MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

KNICKERBOCKER BRIDGE
OVER BACK RIVER
BOOTHBAY, MAINE

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GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to make geotechnical recommendations for the replacement of Knickerbocker Bridge over the Back River and located on Barters Island Road in Boothbay, Maine. The proposed replacement bridge will be a 540-foot structure consisting of six (6) 90-foot spans. The replacement bridge will be built on a new alignment south of the existing bridge. There will be 1060 feet of approach work.

Based on preliminary geotechnical recommendations, the MaineDOT Bridge Program selected foundation types for this site consisting of cantilever-type abutments supported on bedrock and pile bent piers consisting of concrete filled pipe piles, some with bedrock-socketed internal H or W-sections for fixity. The following design recommendations for those foundation types are discussed in detail in the attached report:

Cantilever-type Abutments and Wingwalls – Cantilever-type abutments and in-line wingwalls shall be designed to resist all lateral earth loads, vehicular loads, superstructure loads, and any loads transferred through the superstructure. They shall be designed for all relevant strength and service limit states in accordance with AASHTO LRFD Bridge Design Specifications 4th Edition, 2007, (herein referred to as LRFD).

The design of project abutments founded on spread footings at the strength limit state shall consider nominal bearing resistance, overturning, lateral sliding, structural failure, and foundation resistance at the design flood. A sliding resistance factor, φ_τ, of 0.80 shall be applied to the nominal sliding resistance of abutments and wingwalls founded on spread footings on level bedrock. For footings on level bedrock, the eccentricity of loading at the strength limit state shall not exceed three-eighths (3/8^{ths}) of the footing dimensions. A resistance factor, φ, of 1.0 shall be used to assess spread footing design at the service limit state.

Earth loads shall be calculated using an active earth pressure coefficient, K_a, of 0.307, calculated using Rankine Theory for cantilever-type substructures. The Designer may assume Soil Type 4 for backfill material soil properties. The backfill properties are as follows: ϕ = 32°, γ = 125 pcf.

Pipe Pile Pier Bents - Concrete filled, pipe pile bent piers are recommended for intermediate structure support. The majority of the pipe piles will be socketed in bedrock with an H or W-section to achieve fixity. Pipe piles with diameters ranging from 24 to 30 inches and wall thicknesses of 1/2 to 5/8-inch are recommended. Pipe piles should be fabricated in accordance with ASTM A252, Grade 3, with minimum yield strength of 45 ksi.

Pipe piles should be protected from corrosion; a fusion-bonded epoxy coating of 18 to 20 mils in conjunction with aluminum alloy cathodic protection anodes are recommended.

GEOTECHNICAL DESIGN SUMMARY – CONTINUED

Pipe piles should be equipped with a cutting shoe, constructed from Grade ASTM A148 90/60 steel, and driven open ended. Pier pipe piles should be end bearing, and driven to the required resistance on or within bedrock. The factored structural resistance of eight pipe piles sections analyzed exceeds the factored geotechnical resistance and therefore the factored geotechnical resistance governs.

Pipe piles that do not achieve fixity for the design scour depth should be fixed to bedrock with a small H or W-section placed in 10-foot deep bedrock sockets. These should be 50 ksi steel H or W-sections. Any piles that are found to be in uplift when investigated for the strength, service or extreme limit states shall be fixed to bedrock with a 10-foot deep rock socket.

Piles for pier bents should be analyzed with respect to combined axial compression and flexure using an unsupported length that considers the depth of scour, the exposed pile length, and depth to pile fixity. The horizontal movement induced by lateral loads shall be evaluated using procedures that consider soil-structure interaction, per LRFD Article 10.7.2.4.

The Contractor is required to perform a wave equation analysis and conduct dynamic testing with restrike on the first pipe pile driven for each pier bent. 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 of 0.65. The maximum factored axial pile load required should be shown on the plans.

Bearing Capacity – The factored bearing pressure at the strength limit state for abutment spread footings on sound bedrock should not exceed the factored bearing resistance of 40 kips per square foot (ksf). A factored bearing resistance of 30 ksf may be used when analyzing the service limit state and for preliminary footing sizing, assuming a resistance factor, φ, of 1.0. In no instance shall the bearing stress exceed the nominal resistance of the footing concrete or seal concrete, estimated to be 0.3 *f'c*. No footing shall be less than 2 feet wide regardless of the applied bearing pressure or bearing material.

Scour and Riprap - For scour protection, any footing constructed on native subgrade soils, should be embedded for scour protection and armored with 3 feet of riprap as per Section 2.3.11.3 of the MaineDOT Bridge Design Guide (BDG). Prepared bedrock that is cleaned of all loose and fractured rock is not considered scour susceptible.

The effect of scour shall be considered in the design of pile pier bents. The Bridge Program estimates the design scour depth to range from approximately 22 to 55 feet, which correlates to the depth of soft, fine-grained, silt and clay deposits encountered. Pile pier bents shall be designed so that the pile embedment after the design scour event satisfies the required axial and lateral resistance.

GEOTECHNICAL DESIGN SUMMARY – CONTINUED

Settlement - Settlement of abutment spread footings founded directly on prepared, level, bedrock and sized for the service limit state is anticipated to be less than 0.5-inch. The settlement of pipe piles at the working load should not exceed 0.5-inch, provided the working load per pile does not exceed the calculated, factored geotechnical axial pile resistance.

Frost Protection - Foundations placed on bedrock are not subject to heave by frost, therefore, there are no frost embedment requirements for project footings cast directly on sound bedrock. Foundations placed on the granular fill soils should be founded a minimum of 5.3 feet below finished exterior grade for frost protection.

Approach Design. We recommend a subgrade resilient modulus of 4350 psi for pavement design. Bedrock is close to surface, and in some cases exposed, along the approaches to the bridge. These areas will require blasting to prepare for roadway construction and drainage features. "Fracture-blasting" is recommended to facilitate drainage of the roadway subbase. Therefore, in areas requiring blasting to construct the pavement structure, the contractor should be directed to overdrill blasting holes by 1 to 2 feet.

Seismic Design Considerations – Knickerbocker Bridge on Barters Island Road is not on the National Highway System (NHS) and is therefore not considered to be functionally important. Since the bridge construction costs will not exceed $10 million the bridge is not classified as a major structure. In conformance with the BDG, these criteria eliminate the requirement to design the bridge substructures for seismic earth loads.

The horizontal bedrock acceleration coefficient (A) for Boothbay is approximately 0.045g. Per LRFD Article 3.10.4 the bridge is assigned to Seismic Zone 1. In conformance with LRFD Article 4.7.4.3.1, for multispan bridges, no seismic analysis is required for bridges in Seismic Zone 1 with the exception of bridge seat width requirements specified in LRFD 4.7.4.4 and connection restraint per LRFD 3.10.9.2.

Construction Considerations - Lost drilling tools, cobbles and boulders may obstruct pile driving, pile installation and pile cleaning out operations at the pier locations. Obstructions may be cleared by conventional excavation methods, tool retrievers, preaugering, predrilling, spudding, airlifting, down-hole hammers, or other alternative methods.

At the proposed abutment spread footing locations, the bedrock surface shall be cleared of all loose, fractured and decomposed bedrock and loose soil. The resulting bearing surface shall be sound and competent. The borings indicate that bedrock excavation up to 3 feet may be required in some areas of the abutment footings to remove fractured bedrock and create a level or benched footing subgrade.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present geotechnical recommendations for replacement of Knickerbocker Bridge over the Back River on Barters Island Road in Boothbay, Maine. This report presents the soils information obtained at the site during the subsurface investigations, final foundation recommendations and geotechnical design recommendations for bridge replacement.

The Knickerbocker Bridge was built in 1930. The superstructure was subsequently replaced in 1980. The total bridge span is 535 feet, consisting of thirty-six (36) 14-foot spans and one (1) 28-foot span. The spill-through abutments consist of 2 rows of timber piles with crossbracing with anchor and tie-back systems for resisting lateral loads. The 36 intermediary supports consist of timber pier bents with 1 row of timber piles, with timber crossbracing. The timber piles are severely deteriorated and show evidence of damage by marine borers and ice action. The existing superstructure has a curb-to-curb width of 24 feet.

The proposed replacement bridge will be a 540-foot structure consisting of six (6) 90-foot spans. The superstructure will consist of precast, prestressed butted box beams. The replacement bridge will be built on a new alignment on the south side and downstream of the existing bridge. The proposed bridge under clearance will be increased. The proposed bridge profile will provide a clearance of 8.3 feet at the mid-span at mean high water (MHW). The superstructure curb-to-curb width will be 28.7 feet.

Preliminary foundation alternatives were provided by the geotechnical team member in an internal Geotechnical Design Memorandum, dated November 1, 2005, included as Appendix A of this report. Subsequent preliminary engineering assessments by the MaineDOT Bridge Program identified the most economical and practicable foundation alternatives for this site to be cantilever-type abutments and concrete filled, pipe pile bent piers, some with rock-socketed internal H or W-sections for fixity. The bridge approach side slopes will be retained with cast-in-place concrete wingwalls, in-line with the abutments.

2.0 GEOLOGIC SETTING

Knickerbocker Bridge on Barters Island Road in Boothbay, Maine crosses the Back River as shown on *Sheet 1 - Location Map* presented at the end of this report. The Back River flows southerly into the Sheepscot River and Sheepscot Bay, located south of the Knickerbocker Bridge site.

This portion of the Back River separates Hodgdon Island and Barters Island from Boothbay. This is a tidal location with tidal ebb and flood. The project Designer estimates the extreme tide elevation is at 6.5 feet and occurs about four times a year. This elevation does not include wave action or storm surge or adjustment for sea level trends. Based on these aspects, the extreme tide elevation selected for design is 7.5 feet.

According to the Maine Geologic Survey "Surficial Geology of the Westport Quadrangle, Maine, Open-file No. 76-40" (1976) the surficial soils in the vicinity of Knickerbocker Bridge consist of exposed bedrock and glacial marine deposits. The glacial marine geologic unit consists of mostly silt and clay, which is characterized by low strength and high compressibility. The glacial marine unit is composed of sediment that washed out of the Late Wisconsinan glacier and accumulated on the ocean floor during late-glacial time, when the relative sea level was higher than present.

The Bedrock Geologic Map of Maine (1985), indicates the bedrock at the project site is on the margin of the Cape Elizabeth and Bucksport Formations. The Cape Elizabeth Formation consists of interbedded, pelite and sandstone. The Bucksport Formation consists of calcareous sandstone, interbedded sandstone and impure limestone.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the proposed downstream alignment were explored by drilling nine (9) test borings. Borings BB-BBR-201 through BB-BBR-207 were drilled between August 20 and September 7, 2007. All borings were drilled by the MaineDOT Materials Testing and Exploration Drill Crew. Test borings BB-BBR-201 and BB-BBR-207 were drilled at the location of the proposed west and east abutments, respectively. Borings BB-BBR-202 through BB-BBR-206 were drilled at the approximate locations of the proposed five piers.

Three preliminary borings, BB-BR.BOO-101, BB-BR.BOO-102 and BB-BR.BOO-103 were drilled upstream of the existing bridge, during the preliminary design phase in 2005 for the purpose of identifying subsurface conditions. Whereas the proposed bridge alignment is downstream of the existing bridge, subsurface data from the three upstream explorations will only be discussed where relevant to the characterization of engineering and strength properties of the geologic soil units at the project site.

Boring locations are shown on Sheet 2 - Boring Location Plan, found at the end of this report.

The borings were drilled using cased wash boring techniques. Soil samples were typically obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6 inch interval of penetration are recorded. The standard penetration resistance, N-value, is the sum of the blows for the second and third intervals. The MaineDOT drill rig is newly equipped with a CME automatic hammer to drive the split spoon. The hammer was calibrated by MaineDOT in August of 2007 and was found to deliver approximately 30 percent more energy during driving than the standard rope and cathead system. All N-values discussed in this report are corrected values computed by applying an average energy transfer factor of 0.77 to the raw field N-values. This hammer efficiency factor (0.77) and both the raw field N-value and the corrected N-value are shown on the boring logs.

SPT sampling and testing in the preliminary borings, drilled in 2005, was performed with a standard rope and cathead system, and the N-values obtained in the field are equivalent to corrected N-values.

The bedrock was cored in the borings using an NQ 1.88-in I.D. core barrel and the Rock Quality Designation (RQD) of the core was calculated. The MaineDOT Geotechnical Team member selected the boring locations and drilling methods, designated type and depth of sampling techniques, reviewed field logs for accuracy and identified field and laboratory testing requirements. The MaineDOT Geotechnical Team member and a MaineDOT Certified Subsurface Investigator logged the subsurface conditions encountered. The borings were located in the field by a MaineDOT Survey Crew during the drilling program.

Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix B – Boring Logs and on Sheet 3 - Interpretive Subsurface Profile, found at the end of this report

4.0 LABORATORY TESTING

Laboratory testing for samples obtained in the preliminary and final borings consisted of one (1) standard grain size analysis, twenty-three (23) grain size analyses with hydrometer, twenty-four (24) natural water content tests, fourteen (14) Atterberg Limits tests and one (1) ignition loss test. The results of soil laboratory tests are included as Appendix C - Laboratory Data, at the end of this report. Laboratory test information is also shown on the boring logs provided in Appendix B – Boring Logs and on Sheets 4, 5 and 6 – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered at the explorations generally consisted of **topsoil**, **marine silts, glaciomarine deposits** and a **sand deposit,** all of which are underlain by **bedrock**. An interpretive subsurface profile depicting the detailed soil stratigraphy across the site is show on Sheet 3 - Interpretive Subsurface Profile found at the end of this report. A brief summary description of the strata encountered is as follows:

5.1 Topsoil

A layer of topsoil was encountered in the boring located in the vicinity of the proposed westerly abutment. The topsoil layer is approximately 2.9 feet thick. The topsoil encountered in boring BB-BBR-101 consisted of brown, dry, fine to medium silty SAND. SPT tests were not conducted in the topsoil deposit.

5.2 River Bottom and Marine Silt

Soft, fine-grained river bottom and marine deposits were encountered in all borings drilled in the channel of the Back River. The thickness of the encountered deposit was approximately 3.5 to 13.0 feet in the river channel along the proposed downstream alignment and consisted of grey, wet, very soft, clayey SILT and SILT, with little to some clay, little to trace gravel, sand and shell fragments, with an organic odor.

The thickness of the marine silt deposit encountered in the river channel upstream of the existing bridge ranged from approximately 5.0 to 11.5 feet and consisted of grey, wet, very soft, clayey SILT and organic SILT, with some to no clay, trace to no gravel, fine sand and shell fragments, with an organic odor.

Fifteen (15) attempts to conduct SPT tests in the silt deposit resulted in drill rods advancing through the unit under the dead weight of the rods (WOR) or the weight of the hammer (WOH), indicating that the soil unit is very soft in consistency.

Grain size analyses were conducted on nine samples from the silt deposit. The grain size analyses resulted in the soil being classified as an A-4 or A-6 under the AASHTO Soil Classification System and as an CL-ML or CL under the Unified Soil Classification System.

Natural water content tests on samples from the silt deposit determined moisture contents ranged from approximately 14 to 93 percent. Table 1, below, summarizes the results of natural water content tests made from samples of the silt deposit:

Sample No.	Soil Description	Water Content (%)
BB-BR.BOO-101, 2D	SILT	66.8
BB-BR.BOO-103, 2D/A	Clayey SILT	26.8
BB-BBR-202A, 1D	SILT	75.9
BB-BBR-202, 2D	SILT	31.4
BB-BBR-203, 1D	SILT	93.1
BB-BBR-203, 2D	SILT	13.9
BB-BBR-204, 1D	SILT	39.6
BB-BBR-205, 1D	SILT	53.9
BB-BBR-206, 2D	SILT	44.2

Table 1. Natural Water Content Test Results

Atterberg Limits tests on two samples from two silt deposit determined the plastic limits ranged from 16 to 25. The natural water contents of the two samples exceed the liquid limits, indicating that the soils have a high liquefaction potential. Slight disturbance can cause remolding in these soils and has the potential to transform this type of soil deposit into a viscous liquid. The calculated values of liquidity index for the soils tested where greater than 1, and therefore, the soil deposit is considered unconsolidated.

The following table summarizes the results of Atterberg Limits tests made from samples of the silt deposit:

Sample No.	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-BR.BOO-101, 2D	66.8	38	25	13	3.2
BB-BR.BOO-103, 2D/A	26.8	24	16	8	1.35

Table 2. Atterberg Limits Test Results

Vane shear tests conducted within the silt showed measured undrained shear strengths ranging from approximately 312 to 848 psf while the remolded shear strength ranged from 22 to 134 psf. One vane shear test showed a measured undrained shear strength in excess of 1217 psf, and was probably due to a sand seam. Based on the ratio of peak to remolded shear strengths from the vane shear tests, the marine silt deposit is determined to have a sensitivity ranging from 5 to 20, which correlates to a soil that is "sensitive" to "slightly quick". Slight disturbance can cause remolding in these soils and has the potential to transform this type of deposit into a viscous liquid.

Laboratory test results can be found in Appendix C - Laboratory Data. This testing information is also shown on the boring logs in Appendix B and on Sheets 4, 5 and 6 - Boring Logs found at the end of this report.

5.3 Weathered Glaciomarine Deposit

A relatively shallow and discontinuous layer of weathered glaciomarine soil, known as the Presumpscot Formation, was encountered below the marine silt. Where encountered, the thickness of the layer along the proposed downstream bridge alignment is approximately 5.0 to 9.5 feet thick in the river channel.

The soil deposit along the proposed downstream alignment generally consisted of olive grey or brown grey, mottled, SILT CLAY or silty CLAY, with trace to no fine sand, with blocky structure. Corrected SPT N-values taken in the layer ranged from WOR readings to 12 blows per foot (bpf) indicating that the soil deposit is very soft to stiff in consistency.

Grain size analyses were conducted on four (4) samples from the weathered glaciomarine unit. Grain size analyses resulted in the soil being classified as an A-4 or A-6 under the AASHTO Soil Classification System and as CL under the Unified Soil Classification System.

Vane shear tests conducted within the deposit showed measured undrained shear strengths ranging from approximately 670 to 2200 psf while the remolded shear strength ranged from 143 to 1100 psf. Based on the ratio of peak to remolded shear strengths from the vane shear tests, the glaciomarine unit is determined to have a sensitivity ranging from 2 to 4.7, which correlates to a soil that is "moderately sensitive" to disturbance. Atterberg Limits tests on samples from the deposit determined moisture contents ranged from approximately 23 to 32 percent and plastic limits ranged from 15 to 23. The natural water contents of the tested samples did not exceed the liquid limits, indicating that the soils do not have a liquefaction

potential. The calculated values of liquidity index for the soil tested were less than 1, and therefore, the soil deposit is considered heavily preconsolidated.

The following table summarizes the results of Atterberg Limits tests made from samples of the upper glaciomarine unit:

Sample No.	Soil Description	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-BBR-202, 4D	SILT CLAY	31.6	38	23	15	0.57
BB-BBR-205, 2D	clayey SILT	23.4	33	15	18	0.47
BB-BBR-206, 4D	silty CLAY	31.2	37	23	14	0.59

Table 3. Atterberg Limits Test Results

5.4 Glaciomarine Deposit

An unconsolidated, glaciomarine deposit, known as the Presumpscot Formation, was encountered below marine silt and the weathered glaciomarine deposits. The encountered thickness of the unit is approximately 9.3 to 36.5 feet thick in the river channel along the proposed downsteam bridge alignment. It was not encountered in the explorations located in the vicinity of the proposed abutments. The deposit consisted of predominately grey CLAY or clayey SILT.

Eighteen (18) attempts to conduct SPT tests in the unconsolidated clay silt deposit resulted in drill rods advancing through the soil unit under the dead WOR or the dead WOH, indicating that the soils are very soft in consistency.

Grain size analyses were conducted on ten (10) samples from the unconsolidated glaciomarine unit. Grain size analyses resulted in the soil being classified as an A-4 or A-6 under the AASHTO Soil Classification System and as CL or CL-ML under the Unified Soil Classification System.

Forty (40) undrained vane shear tests, conducted within the glaciomarine deposit, showed measured undrained shear strengths ranging from approximately 137 to 632 psf while the remolded shear strengths ranged from 22 to 134 psf. One vane shear test fell outside this range, with measured undrained peak and remolded shear strengths of 1250 and 179 psf. Based on the ratio of peak to remolded shear strengths from the vane shear tests, the glaciomarine unit is determined to have a sensitivity ranging from 4 to 16, which correlates to a soil that is “sensitive” to “slightly quick” to disturbance. Atterberg Limits tests on samples from the deposit determined moisture contents ranged from approximately 26 to 47 percent and plastic limits ranged from 11 to 25. The natural water contents of the tested samples exceed the liquid limits, indicating that the soils have a high liquefaction potential. Slight disturbance can cause remolding in these soils and has the potential to transform this type of deposit into a viscous liquid. The calculated values of liquid index for the soils tested where

greater than 1 for the nine (9) soil samples, and therefore, the soil deposit is considered unconsolidated.

The following table summarizes the results of Atterberg Limits test made from samples of the glaciomarine unit:

Sample No.	Soil Description	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-BBR-202, 6D	clayey SILT	30.4	28	15	13	1.19
BB-BBR-203, 5D	clayey SILT	44.5	37	25	12	1.62
BB-BBR-203, 9D	clayey SILT	46.6	35	11	24	1.48
BB-BBR-203, 11D	sandy SILT	26.1	-	-	-	-
BB-BBR-204, 3D	clayey SILT	36.0	27	14	13	1.69
BB-BBR-205, 6D	clayey SILT	36.6	25	20	5	3.32
BB-BR.BOO-101, 4D	clayey SILT	44.2	28	20	8	3.03
BB-BR.BOO-102, 3D	clayey SILT	41.7	32	19	13	1.75
BB-BR.BOO-102, 5D	clayey SILT	42.8	31	22	9	2.3
BB-BR.BOO-104, 5D	silty CLAY	43.0	34	22	12	1.75

Table 4. Atterberg Limits Test Results

Laboratory test results can be found in Appendix C - Laboratory Data. This testing information is also shown on the boring logs in Appendix B and on Sheets 4, 5 and 6 - Boring Logs found at the end of this report.

5.5 Glacial Sand

A discontinuous, predominantly sandy glacial deposit was encountered below the glaciomarine silts and clays. The glacial soils were either directly deposited by meltout from the moving ice sheet or were transported by water flowing from the ice sheet. The major portion of the deposit is SAND with the minor portions of silt, fine gravel, clay and boulders. When encountered along the proposed downstream bridge alignment, the thickness of the deposit varied from approximately 5.9 to 27.1 feet. An occasional boulder or indication of a boulder was encountered in the unit. Boulders greater than 12 inches in diameter and cobbles with diameters of 3 to 12 inches should be expected in this soil deposit.

Corrected SPT N-values taken in the layer ranged from 9 to 68 bpf indicating that the unit ranges from a loose to a very dense consistency.

A grain size analysis was conducted on one (1) sample from the deposit. The grain size analysis resulted in the sample being classified as A-1-b under the AASHTO Soil Classification System and SM under the Unified Soil Classification System. The measured water content for the sample was 10.6 percent. Laboratory test results can be found in

Appendix C - Laboratory Data. This testing information is also shown on the boring logs in Appendix B and on Sheet 4, 5 and 6 - Boring Logs found at the end of this report.

5.6 Bedrock

The bedrock below the proposed downstream bridge alignment was encountered in 7 of the 9 explorations, at depths ranging from approximately 2.9 feet to 81.6 feet. The table below summarizes top of bedrock elevations at the proposed abutment and pier locations.

Boring	Station	Structure	Depth to Bedrock, bgs (feet)	Elevation of Bedrock Surface (feet)
BB-BBR-201	10+11	Abutment 1	2.9	6.2
BB-BBR-202A	11+06	Pier 1	45.0	-50.8
BB-BBR-203	11+95	Pier 2	81.6	-93.1
BB-BBR-204	12+86.8	Pier 3	22.6	-38.2
BB-BBR-205	13+77.9	Pier 4	35.9	-47.3
BB-BBR-206	14+68.1	Pier 5	28.4	-34.3
BB-BBR-207A	15+40	Abutment 2	3.5	-7.0

Table 5. Top of Bedrock Elevations

The bedrock generally consists of grey to white BIOTITE SCHIST, fine to medium grained, soft to moderately hard, severely weathered to slightly weathered, with the primary joint set along the banding, dipping at all angles, tight to open with infilling; and beige to discolored and stained, quartz feldspar GNEISS, fine to medium grained, moderately hard, highly weathered to slightly weathered, with the primary joint set along banding, dipping at all angles, tight to open with discolored, with stained surfaces and silt infilling. The cores from BB-BBR-203 contained intrusions of PEGMATITE GRANITE within the gneiss. The Rock Quality Designation (RQD) of the bedrock was determined to range from 35 to 100 percent, which correlates to a rock of poor to excellent quality.

Water was encountered in each of the borings in the Back River; the water level appeared consistent with the water level in the Back River, which is tidal.

Refer to the boring logs in Appendix B for more detailed documentation of the conditions encountered in each exploration.

6.0 FOUNDATION ALTERNATIVES

Prior to the development of the Preliminary Design Report for the Knickerbocker Bridge, foundation alternatives were provided in an internal Geotechnical Design Memorandum, dated November 1, 2005, included as Appendix A of this report. The following foundations were considered for the replacement bridge substructures and evaluated for practicality in the November 1, 2005 memorandum:

- Abutments supported on spread footings (full height cantilever or gravity)
- Spill-through abutments constructed from socketed pipe piles or drilled shafts socketed in bedrock
- Pipe pile bent piers (all concrete filled pipe piles, and some with rock-socketed internal W or H-sections for fixity)
- Drilled shafts with rock sockets.

All of these foundation types are viable, to varying degrees, as foundation alternatives for this site, however, cantilever-type abutments on spread footings with intermediary, fixed pipe pile bent piers were selected by the Team as the most economical and practicable foundation type. The Preliminary Design Report for the Knickerbocker Bridge recommends that the replacement bridge consist of cantilever-type abutments on spread footings founded on bedrock and concrete filled pipe pile bent piers with rock sockets as necessary. This report addresses only those foundation types.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

The following subsections will discuss geotechnical design recommendations for cantilever abutments on spread footings founded on bedrock and pipe pile bent piers with rock sockets as necessary.

7.1 Cantilever-Type Abutments and Walls

Cantilever-type abutments and wingwalls shall be proportioned for all applicable load combinations specified in LRFD Articles 3.4.1 and 11.5.5. The design of walls founded on spread footings at the strength limit state shall consider nominal bearing resistance, overturning, lateral sliding and structural failure. Strength limit state design shall also consider foundation resistance after scour due to the design flood.

A resistance factor, φ, of 1.0 shall be used to assess spread footing design at the service limit state, including: settlement, horizontal movement, overall stability and scour at the design flood.

Extreme limit state design checks for spread footings shall include bearing resistance, eccentricity, sliding and overall stability. Resistance factors, φ, for the extreme event limit state shall be taken as 1.0.

Cantilever-type abutments and wingwalls shall be designed as unrestrained, meaning that they are free to rotate at the top in an active state of earth pressure. Earth loads shall be calculated using an active earth pressure coefficient, K_a, calculated using Rankine Theory for cantilever walls. See *Sheet 7 - Rankine and Coulomb Active Earth Pressure Coefficients* at the end of this report for guidance in calculating this value. Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the Bridge Design Guide (BDG) for the wingwalls if an approach slab is not specified. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from the table below:

Retaining Wall Height (feet)	h_{eq} (feet)	
	Distance from wall backface to edge of traffic = 0 foot	Distance from wall backface to edge of traffic is >= 1 foot
5	5.0	2.0
10	3.5	2.0
20	2.0	2.0

Table 6. Equivalent Height of Soil for Vehicular Loading

The Designer may assume Soil Type 4 (BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: $\phi = 32°$, $\gamma = 125$ pcf. Sliding computations for resistance to lateral loads shall assume a maximum allowable frictional coefficient of 0.70 at the rock-concrete interface for spread footings cast on level bedrock.

A sliding resistance factor, φ_τ, of 0.80 shall be applied to the nominal sliding resistance of walls founded on spread footings on level bedrock.

For footings on bedrock, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed three-eighths ($3/8^{ths}$) of the footing dimension, in either direction.

All abutment and wingwall designs shall include a drainage system to intercept any groundwater. Drainage behind structure shall be in accordance with Section 5.4.1.4 of the BDG. To avoid water intrusion behind the abutment the approach slab should be connected directly to the abutment.

Backfill within 10 feet of the abutments and wingwalls and side slope fill shall conform to Granular Borrow for Underwater Backfill - MaineDOT Specification 709.19. This gradation specifies 10 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.

7.2 Bearing Resistance

Project abutments and return wings on spread footings shall be proportioned to provide stability against bearing capacity failure. Application of permanent and transient loads is specified in LRFD Articles 11.5.5 and 3.4.1.

The bearing resistance for spread footings founded on competent, sound bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 40 ksf. This assumes a bearing resistance factor, φ_b, for spread footings on bedrock to be 0.45, based on bearing resistance evaluation using semi-empirical methods. The calculated factored bearing resistance is based on excavation of fractured bedrock to a depth where the RQD is at least 50%. A factored bearing resistance of 30 ksf and a resistance factor, φ, of 1.0 may be used when analyzing the service limit state. The stress distribution may be assumed to be linearly distributed over the effective base as shown in LRFD Figure 11.6.3.2-2. See Appendix D – Calculations, for supporting documentation.

The bearing resistance for spread footings shall be checked for the extreme limit state with a resistance factor of 1.0. Furthermore, footings shall be designed so that the nominal bearing resistance after the design scour event provides adequate resistance to support the unfactored Strength Limit States loads with a resistance factor of 1.0.

If the recommended value of presumptive bedrock bearing resistance exceeds nominal resistance of the footing concrete or seal concrete, the presumptive bearing resistance of the bedrock shall be taken as the nominal resistance of the concrete, estimated as 30 percent of the design compressive strength of the concrete (0.3 *f'c*). No footing shall be less than 2 feet wide regardless of the applied bearing pressure or bearing material.

7.3 Scour and Riprap

The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered at the strength and service limits states. Bedrock that is cleaned of all loose rock, highly fractured rock or soft weathered rock is not considered scour susceptible. In general, for scour protection, any footings which are constructed on soil should be embedded at least 3 feet below the design scour depth and armored with 3 feet of riprap for scour protection. Refer to BDG Section 2.3.11 for information regarding scour design.

Stone riprap conforming to Item number 703.26 of the Standard Specification shall be placed at the toes of any walls not constructed on bedrock. Riprap shall be 3 feet thick. The riprap shall extend horizontally in front of the wall face before sloping at maximum 1.75H:1V slope to the existing ground surface. The toe of the riprap section shall be constructed 1 foot below the streambed elevation. 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.

7.4 Pipe Pile Pier Bents

Pipe pile bent piers were selected for intermediate structure support. Piles for pile bent piers shall consist of concrete filled pipe piles driven to bedrock. It is estimated that all pipe piles will require rock-socketing with an internal H or W-section as there is insufficient overburden to provide a fixed condition at the pipe pile tip. Refer to Section 7.4.6 of this report for recommendations concerning the rock-socketed H or W-sections.

Pipe piles with diameters ranging from 24 to 30 inches and wall thicknesses of 1/2 to 5/8-inch are recommended. Pipe piles should be fabricated in accordance with ASTM A252, Grade 3, with minimum yield strength of 45 ksi. Pipe piles should be protected from corrosion with a minimum 18-mil fusion bonded epoxy coating from the top of the exposed pile (not the lengh embedded in the pile cap) to a minimum of 3 feet below the total predicted scour depth. Furthermore, sacrificial aluminum alloy anodes should be attached for cathodic protection.

Open-ended pipe piles should be equipped with a cutting shoe, constructed from Grade ASTM A148 90/60 steel, and driven open ended. Pier pipe piles should be end bearing, and driven to the required resistance on or within bedrock. Pipe piles that do not achieve fixity for the design scour depth should be fixed to bedrock with a small H or W-section. These should be end bearing, 50 ksi steel H or W-section piles.

7.4.1 Strength Limit State Design

Structural Resistance of Driven Pipe Piles. The nominal compressive structural resistance (P_n) for piles loaded in compression shall be as specified in LRFD Article 6.9.5.1. The pipe piles have an unbraced length and require calculation of the λ– factor as specified in Article 6.9.5.1.

For the strength limit state, the factored axial compressive structural resistance of the pile (P_r) shall be calculated using the resistance factor (ϕ_c) of 0.70 as specified in 6.5.4.2. The proposed piles at Pier 3 will potentially have the longest exposed pile length, 27 feet, therefore govern the structural resistance of piles at all piers. Factored axial compressive structural resistances in the lower portion of eight pipe pile sections for Pier 3 are provided in the Table 7. Supporting documentation is provided in Appendix D – Calculations.

Outside Pipe Pile Diameter (in.)	Factored Axial Structural Pile Resistance – Lower Portion of Pile @ Pier 3 ϕ_c=0.70 (kips)	
	Wall Thickness (in.)	
	1/2	5/8
24	648	846
26	719	952
28	798	1047
30	877	1151

Table 7. Factored Axial Structural Compressive Resistances for Eight Pipe Pile Sections

For structural analyses of pipe piles in combined compression and bending, resistance factors of 0.80 (ϕ_c) and 1.0 (ϕ_f) shall be applied to the compression and flexure terms, respectively, in the interaction equation. The factored structural resistances for pipe pile sections in combined axial compression and flexure are not provided in this report as these analyses are considered part of the structural design and the responsibility of the structural designer.

LRFD Article 10.5.5.3.2 states that the nominal resistance remaining after the scour resulting from the check flood shall provide adequate foundation resistance to support the unfactored Strength Limit State loads with a resistance factor of 1.0. The nominal structural axial compressive resistance of the pipe piles at Pier 2, due to an increased exposed length of pile due to scour, is provided in the Table 8. Supporting documentation is provided in Appendix D – Calculations. The check of axial pile compressive resistance against the unfactored Strength Limit State load group is left to the structural engineer to investigate.

Outside Pipe Pile Diameter (in.)	Nominal Axial Structural Pile Resistance – Lower Portion of Pile @ Pier 2 ϕ_c=1.0 (kips)	
	Wall Thickness (in.)	
	½	5/8
24	287	374
26	388	509
28	499	656
30	616	812

Table 8. Nominal Axial Structural Compressive Resistances for Eight Pipe Pile Sections after scour from the design flood

Geotechnical Compressive Pile Resistance. The factored axial geotechnical resistances of eight pipe pile sections were calculated using a resistance factor, ϕ_{stat}, of 0.45 for end bearing pile capacity on rock using the Kulhawy and Goodman RQD Method, and a resistance factor, ϕ_{stat}, of 0.45, for side frictional capacity using the Nordlund/Thurman method. The recommended factored axial geotechnical resistances of eight pipe pile sections are provided in Tables 9 and 10, below. Axial geotechnical pile resistances calculations can be found in Appendix D at the end of this report.

Substructure	Strength Limit State Factored Axial Geotechnical Resistance ϕ_{stat} = 0.45 (kips)			
	0.5 inch wall pipe piles			
	24-in	26-in	28-in	30-in
Pier 1	179	194	209	224
Pier 2	294	310	325	340
Pier 3	357	388	418	449
Pier 4	221	240	259	278
Pier 5	221	240	259	278
Recommended Design Resistance	230	245	260	280

Table 9. Strength Limit State factored Axial Geotechnical Resistance for 1/2-wall Pipe Pile Sections

Substructure	Strength Limit State Factored Axial Geotechnical Resistance ϕ_{stat} = 0.45 (kips)			
	5/8- inch wall pipe piles			
	24-in	26-in	28-in	30-in
Pier 1	222	241	260	279
Pier 2	338	357	376	395
Pier 3	444	482	520	558
Pier 4	275	299	322	346
Pier 5	275	299	322	346
Recommended Design Resistance	300	300	320	340

Table 10. Strength Limit State factored Axial Geotechnical Resistance for 5/8-wall Pipe Pile Sections

If there are less than 5 piles in pile bent, the resistance factor value, ϕ_{stat}, of 0.45 should be reduced by 20 percent and the factored axial geotechnical pile resistances in Tables 8 and 9 reduced accordingly, in accordance with LRFD 10.5.5.2.3. Using the assumption that 45 ksi steel will be used, the factored compressive structural resistance of the pipe piles exceeds the factored geotechnical resistance and therefore the geotechnical resistance governs.

The maximum factored axial pile load should not exceed the calculated factored geotechnical pile resistance in Tables 8 and 9, and shall be shown on the plans.

Per LRFD 10.5.5.3.2, the pier bents shall be designed so that the nominal resistance remaining after the design scour event is no less than the unfactored Strength Limit State loads with a resistance factor of 1.0, including any debris loads occurring from the flood event.

7.4.2 Estimated Depths to Pile Fixity

Stability of the pipe piles shall be evaluated in accordance with the provisions of LRFD Article 6.9 using an equivalent length of the pile that accounts for the laterally unsupported length of the exposed pile extending through the air and/or water, plus the embedment depth to pile fixity.

All piles should be designed to achieve a fixed condition for the design scour event. Preliminary depths to fixity for eight pipe pile cross sections were calculated, assuming only axial loading and without consideration of lateral loads, using the buckling methodology in LRFD 10.7.3.13.4. Tables 11 and 12 compare the calculated depths to fixity for piles at each pier bent to the available depth of overburden reduced by the estimated design scour depth. The design scour depth was estimated to be equal to the depth of the fine grained glaciomarine silt and clay deposit. The analyses indicate that pipe piles at all bent locations do not achieve a fixed condition and should be rock-socketed with a W- or H-section. Calculations are provided in Appendix D – Calculations. W- or H-sections will depend on the pipe pile size selected.

When lateral and axial load groups are known, this data should be provided to the geotechnical engineer. A more refined analysis of pile fixity can then be performed using LPile or FBPier software.

	Preliminary Estimate of Depth to Fixity w/ no lateral loads applied (feet)				Depth of Overburden (feet)	Depth of Overburden considering estimated Scour (feet)
	0.5 inch wall pipe piles					
	24-in	26-in	28-in	30-in		
Pier 1	18	19	20	22	45	14
Pier 2	17	18	20	21	81	28
Pier 3	19	21	22	23	23	0
Pier 4	19	20	21	23	36	6
Pier 5	19	20	21	23	28	2

Table 11. Preliminary Estimates of Depth to Fixity for ½ inch Wall Pipe piles.

	Preliminary Estimate of Depth to Fixity w/ no lateral loads applied (feet)				Depth of Overburden (feet)	Depth of Overburden considering estimated scour (feet)
	5/8-inch wall pipe piles					
	24-in	26-in	28-in	30-in		
Pier 1	18	20	21	22	45	14
Pier 2	18	19	20	22	81	28
Pier 3	20	21	23	24	23	0
Pier 4	19	21	22	23	36	6
Pier 5	19	21	22	23	28	2

Table 12. Preliminary Estimates of Depth to Fixity – 5/8-inch wall piles

7.4.3 Buckling and Combined Axial and Flexure

Pile group design shall consider loading effects due to combined axial and flexural loading, as outlined in Article 6.15 of the LRFD Specifications. In designing piles for the bent group the depth to scour and effects of soil-structure interaction shall be considered, in conformance with Article 10.7.3.9 of the LRFD Specifications. The recommended design approach considers the non-linear response of soil with lateral displacement. Soil-structure interaction considering the non-linear response of soil can be modeled using computer software supplied by the geotechnical engineer.

The factored structural resistances for pipe pile sections in combined axial compression and flexure are not provided in this report as these analyses are considered part of the structural design and the responsibility of the structural designer.

7.4.4 Service Limit and Extreme Limit State Designs

Per LRFD 10.5.5.1, the ability of the pier bents to meet deflection criteria at the service limit state shall be investigated using a resistance factor of 1.0, and shall consider deflection after scour due to the design flood. Factored geotechnical axial resistances of eight pipe pile sections were calculated for the service limit state using the Kulhawy and Goodman RQD method, and are summarized below in Tables 13 and 14. Supporting documentation is provided in Appendix D – Calculations.

The pier bents shall be designed for the Extreme Event II limit state. Extreme events include scour for ice, vessel impact and debris loading. Resistance factors for the extreme limit state shall be taken as 1.0, with the exception for pile uplift resistance, which shall be taken as 0.80 or less.

Any piles that are found to be in uplift when investigated for combined axial and lateral loads in the strength, service or extreme limits states, shall be fixed to bedrock with a 10-foot rock socketed H- or W-section designed to resist the uplift load.

Factored geotechnical axial compressive resistances of eight pipe pile sections for the extreme limit state are provided below in Tables 13 and 14.

	Service and Extreme Event II Limit States Factored Axial Compressive Geotechnical Resistance $\phi_{stat} = 1.0$ (kips)			
	0.5 inch wall pipe piles			
	24-in	26-in	28-in	30-in
Pier 1	397	431	465	499
Pier 2	654	688	722	756
Pier 3	794	862	929	997
Pier 4	492	533	575	617
Pier 5	492	533	575	617
Recommended Design Resistance	510	545	584	622

Table 13. Service and Extreme Limit State Factored Axial Geotechnical Resistances for 1/2-wall Pipe Pile Sections

	Service and Extreme Event II Limit State Factored Axial Compressive Geotechnical Resistance ϕ_{stat} = 1.0 (kips)			
	5/8- inch wall pipe piles			
	24-in	26-in	28-in	30-in
Pier 1	494	536	578	621
Pier 2	751	793	835	878
Pier 3	988	1072	1157	1241
Pier 4	611	663	716	768
Pier 5	611	663	716	768
Recommended Design Resistance	617	664	711	758

Table 14. Service and Extreme Limit State Factored Axial Geotechnical Resistances for 5/8-wall Pipe Pile Sections

7.4.5 Ultimate pile resistance and pile quality control program

Contract documents should require that the contractor perform a wave equation analysis of the proposed pipe pile driving system. The first pipe pile driven for each pile bent should be dynamically tested to confirm capacity and verify the stopping criteria developed by the Contractor. Restrikes will be required as part of the pile field quality control program. With this level of quality control, the ultimate resistance that must be achieved in the wave equation analysis and dynamic testing will be the maximum factored axial pile load divided by a resistance factored of 0.65. This resistance factor assumes that a field dynamic testing will be performed on one test pile per pier bent. Calculations for the ultimate pile resistance required by a driveablity analysis are provided in Appendix D – Calculations.

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 $0.90\phi_{da} F_y$, where ϕ_{da} is equal to 1.0, in accordance with LRFD Article 10.7.8. A hammer should be selected which provides the required resistance when the penetration resistance for the final 3 to 6 inches is 8 to 13 blows per 1 inch. 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.

7.4.6 Bedrock-socketed H or W-sections

To satisfy requirements for fixity, pipe piles will be fixed with internal H or W-sections socketed in bedrock. Internal W or H-sections should be placed in minimum 10-foot deep rock sockets and extend a minimum of 10 feet into the lower end of the pipe piles. The H or W- sections should be fixed in the bedrock sockets with concrete.

Bedrock sockets may be drilled using rotary duplex methods with down-the-hole hammers, rotary percussive methods or solid coring methods. The rock socket should have a diameter of at least 2 inches greater than the diagonal H or W-section dimension, but must be at least 12 inches less than pipe pile inside diameter, but also has to fit inside the rebar cage, so as to not compromise the end bearing resistance of the pipe pile on bedrock. The rock socket shall be constructed so as to have a planar bottom. The pipe pile should be driven to the required nominal resistance, and then the 10-foot deep rock socket drilled out. Once the rock socket is drilled and the socket and pipe pile cleaned out, the H or W-section and reinforcing cage should be installed, and the pipe pile tremie-filled with Class A concrete. If the clean-out operations disturb the end bearing capacity of the pipe pile, or if there is indication that the pipe pile is not on bedrock, then the piles shall be re-driven to the stopping criteria established by the wave equation analysis and dynamic pile testing.

7.5 Settlement

Any settlement of bridge abutments will be due to the elastic settlement of the bedrock, and is anticipated to occur during construction of the abutments, and will be negligible.

The settlement of pier bent pipe piles due to compression of the bedrock at the working loads should not exceed 0.5-inch, provided the working load per pile does not exceed the calculated, factored geotechnical axial pile resistance. In general, relatively short penetration into rock is needed to mobilize the required pile resistance.

7.6 Frost Protection

Foundations placed on bedrock are not subject to heave by frost, therefore, there are no frost embedment requirements for project footings cast directly on sound bedrock. In the case that project final engineering introduces foundations placed on fill soils, the foundations should be designed with an appropriate embedment for frost protection. According to BDG Figure 5-1, Maine Design Freezing Index Map, the site has a design freezing index of approximately 1300 F-degree days. An assumed water content of 20% was used for moist, coarse grained soils above the water table. These components correlate to a frost depth of 5.3 feet. Therefore, any foundations placed on native soil should be founded a minimum of 5.3 feet below finished exterior grade for frost protection. Supporting calculations are provided in Appendix D.

7.7 Approach Design

In designing the approaches to the replacement bridge, we recommend a subgrade resilient modulus of 4350 psi for pavement design.

Bedrock is close to surface, and in some cases exposed, along the approaches to the bridge. These areas will require blasting to prepare for roadway construction and drainage features. "Fracture-blasting" is recommended to facilitate drainage of the roadway subbase. Therefore, in areas requiring blasting to construct the pavement structure, the contractor should be directed to overdrill blast holes by 1 to 2 feet.

7.8 Seismic Design Considerations

The horizontal bedrock acceleration coefficient (A) for Boothbay is approximately 0.045g, based on Figure 3-4 of the BDG, Seismic Performance Categories for Maine, August 2003. Per LRFD Article 3.10.4, the bridge is assigned to Seismic Zone 1. Soil Profile Type III is applicable to the site and a Site Coefficient (S) of 1.5 should be used. In conformance with LRFD Article 4.7.4.3.1, for multispan bridges, no seismic analysis is required for single or multiple span bridges in Seismic Zone 1 with the exception of bridge seat width requirements specified in LRFD 4.7.4.4 and connection restraint per LRFD 3.10.9.2.

Per BDG Section 3.7.1.1, bridges located in areas where the horizontal acceleration coefficient is less than or equal to 0.09g are designated a Seismic Performance Category (SPC) classification of A. For SPC A, no detailed analysis is required other than connection design and bearing seat length, except if the bridge is functionally important or classified as a major structure. According to Figure 2-2 of the BDG, Knickerbocker Bridge is not on the National Highway System (NHS) and is therefore not considered to be functionally important, and since the bridge construction costs do not exceed $10 million the bridge is not classified as a major structure.

7.9 Construction Considerations

There is a potential that lost drilling tools, cobbles, and boulders may obstruct pile installation operations, including, but not limited to, driving piles and cleaning out pipe piles. Obstructions may be cleared by conventional excavation methods, tool retrievers, preaugering, predrilling, or down-hole hammers. Alternative methods to clear obstructions may be used as approved by the Resident.

At the proposed abutment spread footing locations, the bedrock surface shall be cleaned of all loose, fractured and decomposed bedrock. The resulting bearing surface shall be sound and competent. The bearing surface shall then be washed with high-pressure water and air prior to concrete being placed for the footing. Excavation of bedrock material may be done using conventional excavation methods and drilling and blasting techniques. Blasting should be conducted in accordance with Section 105.2.6 of the MaineDOT Standard Specifications.

Where the bedrock surface slopes toward the river channel, the bedrock surface shall be stepped to create level benches or excavated to be level overall. Elsewhere, the bedrock surface slope shall be less than 4 horizontal to 1 vertical (4H:1V) or it shall be benched in level steps or excavated to be completely level, to improve sliding resistance.

The borings indicate that bedrock excavation up to 3 feet may be required in some areas of the abutment footings to remove fractured bedrock and create a level or benched footing subgrade. The nature, slope and degree of fracturing in the bedrock bearing surfaces will not be evident until the foundation excavation is made.

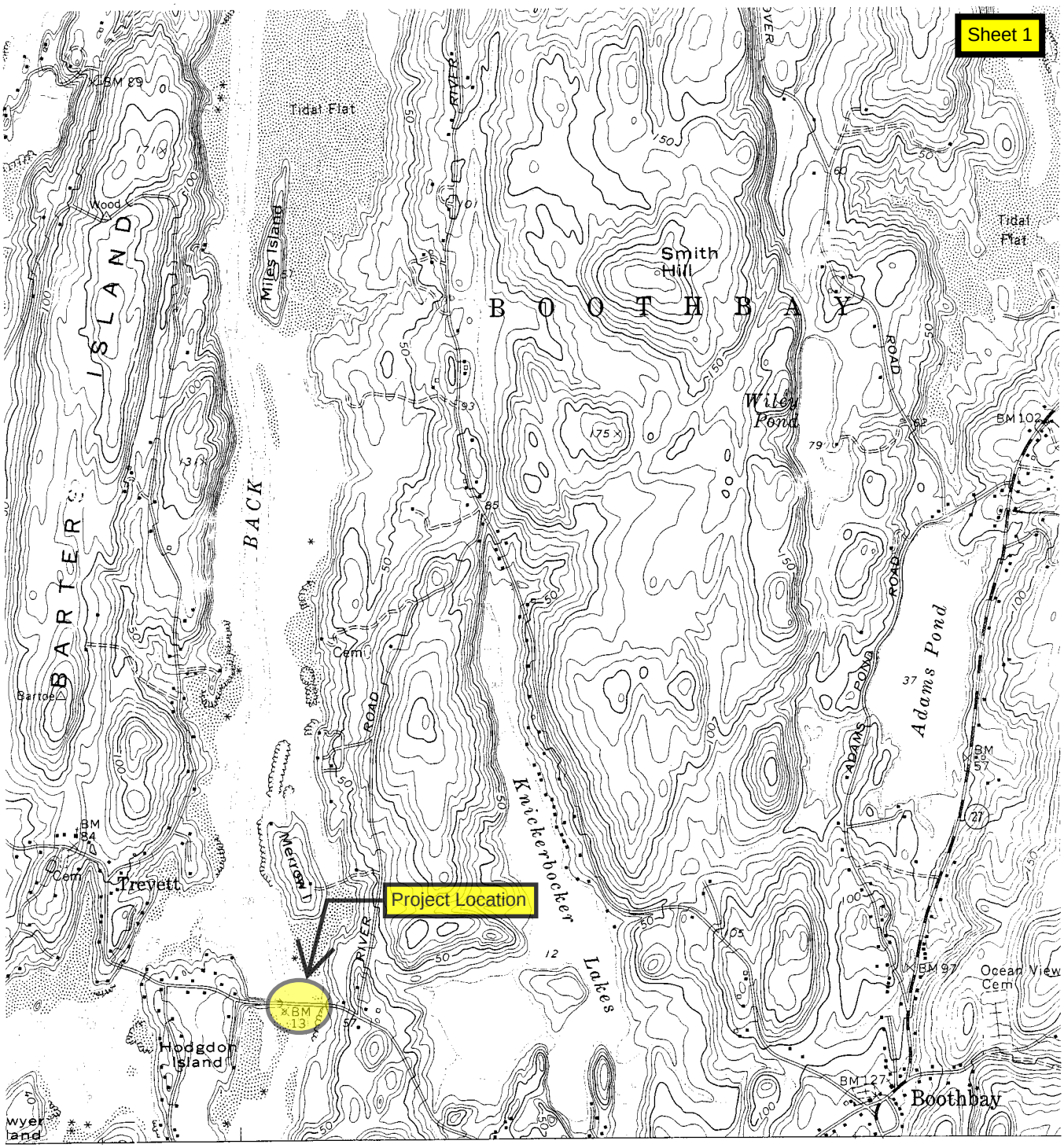
The final bedrock surface shall be approved by the Resident prior to placement of the footing concrete.

8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Knickerbocker Bridge in Boothbay, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use is 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



WESTPORT QUADRANGLE
MAINE
7.5 MINUTE SERIES (TOPOGRAPHIC)
NW/4 BOOTHBAY 15' QUADRANGLE

Boothbay, Maine
Knickerbocker Bridge
over
Back River
PIN. 12630.00

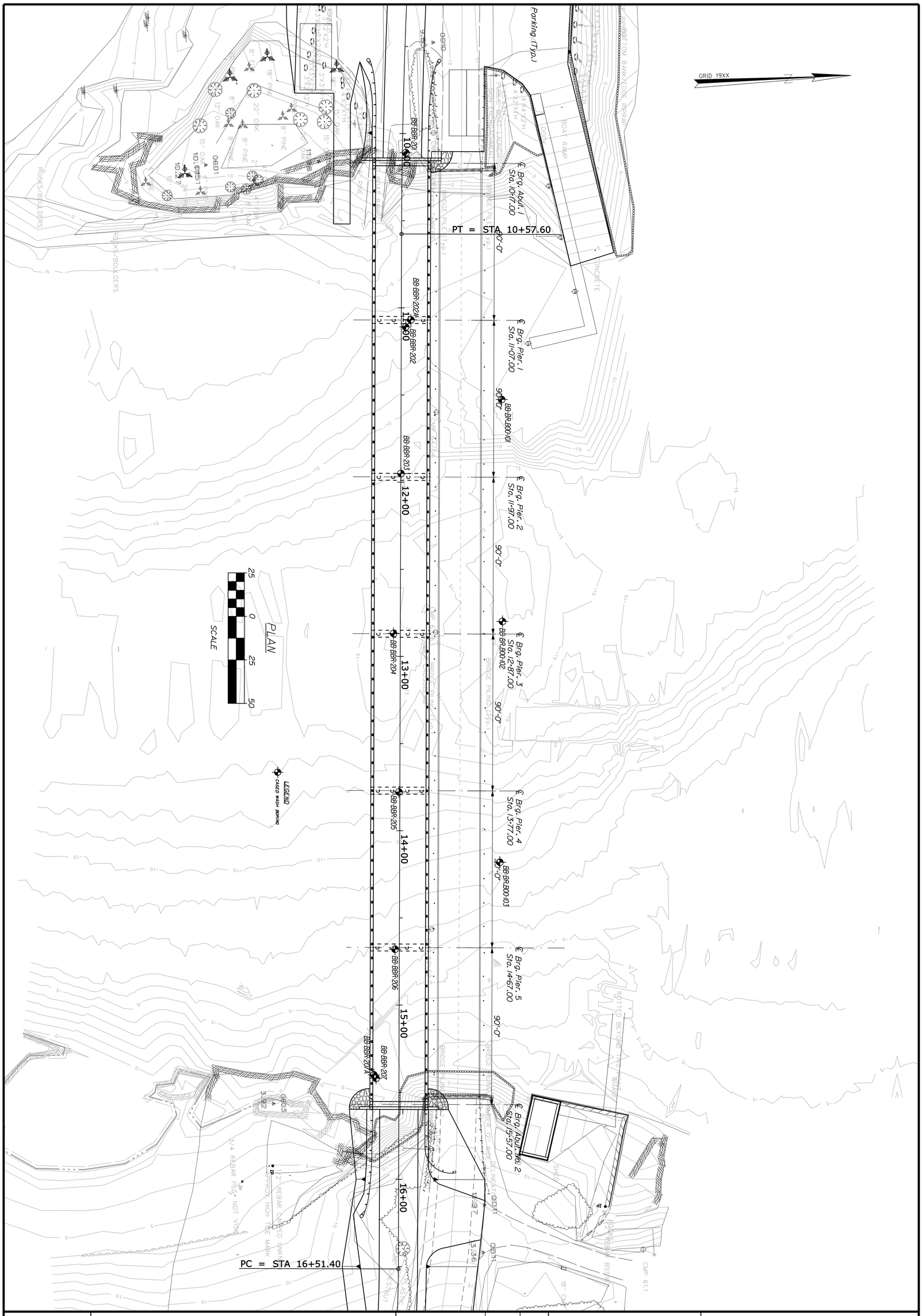
SCALE 1:24 000

1 ½ 0 1 MILE

1000 0 1000 2000 3000 4000 5000 6000 7000 FEET

1 .5 0 1 KILOMETER

CONTOUR INTERVAL 10 FEET
DATUM IS MEAN SEA LEVEL



PROJ. MANAGER W. FRANKHAUSER		BY	DATE	
DESIGN-DETAILED	L. KRUSINSKI	T. WHITE	FEB 2008	
CHECKED-REVIEWED	------	------	------	SIGNATURE
DESIGN2-DETAILED2	------	------	------	
DESIGN3-DETAILED3	------	------	------	P.E. NUMBER
REVISIONS 1	------		------	
REVISIONS 2	------		------	
REVISIONS 3	------		------	DATE
REVISIONS 4	------		------	
FIELD CHANGES	------		------	

KNICKERBOCKER BRIDGE
BACK RIVER
BOOTHBAY LINCOLN COUNTY

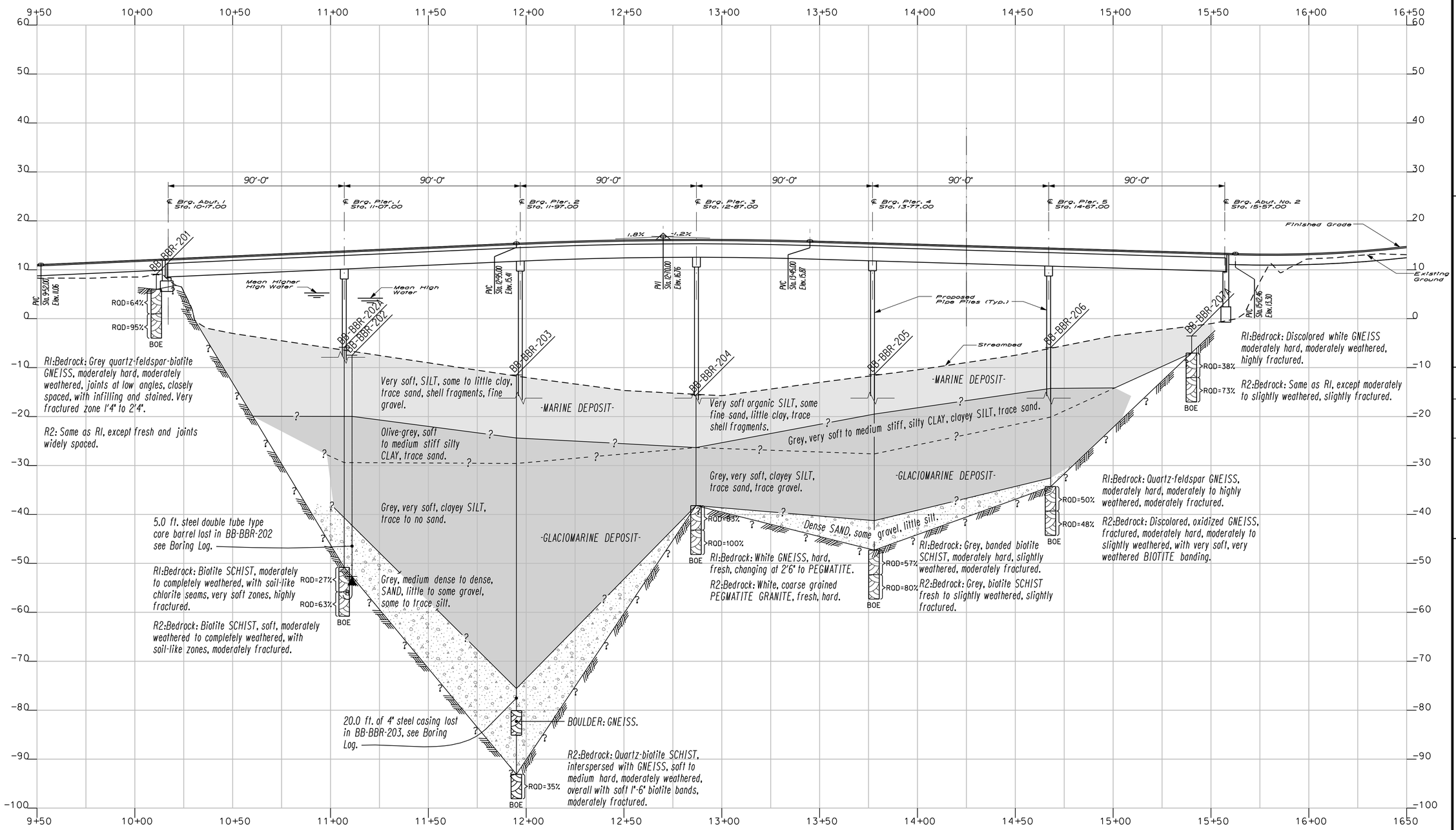
BORING LOCATION PLAN

SHEET NUMBER 2 OF 7

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR 1263(000)X

BRIDGE NO. 2438 PIN 12630.00 BRIDGE PLANS



PROFILE

HORIZ. 25 0 25 50

VERT. 10 0 10 20

SCALE

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.

Borings BB-BR.B00-101, BB-BR.B00-102, BB-BR.B00-103 and BB-BBR-207 are not shown, see Boring Log Sheets.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR 1263(000)X

PIN 12630.00

BRIDGE NO. 2438

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	W. FRANKHAUSER	
DESIGN-DETAILED	L. KRUSINSKI	FEB 2008
CHECKED-REVIEWED	T. WHITE	
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

KNICKERBOCKER BRIDGE
BACK RIVER
BOOTHBAY LINCOLN COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

3

OF 7

Filename: ...\00\geotech\msta\003_ISP1.dgn Division: GEOTECH Username: terry.white Date:2/8/2008

Filename: ...\mstio\004_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:2/8/2008

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

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BBR-201
PIN: 12630.00

Driller:	MaineDOT	Elevation (ft.)	9.1
Operator:	E. Giguere/G. Lidstone	Datum:	NAVD 88
Logged By:	B. Wilder	Rig Type:	CME 45C
Date Start/Finish:	9/6/07; 12:30-15:00	Drilling Method:	Cased Wash Boring
Boring Location:	10+11, 2.0' Lt.	Casing ID/OD:	HW
Auger ID/OD:	5" Solid Stem Auger	Sampler:	Standard Split Spoon
Hammer Wt./Fall:	CME 340 Auto Hammer/30"	Core Barrel:	NQ-1.88
Water Level*:	None Observed		

Hammer Efficiency Factor: .77 — Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA			Brown, dry, fine to medium SAND. (Topsoil).	
	R1	60/60	3.1 - 8.1	RQD = 64%			ND	6.20 / 6.00		Top of Bedrock at Elev. 6.2'	
										R1: Bedrock: Grey, medium grained, quartz-feldspar-biotite GNEISS, moderately hard, moderately weathered, joints along banding at low angles, close, open, with infilling, surfaces stained and weathered. Very fractured zone 1'4" to 2'4" into core. Slightly fractured overall. Core Times (min:sec) 3.1-4.1' (2:08) 4.1-5.1' (1:50) 5.1-6.1' (1:28) 6.1-7.1' (1:33) 7.1-8.1' (1:29) 100% Recovery	
	R2	60/60	8.1 - 13.1	RQD = 95%						R2: Grey, medium grained quartz-feldspar-biotite GNEISS, moderately hard, fresh, joints widely spaced, dipping a low angles, open, weathered, stained. Slightly fractured to massive. Core Times (min:sec) 8.1-9.1' (1:55) 9.1-10.1' (1:40) 10.1-11.1' (1:49) 11.1-12.1' (1:51) 12.1-13.1' (1:55) 100% Recovery	
20								-4.00		Bottom of Exploration at 13.10 feet below ground surface.	

Remarks:

Stratification lines represent approximate boundaries between soil typest 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.

Page 1 of 1
Boring No.: BB-BBR-201

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

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BBR-202A
PIN: 12630.00

Driller:	MaineDOT	Elevation (ft.)	-5.82
Operator:	E. Giguere/G. Lidstone	Datum:	NAVD 88
Logged By:	B. Wilder/L. Krusinski	Rig Type:	CME 45C on Barge
Date Start/Finish:	8/22-07-8/22-07	Drilling Method:	Cased Wash Boring
Boring Location:	11+07, 5.5' Lt.	Casing ID/OD:	HW
Auger ID/OD:	N/A	Sampler:	Standard Split Spoon
Hammer Wt./Fall:	CME 340 Auto Hammer/30"	Core Barrel:	NQ-1.88
Water Level*:	Back River (Tidal)		

Hammer Efficiency Factor: .77 — Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0	10	24/14	0.0 - 2.0	Hydraulic Push	---		WOC			Dark grey, wet, very soft, organic SILT, some fine sand, little clay, trace of gravel, shells and wood fragments.	GA209954 CL-ML, A-4 WC=75.9% Ignition Loss LOI=5.1% H2O=79.3%
							WOC	-7.82			
							WOH			Material descriptions not taken, see BB-BBR-202.	
							WOH				
							4			HP=Hydraulic Push	
							HP				
							HP				
							8				
							22				
							19				
							26			Washed casing out.	
							138				
							403				
							92				
							92				
							74				
							114				
							95				
							106				
							99				
							100				
							132				
							99				
							103				
							90				
							86				
							95				
							62				
							54				
							50				
							67				
							82				
							102				
							128				
							185				
							114				
							104				
							112				
							95				
							91				
							102				
							70				
							114				
							96				
							200				
45	R1	60/60	45.0 - 50.0	RQD = 27%			ND	-50.82		Top of Bedrock at Elev. -50.82'	
										R1: Bedrock: Grey and white banded, fine to medium grained BIOTITE SCHIST. Moderately to completely weathered, soil like schist seams. Joints at low to moderately angles along bedding/banding, second joint set steep, closed, tight, no infilling. Highly fractured, very soft, soil like zones between 3'7" and 5'0", joints along layering, spacing close to very close. Correlation of RQD to rock quality: Poor. Core Times (min:sec) 45.0-46.0' (12:17) 450 psi 46.0-47.0' (21:47) 500 psi 47.0-48.0' (12:15) " 48.0-49.0' (21:48) " 49.0-50.0' (12:53) " 100% Recovery	
50	R2	60/60	50.0 - 55.0	RQD = 63%				-55.82		R2: Bedrock: Grey, fine to medium grained, BIOTITE SCHIST, soft, moderately weathered to completely weathered soil like zones 2'1"-2'10", moderately fractured, joint set horiz. to dipping at low angles, spaced very close to close, tight, Second joint set near vertical, widely spaced, surfaces fresh to oxidized, soil like in softer zones: no layering or banding evident. Correlation of RQD to rock quality: Fair. Core Times (min:sec) 50.0-51.0' (40:09) 500 psi 51.0-52.0' (40:09) " 52.0-53.0' (41:51) " 53.0-54.0' (11:59) " 54.0-55.0' (12:12) " 100% Recovery	
55								-60.82		Bottom of Exploration at 55.00 feet below ground surface.	

Remarks:

Stratification lines represent approximate boundaries between soil typest 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.

Page 1 of 1
Boring No.: BB-BBR-202A

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

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BBR-202
PIN: 12630.00

Driller:	MaineDOT	Elevation (ft.)	-7.46
Operator:	E. Giguere/G. Lidstone	Datum:	NAVD 88
Logged By:	B. Wilder/L. Krusinski	Rig Type:	CME 45C on Barge
Date Start/Finish:	8/20/07-8/22/07	Drilling Method:	Cased Wash Boring
Boring Location:	11+11, 2.49' Lt.	Casing ID/OD:	HW
Auger ID/OD:	N/A	Sampler:	Standard Split Spoon
Hammer Wt./Fall:	CME 340 Auto Hammer/30"	Core Barrel:	NQ-1.88
Water Level*:	Back River (Tidal)		

Hammer Efficiency Factor: .77 — Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							WOH				
							WOH				
							WOH				
							2				
	10	24/2	4.0 - 6.0	WOH/WOH/WOH/WOH			WOH			Dark grey, wet, very soft, clayey SILT, little fine sand, trace shell fragments, plastic.	
							WOH				
							3				
							3				
	20 V1	24/10	9.0 - 11.0 9.5 - 10.0	WOR/WOR/WOR/3 Su=65/22 psf	WOR		5			Dark grey, wet, very soft, SILT, some clay, little fine sand, little fine gravel, trace shell fragments, plastic, slightly organic.	GA209953 CL-ML, A-4 WC=31.4%
	MV		10.2 - 10.6	Could not push			7			55x110 mm vane raw torque readings: V1: 10.0/0.5 ft-lbs Failed 55x110 mm vane attempt.	
							20				
							44	-19.96			
	30 MV	24/24	13.0 - 15.0 15.0 - 13.6	4/4/5/6 Could not push	9	12	39			Olive grey and brown, damp, mottled, silt CLAY, stiff, medium plasticity, blocky. Failed 55x110 mm vane attempt.	
							49	-12.5			
							39				
							74				
							73				
	40 MV	24/24	18.0 - 20.0 20.0 - 18.4	1/2/2/3 Could not push	4	5	60			Olive grey, very little mottling, soft, silt CLAY, damp, plastic, easily rolled into #1/4" threads, blocky, medium stiff.	GA209951 CL, A-6 LL=38 PL=23 PI=15 WC=31.6%
							60				
							54				
							55	-22.0			
							52	-29.46			
	50 V2	24/24	23.0 - 25.0 25.0 - 24.0	WOR/WOR/WOR/WOH Su=87/93 psf	WOH		22			Dark grey, wet, very soft, CLAY, little silt, high plasticity, too high water content to roll threads, homogeneous. 55x110 mm vane raw torque readings: V2: 11.0/2.0 ft-lbs 55x110 mm vane raw torque readings: V3: 10.5/2.0 ft-lbs	
	V3		24.0 - 24.6 25.0	Su=469/89 psf			54				
							50				
							43				
							46				
	60 V4	24/24	28.0 - 30.0 30.0 - 29.0	WOR/WOR/WOR/WOR Su=357/92 psf	WOR		56			Dark grey, wet, very soft, clayey SILT, trace sand, one 1" angular gravel, high plasticity, excess water-can't roll into threads, fat clay, (Two cup sample) 55x110 mm vane raw torque readings: V4: 15.0/3.0 ft-lbs 55x110 mm vane raw torque readings: V5: 13.0/2.0 ft-lbs	GA209952 CL, A-6 LL=28 PL=15 PI=13 WC=30.4%
	V5		29.0 - 29.6 30.0	Su=357/55 psf			61				
							53				
							48	-38.79			
							60				
								-31.3			
	TO	24/13	33.0 - 35.0	6/22/31/21	53	68	43			Brown, wet, very dense, fine to coarse SAND, little silt, changing to medium to coarse sand, some fine gravel (subangular), trace weathered biotite schist, trace coarse angular gravel.	
							75				
							72				
							102				
							95				
	MD	24/0	38.0 - 40.0	11/14/13/12	27	35	86			Failed sample attempt.	
							96				
							82				
							152				
							111				
	BD	22/15	43.0 - 44.8	6/11/9/5(10")	20	26	95			Grey, wet, medium dense, fine to coarse SAND, little fine angular gravel, well graded, trace silt, homogeneous, sorted, (Glaciofluvial). 0142 blows for 10". Roller cored ahead to 45.0' bgs.	
							0142	-52.63			
	R1	0/0	45.0 - 45.0							Failed core attempt, casing too bent for coring. Lost core barrel in boring, moved to BB-BBR-202A.	
50								-52.4		Bottom of Exploration at 45.17 feet below ground surface. Surveyed Bedrock Elev. -52.63'.	

Remarks:

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Page 1 of 1
Boring No.: BB-BBR-202

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

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BR.B00-101
PIN: 12630.00

Driller:	MaineDOT	Elevation (ft.)	-13.2
Operator:	E. Giguere	Datum:	NAVD 88
Logged By:	G. Lidstone	Rig Type:	CME 45C
Date Start/Finish:	8/24/05; 08:00-17:00	Drilling Method:	Cased Wash Boring
Boring Location:	11+52.4, 57.7' Lt.	Casing ID/OD:	HW
Auger ID/OD:	N/A	Sampler:	Standard Split Spoon
Hammer Wt./Fall:	140#/30"	Core Barrel:	NQ-2"
Water Level*:	Tidal		

Hammer Efficiency Factor: 0.60 — Hammer Type: Automatic ☐ Hydraulic ☐ Rope & Cathead ☒

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0	10	24/15	0.0 - 2.0	WOR/WOR/WOR/WOR			WOC			Dark grey, wet, very soft, organic SILT, little fine sand, trace shells. -MARINE DEPOSIT-	
							WOC				
							WOC				
							WOH				
							WOH				
5	20	24/17	5.0 - 7.0	WOR/WOR/WOR/WOR	WOR		WOH	-18.20		Grey, wet, very soft, SILT, some clay, trace sand, trace organics.	GA181934 A-6, CL WC=66.8% LL=38 PL=25 PI=13
							2				
							2				
							2				
10	30 V1	24/14	10.0 - 12.0 10.0 - 11.0	WOR/WOR/WOR/WOH V1: Su=48/134 psf	0	0	12			55x110 mm vane raw torque readings: V1: 15.0/3.0 ft-lbs (probable sand seam) Failed 55x110 mm vane attempt (probable fine sand seam).	
	MV		11.0 - 11.1 11.1 - 11.3	MV1 Could not push			29	-24.70			
							46				
							62				
							51	-41.5		Grey-brown, SILT, trace fine sand, trace clay.	
								-14.0			
15	40	24/24	15.0 - 17.0	WOR/WOR/WOR/WOR	WOR		29			Grey, wet, soft, clayey SILT, trace sand. -GLACIOMARINE DEPOSIT- Washed ahead of casing from 15.0-17.0' bgs.	GA181935 A-6, CL WC=44.2% LL=28 PL=20 PI=8
							28				
	V2		17.6 - 18.0	Su=335/27 psf			20			65x130 mm vane raw torque readings: V2: 12.2/1.0 ft-lbs	
	V3		18.6 - 19.0	Su=343/22 psf			16			65x130 mm vane raw torque readings: V3: 12.5/0.8 ft-lbs	
20	50 V4	24/24	20.0 - 22.0 20.0 - 20.6	WOR/WOR/WOR/WOR V4: Su=335/27 psf	WOR		13			Grey, wet, soft, clayey SILT. 65x130 mm vane raw torque readings: V4: 12.2/1.0 ft-lbs	
	V5		21.0 - 21.6 22.0	V5: Su=330/33 psf			11			65x130 mm vane raw torque readings: V5: 12.0/1.2 ft-lbs	
							11				
							10				
25	60 V6	24/24	25.0 - 27.0 27.0 - 26.0	WOR/WOR/WOR/WOR V6: Su=333/33 psf	WOR		14			Grey, wet, very soft, clayey SILT. 65x130 mm vane raw torque readings: V6: 8.5/1.2 ft-lbs	
	V7		26.0 - 26.6 27.0	V7: Su=233/41 psf			15			65x130 mm vane raw torque readings: V7: 8.5/1.5 ft-lbs	
							14				
30	70 V8	24/24	29.0 - 31.0 31.0 - 30.6	WOR/WOR/WOR/WOR V8: Su=302/55 psf	WOR		21			Grey, wet, soft, clayey SILT. 65x130 mm vane raw torque readings: V8: 11.0/2.0 ft-lbs	
	V9		30.6 - 31.0	V9: Su=308/55 psf			19			65x130 mm vane raw torque readings: V9: 11.2/2.0 ft-lbs	
							17				
							18				
							19	-47.60		Failed 65x130 mm vane attempt.	
	80 MV	24/7	34.0 - 36.0 34.0 - 34.4	WOR/3/2/2 Could not push	5	5	19	-34.4		Grey, wet, loose, silty fine to medium SAND, little coarse sand, little coarse clay in layers.	
							25	-49.20			
							24	-36.0			
							44				
							32				
							31				
40	90	24/1	40.0 - 42.0	12/11/12/27	23	23	48			Grey, wet, medium dense, fine to medium SAND, little coarse sand, trace gravel, trace silt.	
							64				
							79				
							52				
							51				
							68				
45	100	24/4	45.0 - 47.0	16/10/8/10	18	18	45			Grey, wet, medium dense, fine to medium SAND, little coarse sand, trace gravel, trace silt.	
							48				
							62				
							55				
							68				
50	R1	60/60	50.2 - 55.2	RQD = 82%			ND ND	-63.70		DSD blows for 2.4". (Update from 50.1-50.5' bgs. (R1: Core Times (min:sec) 50.2-51.2 (2:28)	
								-60.5		Bedrock at Elev -63.70'	
								-68.40		BEDROCK: Dark grey with white infilling, medium to coarse grained, quartz-feldspar GNEISS, moderately hard, moderately weathered, steep to vertical foliation/jointing, close spacing. R1: Core Times (min:sec) 51.2-52.2 (2:33) 52.2-53.2 (2:42) 53.2-54.2 (2:45) 54.2-55.2 (3:17) 100% Recovery	
55								-65.2		Bottom of Exploration at 55.20 feet below ground surface.	

Remarks:
Barge to Ground 10.2' at 08:00, 8/24/05.
15.5' of Casing and Rods used before reaching ground.
WOC = Weight Of Casing

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

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Page 1 of 1
Boring No.: BB-BR.B00-101

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR 1263(000)X

PIN 12630.00

BRIDGE PLANS

BRIDGE NO. 2438

	BY	SIGNATURE	DATE
PROJ. MANAGER	W. FRANKHAUSER		
DESIGN-DETAILED	L. KRUSINSKI	T. WHITE	FEB 2008
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

P.E. NUMBER

DATE

KNICKERBOCKER BRIDGE
BACK RIVER
BOOTHBAY LINCOLN COUNTY

BORING LOGS

SHEET NUMBER
4
OF 7

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR 1263(000)X

BRIDGE NO. 2438 | PIN 12630.00 | BRIDGE PLANS

PROJ. MANAGER W. FRANKHAUSER	BY	DATE
DESIGN-DETAILED L. KRUSINSKI	T. WHITE	FEB 2008
CHECKED-REVIEWED	-------	-------
DESIGN2-DETAILED2	-------	-------
DESIGN3-DETAILED3	-------	-------
REVISIONS 1	-------	-------
REVISIONS 2	-------	-------
REVISIONS 3	-------	-------
REVISIONS 4	-------	-------
FIELD CHANGES	-------	-------

SIGNATURE

P.E. NUMBER

DATE

KNICKERBOCKER BRIDGE
BACK RIVER
BOOTHBAY LINCOLN COUNTY

BORING LOGS

SHEET NUMBER 5 OF 7

Filename: ...\msta\005_BORING LOGS2.dgn Division: GEOTECH Username: terry.white Date:2/8/2008

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

Project:	Knickerbocker Bridge #2438 over Back River	Location:	Boothbay, Maine	Boring No.:	BB-BBR-203
				PIN:	12630.00
Driller:	MaineDOT	Elevation (ft.)	-11.5	Auger ID/OD:	N/A
Operator:	E. Giguere/G. Lidstone	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder/L. Krusinski	Rig Type:	CME 45C on Barge	Hammer Wt./Fall:	CME 340 Auto Hammer/30"
Date Start/Finish:	8/23/07-8/23/07	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-1.88
Boring Location:	11+95, 0.1' Lt.	Casing ID/OD:	HW	Water Level*:	Back River (Tidal)
Hammer Efficiency Factor: 0.77		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

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
1D	24/13	0.0 - 2.0	WOR/WOR/WOR/WOR	WOR		WOC		Dark grey, wet, very soft, organic SILT, some fine sand, little clay trace gravel and shell fragments, organic odor, medium plastic. -Marine Deposit-	G#209956 CL-ML, A-4 WC=93.1%
						WOC			
						WOC			
						WOH			
						WOH			
						WOH			
						WOH			
						WOH			
2D V1	24/18	8.0 - 10.0 8.6 - 9.0	WOR/WOR/WOR/WOR Su=312/67 psf	WOR		WOH		Olive grey, wet (viscous), very soft, SILT, some clay, trace fine sand, trace shell fragments, medium to highly plastic. -Marine Deposit- 55x110 mm vane raw torque readings: V1: 7.0/1.5 ft-lbs 55x110 mm vane raw torque readings: V1: 9.0/2.0 ft-lbs	G#209957 CL-ML, A-4 WC=13.9%
V2		9.6 - 10.0	Su=402/89 psf			WOH			
						9			
						9			
						13	-24.50	13.0	
MV 3D	24/20	13.0 - 13.2 13.0 - 15.0	Could not push 2/3/3/3	6	8	31		Failed 55x110 mm vane attempt. Olive-grey, moist to wet, soft, silty CLAY, some fine sand, trace organic material fibers, medium plasticity, no dilatent -Glaciomarine Deposit-	
						39			
						35			
						32			
						23	-29.50	18.0	
4D V3	24/24	18.0 - 20.0 18.6 - 19.0	WOR/WOR/WOR/WOR Su=357/89 psf	WOR		16		Dark grey, wet, very soft, clayey SILT, trace sand, high plasticity, homogeneous. - Glaciomarine Deposit - 55x110 mm vane raw torque readings: V3: 8.0/2.0 ft-lbs 55x110 mm vane raw torque readings: V4: 10.0/2.0 ft-lbs	
V4		19.6 - 20.0	Su=446/89 psf			20			
						23			
						21			
						18			
5D V5	24/28	23.0 - 25.0 23.6 - 24.0	WOR/WOR/WOR/WOR Su=439/27 psf	WOR		21		Same as 4D. 65x130 mm vane raw torque readings: V5: 16.0/1.0 ft-lbs 65x130 mm vane raw torque readings: V6: 18.2/2.0 ft-lbs	G#209958 CL, A-6 LL=37 PL=25 PI=12 WC=44.5%
V6		24.6 - 25.0	Su=500/55 psf			23			
						24			
						22			
						19			
6D V7	24/14	28.0 - 30.0 28.6 - 29.0	WOR/WOR/WOR/WOR Su=426/41 psf	WOR		WOC		Grey, wet, very soft, clayey SILT, high plasticity. - Glaciomarine Deposit - Casing broke off at 15.0' bgs, pulled back with casing retriever. 65x130 mm vane raw torque readings: V7: 15.5/1.5 ft-lbs 65x130 mm vane raw torque readings: V6: 18.2/2.0 ft-lbs	
V8		29.6 - 30.0	Su=500/55 psf			WOC			
						WOC			
						WOH			
						WOH			
7D V9	24/24	33.0 - 35.0 33.6 - 34.0	WOR/WOR/WOR/WOR Su=481/55 psf	WOR		11		Same as 6D. 65x130 mm vane raw torque readings: V9: 17.5/2.0 ft-lbs 65x130 mm vane raw torque readings: V10: 19.0/2.2 ft-lbs	
V10		34.6 - 35.0	Su=522/60 psf			10			
						11			
						11			
						11			
8D V11	24/24	38.0 - 40.0 38.6 - 39.0	WOR/WOR/WOR/WOR Su=549/77 psf	WOR		12		Same as 7D. 65x130 mm vane raw torque readings: V11: 20.0/2.8 ft-lbs 65x130 mm vane raw torque readings: V12: 21.5/2.8 ft-lbs	
V12		39.6 - 40.0	Su=590/77 psf			11			
						13			
						13			
						16			
9D V13	24/24	43.0 - 45.0 43.6 - 44.0	WOR/WOR/WOR/WOR Su=467/110 psf	WOR		11		Same as 8D, 2 cup sample. 65x130 mm vane raw torque readings: V13: 17.0/4.0 ft-lbs 65x130 mm vane raw torque readings: V14: 17.5/4.5 ft-lbs	G#209955 CL, A-6 LL=35 PL=11 PI=24 WC=46.6%
V14		44.6 - 45.0	Su=481/124 psf			12			
						19			
						18			
						20			
10D	24/24	48.0 - 50.0	WOR/WOR/WOR/WOR	WOR		26		Similar to 9D, skipped vane testing.	
						26			

Remarks:

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Page 1 of 2 — Boring No.: BB-BBR-203

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

Project:	Knickerbocker Bridge #2438 over Back River	Location:	Boothbay, Maine	Boring No.:	BB-BBR-203
				PIN:	12630.00
Driller:	MaineDOT	Elevation (ft.)	-11.5	Auger ID/OD:	N/A
Operator:	E. Giguere/G. Lidstone	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder/L. Krusinski	Rig Type:	CME 45C on Barge	Hammer Wt./Fall:	CME 340 Auto Hammer/30"
Date Start/Finish:	8/23/07-8/23/07	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-1.88
Boring Location:	11+95, 0.1' Lt.	Casing ID/OD:	HW	Water Level*:	Back River (Tidal)
Hammer Efficiency Factor: 0.77		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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			
						26			
						26			
11D V15	24/24	53.0 - 55.0 53.6 - 54.0	WOR/WOR/WOR/WOH Su=632/115 psf	WOR		16		Grey, wet, very soft, sandy SILT, little clay. 65x130 mm vane raw torque readings: V15: 23.0/4.2 ft-lbs Failed 65x130 mm vane attempt.	G#209961 CL-ML, A-4 WC=26.1%
MV		54.0 - 54.2	Could not push			15	-66.00	54.5 Grey, wet, soft, silty fine SAND, little to trace clay.	
						59			
						107			
						62			
12D	24/22	58.0 - 60.0	9/12/5/5	17	22	44		Grey, wet, medium dense, fine to coarse SAND, little fine gravel, rounded to subrounded, some layered silt.	
						25			
						27			
						62			
						23	-74.50	63.0	
13D	24/16	63.0 - 65.0	6/5/4/5	9	12	25		Grey, wet, medium dense, fine gravelly, rounded to subrounded, medium to coarse SAND, some fine sand, trace silt. 20-foot section of 4-in HW steel casing broke off when pulling casing, and abandoned in drillhole between approximately elev. -60 and -80 ft.	
						35			
						46			
						34			
						19		Grey, wet, very dense, sandy fine GRAVEL with two 1" dia. rounded rock fragments, (white gneiss).	
14D R1	7.2/6 60/60	68.0 - 68.6 68.6 - 73.6	31/30(1.2")	---		75 NQ	-80.10	68.6 R1: 3.6' BOULDER, white and grey, QUARTZ-FELDSPAR GNEISS. R1: Core Times (min:sec) 68.6-69.6' (4:00) 69.6-70.6' (3:30) 70.6-71.6' (3:20) 71.6-72.6' (1:30) 72.6-73.6' (0:30) 72% Recovery	
							-83.70	72.2	
						SPUN		Pulled core barrel, retrieved R1. Spun core barrel ahead from 73.6-81.6' bgs. Two 0.1' rounded rock fragments picked up when advancing core barrel from bottom boulder elevation to top of bedrock.	
						NQ			
R2	60/60	81.6 - 86.6	RQD = 35%			NQ	-93.10	81.6 Top of Bedrock at Elev. -93.10' R2: Bedrock: Grey and white, banded, quartz BIOTITE SCHIST, interspersed with medium grained GNEISS banding, fine to medium grained, soft to medium hard, moderately fractured, moderately weathered overall with intermintant soft, highly weathered banding. Joints chaotic, 1"-6" spacing, tight to open, soft to fresh, with loose biotite, soft infilling to no infilling. Correlation of RQD to rock quality: Poor. R2: Core Times (min:sec) 81.6-82.6' (4:00) 82.6-83.6' (3:30) 83.6-84.6' (4:20) 84.6-85.6' (4:45) 85.6-86.6' (4:30) 100% Recovery	
							-98.10	86.6 Bottom of Exploration at 86.60 feet below ground surface.	

Remarks:

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

Page 2 of 2 — Boring No.: BB-BBR-203

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

Project:	Knickerbocker Bridge #2438 over Back River	Location:	Boothbay, Maine	Boring No.:	BB-BR.BOO-102
				PIN:	12630.00
Driller:	MaineDOT	Elevation (ft.)	-16.9	Auger ID/OD:	N/A
Operator:	E. Giguere	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	G. Lidstone	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/23/05; 10:30-17:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	12+79.7, 58.6' Lt.	Casing ID/OD:	HW	Water Level*:	Tidal
Hammer Efficiency Factor: 0.60		Hammer Type:	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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
1D	24/5	0.0 - 2.0	WOR/WOR/WOR/WOR	WOR		WOC		Dark grey, wet, very soft, organic SILT, little fine sand, trace coarse sand, trace shells. -MARINE DEPOSIT-	
						WOC		Dark grey, wet, very soft, organic SILT, little fine sand, trace shells.	
						WOC			
						WOH			
						WOH	-21.90	5.0	
						3			
						3		Grey and brown, soft, SILT, trace clay.	
2D V1	24/14	7.0 - 9.0 7.6 - 8.0	WOH/1/3/4 Su=670/143 psf	4	4	5		55x110 mm vane raw torque readings: V1: 15.0/3.2 ft-lbs	
						14			
						18	-26.90	10.0	
						16		-GLACIOMARINE DEPOSIT-	
						15			
3D V2	24/19	12.0 - 14.0 12.6 - 13.0	WOR/WOR/WOR/WOR V2: Su=402/76 psf	WOR		15		Grey, wet, soft, clayey SILT, trace sand, with black staining. 55x110 mm vane raw torque readings: V2: 9.0/1.7 ft-lbs 55x110 mm vane raw torque readings: V3: 7.7/0.5 ft-lbs	G#181936 A-6, CL WC=41.7% LL=32 PL=19 PI=13
V3		13.6 - 14.0	V3: Su=344/22 psf			16			
						14			
4D V4	24/7	15.0 - 17.0 15.6 - 16.0	WOR/WOR/WOR/WOH V4: Su=426/69 psf	WOR		19		Grey, wet, soft, clayey SILT. 65x130 mm vane raw torque readings: V4: 15.5/2.5 ft-lbs 65x130 mm vane raw torque readings: V5: 13.2/2.0 ft-lbs	
V5		16.6 - 17.0	V5: Su=363/55 psf			25			
						21			
						18			
						17			
5D V6	24/18	20.0 - 22.0 20.6 - 21.0	WOR/WOR/WOR/WOR V6: Su=137/60 psf	WOR		27		Grey, wet, very soft, clayey SILT, trace sand. 65x130 mm vane raw torque readings: V6: 5.0/2.2 ft-lbs 65x130 mm vane raw torque readings: V7: 9.0/2.5 ft-lbs	G#181937 A-4, CL WC=42.8 LL=31 PL=22 PI=9
V7		21.6 - 22.0	V7: Su=247/69 psf			25			
						20			
						20	-40.90	24.0	
6D/AB	24/13	24.0 - 26.0	WOR/1/9/6	10	10	19	-42.00	Roller Coned ahead from 24.0-24.8' bgs. (6D/A) 24.0-25.1' bgs. Grey, wet, very soft, clayey SILT. 25.1 (6D/B) 25.1-26.0' bgs. Grey, wet, loose, silty fine SAND with clay layers, trace gravel. 26.0	
						23	-42.90 -43.30		
R1	60/56	26.6 - 31.6	RQD = 62%			a222 NQ	-43.50	a222 blows for 7.2". Grey, wet, fine to coarse SAND, little gravel. 26.4 Weathered ROCK. 26.6 Bedrock at Elev. -43.50' BEDROCK: White, medium to coarse grained, quartz-feldspar GNEISS, hard, moderately weathered, low-angle foliation/jointing, very close spacing. R1:Core Times (min:sec) 26.6-27.6 (2:55) 27.6-28.6 (3:25) 28.6-29.6 (2:45) 29.6-30.6 (3:00) 30.6-31.6 (2:55) 93% Recovery	
R2	58.8/56	31.6 - 36.5	RQD = 90%				-48.50	31.6 R2:Core Times (min:sec) 31.6-32.6 (2:15) 32.6-33.6 (2:05) 33.6-34.6 (3:03) 34.6-35.6 (3:09) 35.6-36.5 (3:23) 95% Recovery	
							-53.40	36.5 Bottom of Exploration at 36.50 feet below ground surface.	

Remarks:

Barge to Ground 14.5' at 10:35, 8/23/05.
15.5' of Casing and Rods used before reaching ground.
WOC = Weight Of Casing

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Page 1 of 1 — Boring No.: BB-BR.BOO-102

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

Project:	Knickerbocker Bridge #2438 over Back River	Location:	Boothbay, Maine	Boring No.:	BB-BBR-204
				PIN:	12630.00
Driller:	MaineDOT	Elevation (ft.)	-15.6	Auger ID/OD:	N/A
Operator:	E. Giguere/G. Lidstone	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder/L. Krusinski	Rig Type:	CME 45C on Barge	Hammer Wt./Fall:	CME 340 Auto Hammer/30"
Date Start/Finish:	9/5/07-9/5/07	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-1.88
Boring Location:	12+86.8, 3.9' Rt.	Casing ID/OD:	HW	Water Level*:	Back River (Tidal)
Hammer Efficiency Factor: .77		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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
						WOC			
						WOC			
						WOC			
						WOC			
						WOH			
						WOH			
						4			
1D	24/14	7.0 - 9.0	WOH/WOH/WOH/WOH	WOH		4		Grey, wet, very soft, organic SILT, some fine sand, little clay, little gravel, trace shell fragments.	G#209966 CL-ML, A-4 WC=39.6%
						4			
						6	-25.60	10.0	
						20			
						27			
						21			
2D V1	24/20	13.0 - 15.0 13.6 - 14.0	WOR/WOR/WOR/WOR Su=402/89 psf	WOR		aHP		aHydraulic Push Dark grey, wet, very soft, clayey SILT, trace sand, trace gravel, medium to high plasticity, viscous. -Glaciomarine Deposit- 55x110 mm vane raw torque readings: V1: 9.0/2.0 ft-lbs 55x110 mm vane raw torque readings: V2: 11.0/3.0 ft-lbs	
V2		14.6 - 15.0	Su=491/134 psf			aHP			
						5			
						21			
						22			
						22			
3D	24/24	19.0 - 21.0	WOR/WOR/WOR/WOR	WOR		6		Similar to 2D, except grey. - Glaciomarine Deposit -	G#209967 CL, A-6 LL=27 PL=14 PI=13 WC=36.0%
						6			
						29		b100 blows for 0.6'.	
R1	60/60	22.6 - 27.6	RQD = 83%			b100 NQ	-38.20	22.6 Top of Bedrock at Elev. -38.20' R1:Bedrock: White, fine to medium grained GNEISS, hard, fresh, discontinuties widely spaced, tight, fresh, changing at 2'6" to coarse grained, white, quartz feldspar PEGMATITE, hard, fresh, massive. Correlation of RQD to rock quality: good. R1:Core Times (min:sec) 22.6-23.6' (5:00) 23.6-24.6' (4:30) 24.6-25.6' (4:20) 25.6-26.6' (4:30) 26.6-27.6' (4:10) 100% Recovery	
R2	60/60	27.6 - 32.6	RQD = 100%				-43.20	27.6 R2: Bedrock: White, hard, coarse grained quartz feldspar PEGMATITE, fresh, massive. Correlation of RQD to rock quality: excellent. R2: Core Times (min:sec) 27.6-28.6' (2:43) 28.6-29.6' (2:28) 29.6-30.6' (2:33) 30.6-31.6' (2:54) 31.6-32.6' (2:50) 100% Recovery	
							-48.20	32.6 Bottom of Exploration at 32.60 feet below ground surface.	

Remarks:

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

Page 1 of 1 — Boring No.: BB-BBR-204

Filename: ...\msta\006_BORING LOGS3.dgn Division: GEOTECH Username: terry.white Date:2/8/2008

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

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine
Boring No.: BB-BBR-205
PIN: 12630.00

Driller:	MaineDOT	Elevation (ft.)	-11.4	Auger ID/OD:	N/A
Operator:	E. Giguere/G. Lidstone	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder/L. Krusinski	Rig Type:	CME 45C on Barge	Hammer Wt./Fall:	CME 340 Auto Hammer/30"
Date Start/Finish:	8/28/07-8/29/07	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-1.88
Boring Location:	13+77.9, 0.5' Rt.	Casing ID/OD:	HW	Water Level*:	Back River (Tidal)
Hammer Efficiency Factor:	.77	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							WOC			
							WOC			
							WOC			
							WOC			
							WOC			
5	1D	24/10	5.0 - 7.0	WOR/WOR/WOR/WOR	WOR		WOH		Grey, wet, very soft, organic SILT, some fine sand, little clay, trace gravel, organic fibers, sea shell fragments. -Marine Sediments-	G#209960 CL-ML, A-4 WC=53.9%
							6			
							a3/H		a3 blows for 6"/Hydraulic Push 6"	
							bH/2	-19.40	8.0 bHydraulic Push 6"/25 blows for 6"	
							86			
10	2D	24/24	10.0 - 12.0	Hydraulic Push	---		87		Grey, moist, very soft, clayey SILT, trace fine sand, intermittent brown mottling, blocky, stiff tactile, non to low plasticity. -Glaciomarine Deposit-	G#209959 CL, A-6 LL=33 PL=15 PI=18 WC=23.4%
							101			
							105			
							115			
							117			
15	3D V1	24/24	15.0 - 17.0 15.6 - 16.0	WOH/WOH/WOH/WOH Su=1250/179 psf	WOH		82	-26.40	15.0 Grey, wet, very soft, clayey SILT, medium plasticity -Glaciomarine Deposit- 55x110 mm vane raw torque readings: V1: 28.0/4.0 ft-lbs 55x110 mm vane raw torque readings: V2: 12.8/2.0 ft-lbs	
	V2		16.6 - 17.0	Su=571/89 psf			61			
							60			
							51			
							48			
20	4D	24/24	20.0 - 22.0	WOR/WOR/WOR/WOR	WOR		60		Dark grey, wet, very soft, CLAY SILT, medium to high plasticity. -Glaciomarine Deposit-	
							44			
							36			
							30			
	5D V3	24/24	24.0 - 26.0 24.6 - 25.0	WOR/WOR/WOR/WOR Su=379/98 psf	WOR		27		Grey, wet, very soft, clayey SILT, highly plastic. 55x110 mm vane raw torque readings: V3: 8.5/2.2 ft-lbs 55x110 mm vane raw torque readings: V4: 8.0/2.0 ft-lbs	
25	V4		25.6 - 26.0	Su=357/89 psf			22			
							21			
							21			
	6D	24/24	28.0 - 30.0	WOR/WOR/WOR/WOR	WOR		21		Grey, wet, very soft, clayey SILT, trace sand, trace gravel.	G#209962 CL-ML, A-4 LL=25 PL=20 PI=5 WC=36.6%
							19			
30							43	-41.40	30.0	
							34			
							33			
	7D	24/12	33.0 - 35.0	4/14/14/6	28	36	38		Grey, wet, dense, fine SAND, some fine gravel, rounded to subangular, little silt.	G#209963 SM, A-1-b WC=10.6%
							55			
35	R1	60/57	35.9 - 40.9	RQD = 57%			a60 NQ	-47.30	a60 blows for 0.9'. 35.9 Top of Bedrock @ Elev. -47.30' R1:Bedrock: Grey, moderately hard, banded, fine to medium grained BIOTITE SCHIST, very slightly weathered, discontinuities at horiz. to steep angles, close spacing, tight to open, trace of silt and sand infilling, slightly weathered surface, moderately fractured. Correlation of RQD to rock quality: Good. R1:Core Times (min:sec) 35.9-36.9' (5:30) 36.9-37.9' (5:00) 37.9-38.9' (4:00) 38.9-39.9' (4:30) 39.9-40.9' (4:20) 95% Recovery	
40	R2	60/60	40.9 - 45.9	RQD = 80%				-52.30	40.9 R2:Bedrock: Grey, fine to medium grained BIOTITE SCHIST, fresh to slightly weathered, jointing chaotic, horiz. to steep (visible in lower) along and perpendicular to banding closely spaced, trace silt infilling, slightly fractured. Correlation of RQD to rock quality: Good. R2:Core Times (min:sec) 40.9-41.9' (3:15) 41.9-42.9' (2:30) 42.9-43.9' (2:18) 43.9-44.9' (2:05) 44.9-45.9' (2:05) 100% Recovery	
45								-57.30	45.9 Bottom of Exploration at 45.90 feet below ground surface.	
50										

Remarks:

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

Page 1 of 1
Boring No.: BB-BBR-205

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

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine
Boring No.: BB-BR.BOO-103
PIN: 12630.00

Driller:	MaineDOT	Elevation (ft.)	-7.6	Auger ID/OD:	N/A
Operator:	E. Giguere	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	G. Lidstone	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/22/05-8/23/05	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	14+17.9, 57.5' Lt.	Casing ID/OD:	HW	Water Level*:	Tidal
Hammer Efficiency Factor:	0.60	Hammer Type:	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/9	0.0 - 2.0	WOR/WOR/WOR/WOR	WOR		WOC		Dark grey, wet, very soft, organic SILT, trace fine sand, trace shells. -MARINE DEPOSIT-	
							WOC			
							WOC			
							WOH			
							WOH			
5							1			
	2D/AB MV	24/17	6.0 - 8.0 6.6 - 7.0	WOR/WOR/WOH/5 aSu=>1217 psf	WOH		5		(2D/A) 6.0-7.5' bgs. Grey, wet, very soft, clayey SILT, trace fine sand layers, trace gravel, trace shells. 65x130 mm vane raw torque readings: aMV: >44.3 ft-lbs (probable sand seam)	G#181938 A-4, CL WC=26.8% LL=24 PL=16 PI=8
							15	-15.10	7.5 (2D/B) 7.5-8.0' bgs. Grey-brown, moist, very soft, clayey SILT, trace sand.	G#181939 A-4, CL WC=23.6%
							31			
							36			
10							37			
	3D	24/22	11.0 - 13.0	1/3/3/4	6	6	60		Grey-brown, moist, loose, fine SAND, some silt, trace clay.	
							55			
							50			
							50			
15							47			
	V1 4D	24/24	16.0 - 16.1 16.4 - 18.4	bSu=2201/1100 psf WOR/WOH/WOH/WOH	WOH		30	-24.00	16x32 mm vane raw torque readings: bV1: 14.0/7.0 in-lbs (vane in fine sand) 16.4 Grey, wet, very soft, clayey SILT with black staining. -GLACIOMARINE DEPOSIT-	
							28			
							19			
							17			
20							14			
	5D V2	24/24	21.0 - 23.0 21.6 - 22.0	WOR/WOR/WOR/WOR V2: Su=439/74 psf	WOR		19		Grey, wet, very soft, silty CLAY, trace sand. 65x130 mm vane raw torque readings: V2: 16.0/2.7 ft-lbs 65x130 mm vane raw torque readings: V3: 15.0/2.5 ft-lbs	G#181940 A-6, CL WC=43.0% LL=34 PL=22 PI=12
	V3		22.6 - 23.0	V3: Su=412/69 psf			16			
							18			
							18			
25							20			
	6D V4	24/24	26.0 - 28.0 26.6 - 27.0	WOR/WOR/WOR/WOR V4: Su=439/88 psf	WOR		24		Grey, wet, very soft, clayey SILT. 65x130 mm vane raw torque readings: V4: 16.0/3.2 ft-lbs 65x130 mm vane raw torque readings: V5: 14.2/2.0 ft-lbs	
	V5		27.6 - 28.0	V5: Su=390/55 psf			20			
							16			
							16			
30							17			
	7D	24/1	31.0 - 33.0	18/13/10/11	23	23	68	-38.60	31.0 Grey, wet, medium dense, fine to coarse SAND, little gravel, trace silt.	
							42			
							48			
							77			
35							105			
							159	-44.50 -44.60	36.9 Weathered ROCK. 37.0	
	R1	60/57	37.0 - 42.0	RQD = 53%			NQ		Bedrock at Elev. -44.60' BEDROCK: Dark grey with white infilling, medium to coarse grained, quartz-feldspar GNEISS, moderately hard, moderately weathered, steep to vertical foliation, low angle jointing, close spacing. FAIR QUALITY R1:Core Times (min:sec) 37.0-38.0 (3:25) 38.0-39.0 (2:25) 39.0-40.0 (2:30) 40.0-41.0 (3:20) 41.0-42.0 (3:35) 95% Recovery	
40										
	R2	60/56	42.0 - 47.0	RQD = 67%				-49.60	42.0 R2:Core Times (min:sec) 42.0-43.0 (2:45) 43.0-44.0 (3:05) 44.0-45.0 (3:15) 45.0-46.0 (3:30) 46.0-47.0 (3:20) 93% Recovery	
45								-54.60	47.0 Bottom of Exploration at 47.00 feet below ground surface.	
50										

Remarks:
Barge to Ground 12.2' at 11:35, 8/22/05.
15.5' of Casing and Rods used before reaching ground.
WOC = Weight Of Casing
8/22/05; 11:30-17:00, 8/23/05; 07:30-10:00

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.

Page 1 of 1
Boring No.: BB-BR.BOO-103

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

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine
Boring No.: BB-BBR-206
PIN: 12630.00

Driller:	MaineDOT	Elevation (ft.)	-5.9	Auger ID/OD:	N/A
Operator:	E. Giguere/G. Lidstone	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder/L. Krusinski	Rig Type:	CME 45C on Barge	Hammer Wt./Fall:	CME 340 Auto Hammer/30"
Date Start/Finish:	8/29/07-8/29/07	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-1.88
Boring Location:	14+68.1, 2.5' Rt.	Casing ID/OD:	HW	Water Level*:	Back River (Tidal)
Hammer Efficiency Factor:	.77	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/14	0.0 - 2.0	WOR/WOR/WOR/WOR	WOR		WOC		Olive grey, wet, very soft, clayey organic SILT, trace shell fragments.	
							WOC			
							WOC			
							WOH	-9.90	4.0	
							3			
5	2D	24/16	5.0 - 7.0	WOR/WOR/WOR/WOR	WOR		3		Grey, wet, very soft, SILT, some fine sand, trace clay, trace fine gravel, trace shell fragments.	G#209964 CL-ML, A-4 WC=44.2%
							10			
							29			
							43	-13.90	8.0	
	3D	24/22	9.0 - 11.0	3/3/3/5	6	8	41		Grey and brown, moist, mottled, medium stiff, silty CLAY, trace of sand, shell fragments, blocky, stiff tactilely. -Glaciomarine Deposit- Failed 55x110 mm vane attempt.	
10	MV		10.6 - 10.6	Could not push			40			
							66			
							59			
							81			
	4D MV	24/24	14.0 - 16.0 14.6 - 14.6	2/3/3/3 Could not push	6	8	94		Grey and brown, moist, mottled, medium stiff silty CLAY, trace sand, blocky. Failed 55x110 mm vane attempt. PP=1.1 tsf, 1.5 tsf	G#209965 CL, A-6 LL=37 PL=23 PI=14 WC=31.2%
15							89			
							90	-21.90	16.0	
							79			
							79			
	5D	24/24	19.0 - 21.0	WOR/WOR/WOR/WOR	WOR		62		Dark grey, wet, very soft, marine silty CLAY, highly plasticity. PP=0 tsf	
20							55			
							55			
							47			
							38			
							32			
25	6D/A	24/24	25.0 - 27.0	WOR/WOR/WOR/4	WOR		45		(6D/A) 25.0-26.6' bgs. Similar to above.	
							66	-32.50	26.6 (6D/B) 26.6-28.4' bgs. Grey, wet, soft, silty fine SAND, little clay.	
							117		a200 blows for 0.4'.	
	R1	60/60	28.4 - 33.4	RQD = 50%			a200 NQ	-34.30	28.4 Top of bedrock at Elev. -34.30' R1: Bedrock: Beige, fine to medium grained, quartz feldspar GNEISS, moderately hard, moderately to highly weathered, quartz is discolored and stained, feldspar broken down, jointing chaotic, horiz. to steep, close, open, sandy silt infilling, discolored, oxidized surfaces, moderately fractured. Rock Quality Poor R1:Core Times (min:sec) 28.4-29.4' (2:55) 29.4-30.4' (2:23) 30.4-31.4' (3:48) 31.4-32.4' (2:18) 32.4-33.4' (2:18) 100% Recovery	
30										
	R2	60/58	33.4 - 38.4	RQD = 48%				-39.30	33.4 R2:Bedrock: Upper 2'2' fractured, discolored, stained GNEISS. White to grey, discolored, stained, fine to medium grained, quartz feldspar GNEISS, moderately hard, moderately weathered to slightly weathered, joints along banding at chaotic angles, surfaces open, discolored, stained, decomposed feldspar, very close to close, very weathered biotite schist band 3'9"-4'2". R2:Core Times (min:sec) 33.4-34.4' (3:00) 34.4-35.4' (2:30) 35.4-36.4' (2:10) 36.4-37.4' (2:05) 37.4-38.4' (3:00) 92% Recovery	
35										
								-44.30	38.4 Bottom of Exploration at 38.40 feet below ground surface.	
40										
45										
50										

Remarks:
PP = Pocket Penetrometer

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.

Page 1 of 1
Boring No.: BB-BBR-206

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

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine
Boring No.: BB-BBR-207
PIN: 12630.00

Driller:	MaineDOT	Elevation (ft.)		Auger ID/OD:	N/A
Operator:	E. Giguere/G. Lidstone	Datum:	NAVD 88	Sampler:	N/A
Logged By:	B. Wilder/L. Krusinski	Rig Type:	CME 45C on Barge	Hammer Wt./Fall:	CME 340 Auto Hammer/30"
Date Start/Finish:	9/4/07-9/4/07	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	15+42.1, 13.1' Rt.	Casing ID/OD:	N/A	Water Level*:	Back River (Tidal)
Hammer Efficiency Factor:	.77	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0									Bottom of Exploration at 0.00 feet below ground surface. Cobble or boulder on surface, relocated to BB-BBR-207A.	
5										
10										
15										
20										
25										

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.

Page 1 of 1
Boring No.: BB-BBR-207

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

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine
Boring No.: BB-BBR-207A
PIN: 12630.00

Driller:	MaineDOT	Elevation (ft.)	-3.52	Auger ID/OD:	N/A
Operator:	E. Giguere/G. Lidstone	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder/L. Krusinski	Rig Type:	CME 45C on Barge	Hammer Wt./Fall:	CME 340 Auto Hammer/30"
Date Start/Finish:	9/4/07-9/4/07	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-1.88
Boring Location:	15+40.1, 14.4' Rt.	Casing ID/OD:	HW	Water Level*:	Back River (Tidal)
Hammer Efficiency Factor:	.77	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							aHYD		aHydraulic Push Tidal flat sediments and boulders.	
							3			
							3		b9 blows for 0.5', then no movement.	
	R1	60/60	3.5 - 8.5	RQD = 38%			b9 NQ	-7.02	3.5 Top of Bedrock at Elev. -7.02' R1: Bedrock: White, discolored and stained, fine to medium grained GNEISS, moderately hard, moderately weathered, discontinuities horizontal to steep, tight, discolored, stained, highly fractured. Correlation of RQD to rock quality: poor. R1:Core Times (min:sec) 500 psi 3.5-4.5' (3:26) 4.5-5.5' (2:36) 5.5-6.5' (2:52) 6.5-7.5' (2:38) lost water at 3.5' into run 7.5-8.5' (2:11) 100% Recovery	
5										
	R2	60/59	8.5 - 13.5	RQD = 73%				-12.02	8.5 R2: Bedrock: Same as R1, except moderately to slightly weathered, slightly fractured. Correlation of RQD to rock quality: fair. R2:Core Times (min:sec) 500 psi 8.5-9.5' (2:36) 9.5-10.5' (2:10) 10.5-11.5' (2:00) 11.5-12.5' (2:27) 12.5-13.5' (3:05) 98% Recovery	
10										
								-17.02	13.5 Bottom of Exploration at 13.50 feet below ground surface.	
15										
20										
25										

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.

Page 1 of 1
Boring No.: BB-BBR-207A

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
BR 1263(000)X
PIN 12630.00
BRIDGE NO. 2438
BRIDGE PLANS

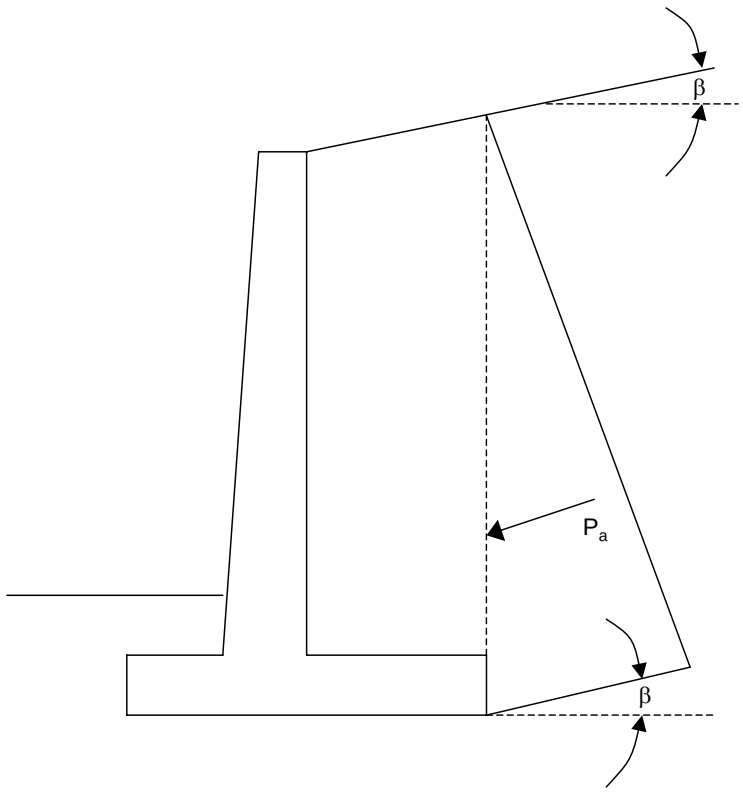
	BY	DATE
PROJ. MANAGER W. FRANKHAUSER		
DESIGN-DETAILED L. KRUSINSKI	T. WHITE	FEB 2008
CHECKED-REVIEWED		
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

KNICKERBOCKER BRIDGE
BACK RIVER
BOOTHBAY LINCOLN COUNTY

BORING LOGS

SHEET NUMBER
6
OF 7



For cases where interface friction between the backfill and wall are 0 or not considered, use Rankine.

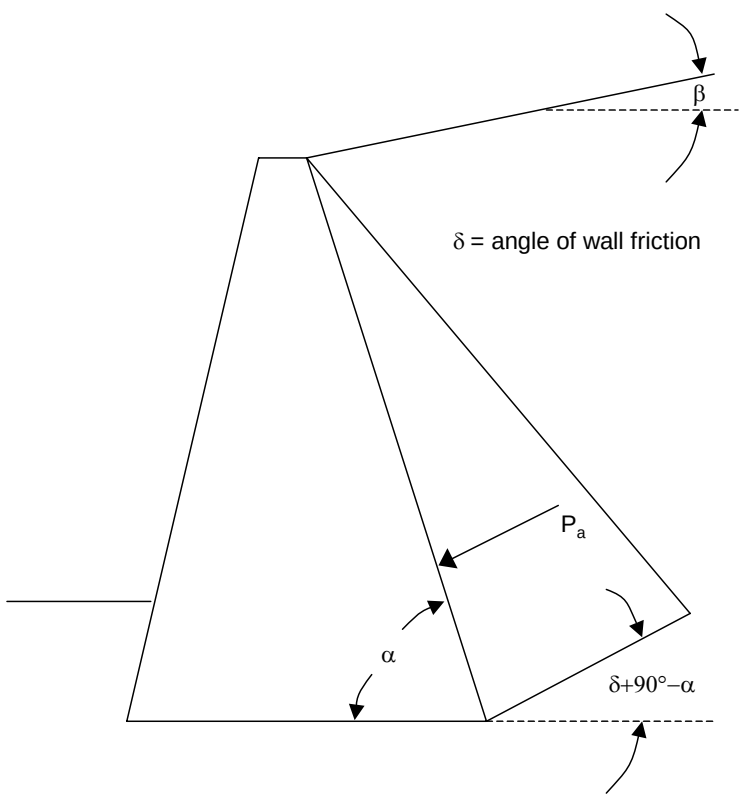
For a horizontal backfill surface, $\beta = 0°$:

$$K_a = \tan^2\left(45° - \frac{\phi}{2}\right)$$

For a sloped backfill surface, $\beta > 0°$:

$$K_a = \cos\beta * \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

P_a is oriented at β



For cases where interface friction is considered, use Coulomb.

For horizontal or sloped backfill surfaces:

$$K_a = \frac{\sin^2(\alpha + \phi)}{\sin^2\alpha * \sin(\alpha - \delta) * \left(1 + \sqrt{\frac{\sin(\phi + \delta) * \sin(\phi - \beta)}{\sin(\alpha - \delta) * \sin(\beta + \alpha)}}\right)^2}$$

P_a is oriented at $\delta + 90° - \alpha$

Rankine and Coulomb Active Earth Pressure Coefficients

Appendix A

Geotechnical Design Memorandum

STATE OF MAINE

DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM - GEOTECHNICAL SECTION
16 STATE HOUSE STATION
AUGUSTA, ME 04333-0016

JEFF TWEEDIE, P.E.
PH: 207.624.3427
FAX: 207.624.3491
jeff.tweedie@maine.gov

INTEROFFICE MEMORANDUM

TO: WAYNE FRANKHAUSER

FROM: JEFF TWEEDIE

SUBJECT: BOOTHBAY, KNICKERBOCKER BRIDGE (12630.00): PRELIMINARY GEOTECHNICAL RECOMMENDATIONS

DATE: NOVEMBER 1, 2005

CC: FILE

A preliminary geotechnical evaluation has been completed for the proposed replacement of Knickerbocker Bridge. The project is located on Barters Island Road, spanning Back River, in the town of Boothbay. This preliminary geotechnical evaluation is based on a site visit, available geologic information, three preliminary borings, and existing plans. Once a replacement structure and alignment are developed, we will examine the requirements for further subsurface explorations, and final geotechnical design/report. This evaluation was completed in general conformance with the Bridge Design Guide (BDG) and considers only replacement.

Summary of Available Information. Knickerbocker Bridge spans Back River, and connects Barters Island to the mainland. The flow of Back River in the area of the bridge is tidal. The Surficial Geology of the Westport Quadrangle, Maine (Smith, Maine Geologic Survey) indicates the soils in the area of the bridge consist of glacial-marine deposits. The glacial-marine deposits consist primarily of silt and clay, which are characterized by their low strength and high compressibility. The glacial-marine deposit originated from sediments washed from the late Wisconsinan glacier, which were deposited on the sea floor during the most recent marine submergence. The Bedrock Geology Map of Maine (Osberg, Hussey, and Boone, Maine Geological Survey), indicate the bedrock to be on the margin of the Cape Elizabeth and Bucksport Formations. The Cape Elizabeth Formation consists of an interbedded pelite and sandstone. The Bucksport formation consists of calcareous sandstone, interbedded sandstone, and impure limestone.

The existing structure was constructed in 1930, and consists of 36, 14-ft spans and one 28-ft span. The structure is constructed from timber stringers, supported by timber pile bent piers. The bearing stratum for the piles is unknown, however, is presumed to be bedrock due to the soft soils encountered during preliminary boring operations. The as-built plans show the abutments are a spill-through configuration, supported on timber piles, with an anchor and tie-back system for resisting lateral earth loads.

Three preliminary borings, BB-BR.BOO-101, BB-BR.BOO-102, and BB-BR.BOO-103, were completed within the river channel, along the northern facia of the structure, as shown on the boring location plan and interpretive subsurface profile included with this correspondence. The borings show very loose river and marine sediments for the top 10.0 to 16.4 ft. These sediments are underlain with 14.6 to 20.4 ft of glacial-marine deposit, consisting of very soft to soft silt. The glacial-marine deposit is underlain with 1.3 to 16.1 ft of loose to medium dense sand. The depth to top of bedrock varied over the site, ranging from 26.4 ft (elev. -43.3 ft) to 50.5 ft (elev. -63.7 ft). A generalized interpretive subsurface profile and a copy of the boring logs are included with this correspondence.

Results of a site visit show apparent bedrock outcrops located within the approaches to the abutments. However, a shallow bedrock depth in this area is not supported by the as-built plans, as abutment support is provided by timber piles.

Preliminary Recommendations. Due to the lack of expansion joints, Integral Abutment Bridges (IAB) are generally preferred by the Bridge Program. However, due to the significant structure length, > 500 ft, an integral structure is not feasible. Feasible substructure alternatives are discussed as follows:

Abutments. For abutment support the designer may consider full height abutments, either cantilever or gravity, founded on a spread footing on bedrock or H-piles. Alternately, the designer may consider a spill-through abutment configuration, constructed from pipe piles driven to bedrock, or drilled shafts socketed into bedrock. For the spill-through abutment configuration with pipe piles, sliding and overturning forces may be resisted by socketing the pipe piles into bedrock with a small H-pile section, or with rock anchors. As noted above, the depth to rock in the area of the abutments is not certain, as a result, we recommend the designer consider the full-height abutment alternative founded on H-piles for development of the Preliminary Design Report (PDR). Adjustments to the PDR can be made, as necessary, after a final alignment is chosen and final borings are completed.

Piers. For intermediate structure support, alternatives that do not require the additional cost associated with a cofferdam are generally preferred by the Bridge Program. Due to the project being located in a tidal area, pile bent piers, constructed from concrete filled steel pipe piles, are optimal, and are recommended. Vessel impact from small boats (i.e. lobster boats) should be considered during design. Depending on the alignment and location of the navigable channel, the depth to bedrock may be shallow, and adequate lateral capacity may not be provided by the native soils. As a result, we recommend that the pipe piles for bents located adjacent to the navigable channel be fixed to bedrock with a small H-pile section. Drilled shafts, socketed into bedrock, will also be able to provide sufficient lateral resistance from vessel impact for the piers, and may also be considered.

Closure. The preceeding alternatives are based on available subsurface information. If the structure is realigned significantly, the applicability of the alternatives will need to be reexamined, and/or subsurface explorations may be required prior to completion of the Preliminary Design Report (PDR).

Appendix B

Boring Logs

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BBR-201
PIN: 12630.00

Driller:	MaineDOT	**Elevation (ft.)**	9.1	**Auger ID/OD:**	5" Solid Stem Auger
Operator:	E. Giguere/G. Lidstone	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	CME 340 Auto Hammer/30"
Date Start/Finish:	9/6/07; 12:30-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-1.88
Boring Location:	10+11, 2.0' Lt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			Brown, dry, fine to medium SAND, (Topsoil).	
								6.20		2.9	
	R1	60/60	3.1 - 8.1	RQD = 64%			NQ	6.00		Top of Bedrock at Elev. 6.2' 3.1 R1: Bedrock: Grey, medium grained, quartz-feldspar-biotite GNEISS, moderately hard, moderately weathered, joints along banding at low angles, close, open, with infilling, surfaces stained and weathered. Very fractured zone 1'4" to 2'4" into core. Slightly fractured overall. Core Times (min:sec) 3.1-4.1' (2:08) 4.1-5.1' (1:50) 5.1-6.1' (1:28) 6.1-7.1' (1:33) 7.1-8.1' (1:29) 100% Recovery	
5											
								1.00			
	R2	60/60	8.1 - 13.1	RQD = 95%						8.1 R2: Grey, medium grained quartz-feldspar-biotite GNEISS, moderately hard, fresh, joints widely spaced, dipping a low angles, open, weathered, stained. Slightly fractured to massive. Core Times (min:sec) 8.1-9.1' (1:55) 9.1-10.1' (1:40) 10.1-11.1' (1:49) 11.1-12.1' (1:51) 12.1-13.1' (1:55) 100% Recovery	
10											
								-4.00		13.1	
15											
20											
25											

Bottom of Exploration at 13.10 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: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BBR-202
PIN: 12630.00

Driller:	MaineDOT	**Elevation (ft.)**	-7.46	**Auger ID/OD:**	N/A
Operator:	E. Giguere/G. Lidstone	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder/L. Krusinski	**Rig Type:**	CME 45C on Barge	**Hammer Wt./Fall:**	CME 340 Auto Hammer/30"
Date Start/Finish:	8/20/07-8/22/07	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-1.88
Boring Location:	11+11, 2.49' Lt.	**Casing ID/OD:**	NW	**Water Level*:**	Back River (Tidal)
Hammer Efficiency Factor:	.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							WOH			
							WOH			
							WOH			
							2			
	1D	24/2	4.0 - 6.0	WOH/WOH/WOH/ WOH	WOH		WOH		Dark grey, wet, very soft, clayey SILT, little fine sand, trace shell fragments, plastic.	
5							WOH			
							3			
							3			
							3			
	2D	24/10	9.0 - 11.0	WOR/WOR/WOR/3	WOR		5		Dark grey, wet, very soft, SILT, some clay, little fine sand, little fine gravel, trace shell fragments, plastic, slightly organic.	G#209953 CL-ML, A-4 WC=31.4%
10	V1		9.6 - 10.0	Su=446/22 psf						
	MV		10.2 - 10.6	Could not push			7		55x110 mm vane raw torque readings: V1: 10.0/0.5 ft-lbs Failed 55x110 mm vane attempt.	
							20			
							44	-19.96	12.5	
	3D	24/24	13.0 - 15.0	4/4/5/6	9	12	39		Olive grey and brown, damp, mottled, silt CLAY, stiff, medium plasticity, blocky.	
	MV		13.2 - 13.6	Could not push					Failed 55x110 mm vane attempt.	
							49			
15							39			
							74			
							73			
	4D	24/24	18.0 - 20.0	1/2/2/3	4	5	60		Olive grey, very little mottling, soft, silt CLAY, damp, plastic, easily rolled into <1/4" threads, blocky, medium stiff.	G#209951 CL, A-6 LL=38 PL=23 PI=15 WC=31.6%
	MV		18.2 - 18.4	Could not push						
							60			
20							54			
							55			
							52	-29.46	22.0	
	5D	24/24	23.0 - 25.0	WOR/WOR/WOH/ WOH	WOH		22		Dark grey, wet, very soft, CLAY, little silt, high plasticity, too high water content to roll threads, homogeneous.	
	V2		23.6 - 24.0	Su=491/89 psf					55x110 mm vane raw torque readings:	
25	V3		24.6 - 25.0	Su=469/89 psf			54		V2: 11.0/2.0 ft-lbs	

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BBR-202
PIN: 12630.00

Driller:	MaineDOT	**Elevation (ft.)**	-7.46	**Auger ID/OD:**	N/A
Operator:	E. Giguere/G. Lidstone	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder/L. Krusinski	**Rig Type:**	CME 45C on Barge	**Hammer Wt./Fall:**	CME 340 Auto Hammer/30"
Date Start/Finish:	8/20/07-8/22/07	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-1.88
Boring Location:	11+11, 2.49' Lt.	**Casing ID/OD:**	NW	**Water Level*:**	Back River (Tidal)
Hammer Efficiency Factor:	.77	**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 = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

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							50			55x110 mm vane raw torque readings: V3: 10.5/2.0 ft-lbs	
							43				
							46				
	6D	24/24	28.0 - 30.0	WOR/WOR/WOR/ WOR	WOR		56			Dark grey, wet, very soft, clayey SILT, trace sand, one 1" angular gravel, high plasticity, excess water-can't roll into threads, fat clay. (Two cup sample)	G#209952 CL, A-6 LL=28 PL=15 PI=13 WC=30.4%
	V4		28.6 - 29.0	Su=522/82 psf						65x130 mm vane raw torque readings: V4: 19.0/3.0 ft-lbs	
	V5		29.6 - 30.0	Su=357/55 psf			61			65x130 mm vane raw torque readings: V5: 13.0/2.0 ft-lbs	
30							53				
							48	-38.79		31.3	
							60				
	7D	24/13	33.0 - 35.0	6/22/31/21	53	68	43			Brown, wet, very dense, fine to coarse SAND, little silt, changing to medium to coarse sand, some fine gravel (subangular), trace weathered biotite schist, trace coarse angular gravel.	
							75				
35							72				
							102				
							95				
	MD	24/0	38.0 - 40.0	11/14/13/12	27	35	86			Failed sample attempt.	
							96				
40							82				
							152				
							111				
	8D	22/15	43.0 - 44.8	6/11/9/5(10")	20	26	95			Grey, wet, medium dense, fine to coarse SAND, little fine angular gravel, well graded, trace silt, homogeneous, sorted, (Glaciofluvial).	
							[a]142			[a]142 blows for 10". Roller coned ahead to 45.0' bgs.	
45	R1	0/0	45.0 - 45.0					-52.63		Failed core attempt, casing too bent for coring. Lost core barrel in boring, moved to BB-BBR-202A.	
										45.2	
										Bottom of Exploration at 45.17 feet below ground surface. Surveyed Bedrock Elev. -52.63'.	
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BBR-202A
PIN: 12630.00

Driller:	MaineDOT	**Elevation (ft.)**	-5.82	**Auger ID/OD:**	N/A
Operator:	E. Giguere/G. Lidstone	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder/L. Krusinski	**Rig Type:**	CME 45C on Barge	**Hammer Wt./Fall:**	CME 340 Auto Hammer/30"
Date Start/Finish:	8/22/07-8/22/07	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-1.88
Boring Location:	11+07, 5.5' Lt.	**Casing ID/OD:**	HW	**Water Level*:**	Back River (Tidal)
Hammer Efficiency Factor:	.77	**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 = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/14	0.0 - 2.0	Hydraulic Push	---		WOC			Dark grey, wet, very soft, organic SILT, some fine sand, little clay, trace of gravel, shells and wood fragments.	G#209954 CL-ML, A-4 WC=75.9% Ignition Loss Loss=5.7% H2O=79.3%
							WOC	-7.82		2.0	
							WOH			Material descriptions not taken, see BB-BBR-202.	
							WOH				
							4				
5							HP			HP=Hydraulic Push	
							HP				
							8				
							22				
							19				
10							26				
							138				
							403				
							92				
							92			Washed casing out.	
15							74				
							114				
							95				
							106				
							99				
20							100				
							132				
							99				
							103				
25							90				

Remarks:

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

Boring No.: BB-BBR-202A

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BBR-202A
PIN: 12630.00

Driller:	MaineDOT	**Elevation (ft.)**	-5.82	**Auger ID/OD:**	N/A
Operator:	E. Giguere/G. Lidstone	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder/L. Krusinski	**Rig Type:**	CME 45C on Barge	**Hammer Wt./Fall:**	CME 340 Auto Hammer/30"
Date Start/Finish:	8/22/07-8/22/07	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-1.88
Boring Location:	11+07, 5.5' Lt.	**Casing ID/OD:**	HW	**Water Level*:**	Back River (Tidal)
Hammer Efficiency Factor:	.77	**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 = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							86				
							95				
							62				
							54				
							50				
30							67				
							82				
							102				
							128				
							185				
35							114				
							104				
							112				
							95				
							91				
40							102				
							70				
							114				
							96				
							200				
45	R1	60/60	45.0 - 50.0	RQD = 27%			NQ	-50.82		45.0 Top of Bedrock at Elev. -50.82' R1: Bedrock: Grey and white banded, fine to medium grained BIOTITE SCHIST. Moderately to completely weathered, soil like schist seams. Joints at low to moderately angles along bedding/banding, second joint set steep, closed, tight, no infilling, highly fractured, very soft, soil like zones between 3'7" and 5'0". Joints along layering, spacing close to very close. Correlation of RQD to rock quality: Poor. R1: Core Times (min:sec) 45.0-46.0' (4:27) 400-500 psi	
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: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BBR-202A
PIN: 12630.00

Driller:	MaineDOT	**Elevation (ft.)**	-5.82	**Auger ID/OD:**	N/A
Operator:	E. Giguere/G. Lidstone	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder/L. Krusinski	**Rig Type:**	CME 45C on Barge	**Hammer Wt./Fall:**	CME 340 Auto Hammer/30"
Date Start/Finish:	8/22/07-8/22/07	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-1.88
Boring Location:	11+07, 5.5' Lt.	**Casing ID/OD:**	HW	**Water Level*:**	Back River (Tidal)
Hammer Efficiency Factor:	.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
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WC = water content, percent
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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	R2	60/60	50.0 - 55.0	RQD = 63%				-55.82		46.0-47.0' (2:47) 500 psi 47.0-48.0' (2:15) " 48.0-49.0' (2:48) " 49.0-50.0' (2:53) " 100% Recovery 50.0 R2: Bedrock: Grey, fine to medium grained, BIOTITE SCHIST, soft, moderately weathered to completely weathered soil like zones 2'1"-2'10", moderately fractured. Joint set horiz. to dipping at low angles, spaced very close to close, tight. Second joint set near vertical, widely spaced, surfaces fresh to oxidized, soil like in softer zones; no layering or banding evident. Correlation of RQD to rock quality: Fair. R2: Core Times (min:sec) 50.0-51.0' (4:09) 500 psi 51.0-52.0' (4:09) " 52.0-53.0' (2:57) " 53.0-54.0' (1:59) " 54.0-55.0' (2:12) " 100% Recovery 55.0	
55								-60.82			
60											
65											
70											
75											

Bottom of Exploration at 55.00 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: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BBR-203
PIN: 12630.00

Driller:	MaineDOT	**Elevation (ft.)**	-11.5	**Auger ID/OD:**	N/A
Operator:	E. Giguere/G. Lidstone	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder/L. Krusinski	**Rig Type:**	CME 45C on Barge	**Hammer Wt./Fall:**	CME 340 Auto Hammer/30"
Date Start/Finish:	8/23/07-8/23/07	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-1.88
Boring Location:	11+95, 0.1' Lt.	**Casing ID/OD:**	HW	**Water Level*:**	Back River (Tidal)
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/13	0.0 - 2.0	WOR/WOR/WOR/ WOR	WOR		WOC		Dark grey, wet, very soft, organic SILT, some fine sand, little clay trace gravel and shell fragments, organic odor, medium plastic. -Marine Deposit-	G#209956 CL-ML, A-4 WC=93.1%
							WOC			
							WOC			
							WOH			
							WOH			
5							WOH			
							WOH			
							WOH			
	2D V1	24/18	8.0 - 10.0 8.6 - 9.0	WOR/WOR/WOR/ WOR	WOR		WOH		Olive grey, wet (viscous), very soft, SILT, some clay, trace fine sand, trace shell fragments, medium to highly plastic. -Marine Deposit- 55x110 mm vane raw torque readings: V1: 7.0/1.5 ft-lbs 55x110 mm vane raw torque readings: V1: 9.0/2.0 ft-lbs	G#209957 CL-ML, A-4 WC=13.9%
	V2		9.6 - 10.0	Su=312/67 psf Su=402/89 psf			WOH			
10							9			
							9			
							13			
	MV 3D	24/20	13.0 - 13.2 13.0 - 15.0	Could not push 2/3/3/3	6	8	31	-24.50	Failed 55x110 mm vane attempt. 13.0 Olive-grey, moist to wet, soft, silty CLAY, some fine sand, trace organic material fibers, medium plasticity, no dilatent -Glaciomarine Deposit-	
							39			
15							35			
							32			
							23			
	4D V3	24/24	18.0 - 20.0 18.6 - 19.0	WOR/WOR/WOR/ WOR	WOR		16	-29.50	18.0 Dark grey, wet, very soft, clayey SILT, trace sand, high plasticity, homogeneous. - Glaciomarine Deposit - 55x110 mm vane raw torque readings: V3: 8.0/2.0 ft-lbs 55x110 mm vane raw torque readings: V4: 10.0/2.0 ft-lbs	
	V4		19.6 - 20.0	Su=357/89 psf Su=446/89 psf			20			
20							23			
							21			
							18			
	5D V5	24/28	23.0 - 25.0 23.6 - 24.0	WOR/WOR/WOR/ WOR	WOR		21		Same as 4D. 65x130 mm vane raw torque readings: V5: 16.0/1.0 ft-lbs 65x130 mm vane raw torque readings:	G#209958 CL, A-6 LL=37 PL=25
25	V6		24.6 - 25.0	Su=439/27 psf Su=500/55 psf			23			

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BBR-203
PIN: 12630.00

Driller:	MaineDOT	**Elevation (ft.)**	-11.5	**Auger ID/OD:**	N/A
Operator:	E. Giguere/G. Lidstone	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder/L. Krusinski	**Rig Type:**	CME 45C on Barge	**Hammer Wt./Fall:**	CME 340 Auto Hammer/30"
Date Start/Finish:	8/23/07-8/23/07	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-1.88
Boring Location:	11+95, 0.1' Lt.	**Casing ID/OD:**	HW	**Water Level*:**	Back River (Tidal)
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
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WOH = weight of 140lb. hammer
WOR = weight of rods
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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							24			V6: 18.2/2.0 ft-lbs	PI=12 WC=44.5%
							22				
							19				
	6D V7	24/14	28.0 - 30.0 28.6 - 29.0	WOR/WOR/WOR/ WOR Su=426/41 psf	WOR		WOC			Grey, wet, very soft, clayey SILT, high plasticity. - Glaciomarine Deposit - Casing broke off at 15.0' bgs, pulled back with casing retriever. 65x130 mm vane raw torque readings: V7: 15.5/1.5 ft-lbs	
	V8		29.6 - 30.0	Su=500/55 psf			WOC			65x130 mm vane raw torque readings: V6: 18.2/2.0 ft-lbs	
30							WOC				
							WOH				
							WOH				
	7D V9	24/24	33.0 - 35.0 33.6 - 34.0	WOR/WOR/WOR/ WOR Su=481/55 psf	WOR		11			Same as 6D. 65x130 mm vane raw torque readings: V9: 17.5/2.0 ft-lbs	
	V10		34.6 - 35.0	Su=522/60 psf			10			65x130 mm vane raw torque readings: V10: 19.0/2.2 ft-lbs	
35							11				
							11				
							11				
	8D V11	24/24	38.0 - 40.0 38.6 - 39.0	WOR/WOR/WOR/ WOR Su=549/77 psf	WOR		12			Same as 7D. 65x130 mm vane raw torque readings: V11: 20.0/2.8 ft-lbs	
	V12		39.6 - 40.0	Su=590/77 psf			11			65x130 mm vane raw torque readings: V12: 21.5/2.8 ft-lbs	
40							13				
							13				
							16				
	9D V13	24/24	43.0 - 45.0 43.6 - 44.0	WOR/WOR/WOR/ WOR Su=467/110 psf	WOR		11			Same as 8D, 2 cup sample. 65x130 mm vane raw torque readings: V13: 17.0/4.0 ft-lbs	G#209955 CL, A-6 LL=35 PL=11 PI=24 WC=46.6%
	V14		44.6 - 45.0	Su=481/124 psf			12			65x130 mm vane raw torque readings: V14: 17.5/4.5 ft-lbs	
45							19				
							18				
							20				
	10D	24/24	48.0 - 50.0	WOR/WOR/WOR/ WOR	WOR		26			Similar to 9D, skipped vane testing.	
50							26				

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BBR-203
PIN: 12630.00

Driller:	MaineDOT	**Elevation (ft.)**	-11.5	**Auger ID/OD:**	N/A
Operator:	E. Giguere/G. Lidstone	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder/L. Krusinski	**Rig Type:**	CME 45C on Barge	**Hammer Wt./Fall:**	CME 340 Auto Hammer/30"
Date Start/Finish:	8/23/07-8/23/07	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-1.88
Boring Location:	11+95, 0.1' Lt.	**Casing ID/OD:**	HW	**Water Level*:**	Back River (Tidal)
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

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.
50							25			
							26			
							26			
	11D	24/24	53.0 - 55.0	WOR/WOR/WOR/ WOH	WOR		16		Grey, wet, very soft, sandy SILT, little clay.	G#209961 CL-ML, A-4 WC=26.1%
	V15		53.6 - 54.0	Su=632/115 psf					65x130 mm vane raw torque readings: V15: 23.0/4.2 ft-lbs	
	MV		54.0 - 54.2	Could not push			15	-66.00	Failed 65x130 mm vane attempt.	
55							59		54.5 Grey, wet, soft, silty fine SAND, little to trace clay.	
							107			
							62			
	12D	24/22	58.0 - 60.0	9/12/5/5	17	22	44		Grey, wet, medium dense, fine to coarse SAND, little fine gravel, rounded to subrounded, some layered silt.	
							25			
60							27			
							62			
							23			
	13D	24/16	63.0 - 65.0	6/5/4/5	9	12	25	-74.50	63.0 Grey, wet, medium dense, fine gravelly, rounded to subrounded, medium to coarse SAND, some fine sand, trace silt.	
							35			
65							46		20-foot section of 4-in HW steel casing broke off when pulling casing, and abandoned in drillhole between approximately elev. -60 and -80 ft.	
							34			
							19			
	14D	7.2/6	68.0 - 68.6	31/30(1.2")	---		75	-80.10	Grey, wet, very dense, sandy fine GRAVEL with two 1" dia. rounded rock fragments, (white gneiss).	
	R1	60/60	68.6 - 73.6				NQ		68.6 R1: 3.6' BOULDER, white and grey, QUARTZ-FELDSPAR GNEISS.	
70									R1: Core Times (min:sec) 68.6-69.6' (4:00) 69.6-70.6' (3:30) 70.6-71.6' (3:20) 71.6-72.6' (1:30)	
								-83.70	72.6-73.6' (0:30) 72% Recovery	
									72.2	
							SPUN		Pulled core barrel, retrieved R1. Spun core barrel ahead from 73.6-81.6' bgs.	
75							NQ		Two 0.1' rounded rock fragments picked up when advancing core barrel	

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BBR-203
PIN: 12630.00

Driller:	MaineDOT	**Elevation (ft.)**	-11.5	**Auger ID/OD:**	N/A
Operator:	E. Giguere/G. Lidstone	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder/L. Krusinski	**Rig Type:**	CME 45C on Barge	**Hammer Wt./Fall:**	CME 340 Auto Hammer/30"
Date Start/Finish:	8/23/07-8/23/07	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-1.88
Boring Location:	11+95, 0.1' Lt.	**Casing ID/OD:**	HW	**Water Level*:**	Back River (Tidal)
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

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.
75										from bottom boulder elevation to top of bedrock.	
80											
	R2	60/60	81.6 - 86.6	RQD = 35%			NQ	-93.10		81.6 Top of Bedrock at Elev. -93.10' R2: Bedrock: Grey and white, banded, quartz BIOTITE SCHIST, interspersed with medium grained GNEISS banding, fine to medium grained, soft to medium hard, moderately fractured, moderately weathered overall with intermintant soft, highly weathered banding. Joints chaotic, 1"-6" spacing, tight to open, soft to fresh, with loose biotite, soft infilling to no infilling. Correlation of RQD to rock quality: Poor. R2: Core Times (min:sec) 81.6-82.6' (4:00) 82.6-83.6' (3:30) 83.6-84.6' (4:20) 84.6-85.6' (4:45) 85.6-86.6' (4:30) 100% Recovery	
85								-98.10		86.6 **Bottom of Exploration at 86.60 feet below ground surface.**	
90											
95											
100											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BBR-204
PIN: 12630.00

Driller:	MaineDOT	**Elevation (ft.)**	-15.6	**Auger ID/OD:**	N/A
Operator:	E. Giguere/G. Lidstone	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder/L. Krusinski	**Rig Type:**	CME 45C on Barge	**Hammer Wt./Fall:**	CME 340 Auto Hammer/30"
Date Start/Finish:	9/5/07-9/5/07	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-1.88
Boring Location:	12+86.8, 3.9' Rt.	**Casing ID/OD:**	HW	**Water Level*:**	Back River (Tidal)
Hammer Efficiency Factor:	.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							WOC			
							WOC			
							WOC			
							WOC			
							WOH			
5							WOH			
							4			
	1D	24/14	7.0 - 9.0	WOH/WOH/WOH/ WOH	WOH		4		Grey, wet, very soft, organic SILT, some fine sand, little clay, little gravel, trace shell fragments.	G#209966 CL-ML, A-4 WC=39.6%
							4			
							6			
10							20	-25.60	10.0	
							27			
							21			
	2D	24/20	13.0 - 15.0	WOR/WOR/WOR/ WOR	WOR		aHP		aHydraulic Push Dark grey, wet, very soft, clayey SILT, trace sand, trace gravel, medium to high plasticity, viscous.	
	V1		13.6 - 14.0	Su=402/89 psf						
	V2		14.6 - 15.0	Su=491/134 psf			aHP			
15							5		-Glaciomarine Deposit- 55x110 mm vane raw torque readings: V1: 9.0/2.0 ft-lbs 55x110 mm vane raw torque readings: V2: 11.0/3.0 ft-lbs	
							21			
							22			
							22			
	3D	24/24	19.0 - 21.0	WOR/WOR/WOR/ WOR	WOR		6		Similar to 2D, except grey. - Glaciomarine Deposit -	G#209967 CL, A-6 LL=27 PL=14 PI=13 WC=36.0%
20							6			
							29			
	R1	60/60	22.6 - 27.6	RQD = 83%			b100 NQ	-38.20	b100 blows for 0.6'. 22.6 Top of Bedrock at Elev. -38.20' R1:Bedrock: White, fine to medium grained GNEISS, hard, fresh, discontinuties widely spaced, tight, fresh, changing at 2'6" to coarse grained, white, quartz feldspar PEGMATITE, hard, fresh, massive.	
25										

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Boring No.: BB-BBR-204

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BBR-204
PIN: 12630.00

Driller:	MaineDOT	**Elevation (ft.)**	-15.6	**Auger ID/OD:**	N/A
Operator:	E. Giguere/G. Lidstone	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder/L. Krusinski	**Rig Type:**	CME 45C on Barge	**Hammer Wt./Fall:**	CME 340 Auto Hammer/30"
Date Start/Finish:	9/5/07-9/5/07	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-1.88
Boring Location:	12+86.8, 3.9' Rt.	**Casing ID/OD:**	HW	**Water Level*:**	Back River (Tidal)
Hammer Efficiency Factor:	.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										Correlation of RQD to rock quality: good.	
										R1:Core Times (min:sec)	
										22.6-23.6' (5:00)	
										23.6-24.6' (4:30)	
										24.6-25.6' (4:20)	
	R2	60/60	27.6 - 32.6	RQD = 100%				-43.20		25.6-26.6' (4:30)	
										26.6-27.6' (4:10) 100% Recovery	
										27.6	
										R2: Bedrock: White, hard, coarse grained quartz feldspar PEGMATITE, fresh, massive. Correlation of RQD to rock quality: excellent.	
30										R2: Core Times (min:sec)	
										27.6-28.6' (2:43)	
										28.6-29.6' (2:28)	
										29.6-30.6' (2:33)	
										30.6-31.6' (2:54)	
										31.6-32.6' (2:50) 100% Recovery	
								-48.20		32.6	
										Bottom of Exploration at 32.60 feet below ground surface.	
35											
40											
45											
50											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BBR-205
PIN: 12630.00

Driller:	MaineDOT	**Elevation (ft.)**	-11.4	**Auger ID/OD:**	N/A
Operator:	E. Giguere/G. Lidstone	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder/L. Krusinski	**Rig Type:**	CME 45C on Barge	**Hammer Wt./Fall:**	CME 340 Auto Hammer/30"
Date Start/Finish:	8/28/07-8/29/07	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-1.88
Boring Location:	13+77.9, 0.5' Rt.	**Casing ID/OD:**	HW	**Water Level*:**	Back River (Tidal)
Hammer Efficiency Factor:	.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							WOC			
							WOC			
							WOC			
							WOC			
							WOC			
5	1D	24/10	5.0 - 7.0	WOR/WOR/WOR/ WOR	WOR		WOH		Grey, wet, very soft, organic SILT, some fine sand, little clay, trace gravel, organic fibers, sea shell fragments. -Marine Sediments-	G#209960 CL-ML, A-4 WC=53.9%
							6			
							[a]3/H		[a]3 blows for 6"/Hydraulic Push 6"	
							[b]H/2	-19.40	8.0 [b]Hydraulic Push 6"/25 blows for 6"	
							86			
10	2D	24/24	10.0 - 12.0	Hydraulic Push	---		87		Grey, moist, very soft, clayey SILT, trace fine sand, intermittent brown mottling, blocky, stiff tactile, non to low plasticity. -Glaciomarine Deposit-	G#209959 CL, A-6 LL=33 PL=15 PI=18 WC=23.4%
							101			
							105			
							115			
							117			
15	3D	24/24	15.0 - 17.0	WOH/WOH/WOH/ WOH	WOH		82	-26.40	15.0 Grey, wet, very soft, clayey SILT, medium plasticity -Glaciomarine Deposit-	
	V1		15.6 - 16.0	Su=1250/179 psf					55x110 mm vane raw torque readings: V1: 28.0/4.0 ft-lbs	
	V2		16.6 - 17.0	Su=571/89 psf			61		55x110 mm vane raw torque readings: V2: 12.8/2.0 ft-lbs	
							60			
							51			
							48			
20	4D	24/24	20.0 - 22.0	WOR/WOR/WOR/ WOR	WOR		60		Dark grey, wet, very soft, CLAY SILT, medium to high plasticity, -Glaciomarine Deposit-	
							44			
							36			
							30			
25	5D	24/24	24.0 - 26.0	WOR/WOR/WOR/ WOR	WOR		27		Grey, wet, very soft, clayey SILT, highly plastic. 55x110 mm vane raw torque readings:	

Remarks:

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Boring No.: BB-BBR-205

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BBR-205
PIN: 12630.00

Driller:	MaineDOT	**Elevation (ft.)**	-11.4	**Auger ID/OD:**	N/A
Operator:	E. Giguere/G. Lidstone	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder/L. Krusinski	**Rig Type:**	CME 45C on Barge	**Hammer Wt./Fall:**	CME 340 Auto Hammer/30"
Date Start/Finish:	8/28/07-8/29/07	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-1.88
Boring Location:	13+77.9, 0.5' Rt.	**Casing ID/OD:**	HW	**Water Level*:**	Back River (Tidal)
Hammer Efficiency Factor:	.77	**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 = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

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	V3 V4		24.6 - 25.0 25.6 - 26.0	Su=379/98 psf Su=357/89 psf			22		V3: 8.5/2.2 ft-lbs 55x110 mm vane raw torque readings: V4: 8.0/2.0 ft-lbs	
							21			
							21			
	6D	24/24	28.0 - 30.0	WOR/WOR/WOR/ WOR	WOR		21		Grey, wet, very soft, clayey SILT, trace sand, trace gravel.	G#209962 CL-ML, A-4 LL=25 PL=20 PI=5 WC=36.6%
							19			
30							43	-41.40	30.0	
							34			
							33			
	7D	24/12	33.0 - 35.0	4/14/14/6	28	36	38		Grey, wet, dense, fine SAND, some fine gravel, rounded to subangular, little silt.	G#209963 SM, A-1-b WC=10.6%
							55			
35	R1	60/57	35.9 - 40.9	RQD = 57%			a60 NQ	-47.30	a60 blows for 0.9'. 35.9 Top of Bedrock @ Elev. -47.30' R1:Bedrock: Grey, moderately hard, banded, fine to medium grained BIOTITE SCHIST, very slightly weathered, discontinuities at horiz. to steep angles, close spacing, tight to open, trace of silt and sand infilling, slightly weathered surface, moderately fractured. Correlation of RQD to rock quality: Good. R1:Core Times (min:sec) 35.9-36.9' (5:30) 36.9-37.9' (5:00) 37.9-38.9' (4:00) 38.9-39.9' (4:30) 39.9-40.9' (4:20) 95% Recovery	
40	R2	60/60	40.9 - 45.9	RQD = 80%				-52.30	40.9 R2:Bedrock: Grey, fine to medium grained BIOTITE SCHIST, fresh to slightly weathered, jointing chaotic, horiz. to steep (visible in lower) along and perpendicular to banding closely spaced, trace silt infilling, slightly fractured. Correlation of RQD to rock quality: Good. R2:Core Times (min:sec) 40.9-41.9' (3:15) 41.9-42.9' (2:30) 42.9-43.9' (2:18) 43.9-44.9' (2:05) 44.9-45.9' (2:05) 100% Recovery	
45								-57.30	45.9 **Bottom of Exploration at 45.90 feet below ground surface.**	
50										

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BBR-206
PIN: 12630.00

Driller:	MaineDOT	**Elevation (ft.)**	-5.9	**Auger ID/OD:**	N/A
Operator:	E. Giguere/G. Lidstone	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder/L. Krusinski	**Rig Type:**	CME 45C on Barge	**Hammer Wt./Fall:**	CME 340 Auto Hammer/30"
Date Start/Finish:	8/29/07-8/29/07	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-1.88
Boring Location:	14+68.1, 2.5' Rt.	**Casing ID/OD:**	HW	**Water Level*:**	Back River (Tidal)
Hammer Efficiency Factor:	.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/14	0.0 - 2.0	WOR/WOR/WOR/ WOR	WOR		WOC		Olive grey, wet, very soft, clayey organic SILT, trace shell fragments.	
							WOC			
							WOC			
							WOH			
							3	-9.90	4.0	
5	2D	24/16	5.0 - 7.0	WOR/WOR/WOR/ WOR	WOR		3		Grey, wet, very soft, SILT, some fine sand, trace clay, trace fine gravel, trace shell fragments.	G#209964 CL-ML, A-4 WC=44.2%
							10			
							29			
							43	-13.90	8.0	
	3D	24/22	9.0 - 11.0	3/3/3/5	6	8	41		Grey and brown, moist, mottled, medium stiff, silty CLAY, trace of sand shell fragments, blocky, stiff tactilely. -Glaciomarine Deposit-	
10	MV		10.6 - 10.6	Could not push			40		Failed 55x110 mm vane attempt.	
							66			
							59			
							81			
	4D	24/24	14.0 - 16.0	2/3/3/3	6	8	94		Grey and brown, moist, mottled, medium stiff silty CLAY, trace sand, blocky.	G#209965 CL, A-6 LL=37 PL=23 PI=14 WC=31.2%
15	MV		14.6 - 14.6	Could not push					Failed 55x110 mm vane attempt. PP=1.1 tsf, 1.5 tsf	
							89			
							90	-21.90	16.0	
							79			
							79			
	5D	24/24	19.0 - 21.0	WOR/WOR/WOR/ WOR	WOR		62		Dark grey, wet, very soft, marine silty CLAY, highly plasticity. PP=0 tsf	
20							55			
							55			
							47			
							38			
25							32			

Remarks:

PP = Pocket Penetrometer

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BBR-206
PIN: 12630.00

Driller:	MaineDOT	**Elevation (ft.)**	-5.9	**Auger ID/OD:**	N/A
Operator:	E. Giguere/G. Lidstone	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder/L. Krusinski	**Rig Type:**	CME 45C on Barge	**Hammer Wt./Fall:**	CME 340 Auto Hammer/30"
Date Start/Finish:	8/29/07-8/29/07	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-1.88
Boring Location:	14+68.1, 2.5' Rt.	**Casing ID/OD:**	HW	**Water Level*:**	Back River (Tidal)
Hammer Efficiency Factor:	.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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SSA = Solid Stem Auger
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RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
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PI = Plasticity Index
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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/A	24/24	25.0 - 27.0	WOR/WOR/WOR/4	WOR		45			(6D/A) 25.0-26.6' bgs. Similar to above.	
							66	-32.50		26.6	
							117			(6D/B) 26.6-28.4' bgs. Grey, wet, soft, silty fine SAND, little clay.	
	R1	60/60	28.4 - 33.4	RQD = 50%			a200 NQ	-34.30		a200 blows for 0.4'. 28.4 Top of bedrock at Elev. -34.30' R1: Bedrock: Beige, fine to medium grained, quartz feldspar GNEISS, moderately hard, moderately to highly weathered, quartz is discolored and stained, feldspar broken down, jointing chaotic, horiz. to steep, close, open, sandy silt infilling, discolored, oxidized surfaces, moderately fractured. Rock Quality Poor R1:Core Times (min:sec) 28.4-29.4' (2:55) 29.4-30.4' (2:23) 30.4-31.4' (3:48) 31.4-32.4' (2:18) 32.4-33.4' (2:18) 100% Recovery	
30											
	R2	60/58	33.4 - 38.4	RQD = 48%				-39.30		33.4 R2:Bedrock: Upper 2'2' fractured, discolored, stained GNEISS. White to grey, discolored, stained, fine to medium grained, quartz feldspar GNEISS, moderately hard, moderately weathered to slightly weathered, joints along banding at chaotic angles, surfaces open, discolored, stained, decomposed feldspar, very close to close, very weathered biotite schist band 3'9"-4'2".	
35											
								-44.30		R2:Core Times (min:sec) 33.4-34.4' (3:00) 34.4-35.4' (2:30) 35.4-36.4' (2:10) 36.4-37.4' (2:05) 37.4-38.4' (3:00) 92% Recovery 38.4 **Bottom of Exploration at 38.40 feet below ground surface.**	
40											
45											
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BBR-207
PIN: 12630.00

Driller:	MaineDOT	**Elevation (ft.)**		**Auger ID/OD:**	N/A
Operator:	E. Giguere/G. Lidstone	**Datum:**	NAVD 88	**Sampler:**	N/A
Logged By:	B. Wilder/L. Krusinski	**Rig Type:**	CME 45C on Barge	**Hammer Wt./Fall:**	CME 340 Auto Hammer/30"
Date Start/Finish:	9/4/07-9/4/07	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	15+42.1, 13.1' Rt.	**Casing ID/OD:**	N/A	**Water Level*:**	Back River (Tidal)
Hammer Efficiency Factor:	.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0										**Bottom of Exploration at 0.00 feet below ground surface.**	
										Cobble or boulder on surface, relocated to BB-BBR-207A.	
5											
10											
15											
20											
25											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BBR-207A
PIN: 12630.00

Driller:	MaineDOT	**Elevation (ft.)**	-3.52	**Auger ID/OD:**	N/A
Operator:	E. Giguere/G. Lidstone	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder/L. Krusinski	**Rig Type:**	CME 45C on Barge	**Hammer Wt./Fall:**	CME 340 Auto Hammer/30"
Date Start/Finish:	9/4/07-9/4/07	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-1.88
Boring Location:	15+40.1, 14.4' Rt.	**Casing ID/OD:**	HW	**Water Level*:**	Back River (Tidal)
Hammer Efficiency Factor:	.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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HSA = Hollow Stem Auger
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N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

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.
0							[a]HYD			[a]Hydraulic Push Tidal flat sediments and boulders.	
							3				
							3				
	R1	60/60	3.5 - 8.5	RQD = 38%			[b]9 NQ	-7.02		[b]9 blows for 0.5', then no movement.	
										3.5 Top of Bedrock at Elev. -7.02'	
5										R1: Bedrock: White, discolored and stained, fine to medium grained GNEISS, moderately hard, moderately weathered, discontinuities horizontal to steep, tight, discolored, stained, highly fractured. Correlation of RQD to rock quality: poor. R1:Core Times (min:sec) 500 psi 3.5-4.5' (3:26) 4.5-5.5' (2:36) 5.5-6.5' (2:52) 6.5-7.5' (2:38) lost water at 3.5' into run 7.5-8.5' (2:11) 100% Recovery	
	R2	60/59	8.5 - 13.5	RQD = 73%				-12.02			
10										8.5 R2: Bedrock: Same as R1, except moderately to slightly weathered, slightly fractured. Correlation of RQD to rock quality: fair. R2:Core Times (min:sec) 500 psi 8.5-9.5' (2:36) 9.5-10.5' (2:10) 10.5-11.5' (2:00) 11.5-12.5' (2:27) 12.5-13.5' (3:05) 98% Recovery	
								-17.02		13.5 **Bottom of Exploration at 13.50 feet below ground surface.**	
15											
20											
25											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BR.BOO-101
PIN: 12630.00

Driller:	MaineDOT	**Elevation (ft.)**	-13.2	**Auger ID/OD:**	N/A
Operator:	E. Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/24/05; 08:00-17:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	11+52.4, 57.7' Lt.	**Casing ID/OD:**	HW	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.60	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

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	24/15	0.0 - 2.0	WOR/WOR/WOR/ WOR	WOR		WOC		Dark grey, wet, very soft, organic SILT, little fine sand, trace shells. -MARINE DEPOSIT-	
							WOC			
							WOC			
							WOH			
							WOH			
5	2D	24/17	5.0 - 7.0	WOR/WOR/WOR/ WOR	WOR		WOH	-18.20	5.0 Grey, wet, very soft, SILT, some clay, trace sand, trace organics.	G#181934 A-6, CL WC=66.8% LL=38 PL=25 PI=13
							2			
							2			
							2			
							4			
10	3D	24/14	10.0 - 12.0	WOR/WOH/WOH/ WOH	0	0	12		55x110 mm vane raw torque readings:	
	V1		10.6 - 11.0	V1: Su=848/134 psf	9	9			V1: 19.0/3.0 ft-lbs (probable sand seam)	
	MV		11.1 - 11.5	MV: Could not push			29	-24.70	Failed 55x110 mm vane attempt (probable fine sand seam).	
							46		11.5 Grey-brown, SILT, trace fine sand, trace clay.	
							62			
							51	-27.20	14.0	
15	4D	24/24	15.0 - 17.0	WOR/WOR/WOR/ WOR	WOR		29		Grey, wet, soft, clayey SILT, trace sand. -GLACIOMARINE DEPOSIT-	G#181935 A-4, CL WC=44.2% LL=28 PL=20 PI=8
							28		Washed ahead of casing from 15.0-17.0' bgs.	
	V2		17.6 - 18.0	Su=335/27 psf			20		65x130 mm vane raw torque readings: V2: 12.2/1.0 ft-lbs	
	V3		18.6 - 19.0	Su=343/22 psf			16		65x130 mm vane raw torque readings: V3: 12.5/0.8 ft-lbs	
							10			
20	5D	24/24	20.0 - 22.0	WOR/WOR/WOR/ WOR	WOR		13		Grey, wet, soft, clayey SILT.	
	V4		20.6 - 21.0	V4: Su=335/27 psf					65x130 mm vane raw torque readings: V4: 12.2/1.0 ft-lbs	
	V5		21.6 - 22.0	V5: Su=330/33 psf			11		65x130 mm vane raw torque readings: V5: 12.0/1.2 ft-lbs	
							11			
							11			
25							10			

Remarks:

Barge to Ground 10.2' at 08:00, 8/24/05.
15.5' of Casing and Rods used before reaching ground.
WOC = Weight Of Casing

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BR.BOO-101
PIN: 12630.00

Driller:	MaineDOT	**Elevation (ft.)**	-13.2	**Auger ID/OD:**	N/A
Operator:	E. Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/24/05; 08:00-17:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	11+52.4, 57.7' Lt.	**Casing ID/OD:**	HW	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.60	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/24	25.0 - 27.0	WOR/WOR/WOR/ WOR	WOR		14			Grey, wet, very soft, clayey SILT.	
	V6		25.6 - 26.0	V6: Su=233/33 psf						65x130 mm vane raw torque readings: V6: 8.5/1.2 ft-lbs	
	V7		26.6 - 27.0	V7: Su=233/41 psf			14			65x130 mm vane raw torque readings: V7: 8.5/1.5 ft-lbs	
							15				
							14				
	7D	24/24	29.0 - 31.0	WOR/WOR/WOR/ WOR	WOR		21			Grey, wet, soft, clayey SILT.	
30	V8		29.6 - 30.0	V8: Su=302/55 psf						65x130 mm vane raw torque readings: V8: 11.0/2.0 ft-lbs	
	V9		30.6 - 31.0	V9: Su=308/55 psf			19			65x130 mm vane raw torque readings: V9: 11.2/2.0 ft-lbs	
							17				
							18				
							19				
	8D	24/7	34.0 - 36.0	WOR/3/2/2	5	5	19	-47.60		Failed 65x130 mm vane attempt.	
35	MV		34.0 - 34.4	Could not push						34.4	
							25			Grey, wet, loose, silty fine to medium SAND, little coarse sand, little coarse clay in layers.	
							24	-49.20		36.0	
							44				
							32				
							31				
40	9D	24/1	40.0 - 42.0	12/11/12/27	23	23	48			Grey, wet, medium dense, fine to medium SAND, little coarse sand, trace gravel, trace silt.	
							64				
							79				
							52				
							51				
45	10D	24/4	45.0 - 47.0	16/10/8/10	18	18	45			Grey, wet, medium dense, fine to medium SAND, little coarse sand, trace gravel, trace silt.	
							48				
							62				
							55				
50							68				

Remarks:

Barge to Ground 10.2' at 08:00, 8/24/05.
15.5' of Casing and Rods used before reaching ground.
WOC = Weight Of Casing

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BR.BOO-101
PIN: 12630.00

Driller:	MaineDOT	**Elevation (ft.)**	-13.2	**Auger ID/OD:**	N/A
Operator:	E. Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/24/05; 08:00-17:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	11+52.4, 57.7' Lt.	**Casing ID/OD:**	HW	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.60	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	R1	60/60	50.2 - 55.2	RQD = 82%			a50 NQ	-63.70		a50 blows for 2.4". Cobble from 50.1-50.5' bgs. R1: Core Times (min:sec) 50.2-51.2 (2:28)	
										50.5	
										Bedrock at Elev -63.70'	
										BEDROCK: Dark grey with white infilling, medium to coarse grained, quartz-feldspar GNEISS, moderately hard, moderately weathered, steep to vertical foliation/jointing, close spacing. R1: Core Times (min:sec)	
55								-68.40		51.2-52.2 (2:33) 52.2-53.2 (2:42) 53.2-54.2 (2:45) 54.2-55.2 (3:17) 100% Recovery	
										55.2	
										Bottom of Exploration at 55.20 feet below ground surface.	
60											
65											
70											
75											

Remarks:

Barge to Ground 10.2' at 08:00, 8/24/05.
15.5' of Casing and Rods used before reaching ground.
WOC = Weight Of Casing

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BR.BOO-102
PIN: 12630.00

Driller:	MaineDOT	**Elevation (ft.)**	-16.9	**Auger ID/OD:**	N/A
Operator:	E. Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/23/05; 10:30-17:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+79.7, 58.6' Lt.	**Casing ID/OD:**	HW	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.60	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

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	0.0 - 2.0	WOR/WOR/WOR/ WOR	WOR		WOC		Dark grey, wet, very soft, organic SILT, little fine sand, trace coarse sand, trace shells. -MARINE DEPOSIT-	
							WOC			
							WOC		Dark grey, wet, very soft, organic SILT, little fine sand, trace shells.	
							WOH			
							WOH			
5							3	-21.90	5.0	
							3			
	2D V1	24/14	7.0 - 9.0 7.6 - 8.0	WOH/1/3/4 Su=670/143 psf	4	4	5		Grey and brown, soft, SILT, trace clay. 55x110 mm vane raw torque readings: V1: 15.0/3.2 ft-lbs	
							14			
							18			
10							16	-26.90	10.0 -GLACIOMARINE DEPOSIT-	
							15			
	3D V2	24/19	12.0 - 14.0 12.6 - 13.0	WOR/WOR/WOR/ WOR V2: Su=402/76 psf	WOR		15		Grey, wet, soft, clayey SILT, trace sand, with black staining. 55x110 mm vane raw torque readings: V2: 9.0/1.7 ft-lbs	G#181936 A-6, CL WC=41.7% LL=32 PL=19 PI=13
	V3		13.6 - 14.0	V3: Su=344/22 psf			16		55x110 mm vane raw torque readings: V3: 7.7/0.5 ft-lbs	
							14			
15	4D V4	24/7	15.0 - 17.0 15.6 - 16.0	WOR/WOR/WOR/ WOH V4: Su=426/69 psf	WOR		19		Grey, wet, soft, clayey SILT. 65x130 mm vane raw torque readings: V4: 15.5/2.5 ft-lbs	
	V5		16.6 - 17.0	V5: Su=363/55 psf			25		65x130 mm vane raw torque readings: V5: 13.2/2.0 ft-lbs	
							21			
							18			
							17			
20	5D V6	24/18	20.0 - 22.0 20.6 - 21.0	WOR/WOR/WOR/ WOR V6: Su=137/60 psf	WOR		27		Grey, wet, very soft, clayey SILT, trace sand. 65x130 mm vane raw torque readings: V6: 5.0/2.2 ft-lbs	G#181937 A-4, CL WC=42.8 LL=31 PL=22 PI=9
	V7		21.6 - 22.0	V7: Su=247/69 psf			25		65x130 mm vane raw torque readings: V7: 9.0/2.5 ft-lbs	
							20			
							20			
25	6D/AB	24/13	24.0 - 26.0	WOR/1/9/6	10	10	19	-40.90	24.0 Roller Coned ahead from 24.0-24.8' bgs.	

Remarks:

Barge to Ground 14.5' at 10:35, 8/23/05.
15.5' of Casing and Rods used before reaching ground.
WOC = Weight Of Casing

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BR.BOO-102
PIN: 12630.00

Driller:	MaineDOT	**Elevation (ft.)**	-16.9	**Auger ID/OD:**	N/A
Operator:	E. Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/23/05; 10:30-17:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+79.7, 58.6' Lt.	**Casing ID/OD:**	HW	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.60	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

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							23	-42.00	(6D/A) 24.0-25.1' bgs. Grey, wet, very soft, clayey SILT.	
	R1	60/56	26.6 - 31.6	RQD = 62%			[a]222 NQ	-42.90	25.1	
								-43.30	(6D/B) 25.1-26.0' bgs. Grey, wet, loose, silty fine SAND with clay layers, trace gravel.	
								-43.50	26.0	
									[a]222 blows for 7.2". Grey, wet, fine to coarse SAND, little gravel.	
									26.4	
									Weathered ROCK.	
									26.6	
30									Bedrock at Elev. -43.50'	
	R2	58.8/56	31.6 - 36.5	RQD = 90%				-48.50	BEDROCK: White, medium to coarse grained, quartz-feldspar GNEISS, hard, moderately weathered, low-angle foliation/jointing, very close spacing.	
									R1:Core Times (min:sec) 26.6-27.6 (2:55) 27.6-28.6 (3:25) 28.6-29.6 (2:45) 29.6-30.6 (3:00) 30.6-31.6 (2:55) 93% Recovery	
35									31.6	
								-53.40	R2:Core Times (min:sec) 31.6-32.6 (2:15) 32.6-33.6 (2:05) 33.6-34.6 (3:03) 34.6-35.6 (3:09) 35.6-36.5 (3:23) 95% Recovery	
									36.5	
40									**Bottom of Exploration at 36.50 feet below ground surface.**	
45										
50										

Remarks:

Barge to Ground 14.5' at 10:35, 8/23/05.
15.5' of Casing and Rods used before reaching ground.
WOC = Weight Of Casing

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BR.BOO-103
PIN: 12630.00

Driller:	MaineDOT	**Elevation (ft.)**	-7.6	**Auger ID/OD:**	N/A
Operator:	E. Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/22/05-8/23/05	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+17.9, 57.5' Lt.	**Casing ID/OD:**	HW	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.60	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

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	24/9	0.0 - 2.0	WOR/WOR/WOR/ WOR	WOR		WOC		Dark grey, wet, very soft, organic SILT, trace fine sand, trace shells. -MARINE DEPOSIT-	
							WOC			
							WOC			
							WOH			
							WOH			
5							1			
	2D/AB MV	24/17	6.0 - 8.0 6.6 - 7.0	WOR/WOR/WOH/5 aSu=>1217 psf	WOH		5		(2D/A) 6.0-7.5' bgs. Grey, wet, very soft, clayey SILT, trace fine sand layers, trace gravel, trace shells.	G#181938 A-4, CL WC=26.8% LL=24 PL=16 PI=8
							15	-15.10	65x130 mm vane raw torque readings: aMV: >44.3 ft-lbs (probable sand seam)	
							31		7.5	
							36		(2D/B) 7.5-8.0' bgs. Grey-brown, moist, very soft, clayey SILT, trace sand.	G#181939 A-4, CL WC=23.6%
10							37			
	3D	24/22	11.0 - 13.0	1/3/3/4	6	6	60		Grey-brown, moist, loose, fine SAND, some silt, trace clay.	
							55			
							50			
							50			
15							47			
	V1 4D	24/24	16.0 - 16.1 16.4 - 18.4	bSu=2201/1100 psf WOR/WOH/WOH/ WOH	WOH		30	-24.00	16x32 mm vane raw torque readings: bV1: 14.0/7.0 in-lbs (vane in fine sand)	
							28		16.4 Grey, wet, very soft, clayey SILT with black staining. -GLACIOMARINE DEPOSIT-	
							19			
							17			
20							14			
	5D V2	24/24	21.0 - 23.0 21.6 - 22.0	WOR/WOR/WOR/ WOR V2: Su=439/74 psf	WOR		19		Grey, wet, very soft, silty CLAY, trace sand. 65x130 mm vane raw torque readings: V2: 16.0/2.7 ft-lbs	G#181940 A-6, CL WC=43.0% LL=34 PL=22 PI=12
	V3		22.6 - 23.0	V3: Su=412/69 psf			16		65x130 mm vane raw torque readings: V3: 15.0/2.5 ft-lbs	
							18			
25							18			

Remarks:

Barge to Ground 12.2' at 11:35, 8/22/05.
15.5' of Casing and Rods used before reaching ground.
WOC = Weight Of Casing
8/22/05; 11:30-17:00, 8/23/05; 07:30-10:00

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: Knickerbocker Bridge #2438 over Back River
Location: Boothbay, Maine

Boring No.: BB-BR.BOO-103
PIN: 12630.00

Driller:	MaineDOT	**Elevation (ft.)**	-7.6	**Auger ID/OD:**	N/A
Operator:	E. Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/22/05-8/23/05	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+17.9, 57.5' Lt.	**Casing ID/OD:**	HW	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.60	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

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							20			
	6D	24/24	26.0 - 28.0	WOR/WOR/WOR/ WOR	WOR		24		Grey, wet, very soft, clayey SILT.	
	V4		26.6 - 27.0	V4: Su=439/88 psf					65x130 mm vane raw torque readings: V4: 16.0/3.2 ft-lbs	
	V5		27.6 - 28.0	V5: Su=390/55 psf			20		65x130 mm vane raw torque readings: V5: 14.2/2.0 ft-lbs	
							16			
							16			
30							17			
	7D	24/1	31.0 - 33.0	18/13/10/11	23	23	68	-38.60	31.0 Grey, wet, medium dense, fine to coarse SAND, little gravel, trace silt.	
							42			
							48			
							77			
35							105			
							159	-44.50	36.9	
	R1	60/57	37.0 - 42.0	RQD = 53%			NQ	-44.60	Weathered ROCK. 37.0 Bedrock at Elev. -44.60'	
									BEDROCK: Dark grey with white infilling, medium to coarse grained, quartz-feldspar GNEISS, moderately hard, moderately weathered, steep to vertical foliation, low angle jointing, close spacing. FAIR QUALITY R1:Core Times (min:sec) 37.0-38.0 (3:25) 38.0-39.0 (2:25) 39.0-40.0 (2:30) 40.0-41.0 (3:20) 41.0-42.0 (3:35) 95% Recovery		
40										
								-49.60		
	R2	60/56	42.0 - 47.0	RQD = 67%					42.0 R2:Core Times (min:sec) 42.0-43.0 (2:45) 43.0-44.0 (3:05) 44.0-45.0 (3:15) 45.0-46.0 (3:30) 46.0-47.0 (3:20) 93% Recovery	
45										
								-54.60	47.0 **Bottom of Exploration at 47.00 feet below ground surface.**	
50										

Remarks:

Barge to Ground 12.2' at 11:35, 8/22/05.
15.5' of Casing and Rods used before reaching ground.
WOC = Weight Of Casing
8/22/05; 11:30-17:00, 8/23/05; 07:30-10:00

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.

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	

Appendix C

Laboratory Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Boothbay Project Number: 12630.00

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-BR.BOO-101, 2D	11+52.4	57.7 Lt.	5.0-7.0	181934	1	66.8	38	13	CL	A-6	III
BB-BR.BOO-101, 4D	11+52.4	57.7 Lt.	15.0-17.0	181935	1	44.2	28	8	CL	A-4	IV
BB-BR.BOO-102, 3D	12+79.7	58.6 Lt.	12.0-14.0	181936	1	41.7	32	13	CL	A-6	III
BB-BR.BOO-102, 5D	12+79.7	58.6 Lt.	20.0-22.0	181937	1	42.8	31	9	CL	A-4	IV
BB-BR.BOO-103, 2D/A	14+17.9	57.5 Lt.	6.0-7.5	181938	2	26.8	24	8	CL	A-4	IV
BB-BR.BOO-103, 2D/B	14+17.9	57.5 Lt.	7.5-8.0	181939	2	23.6			CL	A-4	IV
BB-BR.BOO-103, 5D	14+17.9	57.5 Lt.	21.0-23.0	181940	2	43.0	34	12	CL	A-6	IV
*BB-BBR-202A, 1D	11+07	5.5 Lt.	0.0-2.0	209954	3	75.9			CL-ML	A-4	IV
BB-BBR-202, 2D	11+11	2.49 Lt.	9.0-11.0	209953	3	31.4			CL-ML	A-4	IV
BB-BBR-202, 4D	11+11	2.49 Lt.	18.0-20.0	209951	3	31.6	38	15	CL	A-6	III
BB-BBR-202, 6D	11+11	2.49 Lt.	28.0-30.0	209952	3	30.4	28	13	CL	A-6	III
BB-BBR-203, 1D	11+95	0.1 Lt.	0.0-2.0	209956	4	93.1			CL-ML	A-4	IV
BB-BBR-203, 2D	11+95	0.1 Lt.	8.0-10.0	209957	4	13.9			CL-ML	A-4	IV
BB-BBR-203, 5D	11+95	0.1 Lt.	23.0-25.0	209958	4	44.5	37	12	CL	A-6	IV
BB-BBR-203, 9D	11+95	0.1 Lt.	43.0-45.0	209955	4	46.6	35	24	CL	A-6	III
BB-BBR-203, 11D	11+95	0.1 Lt.	53.0-55.0	209961	4	26.1			CL-ML	A-4	IV
BB-BBR-204, 1D	12+86.8	3.9 Rt.	7.0-9.0	209966	5	39.6			CL-ML	A-4	IV
BB-BBR-204, 3D	12+86.8	3.9 Rt.	19.0-21.0	209967	5	36.0	27	13	CL	A-6	III
BB-BBR-205, 1D	13+77.9	0.5 Rt.	5.0-7.0	209960	6	53.9			CL-ML	A-4	IV
BB-BBR-205, 2D	13+77.9	0.5 Rt.	10.0-12.0	209959	6	23.4	33	18	CL	A-6	III
BB-BBR-205, 6D	13+77.9	0.5 Rt.	28.0-30.0	209962	6	36.6	25	5	CL-ML	A-4	IV
BB-BBR-205, 7D	13+77.9	0.5 Rt.	33.0-35.0	209963	6	10.6			SM	A-1-b	II
BB-BBR-206, 2D	14+68.1	2.5 Rt.	5.0-7.0	209964	6	44.2			CL-ML	A-4	IV
BB-BBR-206, 4D	14+68.1	2.5 Rt.	14.0-16.0	209965	6	31.2	37	14	CL	A-6	III
*BB-BBR-202A, 1D ----- Loss on Ignition (T267) Loss% 5.7, H2O% 79.3											

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

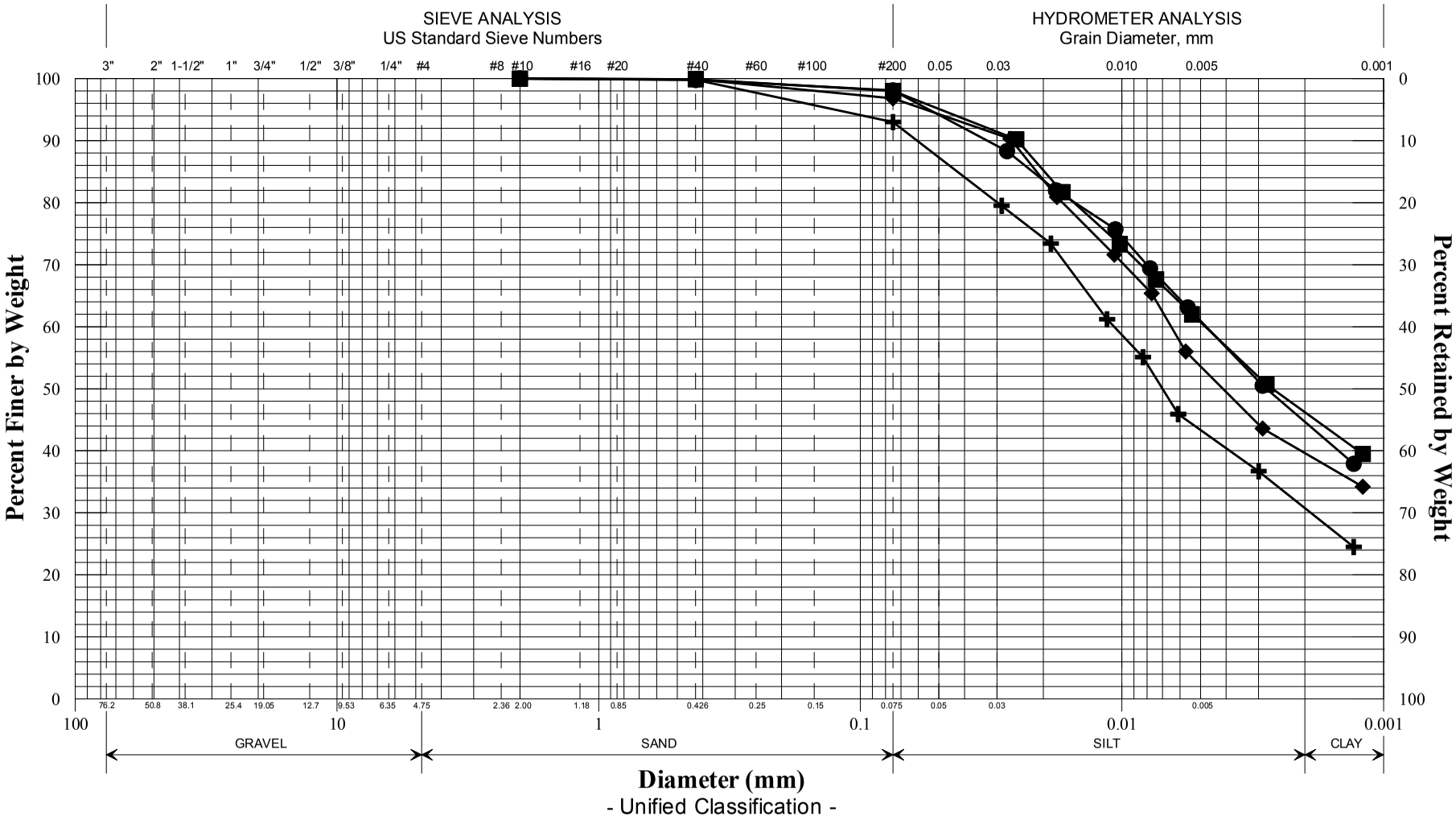
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

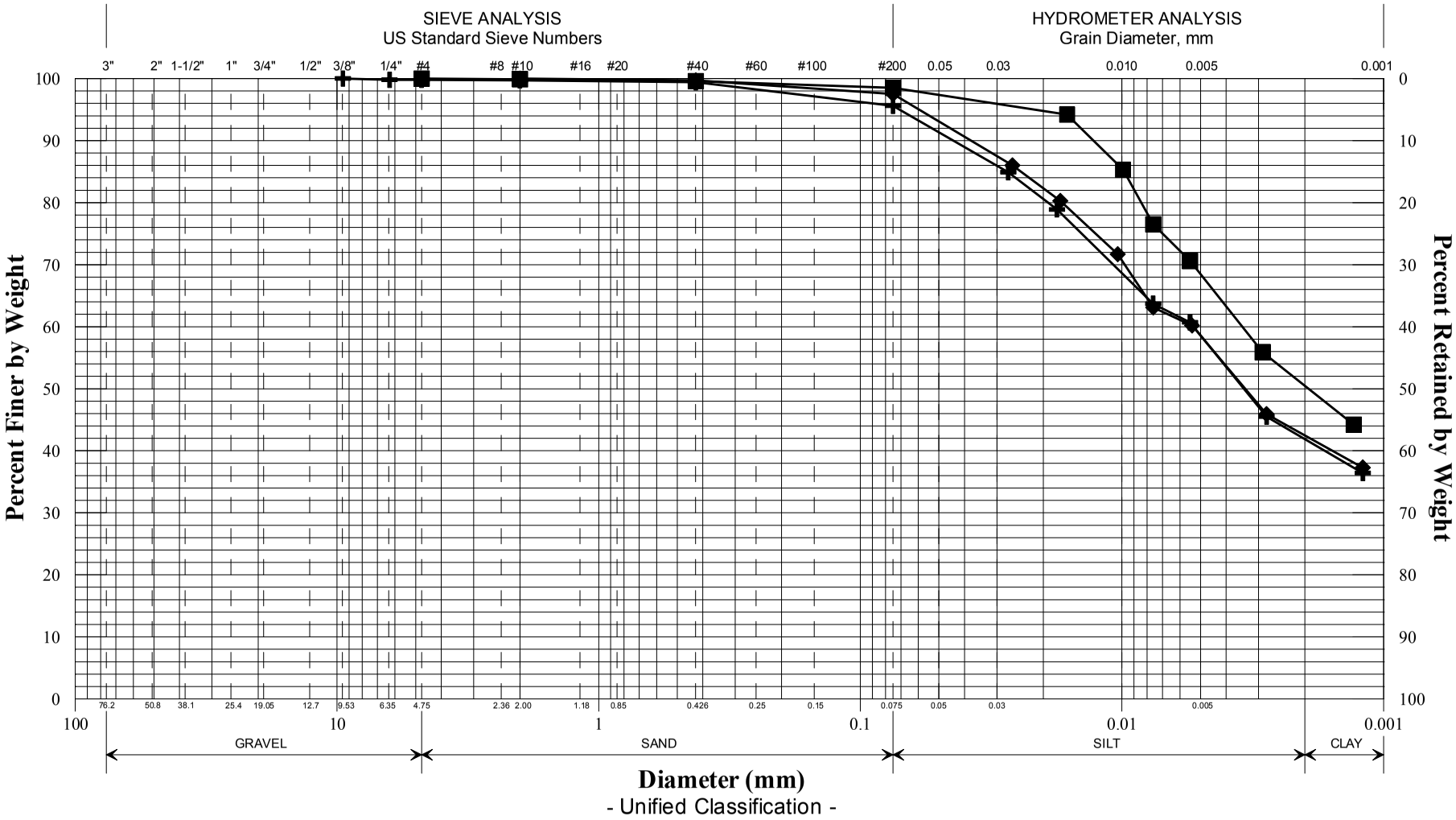


	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-BR-BOO-101/2D	11+52.4	57.7 LT	5.0-7.0	SILT, some clay, trace sand.	66.8	38	25	13
◆	BB-BR-BOO-101/4D	11+52.4	57.7 LT	15.0-17.0	Clayey SILT, trace sand.	44.2	28	20	8
■	BB-BR-BOO-102/3D	12+79.7	58.6 LT	12.0-14.0	Clayey SILT, trace sand.	41.7	32	19	13
●	BB-BR-BOO-102/5D	12+79.7	58.6 LT	20.0-22.0	Clayey SILT, trace sand.	42.8	31	22	9
▲									
✕									

PIN
012630.00
Town
Boothbay
Reported by/Date
WHITE, TERRY A 11/2/2005

SHEET 1

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

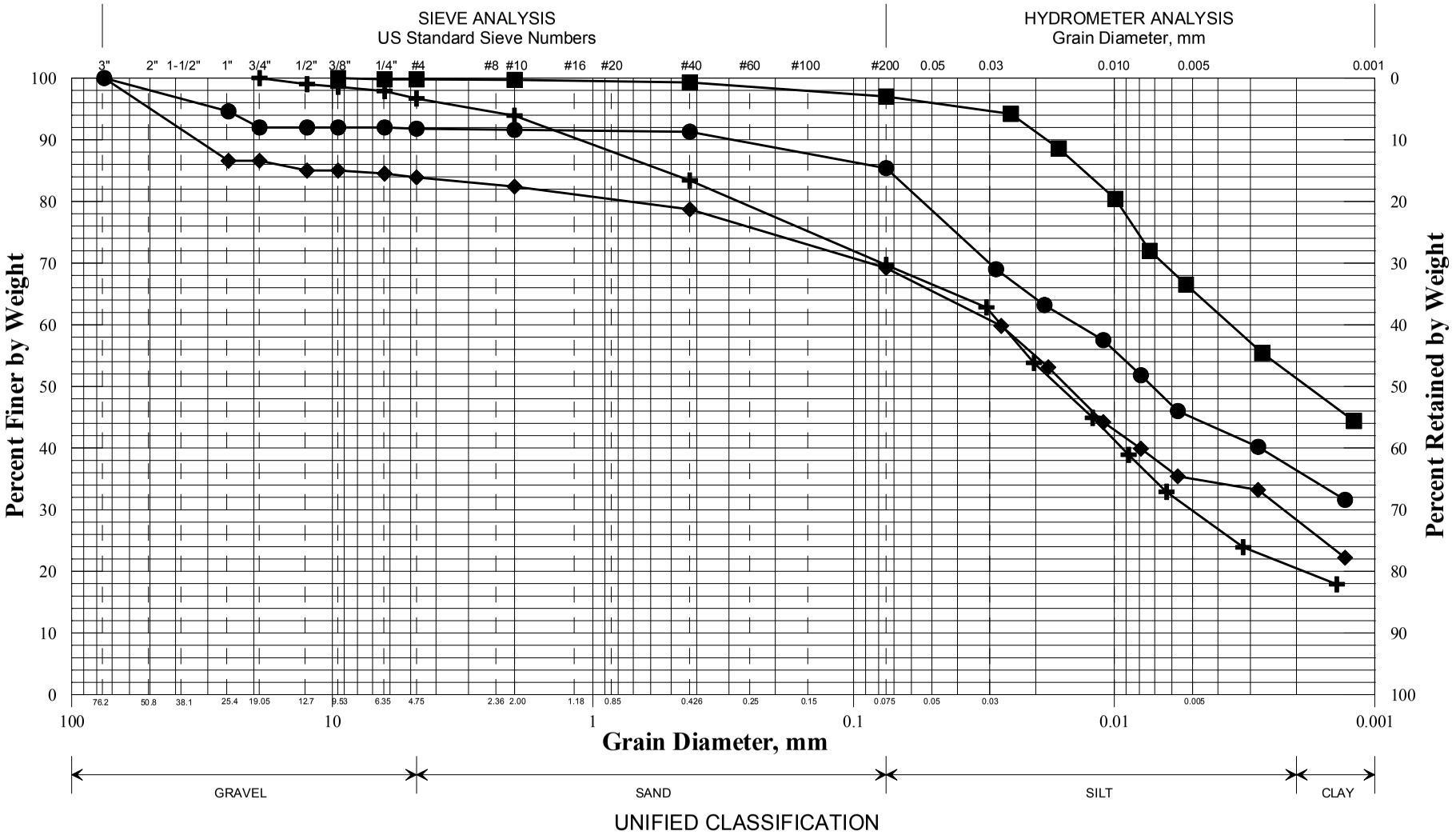


	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-BR-BOO-103/2D(A)	14+17.9	57.5 LT	6.0-7.5	Clayey SILT, trace sand, trace gravel.	26.8	24	16	8
◆	BB-BR-BOO-103/2D(B)	14+17.9	57.5 LT	7.5-8.0	Clayey SILT, trace sand.	23.6			
■	BB-BR-BOO-103/5D	14+17.9	57.5 LT	21.0-23.0	Silty CLAY, trace sand.	43.0	34	22	12
●									
▲									
✕									

PIN
012630.00
Town
Boothbay
Reported by/Date
WHITE, TERRY A 11/2/2005

SHEET 2

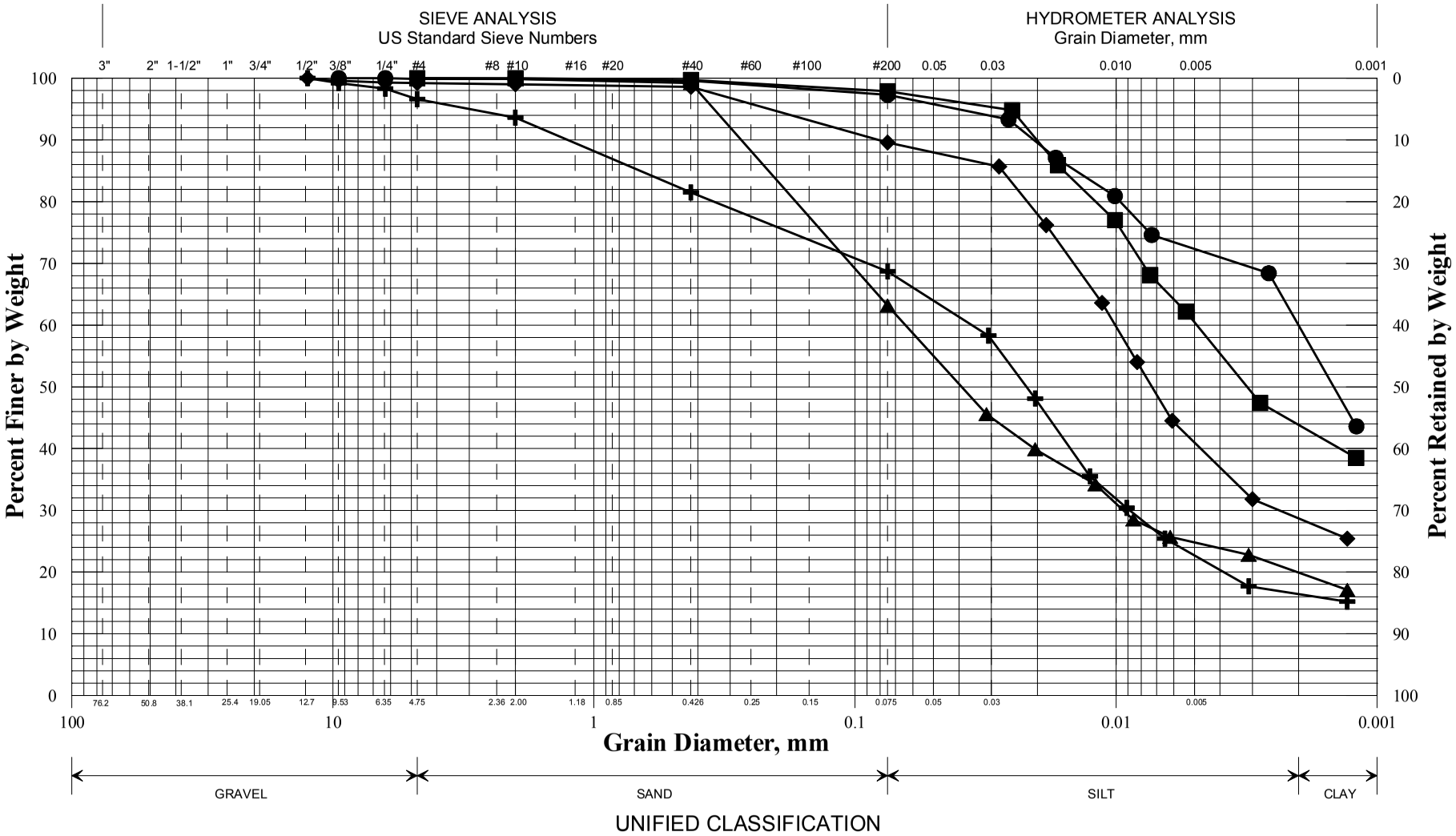
State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-BBR-202A/1D	11+07	5.5 LT	0.0-2.0	SILT, some sand, little clay, trace gravel.	75.9			
◆	BB-BBR-202/2D	11+11	2.49 LT	9.0-11.0	SILT, some clay, little gravel, little sand.	31.4			
■	BB-BBR-202/4D	11+11	2.49 LT	18.0-20.0	Silty CLAY, trace sand, trace gravel.	31.6	38	23	15
●	BB-BBR-202/6D	11+11	2.49 LT	28.0-30.0	Clayey SILT, trace gravel, trace sand.	30.4	28	15	13
▲									
✕									

PIN
012630.00
Town
Boothbay
Reported by/Date
WHITE, TERRY A 10/18/2007

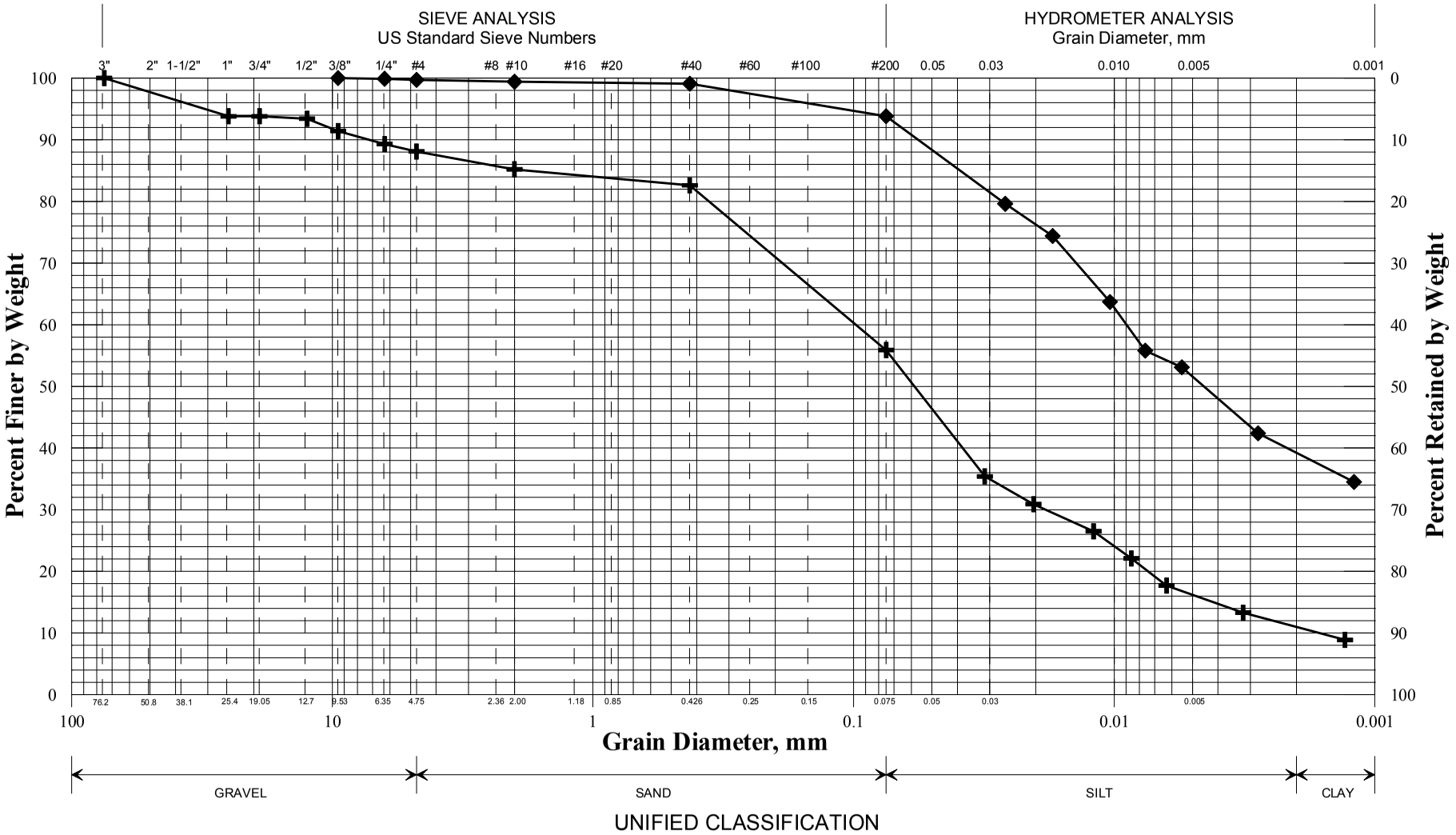
State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-BBR-203/1D	11+95	0.1 LT	0.0-2.0	SILT, some sand, little clay, trace gravel.	93.1			
◆	BB-BBR-203/2D	11+95	0.1 LT	8.0-10.0	SILT, some clay, trace sand, trace gravel.	13.9			
■	BB-BBR-203/5D	11+95	0.1 LT	23.0-25.0	Clayey SILT, trace sand.	44.5	37	25	12
●	BB-BBR-203/9D	11+95	0.1 LT	43.0-45.0	Silty CLAY, trace sand.	46.6	35	11	24
▲	BB-BBR-203/11D	11+95	0.1 LT	53.0-55.0	Sandy SILT, little clay.	26.1			
✕									

PIN
012630.00
Town
Boothbay
Reported by/Date
WHITE, TERRY A 10/18/2007

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

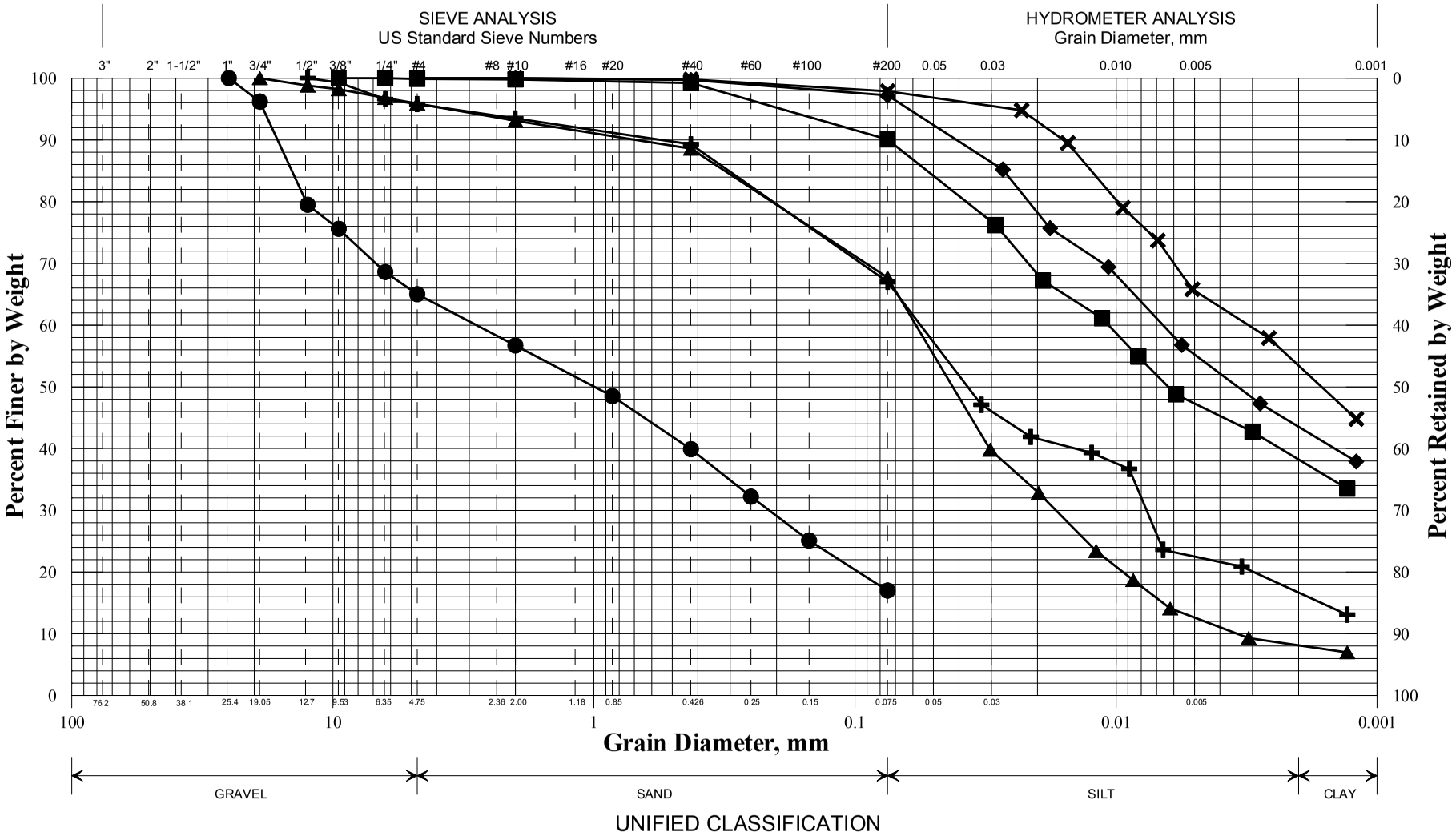


UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-BBR-204/1D	12+86.8	3.9 RT	7.0-9.0	SILT, some sand, little gravel, little clay.	39.6			
◆	BB-BBR-204/3D	12+86.8	3.9 RT	19.0-21.0	Clayey SILT, trace sand, trace gravel.	36.0	27	14	13
■									
●									
▲									
✕									

PIN
012630.00
Town
Boothbay
Reported by/Date
WHITE, TERRY A 10/18/2007

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

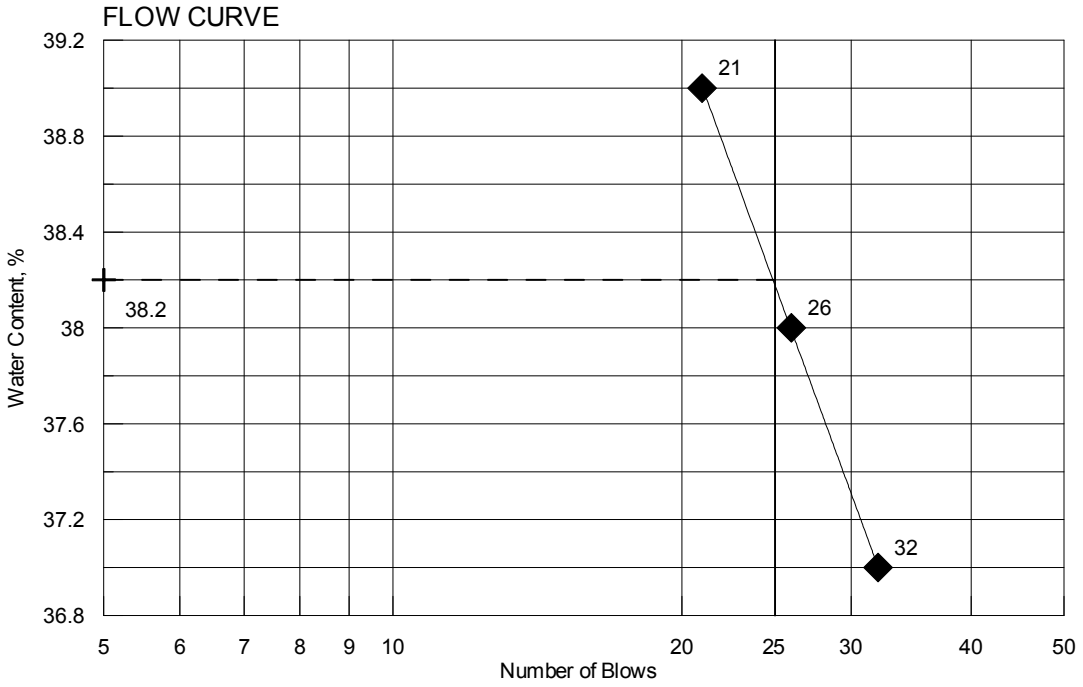


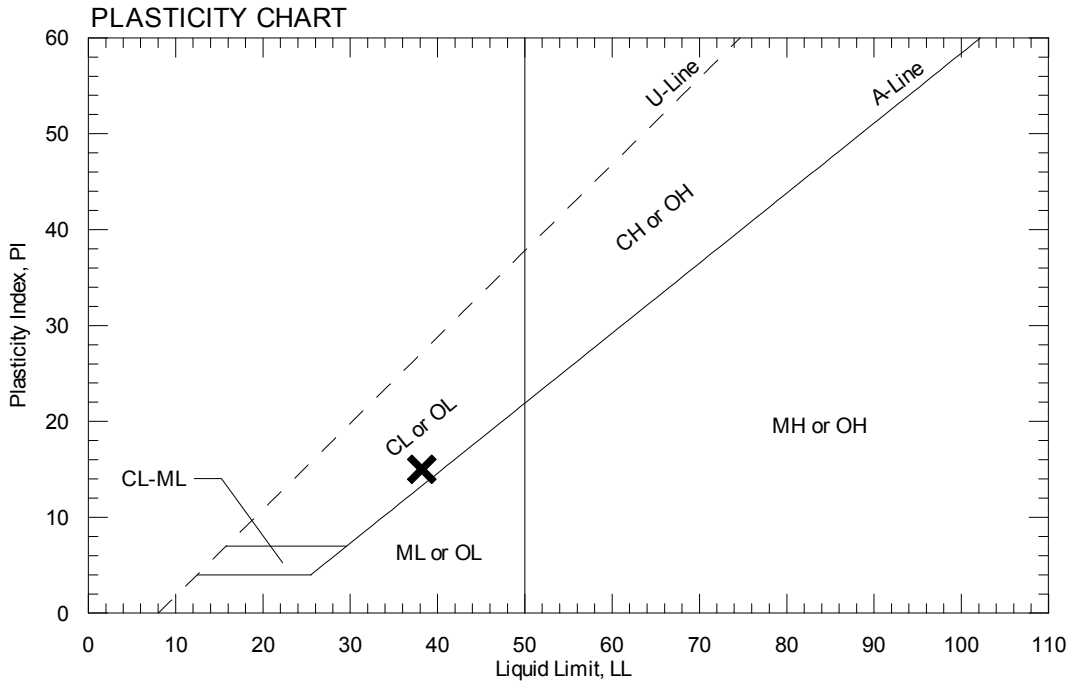
	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-BBR-205/1D	13+77.9	0.5 RT	5.0-7.0	SILT, some sand, little clay, trace gravel.	53.9			
◆	BB-BBR-205/2D	13+77.9	0.5 RT	10.0-12.0	Clayey SILT, trace sand.	23.4	33	15	18
■	BB-BBR-205/6D	13+77	0.5 RT	28.0-30.0	Clayey SILT, trace sand, trace gravel.	36.6	25	20	5
●	BB-BBR-205/7D	13+77.9	0.5 RT	33.0-35.0	SAND, some gravel, little silt.	10.6			
▲	BB-BBR-206/2D	14+68.1	2.5 RT	5.0-7.0	SILT, some sand, trace clay, trace gravel.	44.2			
✕	BB-BBR-206/4D	14+68.1	2.5 RT	14.0-16.0	Silty CLAY, trace sand.	31.2	37	23	14

PIN
012630.00
Town
Boothbay
Reported by/Date
WHITE, TERRY A 10/18/2007

State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

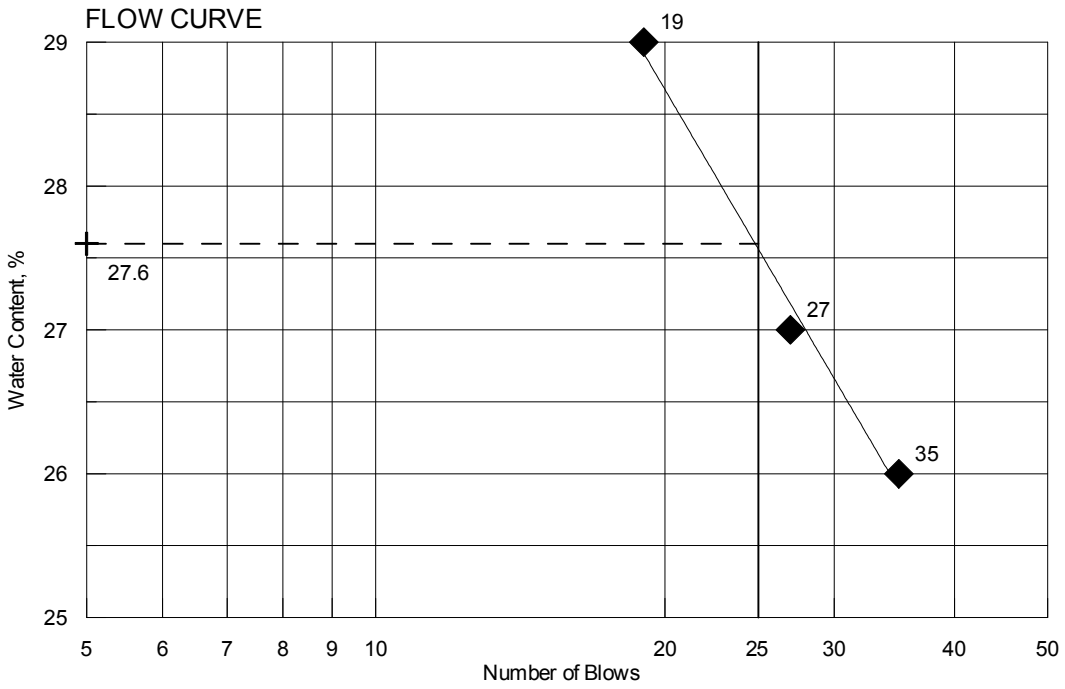
TOWN	Boothbay	Reference No.	209951
PIN	012630.00	Water Content, %	31.6
Sampled	8/21/2007	Plastic Limit	23
Boring No./Sample No.	BB-BBR-202/4D	Liquid Limit	38
Station	11+11	Plasticity Index	15
Depth	18.0-20.0	Tested By	BBURR

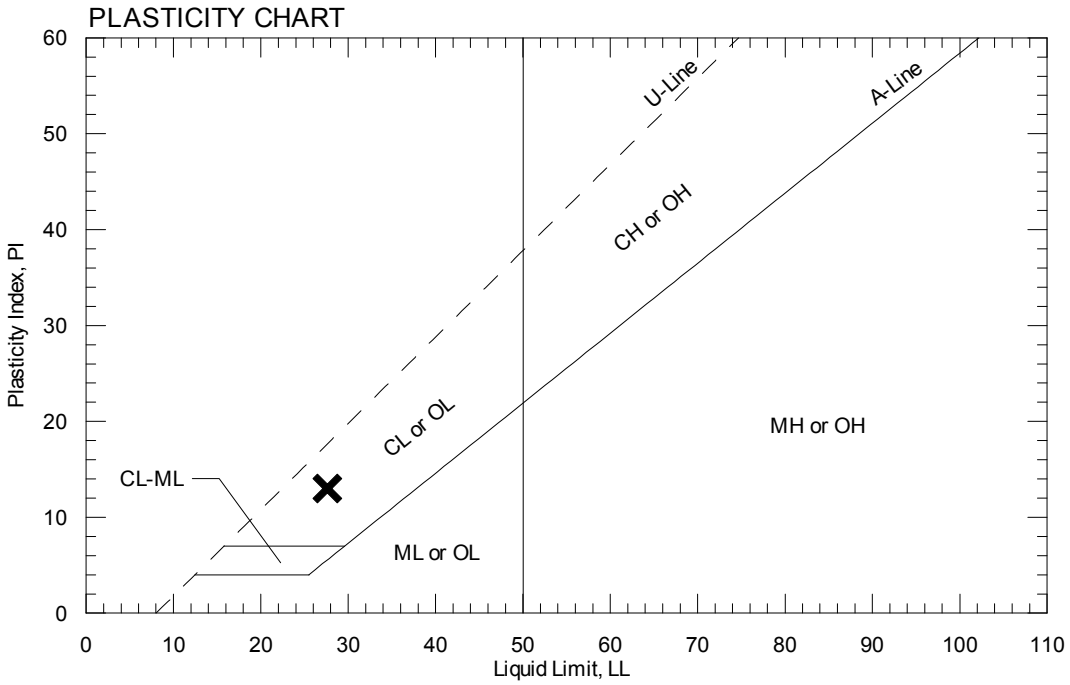




State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

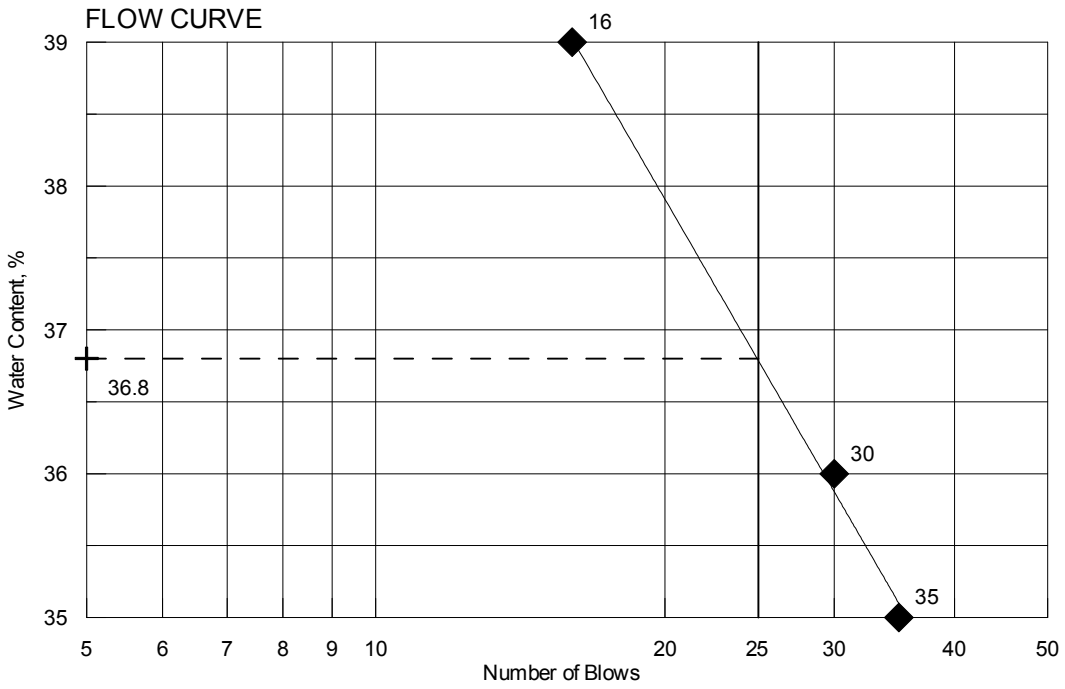
TOWN	Boothbay	Reference No.	209952
PIN	012630.00	Water Content, %	30.4
Sampled	8/21/2007	Plastic Limit	15
Boring No./Sample No.	BB-BBR-202/6D	Liquid Limit	28
Station	11+11	Plasticity Index	13
Depth	28.0-30.0	Tested By	BBURR

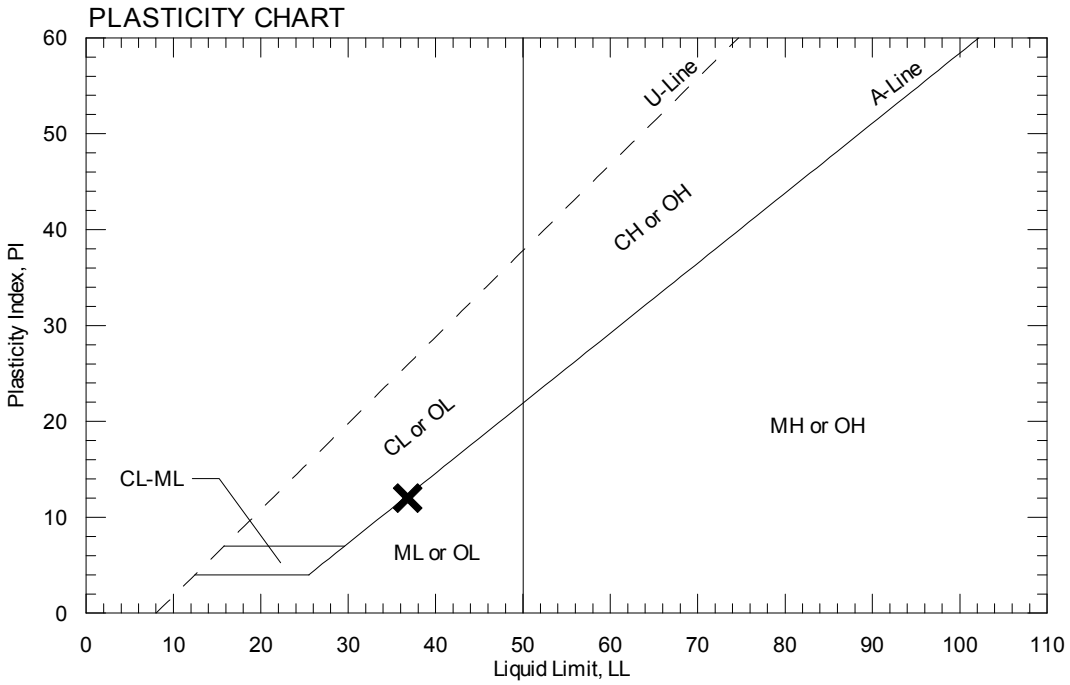




State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

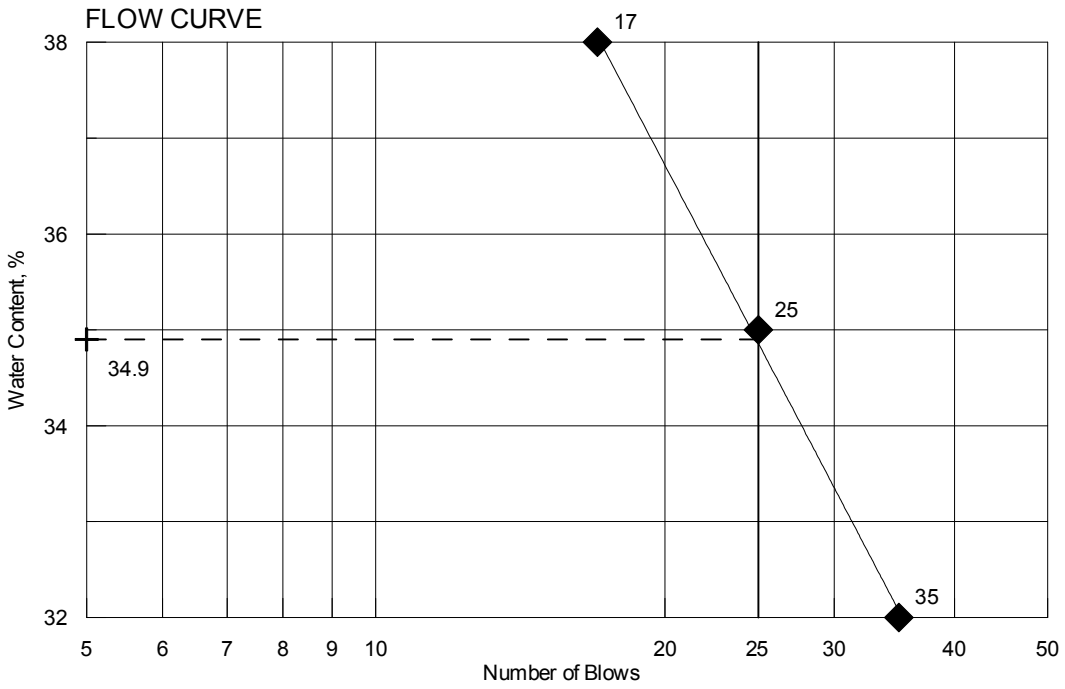
TOWN	Boothbay	Reference No.	209958
PIN	012630.00	Water Content, %	44.5
Sampled	8/23/2007	Plastic Limit	25
Boring No./Sample No.	BB-BBR-203/5D	Liquid Limit	37
Station	11+95	Plasticity Index	12
Depth	23.0-25.0	Tested By	BBURR

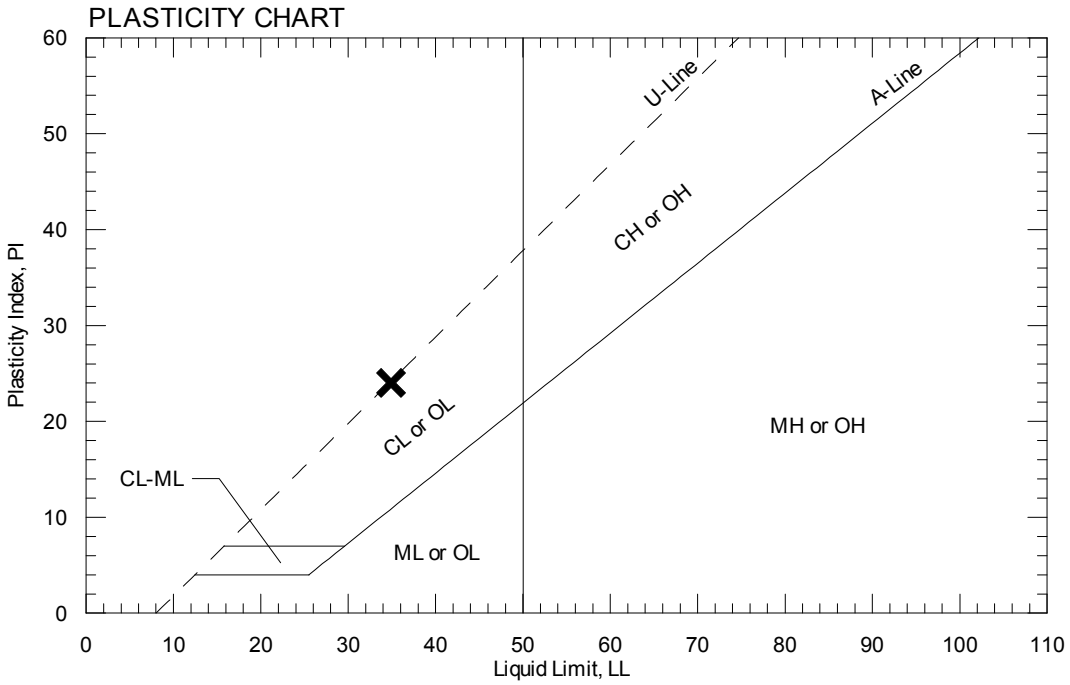




State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

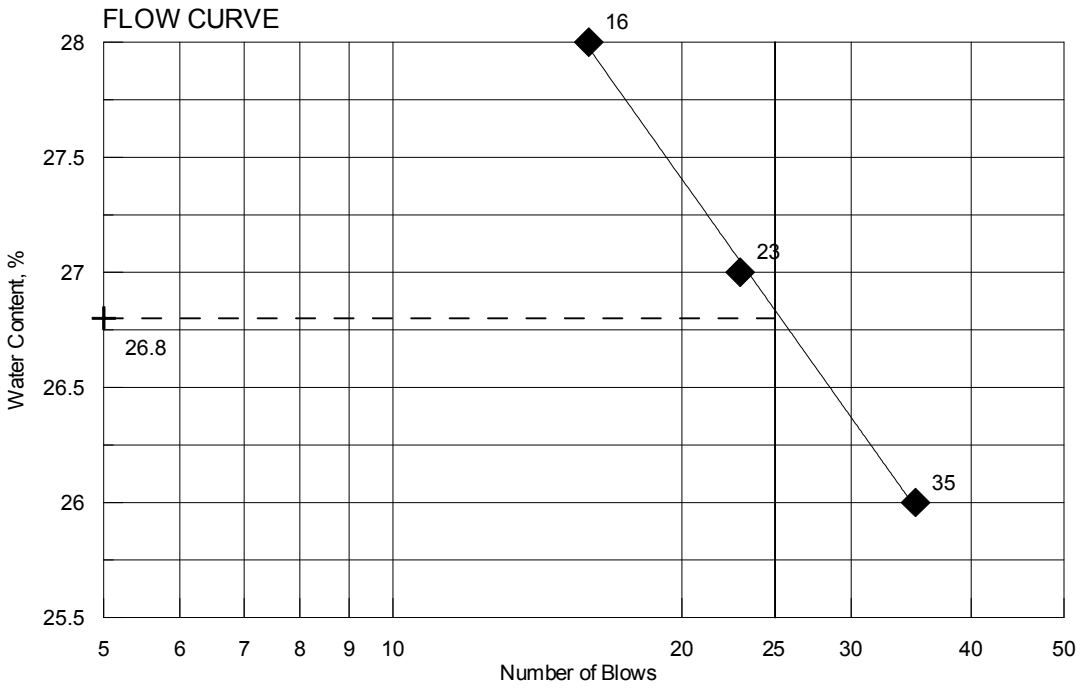
TOWN	Boothbay	Reference No.	209955
PIN	012630.00	Water Content, %	46.6
Sampled	8/21/2007	Plastic Limit	11
Boring No./Sample No.	BB-BBR-203/9D	Liquid Limit	35
Station	11+95	Plasticity Index	24
Depth	43.0-45.0	Tested By	BBURR

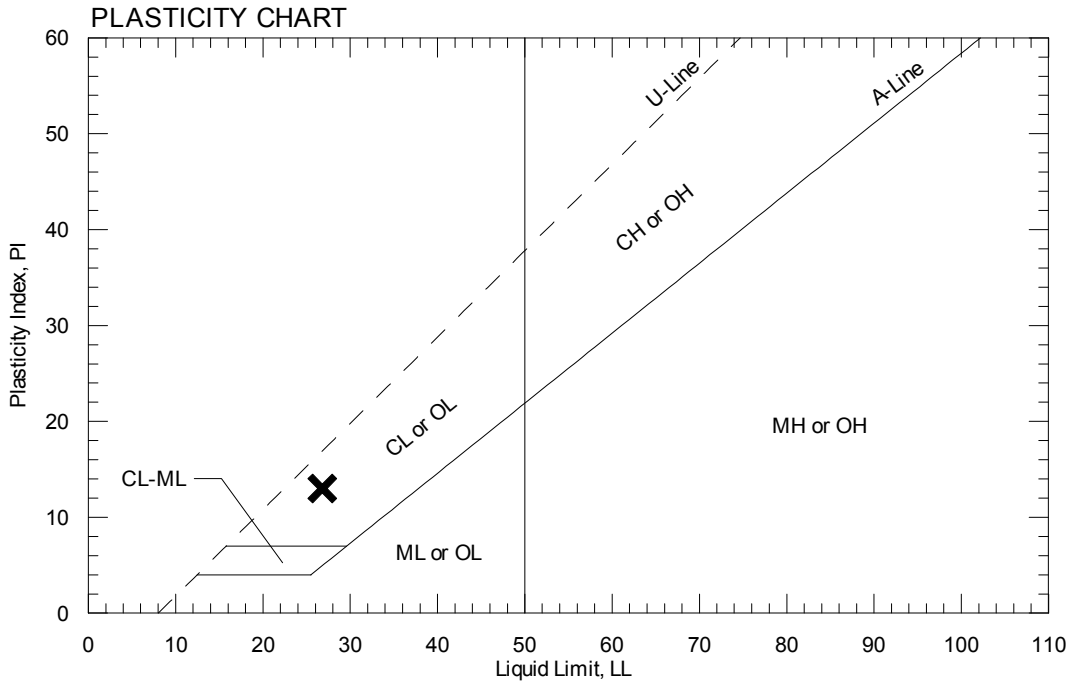




State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

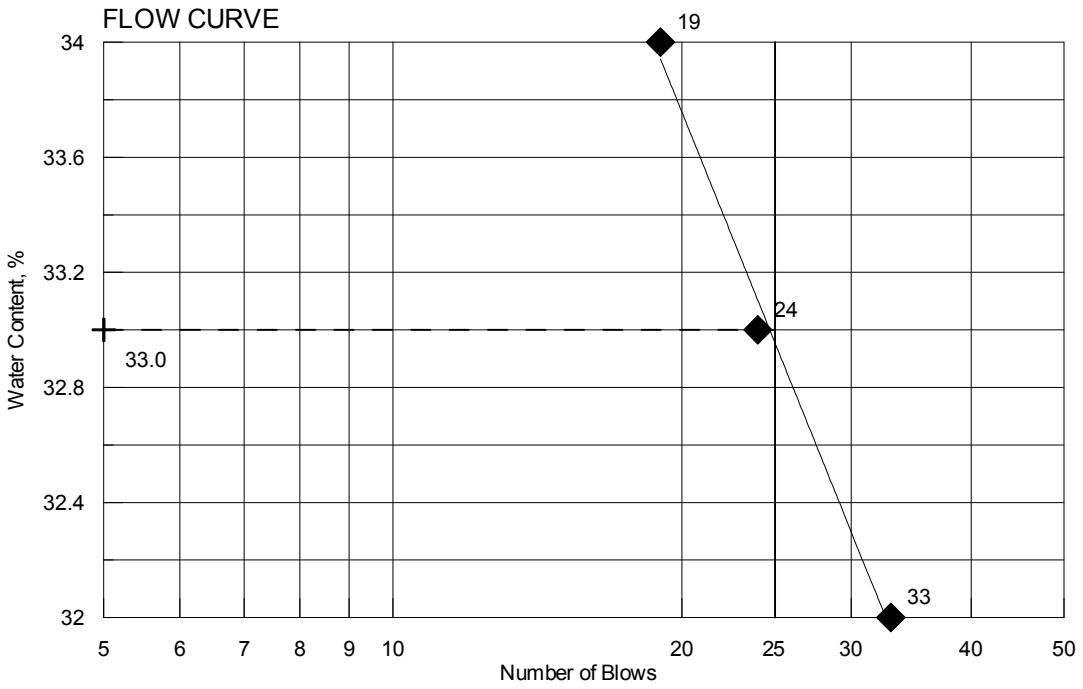
TOWN	Boothbay	Reference No.	209967
PIN	012630.00	Water Content, %	36
Sampled	9/5/2007	Plastic Limit	14
Boring No./Sample No.	BB-BBR-204/3D	Liquid Limit	27
Station	12+86.8	Plasticity Index	13
Depth	19.0-21.0	Tested By	BBURR

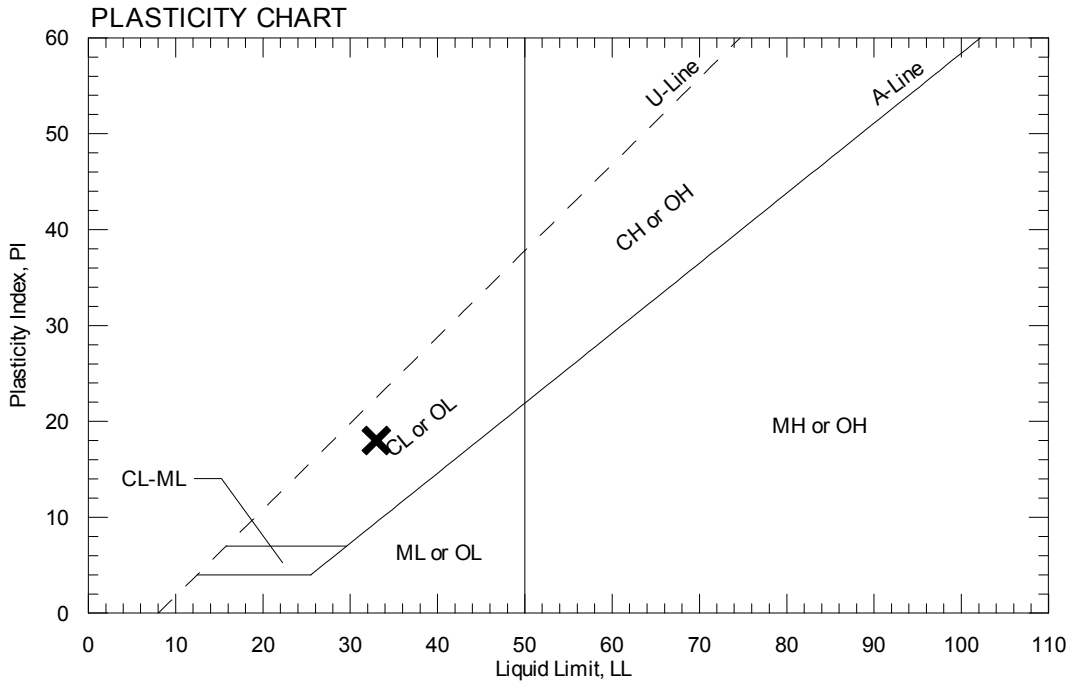




State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

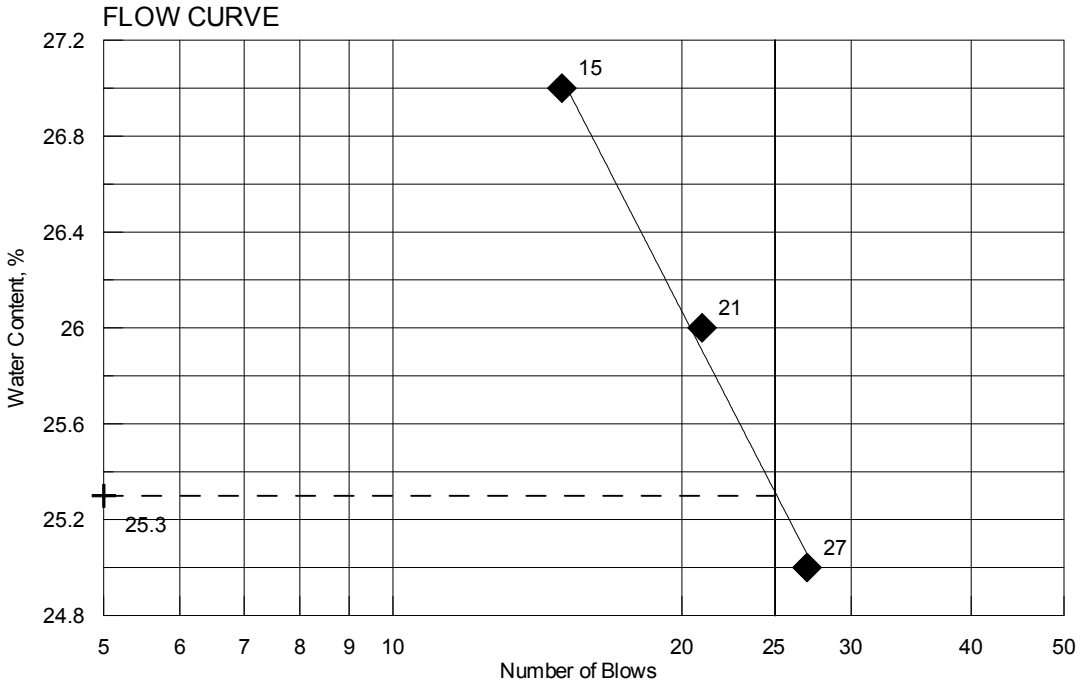
TOWN	Boothbay	Reference No.	209959
PIN	012630.00	Water Content, %	23.4
Sampled	8/28/2007	Plastic Limit	15
Boring No./Sample No.	BB-BBR-205/2D	Liquid Limit	33
Station	13+77.9	Plasticity Index	18
Depth	10.0-12.0	Tested By	BBURR

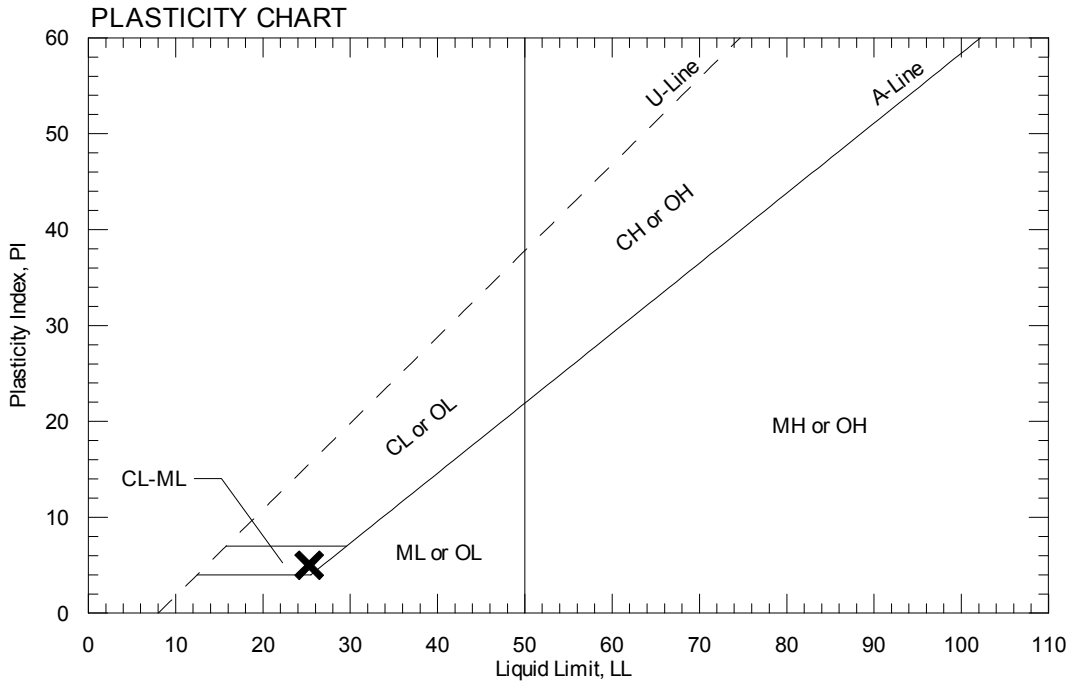




State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

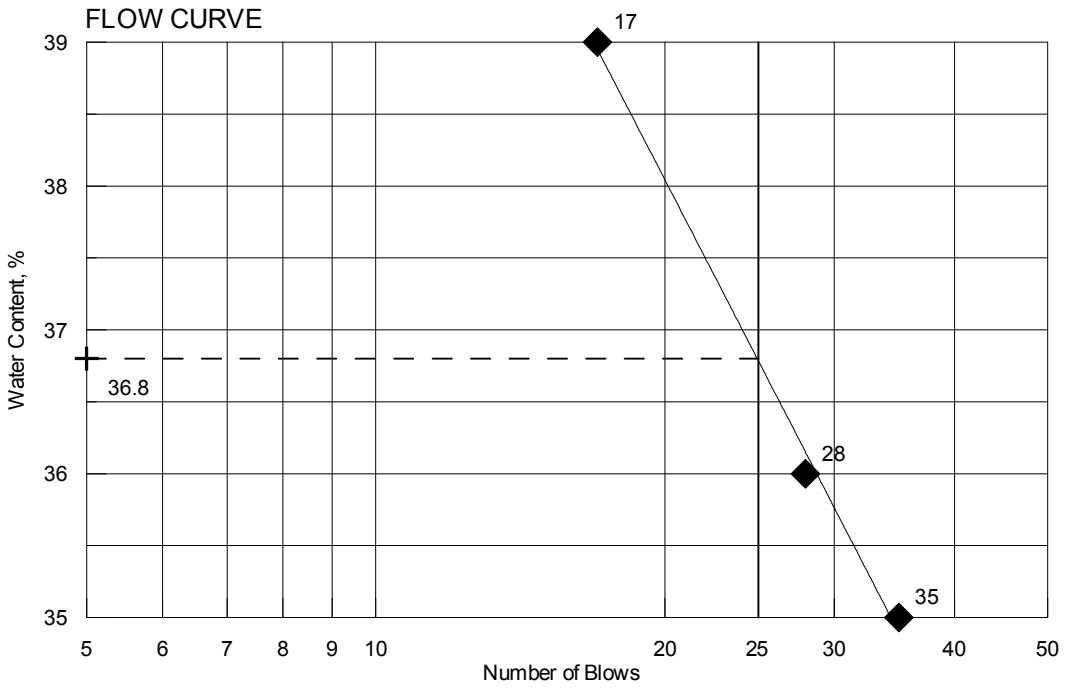
TOWN	Boothbay	Reference No.	209962
PIN	012630.00	Water Content, %	36.6
Sampled	8/28/2007	Plastic Limit	20
Boring No./Sample No.	BB-BBR-205/6D	Liquid Limit	25
Station	13+77	Plasticity Index	5
Depth	28.0-30.0	Tested By	BBURR

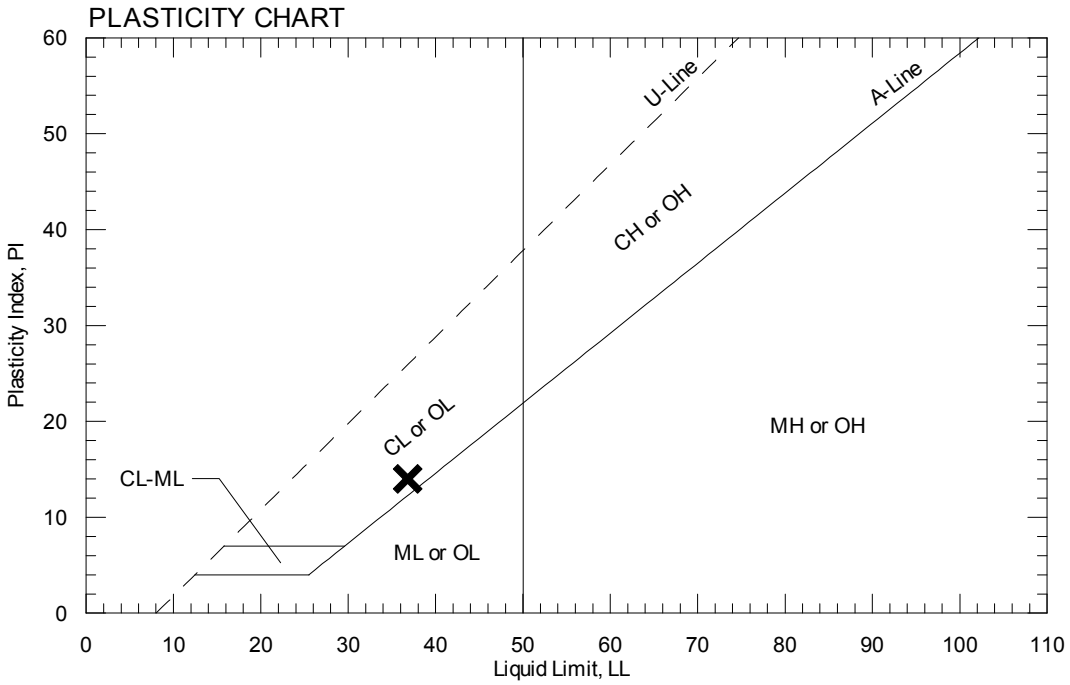




State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

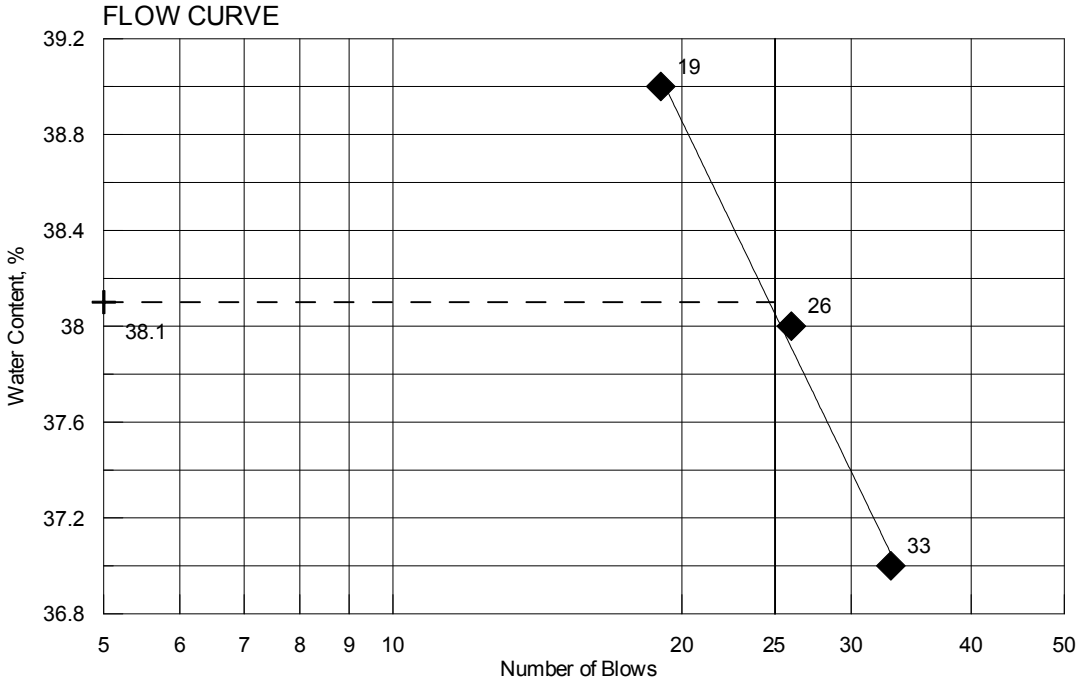
TOWN	Boothbay	Reference No.	209965
PIN	012630.00	Water Content, %	31.2
Sampled	8/29/2007	Plastic Limit	23
Boring No./Sample No.	BB-BBR-206/4D	Liquid Limit	37
Station	14+68.1	Plasticity Index	14
Depth	14.0-16.0	Tested By	BBURR

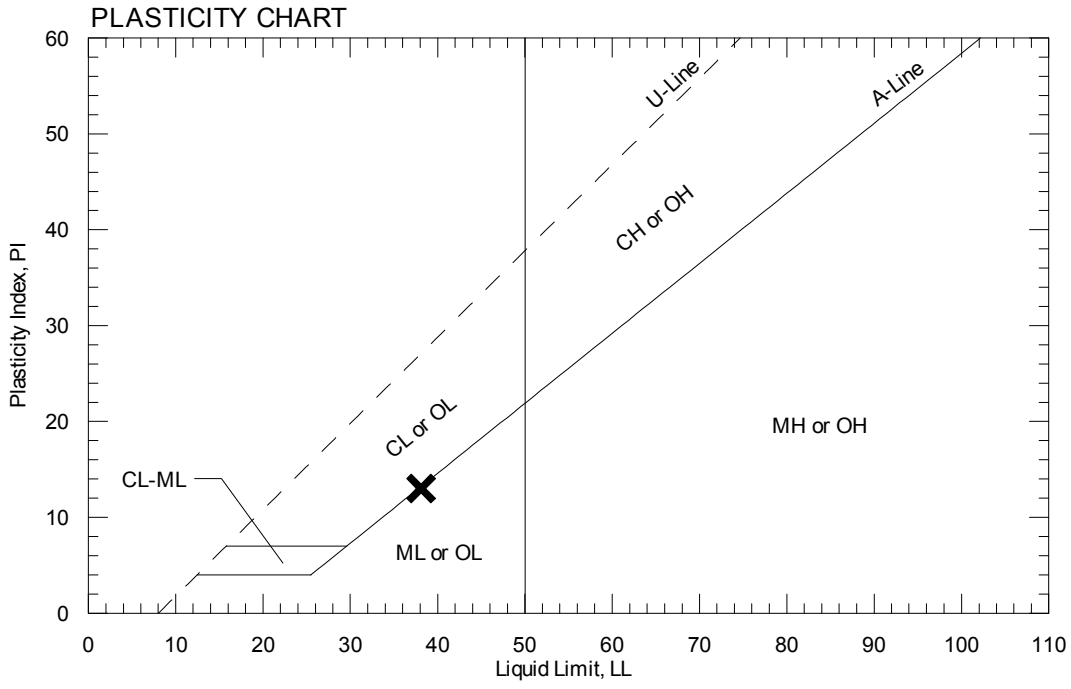




State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

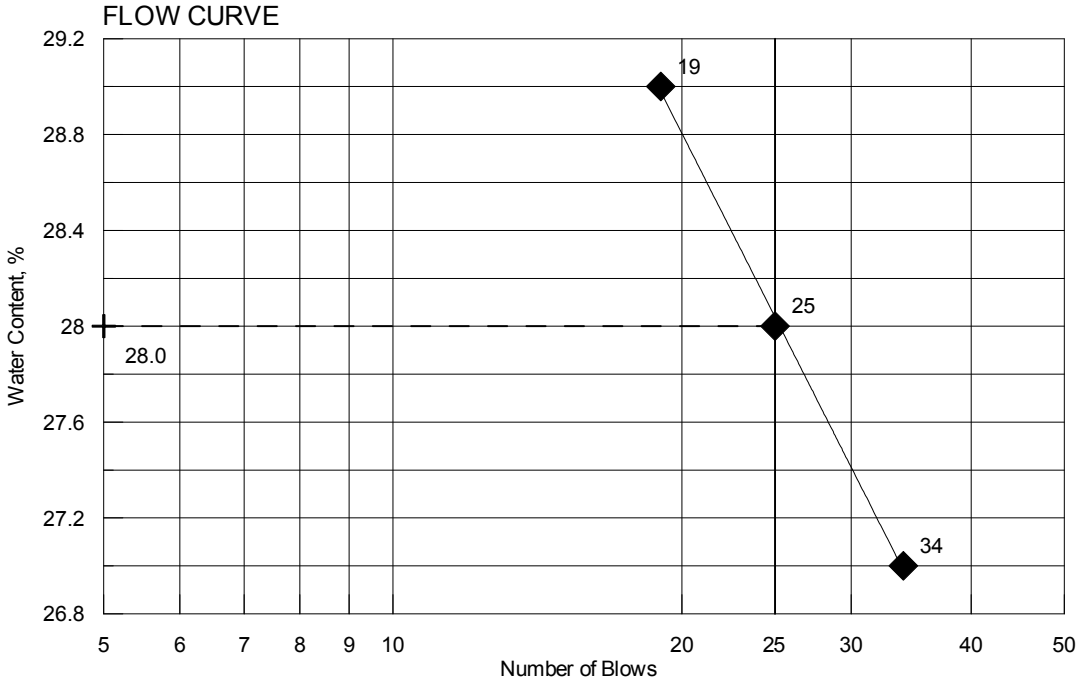
TOWN	Boothbay	Reference No.	181934
PIN	012630.00	Water Content, %	66.8
Sampled	8/24/2005	Plastic Limit	25
Boring No./Sample No.	BB-BR-BOO-101/2D	Liquid Limit	38
Station	11+52.4	Plasticity Index	13
Depth	5.0-7.0	Tested By	BBURR

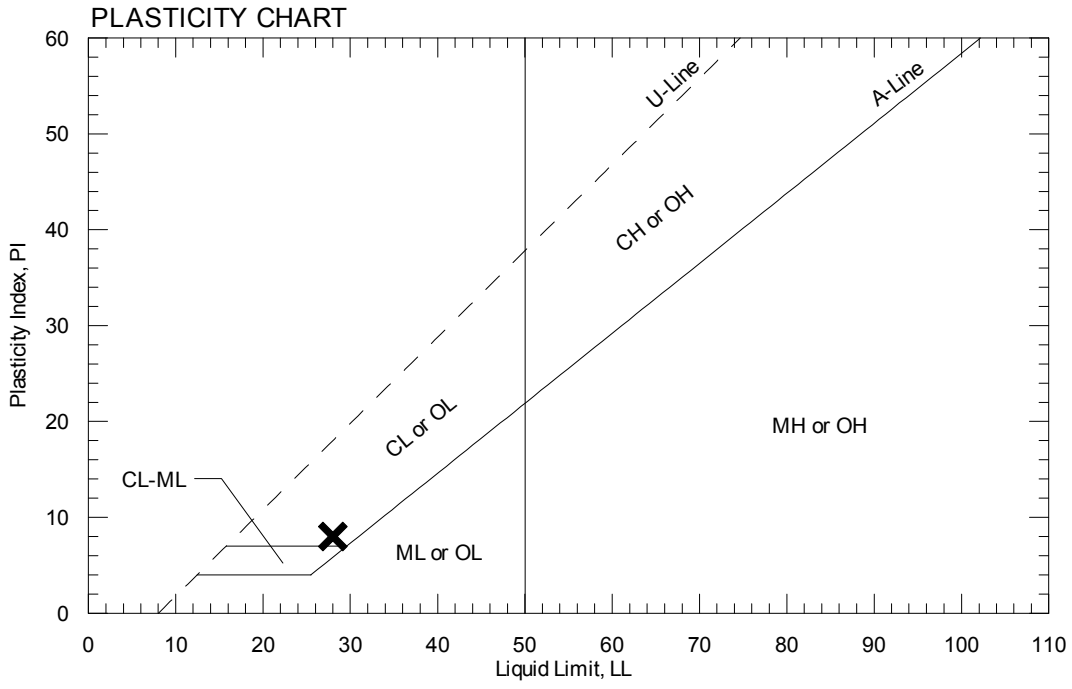




State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

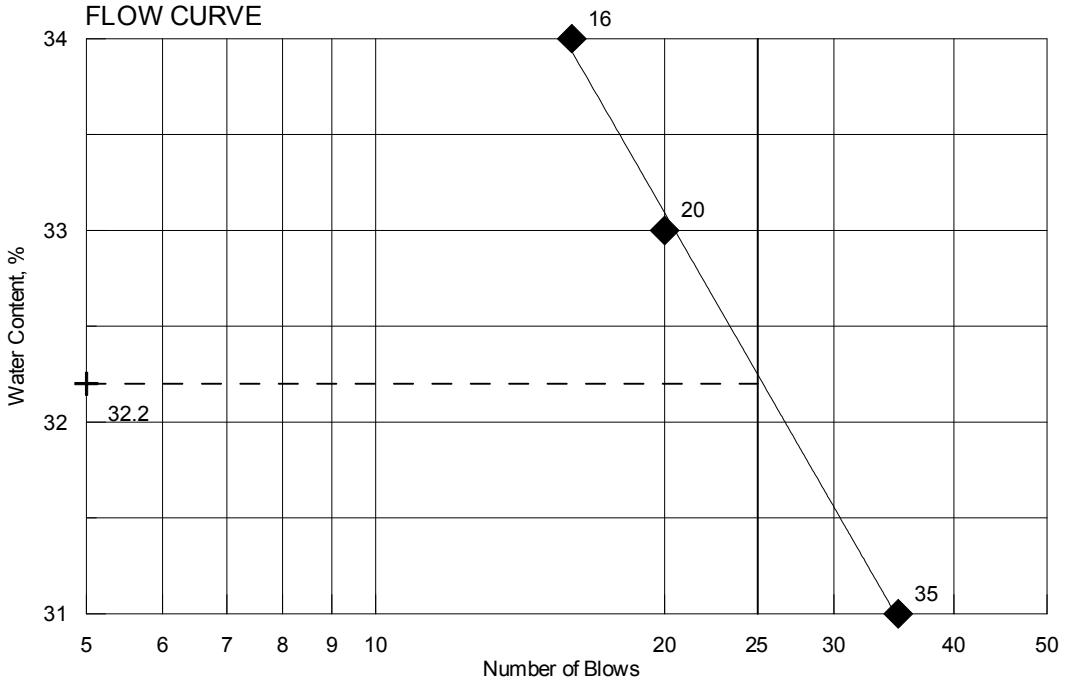
TOWN	Boothbay	Reference No.	181935
PIN	012630.00	Water Content, %	44.2
Sampled	8/24/2005	Plastic Limit	20
Boring No./Sample No.	BB-BR-BOO-101/4D	Liquid Limit	28
Station	11.52.4	Plasticity Index	8
Depth	15.0-17.0	Tested By	BBURR

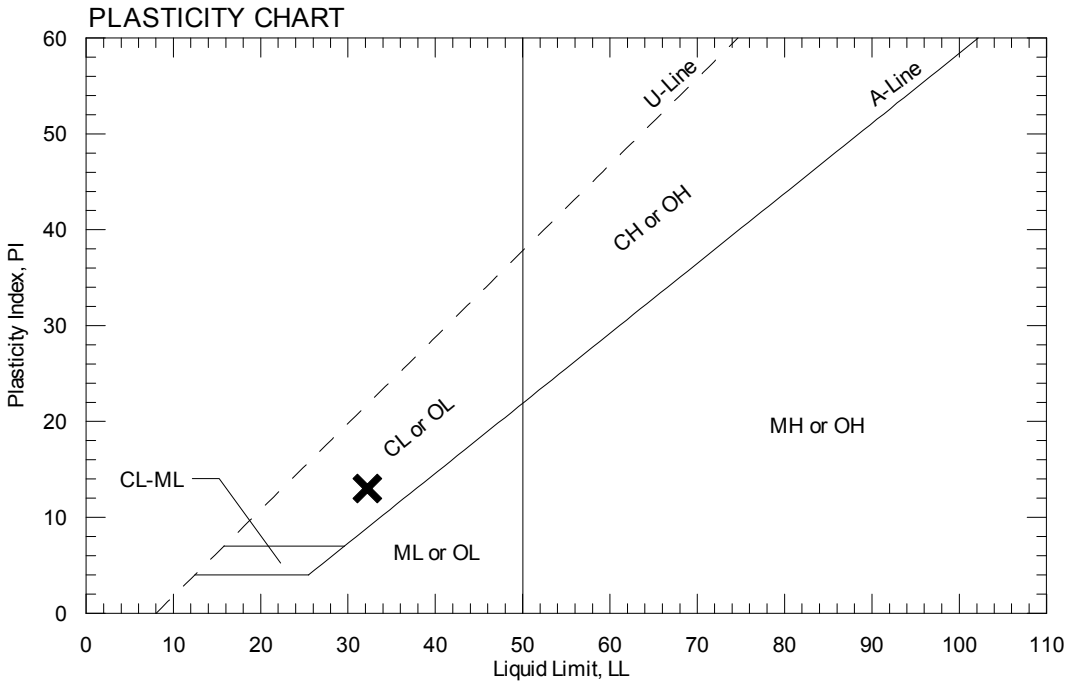




State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

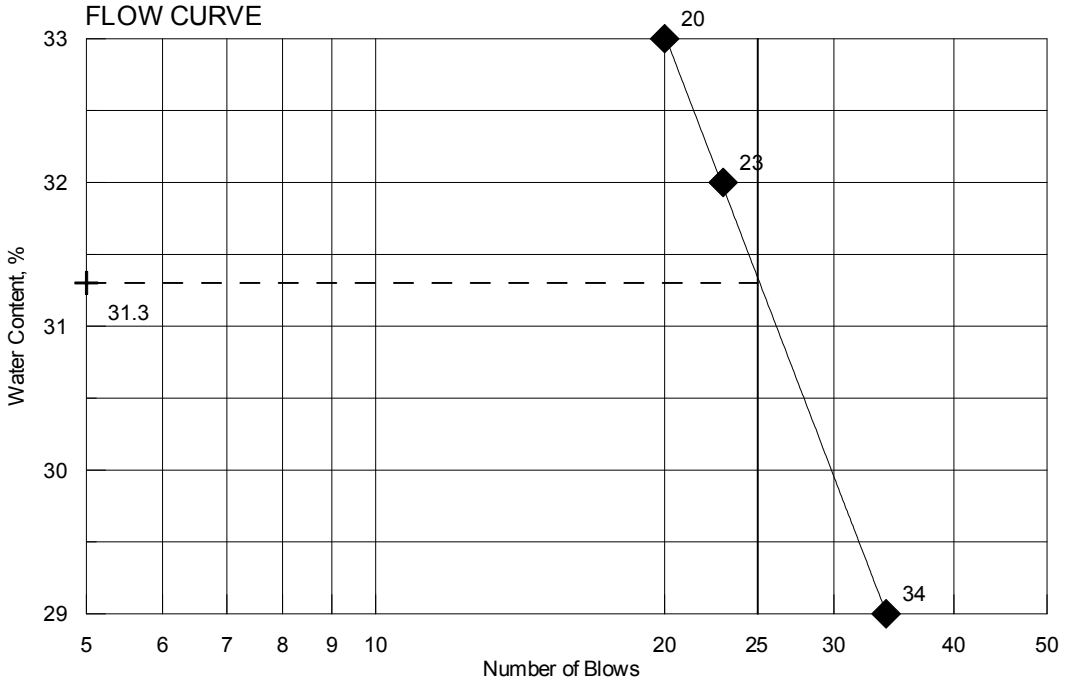
TOWN	Boothbay	Reference No.	181936
PIN	012630.00	Water Content, %	41.7
Sampled	8/23/2005	Plastic Limit	19
Boring No./Sample No.	BB-BR-BOO-102/3D	Liquid Limit	32
Station	12+79.7	Plasticity Index	13
Depth	12.0-14.0	Tested By	BBURR

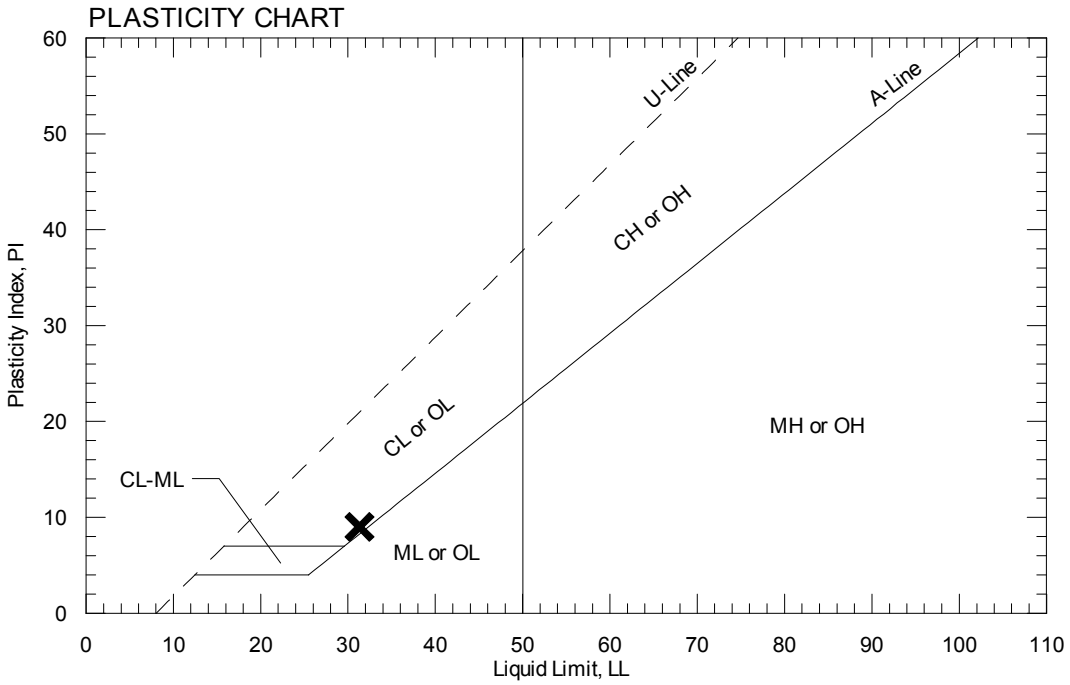




State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

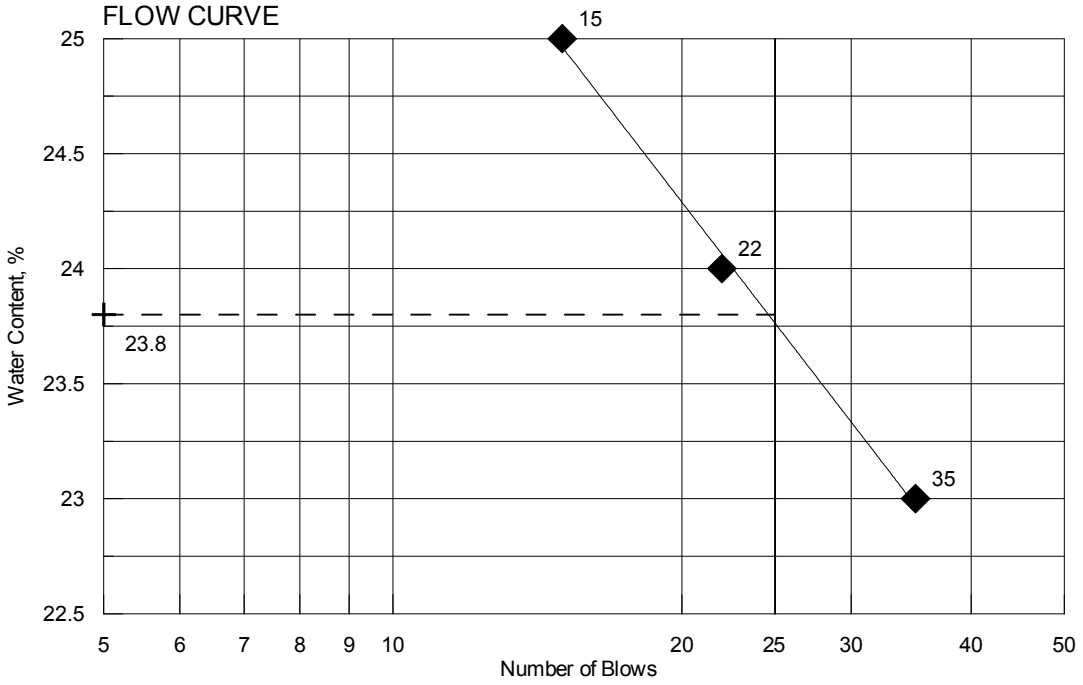
TOWN	Boothbay	Reference No.	181937
PIN	012630.00	Water Content, %	42.8
Sampled	8/23/2005	Plastic Limit	22
Boring No./Sample No.	BB-BR-BOO-102/5D	Liquid Limit	31
Station	12+79.7	Plasticity Index	9
Depth	20.0-22.0	Tested By	BBURR

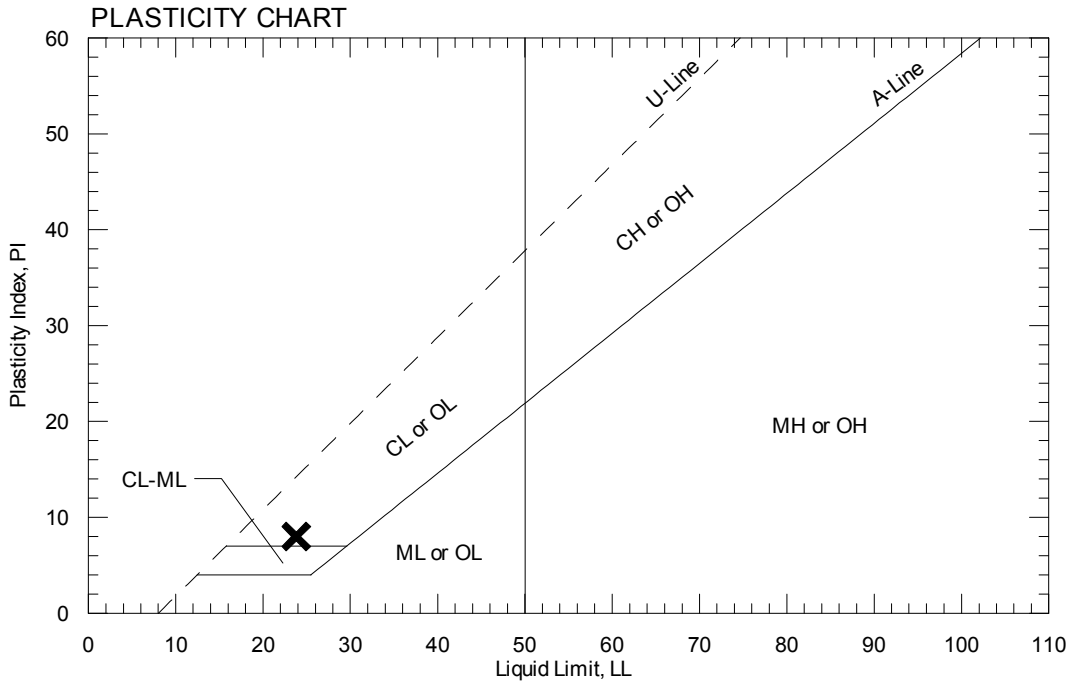




State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

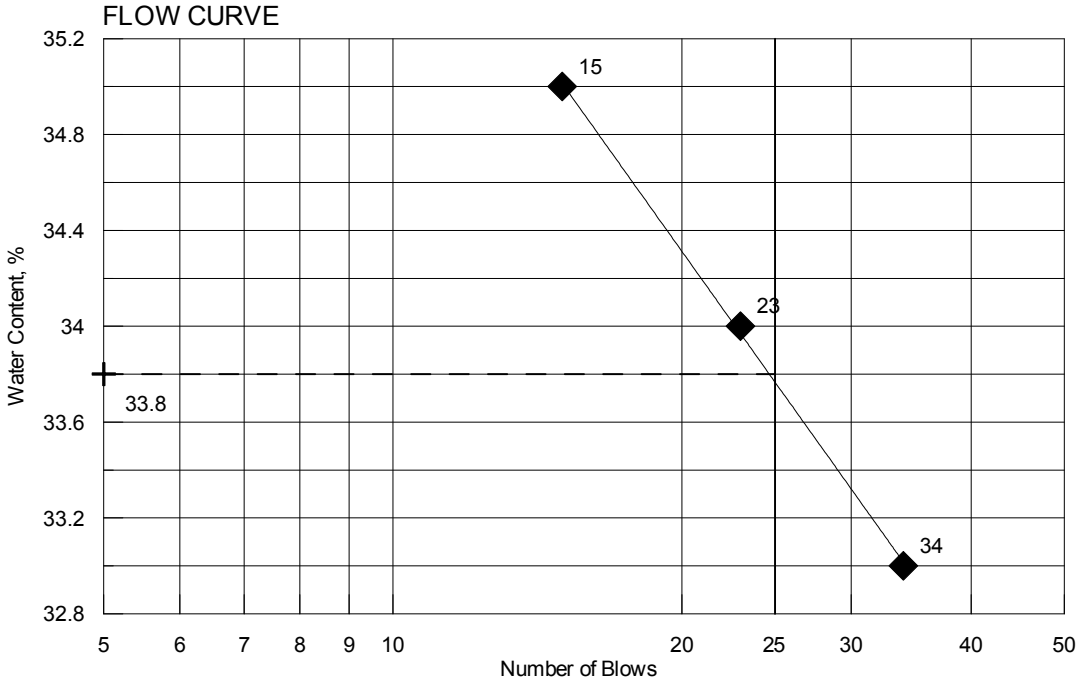
TOWN	Boothbay	Reference No.	181938
PIN	012630.00	Water Content, %	26.8
Sampled	8/22/2005	Plastic Limit	16
Boring No./Sample No.	BB-BR-BOO-103/2D(A)	Liquid Limit	24
Station	14+17.9	Plasticity Index	8
Depth	6.0-7.5	Tested By	BBURR

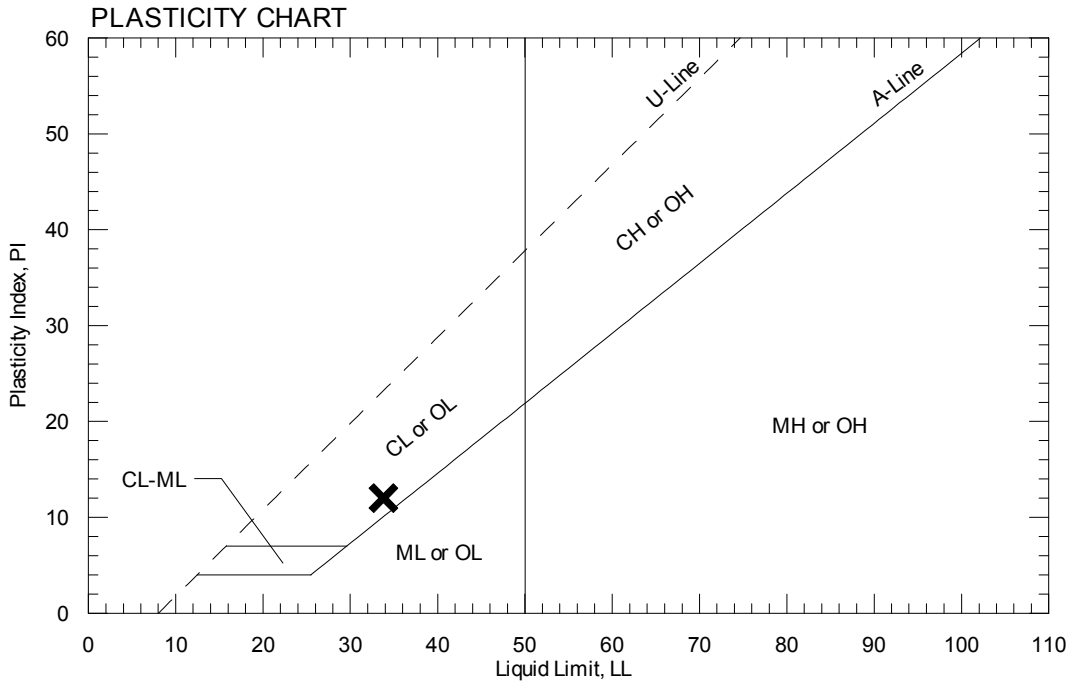




State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

TOWN	Boothbay	Reference No.	181940
PIN	012630.00	Water Content, %	43
Sampled	8/22/2005	Plastic Limit	22
Boring No./Sample No.	BB-BR-BOO-103/5D	Liquid Limit	34
Station	14+17.9	Plasticity Index	12
Depth	21.0-23.0	Tested By	BBURR





Appendix D

Calculations

	Factored Geotechnical Resistance (φ=0.45) (kips) *(Kulhawy and Goodman, RQD method)*				Factored Geotechnical Resistance (φ=0.45) (kips) *(Kulhawy and Goodman, RQD method)*			
	0.5-inch wall Pipe Piles				**5/8 inch wall Pipe Piles**			
	24-in	26-in	28-in	30-in	24-in	26"	28"	30"
Pier 1	179	194	209	224	222	241	260	279
Pier 2	294	310	325	340	338	357	376	395
Pier 3	415	450	485	520	515	560	604	648
Pier 4	221	240	259	278	275	299	322	346
Pier 5	221	240	259	278	275	299	322	346
Average 4 lower values	229	246	263	280	278	299	320	342

***Recommendation** : Use 300 kip for factored Geotechnical Axial Pile Resistance for a 24-inch dia, 5/8 wall pipe pile*

Table 1. Factored Geotechnical Axial Pile Resistances

	Prelim Estimate of Fixity w/ NO lateral loads (feet) *(FBPier analysis required pending loads)*				Prelim Estimate of Fixity w/ NO lateral loads (feet) *(FBPier analysis required pending loads)*				Depth of overburden (feet)	Preliminary Estimate of Fixity achieved assuming no scour?
	0.5-inch wall Pipe Piles				**5/8 inch wall Pipe Piles**					
	24-in	26-in	28-in	30-in	24-in	26"	28"	30"		
Pier 1	18	19	20	22	18	20	21	22	45	yes
Pier 2	17	18	20	21	18	19	20	22	81	yes
Pier 3	19	21	22	23	20	21	23	24	23	no
Pier 4	19	20	21	23	19	21	22	23	36	yes
Pier 5	19	20	21	23	19	21	22	23	28	yes

Table 2. Preliminary Design Stage Estimates of Depth to Fixity for Pipe Piles

	Bottom pile Cap Elev	Streambed Elev	Exposed Length
Pier 1	8	-6	14
Pier 2	10	-12	22
Pier 3	11	-16	27
Pier 4	10	-12	22
Pier 5	8	-6	14

Table 3. Exposed Pile Lengths

Bearing Capacity - Abutment 1 and 2 Spread Footing Foundations

Definition of units

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $Mg := 1000 \cdot kg$ $kN := 1000 \cdot newton$ $kPa := \frac{kN}{m^2}$ $ton := 2000 \cdot lbf$ $tsf := \frac{ton}{ft^2}$ $kip := 1000 \cdot lbf$

$ksf := \frac{kip}{ft^2}$ $i := 1 .. 5$

Method 1

Method: *NavFac DM 7.2, May 1983, Foundations and Earth Structures* , Table 1 7.2-142, "Presumptive Values of Allowable Bearing Pressures for Spread Foundations".

Description of Bearing Material:

Abutment 2: Upper 5 feet is discolored, stained GNEISS, moderately hard, moderately weathered, chaotic joints, tight, stained. HIghly fractured RQD = 38%. Core R2 has a RQD of 73% - same as R1 except moderately to slightly weathered and slightly fractured. Use average of upper RQD's for design: 50%
Abutment 1: Upper core is GNEISS, with RQD of 64%, moderately hard, moderately weathered, slightly fractured overall, but with a very fractured zone 1'4" to 2'4" into the core. Use RQD of 50% for design.

Consistency in Place:	moderately hard, slightly weathered, slightly fractured
Allowable Bearing Pressure	Range: 15-25 tsf for "medium hard sound rock"
Tons Per sq ft	Recommended Value for use is 15 tsf (30 ksf)
	Use 30 ksf for service limit state analysis - sizing footing

Method 2

Method: *AASHTO Standard Specifications - 17th Edition, 2002*

Section 4.4.8.1.1 - Competent Rock

Figure 4.4.8.1.1.A - for footings suported on compent rock.

Averaged RQD of rock is 50%

Allowable contact stress 60 tsf (120 ksf)

Method 3

AASHTO Standard Specifications - 17th Edition, 2002

Section 4.4.8.1.2. Footings on Broken or Jointed Rock

Table 4.4.8.1.2.A - for footings supported on jointed rock.

a. estimated RMR, Rock Mass Rating,	Low value is 50% - Rock Mass Quality is fair
b. Rock Category per 4.4.8.1.2B	E, GNEISS and Schist
c. Unconfined compressive strength, Co	8,000 psi estimated (3500 - 45000 psi for GNEISS)
d. Nms, per Table 4.4.8.1.2A	Table states to use Nms=.081
e. Q ult	Nms x Co

Nominal Bearing Resistance

$Q_{nom} := 0.081 \cdot 8000 \cdot psi$ $\quad Q_{nom} = 93.312\ ksf$

Factored Bearing Resistance

$\phi := 0.45$

$Q_{factored} := Q_{nom} \cdot \phi$

$\boxed{Q_{factored} = 41.99\ ksf}$

Recommend factored bearing resistance 40 ksf

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $tonf := g \cdot ton$ $tsf := \frac{tonf}{ft^2}$ $psi := \frac{lbf}{in^2}$

$kip := 1000 \cdot lbf$ $kN := 10^3 \cdot newton$ $kPa := 10^3 \cdot Pa$ $MPa := 10^6 \cdot Pa$ $ksi := \frac{kip}{in^2}$

$MN := 10^6 \cdot newton$ $ksf := \frac{kip}{ft^2}$

Analysis : This is computation of the structural capacity in axial compression (no flexure) of 8 pipe pile sections, based on that foundation with the longest exposed pile lengths and deepest depth to fixity. This is at Pier 3. This calculation of factored axial compressive structural capacity may be used at all bents if one factored resistance value is desired for all bents.

Pipe Pile Properties

$i := 1, 2 .. 6$

Use the following pipe pile diameters 24" - 30", 1/2 and 5/8 walls

$$dia := \begin{pmatrix} 24 \\ 26 \\ 28 \\ 30 \end{pmatrix} \cdot in \qquad wall := \begin{pmatrix} \frac{1}{2} \\ \frac{5}{8} \end{pmatrix} \cdot in \qquad c := \frac{1}{8} \cdot in$$

Corrosion loss
per Bridge Design Guide

$$dia_{corr} := dia - 2 \cdot c \qquad dia_{corr} = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} in$$

$$wall_{corr} := wall - c \qquad wall_{corr} = \begin{pmatrix} 0.375 \\ 0.5 \end{pmatrix} in$$

Steel Area of 1/2-inch wall pipe piles assuming corrosion loss of 1/8" per BDG

$$A_1 := \left[\pi \cdot \left(\frac{dia_{corr}}{2} \right)^2 - \pi \cdot \left(\frac{dia_{corr} - 2 \cdot wall_{corr_0}}{2} \right)^2 \right] \qquad A_1 = \begin{pmatrix} 27.538 \\ 29.894 \\ 32.25 \\ 34.607 \end{pmatrix} in^2$$

Steel Area of 5/8-inch wall pipe piles

$$A_2 := \left[\pi\cdot\left(\frac{dia_{corr}}{2}\right)^2 - \pi\cdot\left(\frac{dia_{corr} - 2\cdot wall_{corr_1}}{2}\right)^2\right] \qquad A_2 = \begin{pmatrix} 36.521 \\ 39.663 \\ 42.804 \\ 45.946 \end{pmatrix} in^2$$

Composite Pile Properties for 1/2-inch wall pipe piles

unit weight of concrete $\quad wc := 0.15$ $\quad$ in kips per cubic foot

compressive strength of concrete in ksi $\quad fc := 4.450$ $\quad$ ksi

modulus of elasticity - concrete $\quad E_c := 33000\cdot wc^{1.5}\cdot\sqrt{fc}\cdot 1000\cdot psi \qquad E_c = 4.044\times 10^3\, ksi$

steel modulus $\quad E_{steel} := 29000\cdot ksi$

$$n := \frac{E_{steel}}{E_c} \qquad n = 7.171$$

diameter of concrete core $\quad d_c := dia - 2\cdot\frac{1}{2}\cdot in \qquad d_c = \begin{pmatrix} 23 \\ 25 \\ 27 \\ 29 \end{pmatrix} in$

dia of steel pipe $\quad d_s := dia_{corr} \qquad d_s = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} in$

Moment of interia of concrete core $\quad I_c := \frac{\pi\cdot d_c^{\,4}}{64} \qquad I_c = \begin{pmatrix} 0.662 \\ 0.925 \\ 1.258 \\ 1.674 \end{pmatrix} ft^4$

Moment of interia of steel pipe $\quad I_{s\_0.5} := \frac{\pi\cdot\overrightarrow{\left(d_s^{\,4} - d_c^{\,4}\right)}}{64} \qquad I_{s\_0.5} = \begin{pmatrix} 0.091 \\ 0.116 \\ 0.146 \\ 0.18 \end{pmatrix} ft^4$

Composite Moment of Inertia

$$I_t := \overrightarrow{\left(\frac{I_c}{n} + I_{s\_0.5}\right)} \qquad I_t = \begin{pmatrix} 0.183 \\ 0.245 \\ 0.321 \\ 0.414 \end{pmatrix} ft^4$$

Transformed area

$$A_{concrete\_0.5} := \pi \cdot \frac{d_c^{\,2}}{4}$$

$$A_t := A_1 + \frac{A_{concrete\_0.5}}{n} \qquad A_t = \begin{pmatrix} 0.594 \\ 0.683 \\ 0.778 \\ 0.88 \end{pmatrix} ft^2$$

Depth for fixity for 1/2-inch wall piles

Use one calculation of structural capacity of all 1/2-wall pipe piles

Use Preliminary Estimate of Fixity with no lateral loads for 0.5-inch wall pipe piles based on Pier 3, which has the longest estimated depth to fixity for each pipe diameter - reference Table 2 at start of Appendix D - Calculations.

$$Fixity_{0.5} := \begin{pmatrix} 19 \\ 21 \\ 22 \\ 23 \end{pmatrix} \cdot ft$$

Composite Pile Properties of 5/8 wall pipe piles

diameter of concrete core

$$d_c := dia - 2 \cdot \frac{5}{8} \cdot in \qquad d_c = \begin{pmatrix} 22.75 \\ 24.75 \\ 26.75 \\ 28.75 \end{pmatrix} in$$

dia of steel pipe

$$d_s := dia_{corr} \qquad d_s = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} in$$

Moment of interia of concrete core

$$I_c := \frac{\pi \cdot d_c^{\,4}}{64} \qquad I_c = \begin{pmatrix} 0.634 \\ 0.888 \\ 1.212 \\ 1.617 \end{pmatrix} ft^4$$

Moment of interia of steel pipe

$$I_{s\_0.625} := \frac{\pi \cdot \left(\overrightarrow{\left(d_s^{\,4} - d_c^{\,4}\right)}\right)}{64} \qquad I_{s\_0.625} = \begin{pmatrix} 0.119 \\ 0.152 \\ 0.192 \\ 0.237 \end{pmatrix} ft^4$$

Composite Moment of Inertia

$$I_t := \frac{I_c}{n} + I_{s\_0.625} \qquad I_t = \begin{pmatrix} 0.207 \\ 0.276 \\ 0.361 \\ 0.463 \end{pmatrix} ft^4$$

Transformed area

$$A_{concrete\_0.625} := \pi \cdot \frac{d_c^{\,2}}{4}$$

$$A_t := A_2 + \frac{A_{concrete\_0.625}}{n} \qquad A_t = \begin{pmatrix} 0.647 \\ 0.741 \\ 0.842 \\ 0.948 \end{pmatrix} ft^2$$

Depth for fixity for 5/8-inch wall pipe piles

Use 1 calculation of structural capacity of all 5/8-wall pipe piles

Use Preliminary Estimates of Fixity with no lateral loads for 0.625-inch wall pipe piles - reference Table 2 at beginning of Appendix D - Calculations. Use estimates for Pier 3 which has the longest depths to fixity.

$$Fixity_{0.625} := ft \cdot \begin{pmatrix} 20 \\ 21 \\ 23 \\ 24 \end{pmatrix}$$

Nominal Axial Structural Resistance of Pipe piles

Axial pile resistance may be controlled by structural resistance if driven to sound bedrock
Check concurrent axial loading and moments with LRFD Equation 6.9.2.2-1.
Use 6.9.4.1-1 to compute the nominal compressive structural resistance for HP sections and LRFD 6.9.5.1-1 or 6.9.5.1-2 for Pipe Pile Sections

λ in equation 6.9.5.1-2 has to be computed for the pipes since the pipe piles have an unbraced length

Yield strength of steel shell $F_y := 45 \cdot ksi$

Compressive strength of concrete core $f_c := 4000 \cdot psi$

Yield strength of longitudinal reinforcment $F_{yr} := 60 \cdot ksi$

Compute λ per 6.9.4.1-3 for noncomposite members or 6.9.5.1-1 for composite members

Effective length factor per Article 4.6.2.5

Use case (c) in Table C4.6.2.5-1

$K := 1.0$ Because piles are to be fixed at the end with a socketed H or W-section

Exposed length of pile

Bottom of pile cap to streambed elevations range from 14 ft at Pier 1 to 27 feet at Pier 3 - for a single pile design, use the most conservative, 27 ft

$L_{ex} := 27 \cdot ft$

NOTE regarding scour and exposed length of pile variable: LRFD Article 10.5.5.3.2 states that the nomimal resistance remaining after the scour resulting from the check flood shall provide adequate foundation resistance to support the UNFACTORED STRENGTH LIMIT STATE LOADS with a resistance factor of 1.0.

The nominal axial structural compressive resistance of the pipe piles due to an increased exposed length of pile due to scour is provided at the end of these calcuations

The investigation of the nominal axial structural compressive resistance of the pipe piles, and also in combined flexure and axial loading, considering an increased exposed length of pile due to scour, and its resistance to the unfactored stength limit state load group is the responsiblity of the structural engineer.

L unbraced length of column

$$L_{0.5} := L_{ex} + Fixity_{0.5} \qquad L_{0.5} = \begin{pmatrix} 46 \\ 48 \\ 49 \\ 50 \end{pmatrix} ft$$

$$L_{0.625} := L_{ex} + Fixity_{0.625} \qquad L_{0.625} = \begin{pmatrix} 47 \\ 48 \\ 50 \\ 51 \end{pmatrix} ft$$

Longitudinal Reinforcement

Assume longitudinal reinforcement of 12 - #8 (1-in) rebar equally spaced for all pile sections

$$A_r := 12 \cdot \frac{\pi \cdot (1 \cdot in)^2}{4} \qquad A_r = 9.425\ in^2$$

Composite Column Constant per Table 6.9.5.1.1

for filled tube sections C1=1.0, C2=0.85, C3=0.40

$$C1 := 1.0 \qquad C2 := 0.85 \qquad C3 := 0.40$$

Variable Fe

$$F_{e\_0.5} := F_y + C1 \cdot F_{yr} \cdot \frac{A_r}{A_1} + C2 \cdot f_c \cdot \frac{A_{concrete\_0.5}}{A_1} \qquad F_{e\_0.5} = \begin{pmatrix} 116.832 \\ 119.745 \\ 122.896 \\ 126.235 \end{pmatrix} ksi$$

for 1/2-in walls

$$F_{e\_0.625} := F_y + C1 \cdot F_{yr} \cdot \frac{A_r}{A_2} + C2 \cdot f_c \cdot \frac{A_{concrete\_0.625}}{A_2} \qquad F_{e\_0.625} = \begin{pmatrix} 98.327 \\ 100.499 \\ 102.852 \\ 105.347 \end{pmatrix} ksi$$

for 5/8-in walls

Radius of gyration of both sets of steel sections

$$r_{s\_0.5} := \overrightarrow{\sqrt{\frac{I_{s\_0.5}}{A_1}}} \qquad r_{s\_0.5} = \begin{pmatrix} 0.689 \\ 0.748 \\ 0.807 \\ 0.866 \end{pmatrix} ft$$

for 1/2-in walls

$$r_{s\_0.625} := \sqrt{\frac{I_{s\_0.625}}{A_2}} \qquad r_{s\_0.625} = \begin{pmatrix} 0.685 \\ 0.744 \\ 0.803 \\ 0.862 \end{pmatrix} ft$$

for 5/8-in walls

Ee Term

$$E_{e\_0.5} := E_{steel} \cdot \left(1 + \frac{C3}{n} \cdot \overrightarrow{\frac{A_{concrete\_0.5}}{A_1}} \right) \qquad E_{e\_0.5} = \begin{pmatrix} 53406 \\ 55563 \\ 57719 \\ 59876 \end{pmatrix} ksi$$

for 1/2-in walls

$$E_{e\_0.625} := E_{steel} \cdot \left(1 + \frac{C3}{n} \cdot \frac{A_{concrete\_0.625}}{A_2} \right) \qquad E_{e\_0.625} = \begin{pmatrix} 47005 \\ 48622 \\ 50239 \\ 51857 \end{pmatrix} ksi$$

for 5/8-in walls

Lamda term for composite members LRFD Eq. 6.9.5.1-3

$$\lambda_{0.5} := \overrightarrow{\left[\left(\frac{K \cdot L_{0.5}}{r_{s\_0.5} \cdot \pi} \right)^2 \cdot \frac{F_{e\_0.5}}{E_{e\_0.5}} \right]} \qquad \lambda_{0.5} = \begin{pmatrix} 0.989 \\ 0.9 \\ 0.796 \\ 0.713 \end{pmatrix}$$

for 1/2-in walls

$$\lambda_{0.625} := \overrightarrow{\left[\left(\frac{K \cdot L_{0.625}}{r_{s\_0.625} \cdot \pi} \right)^2 \cdot \frac{F_{e\_0.625}}{E_{e\_0.625}} \right]} \qquad \lambda_{0.625} = \begin{pmatrix} 0.997 \\ 0.872 \\ 0.804 \\ 0.721 \end{pmatrix}$$

for 5/8-in walls

Lamda term for noncomposite members LRFD Eq. 6.9.4.1-3

$$\lambda_{0.5\_tip} := \overrightarrow{\left[\left(\frac{K \cdot L_{0.5}}{r_{s\_0.5} \cdot \pi}\right)^2 \cdot \frac{F_y}{E_{steel}}\right]} \qquad \lambda_{0.5\_tip} = \begin{pmatrix} 0.701 \\ 0.648 \\ 0.58 \\ 0.525 \end{pmatrix}$$

for 1/2-in walls

$$\lambda_{0.625\_tip} := \overrightarrow{\left[\left(\frac{K \cdot L_{0.625}}{r_{s\_0.625} \cdot \pi}\right)^2 \cdot \frac{F_y}{E_{steel}}\right]} \qquad \lambda_{0.625\_tip} = \begin{pmatrix} 0.74 \\ 0.654 \\ 0.61 \\ 0.55 \end{pmatrix}$$

for 5/8-in walls

Nominal Axial Structural Resistance of 1/2-inch wall

Since λ<2.25 use Eq. 6.9.5.1-1

$$P_{n\_0.5} := \overrightarrow{\left(0.66^{\lambda_{0.5}} \cdot F_{e\_0.5} \cdot A_1\right)} \qquad P_{n\_0.5} = \begin{pmatrix} 2134 \\ 2463 \\ 2847 \\ 3249 \end{pmatrix} \text{kip}$$

At the bottom of open-ended pile piles, or closed ended pipes where the conical tip or closed tip experiences breaching, the nominal compressive resistance is a function of only the steel pipe

$$P_{n\_0.5tip} := \overrightarrow{\left(0.66^{\lambda_{0.5\_tip}} \cdot F_y \cdot A_1\right)} \qquad P_{n\_0.5tip} = \begin{pmatrix} 926 \\ 1028 \\ 1140 \\ 1252 \end{pmatrix} \text{kip}$$

Use this for design

Nominal Axial Structural Resistance of 5/8-inch wall

$$P_{n\_0.625} := \overrightarrow{\left(0.66^{\lambda_{0.625}} \cdot F_{e\_0.625} \cdot A_2\right)} \qquad P_{n\_0.625} = \begin{pmatrix} 2373 \\ 2775 \\ 3152 \\ 3588 \end{pmatrix} \text{kip}$$

For lower portion of open ended pipe piles, or pipe piles where the tip has breached, the nominal compressive resistance is a fct of just the steel pipe wall

$$P_{n\_0.625tip} := \overrightarrow{\left(0.66^{\lambda_{0.625\_tip}} \cdot F_y \cdot A_2\right)}$$

$$P_{n\_0.625tip} = \begin{pmatrix} 1209 \\ 1360 \\ 1495 \\ 1645 \end{pmatrix} kip$$

Use this value for design

Factored Axial Structural Resistance of single pipe pile

Strength State resistance factor for pipe piles in compression, no damage anticipated - LRFD 6.5.4.2

$$\phi_c := 0.70$$

Factored Structural Resistance (Pr)

$$P_{r\_0.5} := \phi_c \cdot P_{n\_0.5}$$

$$P_{r\_0.5} = \begin{pmatrix} 1493 \\ 1724 \\ 1993 \\ 2274 \end{pmatrix} kip$$

for 1/2-in walls

$$P_{r\_0.625} := \phi_c \cdot P_{n\_0.625}$$

$$P_{r\_0.625} = \begin{pmatrix} 1661 \\ 1943 \\ 2206 \\ 2511 \end{pmatrix} kip$$

for 5/8-in walls

Factored Structural Resistance (Pr) for the lower portion of open-ended pile piles or breached close-ended pipe piles is a function of only the steel shell

$$P_{r\_0.5tip} := \phi_c \cdot P_{n\_0.5tip}$$

$$P_{r\_0.5tip} = \begin{pmatrix} 648 \\ 719 \\ 798 \\ 877 \end{pmatrix} kip$$

for 1/2-in walls

$$P_{r\_0.625tip} := \phi_c \cdot P_{n\_0.625tip}$$

$$P_{r\_0.625tip} = \begin{pmatrix} 846 \\ 952 \\ 1047 \\ 1151 \end{pmatrix} kip$$

for 5/8-in walls

Use these for Factored Structural Resistance

The nominal axial structural compressive resistance of the pipe piles due to an increased exposed length of pile due to the scour event

LRFD Article 10.5.5.3.2 states that the nomimal resistance remaining after the scour resulting from the check flood shall provide adequate foundation resistance to support the UNFACTORED STRENGTH LIMIT STATE LOADS with a resistance factor of 1.0.

The nominal axial structural compressive resistance of the pipe piles due to an increased exposed length of pile due to scour is calculated below. Pier 2 is subject to the greatest exposed pile length due to scour - therefore this analysis is done from Pier 2.

The investigation of the nominal axial structural compressive resistance of the pipe piles, and also in combined flexure and axial loading, considering an increased exposed length of pile due to scour, and its resistance to the unfactored stength limit state load group is the responsiblity of the structural engineer

Compute λ per 6.9.4.1-3 for noncomposite members or 6.9.5.1-1 for composite members

Effective length factor per Article 4.6.2.5

Use case (c) in Table C4.6.2.5-1

$K := 1.0$ Because piles are to be fixed at the end with a socketed H or W-section

Exposed length of pile

Bottom of pile cap to the top of the glacial sand unit (all glaciomarine soils are scoured)

$L_{ex} := 10 \cdot ft - -74.5 \cdot ft$ $L_{ex} = 84.5\,ft$

Depth to fixity is from the top of the glacial sand unit to the top of the H- or W-section in rock socket

$Fixity_{0.5} := -74.5 \cdot ft - -93.10 \cdot ft$

$Fixity_{0.5} = 18.6\,ft$ and $Fixity_{0.625} := Fixity_{0.5}$

L unbraced length of column

$L_{0.5} := L_{ex} + Fixity_{0.5}$ $L_{0.5} = 103.1\,ft$

$L_{0.625} := L_{ex} + Fixity_{0.625}$ $L_{0.625} = 103.1\,ft$

Longitudinal Reinforcement

Assume longitudinal reinforcement of 12 - #8 (1-in) rebar equally spaced for all pile sections

$$A_r := 12 \cdot \frac{\pi \cdot (1 \cdot in)^2}{4} \qquad A_r = 9.425\ in^2$$

Composite Column Constant per Table 6.9.5.1.1

for filled tube sections C1=1.0, C2=0.85, C3=0.40

$$C1 := 1.0 \qquad C2 := 0.85 \qquad C3 := 0.40$$

Variable Fe

$$F_{e\_0.5} := F_y + C1 \cdot F_{yr} \cdot \frac{A_r}{A_1} + C2 \cdot f_c \cdot \frac{A_{concrete\_0.5}}{A_1} \qquad F_{e\_0.5} = \begin{pmatrix} 116.832 \\ 119.745 \\ 122.896 \\ 126.235 \end{pmatrix} ksi$$

for 1/2-in walls

$$F_{e\_0.625} := F_y + C1 \cdot F_{yr} \cdot \frac{A_r}{A_2} + C2 \cdot f_c \cdot \frac{A_{concrete\_0.625}}{A_2} \qquad F_{e\_0.625} = \begin{pmatrix} 98.327 \\ 100.499 \\ 102.852 \\ 105.347 \end{pmatrix} ksi$$

for 5/8-in walls

Radius of gyration of both sets of steel sections

$$r_{s\_0.5} := \overrightarrow{\sqrt{\frac{I_{s\_0.5}}{A_1}}} \qquad r_{s\_0.5} = \begin{pmatrix} 0.689 \\ 0.748 \\ 0.807 \\ 0.866 \end{pmatrix} ft$$

for 1/2-in walls

$$r_{s\_0.625} := \sqrt{\frac{I_{s\_0.625}}{A_2}} \qquad r_{s\_0.625} = \begin{pmatrix} 0.685 \\ 0.744 \\ 0.803 \\ 0.862 \end{pmatrix} ft$$

for 5/8-in walls

Ee Term

$$E_{e\_0.5} := E_{steel} \cdot \left(1 + \frac{C3}{n} \cdot \overrightarrow{\frac{A_{concrete\_0.5}}{A_1}}\right) \qquad E_{e\_0.5} = \begin{pmatrix} 53406 \\ 55563 \\ 57719 \\ 59876 \end{pmatrix} ksi$$

for 1/2-in walls

$$E_{e\_0.625} := E_{steel} \cdot \left(1 + \frac{C3}{n} \cdot \frac{A_{concrete\_0.625}}{A_2}\right) \qquad E_{e\_0.625} = \begin{pmatrix} 47005 \\ 48622 \\ 50239 \\ 51857 \end{pmatrix} ksi$$

for 5/8-in walls

Lamda term for composite members LRFD Eq. 6.9.5.1-3

$$\lambda_{0.5} := \overrightarrow{\left[\left(\frac{K \cdot L_{0.5}}{r_{s\_0.5} \cdot \pi}\right)^2 \cdot \frac{F_{e\_0.5}}{E_{e\_0.5}}\right]} \qquad \lambda_{0.5} = \begin{pmatrix} 4.966 \\ 4.152 \\ 3.524 \\ 3.031 \end{pmatrix}$$

for 1/2-in walls

$$\lambda_{0.625} := \overrightarrow{\left[\left(\frac{K \cdot L_{0.625}}{r_{s\_0.625} \cdot \pi}\right)^2 \cdot \frac{F_{e\_0.625}}{E_{e\_0.625}}\right]} \qquad \lambda_{0.625} = \begin{pmatrix} 4.799 \\ 4.021 \\ 3.419 \\ 2.945 \end{pmatrix}$$

for 5/8-in walls

Lamda term for noncomposite members LRFD Eq. 6.9.4.1-3

$$\lambda_{0.5\_tip} := \overrightarrow{\left[\left(\frac{K \cdot L_{0.5}}{r_{s\_0.5} \cdot \pi}\right)^2 \cdot \frac{F_y}{E_{steel}}\right]} \qquad \lambda_{0.5\_tip} = \begin{pmatrix} 3.523 \\ 2.989 \\ 2.569 \\ 2.231 \end{pmatrix}$$

for 1/2-in walls

$$\lambda_{0.625\_tip} := \overrightarrow{\left[\left(\frac{K \cdot L_{0.625}}{r_{s\_0.625} \cdot \pi}\right)^2 \cdot \frac{F_y}{E_{steel}}\right]} \qquad \lambda_{0.625\_tip} = \begin{pmatrix} 3.56 \\ 3.019 \\ 2.592 \\ 2.25 \end{pmatrix}$$

for 5/8-in walls

Nominal Axial Structural Resistance of 1/2-inch wall

Since λ<2.25 use Eq. 6.9.5.1-1

$$P_{n\_0.5} := \overrightarrow{\left(0.66^{\lambda_{0.5}} \cdot F_{e\_0.5} \cdot A_1\right)}$$

$$P_{n\_0.5} = \begin{pmatrix} 409 \\ 638 \\ 916 \\ 1240 \end{pmatrix} \text{kip}$$

At the bottom of open-ended pile piles, or closed ended pipes where the conical tip or closed tip experiences breaching, the nominal compressive resistance is a function of only the steel pipe

$$P_{n\_0.5tip} := \overrightarrow{\left(0.66^{\lambda_{0.5\_tip}} \cdot F_y \cdot A_1\right)}$$

$$P_{n\_0.5tip} = \begin{pmatrix} 287 \\ 388 \\ 499 \\ 616 \end{pmatrix} \text{kip}$$

Use this for design

Nominal Axial Structural Resistance of 5/8-inch wall

$$P_{n\_0.625} := \overrightarrow{\left(0.66^{\lambda_{0.625}} \cdot F_{e\_0.625} \cdot A_2\right)}$$

$$P_{n\_0.625} = \begin{pmatrix} 489 \\ 750 \\ 1063 \\ 1424 \end{pmatrix} \text{kip}$$

For lower portion of open ended pipe piles, or pipe piles where the tip has breached, the nominal compressive resistance is a fct of just the steel pipe wall

$$P_{n\_0.625tip} := \overrightarrow{\left(0.66^{\lambda_{0.625\_tip}} \cdot F_y \cdot A_2\right)}$$

$$P_{n\_0.625tip} = \begin{pmatrix} 374 \\ 509 \\ 656 \\ 812 \end{pmatrix} \text{kip}$$

Use this value for design

Nominal Axial Structural Compressive Resistance of single pipe pile AFTER SCOUR for comparision to the the unfactored Strength Limit State load case.

Nominal Axial Structural Resistance of 1/2-inch wall

Assuming Composite action of Concrete Filled Pipe Pile

$$P_{n\_0.5} = \begin{pmatrix} 409 \\ 638 \\ 916 \\ 1240 \end{pmatrix} \text{kip}$$

Assuming contribution of only the steel pipe

$$P_{n\_0.5tip} = \begin{pmatrix} 287 \\ 388 \\ 499 \\ 616 \end{pmatrix} \text{kip}$$

Nominal Axial Structural Resistance of 5/8-inch wall

Assuming Composite action of Concrete Filled Pipe Pile

$$P_{n\_0.625} = \begin{pmatrix} 489 \\ 750 \\ 1063 \\ 1424 \end{pmatrix} \text{kip}$$

Assuming contribution of only the steel pipe

$$P_{n\_0.625tip} = \begin{pmatrix} 374 \\ 509 \\ 656 \\ 812 \end{pmatrix} \text{kip}$$

Structural Capacity -ASD Method

1/2-inch Wall

$F_y := 45 \cdot ksi$ $\quad \sigma_{ult} := F_y$ $\quad Q_{ult\_0.5} := \overrightarrow{(\sigma_{ult} \cdot A_1)}$ $\quad Q_{ult\_0.5} = \begin{pmatrix} 1239 \\ 1345 \\ 1451 \\ 1557 \end{pmatrix} kip$

$Q_{all\_0.5} := \frac{\sigma_{ult}}{3} \cdot A_1$ $\quad \boxed{Q_{all\_0.5} = \begin{pmatrix} 413 \\ 448 \\ 484 \\ 519 \end{pmatrix} kip}$

5/8-inch wall

$F_y := 45 \cdot ksi$ $\quad \sigma_{ult} := F_y$ $\quad Q_{ult\_0.625} := \overrightarrow{(\sigma_{ult} \cdot A_2)}$ $\quad Q_{ult\_0.625} = \begin{pmatrix} 1643 \\ 1785 \\ 1926 \\ 2068 \end{pmatrix} kip$

$Q_{all\_0.625} := \frac{\sigma_{ult}}{3} \cdot A_2$ $\quad \boxed{Q_{all\_0.625} = \begin{pmatrix} 548 \\ 595 \\ 642 \\ 689 \end{pmatrix} kip}$

Calibrated Factored Resistances to ASD

$Q_{ult\_0.5} \cdot 0.65 = \begin{pmatrix} 805 \\ 874 \\ 943 \\ 1012 \end{pmatrix} kip$ $\quad Q_{ult\_0.625} \cdot 0.65 = \begin{pmatrix} 1068 \\ 1160 \\ 1252 \\ 1344 \end{pmatrix} kip$

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $tonf := g \cdot ton$ $tsf := \frac{tonf}{ft^2}$

$psi := \frac{lbf}{in^2}$ $kip := 1000 \cdot lbf$ $kN := 10^3 \cdot newton$ $kPa := 10^3 \cdot Pa$

$MN := 10^6 \cdot newton$ $ksf := \frac{kip}{ft^2}$ $MPa := 10^6 \cdot Pa$ $ksi := \frac{kip}{in^2}$

Bedrock Properties

Pipe pile capacity based on steel shell end bearing on bedrock - driven through soft glaciomarine silt clay deposit

RQD from bedrock cores ranges from:
27% to 35% in the most weathered and fractured zones of the bedrock formation
50% to 83% in the moderately to slightly weathered and fractured zones

Rock Type: Mostly banded SCHIST and GNEISS
Biotite SCHIST - more weathered & fractured zones
GNEISS - better RQD
PEGMATITE GRANITE veins

Perform 4 designs:

1. RQD of **27%** for design (for Biotite SCHIST) at Pier 1, ϕ = 20-27 (AASHTO LRFD Table C.10.4.6.4-1); AASHTO TABLE 4.4.8.2.B uniaxial compressive strength = Cu= 1400 to 21,000 psi - use 7500 psi for design

2. RQD of **35%** for design (for Biotite SCHIST) at Pier 2, ϕ = 20-27 (AASHTO LRFD Table C.10.4.6.4-1); AASHTO TABLE 4.4.8.2.B uniaxial compressive strength = Cu= 1400 to 21000 psi - use 7500 psi for design

3. RQD of **57%** SCHIST and **50%** GNEISS at Pier 4 and Pier 5; ϕ = 27-34 (AASHTO LRFD Table C.10.4.6.4-1); AASHTO TABLE 4.4.8.2.B uniaxial compressive strength = 3,500 - 45,000 psi- use 8,000 psi for design

4. Highest RQD of 83% at Pier 3, GNEISS, hard, relative unweathered, slightly fractured, use ϕ = 27-34 (AASHTO LRFD Table C.10.4.6.4-1);
AASHTO TABLE 4.4.8.2.B uniaxial compressive strength = 3,500 - 45,000 psi -- use 15,000 psi for design

Pipe Pile Properties

Use the following pipe pile diameters 24" - 30", 1/2 and 5/8 walls

$i := 1, 2 .. 6$

$$dia := \begin{pmatrix} 24 \\ 26 \\ 28 \\ 30 \end{pmatrix} \cdot in \qquad wall := \begin{pmatrix} \frac{1}{2} \\ \frac{5}{8} \end{pmatrix} \cdot in \qquad c := \frac{1}{8} \cdot in$$

$$dia_{corr} := dia - 2 \cdot c \qquad dia_{corr} = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} in$$

$$wall_{corr} := wall - c \qquad wall_{corr} = \begin{pmatrix} 0.375 \\ 0.5 \end{pmatrix} in$$

Design steel loss due to corrosion per the BDG

Steel Area of 1/2-inch wall pipe piles assuming corrosion loss of 1/8" per BDG

$$A_1 := \left[\pi \cdot \left(\frac{dia_{corr}}{2} \right)^2 - \pi \cdot \left(\frac{dia_{corr} - 2 \cdot wall_{corr_0}}{2} \right)^2 \right] \qquad A_1 = \begin{pmatrix} 27.538 \\ 29.894 \\ 32.25 \\ 34.607 \end{pmatrix} in^2$$

$$A_{1\_noloss} := \left[\pi \cdot \left(\frac{dia}{2} \right)^2 - \pi \cdot \left(\frac{dia - 2 \cdot wall_0}{2} \right)^2 \right] \qquad A_{1\_noloss} = \begin{pmatrix} 36.914 \\ 40.055 \\ 43.197 \\ 46.338 \end{pmatrix} in^2$$

Steel Area of 5/8-inch wall pipe piles

$$A_2 := \left[\pi \cdot \left(\frac{dia_{corr}}{2} \right)^2 - \pi \cdot \left(\frac{dia_{corr} - 2 \cdot wall_{corr_1}}{2} \right)^2 \right] \qquad A_2 = \begin{pmatrix} 36.521 \\ 39.663 \\ 42.804 \\ 45.946 \end{pmatrix} in^2$$

$$A_{2\_noloss} := \left[\pi\cdot\left(\frac{dia}{2}\right)^2 - \pi\cdot\left(\frac{dia - 2\cdot wall_1}{2}\right)^2\right] \qquad A_{2\_noloss} = \begin{pmatrix} 45.897 \\ 49.824 \\ 53.751 \\ 57.678 \end{pmatrix} in^2$$

Depth to Fixity of 1/2-inch wall pipe piles at Pier 1

12.5 ft of very soft marine silt (Su=450) over 9.5 feet of stiff, weathered PF (Su=1500) over 6.3 ft of very soft glaciomarine clayey silt, Su ranges from 360 and 520 psf - use average Su of 440 psf for entire strata

Transformed pile propeties of 1/2 inch wall pipe pile

$Su := 440\cdot psf$

unit weight of concrete in kcf $\quad wc := 0.15$

compressive strength of concrete in ksi $\quad fc := 4.450$

modulus of elasticity - concrete $\quad E_c := 33000\cdot wc^{1.5}\cdot\sqrt{fc}\cdot 1000\cdot psi \qquad E_c = 4.044\times 10^3\, ksi$

steel modulus $\quad E_{steel} := 29000\cdot ksi$

Modular ratio, n $\quad n := \frac{E_{steel}}{E_c} \qquad n = 7.171$

MDOT Stuctural Engr routinely use n = 7.6 $\qquad n := 7.6$

diameter of concrete core $\quad d_c := dia - 2\cdot\frac{1}{2}\cdot in \qquad d_c = \begin{pmatrix} 23 \\ 25 \\ 27 \\ 29 \end{pmatrix} in$

dia of steel pipe

$d_s := dia_{corr} \qquad d_s = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} in$

Moment of interia of concrete core $\quad I_c := \frac{\pi\cdot d_c^4}{64} \qquad I_c = \begin{pmatrix} 0.662 \\ 0.925 \\ 1.258 \\ 1.674 \end{pmatrix} ft^4$

Moment of interia of steel pipe

$$I_{s\_0.5} := \frac{\pi \cdot \left(d_s^{\,4} - d_c^{\,4}\right)}{64} \qquad I_{s\_0.5} = \begin{pmatrix} 0.091 \\ 0.116 \\ 0.146 \\ 0.18 \end{pmatrix} ft^4$$

Composite Moment of Inertia

$$I_t := \frac{I_c}{n} + I_{s\_0.5} \qquad I_t = \begin{pmatrix} 0.178 \\ 0.238 \\ 0.311 \\ 0.4 \end{pmatrix} ft^4$$

Transformed area

$$A_{concrete\_0.5} := \pi \cdot \frac{d_c^{\,2}}{4}$$

$$A_t := A_1 + \frac{A_{concrete\_0.5}}{n} \qquad A_t = \begin{pmatrix} 0.571 \\ 0.656 \\ 0.747 \\ 0.844 \end{pmatrix} ft^2$$

LRFD Eq. 10.7.3.13.4-1 for fixity in feet : 1.4*(EpIp/Es)^0.25.
E must be in ksi and I in ft^4.
Es=0.465*Su (Su must be in ksf, result is in ksi)
Su=325 psf

Use same equation in NCHRP#343 pg 61:
Leq=Lu +1.4R
where Leq = equivalent free standing length of pile
Lu=unsupported length of pile extending above ground
for clays, R=(Ep*Ip/Es)^0.25

Soil Modulus of clay

$$E_{soil} := 67 \cdot Su \qquad E_{soil} = 29.48\ ksf$$

R parameter

$$R := \left(\frac{E_{steel} \cdot I_t}{E_{soil}}\right)^{0.25} \qquad R = \begin{pmatrix} 12.599 \\ 13.547 \\ 14.491 \\ 15.432 \end{pmatrix} ft$$

Depth for fixity $D_{0.5} := 1.4 \cdot R$

$$D_{0.5} = \begin{pmatrix} 17.639 \\ 18.965 \\ 20.287 \\ 21.605 \end{pmatrix} ft$$

Check first value in Array with LRFD Eq. 10.7.3.13.4-1

$E_{steel} = 29000\,ksi$ $\quad E_{soil} := 0.465 \cdot (0.440) \cdot ksi$ $\quad E_{soil} = 0.205\,ksi$

$$I_t = \begin{pmatrix} 0.178 \\ 0.238 \\ 0.311 \\ 0.4 \end{pmatrix} ft^4$$

$$Check := 1.4 \cdot \left(\frac{29000 \cdot 0.178}{0.205} \right)^{0.25}$$

$Check = 17.636$ ***OK***

Depth to Fixity of 5/8 wall pipe piles

Undrained shear strength - 440 psf average $\quad Su := 440 \cdot psf$

Transformed pile propeties of 5/8 inch wall pipe pile

$n = 7.6$

diameter of concrete core $\quad d_c := dia - 2 \cdot \frac{5}{8} \cdot in$

$$d_c = \begin{pmatrix} 22.75 \\ 24.75 \\ 26.75 \\ 28.75 \end{pmatrix} in$$

dia of steel pipe $\quad d_s := dia_{corr}$

$$d_s = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} in$$

Moment of interia of concrete core $\quad I_c := \frac{\pi \cdot d_c^{\,4}}{64}$

$$I_c = \begin{pmatrix} 0.634 \\ 0.888 \\ 1.212 \\ 1.617 \end{pmatrix} ft^4$$

Moment of interia of steel pipe

$$I_{s\_0.625} := \frac{\pi \cdot \left(d_s^{\,4} - d_c^{\,4}\right)}{64} \qquad I_{s\_0.625} = \begin{pmatrix} 0.119 \\ 0.152 \\ 0.192 \\ 0.237 \end{pmatrix} ft^4$$

Composite Moment of Inertia

$$I_t := \frac{I_c}{n} + I_{s\_0.625} \qquad I_t = \begin{pmatrix} 0.202 \\ 0.269 \\ 0.351 \\ 0.45 \end{pmatrix} ft^4$$

Transformed area

$$A_{concrete\_0.625} := \pi \cdot \frac{d_c^{\,2}}{4}$$

$$A_t := A_2 + \frac{A_{concrete\_0.625}}{n} \qquad A_t = \begin{pmatrix} 0.625 \\ 0.715 \\ 0.811 \\ 0.912 \end{pmatrix} ft^2$$

LRFD Eq. 10.7.3.13.4-1 for fixity in feet : 1.4*(EpIp/Es)^0.25.
E must be in ksi and I in ft^4.
Es=0.465*Su (Su must be in ksf, result is in ksi)
Su=3000 psf or 3 ksf

Use same equation in NCHRP#343 pg 61

Soil Modulus of clay

$$E_{soil} := 67 \cdot Su \qquad E_{soil} = 29.48\ ksf$$

R parameter

$$R := \left(\frac{E_{steel} \cdot I_t}{E_{soil}}\right)^{0.25} \qquad R = \begin{pmatrix} 13.014 \\ 13.976 \\ 14.934 \\ 15.888 \end{pmatrix} ft$$

Depth for fixity

$$D_{0.625} := 1.4 \cdot R \qquad D_{0.625} = \begin{pmatrix} 18.22 \\ 19.567 \\ 20.908 \\ 22.243 \end{pmatrix} ft$$

Check first value in Array with LRFD Eq. 10.7.3.13.4-1 which calls for specific units

$$E_{steel} = 29000\ ksi \qquad E_{soil} := 0.465 \cdot 0.440 \cdot ksi \qquad E_{soil} = 0.205\ ksi \qquad I_t = \begin{pmatrix} 0.202 \\ 0.269 \\ 0.351 \\ 0.45 \end{pmatrix} ft^4$$

$$Check := 1.4 \cdot \left(\frac{29000 \cdot 0.202}{0.205} \right)^{0.25} \qquad Check = 18.202$$ ***OK***

Nominal Axial Structural Resistance of Pipe piles

Axial pile resistance may be controlled by structural resistance if driven to sound bedrock - unlikely at Boothbay site.
Check concurrent axial loading and moments with LRFD Equation 6.9.2.2-1.
Use 6.9.4.1-1 to compute the nominal compressive structural resistance for HP sections and LRFD 6.9.5.1-1 or 6.9.5.1-2 for Pipe Pile Sections

λ in equation 6.9.5.1-2 has to be computed for the pipes since the pipe piles have an unbraced length

Yield strength of steel shell $F_y := 45 \cdot ksi$

Compressive strength of concrete core $f_c := 4000 \cdot psi$

Yield strength of longitudinal reinforcment $F_{yr} := 60 \cdot ksi$

Compute λ per 6.9.4.1-3 for noncomposite members or 6.9.5.1-1 for composite members

Effective length factor per Article 4.6.2.5

Use case (c) in Table C4.6.2.5-1 - end of pipe pile is socketed in bedrock and fixed with an H or W section, so fixed against rotation and translation, K=1.0

$K := 1.0$

L unbraced length of column SEE Table 3 at the start of this Appendix for unbraced pile lengths

$$L_{0.5} := 14 \cdot ft + D_{0.5} \qquad L_{0.5} = \begin{pmatrix} 31.639 \\ 32.965 \\ 34.287 \\ 35.605 \end{pmatrix} ft$$

$$L_{0.625} := 14 \cdot ft + D_{0.625} \qquad L_{0.625} = \begin{pmatrix} 32.22 \\ 33.567 \\ 34.908 \\ 36.243 \end{pmatrix} ft$$

Longitudinal Reinforcement

Assume longitudinal reinforcement of 12 - #8 (1-in) rebar equally spaced for all pile sections

$$A_r := 12 \cdot \frac{\pi \cdot (1 \cdot in)^2}{4} \qquad A_r = 9.425\ in^2$$

Composite Column Constant per Table 6.9.5.1.1

for filled tube sections C1=1.0, C2=0.85, C3=0.40

$C1 := 1.0 \qquad C2 := 0.85 \qquad C3 := 0.40$

Variable Fe

$$F_{e\_0.5} := F_y + C1 \cdot F_{yr} \cdot \frac{A_r}{A_1} + C2 \cdot f_c \cdot \frac{A_{concrete\_0.5}}{A_1} \qquad F_{e\_0.5} = \begin{pmatrix} 116.832 \\ 119.745 \\ 122.896 \\ 126.235 \end{pmatrix} ksi$$

$$F_{e\_0.625} := F_y + C1 \cdot F_{yr} \cdot \frac{A_r}{A_2} + C2 \cdot f_c \cdot \frac{A_{concrete\_0.625}}{A_2} \qquad F_{e\_0.625} = \begin{pmatrix} 98.327 \\ 100.499 \\ 102.852 \\ 105.347 \end{pmatrix} ksi$$

Radius of gyration of both sets of steel sections

$$r_{s\_0.5} := \sqrt{\frac{I_{s\_0.5}}{A_1}} \qquad r_{s\_0.5} = \begin{pmatrix} 0.689 \\ 0.748 \\ 0.807 \\ 0.866 \end{pmatrix} ft$$

$$r_{s\_0.625} := \sqrt{\frac{I_{s\_0.625}}{A_2}} \qquad r_{s\_0.625} = \begin{pmatrix} 0.685 \\ 0.744 \\ 0.803 \\ 0.862 \end{pmatrix} ft$$

Ee Term

$$E_{e\_0.5} := E_{steel} \cdot \left(1 + \frac{C3}{n} \cdot \frac{A_{concrete\_0.5}}{A_1}\right) \qquad E_{e\_0.5} = \begin{pmatrix} 52028 \\ 54063 \\ 56097 \\ 58132 \end{pmatrix} ksi$$

$$E_{e\_0.625} := E_{steel} \cdot \left(1 + \frac{C3}{n} \cdot \frac{A_{concrete\_0.625}}{A_2}\right) \qquad E_{e\_0.625} = \begin{pmatrix} 45988 \\ 47514 \\ 49040 \\ 50566 \end{pmatrix} ksi$$

Lamda term

$$\lambda_{0.5} := \overrightarrow{\left[\left(\frac{K \cdot L_{0.5}}{r_{s\_0.5} \cdot \pi}\right)^2 \cdot \frac{F_{e\_0.5}}{E_{e\_0.5}}\right]} \qquad \lambda_{0.5} = \begin{pmatrix} 0.48 \\ 0.436 \\ 0.401 \\ 0.372 \end{pmatrix}$$

$$\lambda_{0.625} := \overrightarrow{\left[\left(\frac{K \cdot L_{0.625}}{r_{s\_0.625} \cdot \pi}\right)^2 \cdot \frac{F_{e\_0.625}}{E_{e\_0.625}}\right]} \qquad \lambda_{0.625} = \begin{pmatrix} 0.479 \\ 0.436 \\ 0.402 \\ 0.373 \end{pmatrix}$$

Lamda term for noncomposite members LRFD Eq. 6.9.4.1-3

$$\lambda_{0.5\_tip} := \overrightarrow{\left[\left(\frac{K \cdot L_{0.5}}{r_{s\_0.5} \cdot \pi}\right)^2 \cdot \frac{F_y}{E_{steel}}\right]} \qquad \lambda_{0.5\_tip} = \begin{pmatrix} 0.332 \\ 0.306 \\ 0.284 \\ 0.266 \end{pmatrix}$$

$$\lambda_{0.625\_tip} := \overrightarrow{\left[\left(\frac{K \cdot L_{0.625}}{r_{s\_0.625} \cdot \pi}\right)^2 \cdot \frac{F_y}{E_{steel}}\right]} \qquad \lambda_{0.625\_tip} = \begin{pmatrix} 0.348 \\ 0.32 \\ 0.297 \\ 0.278 \end{pmatrix}$$

Nominal Axial Structural Resistance of 1/2-inch wall

Since λ<2.25 use Eq. 6.9.5.1-1

$$P_{n\_0.5} := \overrightarrow{\left(0.66^{\lambda_{0.5}} \cdot F_{e\_0.5} \cdot A_1\right)} \qquad P_{n\_0.5} = \begin{pmatrix} 2635 \\ 2986 \\ 3355 \\ 3742 \end{pmatrix} kip$$

At the bottom of open-ended pile piles, or closed ended pipes where the conical tip or closed tip experiences breaching, the nominal compressive resistance is a function of only the steel pipe

$$P_{n\_0.5tip} := \overrightarrow{\left(0.66^{\lambda_{0.5\_tip}} \cdot F_y \cdot A_1\right)}$$

$$P_{n\_0.5tip} = \begin{pmatrix} 1080 \\ 1185 \\ 1290 \\ 1394 \end{pmatrix} kip$$

Nominal Axial Structural Resistance of 5/8-inch wall

$$P_{n\_0.625} := \overrightarrow{\left(0.66^{\lambda_{0.625}} \cdot F_{e\_0.625} \cdot A_2\right)}$$

$$P_{n\_0.625} = \begin{pmatrix} 2943 \\ 3325 \\ 3726 \\ 4145 \end{pmatrix} kip$$

For lower portion of open ended pipe piles, or pipe piles where the tip has breached, the nominal compressive resistance is a fct of just the steel pipe wall

$$P_{n\_0.625tip} := \overrightarrow{\left(0.66^{\lambda_{0.625\_tip}} \cdot F_y \cdot A_2\right)}$$

$$P_{n\_0.625tip} = \begin{pmatrix} 1422 \\ 1563 \\ 1702 \\ 1842 \end{pmatrix} kip$$

Factored Axial Structural Resistance of single pipe pile

Strength State resistance factor for pipe piles in compression, no damage anticipated - LRFD 6.5.4.2

$$\phi_c := 0.70$$

Factored Structural Resistance (Pr)

$$P_{r\_0.5} := \phi_c \cdot P_{n\_0.5}$$

$$P_{r\_0.625} := \phi_c \cdot P_{n\_0.625}$$

$$P_{r\_0.5} = \begin{pmatrix} 1845 \\ 2090 \\ 2349 \\ 2620 \end{pmatrix} kip \qquad P_{r\_0.625} = \begin{pmatrix} 2060 \\ 2328 \\ 2608 \\ 2901 \end{pmatrix} kip$$

Factored Structural Resistance (Pr) for the lower portion of open-ended pile piles or breached close-ended pipe piles is a function of only the steel shell

$$P_{n\_0.5tip} = \begin{pmatrix} 1080 \\ 1185 \\ 1290 \\ 1394 \end{pmatrix} kip \qquad P_{n\_0.625tip} = \begin{pmatrix} 1422 \\ 1563 \\ 1702 \\ 1842 \end{pmatrix} kip$$

$$P_{r\_0.5tip} := \phi_c \cdot P_{n\_0.5tip}$$

$$P_{r\_0.625tip} := \phi_c \cdot P_{n\_0.625tip}$$

$$P_{r\_0.5tip} = \begin{pmatrix} 756 \\ 829 \\ 903 \\ 976 \end{pmatrix} kip \qquad P_{r\_0.625tip} = \begin{pmatrix} 996 \\ 1094 \\ 1192 \\ 1289 \end{pmatrix} kip$$

Use these for Factored Structural Resistance at Pier 1

Geotechnical Pile Resistance - Kulhawy and Goodman - RQD method

Ref: "Pile Design & Construction Practice", Tomlinson, page 139

For calculating maximum end bearing allowable load -this method ignores side resistance - use Driven to calculate that.

Correct for wedge failure under strip footing:

multiply cNc by 1.25 - square piles, 1.2 for circular piles
multiply $\gamma BN\gamma$ by 0.8 - square pile and 0.7 for circular piles

For RQD 0 -70 %

qc=0.33 x Quc
c=0.1 x Quc
ϕ=30 degrees

For RQD 70 - 100 %

qc=0.33 to 0.88 x Quc
c=0.10 x Quc
ϕ=30 to 60 degrees

Calculation

assume RQD = 27%
ϕ = 27 ***PELLS & TURNER, TOMLINSON, PAGE 140***
c= .10 x Quc *Assume pile penetrates 0 inches into bedrock*
qc = 0.33 x Quc *Assume schist has an unconfined Qu of 7500 psi*

unconfined compression strength of bedrock

$q_{uc} := 7500 \cdot psi$ $c := 0.1 \cdot q_{uc}$ $c = 750\,psi$

$D := .0 \cdot in$

$B_{min} := 24 \cdot in$

$\gamma := 145 \cdot \frac{lbf}{ft^3}$ $q_c := 0.33 \cdot q_{uc}$ $q_c = 2475\,psi$

$N_c := 11.95$ $N_q := 7.09$ $N_\gamma := 9.94$ ***(BASED ON PELLS & TURNER, TOMLINSON, PAGE 140)***

$$q_{ub} := 1.2 \cdot c \cdot N_c + \frac{\gamma \cdot B_{min} \cdot N_\gamma}{2} \cdot 0.7 + \gamma \cdot D \cdot N_q$$

This is ultimate base resistance, settlement up to 20% of base dia. needed to mobilize. So to ensure settlements at working load are within allowable limits, limit working load to q ub divided by 3.5

$q_{ub} = 10.762\,ksi$

$R_{p\_nom\_0.5\_kulh} := q_{ub} \cdot A_{1\_noloss}$ $R_{p\_nom\_0.5\_kulh} = \begin{pmatrix} 397 \\ 431 \\ 465 \\ 499 \end{pmatrix} kip$

$R_{p\_nom\_0.625\_kulh} := q_{ub} \cdot A_{2\_noloss}$ $R_{p\_nom\_0.625\_kulh} = \begin{pmatrix} 494 \\ 536 \\ 578 \\ 621 \end{pmatrix} kip$

Factored Axial Geotechnical Tip Resistance of single pipe pile

Factor to account for method controlling pile installation- dynamic tests on 2% piles and CAPWAP $\lambda_v := 1.0$

Resistance factor, end bearing on rock CGS method $\phi_{qp} := 0.45 \cdot \lambda_v$

Factored Geotechnical Tip Resistance (Rr) assuming no soil plug

$R_{r\_tip\_0.5\_kulh} := R_{p\_nom\_0.5\_kulh} \cdot \phi_{qp}$

$$R_{r\_tip\_0.5\_kulh} = \begin{pmatrix} 179 \\ 194 \\ 209 \\ 224 \end{pmatrix} kip$$

$R_{r\_tip\_0.625\_kulh} := R_{p\_nom\_0.625\_kulh} \cdot \phi_{qp}$

$$R_{r\_tip\_0.625\_kulh} = \begin{pmatrix} 222 \\ 241 \\ 260 \\ 279 \end{pmatrix} kip$$

Nominal Geotechnical Side Frictional Resistance

The piles will be primarily end bearing. At piers 1, 3, 4 and 5 no skin friction in the overlying glaciomarine and marine clayey silt.

The nominal skin friction is computed using FHWA program Driven 1.0

Driven software uses Nordlund/Thurman Method for side frictional resistance in cohesionless soils. Use a ϕ of 0.45 per LRFD Table 10.5.5.2.3-1

$\phi_{side} := 0.45$

Nominal side resistance (from Driven)

$$R_{s\_0.5} := \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kip \qquad R_{s\_0.625} := \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kip$$

Factored Geotechnical Side Frictional Resistance

$R_{r\_s0.5} := R_{s\_0.5} \cdot \phi_{side}$ $\qquad$ $R_{r\_s0.625} := R_{s\_0.625} \cdot \phi_{side}$

$$R_{r\_s0.5} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} kip \qquad R_{r\_s0.625} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} kip$$

Total Factored Geotechnical Bearing Resistance of Piles - End bearing of shell + side frictional resistance + 0% of the plugged area on bedrock

$R_{r\_0.5total} := R_{r\_tip\_0.5\_kulh} + R_{r\_s0.5}$

$$R_{r\_0.5total} = \begin{pmatrix} 179 \\ 194 \\ 209 \\ 224 \end{pmatrix} kip$$

$R_{r\_0.625total} := R_{r\_tip\_0.625\_kulh} + R_{r\_s0.625}$

$$R_{r\_0.625total} = \begin{pmatrix} 222 \\ 241 \\ 260 \\ 279 \end{pmatrix} kip$$

Service Limit State" Factored " Axial Geotechnical Tip Resistance of single pipe pile

Resistance factor for service limit state $\phi_c := 1.0$

Service Limit State Geotechnical Resistance

$$R_{service\_0.5} := R_{p\_nom\_0.5\_kulh} \cdot \phi_c \qquad R_{service\_0.5} = \begin{pmatrix} 397 \\ 431 \\ 465 \\ 499 \end{pmatrix} kip$$

$$R_{service\_0.625} := R_{p\_nom\_0.625\_kulh} \cdot \phi_c \qquad R_{service\_0.625} = \begin{pmatrix} 494 \\ 536 \\ 578 \\ 621 \end{pmatrix} kip$$

Compute Ultimate Resistance that must be achieved during the driveability wave equation analysis

The ultimate resistance that must be achieved during wave equation analysis will be the APPLIED MAXIMUM AXIAL PILE LOAD {must be < FACTORED RESISTANCE (the factored geotechnical resistance governs for all pile sections)} divided by the appropriate resistance factor for wave equation analysis & dynamic test.

Resistance factor per Table 10.5.5.2.3-1 for dynamic testing is ϕ=0.65, per Table 3 which requires no less than 3 to 4 piles are to be tested considering low to medium site variablity.

$\phi_{dyn} := 0.65$

For the 1/2-in wall piles Rr_0.5total governs the factored resistance

$$Q_{p\_0.5} := \frac{R_{r\_0.5total}}{\phi_{dyn}} \qquad Q_{p\_0.5} = \begin{pmatrix} 275 \\ 298 \\ 322 \\ 345 \end{pmatrix} kip$$

For the 5/8-in wall piles, the factored geotech resistance governs (Rr_0.625total)

$$Q_{p\_0.625} := \frac{R_{r\_0.625total}}{\phi_{dyn}} \qquad Q_{p\_0.625} = \begin{pmatrix} 342 \\ 371 \\ 400 \\ 430 \end{pmatrix} kip$$

The resistance required by the driveablity wave equation analysis is:

$$Q_{p\_0.5} = \begin{pmatrix} 275 \\ 298 \\ 322 \\ 345 \end{pmatrix} \text{kip}$$

$$Q_{p\_0.625} = \begin{pmatrix} 342 \\ 371 \\ 400 \\ 430 \end{pmatrix} \text{kip}$$

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $tonf := g \cdot ton$ $tsf := \frac{tonf}{ft^2}$

$psi := \frac{lbf}{in^2}$ $kip := 1000 \cdot lbf$ $kN := 10^3 \cdot newton$ $kPa := 10^3 \cdot Pa$

$MN := 10^6 \cdot newton$ $ksf := \frac{kip}{ft^2}$ $MPa := 10^6 \cdot Pa$ $ksi := \frac{kip}{in^2}$

Bedrock Properties

Pipe pile capacity based on steel shell end bearing on bedrock - assume driven through soft glaciomarine silt clay deposit

RQD from bedrock cores ranges from:
27% to 35% in the most weathered and fractured zones of the bedrock formation
50% to 83% in the moderately to slightly weathered and fractured zones

Rock Type: Mostly banded SCHIST and GNEISS
Biotite SCHIST - more weathered & fractured zones
GNEISS - better RQD
PEGMATITE GRANITE veins

Perform 4 designs:

1. RQD of 27% for design (for Biotite SCHIST) at Pier 1, ϕ = 20-27 (AASHTO LRFD Table C.10.4.6.4-1); AASHTO TABLE 4.4.8.2.B uniaxial compressive strength = Cu= 1400 to 21,000 psi - use 7500 psi for design

2. RQD of **35%** for design (for Biotite SCHIST) at Pier 2, ϕ = 20-27 (AASHTO LRFD Table C.10.4.6.4-1); AASHTO TABLE 4.4.8.2.B uniaxial compressive strength = Cu= 1400 to 21000 psi - use 7500 psi for design

3. RQD of 57% SCHIST and 50% GNEISS at Piers 4 and 5; ϕ = 27-34 (AASHTO LRFD Table C.10.4.6.4-1); AASHTO TABLE 4.4.8.2.B uniaxial compressive strength = 3,500 - 45,000 psi- 33; use 8,000 psi for design

4. Highest RQD of 83% at Pier 3, GNEISS, hard, relative unweathered, slightly fractured, use ϕ = 27-34 (AASHTO LRFD Table C.10.4.6.4-1);
AASHTO TABLE 4.4.8.2.B uniaxial compressive strength = 3,500 - 45,000 psi; use 15,000 psi for design

Pipe Pile Properties

Use the following pipe pile diameters 24" - 30", 1/2 and 5/8 walls

$i := 1, 2 .. 6$

$$dia := \begin{pmatrix} 24 \\ 26 \\ 28 \\ 30 \end{pmatrix} \cdot in \qquad wall := \begin{pmatrix} \frac{1}{2} \\ \frac{5}{8} \end{pmatrix} \cdot in \qquad c := \frac{1}{8} \cdot in$$

c = corrosion loss per BDG

$$dia_{corr} := dia - 2 \cdot c \qquad dia_{corr} = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} in$$

$$wall_{corr} := wall - c \qquad wall_{corr} = \begin{pmatrix} 0.375 \\ 0.5 \end{pmatrix} in$$

Steel Area of 1/2-inch wall pipe piles assuming corrosion loss of 1/8" per BDG

$$A_1 := \left[\pi \cdot \left(\frac{dia_{corr}}{2} \right)^2 - \pi \cdot \left(\frac{dia_{corr} - 2 \cdot wall_{corr_0}}{2} \right)^2 \right] \qquad A_1 = \begin{pmatrix} 27.538 \\ 29.894 \\ 32.25 \\ 34.607 \end{pmatrix} in^2$$

$$A_{1\_noloss} := \left[\pi \cdot \left(\frac{dia}{2} \right)^2 - \pi \cdot \left(\frac{dia - 2 \cdot wall_0}{2} \right)^2 \right] \qquad A_{1\_noloss} = \begin{pmatrix} 36.914 \\ 40.055 \\ 43.197 \\ 46.338 \end{pmatrix} in^2$$

Steel Area of 5/8-inch wall pipe piles

$$A_2 := \left[\pi \cdot \left(\frac{dia_{corr}}{2} \right)^2 - \pi \cdot \left(\frac{dia_{corr} - 2 \cdot wall_{corr_1}}{2} \right)^2 \right] \qquad A_2 = \begin{pmatrix} 36.521 \\ 39.663 \\ 42.804 \\ 45.946 \end{pmatrix} in^2$$

$$A_{2\_noloss} := \left[\pi\cdot\left(\frac{dia}{2}\right)^2 - \pi\cdot\left(\frac{dia - 2\cdot wall_1}{2}\right)^2\right] \qquad A_{2\_noloss} = \begin{pmatrix} 45.897 \\ 49.824 \\ 53.751 \\ 57.678 \end{pmatrix} in^2$$

Depth to Fixity of 1/2-inch wall pipe piles at Pier 2

13 ft of very soft marine silt (Su=310, 400 psf) over 5 feet of stiff, weathered PF (Su correlation of 750) over 36.5 ft of very soft glaciomarine clayey silt, 9 Su vane readings averaged to 491psf - use average Su of 490 psf for entire strata

Transformed pile propeties of 1/2 inch wall pipe pile

$$Su := 490\cdot psf$$

unit weight of concrete in kcf $\quad wc := 0.15$

compressive strength of concrete in ksi $\quad fc := 4.450$

modulus of elasticity - concrete $\quad E_c := 33000\cdot wc^{1.5}\cdot\sqrt{fc}\cdot 1000\cdot psi \qquad E_c = 4.044\times 10^3\, ksi$

steel modulus $\quad E_{steel} := 29000\cdot ksi$

Modular ratio, n $\quad n := \frac{E_{steel}}{E_c} \qquad n = 7.171$

MDOT structural engineers routinely use n = 7.6 $\qquad n := 7.6$

diameter of concrete core $\quad d_c := dia - 2\cdot\frac{1}{2}\cdot in \qquad d_c = \begin{pmatrix} 23 \\ 25 \\ 27 \\ 29 \end{pmatrix} in$

dia of steel pipe

$$d_s := dia_{corr} \qquad d_s = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} in$$

Moment of interia of concrete core $\quad I_c := \frac{\pi\cdot d_c^4}{64} \qquad I_c = \begin{pmatrix} 0.662 \\ 0.925 \\ 1.258 \\ 1.674 \end{pmatrix} ft^4$

Moment of interia of steel pipe

$$I_{s\_0.5} := \frac{\pi \cdot \left(d_s^{\,4} - d_c^{\,4}\right)}{64} \qquad I_{s\_0.5} = \begin{pmatrix} 0.091 \\ 0.116 \\ 0.146 \\ 0.18 \end{pmatrix} ft^4$$

Composite Moment of Inertia

$$I_t := \frac{I_c}{n} + I_{s\_0.5} \qquad I_t = \begin{pmatrix} 0.178 \\ 0.238 \\ 0.311 \\ 0.4 \end{pmatrix} ft^4$$

Transformed area

$$A_{concrete\_0.5} := \pi \cdot \frac{d_c^{\,2}}{4}$$

$$A_t := A_1 + \frac{A_{concrete\_0.5}}{n} \qquad A_t = \begin{pmatrix} 0.571 \\ 0.656 \\ 0.747 \\ 0.844 \end{pmatrix} ft^2$$

LRFD Eq. 10.7.3.13.4-1 for fixity in feet : 1.4*(EpIp/Es)^0.25.
E must be in ksi and I in ft^4.
Es=0.465*Su (Su must be in ksf, result is in ksi)
Su=325 psf

Use same equation in NCHRP#343 pg 61:
Leq=Lu +1.4R
where Leq = equivalent free standing length of pile
Lu=unsupported length of pile extending above ground
for clays, R=(Ep*Ip/Es)^0.25

Soil Modulus of clay

$$E_{soil} := 67 \cdot Su \qquad E_{soil} = 32.83\ ksf$$

R parameter

$$R := \left(\frac{E_{steel} \cdot I_t}{E_{soil}}\right)^{0.25} \qquad R = \begin{pmatrix} 12.265 \\ 13.187 \\ 14.106 \\ 15.022 \end{pmatrix} ft$$

Depth for fixity $D_{0.5} := 1.4 \cdot R$

$$D_{0.5} = \begin{pmatrix} 17.171 \\ 18.462 \\ 19.748 \\ 21.031 \end{pmatrix} ft$$

Check first value in Array with LRFD Eq. 10.7.3.13.4-1

$E_{steel} = 29000\,ksi$ $E_{soil} := 0.465 \cdot (0.490) \cdot ksi$ $E_{soil} = 0.228\,ksi$

$$I_t = \begin{pmatrix} 0.178 \\ 0.238 \\ 0.311 \\ 0.4 \end{pmatrix} ft^4$$

$$Check := 1.4 \cdot \left(\frac{29000 \cdot 0.178}{0.228} \right)^{0.25}$$ $Check = 17.173$ ***OK***

Depth to Fixity of 5/8 wall pipe piles

Undrained shear strength - 490 psf average $Su := 490 \cdot psf$

Transformed pile propeties of 5/8 inch wall pipe pile

$n = 7.6$

diameter of concrete core $d_c := dia - 2 \cdot \frac{5}{8} \cdot in$

$$d_c = \begin{pmatrix} 22.75 \\ 24.75 \\ 26.75 \\ 28.75 \end{pmatrix} in$$

dia of steel pipe $d_s := dia_{corr}$

$$d_s = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} in$$

Moment of interia of concrete core $I_c := \frac{\pi \cdot d_c^4}{64}$

$$I_c = \begin{pmatrix} 0.634 \\ 0.888 \\ 1.212 \\ 1.617 \end{pmatrix} ft^4$$

Moment of interia of steel pipe

$$I_{s\_0.625} := \frac{\pi \cdot \left(d_s^{\,4} - d_c^{\,4}\right)}{64} \qquad I_{s\_0.625} = \begin{pmatrix} 0.119 \\ 0.152 \\ 0.192 \\ 0.237 \end{pmatrix} ft^4$$

Composite Moment of Inertia

$$I_t := \frac{I_c}{n} + I_{s\_0.625} \qquad I_t = \begin{pmatrix} 0.202 \\ 0.269 \\ 0.351 \\ 0.45 \end{pmatrix} ft^4$$

Transformed area

$$A_{concrete\_0.625} := \pi \cdot \frac{d_c^{\,2}}{4}$$

$$A_t := A_2 + \frac{A_{concrete\_0.625}}{n} \qquad A_t = \begin{pmatrix} 0.625 \\ 0.715 \\ 0.811 \\ 0.912 \end{pmatrix} ft^2$$

LRFD Eq. 10.7.3.13.4-1 for fixity in feet : 1.4*(EpIp/Es)^0.25.
E must be in ksi and I in ft^4.
Es=0.465*Su (Su must be in ksf, result is in ksi)
Su=3000 psf or 3 ksf

Use same equation in NCHRP#343 pg 61

Soil Modulus of clay

$$E_{soil} := 67 \cdot Su \qquad E_{soil} = 32.83\ ksf$$

R parameter

$$R := \left(\frac{E_{steel} \cdot I_t}{E_{soil}}\right)^{0.25} \qquad R = \begin{pmatrix} 12.669 \\ 13.605 \\ 14.538 \\ 15.466 \end{pmatrix} ft$$

Depth for fixity

$$D_{0.625} := 1.4 \cdot R \qquad D_{0.625} = \begin{pmatrix} 17.736 \\ 19.048 \\ 20.353 \\ 21.653 \end{pmatrix} ft$$

Check first value in Array with LRFD Eq. 10.7.3.13.4-1 which calls for specific units

$$E_{steel} = 29000\ ksi \qquad E_{soil} := 0.465 \cdot 0.490 \cdot ksi \qquad E_{soil} = 0.228\ ksi \qquad I_t = \begin{pmatrix} 0.202 \\ 0.269 \\ 0.351 \\ 0.45 \end{pmatrix} ft^4$$

$$\text{Check} := 1.4\cdot\left(\frac{29000\cdot 0.202}{0.228}\right)^{0.25} \qquad \text{Check} = 17.725$$

OK

Nominal Axial Structural Resistance of Pipe piles

Axial pile resistance may be controlled by structural resistance if driven to sound bedrock - unlikely at Boothbay site.
Check concurrent axial loading and moments with LRFD Equation 6.9.2.2-1.
Use 6.9.4.1-1 to compute the nominal compressive structural resistance for HP sections and LRFD 6.9.5.1-1 or 6.9.5.1-2 for Pipe Pile Sections

λ in equation 6.9.5.1-2 has to be computed for the pipes since the pipe piles have an unbraced length

Yield strength of steel shell $\qquad F_y := 45\cdot ksi$

Compressive strength of concrete core $\qquad f_c := 4000\cdot psi$

Yield strength of longitudinal reinforcment $\qquad F_{yr} := 60\cdot ksi$

Compute λ per 6.9.4.1-3 for noncomposite members or 6.9.5.1-1 for composite members

Effective length factor per Article 4.6.2.5

Use case (c) in Table C4.6.2.5-1 ; since pipe pile tips are fixed in rock sockets, K=1.0

$$K := 1.0$$

L unbraced length of column; see Table 3 on Sheet 1 of this Appendix D for unbraced pile lengths

$$L_{0.5} := 22\cdot ft + D_{0.5} \qquad L_{0.5} = \begin{pmatrix} 39.171 \\ 40.462 \\ 41.748 \\ 43.031 \end{pmatrix} ft$$

$$L_{0.625} := 22\cdot ft + D_{0.625} \qquad L_{0.625} = \begin{pmatrix} 39.736 \\ 41.048 \\ 42.353 \\ 43.653 \end{pmatrix} ft$$

Longitudinal Reinforcement

Assume longitudinal reinforcement of 12 - #8 (1-in) rebar equally spaced for all pile sections

$$A_r := 12 \cdot \frac{\pi \cdot (1 \cdot in)^2}{4} \qquad A_r = 9.425\ in^2$$

Composite Column Constant per Table 6.9.5.1.1

for filled tube sections C1=1.0, C2=0.85, C3=0.40

$C1 := 1.0 \qquad C2 := 0.85 \qquad C3 := 0.40$

Variable Fe

$$F_{e\_0.5} := F_y + C1 \cdot F_{yr} \cdot \frac{A_r}{A_1} + C2 \cdot f_c \cdot \frac{A_{concrete\_0.5}}{A_1} \qquad F_{e\_0.5} = \begin{pmatrix} 116.832 \\ 119.745 \\ 122.896 \\ 126.235 \end{pmatrix} ksi$$

$$F_{e\_0.625} := F_y + C1 \cdot F_{yr} \cdot \frac{A_r}{A_2} + C2 \cdot f_c \cdot \frac{A_{concrete\_0.625}}{A_2} \qquad F_{e\_0.625} = \begin{pmatrix} 98.327 \\ 100.499 \\ 102.852 \\ 105.347 \end{pmatrix} ksi$$

Radius of gyration of both sets of steel sections

$$r_{s\_0.5} := \sqrt{\frac{I_{s\_0.5}}{A_1}} \qquad r_{s\_0.5} = \begin{pmatrix} 0.689 \\ 0.748 \\ 0.807 \\ 0.866 \end{pmatrix} ft$$

$$r_{s\_0.625} := \sqrt{\frac{I_{s\_0.625}}{A_2}} \qquad r_{s\_0.625} = \begin{pmatrix} 0.685 \\ 0.744 \\ 0.803 \\ 0.862 \end{pmatrix} ft$$

Ee Term

$$E_{e\_0.5} := E_{steel} \cdot \left(1 + \frac{C3}{n} \cdot \frac{A_{concrete\_0.5}}{A_1}\right) \qquad E_{e\_0.5} = \begin{pmatrix} 52028 \\ 54063 \\ 56097 \\ 58132 \end{pmatrix} ksi$$

$$E_{e\_0.625} := E_{steel} \cdot \left(1 + \frac{C3}{n} \cdot \frac{A_{concrete\_0.625}}{A_2}\right) \qquad E_{e\_0.625} = \begin{pmatrix} 45988 \\ 47514 \\ 49040 \\ 50566 \end{pmatrix} ksi$$

Lamda term

$$\lambda_{0.5} := \overrightarrow{\left[\left(\frac{K \cdot L_{0.5}}{r_{s\_0.5} \cdot \pi}\right)^2 \cdot \frac{F_{e\_0.5}}{E_{e\_0.5}}\right]} \qquad \lambda_{0.5} = \begin{pmatrix} 0.736 \\ 0.657 \\ 0.595 \\ 0.544 \end{pmatrix}$$

$$\lambda_{0.625} := \overrightarrow{\left[\left(\frac{K \cdot L_{0.625}}{r_{s\_0.625} \cdot \pi}\right)^2 \cdot \frac{F_{e\_0.625}}{E_{e\_0.625}}\right]} \qquad \lambda_{0.625} = \begin{pmatrix} 0.729 \\ 0.652 \\ 0.591 \\ 0.541 \end{pmatrix}$$

Lamda term for noncomposite members LRFD Eq. 6.9.4.1-3

$$\lambda_{0.5\_tip} := \overrightarrow{\left[\left(\frac{K \cdot L_{0.5}}{r_{s\_0.5} \cdot \pi}\right)^2 \cdot \frac{F_y}{E_{steel}}\right]} \qquad \lambda_{0.5\_tip} = \begin{pmatrix} 0.508 \\ 0.46 \\ 0.421 \\ 0.389 \end{pmatrix}$$

$$\lambda_{0.625\_tip} := \overrightarrow{\left[\left(\frac{K \cdot L_{0.625}}{r_{s\_0.625} \cdot \pi}\right)^2 \cdot \frac{F_y}{E_{steel}}\right]} \qquad \lambda_{0.625\_tip} = \begin{pmatrix} 0.529 \\ 0.478 \\ 0.437 \\ 0.403 \end{pmatrix}$$

Nominal Axial Structural Resistance of 1/2-inch wall

Since λ<2.25 use Eq. 6.9.5.1-1

$$P_{n\_0.5} := \overrightarrow{\left(0.66^{\lambda_{0.5}} \cdot F_{e\_0.5} \cdot A_1\right)} \qquad P_{n\_0.5} = \begin{pmatrix} 2370 \\ 2724 \\ 3096 \\ 3485 \end{pmatrix} kip$$

At the bottom of open-ended pile piles, or closed ended pipes where the conical tip or closed tip experiences breaching, the nominal compressive resistance is a function of only the steel pipe

$$P_{n\_0.5tip} := \overrightarrow{\left(0.66^{\lambda_{0.5\_tip}} \cdot F_y \cdot A_1\right)}$$

$$P_{n\_0.5tip} = \begin{pmatrix} 1003 \\ 1111 \\ 1218 \\ 1325 \end{pmatrix} kip$$

Nominal Axial Structural Resistance of 5/8-inch wall

$$P_{n\_0.625} := \overrightarrow{\left(0.66^{\lambda_{0.625}} \cdot F_{e\_0.625} \cdot A_2\right)}$$

$$P_{n\_0.625} = \begin{pmatrix} 2653 \\ 3040 \\ 3444 \\ 3865 \end{pmatrix} kip$$

For lower portion of open ended pipe piles, or pipe piles where the tip has breached, the nominal compressive resistance is a fct of just the steel pipe wall

$$P_{n\_0.625tip} := \overrightarrow{\left(0.66^{\lambda_{0.625\_tip}} \cdot F_y \cdot A_2\right)}$$

$$P_{n\_0.625tip} = \begin{pmatrix} 1319 \\ 1463 \\ 1606 \\ 1749 \end{pmatrix} kip$$

Nominal Axial Geotechnical Resistance of pipe piles

Nominal unit bearing resistance of pile point, qub

Nominal geotechnical tip resistance, Rp, of only steel pipe area - use full pile area for geotechnical capacity, not corroded section

Kulhawy and Goodman - RQD method

Ref: "Pile Design & Construction Practice", Tomlinson, page 139

For calculating maximum end bearing allowable load -this method ignores side resistance - use Driven to calculate that.

Correct for wedge failure under strip footing:

multiply cNc by 1.25 - square piles, 1.2 for circular piles
multiply $\gamma BN\gamma$ by 0.8 - square pile and 0.7 for circular piles

For RQD 0 -70 %

qc=0.33 x Quc
c=0.1 x Quc
ϕ=30 degrees

For RQD 70 - 100 %

qc=0.33 to 0.88 x Quc
c=0.10 x Quc
ϕ=30 to 60 degrees

Calculation

assume RQD = 35%
ϕ = 27 ***PELLS & TURNER, TOMLINSON, PAGE 140***
c= .10 x Quc *Assume pile penetrates 0 inches into bedrock*
qc = 0.33 x Quc

$q_{uc} := 7500 \cdot psi$ $c := 0.1 \cdot q_{uc}$ $c = 750\,psi$

$D := .0 \cdot in$

$B_{min} := 24 \cdot in$

$\gamma := 145 \cdot \frac{lbf}{ft^3}$ $q_c := 0.33 \cdot q_{uc}$ $q_c = 2475\,psi$

$N_c := 11.95$ $N_q := 7.09$ $N_\gamma := 9.94$

Reference: PELLS & TURNER, TOMLINSON, Page 140

$$q_{ub} := 1.2 \cdot c \cdot N_c + \frac{\gamma \cdot B_{min} \cdot N_\gamma}{2} \cdot 0.7 + \gamma \cdot D \cdot N_q$$

This is ultimate base resistance, settlement up to 20% of base dia. needed to mobilize. So to ensure settlements at working load are within allowable limits, limit working load to q ub divided by 3.5

$$q_{ub} = 10.762\ \text{ksi}$$

$$R_{p\_0.5\_kulh} := q_{ub} \cdot A_{1\_noloss}$$

$$R_{p\_0.5\_kulh} = \begin{pmatrix} 397 \\ 431 \\ 465 \\ 499 \end{pmatrix} \text{kip}$$

$$R_{p\_0.625\_kulh} := q_{ub} \cdot A_{2\_noloss}$$

$$R_{p\_0.625\_kulh} = \begin{pmatrix} 494 \\ 536 \\ 578 \\ 621 \end{pmatrix} \text{kip}$$

Nominal geotechnical tip resistance, Rp, of plug and steel shell area (ref FHWA LRFD Design Example) assume unplugged section

$$A_{plug\_0.5} := 0 \cdot \left(A_{concrete\_0.5} \cdot \frac{1}{3} + A_{1\_noloss} \right)$$

$$A_{plug\_0.625} := 0 \cdot \left(A_{concrete\_0.625} \cdot \frac{1}{3} + A_{2\_noloss} \right)$$

$$R_{p\_plugged\_0.5} := \overrightarrow{\left(q_{ub} \cdot A_{plug\_0.5} \right)}$$

$$R_{p\_plugged\_0.5} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \text{kip}$$

$$R_{p\_plugged\_0.625} := \overrightarrow{\left(q_{ub} \cdot A_{plug\_0.625} \right)}$$

$$R_{p\_plugged\_0.625} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \text{kip}$$

Factored Axial Geotechnical Tip Resistance of single pipe pile

Resistance factor, end bearing on rock CGS method $\phi_{qp} := 0.45$

Factored Geotechnical Tip Resistance (Rr) assuming no soil plug and only steel shell bearing on bedrock

$R_{r\_0.5} := R_{p\_0.5\_kulh} \cdot \phi_{qp} + \phi_{qp} \cdot R_{p\_plugged\_0.5}$

$$R_{r\_0.5} = \begin{pmatrix} 179 \\ 194 \\ 209 \\ 224 \end{pmatrix} \text{kip}$$

$R_{r\_0.625} := R_{p\_0.625\_kulh} \cdot \phi_{qp} + \phi_{qp} \cdot R_{p\_plugged\_0.625}$

$$R_{r\_0.625} = \begin{pmatrix} 222 \\ 241 \\ 260 \\ 279 \end{pmatrix} \text{kip}$$

Factored Axial Structural Resistance of single pipe pile

Strength State resistance factor for pipe piles in compression, no damage anticipated - LRFD 6.5.4.2 $\phi_c := 0.70$

Factored Structural Resistance (Pr)

$P_{r\_0.5} := \phi_c \cdot P_{n\_0.5}$

$P_{r\_0.625} := \phi_c \cdot P_{n\_0.625}$

$$P_{r\_0.5} = \begin{pmatrix} 1659 \\ 1907 \\ 2167 \\ 2440 \end{pmatrix} \text{kip} \qquad P_{r\_0.625} = \begin{pmatrix} 1857 \\ 2128 \\ 2411 \\ 2706 \end{pmatrix} \text{kip}$$

Factored Structural Resistance (Pr) for the lower portion of open-ended pile piles or breached close-ended pipe piles is a function of only the steel shell

$$P_{n\_0.5tip} = \begin{pmatrix} 1003 \\ 1111 \\ 1218 \\ 1325 \end{pmatrix} \text{kip} \qquad P_{n\_0.625tip} = \begin{pmatrix} 1319 \\ 1463 \\ 1606 \\ 1749 \end{pmatrix} \text{kip}$$

$P_{r\_0.5tip} := \phi_c \cdot P_{n\_0.5tip}$

$P_{r\_0.625tip} := \phi_c \cdot P_{n\_0.625tip}$

$P_{r\_0.5tip} = \begin{pmatrix} 702 \\ 778 \\ 853 \\ 928 \end{pmatrix} kip$

$P_{r\_0.625tip} = \begin{pmatrix} 923 \\ 1024 \\ 1124 \\ 1224 \end{pmatrix} kip$

Use these for Factored Structural Resistances at Pier 2

Nominal Geotechnical Side Frictional Resistance

The piles will be primarily end bearing. At piers 1, 3, 4 and 5 no skin friction in the overlying glaciomarine and marine clayey silt. Some side friction will develope at Pier 2. The nominal skin friction is computed using FHWA program Driven 1.0

Driven software uses Nordlund/Thurman Method for side frictional resistance in cohesionless soils. Use a ϕ of 0.45 per LRFD Table 10.5.5.2.3-1

Since piles may be driven open ended or closed ended calculated skin friction for both cases, and use that which is less for design. Open ended skin friction calculations are slightly less or equal to closed ended case .

$\phi_{side} := 0.45$

Nominal side resistance (from Driven) for a 24-in 5/8-inch wall pipe pile

$R_{s\_0.5} := \begin{pmatrix} 257 \\ 257 \\ 257 \\ 257 \end{pmatrix} \cdot kip$

$R_{s\_0.625} := \begin{pmatrix} 257 \\ 257 \\ 257 \\ 257 \end{pmatrix} \cdot kip$

Factored Geotechnical Side Frictional Resistance

$R_{r\_s0.5} := R_{s\_0.5} \cdot \phi_{side}$

$R_{r\_s0.625} := R_{s\_0.625} \cdot \phi_{side}$

$R_{r\_s0.5} = \begin{pmatrix} 116 \\ 116 \\ 116 \\ 116 \end{pmatrix} kip$

$R_{r\_s0.625} = \begin{pmatrix} 116 \\ 116 \\ 116 \\ 116 \end{pmatrix} kip$

Total Factored Geotechnical Bearing Resistance of Piles - End bearing of shell and 0% of the plugged area on rock and side frictional resistance

$R_{r\_0.5total} := R_{r\_0.5} + R_{r\_s0.5}$

$$R_{r\_0.5total} = \begin{pmatrix} 294 \\ 310 \\ 325 \\ 340 \end{pmatrix} kip$$

$R_{r\_0.625total} := R_{r\_0.625} + R_{r\_s0.625}$

$$R_{r\_0.625total} = \begin{pmatrix} 338 \\ 357 \\ 376 \\ 395 \end{pmatrix} kip$$

Service Limit State" Factored " Axial Geotechnical Tip Resistance of single pipe pile

Resistance factor for service limit state $\phi_c := 1.0$

Service Limit State Geotechnical Resistance

$R_{service\_0.5} := R_{p\_0.5\_kulh} + R_{s\_0.5}$ $$R_{service\_0.5} = \begin{pmatrix} 654 \\ 688 \\ 722 \\ 756 \end{pmatrix} kip$$

$R_{service\_0.625} := R_{s\_0.625} + R_{p\_0.625\_kulh}$ $$R_{service\_0.625} = \begin{pmatrix} 751 \\ 793 \\ 835 \\ 878 \end{pmatrix} kip$$

Compute Ultimate Resistance that must be achieved during the driveability wave equation analysis

The ultimate resistance that must be achieved during wave equation analysis will be the APPLIED MAXIMUM AXIAL PILE LOAD {must be < FACTORED RESISTANCE (the factored geotechnical resistance governs for all pile sections)} divided by the appropriate resistance factor for wave equation analysis & dynamic test.

Resistance factor per Table 10.5.5.2.3-1 for dynamic testing is ϕ=0.65, per Table 3 which requires no less than 3 to 4 piles are to be tested considering low to medium site variablity.

$\phi_{dyn} := 0.65$

For the 1/2-in wall piles Rr_0.5total governs the factored resistance

$$Q_{p\_0.5} := \frac{R_{r\_0.5total}}{\phi_{dyn}}$$

$$Q_{p\_0.5} = \begin{pmatrix} 453 \\ 476 \\ 500 \\ 523 \end{pmatrix} kip$$

For the 5/8-in wall piles, the factored geotech resistance governs (Rr_0.625total)

$$Q_{p\_0.625} := \frac{R_{r\_0.625total}}{\phi_{dyn}}$$

$$Q_{p\_0.625} = \begin{pmatrix} 520 \\ 549 \\ 578 \\ 608 \end{pmatrix} kip$$

The resistance required by the driveablity wave equation analysis is:

$$Q_{p\_0.5} = \begin{pmatrix} 453 \\ 476 \\ 500 \\ 523 \end{pmatrix} kip$$

$$Q_{p\_0.625} = \begin{pmatrix} 520 \\ 549 \\ 578 \\ 608 \end{pmatrix} kip$$

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $tonf := g \cdot ton$ $tsf := \frac{tonf}{ft^2}$

$psi := \frac{lbf}{in^2}$ $kip := 1000 \cdot lbf$ $kN := 10^3 \cdot newton$ $kPa := 10^3 \cdot Pa$

$MN := 10^6 \cdot newton$ $ksf := \frac{kip}{ft^2}$ $MPa := 10^6 \cdot Pa$ $ksi := \frac{kip}{in^2}$

Bedrock Properties

Pipe pile capacity based on steel shell end bearing on bedrock - assume driven through soft glaciomarine silt clay deposit

RQD from bedrock cores ranges from:
27% to 35% in the most weathered and fractured zones of the bedrock formation
50% to 83% in the moderately to slightly weathered and fractured zones

Rock Type: Mostly banded SCHIST and GNEISS
- Biotite SCHIST - more weathered & fractured zones
- GNEISS - better RQD
- PEGMATITE GRANITE veins

Perform 3 designs:

1. RQD of 27 to 35% for design (for Biotite SCHIST) at Pier 1, Pier 2, ϕ = 20-27 (AASHTO LRFD Table C.10.4.6.4-1); AASHTO TABLE 4.4.8.2.B uniaxial compressive strength = Cu= 1400 to 21000 psi - use 7500 psi for design

2. RQD of 50% GNEISS and 57% SCHIST banding at Pier 4 and 5; ϕ = 27-34 (AASHTO LRFD Table C.10.4.6.4-1); AASHTO TABLE 4.4.8.2.B uniaxial compressive strength = 3,500 - 45,000 psi-use 8,000 psi for design

3. Highest RQD of 83% at Pier 3, GNEISS, hard, relative unweathered, slightly fractured, use ϕ = 27-34 (AASHTO LRFD Table C.10.4.6.4-1); AASHTO TABLE 4.4.8.2.B uniaxial compressive strength = 3,500 - 45,000 psi; use 15,000 psi for design

Pipe Pile Properties

Use the following pipe pile diameters 24" - 30", 1/2 and 5/8 walls

$i := 1, 2 .. 6$

$$dia := \begin{pmatrix} 24 \\ 26 \\ 28 \\ 30 \end{pmatrix} \cdot in \qquad wall := \begin{pmatrix} \frac{1}{2} \\ \frac{5}{8} \end{pmatrix} \cdot in \qquad c := \frac{1}{8} \cdot in$$

corrosion loss per BDG

$$dia_{corr} := dia - 2 \cdot c \qquad dia_{corr} = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} in$$

$$wall_{corr} := wall - c \qquad wall_{corr} = \begin{pmatrix} 0.375 \\ 0.5 \end{pmatrix} in$$

Steel Area of 1/2-inch wall pipe piles assuming corrosion loss of 1/8" per BDG

$$A_1 := \left[\pi \cdot \left(\frac{dia_{corr}}{2} \right)^2 - \pi \cdot \left(\frac{dia_{corr} - 2 \cdot wall_{corr_0}}{2} \right)^2 \right] \qquad A_1 = \begin{pmatrix} 27.538 \\ 29.894 \\ 32.25 \\ 34.607 \end{pmatrix} in^2$$

$$A_{1\_noloss} := \left[\pi \cdot \left(\frac{dia}{2} \right)^2 - \pi \cdot \left(\frac{dia - 2 \cdot wall_0}{2} \right)^2 \right] \qquad A_{1\_noloss} = \begin{pmatrix} 36.914 \\ 40.055 \\ 43.197 \\ 46.338 \end{pmatrix} in^2$$

Steel Area of 5/8-inch wall pipe piles

$$A_2 := \left[\pi \cdot \left(\frac{dia_{corr}}{2} \right)^2 - \pi \cdot \left(\frac{dia_{corr} - 2 \cdot wall_{corr_1}}{2} \right)^2 \right] \qquad A_2 = \begin{pmatrix} 36.521 \\ 39.663 \\ 42.804 \\ 45.946 \end{pmatrix} in^2$$

$$A_{2\_noloss} := \left[\pi\cdot\left(\frac{dia}{2}\right)^2 - \pi\cdot\left(\frac{dia - 2\cdot wall_1}{2}\right)^2\right] \qquad A_{2\_noloss} = \begin{pmatrix} 45.897 \\ 49.824 \\ 53.751 \\ 57.678 \end{pmatrix} in^2$$

Depth to Fixity of 1/2-inch wall pipe piles at Pier 3

10 ft of very soft marine silt over 12.5 feet of very soft glaciomarine clayey silt, Su ranges from 250 to 400 psf

Transformed pile propeties of 1/2 inch wall pipe pile

$Su := 325\cdot psf$

unit weight of concrete in kcf $\quad wc := 0.15$

compressive strength of concrete in ksi $\quad fc := 4.450$

modulus of elasticity - concrete $\quad E_c := 33000\cdot wc^{1.5}\cdot\sqrt{fc}\cdot 1000\cdot psi \qquad E_c = 4.044\times 10^3\, ksi$

steel modulus $\quad E_{steel} := 29000\cdot ksi$

Modular ratio, n $\quad n := \frac{E_{steel}}{E_c} \qquad n = 7.171$

MDOT Structural Engrs routinely use n = 7.6 $\qquad n := 7.6$

diameter of concrete core $\quad d_c := dia - 2\cdot\frac{1}{2}\cdot in \qquad d_c = \begin{pmatrix} 23 \\ 25 \\ 27 \\ 29 \end{pmatrix} in$

dia of steel pipe

$$d_s := dia_{corr} \qquad d_s = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} in$$

Moment of interia of concrete core

$$I_c := \frac{\pi\cdot d_c^4}{64} \qquad I_c = \begin{pmatrix} 0.662 \\ 0.925 \\ 1.258 \\ 1.674 \end{pmatrix} ft^4$$

Moment of interia of steel pipe

$$I_{s\_0.5} := \frac{\pi \cdot \left(d_s^{\,4} - d_c^{\,4}\right)}{64} \qquad I_{s\_0.5} = \begin{pmatrix} 0.091 \\ 0.116 \\ 0.146 \\ 0.18 \end{pmatrix} ft^4$$

Composite Moment of Inertia

$$I_t := \frac{I_c}{n} + I_{s\_0.5} \qquad I_t = \begin{pmatrix} 0.178 \\ 0.238 \\ 0.311 \\ 0.4 \end{pmatrix} ft^4$$

Transformed area

$$A_{concrete\_0.5} := \pi \cdot \frac{d_c^{\,2}}{4}$$

$$A_t := A_1 + \frac{A_{concrete\_0.5}}{n} \qquad A_t = \begin{pmatrix} 0.571 \\ 0.656 \\ 0.747 \\ 0.844 \end{pmatrix} ft^2$$

LRFD Eq. 10.7.3.13.4-1 for fixity in feet : 1.4*(EpIp/Es)^0.25.
E must be in ksi and I in ft^4.
Es=0.465*Su (Su must be in ksf, result is in ksi)
Su=325 psf

Use same equation in NCHRP#343 pg 61:
Leq=Lu +1.4R
where Leq = equivalent free standing length of pile
Lu=unsupported length of pile extending above ground
for clays, R=(Ep*Ip/Es)^0.25

Soil Modulus of clay

$$E_{soil} := 67 \cdot Su \qquad E_{soil} = 21.775\,ksf$$

R parameter

$$R := \left(\frac{E_{steel} \cdot I_t}{E_{soil}}\right)^{0.25} \qquad R = \begin{pmatrix} 13.591 \\ 14.612 \\ 15.631 \\ 16.646 \end{pmatrix} ft$$

Depth for fixity $D_{0.5} := 1.4 \cdot R$

$$D_{0.5} = \begin{pmatrix} 19 \\ 20.5 \\ 21.9 \\ 23.3 \end{pmatrix} ft$$

Check first value in Array with LRFD Eq. 10.7.3.13.4-1

$E_{steel} = 29000\,ksi$ $E_{soil} := 0.465 \cdot (0.325) \cdot ksi$ $E_{soil} = 0.151\,ksi$

$$I_t = \begin{pmatrix} 0.178 \\ 0.238 \\ 0.311 \\ 0.4 \end{pmatrix} ft^4$$

$$Check := 1.4 \cdot \left(\frac{29000 \cdot 0.178}{0.151} \right)^{0.25}$$ $Check = 19.037$ ***OK***

Depth to Fixity of 5/8 wall pipe piles

Stiff to hard sandy silt, Su ranges from 250 to 400 psf - use average $Su := 325 \cdot psf$

Transformed pile propeties of 5/8 inch wall pipe pile

$n = 7.6$

diameter of concrete core $d_c := dia - 2 \cdot \frac{5}{8} \cdot in$

$$d_c = \begin{pmatrix} 22.75 \\ 24.75 \\ 26.75 \\ 28.75 \end{pmatrix} in$$

dia of steel pipe $d_s := dia_{corr}$

$$d_s = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} in$$

Moment of interia of concrete core $I_c := \frac{\pi \cdot d_c^4}{64}$

$$I_c = \begin{pmatrix} 0.634 \\ 0.888 \\ 1.212 \\ 1.617 \end{pmatrix} ft^4$$

Moment of interia of steel pipe $I_{s\_0.625} := \frac{\pi \cdot \left(d_s^4 - d_c^4 \right)}{64}$

$$I_{s\_0.625} = \begin{pmatrix} 0.119 \\ 0.152 \\ 0.192 \\ 0.237 \end{pmatrix} ft^4$$

Composite Moment of Inertia

$$I_t := \frac{I_c}{n} + I_{s\_0.625}$$

$$I_t = \begin{pmatrix} 0.202 \\ 0.269 \\ 0.351 \\ 0.45 \end{pmatrix} ft^4$$

Transformed area

$$A_{concrete\_0.625} := \pi \cdot \frac{d_c^{\,2}}{4}$$

$$A_t := A_2 + \frac{A_{concrete\_0.625}}{n}$$

$$A_t = \begin{pmatrix} 0.625 \\ 0.715 \\ 0.811 \\ 0.912 \end{pmatrix} ft^2$$

LRFD Eq. 10.7.3.13.4-1 for fixity in feet : 1.4*(EpIp/Es)^0.25.
E must be in ksi and I in ft^4.
Es=0.465*Su (Su must be in ksf, result is in ksi)
Su=3000 psf or 3 ksf

Use same equation in NCHRP#343 pg 61

Soil Modulus of clay

$$E_{soil} := 67 \cdot Su \qquad E_{soil} = 21.775\ ksf$$

R parameter

$$R := \left(\frac{E_{steel} \cdot I_t}{E_{soil}}\right)^{0.25}$$

$$R = \begin{pmatrix} 14.038 \\ 15.076 \\ 16.109 \\ 17.138 \end{pmatrix} ft$$

Depth for fixity

$$D_{0.625} := 1.4 \cdot R$$

$$D_{0.625} = \begin{pmatrix} 19.7 \\ 21.1 \\ 22.6 \\ 24 \end{pmatrix} ft$$

Check first value in Array with LRFD Eq. 10.7.3.13.4-1 which calls for specific units

$$E_{steel} = 29000\ ksi \qquad E_{soil} := 0.465 \cdot 0.325 \cdot ksi \qquad E_{soil} = 0.151\ ksi \qquad I_t = \begin{pmatrix} 0.202 \\ 0.269 \\ 0.351 \\ 0.45 \end{pmatrix} ft^4$$

$$Check := 1.4 \cdot \left(\frac{29000 \cdot 0.202}{0.151}\right)^{0.25} \qquad Check = 19.648$$

OK

Nominal Axial Structural Resistance of Pipe piles

Axial pile resistance may be controlled by structural resistance if driven to sound bedrock - unlikely at Boothbay site.
Check concurrent axial loading and moments with LRFD Equation 6.9.2.2-1.
Use 6.9.4.1-1 to compute the nominal compressive structural resistance for HP sections and LRFD 6.9.5.1-1 or 6.9.5.1-2 for Pipe Pile Sections

λ in equation 6.9.5.1-2 has to be computed for the pipes since the pipe piles have an unbraced length

Yield strength of steel shell $F_y := 45 \cdot ksi$

Compressive strength of concrete core $f_c := 4000 \cdot psi$

Yield strength of longitudinal reinforcment $F_{yr} := 60 \cdot ksi$

Compute λ per 6.9.4.1-3 for noncomposite members or 6.9.5.1-1 for composite members

Effective length factor per Article 4.6.2.5

Use case (c) in Table C4.6.2.5-1; since piles are fixed at tip with rock socketed H- or W-Sections, use K=1.0

$K := 1.0$

Exposed length of pile - see table 3 on Sheet of this Appendix D for exposed pile lengths

$L_{ex} := 27 \cdot ft$

L unbraced length of column; $L_{0.5} := L_{ex} + D_{0.5}$ $L_{0.5} = \begin{pmatrix} 46.027 \\ 47.457 \\ 48.883 \\ 50.304 \end{pmatrix} ft$

$L_{0.625} := L_{ex} + D_{0.625}$ $L_{0.625} = \begin{pmatrix} 46.653 \\ 48.107 \\ 49.553 \\ 50.993 \end{pmatrix} ft$

Longitudinal Reinforcement

Assume longitudinal reinforcement of 12 - #8 (1-in) rebar equally spaced for all pile sections

$A_r := 12 \cdot \frac{\pi \cdot (1 \cdot in)^2}{4}$ $A_r = 9.425\ in^2$

Composite Column Constant per Table 6.9.5.1.1

for filled tube sections C1=1.0, C2=0.85, C3=0.40

$C1 := 1.0 \qquad C2 := 0.85 \qquad C3 := 0.40$

Variable Fe

$$F_{e\_0.5} := F_y + C1 \cdot F_{yr} \cdot \frac{A_r}{A_1} + C2 \cdot f_c \cdot \frac{A_{concrete\_0.5}}{A_1} \qquad F_{e\_0.5} = \begin{pmatrix} 116.832 \\ 119.745 \\ 122.896 \\ 126.235 \end{pmatrix} ksi$$

$$F_{e\_0.625} := F_y + C1 \cdot F_{yr} \cdot \frac{A_r}{A_2} + C2 \cdot f_c \cdot \frac{A_{concrete\_0.625}}{A_2} \qquad F_{e\_0.625} = \begin{pmatrix} 98.327 \\ 100.499 \\ 102.852 \\ 105.347 \end{pmatrix} ksi$$

Radius of gyration of both sets of steel sections

$$r_{s\_0.5} := \sqrt{\frac{I_{s\_0.5}}{A_1}} \qquad r_{s\_0.5} = \begin{pmatrix} 0.689 \\ 0.748 \\ 0.807 \\ 0.866 \end{pmatrix} ft$$

$$r_{s\_0.625} := \sqrt{\frac{I_{s\_0.625}}{A_2}} \qquad r_{s\_0.625} = \begin{pmatrix} 0.685 \\ 0.744 \\ 0.803 \\ 0.862 \end{pmatrix} ft$$

Ee Term

$$E_{e\_0.5} := E_{steel} \cdot \left(1 + \frac{C3}{n} \cdot \frac{A_{concrete\_0.5}}{A_1}\right) \qquad E_{e\_0.5} = \begin{pmatrix} 52028 \\ 54063 \\ 56097 \\ 58132 \end{pmatrix} ksi$$

$$E_{e\_0.625} := E_{steel} \cdot \left(1 + \frac{C3}{n} \cdot \frac{A_{concrete\_0.625}}{A_2}\right) \qquad E_{e\_0.625} = \begin{pmatrix} 45988 \\ 47514 \\ 49040 \\ 50566 \end{pmatrix} ksi$$

Lamda term

$$\lambda_{0.5} := \overrightarrow{\left[\left(\frac{K \cdot L_{0.5}}{r_{s\_0.5} \cdot \pi}\right)^2 \cdot \frac{F_{e\_0.5}}{E_{e\_0.5}}\right]} \qquad \lambda_{0.5} = \begin{pmatrix} 1.016 \\ 0.904 \\ 0.815 \\ 0.743 \end{pmatrix}$$

$$\lambda_{0.625} := \overrightarrow{\left[\left(\frac{K \cdot L_{0.625}}{r_{s\_0.625} \cdot \pi}\right)^2 \cdot \frac{F_{e\_0.625}}{E_{e\_0.625}}\right]} \qquad \lambda_{0.625} = \begin{pmatrix} 1.004 \\ 0.896 \\ 0.809 \\ 0.739 \end{pmatrix}$$

Lamda term for noncomposite members LRFD Eq. 6.9.4.1-3

$$\lambda_{0.5\_tip} := \overrightarrow{\left[\left(\frac{K \cdot L_{0.5}}{r_{s\_0.5} \cdot \pi}\right)^2 \cdot \frac{F_y}{E_{steel}}\right]} \qquad \lambda_{0.5\_tip} = \begin{pmatrix} 0.702 \\ 0.633 \\ 0.577 \\ 0.531 \end{pmatrix}$$

$$\lambda_{0.625\_tip} := \overrightarrow{\left[\left(\frac{K \cdot L_{0.625}}{r_{s\_0.625} \cdot \pi}\right)^2 \cdot \frac{F_y}{E_{steel}}\right]} \qquad \lambda_{0.625\_tip} = \begin{pmatrix} 0.729 \\ 0.657 \\ 0.599 \\ 0.55 \end{pmatrix}$$

Nominal Axial Structural Resistance of 1/2-inch wall

Since λ<2.25 use Eq. 6.9.5.1-1

$$P_{n\_0.5} := \overrightarrow{\left(0.66^{\lambda_{0.5}} \cdot F_{e\_0.5} \cdot A_1\right)} \qquad P_{n\_0.5} = \begin{pmatrix} 2109 \\ 2459 \\ 2825 \\ 3208 \end{pmatrix} \text{kip}$$

At the bottom of open-ended pile piles, or closed ended pipes where the conical tip or closed tip experiences breaching, the nominal compressive resistance is a function of only the steel pipe

$$P_{n\_0.5tip} := \overrightarrow{\left(0.66^{\lambda_{0.5\_tip}} \cdot F_y \cdot A_1\right)} \qquad P_{n\_0.5tip} = \begin{pmatrix} 926 \\ 1034 \\ 1142 \\ 1249 \end{pmatrix} \text{kip}$$

Use this value for structural capacity

Nominal Axial Structural Resistance of 5/8-inch wall

$$P_{n\_0.625} := \overrightarrow{\left(0.66^{\lambda_{0.625}} \cdot F_{e\_0.625} \cdot A_2\right)}$$

$$P_{n\_0.625} = \begin{pmatrix} 2366 \\ 2747 \\ 3145 \\ 3561 \end{pmatrix} \text{kip}$$

For lower portion of open ended pipe piles, or pipe piles where the tip has breached, the nominal compressive resistance is a fct of just the steel pipe wall

$$P_{n\_0.625tip} := \overrightarrow{\left(0.66^{\lambda_{0.625\_tip}} \cdot F_y \cdot A_2\right)}$$

$$P_{n\_0.625tip} = \begin{pmatrix} 1214 \\ 1358 \\ 1502 \\ 1645 \end{pmatrix} \text{kip}$$

Use this for nominal structural capacity at Pier 3

Factored Axial Structural Resistance of single pipe pile

Strength State resistance factor for pipe piles in compression, no damage anticipated - LRFD 6.5.4.2

$$\phi_c := 0.70$$

Factored Structural Resistance (Pr)

$$P_{r\_0.5} := \phi_c \cdot P_{n\_0.5}$$

$$P_{r\_0.625} := \phi_c \cdot P_{n\_0.625}$$

$$P_{r\_0.5} = \begin{pmatrix} 1477 \\ 1721 \\ 1977 \\ 2246 \end{pmatrix} \text{kip} \qquad P_{r\_0.625} = \begin{pmatrix} 1656 \\ 1923 \\ 2202 \\ 2492 \end{pmatrix} \text{kip}$$

Factored Structural Resistance (Pr) for the lower portion of open-ended pile piles or breached close-ended pipe piles is a function of only the steel shell

$$P_{n\_0.5tip} = \begin{pmatrix} 926 \\ 1034 \\ 1142 \\ 1249 \end{pmatrix} \text{kip} \qquad P_{n\_0.625tip} = \begin{pmatrix} 1214 \\ 1358 \\ 1502 \\ 1645 \end{pmatrix} \text{kip}$$

$$P_{r\_0.5tip} := \phi_c \cdot P_{n\_0.5tip}$$

$$P_{r\_0.625tip} := \phi_c \cdot P_{n\_0.625tip}$$

$$P_{r\_0.5tip} = \begin{pmatrix} 648 \\ 724 \\ 799 \\ 874 \end{pmatrix} kip \qquad P_{r\_0.625tip} = \begin{pmatrix} 850 \\ 951 \\ 1051 \\ 1151 \end{pmatrix} kip$$

Use these for Factored Structural Resistance at Pier 3

Nominal Pile Geotechnical Resistance- Kulhawy and Goodman - RQD method

Ref: "Pile Design & Construction Practice", Tomlinson, page 139

For calculating maximum end bearing allowable load -this method ignores side resistance - use Driven to calculate that.

Correct for wedge failure under strip footing:

multiply cNc by 1.25 - square piles, 1.2 for circular piles
multiply $\gamma BN\gamma$ by 0.8 - square pile and 0.7 for circular piles

For RQD 0 -70 %

qc=0.33 x Quc
c=0.1 x Quc
ϕ=30 degrees

For RQD 70 - 100 %

qc=0.33 to 0.88 x Quc
c=0.10 x Quc
ϕ=30 to 60 degrees

Calculation

assume RQD = 83%
ϕ = 27 degrees ***PELLS & TURNER, TOMLINSON, PAGE 140***
c= .10 x Quc *Assume pile penetrates 0 inches into bedrock*
qc = 0.58 x Quc

$q_{uc} := 15000 \cdot psi$ $\quad c := 0.1 \cdot q_{uc}$ $\quad c = 1500\,psi$

$D := .0 \cdot in$

$B_{min} := 24 \cdot in$

$\gamma_r := 145 \cdot \frac{lbf}{ft^3}$ $\quad q_c := 0.58 \cdot q_{uc}$ $\quad q_c = 8700\,psi$

$N_c := 11.95$ $N_q := 7.09$ $N_\gamma := 9.94$ ***(BASED ON PELLS & TURNER, TOMLINSON, PAGE 140)***

$$q_{ub} := 1.2 \cdot c \cdot N_c + \frac{\gamma_r \cdot B_{min} \cdot N_\gamma}{2} \cdot 0.7 + \gamma_r \cdot D \cdot N_q$$

This is ultimate base resistance, settlement up to 20% of base dia. needed to mobilize. So to ensure settlements at working load are within allowable limits, limit working load to q ub divided by 3.5

$q_{ub} = 21.517\ \text{ksi}$

$R_{p\_nom\_0.5\_kulh} := q_{ub} \cdot A_{1\_noloss}$ $R_{p\_nom\_0.5\_kulh} = \begin{pmatrix} 794 \\ 862 \\ 929 \\ 997 \end{pmatrix} \text{kip}$

$R_{p\_nom\_0.625\_kulh} := q_{ub} \cdot A_{2\_noloss}$ $R_{p\_nom\_0.625\_kulh} = \begin{pmatrix} 988 \\ 1072 \\ 1157 \\ 1241 \end{pmatrix} \text{kip}$

Factored Resistances

Factor to account for method controlling pile installation- dynamic tests on 2% piles and Capwap $\lambda_v := 1.0$

Resistance factor, end bearing on rock CGS method $\phi_{qp} := 0.45 \cdot \lambda_v$

$R_{r\_tip\_0.5\_kulh} := R_{p\_nom\_0.5\_kulh} \cdot \phi_{qp}$ $R_{r\_tip\_0.5\_kulh} = \begin{pmatrix} 357 \\ 388 \\ 418 \\ 449 \end{pmatrix} \text{kip}$

$R_{r\_tip\_0.625\_kulh} := R_{p\_nom\_0.625\_kulh} \cdot \phi_{qp}$ $R_{r\_tip\_0.625\_kulh} = \begin{pmatrix} 444 \\ 482 \\ 520 \\ 558 \end{pmatrix} \text{kip}$

Nominal geotechnical tip resistance, Rp, of plug and steel shell area (ref FHWA LRFD Design Example) assume unplugged section

$$A_{plug\_0.5} := 0 \cdot \left(A_{concrete\_0.5} \cdot \frac{1}{3} + A_{1\_noloss} \right)$$

$$A_{plug\_0.625} := 0 \cdot \left(A_{concrete\_0.625} \cdot \frac{1}{3} + A_{2\_noloss} \right)$$

$$R_{p\_plugged\_0.5} := \overrightarrow{\left(q_{ub} \cdot A_{plug\_0.5} \right)}$$

$$R_{p\_plugged\_0.5} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} kip$$

$$R_{p\_plugged\_0.625} := \overrightarrow{\left(q_{ub} \cdot A_{plug\_0.625} \right)}$$

$$R_{p\_plugged\_0.625} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} kip$$

Nominal Geotechnical Side Frictional Resistance

The piles will be primarily end bearing. At piers 1, 3, 4 and 5 no skin friction in the overlying glaciomarine and marine clayey silt.

The nominal skin friction is computed using FHWA program Driven 1.0

Driven software uses Norlund/Thurman Method for side frictional resistance in cohesionless soils. Use a ϕ of 0.45 per LRFD Table 10.5.5.2.3-1

$$\phi_{side} := 0.45$$

Nominal side resistance (from Driven)

$$R_{s\_0.5} := \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kip \qquad R_{s\_0.625} := \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kip$$

Factored Geotechnical Side Frictional Resistance

$$R_{r\_s0.5} := R_{s\_0.5} \cdot \phi_{side} \qquad R_{r\_s0.625} := R_{s\_0.625} \cdot \phi_{side}$$

$$R_{r\_s0.5} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} kip \qquad R_{r\_s0.625} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} kip$$

Total Factored Geotechnical Bearing Resistance of Piles - End bearing of shell + 0% of the plugged area on rock + side frictional resistance

$R_{r\_0.5total} := R_{r\_tip\_0.5\_kulh} + R_{r\_s0.5}$

$$R_{r\_0.5total} = \begin{pmatrix} 357 \\ 388 \\ 418 \\ 449 \end{pmatrix} kip$$

$R_{r\_0.625total} := R_{r\_tip\_0.625\_kulh} + R_{r\_s0.625}$

$$R_{r\_0.625total} = \begin{pmatrix} 444 \\ 482 \\ 520 \\ 558 \end{pmatrix} kip$$

USE THIS FOR PROJECT for both open and closed ended pipe piles - as conical tips and flat pile tips have ruptured on some piles

Service Limit State" Factored " Axial Geotechnical Tip Resistance of single pipe pile

Resistance factor for service limit state $\phi_c := 1.0$

Nominal side resistance (from Driven) and tip resistance

$$R_{s\_0.5} := \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kip \qquad R_{s\_0.625} := \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kip$$

$$R_{p\_plugged\_0.5} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} kip \qquad R_{p\_plugged\_0.625} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} kip$$

Service Limit State Geotechnical Resistance

$$R_{service\_0.5} := R_{p\_nom\_0.5\_kulh} \cdot \phi_c \qquad R_{service\_0.5} = \begin{pmatrix} 794 \\ 862 \\ 929 \\ 997 \end{pmatrix} kip$$

$$R_{service\_0.625} := R_{p\_nom\_0.625\_kulh} \cdot \phi_c \qquad R_{service\_0.625} = \begin{pmatrix} 988 \\ 1072 \\ 1157 \\ 1241 \end{pmatrix} kip$$

Compute Ultimate Resistance that must be achieved during the driveability wave equation analysis

The ultimate resistance that must be achieved during wave equation analysis will be the APPLIED MAXIMUM AXIAL PILE LOAD {must be < FACTORED RESISTANCE (the factored geotechnical resistance governs for all pile sections)} divided by the appropriate resistance factor for wave equation analysis & dynamic test.

Resistance factor per Table 10.5.5.2.3-1 for dynamic testing is ϕ=0.65, per Table 3 which requires no less than 3 to 4 piles are to be tested considering low to medium site variablity.

$\phi_{dyn} := 0.65$

For the 1/2-in wall piles Rr_0.5total governs the factored resistance

$$Q_{p\_0.5} := \frac{R_{r\_0.5total}}{\phi_{dyn}}$$

$$Q_{p\_0.5} = \begin{pmatrix} 550 \\ 597 \\ 643 \\ 690 \end{pmatrix} kip$$

For the 5/8-in wall piles, the factored geotech resistance governs (Rr_0.625total)

$$Q_{p\_0.625} := \frac{R_{r\_0.625total}}{\phi_{dyn}}$$

$$Q_{p\_0.625} = \begin{pmatrix} 684 \\ 742 \\ 801 \\ 859 \end{pmatrix} kip$$

The resistance required by the driveablity wave equation analysis is:

$$Q_{p\_0.5} = \begin{pmatrix} 550 \\ 597 \\ 643 \\ 690 \end{pmatrix} kip$$

$$Q_{p\_0.625} = \begin{pmatrix} 684 \\ 742 \\ 801 \\ 859 \end{pmatrix} kip$$

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $tonf := g \cdot ton$ $tsf := \frac{tonf}{ft^2}$

$psi := \frac{lbf}{in^2}$ $kip := 1000 \cdot lbf$ $kN := 10^3 \cdot newton$ $kPa := 10^3 \cdot Pa$

$MN := 10^6 \cdot newton$ $ksf := \frac{kip}{ft^2}$ $MPa := 10^6 \cdot Pa$ $ksi := \frac{kip}{in^2}$

Bedrock Properties

Pipe pile capacity based on steel shell end bearing on bedrock - assume driven through soft glaciomarine silt clay deposit

RQD from bedrock cores ranges from:
27% to 35% in the most weathered and fractured zones of the bedrock formation
50% to 83% in the moderately to slightly weathered and fractured zones

Rock Type: Mostly banded SCHIST and GNEISS
- Biotite SCHIST - more weathered & fractured zones
- GNEISS - better RQD
- PEGMATITE GRANITE veins

Perform 4 designs:

1 & 2. RQD of 27% for design (for Biotite SCHIST) at Pier 1, 35% SCHIST Pier 2, ϕ = 20-27 (AASHTO LRFD Table C.10.4.6.4-1); AASHTO TABLE 4.4.8.1.2.B uniaxial compressive strength = Cu= 1400 to 21000 psi - use 7500 psi for design

3. RQD of 57% biotite SCHIST at Pier 4 and RQD of 50% at Pier 5 in weathered GNEISS; ϕ = 20-34 (AASHTO LRFD Table C.10.4.6.4-1) for GNEISS and SCHIST; use 30 for design. AASHTO TABLE 4.4.8.2.1.B uniaxial compressive strength = 3,500 - 45,000 psi for gneiss and 1,400 to 21,000 for schist- 8000 psi for design considering the weathered and fractured nature, and open joints with infilling.

4. RQD of 83% at Pier 3, GNEISS, hard, relative unweathered, slightly fractured, use ϕ = 27-34 (AASHTO LRFD Table C.10.4.6.4-1); AASHTO TABLE 4.4.8.1.2.B uniaxial compressive strength = 3,500 - 45,000 psi; use 15,000 psi for design

Pipe Pile Properties

Use the following pipe pile diameters 24" - 30", 1/2 and 5/8 walls

$i := 1, 2 .. 6$

$$dia := \begin{pmatrix} 24 \\ 26 \\ 28 \\ 30 \end{pmatrix} \cdot in \qquad wall := \begin{pmatrix} \frac{1}{2} \\ \frac{5}{8} \end{pmatrix} \cdot in \qquad c := \frac{1}{8} \cdot in$$

Sacrifial steel loss due to corrosion per BDG

$$dia_{corr} := dia - 2 \cdot c \qquad dia_{corr} = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} in$$

$$wall_{corr} := wall - c \qquad wall_{corr} = \begin{pmatrix} 0.375 \\ 0.5 \end{pmatrix} in$$

Steel Area of 1/2-inch wall pipe piles assuming corrosion loss of 1/8" per BDG

$$A_1 := \left[\pi \cdot \left(\frac{dia_{corr}}{2} \right)^2 - \pi \cdot \left(\frac{dia_{corr} - 2 \cdot wall_{corr_0}}{2} \right)^2 \right] \qquad A_1 = \begin{pmatrix} 27.538 \\ 29.894 \\ 32.25 \\ 34.607 \end{pmatrix} in^2$$

$$A_{1\_noloss} := \left[\pi \cdot \left(\frac{dia}{2} \right)^2 - \pi \cdot \left(\frac{dia - 2 \cdot wall_0}{2} \right)^2 \right] \qquad A_{1\_noloss} = \begin{pmatrix} 36.914 \\ 40.055 \\ 43.197 \\ 46.338 \end{pmatrix} in^2$$

Steel Area of 5/8-inch wall pipe piles

$$A_2 := \left[\pi \cdot \left(\frac{dia_{corr}}{2} \right)^2 - \pi \cdot \left(\frac{dia_{corr} - 2 \cdot wall_{corr_1}}{2} \right)^2 \right] \qquad A_2 = \begin{pmatrix} 36.521 \\ 39.663 \\ 42.804 \\ 45.946 \end{pmatrix} in^2$$

$$A_{2\_noloss} := \left[\pi\cdot\left(\frac{dia}{2}\right)^2 - \pi\cdot\left(\frac{dia - 2\cdot wall_1}{2}\right)^2\right] \qquad A_{2\_noloss} = \begin{pmatrix} 45.897 \\ 49.824 \\ 53.751 \\ 57.678 \end{pmatrix} in^2$$

Depth to Fixity of 1/2-inch wall pipe piles at Pier 4 and 5

27 to 30 feet of very soft to soft marine and glaciomarine clayey silt over 2 to 6 feet of very soft glaciomarine clayey silt, Su ranges from 125 psf (correlation to WOR) to 400 psf with 2 high values of 571 and 1250

Transformed pile propeties of 1/2 inch wall pipe pile

$Su := 360\cdot psf$

unit weight of concrete in kcf $\quad wc := 0.15$

compressive strength of concrete in ksi $\quad fc := 4.450$

modulus of elasticity - concrete $\quad E_c := 33000\cdot wc^{1.5}\cdot\sqrt{fc}\cdot 1000\cdot psi \qquad E_c = 4.044\times 10^3\, ksi$

steel modulus $\quad E_{steel} := 29000\cdot ksi$

Modular ratio, n $\quad n := \frac{E_{steel}}{E_c} \qquad n = 7.171$

MDOT Structural engineers routinely just use n = 7.6 $\qquad n := 7.6$

diameter of concrete core $\quad d_c := dia - 2\cdot\frac{1}{2}\cdot in \qquad d_c = \begin{pmatrix} 23 \\ 25 \\ 27 \\ 29 \end{pmatrix} in$

dia of steel pipe

$d_s := dia_{corr} \qquad d_s = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} in$

Moment of interia of concrete core $\quad I_c := \frac{\pi\cdot d_c^4}{64} \qquad I_c = \begin{pmatrix} 0.662 \\ 0.925 \\ 1.258 \\ 1.674 \end{pmatrix} ft^4$

Moment of interia of steel pipe

$$I_{s\_0.5} := \frac{\pi \cdot \left(d_s^{\,4} - d_c^{\,4}\right)}{64} \qquad I_{s\_0.5} = \begin{pmatrix} 0.091 \\ 0.116 \\ 0.146 \\ 0.18 \end{pmatrix} ft^4$$

Composite Moment of Inertia

$$I_t := \frac{I_c}{n} + I_{s\_0.5} \qquad I_t = \begin{pmatrix} 0.178 \\ 0.238 \\ 0.311 \\ 0.4 \end{pmatrix} ft^4$$

Transformed area

$$A_{concrete\_0.5} := \pi \cdot \frac{d_c^{\,2}}{4}$$

$$A_t := A_1 + \frac{A_{concrete\_0.5}}{n} \qquad A_t = \begin{pmatrix} 0.571 \\ 0.656 \\ 0.747 \\ 0.844 \end{pmatrix} ft^2$$

LRFD Eq. 10.7.3.13.4-1 for fixity in feet : 1.4*(EpIp/Es)^0.25.
E must be in ksi and I in ft^4.
Es=0.465*Su (Su must be in ksf, result is in ksi)
Su=325 psf

Use same equation in NCHRP#343 pg 61:
Leq=Lu +1.4R
where Leq = equivalent free standing length of pile
Lu=unsupported length of pile extending above ground
for clays, R=(Ep*Ip/Es)^0.25

Soil Modulus of clay

$$E_{soil} := 67 \cdot Su \qquad E_{soil} = 24.12\ ksf$$

R parameter

$$R := \left(\frac{E_{steel} \cdot I_t}{E_{soil}}\right)^{0.25} \qquad R = \begin{pmatrix} 13.248 \\ 14.244 \\ 15.236 \\ 16.226 \end{pmatrix} ft$$

Depth for fixity $D_{0.5} := 1.4 \cdot R$

$$D_{0.5} = \begin{pmatrix} 18.547 \\ 19.941 \\ 21.331 \\ 22.716 \end{pmatrix} ft$$

Check first value in Array with LRFD Eq. 10.7.3.13.4-1

$E_{steel} = 29000\,ksi$ $E_{soil} := 0.465 \cdot (0.360) \cdot ksi$ $E_{soil} = 0.167\,ksi$

$$I_t = \begin{pmatrix} 0.178 \\ 0.238 \\ 0.311 \\ 0.4 \end{pmatrix} ft^4$$

$$Check := 1.4 \cdot \left(\frac{29000 \cdot 0.178}{0.167} \right)^{0.25}$$ $Check = 18.563$ ***OK***

Depth to Fixity of 5/8 wall pipe piles

Su estimated at 125 psf in upper 10 ft, and vanes in lower PF are 357, 380, 571, 1250 psf - use 360 psf $Su := 360 \cdot psf$

Transformed pile propeties of 5/8 inch wall pipe pile

$n = 7.6$

diameter of concrete core $d_c := dia - 2 \cdot \frac{5}{8} \cdot in$

$$d_c = \begin{pmatrix} 22.75 \\ 24.75 \\ 26.75 \\ 28.75 \end{pmatrix} in$$

dia of steel pipe $d_s := dia_{corr}$

$$d_s = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} in$$

Moment of interia of concrete core $I_c := \frac{\pi \cdot d_c^4}{64}$

$$I_c = \begin{pmatrix} 0.634 \\ 0.888 \\ 1.212 \\ 1.617 \end{pmatrix} ft^4$$

Moment of interia of steel pipe

$$I_{s\_0.625} := \frac{\pi \cdot \left(d_s^{\,4} - d_c^{\,4}\right)}{64} \qquad I_{s\_0.625} = \begin{pmatrix} 0.119 \\ 0.152 \\ 0.192 \\ 0.237 \end{pmatrix} ft^4$$

Composite Moment of Inertia

$$I_t := \frac{I_c}{n} + I_{s\_0.625} \qquad I_t = \begin{pmatrix} 0.202 \\ 0.269 \\ 0.351 \\ 0.45 \end{pmatrix} ft^4$$

Transformed area

$$A_{concrete\_0.625} := \pi \cdot \frac{d_c^{\,2}}{4}$$

$$A_t := A_2 + \frac{A_{concrete\_0.625}}{n} \qquad A_t = \begin{pmatrix} 0.625 \\ 0.715 \\ 0.811 \\ 0.912 \end{pmatrix} ft^2$$

LRFD Eq. 10.7.3.13.4-1 for fixity in feet : 1.4*(EpIp/Es)^0.25.
E must be in ksi and I in ft^4.
Es=0.465*Su (Su must be in ksf, result is in ksi)
Su=3000 psf or 3 ksf

Use same equation in NCHRP#343 pg 61

Soil Modulus of clay

$$E_{soil} := 67 \cdot Su \qquad E_{soil} = 24.12\ ksf$$

R parameter

$$R := \left(\frac{E_{steel} \cdot I_t}{E_{soil}}\right)^{0.25} \qquad R = \begin{pmatrix} 13.684 \\ 14.696 \\ 15.703 \\ 16.706 \end{pmatrix} ft$$

Depth for fixity

$$D_{0.625} := 1.4 \cdot R \qquad D_{0.625} = \begin{pmatrix} 19.157 \\ 20.574 \\ 21.984 \\ 23.388 \end{pmatrix} ft$$

Check first value in Array with LRFD Eq. 10.7.3.13.4-1 which calls for specific units

$$E_{steel} = 29000\ ksi \qquad E_{soil} := 0.465 \cdot 0.360 \cdot ksi \qquad E_{soil} = 0.167\ ksi \qquad I_t = \begin{pmatrix} 0.202 \\ 0.269 \\ 0.351 \\ 0.45 \end{pmatrix} ft^4$$

$$Check := 1.4 \cdot \left(\frac{29000 \cdot 0.202}{0.167} \right)^{0.25} \qquad Check = 19.16$$ ***OK***

Nominal Axial Structural Resistance of Pipe piles

Axial pile resistance may be controlled by structural resistance if driven to sound bedrock - unlikely at Boothbay site.
Check concurrent axial loading and moments with LRFD Equation 6.9.2.2-1.
Use 6.9.4.1-1 to compute the nominal compressive structural resistance for HP sections and LRFD 6.9.5.1-1 or 6.9.5.1-2 for Pipe Pile Sections

λ in equation 6.9.5.1-2 has to be computed for the pipes since the pipe piles have an unbraced length

Yield strength of steel shell $F_y := 45 \cdot ksi$

Compressive strength of concrete core $f_c := 4000 \cdot psi$

Yield strength of longitudinal reinforcment $F_{yr} := 60 \cdot ksi$

Compute λ per 6.9.4.1-3 for noncomposite members or 6.9.5.1-1 for composite members

Effective length factor per Article 4.6.2.5

Use case (c) in Table C4.6.2.5-1; appropriate for fixed pile tips in bedrock sockets

$K := 1.0$

Exposed Pile Length - see Table 3 on sheet 1 of Appendix D. $L_{ex} := 22 \cdot ft$
Use exposed pile length at Pier 4.

L unbraced length of column $L_{0.5} := L_{ex} + D_{0.5}$ $\qquad L_{0.5} = \begin{pmatrix} 40.547 \\ 41.941 \\ 43.331 \\ 44.716 \end{pmatrix} ft$

$L_{0.625} := L_{ex} + D_{0.625}$ $\qquad L_{0.625} = \begin{pmatrix} 41.157 \\ 42.574 \\ 43.984 \\ 45.388 \end{pmatrix} ft$

Longitudinal Reinforcement

Assume longitudinal reinforcement of 12 - #8 (1-in) rebar equally spaced for all pile sections

$$A_r := 12 \cdot \frac{\pi \cdot (1 \cdot in)^2}{4} \qquad A_r = 9.425\ in^2$$

Composite Column Constant per Table 6.9.5.1.1

for filled tube sections C1=1.0, C2=0.85, C3=0.40

$$C1 := 1.0 \qquad C2 := 0.85 \qquad C3 := 0.40$$

Variable Fe

$$F_{e\_0.5} := F_y + C1 \cdot F_{yr} \cdot \frac{A_r}{A_1} + C2 \cdot f_c \cdot \frac{A_{concrete\_0.5}}{A_1} \qquad F_{e\_0.5} = \begin{pmatrix} 116.832 \\ 119.745 \\ 122.896 \\ 126.235 \end{pmatrix} ksi$$

$$F_{e\_0.625} := F_y + C1 \cdot F_{yr} \cdot \frac{A_r}{A_2} + C2 \cdot f_c \cdot \frac{A_{concrete\_0.625}}{A_2} \qquad F_{e\_0.625} = \begin{pmatrix} 98.327 \\ 100.499 \\ 102.852 \\ 105.347 \end{pmatrix} ksi$$

Radius of gyration of both sets of steel sections

$$r_{s\_0.5} := \sqrt{\frac{I_{s\_0.5}}{A_1}} \qquad r_{s\_0.5} = \begin{pmatrix} 0.689 \\ 0.748 \\ 0.807 \\ 0.866 \end{pmatrix} ft$$

$$r_{s\_0.625} := \sqrt{\frac{I_{s\_0.625}}{A_2}} \qquad r_{s\_0.625} = \begin{pmatrix} 0.685 \\ 0.744 \\ 0.803 \\ 0.862 \end{pmatrix} ft$$

Ee Term

$$E_{e\_0.5} := E_{steel} \cdot \left(1 + \frac{C3}{n} \cdot \frac{A_{concrete\_0.5}}{A_1}\right) \qquad E_{e\_0.5} = \begin{pmatrix} 52028 \\ 54063 \\ 56097 \\ 58132 \end{pmatrix} ksi$$

$$E_{e\_0.625} := E_{steel} \cdot \left(1 + \frac{C3}{n} \cdot \frac{A_{concrete\_0.625}}{A_2}\right) \qquad E_{e\_0.625} = \begin{pmatrix} 45988 \\ 47514 \\ 49040 \\ 50566 \end{pmatrix} ksi$$

Lamda term

$$\lambda_{0.5} := \overrightarrow{\left[\left(\frac{K \cdot L_{0.5}}{r_{s\_0.5} \cdot \pi}\right)^2 \cdot \frac{F_{e\_0.5}}{E_{e\_0.5}}\right]} \qquad \lambda_{0.5} = \begin{pmatrix} 0.788 \\ 0.706 \\ 0.641 \\ 0.587 \end{pmatrix}$$

$$\lambda_{0.625} := \overrightarrow{\left[\left(\frac{K \cdot L_{0.625}}{r_{s\_0.625} \cdot \pi}\right)^2 \cdot \frac{F_{e\_0.625}}{E_{e\_0.625}}\right]} \qquad \lambda_{0.625} = \begin{pmatrix} 0.782 \\ 0.702 \\ 0.638 \\ 0.585 \end{pmatrix}$$

Lamda term for noncomposite members LRFD Eq. 6.9.4.1-3

$$\lambda_{0.5\_tip} := \overrightarrow{\left[\left(\frac{K \cdot L_{0.5}}{r_{s\_0.5} \cdot \pi}\right)^2 \cdot \frac{F_y}{E_{steel}}\right]} \qquad \lambda_{0.5\_tip} = \begin{pmatrix} 0.545 \\ 0.495 \\ 0.454 \\ 0.42 \end{pmatrix}$$

$$\lambda_{0.625\_tip} := \overrightarrow{\left[\left(\frac{K \cdot L_{0.625}}{r_{s\_0.625} \cdot \pi}\right)^2 \cdot \frac{F_y}{E_{steel}}\right]} \qquad \lambda_{0.625\_tip} = \begin{pmatrix} 0.567 \\ 0.515 \\ 0.472 \\ 0.436 \end{pmatrix}$$

Nominal Axial Structural Resistance of 1/2-inch wall

Since λ<2.25 use Eq. 6.9.5.1-1

$$P_{n\_0.5} := \overrightarrow{\left(0.66^{\lambda_{0.5}} \cdot F_{e\_0.5} \cdot A_1\right)} \qquad P_{n\_0.5} = \begin{pmatrix} 2319 \\ 2669 \\ 3037 \\ 3423 \end{pmatrix} kip$$

At the bottom of open-ended pile piles, or closed ended pipes where the conical tip or closed tip experiences breaching, the nominal compressive resistance is a function of only the steel pipe

$$P_{n\_0.5tip} := \overrightarrow{\left(0.66^{\lambda_{0.5\_tip}} \cdot F_y \cdot A_1\right)}$$

$$P_{n\_0.5tip} = \begin{pmatrix} 988 \\ 1095 \\ 1202 \\ 1308 \end{pmatrix} kip$$

Use this for nominal structural capacity at Piers 4 and 5

Nominal Axial Structural Resistance of 5/8-inch wall

$$P_{n\_0.625} := \overrightarrow{\left(0.66^{\lambda_{0.625}} \cdot F_{e\_0.625} \cdot A_2\right)}$$

$$P_{n\_0.625} = \begin{pmatrix} 2595 \\ 2978 \\ 3378 \\ 3795 \end{pmatrix} kip$$

For lower portion of open ended pipe piles, or pipe piles where the tip has breached, the nominal compressive resistance is a fct of just the steel pipe wall

$$P_{n\_0.625tip} := \overrightarrow{\left(0.66^{\lambda_{0.625\_tip}} \cdot F_y \cdot A_2\right)}$$

$$P_{n\_0.625tip} = \begin{pmatrix} 1298 \\ 1441 \\ 1583 \\ 1725 \end{pmatrix} kip$$

Use this for nominal structural capacity at Piers 4 and 5

Nominal Axial Geotechnical Resistance of pipe piles

Kulhawy and Goodman - RQD method

Ref: "Pile Design & Construction Practice", Tomlinson, page 139

For calculating maximum end bearing allowable load -this method ignores side resistance - use Driven to calculate that.

Correct for wedge failure under strip footing:

multiply cNc by 1.25 - square piles, 1.2 for circular piles
multiply $\gamma BN\gamma$ by 0.8 - square pile and 0.7 for circular piles

For RQD 0 -70 %

qc=0.33 x Quc
c=0.1 x Quc
ϕ=30 degrees

For RQD 70 - 100 %

qc=0.33 to 0.88 x Quc
c=0.10 x Quc
ϕ=30 to 60 degrees

Calculation

assume RQD = 83%
ϕ = 30 ***PELLS & TURNER, TOMLINSON, PAGE 140***
c= .10 x Quc *Assume pile penetrates 0 inches into bedrock*
qc = 0.58 x Quc

$q_{uc} := 8000 \cdot psi$ $\quad c := 0.1 \cdot q_{uc}$ $\quad c = 800\,psi$

$D := .0 \cdot in$

$B_{min} := 24 \cdot in$

$\gamma := 145 \cdot \frac{lbf}{ft^3}$ $\quad q_c := 0.58 \cdot q_{uc}$ $\quad q_c = 4640\,psi$

$N_c := 13.86$ $\quad N_q := 9.0$ $\quad N_\gamma := 13.86$ ***(BASED ON PELLS & TURNER, TOMLINSON, PAGE 140)***

$$q_{ub} := 1.2 \cdot c \cdot N_c + \frac{\gamma \cdot B_{min} \cdot N_\gamma}{2} \cdot 0.7 + \gamma \cdot D \cdot N_q$$

This is ultimate base resistance, settlement up to 20% of base dia. needed to mobilize. So to ensure settlements at working load are within allowable limits, limit working load to q ub divided by 3.5

$q_{ub} = 13.315\,ksi$

$R_{p\_0.5\_kulh} := q_{ub} \cdot A_{1\_noloss}$ $\quad R_{p\_0.5\_kulh} = \begin{pmatrix} 492 \\ 533 \\ 575 \\ 617 \end{pmatrix} kip$

$R_{p\_0.625\_kulh} := q_{ub} \cdot A_{2\_noloss}$ $\quad R_{p\_0.625\_kulh} = \begin{pmatrix} 611 \\ 663 \\ 716 \\ 768 \end{pmatrix} kip$

Nominal geotechnical tip resistance, Rp, of plug and steel shell area (ref FHWA LRFD Design Example) assume unplugged section

$$A_{plug\_0.5} := 0 \cdot \left(A_{concrete\_0.5} \cdot \frac{1}{3} + A_{1\_noloss} \right)$$

$$A_{plug\_0.625} := 0 \cdot \left(A_{concrete\_0.625} \cdot \frac{1}{3} + A_{2\_noloss} \right)$$

$$R_{p\_plugged\_0.5} := \overrightarrow{\left(q_{ub} \cdot A_{plug\_0.5} \right)}$$

$$R_{p\_plugged\_0.5} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \text{kip}$$

$$R_{p\_plugged\_0.625} := \overrightarrow{\left(q_{ub} \cdot A_{plug\_0.625} \right)}$$

$$R_{p\_plugged\_0.625} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \text{kip}$$

Factored Axial Structural Resistance of single pipe pile

Strength State resistance factor for pipe piles in compression, no damage anticipated - LRFD 6.5.4.2

$$\phi_c := 0.70$$

Factored Structural Resistance (Pr)

$$P_{r\_0.5} := \phi_c \cdot P_{n\_0.5}$$

$$P_{r\_0.625} := \phi_c \cdot P_{n\_0.625}$$

$$P_{r\_0.5} = \begin{pmatrix} 1623 \\ 1869 \\ 2126 \\ 2396 \end{pmatrix} \text{kip}$$

$$P_{r\_0.625} = \begin{pmatrix} 1817 \\ 2085 \\ 2365 \\ 2657 \end{pmatrix} \text{kip}$$

Factored Structural Resistance (Pr) for the lower portion of open-ended pile piles or breached close-ended pipe piles is a function of only the steel shell

$$P_{n\_0.5tip} = \begin{pmatrix} 988 \\ 1095 \\ 1202 \\ 1308 \end{pmatrix} kip \qquad P_{n\_0.625tip} = \begin{pmatrix} 1298 \\ 1441 \\ 1583 \\ 1725 \end{pmatrix} kip$$

$$P_{r\_0.5tip} := \phi_c \cdot P_{n\_0.5tip}$$

$$P_{r\_0.625tip} := \phi_c \cdot P_{n\_0.625tip}$$

$$P_{r\_0.5tip} = \begin{pmatrix} 692 \\ 767 \\ 841 \\ 916 \end{pmatrix} kip \qquad P_{r\_0.625tip} = \begin{pmatrix} 909 \\ 1009 \\ 1108 \\ 1207 \end{pmatrix} kip$$

Use these for Factored Structural Resistances at Piers 4 and 5

Factored Axial Geotechnical Tip Resistance of single pipe pile

Resistance factor, end bearing on rock CGS method $\phi_{qp} := 0.45$

Factored Resistances

Factored Geotechnical Tip Resistance (Rr) assuming no soil plug

$$R_{r\_0.5} := \phi_{qp} \cdot R_{p\_0.5\_kulh}$$

$$R_{r\_0.625} := \phi_{qp} \cdot R_{p\_0.625\_kulh}$$

Factored tip resistance, Rr_p, assuming no soil plug

$$R_{r\_0.5} = \begin{pmatrix} 221 \\ 240 \\ 259 \\ 278 \end{pmatrix} kip$$

$$R_{r\_0.625} = \begin{pmatrix} 275 \\ 299 \\ 322 \\ 346 \end{pmatrix} kip$$

Factored Geotechnical Tip Resistance (Rr) assuming soil plug develops over 0% of the inside area and the shell and plug endbearing on rock

$R_{r\_plug\_0.5} := \phi_{qp} \cdot R_{p\_plugged\_0.5}$

$R_{r\_plug\_0.625} := \phi_{qp} \cdot R_{p\_plugged\_0.625}$

$$R_{r\_plug\_0.5} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} kip$$

$$R_{r\_plug\_0.625} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} kip$$

Nominal Geotechnical Side Frictional Resistance

The piles will be primarily end bearing. At piers 1, 3, 4 and 5 no skin friction in the overlying glaciomarine and marine clayey silt.

The nominal skin friction is computed using FHWA program Driven 1.0

Driven software uses Norlund/Thurman Method for side frictional resistance in cohesionless soils. Use a ϕ of 0.45 per LRFD Table 10.5.5.2.3-1

$\phi_{side} := 0.45$

Nominal side resistance (from Driven)

$$R_{s\_0.5} := \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kip \qquad R_{s\_0.625} := \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kip$$

Factored Geotechnical Side Frictional Resistance

$R_{r\_s0.5} := R_{s\_0.5} \cdot \phi_{side}$ $\qquad$ $R_{r\_s0.625} := R_{s\_0.625} \cdot \phi_{side}$

$$R_{r\_s0.5} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} kip$$

$$R_{r\_s0.625} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} kip$$

Total Factored Geotechnical Bearing Resistance of Piles - End bearing of shell + 0% of the plugged area on rock + side frictional resistance

$$R_{r\_0.5total} := R_{r\_0.5} + R_{r\_s0.5}$$

$$R_{r\_0.5total} = \begin{pmatrix} 221 \\ 240 \\ 259 \\ 278 \end{pmatrix} kip$$

USE THIS FOR PROJECT

$$R_{r\_0.625total} := R_{r\_0.625} + R_{r\_s0.625}$$

$$R_{r\_0.625total} = \begin{pmatrix} 275 \\ 299 \\ 322 \\ 346 \end{pmatrix} kip$$

USE THIS FOR PROJECT

Service Limit State" Factored " Axial Geotechnical Tip Resistance of single pipe pile

Resistance factor for service limit state $\phi_c := 1.0$

Nominal side resistance (from Driven) and tip resistance

$$R_{s\_0.5} := \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kip$$

$$R_{s\_s.625} := \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kip$$

$$R_{p\_plugged\_0.5} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} kip$$

$$R_{p\_plugged\_0.625} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} kip$$

Service Limit State Geotechnical Resistance

$$R_{service\_0.5} := R_{s\_0.5} + R_{p\_0.5\_kulh} \qquad R_{service\_0.5} = \begin{pmatrix} 492 \\ 533 \\ 575 \\ 617 \end{pmatrix} kip$$

$$R_{service\_0.625} := R_{s\_0.625} + R_{p\_0.625\_kulh} \qquad R_{service\_0.625} = \begin{pmatrix} 611 \\ 663 \\ 716 \\ 768 \end{pmatrix} kip$$

Compute Ultimate Resistance that must be achieved during the driveability wave equation analysis

The ultimate resistance that must be achieved during wave equation analysis will be the APPLIED MAXIMUM AXIAL PILE LOAD {must be < FACTORED RESISTANCE} divided by the appropriate resistance factor for wave equation analysis & dynamic test.

Resistance factor per Table 10.5.5.2.3-1 for dynamic testing is ϕ=0.65, per Table 3 which requires no less than 3 to 4 piles are to be tested considering low to medium site variablity.

$$\phi_{dyn} := 0.65$$

For the 1/2-in wall piles Rr_0.5total governs the factored resistance

$$Q_{p\_0.5} := \frac{R_{r\_0.5total}}{\phi_{dyn}} \qquad Q_{p\_0.5} = \begin{pmatrix} 340 \\ 369 \\ 398 \\ 427 \end{pmatrix} kip$$

For the 5/8-in wall piles,
the factored geotech resistance
governs (Rr_0.625total)

$$Q_{p\_0.625} := \frac{R_{r\_0.625total}}{\phi_{dyn}}$$

$$Q_{p\_0.625} = \begin{pmatrix} 423 \\ 459 \\ 495 \\ 532 \end{pmatrix} kip$$

The resistance required by the driveablity wave equation analysis is:

$$Q_{p\_0.5} = \begin{pmatrix} 340 \\ 369 \\ 398 \\ 427 \end{pmatrix} kip$$

$$Q_{p\_0.625} = \begin{pmatrix} 423 \\ 459 \\ 495 \\ 532 \end{pmatrix} kip$$