to above. Roller Coned ahead to 30.0 ft bgs.	G#261836 A-4, SM WC=8.8%
30	6D	24/20	30.00 - 32.00	8/6/8/6	14	18			Similar to above. Roller Coned ahead to 35.0 ft bgs.	
35	MD	24/0	35.00 - 37.00	6/7/8/11	15	20			Failed sample attempt, similar to above in wash water. Roller Coned ahead to 40.0 ft bgs.	
								-26.10	38.00	
40	7D	24/24	40.00 - 42.00	4/4/4/7	8	10			Grey, wet, stiff, Silty CLAY, trace fine sand, (Glacial Till).	G#261837 A-6, CL WC=27.2% LL=35 PL=18 PI=17
								-32.10	44.00	
45	8D	24/24	45.00 - 47.00	6/6/6/8	12	16			Grey, wet, very stiff, Sandy SILT, trace gravel, (Glacial Till).	
50										

Remarks:

Auto Hammer #185

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Englishman River Bridge #3964 carries Johnson Cove Road over Englishman River
Location: Roque Bluffs, Maine

Boring No.: BB-RBER-104
PIN: 17881.00

Driller:	Northern Test Boring	**Elevation (ft.)**	11.9	**Auger ID/OD:**	Hollow Stem 2¼-6¼"
Operator:	Mike/Derreck	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/13/11-9/14/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+61.7, 50.0 ft Lt.	**Casing ID/OD:**	HW	**Water Level*:**	8.5 ft bgs.
Hammer Efficiency Factor:	0.783	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
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WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

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	9D	24/13	50.00 - 52.00	8/10/12/20	22	29				Grey, wet, very stiff, SILT, some sand, some gravel.	G#261838 A-4, SM WC=15.2%
	R1	60/60	53.80 - 58.80	RQD = 67%			NQ-2	-41.90		53.80	
55										Top of Bedrock at Elev. -41.9 ft. R1:Bedrock: Mottled maroon with dark green bands, BASALT, fresh, low angle joints and fractures are close, tight to open with minor silt infilling. Basalt member of the Eastport Formation. Rock Mass Quality = Fair R1:Core Times (min:sec) 53.8-54.8 ft (4:00) 54.8-55.8 ft (3:00) 55.8-56.8 ft (2:45) 2" soft layer of rock 56.8-57.8 ft (3:30) 57.8-58.8 ft (4:30) 100% Recovery	
	R2	49.2/47	58.80 - 62.90	RQD = 75%						R2:Bedrock: Similar to above except steep healed joints. Rock Mass Quality = Fair R2:Core Times (min:sec) 58.8-59.8 ft (4:30) 59.8-60.8 ft (4:00) 60.8-61.8 ft (4:15) 61.8-62.8 ft (4:30) 62.8-62.9 ft (2:00) 96% Recovery Core Blocked	
60								-51.00		62.90	
65										**Bottom of Exploration at 62.90 feet below ground surface.**	
70											
75											

Remarks:

Auto Hammer #185

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Appendix B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Roque Bluffs Work Number: 17881.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-RBER-101, 2D	14+55.3	30.4 Lt.	5.0-7.0	261768	1	3.5			GW	A-1-a	0
BB-RBER-101, 3D	14+55.3	30.4 Lt.	10.0-12.0	261772	1	12.6			SW	A-1-a	0
BB-RBER-101, 5D	14+55.3	30.4 Lt.	20.0-22.0	261769	1	24.8			SP-SM	A-3	0
BB-RBER-101, 6D	14+55.3	30.4 Lt.	25.0-27.0	261770	1	10.4			SC-SM	A-4	III
BB-RBER-101, 9D	14+55.3	30.4 Lt.	40.0-42.0	261771	1	17.5			CL-ML	A-4	IV
BB-RBER-102, 1D	14+28.6	0.9 Rt.	1.0-3.0	261773	2	1.7			SW	A-1-a	0
BB-RBER-102, 2D	14+28.6	0.9 Rt.	5.0-7.0	261774	2	7.9			SW-SM	A-1-a	0
BB-RBER-102, 5D	14+28.6	0.9 Rt.	20.0-22.0	261775	2	18.0			SW-SM	A-1-b	II
BB-RBER-102, 7D	14+28.6	0.9 Rt.	30.5-32.0	261826	2	19.5			SM	A-2-4	II
BB-RBER-102, 8D	14+28.6	0.9 Rt.	35.0-37.0	261827	2	11.2			SM	A-4	III
BB-RBER-103, 4D	12+91.4	1.5 Rt.	14.5-16.5	261828	3	12.7			SP	A-1-a	0
BB-RBER-103, 5D	12+91.4	1.5 Rt.	19.0-21.0	261829	3	21.6	33	14	CL	A-6	III
BB-RBER-103, 6D	12+91.4	1.5 Rt.	24.0-26.0	261830	3	13.3			SM	A-4	III
BB-RBER-103, 7D	12+91.4	1.5 Rt.	29.0-31.0	261831	3	10.5			SM	A-4	III
BB-RBER-103, 9D	12+91.4	1.5 Rt.	39.5-41.0	261832	3	11.5			SM	A-4	III
BB-RBER-104, 1D	12+61.7	50.0 Lt.	5.0-7.0	261833	4	2.1			SP	A-1-a	0
BB-RBER-104, 2D	12+61.7	50.0 Lt.	10.0-12.0	261834	4	9.7			GW-GM	A-1-a	0
BB-RBER-104, 3D	12+61.7	50.0 Lt.	15.0-17.0	261835	4	22.9	31	17	CL	A-6	III
BB-RBER-104, 5D	12+61.7	50.0 Lt.	25.0-27.0	261836	4	8.8			SM	A-4	III
BB-RBER-104, 7D	12+61.7	50.0 Lt.	40.0-42.0	261837	4	27.2	39	21	CL	A-6	III
BB-RBER-104, 9D	12+61.7	50.0 Lt.	50.0-52.0	261838	4	15.2			SM	A-4	III

Borings are not in station order.

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.

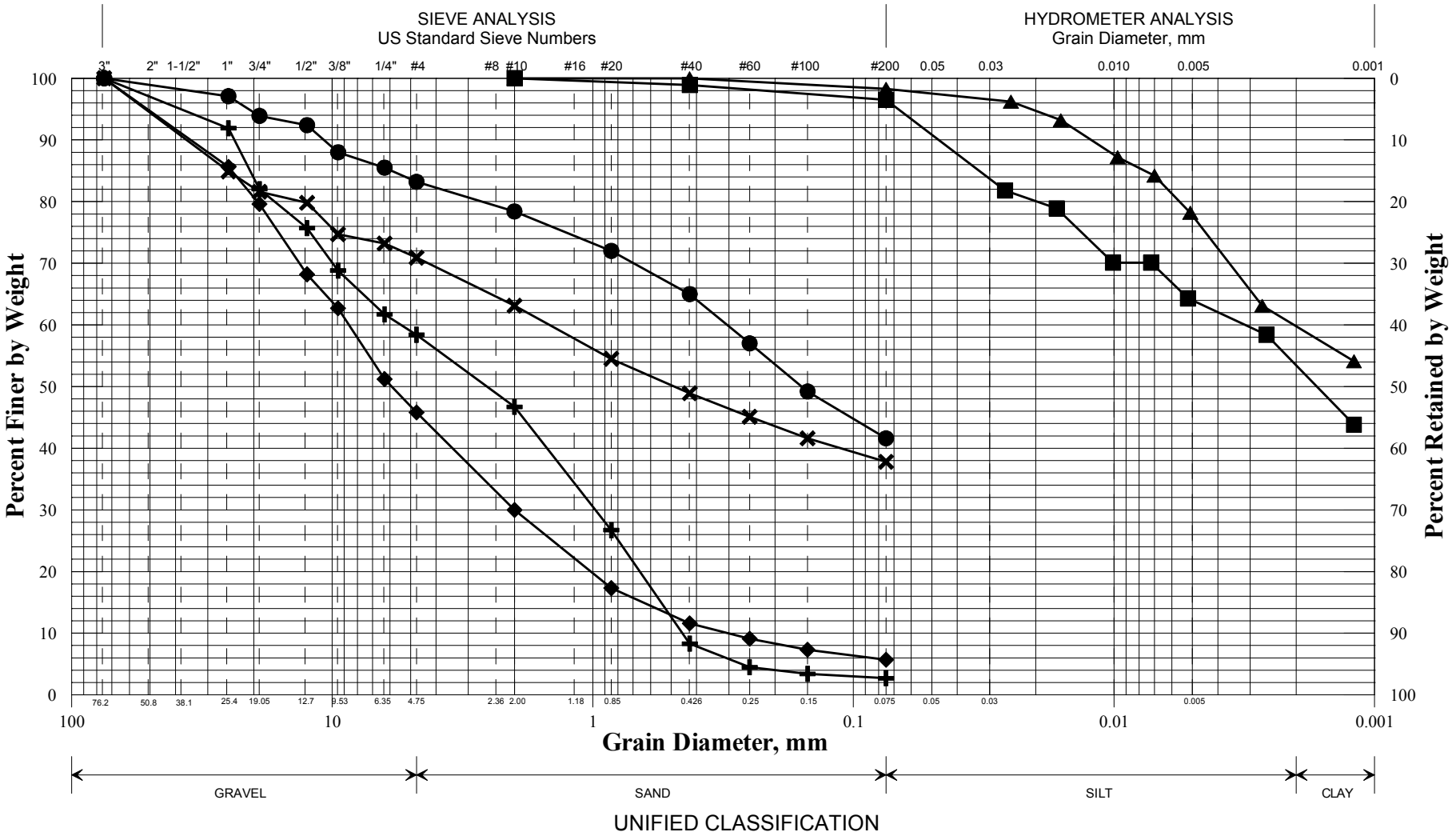
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

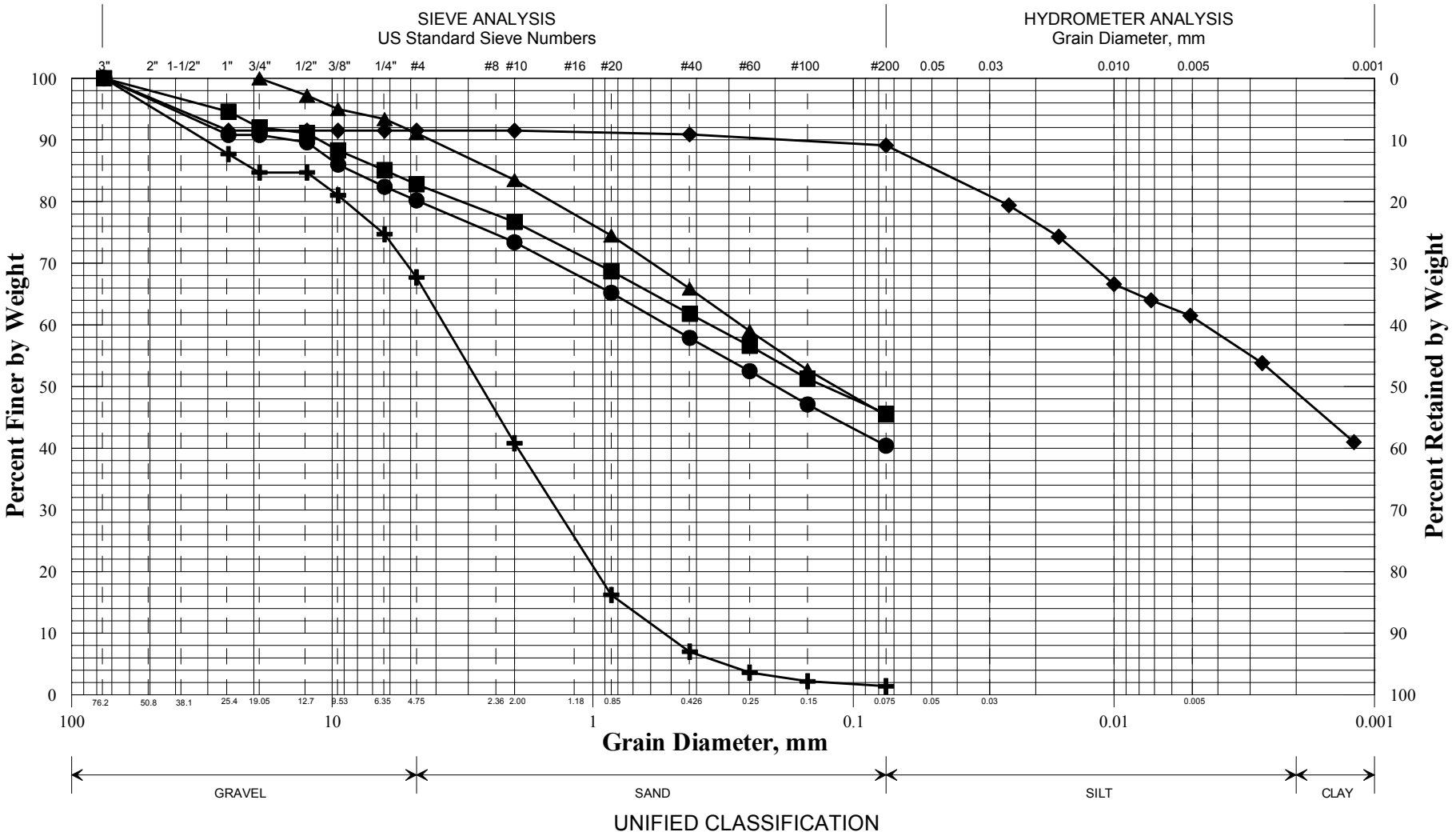


UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-RBER-104/1D	12+61.7	50.0 LT	5.0-7.0	Gravelly SAND, trace silt.	2.1			
◆	BB-RBER-104/2D	12+61.7	50.0 LT	10.0-12.0	Sandy GRAVEL, trace silt.	9.7			
■	BB-RBER-104/3D	12+61.7	50.0 LT	15.0-17.0	Silty CLAY, trace sand.	22.9	31	17	14
●	BB-RBER-104/5D	12+61.7	50.0 LT	25.0-27.0	Silty SAND, little gravel.	8.8			
▲	BB-RBER-104/7D	12+61.7	50.0 LT	40.0-42.0	Silty CLAY, trace sand.	27.2	39	18	21
✖	BB-RBER-104/9D	12+61.7	50.0 LT	50.0-52.0	SILT, some sand, some gravel.	15.2			

WIN
017881.00
Town
Roque Bluffs
Reported by/Date
WHITE, TERRY A 10/21/2011

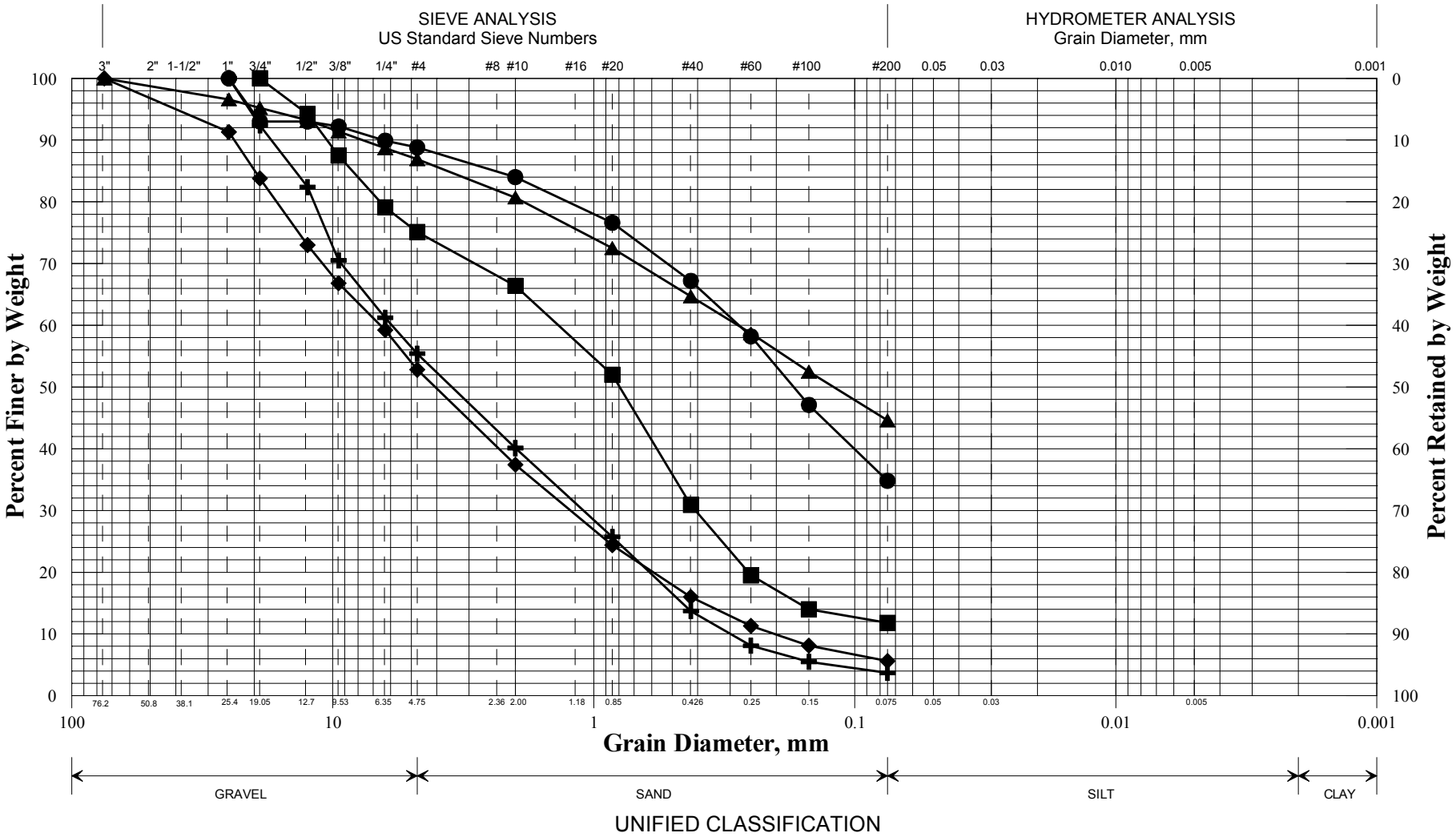
State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-RBER-103/4D	12+91.4	1.5 RT	14.5-16.5	SAND, some gravel, trace silt.	12.7			
◆	BB-RBER-103/5D	12+91.4	1.5 RT	19.0-21.0	Silty CLAY, trace gravel, trace sand.	21.6	33	19	14
■	BB-RBER-103/6D	12+91.4	1.5 RT	24.0-26.0	Sandy SILT, little gravel.	13.3			
●	BB-RBER-103/7D	12+91.4	1.5 RT	29.0-31.0	Sandy SILT, little gravel.	10.5			
▲	BB-RBER-103/9D	12+91.4	1.5 RT	39.5-41.0	Silty SAND, trace gravel.	11.5			
✖									

WIN
017881.00
Town
Roque Bluffs
Reported by/Date
WHITE, TERRY A 10/21/2011

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-RBER-102/1D	14+28.6	0.9 RT	1.0-3.0	Gravelly SAND, trace silt.	1.7			
◆	BB-RBER-102/2D	14+28.6	0.9 RT	5.0-7.0	Gravelly SAND, trace silt.	7.9			
■	BB-RBER-102/5D	14+28.6	0.9 RT	20.0-22.0	SAND, some gravel, little silt.	18.0			
●	BB-RBER-102/7D	14+28.6	0.9 RT	30.5-32.0	SAND, some silt, little gravel.	19.5			
▲	BB-RBER-102/8D	14+28.6	0.9 RT	35.0-37.0	Sandy SILT, little gravel.	11.2			
✖									

WIN
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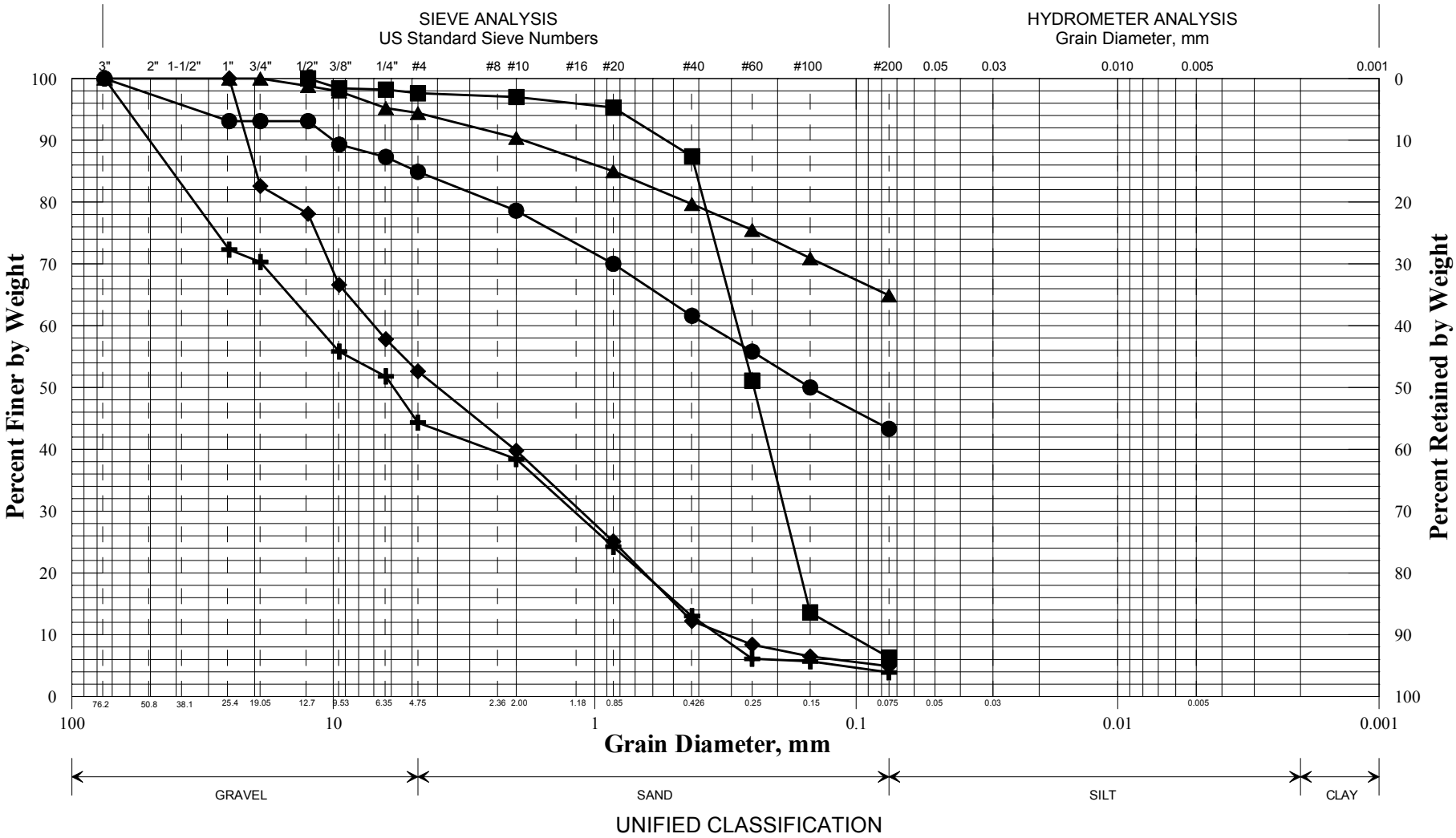
WHITE, TERRY A 10/19/2011

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-RBER-101/2D	14+55.3	30.4 LT	5.0-7.0	Sandy GRAVEL, trace silt.	3.5			
◆	BB-RBER-101/3D	14+55.3	30.4 LT	10.0-12.0	Gravelly SAND, trace silt.	12.6			
■	BB-RBER-101/5D	14+55.3	30.4 LT	20.0-22.0	SAND, trace silt, trace gravel.	24.8			
●	BB-RBER-101/6D	14+55.3	30.4 LT	25.0-27.0	Sandy SILT, little gravel.	10.4			
▲	BB-RBER-101/9D	14+55.3	30.4 LT	40.0-42.0	SILT, some sand, trace gravel.	17.5			
✖									

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SHEET 1

Appendix C

Calculations

Method 1 - MaineDOT Design Freezing Index (DFI) Map and Depth of Frost Penetration Table, BDG Section 5.2.1.

From Design Freezing Index Map: **Roque Bluffs, Maine**
DFI = 1150 degree-days.
Case 1 - coarse grained granular fill soils W=10% (assumed).

For DFI = 1100 $d := 69.8in$

Depth of Frost Penetration $d = 70 \cdot in$ $d = 5.8 \cdot ft$

Method 2 - ModBerg Software

Examine foundations placed on coarse grained fill soils

Eastport lies along the same Maine Design Freezing Index contour - use Eastport data from Modberg's freezing index database.

--- ModBerg Results ---

Project Location: Eastport, Maine
Air Design Freezing Index = 1102 F-days
N-Factor = 0.80
Surface Design Freezing Index = 882 F-days
Mean Annual Temperature = 43.5 deg F
Design Length of Freezing Season = 122 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	63.3	10.0	125.0	28	34	2.0	1.6	1,800

t = Layer thickness, in inches.
w% = Moisture content, in percentage of dry density.
d = Dry density, in lbs/cubic ft.
Cf = Heat Capacity of frozen phase, in BTU/(cubic ft degree F).
Cu = Heat Capacity of thawed phase, in BTU/(cubic ft degree F).
Kf = Thermal conductivity in frozen phase, in BTU/(ft hr degree).
Ku = Thermal conductivity in thawed phase, in BTU/(ft hr degree).
L = Latent heat of fusion, in BTU / cubic f

Total Depth of Frost Penetration = 5.27 ft = 63.3 in.

Recommendation: 5.5 feet for design of foundations constructed on coarse grained soils

Earth Pressure:

Backfill engineering strength parameters

Soil Type 4 Properties from MaineDOT Bridge Design Guide (BDG)

Unit weight $\gamma_1 := 125 \cdot pcf$

Internal friction angle $\phi_1 := 32 \cdot deg$

Cohesion $c_1 := 0 \cdot psf$

Integral Abutment and Wingwall - Passive Earth Pressure - Coulomb Theory

β = Angle of fill slope to the horizontal $\beta := 0 \cdot deg$

ϕ_1 = Angle of internal friction $\phi_1 := 32 \cdot deg$

θ = Angle of back face of wall to the horizontal $\theta := 90 \cdot deg$

For cases where interface friction is considered (this is for gravity shaped structures), use Coulomb.

For precast IAB abutment against clean sand, silty sand-gravel mixture use δ = 17 - 22, per LRFD Table 3.11.5.3-1 - because of the interface of the integral abutment backface and backfill soil

δ = friction angle between fill and wall taken as specified in LRFD Table 3.11.5.3-1 (degrees)

$\delta := 19.5 \cdot deg$

$$K_{p\_coul} := \frac{\sin(\theta - \phi_1)^2}{\sin(\theta)^2 \cdot \sin(\theta + \delta) \cdot \left(1 - \sqrt{\frac{\sin(\phi_1 + \delta) \cdot \sin(\phi_1 + \beta)}{\sin(\theta + \delta) \cdot \sin(\theta + \beta)}}\right)^2}$$

$K_{p\_coul} = 6.73$

Integral Abutment and Wingwall - Passive Earth Pressure - Rankine Theory

Bowles does not recommend use of Rankine method for Kp when B>0.

β = Angle of fill slope to the horizontal $\beta := 0 \cdot deg$

$$K_{p\_rank} := \frac{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}}{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}}$$

$K_{p\_rank} = 3.25$

Pp is oriented at an angle of β to the vertical plane

Design of H-piles

Reference: AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014
Vtrans Integral Abutment Bridge Design Guidelines, 2nd Edition, 2008

Generalized Bedrock Properties

BB-RBER-102, R1 RQD= 38%, R2 RQD= 18%
Rock Type: Basalt (Andesite; igneous), hard, fresh to slightly weathered.

BB-RBER-103, R1 RQD= 0%, R2 RQD= 63%
Rock Type: Welded Tuff, highly weathered (R1) and Basalt, slightly weathered (R2).

ϕ = 34-40 (AASHTO LRFD Table C.10.4.6.4-1);
C_o = 14,000 - 26,000 psi (AASHTO Standard Specifications for Bridges 17th Edition, Table 4.4.8.1.2B)

For Design Purposes, use bedrock data from BB-RBER-102: RQD = 38% and an Unconfined Compressive Strength of 15,000 psi.

Pile Properties

Use the following piles:

12x53
12x74
14x73 Note: all matrices set up in this order
14x89
14x117

$$A_s := \begin{pmatrix} 15.5 \\ 21.8 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot in^2 \qquad d := \begin{pmatrix} 11.78 \\ 12.13 \\ 13.6 \\ 13.83 \\ 14.21 \end{pmatrix} \cdot in \qquad b := \begin{pmatrix} 12.045 \\ 12.215 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot in$$

$$A_{box} := \overrightarrow{(d \cdot b)} \qquad A_{box} = \begin{pmatrix} 141.89 \\ 148.168 \\ 198.356 \\ 203.232 \\ 211.516 \end{pmatrix} \cdot in^2$$

Pile yield strength $F_y := 50 \cdot ksi$

1. Nominal and Factored Structural Compressive Resistance of H-piles

Use LRFD Equation 6.9.2.1-1 $P_r = \phi P_n$

Nominal Axial Structural Resistance

Determine equivalent yield resistance

$Q := 1.0$ LRFD Article 6.9.4.2

$P_o := Q \cdot F_y \cdot A_s$ LRFD Article 6.9.4.1.1

$$P_o = \begin{pmatrix} 775 \\ 1090 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot kip$$

Slender element reduction factor, Q, should be reduced for 12x53 and 14x73 H-pile sections per LRFD 6.9.4.2.

Assume a 1 foot unbraced section of pile due to settlement exposure or scour, L=1. Assume one end fixed and one end subject to translation, rotation fixed K=1.2.

A. Structural Resistance of unbraced segment of pile

Determine elastic critical buckling resistance P_e, LRFD eq. 6.9.4.1.2-1

E = Elastic Modulus $E := 29000 \cdot ksi$

K = effective length factor $K_{eff} := 1.2$ LRFD Table C4.6.2.5-1

l = unbraced length $l_{unbraced} := 1.0 \cdot ft$

r_s = radius of gyration

$$r_s := \begin{pmatrix} 2.86 \\ 2.92 \\ 3.49 \\ 3.53 \\ 3.59 \end{pmatrix} \cdot in$$

radius of gyration about the Y-Y or weak axis per LRFD Article C6.9.4.1.2.

LRFD eq. 6.9.4.1.2-1

$$P_e := \overrightarrow{\left[\frac{\pi^2 \cdot E}{\left(\frac{K_{eff} \cdot l_{unbraced}}{r_s} \right)^2} \cdot A_s \right]}$$

$$P_e = \begin{pmatrix} 174999 \\ 256564 \\ 359780 \\ 448914 \\ 611956 \end{pmatrix} \cdot kip$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 225.806 \\ 235.379 \\ 336.243 \\ 343.995 \\ 355.788 \end{pmatrix}$$

If Pe/Po > or = 0.44, then:

$$P_n := \overrightarrow{\left(0.658^{\frac{P_o}{P_e}} \cdot P_o\right)}$$

LRFD Eq. 6.9.4.1.1-1

then:

$$P_n = \begin{pmatrix} 774 \\ 1088 \\ 1069 \\ 1303 \\ 1718 \end{pmatrix} \cdot kip$$

12x53
12x74
14x73
14x89
14x117

Factored Axial Structural Resistance at the Strength Limit State

Resistance factor for unbraced segements of H-pile in compression under good driving conditions per LRFD 6.5.4.2:

$$\phi_{cu} := 0.6$$

The Factored Structural Resistance (Pr) per LRFD 6.9.2.1-1 is

$$P_r := \phi_{cu} \cdot P_n$$

Factored structural compressive resistance, Pr

$$P_r = \begin{pmatrix} 464 \\ 653 \\ 641 \\ 782 \\ 1031 \end{pmatrix} \cdot kip$$

12x53
12x74
14x73
14x89
14x117

LRFD 10.7.3.2.3 - Piles Driven to Hard Rock -

Article 10.7.3.2.3 states "The nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. The nominal bearing resistance shall not exceed the values obtained from Article 6.9.4.1 with the resistance factors specified in Article 6.5.4.2 and Article 6.15 for severe driving conditions. A pile driving acceptance criteria shall be developed that will prevent pile damage."

Therefore limit the nominal axial geotechnical pile resistance to the nominal structural resistance with a resistance factor for severe driving conditions of 0.50 applied per 10.7.3.2.3.

Nominal Structural Resistance Previously Calculated (lower braced segment):

$$P_n = \begin{pmatrix} 774 \\ 1088 \\ 1069 \\ 1303 \\ 1718 \end{pmatrix} \cdot kip$$

12x53
12x74
14x73
14x89
14x117

The factored geotechnical compressive resistance (P_r) for the **Strength Limit State**, per LRFD 6.9.2.1-1 is

$$\phi_c := 0.5$$

$$P_r := \phi_c \cdot P_n$$

12x53
12x74
14x73
14x89
14x117

$$P_r = \begin{pmatrix} 387 \\ 544 \\ 534 \\ 652 \\ 859 \end{pmatrix} \cdot kip$$

The factored geotechnical compressive resistance (P_r) for the **Extreme and Service Limit States**, per LRFD 6.9.2.1-1 is

$$\phi_c := 1.0$$

$$P_{r\_ee} := \phi_c \cdot P_n$$

12x53
12x74
14x73
14x89
14x117

$$P_{r\_ee} = \begin{pmatrix} 774 \\ 1088 \\ 1069 \\ 1303 \\ 1718 \end{pmatrix} \cdot kip$$

Nominal and Factored Axial Geotechnical Resistance of HP piles

Geotechncial axial pile resistance for pile end bearing on rock is determined by CGS method (LRFD Table 10.5.5.2.3-1) and outlined in Canadian Foundation Engineering Manual, 4th Edition 2006, and FHWA LRFD Pile Foundation Design Example in FHWA-NHI-05-094.

Nominal unit bearing resistance of pile point, q_p

Design value of compressive strength of rock core $q_{u\_1} := 15000 \cdot psi$

Spacing of discontinuities $s_d := 12 \cdot in$

Width of discontinuities. Joints are open to tight per boring logs $t_d := \frac{1}{64} \cdot in$

Pile width is b - matrix $D := b$

Embedment depth of pile in socket - pile is end bearing on rock $H_s := 0 \cdot ft$

Diameter of socket: $D_s := 12 \cdot in$

Depth factor $dd := 1 + 0.4 \cdot \frac{H_s}{D_s}$ and dd < 3

$dd = 1$ OK

K_{sp}

$$K_{sp} := \frac{3 + \frac{s_d}{D}}{10 \cdot \left(1 + 300 \cdot \frac{t_d}{s_d}\right)^{0.5}}$$

$$K_{sp} = \begin{pmatrix} 0.339 \\ 0.338 \\ 0.324 \\ 0.324 \\ 0.323 \end{pmatrix}$$

K_{sp} has a factor of safety of 3.0 in the CGS method. Remove in calculation of pile tip resistance, below.

Geotechnical tip resistance.

$$q_{p\_1} := 3 \cdot q_{u\_1} \cdot K_{sp} \cdot dd \qquad q_{p\_1} = \begin{pmatrix} 2196 \\ 2188 \\ 2101 \\ 2097 \\ 2092 \end{pmatrix} \cdot ksf$$

Nominal geotechnical tip resistance, R_p - *Extreme Limit States and Service Limit States*

Case I

$$R_{p\_1} := \overrightarrow{\left(q_{p\_1} \cdot A_s\right)} \qquad R_{p\_1} = \begin{pmatrix} 236 \\ 331 \\ 312 \\ 380 \\ 500 \end{pmatrix} \cdot kip$$

$\phi = 1.0$

Factored Axial Geotechnical Compressive Resistance - *Strength Limit States*

Resistance factor, end bearing on rock Canadian Geotechnical Society method

$\phi_{stat} := 0.45$ LRFD Table 10.5.5.2.3-1

Factored Geotechnical Tip Resistance (R_r)

$$R_{r\_p1} := \phi_{stat} \cdot R_{p\_1} \qquad R_{r\_p1} = \begin{pmatrix} 106 \\ 149 \\ 140 \\ 171 \\ 225 \end{pmatrix} \cdot kip$$

CGS method is superceded by LRFD 10.7.3.2.3 - Piles Driven to hard rock.

Nominal and Factored Axial Geotechnical Resistance of HP piles

Geotechncial axial pile resistance for pile end bearing on rock determined by Intact Rock Method, proposed by Sandford, MaineDOT Transportation Research Division Technical Report 14-01, Phase 2 (January 2014), based on Rowe and Armitage (1987) equation cited by NCHRP Synthesis 360, Turner, (2006).

Nominal unit bearing resistance of pile point, q_p

Design value of compressive strength of rock core

$$q_{u\_1} := 15000 \cdot psi$$

Geotechnical tip resistance.

$$q_{p\_2} := 2.5 \cdot q_{u\_1}$$

$$q_{p\_1} = \begin{pmatrix} 2196 \\ 2188 \\ 2101 \\ 2097 \\ 2092 \end{pmatrix} \cdot ksf$$

Nominal geotechnical tip resistance, Rp - *Extreme Limit States and Service Limit States*

$$R_{p\_2} := \overrightarrow{\left(q_{p\_2} \cdot A_s\right)}$$

$$R_{p\_2} = \begin{pmatrix} 581 \\ 818 \\ 803 \\ 979 \\ 1290 \end{pmatrix} \cdot kip$$

$\phi = 1.0$

Factored Axial Geotechnical Compressive Resistance - *Strength Limit States*

Resistance factor, end bearing on rock Canadian Geotechnical Society method

$\phi_{stat} := 0.45$ LRFD Table 10.5.5.2.3-1

Factored Geotechnical Tip Resistance (R_r)

$$R_{r\_p2} := \phi_{stat} \cdot R_{p\_2}$$

$$R_{r\_p2} = \begin{pmatrix} 262 \\ 368 \\ 361 \\ 440 \\ 581 \end{pmatrix} \cdot kip$$

Drivability Analyses

Ref: LRFD Article 10.7.8

For steel piles in compression or tension, driving stresses are limited to 90% of f_y

$\phi_{da} := 1.0$ resistance factor from LRFD Table 10.5.5.2.3-1, Drivablity Analysis, steel piles

$\sigma_{dr} := 0.90 \cdot 50 \cdot (ksi) \cdot \phi_{da}$

$\sigma_{dr} = 45 \cdot ksi$ driving stress cannot exceed 45 ksi

Limit driving stress to 45 ksi or limit blow count to 5-15 blows per inch (bpi) per Section 501 (Note: 6-10 bpi is considered optimal for diesel hammers).

Compute the resistance that can be achieved in a drivablity analysis:

The resistance that must be achieved in a drivablity analysis will be the maximum factored pile load divided by the appropriate resistance factor for wave equation analysis and dynamic test which will be required for construction.

$\phi_{dyn} := 0.65$ Reference LRFD Table 10.5.5.2.3-1 - for Strength Limit State

$\phi := 1.0$ For Extreme and Service Limit States

GRLWeap Soil and Pile Model Assumptions

Based on Table 4 of this Report, estmated pile lengths at Abutment 1 will be approx. 40 feet.

Minimum length of pile embedment in soil is 40 ft.

Assume the Contractor drives pile lengths of 55 ft. (40' + 2' (abutment embedment with cut-off) + 5' (testing) + extra embedment into poor quality rock.

Use constant shaft resistance at a percentage of the ultimate capacity such that GRLWeap will assign 50 kip shaft resistance to all ultimate capacities analyzed.

Pile Size is 12 x 53

The 12x53 pile can be driven to the resistances below with a Delmag D 19-42 at a reasonable blow count and level of driving stress. See GRLWEAP results below:
Fuel Setting: 90% of Max

State of Maine Dept. Of Transportation
17881 Roque Bluffs 12x53 Delmag 19-42

19-May-2015
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
400.0	39.46	6.21	8.0	8.09	18.71
460.0	41.56	6.86	12.5	8.53	19.63
465.0	41.71	6.91	13.0	8.56	19.69
470.0	41.84	6.95	13.6	8.59	19.74
475.0	41.99	7.00	14.1	8.63	19.80
480.0	42.12	7.05	14.7	8.66	19.85
482.5	42.19	7.06	15.0	8.67	19.91
485.0	42.24	7.08	15.4	8.69	19.92
490.0	42.37	7.15	16.1	8.72	19.97
495.0	42.53	7.20	16.7	8.75	20.07

Limit blow counts to 15 bpi.
Increasing fuel setting overstresses pile at smaller capacity.

$R_{ndr} := 482.5 \cdot kip$

Strength Limit State

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

$R_{fdr} = 314 \cdot kip$

Extreme and Service Limit States

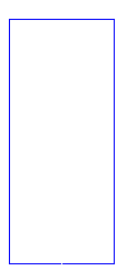
$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 483 \cdot kip$

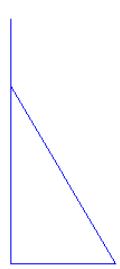
DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer C	109975	kips/in
Skin Quake	0.040	in
Toe Quake	0.100	in
Skin Dampin	0.050	sec/ft
Toe Dampin	0.150	sec/ft
Pile Length	55.00	ft
Pile Penetrat	40.00	ft
Pile Top Are	15.50	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 12 %
(Proportional)

Pile Size is 12 x 74

The 12 x 74 pile can be driven to the resistances below with a Delmag D 19-42 at a reasonable blow count and level of driving stress. See GRLWEAP results below:
Fuel Setting: Max

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17881 Roque Bluffs 12x74 Delmag 19-42

19-May-2015
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
500.0	40.19	4.06	9.6	9.25	21.30
540.0	41.40	3.89	12.0	9.46	21.81
560.0	41.56	3.84	13.8	9.46	21.76
565.0	42.02	3.82	13.8	9.57	22.08
570.0	41.83	3.83	14.4	9.50	21.94
576.0	41.95	3.81	15.0	9.52	21.93
580.0	42.01	3.80	15.3	9.54	21.98
585.0	42.13	3.80	15.7	9.56	22.06
590.0	42.22	3.82	16.3	9.58	22.08
600.0	42.41	3.88	17.3	9.62	22.18

DELMAG D 19-42

Limit to 15 bpi.

$R_{ndr} := 576 \cdot kip$

Efficiency 0.800

Helmet 2.70 kips
Hammer C 109975 kips/in

Skin Quake 0.040 in
Toe Quake 0.100 in
Skin Dampin 0.050 sec/ft
Toe Dampin 0.150 sec/ft

Pile Length 55.00 ft
Pile Penetrat 40.00 ft
Pile Top Are 21.80 in2

Strength Limit State

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

$R_{fdr} = 374 \cdot kip$

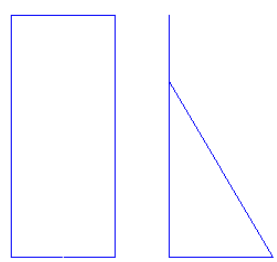
Extreme and Service Limit States

$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 576 \cdot kip$

Pile Model

Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size is 14 x 73

The 14 x 73 pile can be driven to the resistances below with a Delmag D 19-42 at a reasonable blow count and level of driving stress. See GRLWEAP results below.
Fuel Setting: Max

State of Maine Dept. Of Transportation
17881 Roque Bluffs 14x73 Delmag 19-42

19-May-2015
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
500.0	40.59	4.26	9.7	9.28	21.39
540.0	41.81	4.12	12.0	9.50	21.98
560.0	42.28	4.06	13.6	9.59	22.19
565.0	42.13	4.06	14.4	9.52	21.99
570.0	42.23	4.04	14.8	9.54	22.02
573.0	42.26	4.05	15.0	9.54	22.07
580.0	42.42	4.04	15.6	9.58	22.15
585.0	42.55	4.02	16.2	9.60	22.17
590.0	42.62	4.01	16.7	9.62	22.20
600.0	42.79	4.01	17.7	9.65	22.33

Limit to 15 bpi

$R_{ndr} := 573 \cdot kip$

Strength Limit State

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

$R_{fdr} = 372 \cdot kip$

Extreme and Service Limit States

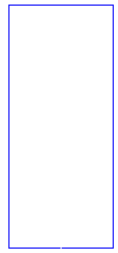
$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 573 \cdot kip$

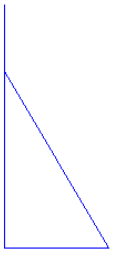
DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer C	109975	kips/in
Skin Quake	0.040	in
Toe Quake	0.100	in
Skin Dampin	0.050	sec/ft
Toe Dampin	0.150	sec/ft
Pile Length	55.00	ft
Pile Penetrat	40.00	ft
Pile Top Are	21.40	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size is 14 x 89

The 14 x 89 pile can be driven to the resistances below with a Delmag D 19-42 at a reasonable blow count and level of driving stress. See GRLWEAP results below:
Fuel Setting: Max

State of Maine Dept. Of Transportation
17881 Roque Bluffs 14x89 Delmag 19-42

19-May-2015
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
500.0	36.24	2.75	8.9	9.00	20.20
550.0	37.71	2.89	10.9	9.22	20.71
575.0	38.32	3.08	12.2	9.31	20.98
600.0	38.92	3.37	13.7	9.41	21.18
610.0	38.84	3.46	14.8	9.35	21.00
613.0	38.84	3.49	15.0	9.36	21.03
615.0	38.93	3.52	15.1	9.37	21.05
620.0	39.05	3.59	15.4	9.38	21.13
630.0	39.20	3.70	16.2	9.41	21.21
640.0	39.36	3.81	17.1	9.44	21.28

DELMAG D 19-42

Limit to 15 bpi.

$R_{ndr} := 613 \cdot kip$

Efficiency 0.800

Helmet 2.70 kips
Hammer C 109975 kips/in

Skin Quake 0.040 in
Toe Quake 0.100 in
Skin Dampin 0.050 sec/ft
Toe Dampin 0.150 sec/ft

Pile Length 55.00 ft
Pile Penetrat 40.00 ft
Pile Top Are 26.10 in2

Strength Limit State

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

$R_{fdr} = 398 \cdot kip$

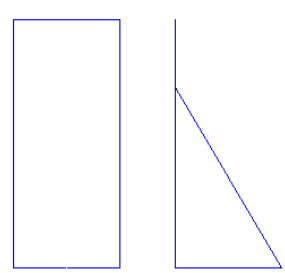
Extreme and Service Limit States

$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 613 \cdot kip$

Pile Model

Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size is 14 x 117

The 14 x 117 pile can be driven to the resistances below with a Delmag D 19-42 at a reasonable blow count and level of driving stress. See GRLWEAP results below:
Fuel Setting: Max

State of Maine Dept. Of Transportation
17881 Roque Bluffs 14x117 Delmag 19-42

20-May-2015
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
500.0	29.97	1.75	8.3	8.64	18.38
550.0	31.45	1.72	9.8	8.81	18.82
600.0	32.67	1.95	11.7	8.99	19.21
625.0	33.26	2.16	12.8	9.06	19.42
650.0	33.76	2.37	14.1	9.13	19.58
660.0	33.90	2.43	14.7	9.16	19.63
665.0	34.06	2.47	15.0	9.17	19.66
670.0	34.12	2.53	15.2	9.19	19.75
680.0	34.35	2.60	15.8	9.22	19.81
690.0	34.50	2.63	16.5	9.24	19.82

DELMAG D 19-42

Limit to 15 bpi.

$R_{ndr} := 665 \cdot kip$

Efficiency 0.800

Helmet 2.70 kips
Hammer C 109975 kips/in

Skin Quake 0.040 in
Toe Quake 0.100 in
Skin Dampin 0.050 sec/ft
Toe Dampin 0.150 sec/ft

Pile Length 55.00 ft
Pile Penetrat 40.00 ft
Pile Top Are 34.40 in2

Strength Limit State

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

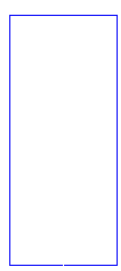
$R_{fdr} = 432 \cdot kip$

Extreme and Service Limit States

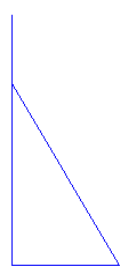
$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 665 \cdot kip$

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size is 14 x 117

The 14 x 117 pile can be driven to the resistances below with a Delmag D 36-32 at a reasonable blow count and level of driving stress. See GRLWEAP results below:
Fuel Setting: 90 % of Max

State of Maine Dept. Of Transportation
17881 Roque Bluffs 14x117 Delmag 36-32

20-May-2015
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
500.0	34.14	1.78	3.1	7.26	33.46
750.0	43.46	3.83	6.3	8.52	38.62
785.0	44.42	3.70	7.0	8.68	39.54
795.0	44.66	3.69	7.2	8.72	39.69
800.0	44.84	3.70	7.3	8.73	39.68
805.0	44.91	3.70	7.4	8.76	39.81
810.0	45.01	3.67	7.5	8.77	39.97
815.0	45.13	3.68	7.7	8.79	39.97
820.0	45.36	3.67	7.8	8.81	40.10
825.0	45.48	3.65	7.9	8.83	40.25

Limit stress to 45 ksi. Rounding up blow counts > 7 will exceed factored structural resistance of the pile

$R_{ndr} := 785 \cdot kip$

Strength Limit State

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

$R_{fdr} = 510 \cdot kip$

Extreme and Service Limit States

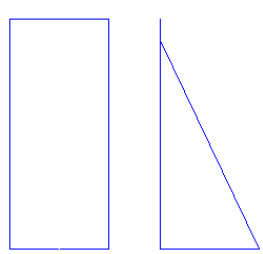
$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 785 \cdot kip$

DELMAG D 36-32

Efficiency	0.800	
Helmet	2.70	kips
Hammer C	109975	kips/in
Skin Quake	0.040	in
Toe Quake	0.100	in
Skin Dampin	0.050	sec/ft
Toe Dampin	0.150	sec/ft
Pile Length	55.00	ft
Pile Penetrat	50.00	ft
Pile Top Are	34.40	in2

Pile Model | Skin Friction Distribution



Res. Shaft = 10 %
(Constant Res. Shaft)

BB-101					BB-102					BB-103					BB-104				
Depth	**SPT N**		**di**	**di/N**	**Depth**	**SPT N**		**di**	**di/N**	**Depth**	**SPT N**		**di**	**di/N**	**Depth**	**SPT N**		**di**	**di/N**
1	5		2	2.5	1	17		2	0.12	1	15		2	0.13	5	17		9	0.53
5	55		7	0.13	5	48		4	0.08	5	83		7	0.08	10	35		5	0.14
10	30		6	0.20	10	22		7	0.32	10	31		6	0.19	15	10		5	0.50
15	20		5	0.25	15	36		7	0.19	15	55		5	0.09	20	21		5	0.24
20	18		5	0.28	20	6		5	0.83	20	11		5	0.45	25	29		5	0.17
25	20		5	0.25	25	14		5	0.36	25	14		5	0.36	30	18		5	0.28
30	27		5	0.19	30	20		5	0.25	30	14		5	0.36	35	20		5	0.25
35	21		5	0.24	35	18		5	0.28	35	14		5	0.36	40	10		5	0.50
40	12		5	0.42	40	24		5	0.21	40	34		5	0.15	45	16		5	0.31
45	21		5	0.24	45	31		5	0.16	45	32		5	0.16	50	29		4	0.14
50	100	**Bedrock**	50	0.50	50	50	**Bedrock**	50	1.00	50	100	**Bedrock**	50	0.50	54	100	**Bedrock**	47	0.47
SUM			**100**	**2.68**				**100**	**2.68**				**100**	**2.70**				**100**	**3.06**

di/di/N **37.27** **di/di/N** **37.26** **di/di/N** **37.06** **di/di/N** **32.67**

SUM	**Nav.**	**36.07**

Conclusion: Site Class D

Site Classification per LRFD Table C3.10.3.1-1 - Method B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 44.610100
Longitude = -067.476800
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)		
0.0	0.068	PGA	- Site Class B
0.2	0.139	Ss	- Site Class B
1.0	0.038	S1	- Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
Latitude = 44.610100
Longitude = -067.476800
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)		
0.0	0.109	As	- Site Class D
0.2	0.222	SDs	- Site Class D
1.0	0.092	SD1	- Site Class D