MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

WHITE'S BRIDGE
OVER SEBAGO LAKE BASIN
STANDISH-WINDHAM, MAINE

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GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to make geotechnical recommendations for the replacement of White's Bridge over Sebago Lake Basin between the towns of Standish and Windham, Maine. The proposed replacement structure will be a three-span structure on H-pile supported integral abutments and pipe pile pier bents. The following design recommendations are discussed in detail in the attached report:

Integral Abutment H-piles - The use of stub abutments founded on a single row of driven integral H-piles is a viable foundation system for use at the site. The piles should be end bearing, driven to the required resistance on or within the bedrock. Piles may be HP 12x53, HP 14x73, HP 14x89, or HP 14x117 depending on the design axial loads. Piles should be 50 ksi, Grade A572 steel H-piles. Piles should be fitted with driving points to protect the tips, improve penetration. The designer shall design the H-piles at the strength limit state considering the structural resistance of the piles, the geotechnical resistance of the pile and loss of the lateral support due to scour at the design flood event. The structural resistance check should include checking axial, lateral, and flexural resistance. The design of the H-piles at the service limit state shall consider tolerable horizontal movement of the piles, overall stability of the pile group and scour at the design flow event. Since the abutment piles will be subjected to lateral loading, piles should be analyzed for axial loading and combined axial and flexure. The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at each abutment. The first pile driven at each abutment should be dynamically tested to confirm capacity and verify the stopping criteria developed by the Contractor in the wave equation analysis. The ultimate pile 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 factored pile load should be shown on the plans.

Bearing Resistance - Spread footing supported abutments and wingwalls, if used, will be founded on native soils at the site. These elements will need to be designed to provide stability against bearing capacity failure. Bearing resistance for any structure founded on the native soils shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 14 ksf for footings with widths between 10 and 15 feet. Footings with a width of 9 feet or less should be assessed for a factored bearing resistance of 12 ksf. A factored bearing resistance of 6 ksf may be used when analyzing the service limit state and for preliminary sizing of footings. Footings shall be designed so that the nominal bearing resistance after the design scour event provides adequate resistance to support the unfactored strength limit state loads with a resistance factor of 1.0.

Abutments and Wingwalls – Abutments and wingwalls shall be designed for all relevant strength, service and extreme limit states and load combinations specified in LRFD Articles 3.4.1 and 11.5.5. The design of abutments and wingwalls at the strength limit state shall consider nominal bearing resistance, overturning (eccentricity), lateral sliding and structural failure. Extreme limit state design shall also consider foundation resistance after scour due to the design flood. For abutments that are pile supported, design for resistance against sliding and overturning is not required. In designing integral abutments for passive earth pressure,

the Rankine earth pressure coefficient (K_p) of 3.25 is recommended. In designing cantilever abutments for active earth pressure, the Rankine earth pressure coefficient (K_a) of 0.31 is recommended. All abutment designs shall include a drainage system to intercept any water. To avoid water intrusion behind the abutment, the approach slab should connect directly to the abutment.

Pile Bent Piers – Pile bent piers were selected for intermediate structure support. Piles for the pier bents may consist of concrete filled pipe piles driven to bedrock. Pipe piles with diameters ranging from 24 to 30 inches and wall thicknesses of ½ to 5/8 inch are recommended. Pipe piles should be fabricated in accordance with ASTM A252, Grade 3, with a minimum yield strength of 45 ksi. Open ended piles should be equipped with a cutting shoe, constructed from Grade ASTM A148 90/60 steel. Pipe pile pier bent piles should be driven to the required resistance on or within the bedrock. The designer shall design the piles at the strength limit state considering the structural and geotechnical resistance of the pile. The structural resistance check should include checking axial, lateral, and flexural resistance. The design of the piles at the service limit state shall consider tolerable horizontal movement of the piles and overall stability of the pile group. Since the pier piles will be subjected to lateral loading and have a substantial unbraced length, piles should be analyzed for axial loading and combined axial and lateral loading. The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at each pier. The first pile driven at each pier should be dynamically tested to confirm capacity and verify the stopping criteria developed by the Contractor in the wave equation analysis. The ultimate pile 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 factored pile load should be shown on the plans.

Scour and Riprap - The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered at the strength, service and extreme limit states. These changes in foundation conditions shall be investigated at the abutments, wingwalls and piers. For scour protection, any footings which are constructed on granular deposits, should be embedded a minimum of 3 feet below the design scour depth and armored with 3 feet of riprap.

Settlement - Post-construction settlements are anticipated to be less than 0.5 inches and will occur during construction having negligible effect of the finished structure. Any settlement of the bridge abutments will be due to the elastic compression of the piling and will also be negligible.

Frost Protection - Any foundations placed on granular soils should be founded a minimum of 6.5 feet below finished exterior grade for frost protection. Integral abutments shall be embedded a minimum of 4.0 feet for frost protection.

Seismic Design Considerations - White's Bridge is not the National Highway System and is therefore not considered to be functionally important and since the bridge construction costs should not exceed $10 million the bridge is not classified as a major structure. The site is assigned to Seismic Zone 1. A detailed seismic analysis is not required for multi-span

bridges in Seismic Zone 1. However, superstructure connections and minimum support length requirements shall be satisfied.

Construction Considerations – The proposed pile bent piers will be placed to coincide with the existing pier locations. The existing H-pile pier bents will need to be removed in their entirety in order to successfully install the pipe piles for the proposed pier bents. Boulders and cobbles were encountered within the interbedded sand and gravel layers in all of the borings. There is potential for these obstructions to impact the pile installation operations. These impacts include, but are not limited to, driving the piles and cleaning out pipe piles. Obstruction may be cleared by conventional excavation methods, pre-augering, pre-drilling, or down-hole hammers.

1.0 INTRODUCTION

A subsurface investigation for the replacement of White's Bridge over Sebago Lake Basin between the towns of in Standish and Windham, Cumberland County, Maine has been completed. The purpose of the investigation was to explore subsurface conditions at the site in order to develop geotechnical recommendations for the bridge replacement. This report presents the soils information obtained at the site, geotechnical design recommendations, and foundation recommendations.

The existing bridge White's Bridge was constructed in 1949 and consists of a 160 foot long, three-span steel girder superstructure supported on two (2) driven H-pile pier bents and concrete abutments on driven H-piles. The 2006 Maine Department of Transportation (MaineDOT) maintenance inspection reports indicate that the bridge superstructure and substructures are in "fair" (rating of 5) condition while the deck is in "serious" condition (rating of 3). The inspection reports state that the H-pile in the pier bents show moderate section loss. The Bridge Sufficiency Rating is 51.

The proposed replacement structure will be a three-span structure on H-pile supported integral abutments and pipe pile pier bents. The contract will include options to construct both a steel beam superstructure and a per-cast box beam superstructure. The proposed bridge alignment will match into the existing with some minor changes to ensure that the alignment meets standards. The bridge will be widened in order to add 4 foot shoulders to the bridge. The bridge will be closed to traffic during the replacement.

2.0 GEOLOGIC SETTING

White's Bridge on the Standish-Windham town line is over Sebago Lake Basin approximately 0.9 miles west of Route 302 as shown on Sheet 1 - Location Map found at the end of this report. Sebago Lake Basin flows in a southerly direction into the Presumpscot River.

According to the Surficial Geologic Map of Maine published by the Maine Geological Survey (1985) the surficial soils in the vicinity of the site consist of glacial till soils. These soils consist of a heterogeneous mixture of sand, silt, clay, and stones and may include boulders. These soils are generally deposited in a blanket deposit that conforms to the underlying bedrock topography. These soils are deposited directly by glacial ice. The site is located in the vicinity of the inland marine limit of the late-glacial marine submergence as mapped by Thompson (1983).

According to the Surficial Bedrock Map of Maine, published by the Maine Geological Survey (1985), the bedrock at the site is identified as igneous carboniferous muscovite-biotite granite commonly known as the Sebago pluton. This bedrock is anticipated to be hard and sound.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions were explored by drilling four (4) test borings at the site. Test boring BB-SWSL-103 was drilled at the location of Abutment No. 1 (west). Test boring BB- SWSL -102 was drilled at the center of the crossing. Test borings BB- SWSL -101 and BB- SWSL -101A were drilled at the location of Abutment No. 1 (east).

The exploration locations are shown on Sheet 2 - Boring Location Plan found at the end of this report. An interpretive subsurface profile depicting the site stratigraphy is shown on Sheet 3 - Interpretive Subsurface Profile found at the end of this report. The borings were drilled between April 15 and 25, 2008 by the MaineDOT drill crew. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A - Boring Logs and on Sheets 4 and 5 - Boring Logs found end of this report.

The borings were drilled using driven cased wash boring and solid stem auger techniques. Soil samples were obtained where possible 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. MaineDOT drill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated 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.

In-situ vane shear tests were made where possible in soft soil deposits to measure the shear strength of the strata. The bedrock was cored in the borings using an NQ 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 and identified field and laboratory testing requirements. The geotechnical team member and a MaineDOT Certified Subsurface Inspector logged the subsurface conditions encountered. The borings were located in the field by use of a tape after completion of the drilling program.

4.0 LABORATORY TESTING

Laboratory testing for samples obtained in the borings consisted of thirty-six (36) standard grain size analyses and two (2) Atterberg Limits tests. The results of these laboratory tests are provided in Appendix B - Laboratory Data at the end of this report. Moisture content information and other soil test results are included on the Boring Logs in Appendix A and on Sheets 4 and 5 - Boring Logs found at the end of this report.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered at the test borings generally consisted of interbedded silts, sands, gravels, cobbles and boulders underlain by granite. An interpretive subsurface profile depicting the site stratigraphy is shown on Sheet 3 - Interpretive Subsurface Profile found at the end of this report. The following paragraphs discuss the subsurface conditions encountered in detail:

5.1 Fill

A layer of fill was encountered behind the abutments. The layers ranged from approximately 13.0 feet thick in boring BB-SWSL-101/101A to approximately 17.0 feet thick in boring BB-SWSL-103. The soil generally consisted of brown and light brown, damp to wet, fine to coarse sand with little to trace silt and some to trace gravel. A large cobble was encountered at a depth of 13.0 feet in boring BB-SWSL-103. Corrected SPT N-values in the fill ranged from 5 to 19 blows per foot (bpf) indicating that the soil is loose to medium dense in consistency. Water contents from five (5) samples obtained within the fill layer range from approximately 5% to 15%. Five (5) grain size analyses conducted on samples of the fill indicate that the soil is classified as an A-1-b by the AASHTO Classification System and a SW-SM, SM, or SW by the Unified Soil Classification System.

5.2 Silt

Layers of silt were encountered in two of the borings. In boring BB-SWSL-103, a silt layer was encountered directly underlying the fill. This silt layer was approximately 4.0 feet thick and generally consisted of brown, wet, silt with some fine to coarse sand, trace gravel and trace silt. One (1) corrected SPT N-value in this silt layer was 24 bpf indicating that the silt is very stiff in consistency. One (1) water content from a sample obtained within this silt layer was approximately 17%. One (1) grain size analysis conducted on a sample from this silt layer indicates that the soil is classified as an A-4 by the AASHTO Classification System and a ML by the Unified Soil Classification System.

In boring BB-SWSL-102, a silt layer was encountered within a deeper sand layer. This silt layer was approximately 6.5 feet thick and generally consisted of grey, wet, silt with some clay and some to trace fine to medium sand. Corrected SPT N-values in this silt layer ranged from weight of hammer (WOH) to 17 bpf indicating that the soil is very soft to very stiff in consistency. Vane shear testing conducted in the silt showed measured undrained shear strengths ranging from approximately 1317 to >1978 pounds per square foot (psf) while the remolded shear strength was approximately 268 psf. Based on the ratio of peak to remolded shear strengths from the vane shear tests, the silt was determined to have sensitivity of approximately 4.9 and is classified as sensitive. Water contents from two (2) samples obtained within this silt layer range from approximately 27% to 31%. Two (2) grain size analyses conducted on samples from this silt layer indicate that the soil is classified as an A-4 by the AASHTO Classification System and a ML by the Unified Soil Classification System.

The following table summarizes the results of Atterberg Limits testing on the silt samples:

Sample No.	Soil Type	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-SWSL-102 7D	Silt	30.8	24	21	3	3.27
BB-SWSL-102 8D	Silt	27.2	Non Plastic			

Interpretation of these results indicates the silt is generally on the verge of becoming a viscous liquid if disturbed as the natural water content of the sample exceeds the liquid limit. This indicates that the silt has a high liquefaction potential. It can be inferred that overburden pressure and inter-particle cementation are providing stability for these soils. Under these conditions the slightest disturbance causing remolding has the potential to convert this type of deposit into a viscous liquid. Liquidity index values greater than or equal to 1 are indicative of soils that are unconsolidated and have a high liquefaction potentially commonly referred to as "quick".

5.3 Upper Sand Layer

An upper layer of sand was encountered in all of the borings. The layer ranged from approximately 12.0 feet thick in boring BB-SWSL-103 to approximately 27.0 feet thick boring BB-SWSL-101/101A. The upper sand generally consisted of brown and light brown, moist to wet, fine to coarse sand with little to trace silt and some to little gravel and gravelly fine to coarse sand with trace silt. A layer of cobbles and boulders was encountered at the bottom of this layer in boring BB-SWSL-101/101A. Corrected SPT N-values in the upper sand layer ranged from 6 to 45 bpf indicating that the soil is loose to very dense in consistency. Water contents from four (4) samples obtained within the upper sand layer range from approximately 10% to 19%. Four (4) grain size analyses conducted on samples from the upper sand layer indicate that the soil is classified as an A-1-b, A-1-a or A-3 by the AASHTO Classification System and a SW, SW-SM or SP by the Unified Soil Classification System.

5.4 Lower Sand Layer

A lower layer of sand was encountered in all of the borings. The layer ranged from approximately 49.3 feet thick in boring BB-SWSL-101/101A to approximately 53.4 feet thick boring BB-SWSL-103. The lower sand generally consisted of grey, wet, fine to coarse sand with some to trace silt and some to trace gravel. A layer of silt (described above) was encountered within the lower sand layer in boring BB-SWSL-102. Corrected SPT N-values in the lower sand layer ranged from 8 to >50 bpf indicating that the soil is loose to very dense in consistency. Water contents from twenty-two (22) samples obtained within the lower sand layer range from approximately 11% to 31%. Twenty-two (22) grain size analyses conducted on samples from the lower sand layer indicate that the soil is classified as an A-1-b, A-3 or A-2-4 by the AASHTO Classification System and a SW, SM, SP, SP-SM or SW-SM by the Unified Soil Classification System.

5.5 Bedrock

Bedrock was encountered and cored in three of the borings. The following table summarizes the depths to bedrock and corresponding elevations of the top of bedrock:

Boring Number/ Location	Depth to Bedrock	Bedrock Elevation	RQD
BB- SWSL -103/ Abutment No. 1	86.4 feet	194.9 feet	17 – 52%
BB- SWSL -102/ Channel Center	67.4 feet	183.6 feet	63 – 100%
BB- SWSL -101A/ Abutment No. 2	89.3 feet	188.9 feet	80%

The bedrock is identified as black, grey and white plutonic GRANITE with mica. The rock quality designation (RQD) of the bedrock was determined to range from 17 to 100 percent indicating a rock mass quality of very poor to excellent quality.

6.0 FOUNDATION ALTERNATIVES

Based on the subsurface conditions encountered during the subsurface exploration program, the following foundation alternatives, with varying levels of risk and durability, may be considered for the bridge replacement:

- Cast-in-place concrete or precast concrete integral abutments supported on driven steel H-piles
- Cast-in-place concrete or precast concrete abutments supported on spread footings
- Pile bent piers

The Preliminary Design Report (PDR) for this project recommends that the replacement bridge be supported on H-pile supported integral abutments and two (2) pile bent piers. This report addresses those foundation types as well as the spread footing supported abutment option for consideration.

7.0 FOUNDATION CONSIDERATIONS AND RECOMMENDATIONS

The following sections will discuss geotechnical design recommendations for cast-in-place concrete or precast concrete integral abutments supported on driven steel H-piles, pile bent piers and spread footing supported abutments.

7.1 Integral Abutment H-piles

The use of stub abutments founded on a single row of driven integral H-piles is a viable foundation system for use at the site. The piles should be end bearing, driven to the required resistance on or within the bedrock. Piles may be HP 12x53, HP 14x73, HP 14x89, or HP 14x117 depending on the design axial loads. Piles should be 50 ksi, Grade A572 steel H-piles. Piles should be fitted with driving points to protect the tips and improve penetration.

Pile lengths at the proposed abutments may be estimated based on the table below:

Location	Estimated Pile Cap Bottom Elevation	Depth to Bedrock From Ground Surface	Top of Rock Elevation	Estimated Pile Length
Abutment No.1 BB-SWSL-103	272.2 feet	86.4 feet	194.9 feet	78 feet
Abutment No.2 BB-SWSL-101/101A	269.8 feet	89.3 feet	188.9 feet	81 feet

These pile lengths do not take into account the additional five (5) feet of pile required for dynamic testing instrumentation or any additional pile length needed to accommodate the Contractor's leads and driving equipment.

The designer shall design the H-piles at the strength limit state considering the structural resistance of the piles, the geotechnical resistance of the pile and loss of the lateral support due to scour at the design flood event. The structural resistance check should include checking axial, lateral, and flexural resistance. Resistance factors for use in the design of piles at the strength limit state are discussed below.

The design of the H-piles at the service limit state shall consider tolerable horizontal movement of the piles, overall stability of the pile group and scour at the design flow event. Extreme limit state design shall check that the nominal pile resistance remaining after scour due to the design flood can support the unfactored strength limit state loads with a resistance factor of 1.0. The design flood scour is defined in AASHTO LRFD Bridge Design Specifications 4th Edition (LRFD) Articles 2.6.4.4.2 and 3.7.5. Since the abutment piles will be subjected to lateral loading, piles should be analyzed for axial loading and combined axial and flexure as defined in LRFD Article 6.15.2 and specified in LRFD Article 6.9.2.2.

7.1.1 Strength Limit State

The nominal structural compressive resistance (P_n) in the strength limit state for piles loaded in compression shall be as specified in LRFD Article 6.9.4.1. The H-piles are fully embedded and λ shall be taken as 0. It is the responsibility of the structural engineer to recalculate the column slenderness factor (λ) for the upper and lower portions of the H-pile based on unbraced lengths and K-values from project specific L-Pile® analyses and determine structural pile resistances. The factored structural axial compressive resistances of the four proposed H-pile sections were calculated using a resistance factor, ϕ_c, of 0.50 (severe driving conditions) and a λ of 0.

The nominal geotechnical compressive resistance in the strength limit state was calculated using Canadian Foundation Engineering Manual methods. The factored geotechnical compressive resistances of the four proposed H-pile sections were calculated using a resistance factor, ϕ_{stat}, of 0.45.

The drivability of the four proposed H-pile sections was considered. The maximum driving stresses in the pile, assuming the use of 50 ksi steel, shall be less than 45 ksi. As the piles will be driven to refusal on bedrock a drivability analysis to determine the resistance that must be achieved was conducted. The resistance factor for a single pile in axial compression when a dynamic test is done, given in LRFD Table 10.5.5.2.3-1, is ϕ_{dyn}= 0.65.

The calculated factored axial compressive structural, geotechnical and drivability resistances of the four proposed H-pile sections for the abutments are summarized in the table below. Supporting calculations are included in Appendix C- Calculations found at the end of this report.

Factored Axial Resistances for Abutment Piles at the Strength Limit State

Pile Section	Factored Resistance			
	Structural Resistance*	Geotechnical Resistance	Drivability Resistance	Design Resistance
HP 12 x 53	388 kips	354 kips	296 kips	354 kips
HP 14 x 73	535 kips	446 kips	339 kips	446 kips
HP 14 x 89	653 kips	542 kips	406 kips	542 kips
HP 14 x 117	860 kips	710 kips	483 kips	710 kips

* based on preliminary assumption of λ=0 for the lower portion of the pile in only axial compression (no flexure)

LRFD Article 10.7.8 states that for routine pile installation applications where significant local experience can be applied to keep the risk of pile installation problems low, a project specific drivability analysis using the wave equation may be waived. In light of this, it is recommended that the governing resistance used in design be the factored geotechnical resistance indicated in the table above.

Per LRFD Article 6.5.4.2, at the strength limit state, for H-piles in compression and bending, the axial resistance factor ϕ_c=0.7 and the flexural resistance factor ϕ_f =1.0 shall be applied to the combined axial and flexural resistance of the pile in the interaction equation (LFRD Eq. 6.12.2.2.1-1 or -2). The combined axial compression and flexure should be evaluated in accordance with the applicable sections of LRFD Articles 6.9.2.2 and 6.12.2.

7.1.2 Service and Extreme Limit States

For the service and extreme limit states resistance factors, ϕ, of 1.0 are recommended for structural and geotechnical pile resistances. For preliminary analysis, the H-piles can be assumed fully embedded and λ can be taken as 0. It is the responsibility of the structural engineer to recalculate the column slenderness factor (λ) for the upper and lower portions of the H-pile based on unbraced lengths and K-values from project specific L-Pile® analyses and determine structural pile resistances.

The calculated factored axial structural, geotechnical and drivability resistances of the four proposed H-pile sections for each abutment are summarized in the table below. Supporting calculations are included in Appendix C- Calculations found at the end of this report.

Factored Axial Resistances for Abutment Piles at the Service and Extreme Limit States

Pile Section	Factored Resistance			
	Structural Resistance*	Geotechnical Resistance	Drivability Resistance	Design Resistance
HP 12 x 53	775 kips	786 kips	456 kips	775 kips
HP 14 x 73	1070 kips	991 kips	522 kips	991 kips
HP 14 x 89	1305 kips	1204 kips	624 kips	1204 kips
HP 14 x 117	1720 kips	1578 kips	743 kips	1578 kips

*based on preliminary assumption of λ=0 for the lower portion of the pile in only axial compression (no flexure)

Although the factored axial drivability resistance is less than both the factored axial structural and geotechnical resistances, LRFD Article 10.7.8 states that for routine pile installation applications where significant local experience can be applied to keep the risk of pile installation problems low, a project specific drivability analysis using the wave equation may be waived. In light of this, it is recommended that the governing resistance used in design be the factored resistance shown in the last column of the table above. It should be noted that the design resistance for the HP 12x 53 pile is govern by the factored structural resistance while the remaining pile sections are governed by the factored geotechnical resistance.

7.1.3 Pile Resistance and Pile Quality Control

Based on the anticipated depth to bedrock at the site, pile splices will be required. The location and number of pile splices shall be in conformance with MaineDOT Standard Specification 501 and be subject to the approval of the Resident. The splices shall be the Champion HP-30000, or approved equivalent, mechanical splicer. Evaluation of equivalent products will be based on the submission of data demonstrating the capability of transferring the full pile strength in compression and tension and developing the bending moment capacity of the pile in both the x-x and y-y axes. The splicers shall be installed and welded as recommended by the manufacturer. Welding shall not be done when the temperature in the immediate vicinity of the weld is below 0°F; when the surfaces are damp or exposed to rain, snow, or high wind; or when the welders or welding operators are exposed to inclement conditions. The pile shall be preheated to and maintained at 150°F minimum within 15 cm (6 inches) from the weld during welding. Formal welding procedures are not required. Welders shall be prequalified in accordance with Section 504 - Structural Steel.

The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at each abutment. The first pile driven at each abutment should be dynamically tested to confirm capacity and verify the stopping criteria developed by the Contractor in the wave equation analysis. The ultimate pile 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 factored pile load should be shown on the plans.

Piles should be driven to an acceptable penetration resistance as determined by the Contractor based on the results of a wave equation analysis and as approved by the Resident.

Driving stresses in the pile determined in the drivability analysis shall be less than 45 ksi in accordance with LRFD Article 10.7.8. A hammer should be selected which provides the required resistance when the penetration resistance for the final 3 to 6 inches is 8 to 15 blows per 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.

7.2 Bearing Resistance

Spread footing supported abutments and wingwalls, if used, will be founded on native soils at the site. These elements will need to be designed to provide stability against bearing capacity failure. Applicable permanent and transient loads are specified in LRFD Articles 3.4.1 and 11.5.5.

Bearing resistance for any structure founded on the native soils shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 14 ksf for footings with widths between 10 and 15 feet. Footings with a width of 9 feet or less should be assessed for a factored bearing resistance of 12 ksf. The bearing resistance factor, ϕ_b, for spread footings on soil is 0.45 based on bearing resistance evaluation using semi-empirical methods. The applied stress distribution may be assumed to be a uniform distribution over the effective base as shown in LRFD Figure 11.6.3.2-1. The eccentricity of loading at the strength limit state evaluated based on factored loads shall not exceed one-fourth of the corresponding footing dimension, B or L, for footings on soil. A factored bearing resistance of 6 ksf may be used when analyzing the service limit state and for preliminary sizing of footings assuming a resistance factor of 1.0. See Appendix C - 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 state loads with a resistance factor of 1.0.

In no instance shall the factored bearing stress exceed the nominal resistance of the footing concrete, which is taken as $0.3f'c$. No footing shall be less than 2 feet wide regardless of the applied bearing pressure or bearing material. Any organic material encountered shall be removed to the full depth and replaced with compacted Granular Borrow, MaineDOT 703.19.

7.3 Abutments and Wingwalls

Abutments and wingwalls shall be designed for all relevant strength, service and extreme limit states and load combinations specified in LRFD Articles 3.4.1 and 11.5.5. The design of abutments and wingwalls at the strength limit state shall consider nominal bearing resistance, overturning (eccentricity), lateral sliding and structural failure.

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

The overall global stability of the foundation should be investigated at the Service I Load Combination and a resistance factor,ϕ, of 0.65. Extreme limit state design checks for abutments supported on piles shall include bearing resistance, pile structural resistance. Pile geotechnical resistance, pile resistance in combined axial and flexure, and overall stability. Extreme limit state design checks for abutments supported on spread footings shall include bearing resistance, eccentricity, sliding and overall stability. Resistance factors, ϕ, for the extreme limit state shall be taken as 1.0. Extreme limit state design shall also check that the nominal resistance remaining after scour due to the design flood can support the unfactored strength limit state loads with a resistance factor of 1.0. The unfactored strength limit state loads include any debris loads occurring during the flood event.

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 of 0.31, calculated using Rankine Theory for cantilever type abutments and wingwalls. Coulomb Theory should be used for gravity shaped structures. See Sheet 6 - 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 MaineDOT Bridge Design Guide (BDG) for the wingwalls if an approach slab is not specified. In the situation where a structural approach slab is specified, reduction of the surcharge loads is permitted per LRFD Article 3.11.6.2. Use of an approach slab may be required per the MaineDOT BDG Sections 5.4.2.10 and 5.4.4.

The live load surcharge on walls may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (H_{eq}) of 2.0 feet per LRFD Article 3.11.6.4-2. The live load surcharge on abutments may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (H_{eq}) taken form the table below:

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

The Designer may assume Soil Type 4 (MaineDOT BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf. Sliding computations for resistance to lateral loads shall assume a maximum allowable frictional coefficient of 0.45 at the soil-concrete interface. A sliding resistance factor of ϕ_τ=0.8 shall be applied to the nominal sliding resistance of walls founded on spread footings on sand.

Integral abutments and wingwall sections that are integral with the abutment should be designed to withstand a passive earth pressure state. In designing for passive earth pressure associated with integral abutments, the Coulomb state is recommended. Experience in designing wingwalls for integral abutments has shown that the use of the Coulomb passive earth pressure K_p=6.89 may result in uneconomical wall sections. For this reason, a Rankine

passive earth pressure, K_p=3.25, is recommended when designing integral abutments and integral wingwall extensions.

All abutment and wingwall designs shall include a drainage system behind the abutments to intercept any groundwater. Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage of the MaineDOT BDG. Geocomposite drainage board applied to the backsides of the abutments and wingwalls with weep holes will provide adequate drainage. To avoid water intrusion behind the abutment, the approach slab should connect 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.

7.4 Pile Bent Piers

Pile bent piers were selected for intermediate structure support. Piles for the pier bents may consist of concrete filled pipe piles driven to bedrock. Pipe piles with diameters ranging from 24 to 30 inches and wall thicknesses of ½ to 5/8 inch are recommended. Pipe piles should be fabricated in accordance with ASTM A252, Grade 3, with a minimum yield strength of 45 ksi. Pipe piles can be driven open-ended or closed-ended. Open ended piles should be equipped with a cutting shoe constructed from Grade ASTM A148 90/60 steel. Pipe pile pier bent piles should be end bearing and driven to the required resistance on or within the bedrock.

Pile lengths at the proposed pier may be roughly estimated based on the table below:

Location	Estimated Pile Cap Bottom Elevation	Depth to Bedrock From Ground Surface	Top of Rock Elevation	Estimated Pile Length
Channel Pier BB-SLSW-102	Pier No 1 – 273.6 feet Pier No. 2 – 272.3 feet	63.4 feet	183.6 feet	90 feet

This estimated pile length does not take into account the variability of the bedrock surface within the channel or the additional eight (8) feet of pile required for dynamic testing instrumentation or any additional pile length needed to accommodate the Contractor's leads and driving equipment.

The designer shall design the piles at the strength limit state considering the structural, geotechnical and drivability resistance of the pile. The structural resistance check should include checking axial, lateral, and flexural resistance. Resistance factors for use in the design of piles at the strength limit state are discussed below.

The design of the piles at the service limit state shall consider tolerable horizontal movement of the piles and overall stability of the pile group. Since the pier piles will be subjected to lateral loading and have a substantial unbraced length, piles should be analyzed for axial loading and combined axial and lateral loading as defined in LRFD Article 6.15.2.

7.4.1 Strength Limit State

The nominal compressive structural resistance (P_n) for piles in the strength limit state loaded in compression shall be as specified in LRFD Article 6.9.4.1 for non-composite members (H-pile) and Article 6.9.5.1 for composite members (pipe pile). The piles have an unbraced length and require calculation of the λ-factor as specified in LRFD Article 6.9.

For the strength limit state, the factored axial compressive structural resistance of the pile (P_r) shall be calculated using the resistance factors (ϕ_c) of 0.6 for pipe pile in severe driving conditions as specified in LRFD Article 6.5.4.2. The proposed pier bent piles will have an unbraced pile length ranging from 26 to 28 feet.

Per LRFD Article 6.5.4.2, at the strength limit state, for piles in compression and bending, the axial resistance factor ϕ_c=0.8 and the flexural resistance factor ϕ_f =1.0 shall be applied to the combined nominal axial and flexural resistance of the pile in the interaction equation, (LRFD Eq. 6.9.2.2-1 or -2) with flexural resistance determined as specified in LRFD 6.12. The factored structural resistance for 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.

The nominal geotechnical compressive resistance in the strength limit state was calculated using Canadian Foundation Engineering Manual methods. The factored geotechnical compressive resistances of the eight (8) proposed pipe pile sections were calculated using a resistance factor, ϕ_{stat}, of 0.45 for end bearing piles on bedrock.

The drivability of the eight (8) proposed pipe pile sections was considered. The maximum driving stresses in the pipe pile, assuming the use of 45 ksi steel, shall be less than 40 ksi. As the piles will be driven to refusal on bedrock a drivability analysis to determine the resistance that must be achieved was conduced. The resistance factor for a single pile in axial compression when a dynamic test is done given in LRFD Table 10.5.5.2.3-1 is ϕ_{dyn}= 0.65.

Factored axial compressive structural, geotechnical and drivability resistances for eight (8) pipe pile sections are summarized in the table below. Supporting calculations are included in Appendix C- Calculations found at the end of this report.

Factored Axial Resistances for Pipe Piles at the Strength Limit State

Pipe Pile		Factored Resistance			
Diameter	Wall Thickness	Structural Resistance	Geotechnical Resistance	Drivability Resistance	Governing Resistance
24 inches	½ inch	677 kips	474 kips	341 kips	474 kips
26 inches	½ inch	742 kips	501 kips	367 kips	501 kips
28 inches	½ inch	807 kips	528 kips	396 kips	528 kips
30 inches	½ inch	872 kips	555 kips	420 kips	555 kips
24 inches	5/8 inch	894 kips	628 kips	442 kips	628 kips
26 inches	5/8 inch	982 kips	665 kips	483 kips	665 kips
28 inches	5/8 inch	1069 kips	701 kips	525 kips	701 kips
30 inches	5/8 inch	1155 kips	737 kips	570 kips	737 kips

Although the factored axial drivability resistance is less than both the factored axial structural and geotechnical resistances for the first two pile sections analyzed, LRFD Article 10.7.8 states that for routine pile installation applications where significant local experience can be applied to keep the risk of pile installation problems low, a project specific drivability analysis using the wave equation may be waived. In light of this, it is recommended that the governing resistance in the lower portion of the pile used in design be the factored geotechnical resistance in the table above. The upper portion of the pile may be governed by a lesser axial pile load in order to satisfy the interaction equation (LRFD Article 6.9.2.2).

7.4.2 Service Limit and Extreme Limit State Designs

Per LRFD Article 10.5.5.1 the ability of the pier piles to meet defection criteria at the service limit state shall be investigated using a resistance factor of 1.0. Per LRFD Article 10.5.5.3.3 the ability of the pier piles at the extreme limit state shall be investigated using a resistance factor of 1.0. Extreme limit state design shall check that the nominal pile resistance remaining after scour due to the design flood can support the unfactored strength limit state loads with a resistance factor of 1.0.

The axial structural resistance of eight (8) proposed pipe pile sections and four (4) proposed H-pile sections was investigated using a resistance factor of 1.0. The piles have an unbraced length and require calculation of the λ factor as specified in LRFD Article 6.9. The axial geotechnical compressive resistance of eight (8) proposed pipe pile sections and four (4) proposed H-pile sections was calculated using Canadian Foundation Engineering Manual methods and a resistance factor of 1.0. The drivability of the eight (8) proposed pipe pile sections and four (4) proposed H-pile sections was considered. The maximum driving stresses in the pipe pile, assuming the use of 45 ksi steel, shall be less than 40 ksi. The maximum driving stresses in the H-pile, assuming the use of 50 ksi steel, shall be less than 45 ksi. The resistance factor for a single pile in axial compression for the service and extreme limit states of 1.0 was used.

The calculated factored axial structural, geotechnical and drivability resistances for the eight (8) pipe pile sections are summarized in the table below. Supporting calculations are included in Appendix C- Calculations found at the end of this report.

Factored Axial Resistances for Pipe Piles at the Service and Extreme Limit States

Pipe Pile		Factored Resistance			
Diameter	Wall Thickness	Structural Resistance	Geotechnical Resistance	Drivability Resistance	Governing Resistance
24 inches	½ inch	1128 kips	1053 kips	525 kips	1053 kips
26 inches	½ inch	1237 kips	1113 kips	565 kips	1113 kips
28 inches	½ inch	1345 kips	1173 kips	609 kips	1173 kips
30 inches	½ inch	1453 kips	1233 kips	646 kips	1233 kips
24 inches	5/8 inch	1491 kips	1396 kips	680 kips	1396 kips
26 inches	5/8 inch	1636 kips	1477 kips	743 kips	1477 kips
28 inches	5/8 inch	1781 kips	1557 kips	808 kips	1557 kips
30 inches	5/8 inch	1925 kips	1638 kips	877 kips	1638 kips

Although the factored axial drivability resistance is less than both the factored axial structural and geotechnical resistances for the first two pile sections analyzed, LRFD Article 10.7.8 states that for routine pile installation applications where significant local experience can be applied to keep the risk of pile installation problems low, a project specific drivability analysis using the wave equation may be waived. In light of this, it is recommended that the governing resistance in the lower portion of the pile used in design be the factored geotechnical resistance shown in the table above. The upper portion of the pile may be governed by a lesser axial pile load in order to satisfy the interaction equation (LRFD Article 6.9.2.2).

7.4.3 Estimated Depths to Pile Fixity

Stability of the piles shall be evaluated in accordance with the provisions in LRFD Articles 6.9, 6.12 and 6.15 using an equivalent pile length of the pile that accounts for the laterally supported 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 (8) proposed pipe pile sections were calculated, assuming only axial loading and without consideration of lateral loads, using the buckling methodology in LRFD Article 10.7.3.13.4. The table below summarizes the calculated depths to fixity for the eight (8) proposed pile sections and the estimated design scour depth. The design scour depth provided by the Structural Designer was estimated to be less than 13 feet. For the purposes of the geotechnical calculations the unbraced length of the pile was assumed to be the length of pile above the lake bed (approximately 15 feet) plus the depth to fixity calculated for each proposed pile section. Supporting calculations are included in Appendix C- Calculations found at the end of this report.

Preliminary Estimates of Depth to Fixity

Outside Pipe Pile Diameter	Wall thickness	Preliminary Estimates of Depth to Fixity w/ no lateral loads applied	Estimated Exposed Pile Length Due to Scour
24-in	1/2–in	11 feet	<13 feet
26-in	1/2–in	12 feet	<13 feet
28-in	1/2–in	13 feet	<13 feet
30-in	1/2–in	13 feet	<13 feet
24-in	5/8–in	11 feet	<13 feet
26-in	5/8–in	12 feet	<13 feet
28-in	5/8–in	13 feet	<13 feet
30-in	5/8–in	13 feet	<13 feet

Due to the depth of the overburden at the site, the pile sections will all achieve a fixed condition under normal conditions and the design scour event if they are driven to end bearing on bedrock.

When the lateral and axial pile 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.

7.4.4 Buckling and Combined Axial and Flexure

Pile group design shall consider loading effects due to combined axial and flexural loading, as outlined in LRFD Article 6.15. In designing piles for the bent group the effects of soil-structure interaction shall be considered in conformance with LRFD Article 10.7.3.9. 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 engineer.

7.4.5 Pile Resistance and Pile Quality Control

Based on the anticipated depth to bedrock at the site, pile splices will be required. The location and number of pile splices shall be in conformance with MaineDOT Standard Specification 501 and be subject to the approval of the Resident.

The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at each pier. The first pile driven at each pier should be dynamically tested to confirm capacity and verify the stopping criteria developed by the Contractor in the wave equation analysis. The ultimate pile 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 factored pile load should be shown on the plans per LRFD Article 3.6.5.2. Calculations for the pile resistance required by a drivability wave equation analysis are included the Appendix C- 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 pipe pile determined in the drivability analysis shall be less than 40 ksi 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 15 blows per 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.

7.5 Scour and Riprap

The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered at the strength, service and extreme limit states. These changes in foundation conditions shall be investigated at the abutments and wingwalls. For scour protection, any footings which are constructed on granular deposits, should be embedded a minimum of 3 feet below the design scour depth and armored with 3 feet of riprap. Refer to MaineDOT BDG Section 2.3.11 for information regarding scour design.

Riprap conforming to item number 703.26 of the Standard Specification shall be placed at the toes of abutments and wingwalls. Riprap shall be 3 feet thick. In front of the wingwalls, the bottom of the riprap section shall be constructed 6.5 feet above the bottom of the structures for frost protection. The riprap shall extend 1.5 feet horizontally in front of the wall before sloping at a 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 and Class "A" Erosion Control Geotextile per Standard Detail 610 (02-04).

7.6 Settlement

The grades of the existing bridge approaches will not be changed in the construction of the proposed bridge; therefore, post-construction settlements are anticipated to be less than 0.5 inches and will occur during construction having negligible effect of the finished structure. Any settlement of the bridge abutments will be due to the elastic compression of the piling and will also be negligible.

7.7 Frost Protection

Any foundation placed on granular subgrade soils should be designed with an appropriate embedment for frost protection. According to the MaineDOT frost depth maps for the State of Maine (MaineDOT BDG Figure 5-1) the site has a design-freezing index of approximately 1330 F-degree days. This correlates to a frost depth of 6.5 feet. Therefore, any foundations placed on granular soils should be founded a minimum of 6.5 feet below finished exterior

grade for frost protection. Integral abutments shall be embedded a minimum of 4.0 feet for frost protection per Figure 5-2 of the MaineDOT BDG. See Appendix C- Calculations at the end of this report for supporting documentation.

7.8 Seismic Design Considerations

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

- Peak Ground Acceleration coefficient (PGA) = 0.095g
- Short-term (0.2-second period) spectral acceleration coefficient = 0.186g
- Long-term (1.0-second period) spectral acceleration coefficient = 0.047g

Per LRFD Article 3.10.3.1 the site is assigned to Site Class C due to the presence of soils with N-values greater than 50 blows per foot at the site. Per LRFD Article 3.10.6 the site is assigned to Seismic Zone 1 based on a calculated S_{D1} of 0.079g (LRFD Eq. 3.10.4.2-6).

According to Figure 2-2 of the MaineDOT BDG, White's Bridge is not the National Highway System (NHS) and is therefore not considered to be functionally important and since the bridge construction costs should not exceed $10 million the bridge is not classified as a major structure. In conformance with LRFD Article 4.7.4.3, a detailed seismic analysis is not required for multi-span bridges in Seismic Zone 1. However, superstructure connections and minimum support length requirements shall be satisfied per LRFD Articles 3.10.9 and 4.7.4.4, respectively.

7.9 Construction Considerations

During the PDR phase of the project it was determined that the existing pier locations were the optimal pier locations and that the proposed pile bent piers would be placed to coincide with the existing pier locations. The existing H-pile pier bents will need to be removed in their entirety in order to successfully install the pipe piles for the proposed pier bents.

Boulders and cobbles were encountered within the interbedded sand and gravel layers in all of the borings. There is potential for these obstructions to impact the pile installation operations. These impacts include, but are not limited to, driving the piles and cleaning out pipe piles. Obstructions may be cleared by conventional excavation methods, pre-augering, pre-drilling, or down-hole hammers. Care should be taken to drive piles within allowable tolerances. Alternative methods to clear obstructions may be used as approved by the Resident.

7.10 Additional Geotechnical Work

It is generally the policy of the MaineDOT geotechnical team to drill one boring at each proposed substructure as required by AASHTO LRFD Article 10.4.2. During the Preliminary Design Report (PDR) phase of this project the proposed replacement structure

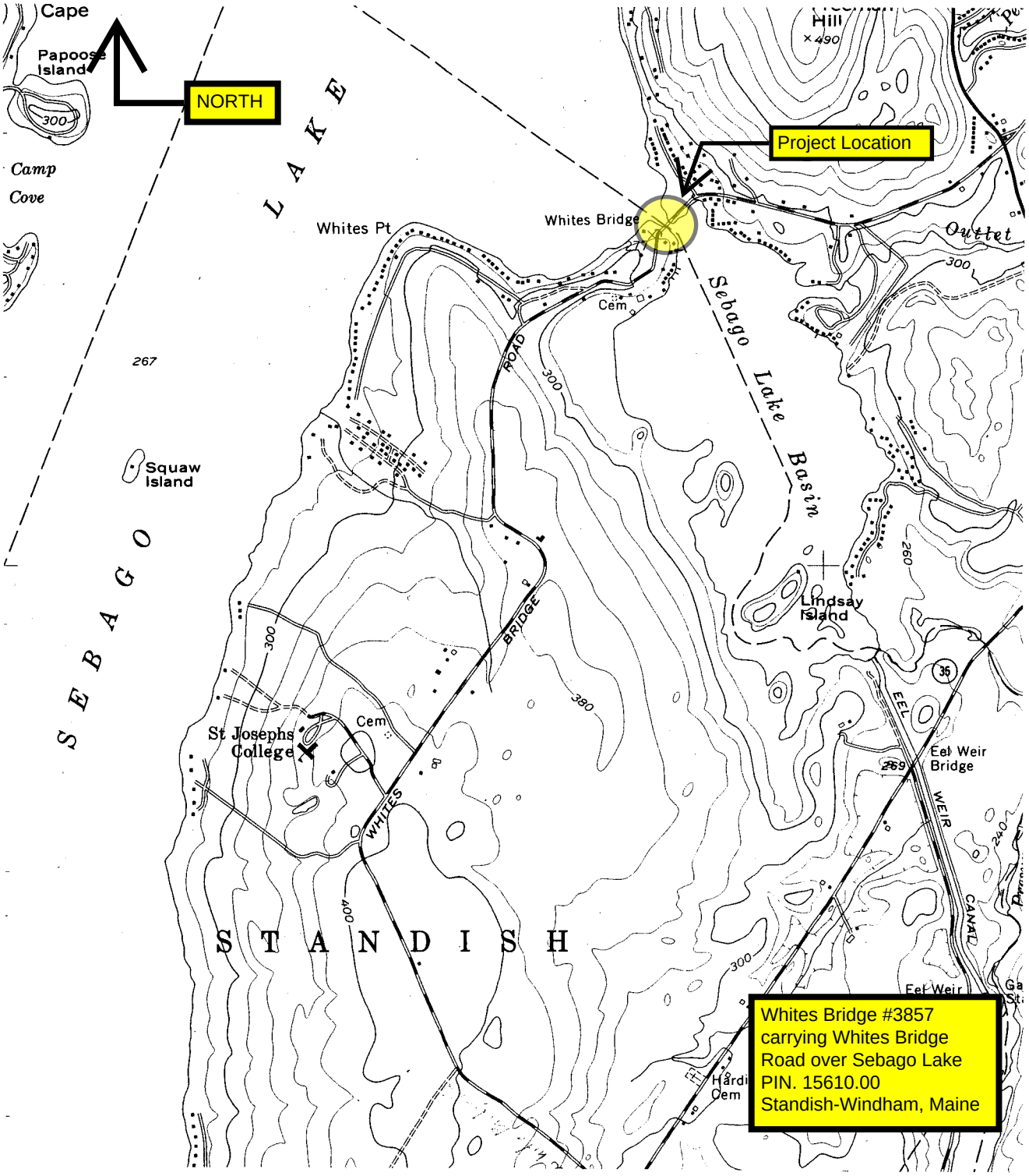
was changed from a two-span structure to a three-span structure. The three borings drilled at the site had already been completed before this decision was final. The geotechnical team recommends that additional borings be conducted at the location of the two proposed piers in order to assist the Contractor in accurately determining pile lengths for ordering pile during construction.

8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of White's Bridge on the Standish-Windham town line 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



NORTH WINDHAM QUADRANGLE
MAINE—CUMBERLAND CO.
7.5 MINUTE SERIES (TOPOGRAPHIC)
SW/4 GRAY 15′ QUADRANGLE

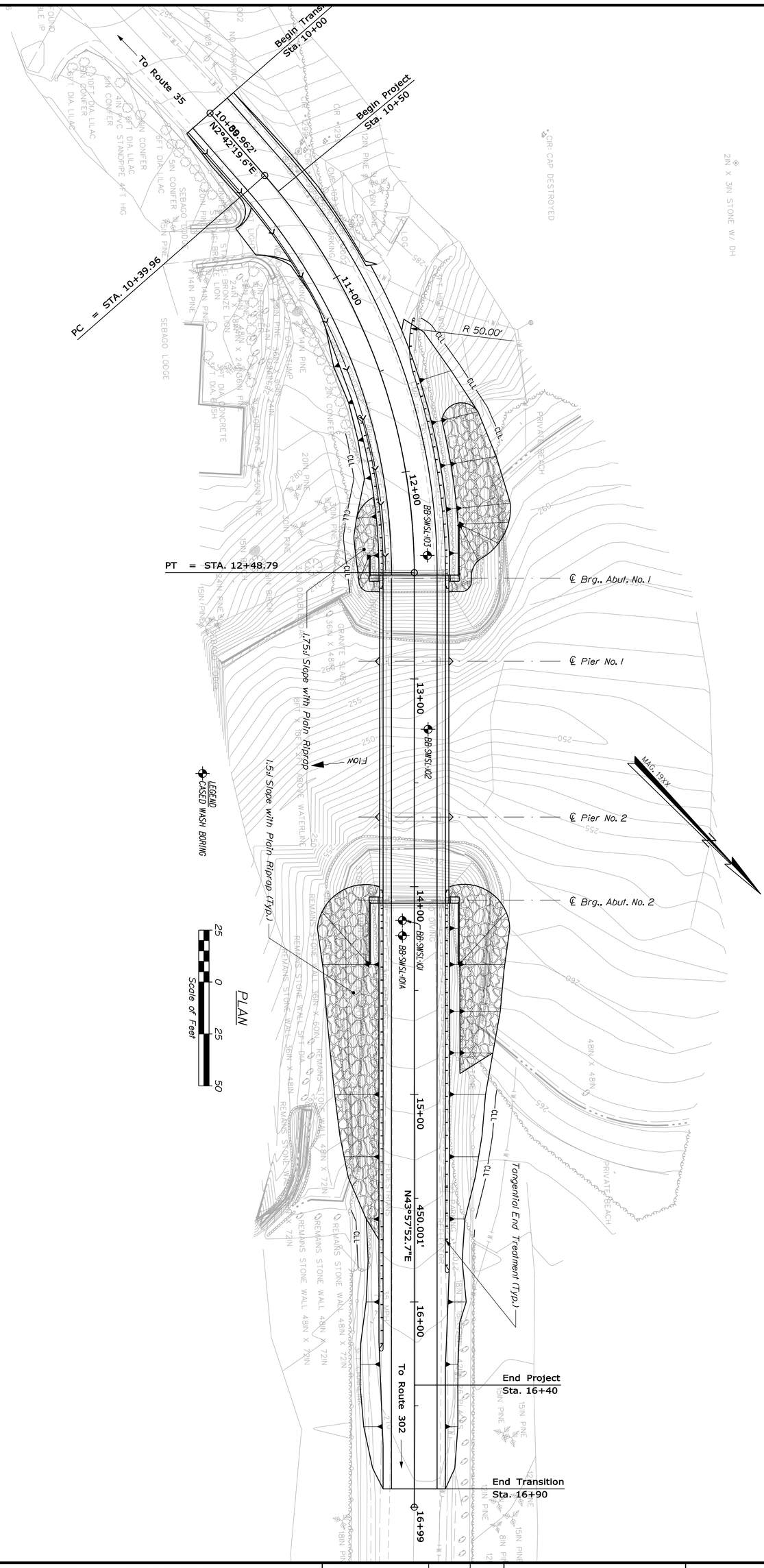
SCALE 1:24 000

1 ½ 0 1 MILE

1000 0 1000 2000 3000 4000 5000 6000 7000 FEET

1 .5 0 1 KILOMETER

CONTOUR INTERVAL 20 FEET
DATUM IS MEAN SEA LEVEL



WHITES BRIDGE WHITES BRIDGE RD. OVER SEBAGO LAKE STANDISH-WINDHAM CUMBERLAND CNTY	PROJ. MANAGER		BY	DATE
	DESIGN-DETAILED	K. MAGUIRE	T. WHITE	MAY 2008
	CHECKED-REVIEWED			
	DESIGN2-DETAILED2			
	DESIGN3-DETAILED3			
	REVISIONS 1			
	REVISIONS 2			
	REVISIONS 3			
	REVISIONS 4			
	FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

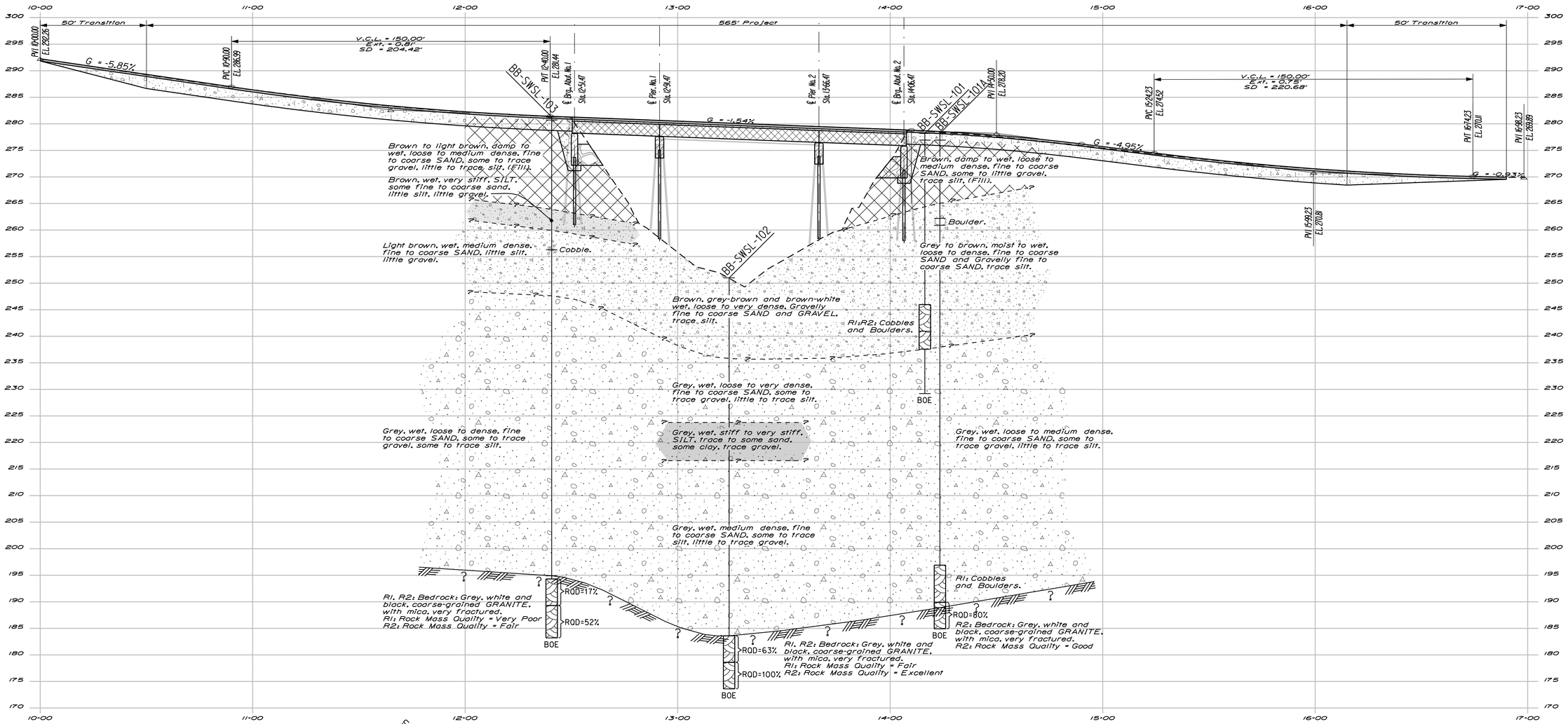
STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-A561(000)X

BRIDGE NO. 3857 PIN 15610.00 BRIDGE PLANS

BORING LOCATION PLAN

SHEET NUMBER 2 OF 6



Note: This generalized interpretive soil profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-A561(000)X

PIN 15610.00

BRIDGE NO. 3857

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	T. WHITE	MAY 2008
CHECKED-REVIEWED	K. MAGUIRE	
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

WHITES BRIDGE
WHITES BRIDGE RD. OVER SEBAGO LAKE
STANDISH-WINDHAM CUMBERLAND CNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

3

OF 6

Filename: ...\00\geotech\msta\007_ISP1.dgn Division: GEOTECH Username: terry.white Date:12/15/2008

Filename: ...\msta\008_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:12/15/2008

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

Project: White's Bridge #3857 carries White's Bridge Rd over Sebago
Location: Standish-Windham, Maine

Boring No.: BB-SWSL-103
PIN: 15610.00

Driller:	MaineDOT	Elevation (ft.) 281.3	Auger ID/OD: N/A
Operator:	E. Giguere/C. Giles	Datum: NAVD 88	Sampler: Standard Split Spoon
Logged By:	K. Maguire/B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish:	4/15-17/08	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2*
Boring Location:	12+40.7, 6.4 Lt.	Casing ID/OD: NW & HW	Water Level*: None Observed
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, FP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Rotary Cone
MDH = weight of 140lb. hammer
WOR/WOH = weight of rods or 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. Soil; Strength (tsf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing/ Results/ AASHTO and Unified Class
0							SSA	280.30	Pavement — 0.40	
	10	24/16	2.00 - 4.00	6/7/8/7	15	19			Brown, damp, medium dense, fine to coarse SAND, some gravel, trace silt. (Fill).	G#207822 A-1-b, SW-SM WC=5.7%
5	20	24/16	5.00 - 7.00	3/2/2/2	4	5	6		Light brown, moist, loose, fine to coarse SAND, little silt, little gravel. (Fill).	G#207823 A-1-b, SM WC=9.9%
10	30	24/6	9.00 - 11.00	3/4/3/4	7	9	3		Light brown, wet, loose, fine to coarse SAND, trace gravel, trace silt. (Fill).	G#207824 A-1-b, SM WC=14.4%
							243		Large Cobble from 13.0-13.8' bgs.	
15	40	24/18	14.00 - 16.00	2/6/6/4	12	15	63		Light brown, wet, medium dense, fine to coarse SAND, trace gravel, trace silt. (Fill). Bent NW Casing, switched to HW Casing.	G#207825 A-1-b, SW-SM WC=10.1%
							80	264.30	17.00	
20	50	18/14	19.00 - 20.50	8/11/8	19	24	60		Brown, wet, very stiff, SILT, some fine to coarse SAND, little clay, little gravel.	G#209901 A-4, ML WC=16.5%
							85	260.30	21.00	
25	MD	2.4/0	24.20 - 24.40	50(2.4")	---		11		Failed sample attempt, large Cobble. Roller Coned ahead from 24.2-25.0' bgs. Switched back to NW Casing.	
							58			
30	60	24/6	29.00 - 31.00	16/13/9/9	22	28	24		Light brown, wet, medium dense, fine to coarse SAND, little silt, little gravel.	
							34	248.30	33.00	
							49			
35	TO	24/15	34.00 - 36.00	5/10/11/13	21	27	19		Grey, wet, medium dense, fine to coarse SAND, some gravel, trace silt.	G#209902 A-1-b, SW WC=13.9%
							74			
							59			
							34			
40	80	24/12	39.00 - 41.00	11/6/6/14	12	15	25		Grey, wet, medium dense, fine to coarse SAND, trace silt, trace gravel.	G#209903 A-1-b, SP-SM WC=19.5%
							55			
							40			
							37			
							41			
45	90	24/13	44.00 - 46.00	12/7/4/5	11	14	17		Similar to above.	G#209904 A-1-b, SP-SM WC=16.3%
							48			
							44			
							44			
							62			
50	100	24/2	49.00 - 51.00	14/13/13/11	26	33	19		Grey, wet, dense, fine to medium SAND, trace silt.	G#209905 A-3, SP-SM

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 2 — Boring No.: BB-SWSL-103

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

Project: White's Bridge #3857 carries White's Bridge Rd over Sebago
Location: Standish-Windham, Maine

Boring No.: BB-SWSL-103
PIN: 15610.00

Driller:	MaineDOT	Elevation (ft.) 281.3	Auger ID/OD: N/A
Operator:	E. Giguere/C. Giles	Datum: NAVD 88	Sampler: Standard Split Spoon
Logged By:	K. Maguire/B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish:	4/15-17/08	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2*
Boring Location:	12+40.7, 6.4 Lt.	Casing ID/OD: NW & HW	Water Level*: None Observed
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, FP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Rotary Cone
MDH = weight of 140lb. hammer
WOR/WOH = weight of rods or 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. Soil; Strength (tsf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing/ Results/ AASHTO and Unified Class
50							41			WC=25.0%
							71			
							72			
							68			
55	110	24/16	54.00 - 56.00	9/5/5/6	10	13	26		Grey, wet, medium dense, fine to coarse SAND, trace silt, trace gravel.	G#209906 A-1-b, SP-SM WC=17.2%
							45			
							73			
							81			
							78			
60	120	24/14	59.00 - 61.00	6/4/5/6	9	12	44		Similar to above.	
							61			
							79			
							89			
							97			
65	130	24/15	64.00 - 66.00	9/5/5/5	10	13	54		Similar to above.	G#209907 A-1-b, SP-SM WC=18.8%
							57			
							79			
							83			
							102			
70	140	24/12	69.00 - 71.00	5/5/3/4	8	10	63		Grey, wet, loose, fine to coarse SAND, little gravel, trace silt.	G#209908 A-1-b, SW-SM WC=15.5%
							64			
							73			
							81			
							84			
75	150	24/6	74.00 - 76.00	6/3/3/4	6	8	59		Grey, saturated, loose, fine SAND, trace silt.	G#209909 A-3, SP-SM WC=30.9%
							71			
							94			
							129			
							212			
80	160	24/16	79.00 - 81.00	16/18/26/20	44	56	94		Grey, wet, very dense, fine to coarse SAND, some gravel, little silt.	G#209910 A-2-4, SM WC=11.0%
							138			
							145			
							144			
							201			
							200			
85	170	16.8/13	85.00 - 86.40	28/25/50(4.8")	---		103		Grey, wet, very dense, fine to coarse SAND, some silt, little gravel.	G#209911 A-1-b, SM WC=11.3%
							050	194.90	050 blows for 0.4'. — 86.40	
							ND CORE		Top of Bedrock at Elev. 194.9'. Roller Coned ahead to 87.0' bgs. R1:Bedrock: Grey, white and black, coarse grained GRANITE with mica. very fractured. Rock Mass Quality = Very Poor. R1:Core Times (min:sec) 87.0-88.0' (2:43) 300# down pressure 88.0-89.0' (2:14) 89.0-90.0' (1:56) 90.0-91.0' (1:19) 91.0-92.0' (2:58) 80% Recovery Lost water return at 90.5' bgs. R2: Similar to R1. Rock Mass Quality = Fair. Core Times (min:sec) 92.0-93.0' (1:47) 300# down pressure 93.0-94.0' (2:22) 94.0-95.0' (2:20) 95.0-96.0' (1:50) 95.5-96.1' core barrel dropped, possible void. 96.0-97.0' (0:45) 97.0-98.0' (1:21) 98.0-98.1' (0:30) 80% Recovery No water return	
90	R1	60/48	87.00 - 92.00	ROD = 17%						
								183.20	98.10	
	R2	73.2/58.8	92.00 - 98.10	ROD = 52%						
95										
100										

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 2 of 2 — Boring No.: BB-SWSL-103

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

Project: White's Bridge #3857 carries White's Bridge Rd over Sebago
Location: Standish-Windham, Maine

Boring No.: BB-SWSL-102
PIN: 15610.00

Driller:	MaineDOT	Elevation (ft.) 251.0	Auger ID/OD: N/A
Operator:	E. Giguere/C. Giles	Datum: NAVD 88	Sampler: Standard Split Spoon
Logged By:	K. Maguire/B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish:	4/17/08, 4/23/08	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2*
Boring Location:	13+24.2, 6.6 Lt.	Casing ID/OD: HW & NW	Water Level*: None Observed
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, FP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Rotary Cone
MDH = weight of 140lb. hammer
WOR/WOH = weight of rods or 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. Soil; Strength (tsf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing/ Results/ AASHTO and Unified Class
0	10	24/5	0.00 - 2.00	9/5/27/32	32	41	9		Brown, wet, very dense, Gravelly fine to coarse SAND, trace silt.	G#207810 A-1-b, SM WC=13.3%
							16			
	20	24/8	2.80 - 4.80	5/6/6/6	12	15	54		Grey-brown, wet, medium dense, Gravelly fine to coarse SAND, trace silt, with broken rock fragments.	
							50			
5							48			
							58			
							38			
							25			
	30	24/6	8.00 - 10.00	3/3/2/2	5	6	9		Brown-white, black-green, GRAVEL, wet, loose, no sand.	
10							26			
							27			
							17			
							54			
	40	24/13	13.00 - 15.00	5/5/3/4	8	10	30		Similar to Gravel above from 13.0-14.5' bgs.	G#207811 A-3, SP WC=19.4%
							41	236.50	14.50	
15							40		Grey, wet, loose, fine to coarse SAND, trace gravel, trace silt.	
							41			
							40			
	50	24/6	18.00 - 20.00	2/2/3/4	5	6	15		Grey, wet, loose, fine to coarse SAND, little silt, trace gravel. Telescoped NW Casing at 18.0' bgs.	G#207812 A-2-4, SM WC=21.6%
20							21			
							33			
							54			
							34			
	60	24/12	23.50 - 25.50	22/32/15/12	47	60	73		Grey, wet, very dense, fine to coarse SAND, some gravel, little silt.	G#207813 A-1-b, SW-SM WC=12.3%
							53			
25							30			
							38	224.00	27.00	
	TO	24/17	27.00 - 29.00	3/6/7/9	13	17	25		Grey, wet, very stiff, SILT, some clay, trace fine sand.	G#207814 A-4, ML WC=30.8% LL=24 PL=21 PI=3
							25			
							29			
30	VI MV		30.13 - 30.50 30.83 30.70	Su=1317/268 psf Would not push			31		55x110 mm vane vor torque readings: V1: 29.5/6.0 ft-lbs Failed 55x110 mm vane attempt.	
							33			
	80	24/18	32.50 - 34.50	WOR/WOH/WOH/1	---		38		Grey, wet, stiff, non-plastic, SILT, some fine to medium sand, some clay, trace gravel.	G#207815 A-4, ML WC=27.2% Non-Plastic
	V2		33.13 - 33.50	Su= @1978 psf			37	217.50	Over vane reading capacity (96.0 ft-lbs) — 33.50	
35							32		Grey, wet, very loose, silty fine SAND, trace gravel.	
							37			
							41			
	90	24/16	37.00 - 39.00	3/4/5/7	9	12	34		Grey, wet, medium dense, fine to coarse SAND, little silt, trace gravel.	G#207816 A-2-4, SM WC=16.6%
							31			
40							59			
							79			
							110			
	100	24/15	42.00 - 44.00	9/11/10/11	21	27	54		Grey, wet, medium dense, fine to coarse SAND, some silt, little gravel.	G#207817 A-2-4, SM WC=14.9%
							51			
45							59			
							81			
							78			
	110	24/16	47.00 - 49.00	8/7/8/10	15	19	51		Grey, wet, medium dense, fine to coarse SAND, some silt, trace gravel.	G#207818 A-1-b, SM WC=17.6%
							42			
50							59			

Remarks:
Cored thru concrete Bridge Deck, concrete thickness 0.8'.
32.2' from Bridge Deck to top of Streambed.

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 2 — Boring No.: BB-SWSL-102

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

Project: White's Bridge #3857 carries White's Bridge Rd over Sebago
Location: Standish-Windham, Maine

Boring No.: BB-SWSL-102
PIN: 15610.00

Driller:	MaineDOT	Elevation (ft.) 251.0	Auger ID/OD: N/A
Operator:	E. Giguere/C. Giles	Datum: NAVD 88	Sampler: Standard Split Spoon
Logged By:	K. Maguire/B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish:	4/17/08, 4/23/08	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2*
Boring Location:	13+24.2, 6.6 Lt.	Casing ID/OD: HW & NW	Water Level*: None Observed
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, FP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Rotary Cone
MDH = weight of 140lb. hammer
WOR/WOH = weight of rods or 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. Soil; Strength (tsf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing/ Results/ AASHTO and Unified Class
50							67			
							66			
	120	24/10	52.00 - 54.00	6/8/8/11	16	21	51		Grey, wet, medium dense, fine to coarse SAND, little gravel, trace silt.	G#207819 A-1-b, SW-SM WC=14.8%
							42			
							67			
							49			
							56			
55	130	24/12	57.00 - 59.00	3/5/9/11	14	18	34		Grey, wet, medium dense, fine to coarse SAND, trace silt, trace gravel.	G#207820 A-1-b, SP WC=19.8%
							20			
							101			
							126			
							140			
							157			
60	140	24/20	63.00 - 65.00	9/10/9/11	19	24	64		Grey, wet, medium dense, fine to coarse SAND, trace silt.	G#207821 A-1-b, SP-SM WC=17.9%
							129			
							178			
							243			
	R1	60/50	67.40 - 72.40	ROD = 63%			0227 ND CORE	183.60	0227 blows for 0.4'. — 67.40	
65									Top of Bedrock at Elev. 183.6'. R1:Bedrock: Grey, white and black, coarse grained GRANITE with mica. Rock Mass Quality = Fair. R1:Core Times (min:sec) 67.4-68.4' (2:49) 68.4-69.4' (2:36) 69.4-70.4' (2:44) 70.4-71.4' (1:56) 71.4-72.4' (1:27) 83% Recovery No water return. 2-5" seams	
	R2	60/60	72.40 - 77.40	ROD = 100%					R2:Bedrock: Grey, white and black, coarse grained GRANITE with mica. Rock Mass Quality = Excellent. R2:Core Times (min:sec) 72.4-73.4' (2:10) 73.4-74.4' (2:35) 74.4-75.4' (2:21) 75.4-76.4' (2:31) 76.4-77.4' (2:35) 100% Recovery No water return	
70								173.60	77.40	
									Bottom of Exploration at 77.40 feet below ground surface.	
75										
80										
85										
90										
95										
100										

Remarks:
Cored thru concrete Bridge Deck, concrete thickness 0.8'.
32.2' from Bridge Deck to top of Streambed.

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 2 of 2 — Boring No.: BB-SWSL-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-A561(000)X

PIN 15610.00

BRIDGE NO. 3857

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	K. MAGUIRE	MAY 2008
CHECKED-REVIEWED	T. WHITE	
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

WHITES BRIDGE
WHITES BRIDGE RD. OVER SEBAGO LAKE
STANDISH-WINDHAM CUMBERLAND CNTY

BORING LOGS

SHEET NUMBER
4
OF 6

Filename: ...\msto\009_BORING LOGS2.dgn Division: GEOTECH Username: terry.white Date:12/15/2008

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

Project: White's Bridge #3857 carries White's Bridge Rd over Sebago
Location: Standish-Windham, Maine

Boring No.: BB-SWSL-101
PIN: 15610.00

Driller:	MaineDOT	Elevation (ft.)	278.2
Operator:	E. Giguere/C. Giles	Datum:	NAVD 88
Logged By:	B. Wilder	Rig Type:	CME 45C
Date Start/Finish:	4/23/08-4/24/08	Drilling Methods:	Cased Wash Boring
Boring Location:	14+16.3, 5.8 Rt.	Casing ID/OD:	HW & NW
Hammer Efficiency Factor:	0.77	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐
Auger ID/OD:	5" Solid Stem	Sampler:	Standard Split Spoon
Hammer Wt./Fall:	140#/30"	Core Barrel:	NQ-2"
Water Level*:	None Observed		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							5SA	277.70	Pavement 0.50	
5	10	24/18	5.00 - 7.00	2/4/5/7	9	12	32		Brown, damp, medium dense, fine to coarse SAND, little gravel, trace silt, (Fill).	GM207801 A-1-b, SW-SM WC=4.8%
							24			
							28			
							24			
							21			
10	20	24/8	10.00 - 12.00	2/2/2/2	4	5	5		Brown, wet, loose, fine to coarse SAND, some gravel, trace silt, (Fill).	GM207802 A-1-b, SM WC=14.9%
							16			
							30			
							123	265.20	13.00	
							144			
15	30	24/10	15.00 - 17.00	9/8/5/8	13	17	28		Grey, wet, medium dense, fine to coarse SAND, some gravel, trace silt.	GM207803 A-1-b, SM-SW WC=11.2%
							49			
							67			
							72			
							88	259.20	19.00	
20	40	24/13	20.00 - 22.00	13/18/17/15	35	45	81		Brown, moist, dense, Gravelly fine to coarse SAND, trace silt.	GM207804 A-1-b, SM-SW WC=9.8%
							109			
							89			
							71			
							63			
25	50	24/4	25.00 - 27.00	4/4/3/3	7	9	4		Similar to above, loose.	
							10			
							11			
							15			
							17			
30	60	24/20	30.00 - 32.00	1/2/3/40	5	6	4		Brown, moist, loose, fine to coarse SAND, some gravel, trace silt.	GM207805 A-1-b, SW WC=17.6%
							025		025 blows for 0.4'. Roller Coned ahead to 32.0' bgs.	
	R1	60/60	32.00 - 37.00				NO CORE	246.20	32.00 R1:BOULDER. R1:Core Times (minsec) 32.0-33.0' (2:41) 33.0-34.0' (1:47) 34.0-35.0' (2:25) 35.0-36.0' (2:26) 36.0-37.0' (2:37) 100% Recovery	
35										
	R2	36/23	37.00 - 40.00						R2:COBBLES and BOULDERS. R2:Core Times (minsec) 37.0-38.0' (2:18) 38.0-39.0' (1:00) 39.0-40.0' (0:15) 63% Recovery Drilled 3.5" Button Bit thru Boulder, then telescoped NW Casing thru.	
40	70	24/13	40.00 - 42.00	4/5/4/4	9	12	37	238.20	40.00 Grey, wet, medium dense, fine to coarse SAND, some gravel, trace silt.	GM207806 A-1-b, SW WC=13.3%
							41			
							49			
							63			
							114			
45	80	24/4	45.00 - 47.00	9/9/12/18	21	27	58		Similar to above.	
							83			
							101			
							171			
							191	229.20	49.00 Bottom of Exploration at 49.00 feet below ground surface.	
50										

Remarks: Broke NW Casing, 10.0' left in hole, moved to BB-SWSL-101A.

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

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Page 1 of 1
Boring No.: BB-SWSL-101

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

Project: White's Bridge #3857 carries White's Bridge Rd over Sebago
Location: Standish-Windham, Maine

Boring No.: BB-SWSL-101A
PIN: 15610.00

Driller:	MaineDOT	Elevation (ft.)	278.2
Operator:	E. Giguere/C. Giles	Datum:	NAVD 88
Logged By:	B. Wilder	Rig Type:	CME 45C
Date Start/Finish:	4/24/08-4/25/08	Drilling Methods:	Cased Wash Boring
Boring Location:	14+23.4, 5.8 Rt.	Casing ID/OD:	HW & NW
Hammer Efficiency Factor:	0.77	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐
Auger ID/OD:	5" Solid Stem	Sampler:	Standard Split Spoon
Hammer Wt./Fall:	140#/30"	Core Barrel:	NQ-2"
Water Level*:	None Observed		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							HW		Drove HW Casing 16.0' bgs., then telescoped with NW Casing, drove to 50.0' bgs and started sampling at 50.0' bgs.	
5									See boring BB-SWSL-101 for soil descriptions in upper 50 feet of strata.	
10										
15							RC	262.20	16.00 BOULDER from 16.0-17.3' bgs. Roller Coned ahead to 45.0' bgs.	
								260.90	17.30	
20										
25										
30										
35										
40										
45							41			
							47			
							48			
							40			
							40			
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.

Page 1 of 2
Boring No.: BB-SWSL-101A

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

Project: White's Bridge #3857 carries White's Bridge Rd over Sebago
Location: Standish-Windham, Maine

Boring No.: BB-SWSL-101A
PIN: 15610.00

Driller:	MaineDOT	Elevation (ft.)	278.2
Operator:	E. Giguere/C. Giles	Datum:	NAVD 88
Logged By:	B. Wilder	Rig Type:	CME 45C
Date Start/Finish:	4/24/08-4/25/08	Drilling Methods:	Cased Wash Boring
Boring Location:	14+23.4, 5.8 Rt.	Casing ID/OD:	HW & NW
Hammer Efficiency Factor:	0.77	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐
Auger ID/OD:	5" Solid Stem	Sampler:	Standard Split Spoon
Hammer Wt./Fall:	140#/30"	Core Barrel:	NQ-2"
Water Level*:	None Observed		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
50	10	24/16	50.00 - 52.00	6/7/7/5	14	18	35	228.20	50.00 Grey, wet, medium dense, fine to coarse SAND, some gravel, trace silt.	
							49			
							43			
							47			
55							69			
							98			
							116			
							49			
							57			
							69			
60	20	24/10	60.00 - 62.00	3/4/4/5	8	10	53		Grey, wet, loose, fine to coarse SAND, little gravel, trace silt.	GM207807 A-1-b, SW WC=15.9%
							69			
							78			
							83			
							106			
65							112			
							110			
							97			
							130			
							117			
70	30	24/18	70.00 - 72.00	4/5/7/12	12	15	79		Grey, wet, medium dense, fine to coarse SAND, trace silt, trace gravel.	GM207808 A-3, SP WC=23.6%
							87			
							119			
							161			
							215			
							288			
							221			
							160			
							121			
							180			
75									Roller Coned ahead to 80.0' bgs.	
80	40	15.6/14	80.00 - 81.30	21/31/50(3.6")	---		223		Grey, wet, very dense, fine to coarse SAND, some gravel, little silt.	GM207809 A-1-b, SW-SM WC=12.2%
	R1	84/26	81.30 - 88.30				050 NO CORE	196.90	050 blows for 0.3'. 81.30 R1:COBBLES and BOULDERS. R1:Core Times (minsec) 81.3-82.3' (2:51) 82.3-83.3' (1:30) 83.3-84.3' (0:49) 84.3-85.3' (0:45) 85.3-86.3' (0:47) 86.3-87.3' (0:40) 87.3-88.3' (0:15) 30% Recovery	
85										
	R2	60/60	88.30 - 93.30	RQD = 80%				188.90	R2:COBBLES. R2:Core Times (minsec) 88.3-89.3' (1:44) 89.30 Top of Bedrock at Elev. 188.9'. R2:Bedrock: Grey, white and black, coarse grained GRANITE with mica. Rock Mass Quality = Good. R2:Core Times (minsec)(continued) 89.3-90.3' (2:24) 90.3-91.3' (2:36) 91.3-92.3' (2:34) 92.3-93.3' (2:37) 100% Recovery	
90										
95								184.90	93.30 Bottom of Exploration at 93.30 feet below ground surface.	
100										

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 2 of 2
Boring No.: BB-SWSL-101A

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-A561(000)X

PIN 15610.00

BRIDGE NO. 3857

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	K. MAGUIRE	MAY 2008
CHECKED-REVIEWED		
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

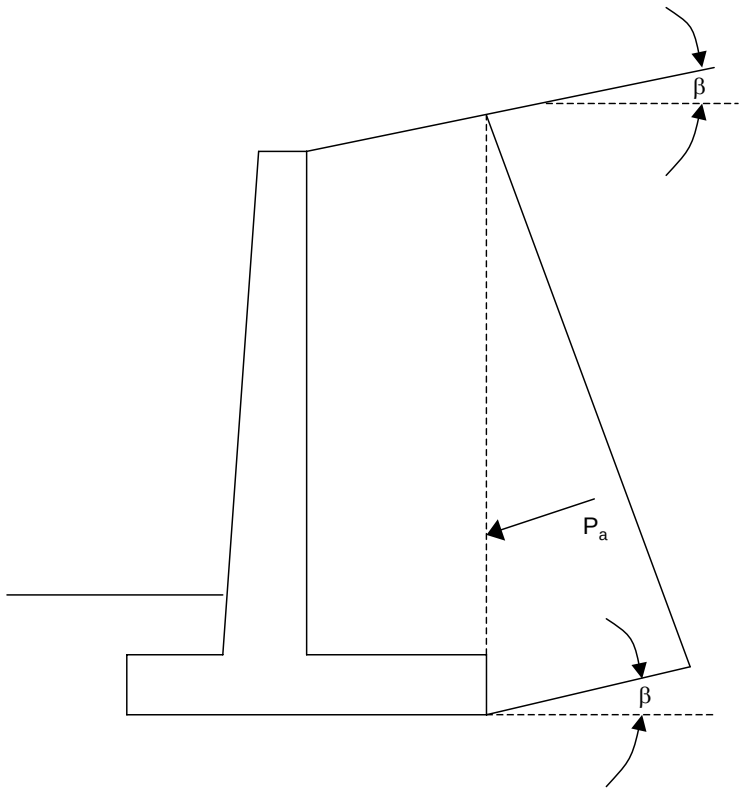
P.E. NUMBER

DATE

WHITES BRIDGE
WHITES BRIDGE RD. OVER SEBAGO LAKE
STANDISH-WINDHAM CUMBERLAND CNTY

BORING LOGS

SHEET NUMBER
5
OF 6



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

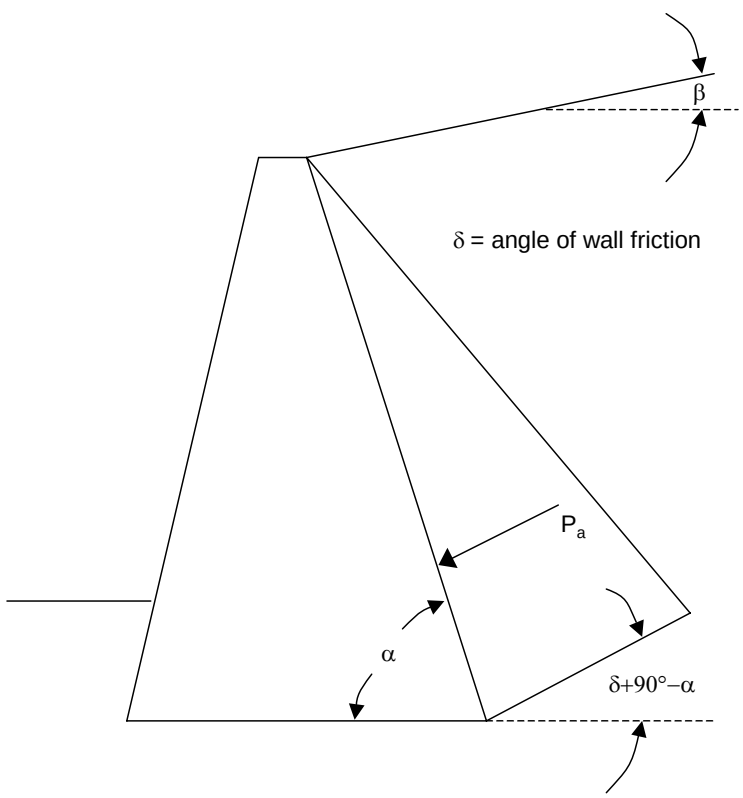
For a horizontal backfill surface, β = 0°:

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

For a sloped backfill surface, β > 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 δ + 90° - α

Rankine and Coulomb Active Earth Pressure Coefficients

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

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

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: White's Bridge #3857 carries White's Bridge Rd over Sebago Lake Basin
Location: Standish-Windham, Maine

Boring No.: BB-SWSL-101
PIN: 15610.00

Driller:	MaineDOT	**Elevation (ft.)**	278.2	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/23/08-4/24/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+16.3, 5.8 Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
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, PP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
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WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
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N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	277.7	Pavement — 0.5	
5	1D	24/18	5.0 - 7.0	2/4/5/7	9	12	32		Brown, damp, medium dense, fine to coarse SAND, little gravel, trace silt, (Fill).	G#207801 A-1-b, SW-SM WC=4.8%
							24			
							28			
							24			
							21			
10	2D	24/8	10.0 - 12.0	2/2/2/2	4	5	5		Brown, wet, loose, fine to coarse SAND, some gravel, trace silt, (Fill).	G#207802 A-1-b, SW WC=14.9%
							16			
							30			
							123	265.2	13.0	
							144			
15	3D	24/10	15.0 - 17.0	9/8/5/8	13	17	28		Grey, wet, medium dense, fine to coarse SAND, some gravel, trace silt.	G#207803 A-1-b, SW-SM WC=11.2%
							49			
							67			
							72			
							88	259.2	19.0	
20	4D	24/13	20.0 - 22.0	13/18/17/15	35	45	81		Brown, moist, dense, Gravelly fine to coarse SAND, trace silt.	G#207804 A-1-a, SW-SM WC=9.8%
							109			
							89			
							71			
25							63			

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Boring No.: BB-SWSL-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: White's Bridge #3857 carries White's Bridge Rd over Sebago Lake Basin
Location: Standish-Windham, Maine

Boring No.: BB-SWSL-101
PIN: 15610.00

Driller:	MaineDOT	**Elevation (ft.)**	278.2	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/23/08-4/24/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+16.3, 5.8 Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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HSA = Hollow Stem Auger
RC = Roller Cone
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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} = 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	5D	24/4	25.0 - 27.0	4/4/3/3	7	9	4			Similar to above, loose.	
							10				
							11				
							15				
							17				
30	6D	24/20	30.0 - 32.0	1/2/3/40	5	6	4			Brown, moist, loose, fine to coarse SAND, some gravel, trace silt.	G#207805 A-1-b, SW WC=17.6%
							[a]25			[a]25 blows for 0.4'. Roller Coned ahead to 32.0' bgs.	
								246.2		32.0	
	R1	60/60	32.0 - 37.0				NQ CORE			R1:BOULDER. R1:Core Times (min:sec) 32.0-33.0' (2:41) 33.0-34.0' (1:47) 34.0-35.0' (2:25) 35.0-36.0' (2:26) 36.0-37.0' (2:37) 100% Recovery	
35											
	R2	36/23	37.0 - 40.0							R2:COBBLES and BOULDERS. R2:Core Times (min:sec) 37.0-38.0' (2:18) 38.0-39.0' (1:00) 39.0-40.0' (0:15) 63% Recovery Drilled 3.5" Button Bit thru Boulder, then telescoped NW Casing thru.	
40								238.2		40.0	
	7D	24/13	40.0 - 42.0	4/5/4/4	9	12	37			Grey, wet, medium dense, fine to coarse SAND, some gravel, trace silt.	G#207806 A-1-b, SW WC=13.3%
							41				
							49				
							63				
							114				
45	8D	24/4	45.0 - 47.0	9/9/12/18	21	27	58			Similar to above.	
							83				
							101				
							171				
							191	229.2		49.0 **Bottom of Exploration at 49.00 feet below ground surface.** Broke NW Casing, 10.0' left in hole, moved to BB-SWSL-101A.	
50											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: White's Bridge #3857 carries White's Bridge Rd over Sebago Lake Basin
Location: Standish-Windham, Maine

Boring No.: BB-SWSL-101A
PIN: 15610.00

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Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/24/08-4/25/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+23.4, 5.8 Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							HW			Drove HW Casing 16.0' bgs., then telescoped with NW Casing, drove to 50.0' bgs and started sampling at 50.0' bgs.	
										See boring BB-SWSL-101 for soil descriptions in upper 50 feet of strata.	
5											
10											
15											
							RC	262.2		16.0 BOULDER from 16.0-17.3' bgs. Roller Coned ahead to 45.0' bgs. 17.3	
								260.9			
20											
25											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: White's Bridge #3857 carries White's Bridge Rd over Sebago Lake Basin
Location: Standish-Windham, Maine

Boring No.: BB-SWSL-101A
PIN: 15610.00

Driller:	MaineDOT	**Elevation (ft.)**	278.2	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/24/08-4/25/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+23.4, 5.8 Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
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, PP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25											
30											
35											
40											
45							41				
							47				
							48				
							40				
50							40				

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: White's Bridge #3857 carries White's Bridge Rd over Sebago Lake Basin
Location: Standish-Windham, Maine

Boring No.: BB-SWSL-101A
PIN: 15610.00

Driller:	MaineDOT	**Elevation (ft.)**	278.2	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/24/08-4/25/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+23.4, 5.8 Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
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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	1D	24/16	50.0 - 52.0	6/7/7/5	14	18	35	228.2		50.0 Grey, wet, medium dense, fine to coarse SAND, some gravel, trace silt.	
							49				
							43				
							47				
							69				
55							98				
							116				
							49				
							57				
							69				
60	2D	24/10	60.0 - 62.0	3/4/4/5	8	10	53			Grey, wet, loose, fine to coarse SAND, little gravel, trace silt.	G#207807 A-1-b, SW WC=15.9%
							69				
							78				
							83				
							106				
65							112				
							110				
							97				
							130				
							117				
70	3D	24/18	70.0 - 72.0	4/5/7/12	12	15	79			Grey, wet, medium dense, fine to coarse SAND, trace silt, trace gravel.	G#207808 A-3, SP WC=23.6%
							87				
							119				
							161				
75							215				

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: White's Bridge #3857 carries White's Bridge Rd over Sebago Lake Basin
Location: Standish-Windham, Maine

Boring No.: BB-SWSL-101A
PIN: 15610.00

Driller:	MaineDOT	**Elevation (ft.)**	278.2	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/24/08-4/25/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+23.4, 5.8 Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
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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							288			Roller Coned ahead to 80.0' bgs.	
							221				
							160				
							121				
							180				
80	4D	15.6/14	80.0 - 81.3	21/31/50(3.6")	---		223			Grey, wet, very dense, fine to coarse SAND, some gravel, little silt.	G#207809 A-1-b, SW-SM WC=12.2%
	R1	84/26	81.3 - 88.3				a50 NQ CORE	196.9		a50 blows for 0.3'. 81.3	
										R1:COBBLES and BOULDERS. R1:Core Times (min:sec) 81.3-82.3' (2:51) 82.3-83.3' (1:30) 83.3-84.3' (0:49) 84.3-85.3' (0:15) 85.3-86.3' (0:47) 86.3-87.3' (0:40) 87.3-88.3' (0:15) 30% Recovery	
85											
	R2	60/60	88.3 - 93.3	RQD = 80%				188.9		R2:COBBLES. R2:Core Times (min:sec) 88.3-89.3' (1:44) 89.3	
90										Top of Bedrock at Elev. 188.9'. R2:Bedrock: Grey, white and black, coarse grained GRANITE with mica. Rock Mass Quality = Good. R2:Core Times (min:sec)(continuted) 89.3-90.3' (2:24) 90.3-91.3' (2:36) 91.3-92.3' (2:34)	
								184.9		92.3-93.3' (2:37) 100% Recovery 93.3	
										Bottom of Exploration at 93.30 feet below ground surface.	
95											
100											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: White's Bridge #3857 carries White's Bridge Rd over Sebago Lake Basin
Location: Standish-Windham, Maine

Boring No.: BB-SWSL-102
PIN: 15610.00

Driller:	MaineDOT	**Elevation (ft.)**	251.0	**Auger ID/OD:**	N/A
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire/B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/17/08, 4/23/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	13+24.2, 6.6 Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
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N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/5	0.0 - 2.0	9/5/27/32	32	41	9		Brown, wet, very dense, Gravelly fine to coarse SAND, trace silt.	G#207810 A-1-b, SW WC=13.3%
							16			
	2D	24/8	2.8 - 4.8	5/6/6/6	12	15	54		Grey-brown, wet, medium dense, Gravelly fine to coarse SAND, trace silt, with broken rock fragments.	
							50			
							48			
5							58			
							38			
							25			
	3D	24/6	8.0 - 10.0	3/3/2/2	5	6	9		Brown-white, black-green, GRAVEL, wet, loose, no sand.	
							26			
10							27			
							17			
							54			
	4D	24/13	13.0 - 15.0	5/5/3/4	8	10	30		Similar to Gravel above from 13.0-14.5' bgs.	G#207811 A-3, SP WC=19.4%
							41	236.5	14.5	
15							40		Grey, wet, loose, fine to coarse SAND, trace gravel, trace silt.	
							41			
							40			
	5D	24/6	18.0 - 20.0	2/2/3/4	5	6	15		Grey, wet, loose, fine to coarse SAND, little silt, trace gravel. Telescoped NW Casing at 18.0' bgs.	G#207812 A-2-4, SM WC=21.6%
							21			
20							33			
							54			
							34			
	6D	24/12	23.5 - 25.5	22/32/15/12	47	60	73		Grey, wet, very dense, fine to coarse SAND, some gravel, little silt.	G#207813 A-1-b, SW-SM WC=12.3%
25							53			

Remarks:

Cored thru concrete Bridge Deck, concrete thickness 0.8'.
32.2' from Bridge Deck to top of Streambed.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: White's Bridge #3857 carries White's Bridge Rd over Sebago Lake Basin
Location: Standish-Windham, Maine

Boring No.: BB-SWSL-102
PIN: 15610.00

Driller:	MaineDOT	**Elevation (ft.)**	251.0	**Auger ID/OD:**	N/A
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire/B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/17/08, 4/23/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	13+24.2, 6.6 Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
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/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							30			
							38			
	7D	24/17	27.0 - 29.0	3/6/7/9	13	17	25	224.0	27.0 Grey, wet, very stiff, SILT, some clay, trace fine sand.	G#207814 A-4, ML WC=30.8% LL=24 PL=21 PI=3
							25			
							29			
30	V1		30.1 - 30.5	Su=1317/268 psf			31		55x110 mm vane raw torque readings: V1: 29.5/6.0 ft-lbs	
	MV		~~30.3 - 30.7~~	~~Would not push~~					Failed 55x110 mm vane attempt.	
							33			
	8D	24/18	32.5 - 34.5	WOR/WOH/WOH/1	---		38		Grey, wet, stiff, non-plastic, SILT, some fine to medium sand, some clay, trace gravel.	G#207815 A-4, ML WC=27.2% Non-Plastic
	V2		33.1 - 33.5	Su= >1978 psf			37	217.5	Over vane reading capacity >46.0 ft-lbs 33.5 Grey, wet, very loose, silty fine SAND, trace gravel.	
							32			
35							37			
							41			
	9D	24/16	37.0 - 39.0	3/4/5/7	9	12	34		Grey, wet, medium dense, fine to coarse SAND, little silt, trace gravel.	G#207816 A-2-4, SM WC=16.6%
							31			
							59			
40							79			
							110			
	10D	24/15	42.0 - 44.0	9/11/10/11	21	27	54		Grey, wet, medium dense, fine to coarse SAND, some silt, little gravel.	G#207817 A-2-4, SM WC=14.9%
							51			
							59			
45							81			
							78			
	11D	24/16	47.0 - 49.0	8/7/8/10	15	19	51		Grey, wet, medium dense, fine to coarse SAND, some silt, trace gravel.	G#207818 A-1-b, SM WC=17.6%
							42			
50							59			

Remarks:

Cored thru concrete Bridge Deck, concrete thickness 0.8'.
32.2' from Bridge Deck to top of Streambed.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: White's Bridge #3857 carries White's Bridge Rd over Sebago Lake Basin
Location: Standish-Windham, Maine

Boring No.: BB-SWSL-102
PIN: 15610.00

Driller:	MaineDOT	**Elevation (ft.)**	251.0	**Auger ID/OD:**	N/A
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire/B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/17/08, 4/23/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	13+24.2, 6.6 Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
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N-uncorrected = Raw field SPT N-value
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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
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WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50							67				
							66				
	12D	24/10	52.0 - 54.0	6/8/8/11	16	21	51			Grey, wet, medium dense, fine to coarse SAND, little gravel, trace silt.	G#207819 A-1-b, SW-SM WC=14.8%
							42				
							67				
55							49				
							56				
	13D	24/12	57.0 - 59.0	3/5/9/11	14	18	34			Grey, wet, medium dense, fine to coarse SAND, trace silt, trace gravel.	G#207820 A-1-b, SP WC=19.8%
							20				
							101				
60							126				
							140				
							157				
	14D	24/20	63.0 - 65.0	9/10/9/11	19	24	64			Grey, wet, medium dense, fine to coarse SAND, trace silt.	G#207821 A-1-b, SP-SM WC=17.9%
							129				
65							178				
							243				
	R1	60/50	67.4 - 72.4	RQD = 63%			[a]227 NQ CORE	183.6		[a]227 blows for 0.4'. 67.4 Top of Bedrock at Elev. 183.6'. R1:Bedrock: Grey, white and black, coarse grained GRANITE with mica. Rock Mass Quality = Fair. R1:Core Times (min:sec) 67.4-68.4' (2:49) 68.4-69.4' (2:36) 69.4-70.4' (2:44) 70.4-71.4' (1:56) 71.4-72.4' (1:27) 83% Recovery No water return, 2-5" seams	
70											
	R2	60/60	72.4 - 77.4	RQD = 100%						R2:Bedrock: Grey, white and black coarse grained GRANITE with mica. Rock Mass Quality = Excellent. R2:Core Times (min:sec) 72.4-73.4' (2:10) 73.4-74.4' (2:35)	
75											

Remarks:

Cored thru concrete Bridge Deck, concrete thickness 0.8'.
32.2' from Bridge Deck to top of Streambed.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: White's Bridge #3857 carries White's Bridge Rd over Sebago Lake Basin
Location: Standish-Windham, Maine

Boring No.: BB-SWSL-102
PIN: 15610.00

Driller:	MaineDOT	**Elevation (ft.)**	251.0	**Auger ID/OD:**	N/A
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire/B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/17/08, 4/23/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	13+24.2, 6.6 Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
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, PP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75										74.4-75.4' (2:21) 75.4-76.4' (2:31) 76.4-77.4' (2:35) 100% Recovery No water return	
								173.6		77.4 **Bottom of Exploration at 77.40 feet below ground surface.**	
80											
85											
90											
95											
100											

Remarks:

Cored thru concrete Bridge Deck, concrete thickness 0.8'.
32.2' from Bridge Deck to top of Streambed.

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: White's Bridge #3857 carries White's Bridge Rd over Sebago Lake Basin
Location: Standish-Windham, Maine

Boring No.: BB-SWSL-103
PIN: 15610.00

Driller:	MaineDOT	**Elevation (ft.)**	281.3	**Auger ID/OD:**	N/A
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire/B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/15-17/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+40.7, 6.4 Lt.	**Casing ID/OD:**	NW & HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	280.9	Pavement — 0.4	
	1D	24/16	2.0 - 4.0	6/7/8/7	15	19			Brown, damp, medium dense, fine to coarse SAND, some gravel, trace silt, (Fill).	G#207822 A-1-b, SW-SM WC=5.7%
5	2D	24/16	5.0 - 7.0	3/2/2/2	4	5	6		Light brown, moist, loose, fine to coarse SAND, little silt, little gravel, (Fill).	G#207823 A-1-b, SM WC=9.9%
							12			
							20			
							18			
	3D	24/6	9.0 - 11.0	3/4/3/4	7	9	3		Light brown, wet, loose, fine to coarse SAND, trace gravel, trace silt, (Fill).	G#207824 A-1-b, SW WC=14.4%
10							12			
							14			
							243			
							73		Large Cobble from 13.0-13.8' bgs.	
	4D	24/18	14.0 - 16.0	2/6/6/4	12	15	63		Light brown, wet, medium dense, fine to coarse SAND, trace gravel, trace silt, (Fill). Bent NW Casing, switched to HW Casing.	G#207825 A-1-b, SW-SM WC=10.1%
15							77			
							80	264.3	17.0	
							97			
							84			
	5D	18/14	19.0 - 20.5	8/11/8	19	24	60		Brown, wet, very stiff, SILT, some fine to coarse SAND, little clay, little gravel.	G#209901 A-4, ML WC=16.5%
20							85	260.3	21.0	
							107			
							90			
							7			
25	MD	2.4/0	24.2 - 24.4	50(2.4")	---		11		Failed sample attempt, large Cobble. Roller Coned ahead from 24.2-25.0' bgs.	

Remarks:

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Boring No.: BB-SWSL-103

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: White's Bridge #3857 carries White's Bridge Rd over Sebago Lake Basin
Location: Standish-Windham, Maine

Boring No.: BB-SWSL-103
PIN: 15610.00

Driller:	MaineDOT	**Elevation (ft.)**	281.3	**Auger ID/OD:**	N/A
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire/B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/15-17/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+40.7, 6.4 Lt.	**Casing ID/OD:**	NW & HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

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							26			Switched back to NW Casing.	
							20				
							34				
							58				
	6D	24/6	29.0 - 31.0	16/13/9/9	22	28	24			Light brown, wet, medium dense, fine to coarse SAND, little silt, little gravel.	
30							38				
							29				
							34				
							49	248.3		33.0	
	7D	24/15	34.0 - 36.0	5/10/11/13	21	27	19			Grey, wet, medium dense, fine to coarse SAND, some gravel, trace silt.	G#209902 A-1-b, SW WC=13.9%
35							49				
							74				
							59				
							34				
	8D	24/12	39.0 - 41.0	11/6/6/14	12	15	25			Grey, wet, medium dense, fine to coarse SAND, trace silt, trace gravel.	G#209903 A-1-b, SP-SM WC=19.5%
40							55				
							40				
							37				
							41				
	9D	24/13	44.0 - 46.0	12/7/4/5	11	14	17			Similar to above.	G#209904 A-1-b, SP-SM WC=16.3%
45							48				
							44				
							44				
							62				
50	10D	24/2	49.0 - 51.0	14/13/13/11	26	33	19			Grey, wet, dense, fine to medium SAND, trace silt.	G#209905 A-3, SP-SM

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: White's Bridge #3857 carries White's Bridge Rd over Sebago Lake Basin
Location: Standish-Windham, Maine

Boring No.: BB-SWSL-103
PIN: 15610.00

Driller:	MaineDOT	**Elevation (ft.)**	281.3	**Auger ID/OD:**	N/A
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire/B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/15-17/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+40.7, 6.4 Lt.	**Casing ID/OD:**	NW & HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50							41				WC=25.0%
							71				
							72				
							68				
	11D	24/16	54.0 - 56.0	9/5/5/6	10	13	26			Grey, wet, medium dense, fine to coarse SAND, trace silt, trace gravel.	G#209906 A-1-b, SP-SM WC=17.2%
55							45				
							73				
							81				
							78				
	12D	24/14	59.0 - 61.0	6/4/5/6	9	12	44			Similar to above.	
60							61				
							79				
							89				
							97				
	13D	24/15	64.0 - 66.0	9/5/5/5	10	13	54			Similar to above.	G#209907 A-1-b, SP-SM WC=18.8%
65							57				
							79				
							83				
							102				
	14D	24/12	69.0 - 71.0	5/5/3/4	8	10	63			Grey, wet, loose, fine to coarse SAND, little gravel, trace silt.	G#209908 A-1-b, SW-SM WC=15.5%
70							64				
							73				
							81				
							84				
75	15D	24/6	74.0 - 76.0	6/3/3/4	6	8	59			Grey, saturated, loose, fine SAND, trace silt.	G#209909 A-3, SP-SM

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: White's Bridge #3857 carries White's Bridge Rd over Sebago Lake Basin
Location: Standish-Windham, Maine

Boring No.: BB-SWSL-103
PIN: 15610.00

Driller:	MaineDOT	**Elevation (ft.)**	281.3	**Auger ID/OD:**	N/A
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire/B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/15-17/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+40.7, 6.4 Lt.	**Casing ID/OD:**	NW & HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75							71				WC=30.9%
							94				
							129				
							212				
	16D	24/16	79.0 - 81.0	16/18/26/20	44	56	94			Grey, wet, very dense, fine to coarse SAND, some gravel, little silt.	G#209910 A-2-4, SM WC=11.0%
80							138				
							145				
							144				
							201				
							200				
85	17D	16.8/13	85.0 - 86.4	28/25/50(4.8")	---		103			Grey, wet, very dense, fine to coarse SAND, some silt, little gravel.	G#209911 A-1-b, SM WC=11.3%
							a50	194.9		a50 blows for 0.4'. 86.4	
	R1	60/48	87.0 - 92.0	RQD = 17%			NQ CORE			Top of Bedrock at Elev. 194.9'. Roller Coned ahead to 87.0' bgs. R1:Bedrock: Grey, white and black, coarse grained GRANITE with mica, very fractured. Rock Mass Quality = Very Poor R1:Core Times (min:sec) 87.0-88.0' (2:43) 300# down pressure 88.0-89.0' (2:14) 89.0-90.0' (1:56) 90.0-91.0' (1:19) 91.0-92.0' (2:58) 80% Recovery Lost water return at 90.5' bgs.	
90											
	R2	73.2/58.8	92.0 - 98.1	RQD = 52%						R2: Similar to R1. Rock Mass Quality = Fair. Core Times (min:sec) 92.0-93.0' (1:47) 300# down pressure 93.0-94.0' (2:22) 94.0-95.0' (2:20) 95.0-96.0' (1:00) 95.5-96.7' core barrel dropped, possible void. 96.0-97.0' (0:45) 97.0-98.0' (1:21) 98.0-98.1' (0:30) 80% Recovery No water return	
95								183.2		98.1	
100										**Bottom of Exploration at 98.10 feet below ground surface.**	

Remarks:

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Appendix B

Laboratory Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Standish-Windham Project Number: 15610.00

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-SWSL-103, 1D	12+40.7	6.4 Lt.	2.0-4.0	207822	1	5.7			SW-SM	A-1-b	0
BB-SWSL-103, 2D	12+40.7	6.4 Lt.	5.0-7.0	207823	1	9.9			SM	A-1-b	II
BB-SWSL-103, 3D	12+40.7	6.4 Lt.	9.0-11.0	207824	1	14.4			SW	A-1-b	0
BB-SWSL-103, 4D	12+40.7	6.4 Lt.	14.0-16.0	207825	1	10.1			SW-SM	A-1-b	0
BB-SWSL-103, 5D	12+40.7	6.4 Lt.	19.0-20.5	209901	1	16.5			ML	A-4	IV
BB-SWSL-103, 7D	12+40.7	6.4 Lt.	34.0-36.0	209902	2	13.9			SW	A-1-b	0
BB-SWSL-103, 8D	12+40.7	6.4 Lt.	39.0-41.0	209903	2	19.5			SP-SM	A-1-b	0
BB-SWSL-103, 9D	12+40.7	6.4 Lt.	44.0-46.0	209904	2	16.3			SP-SM	A-1-b	0
BB-SWSL-103, 10D	12+40.7	6.4 Lt.	49.0-51.0	209905	2	25.0			SP-SM	A-3	0
BB-SWSL-103, 11D	12+40.7	6.4 Lt.	54.0-56.0	209906	2	17.2			SP-SM	A-1-b	0
BB-SWSL-103, 13D	12+40.7	6.4 Lt.	64.0-66.0	209907	3	18.8			SP-SM	A-1-b	0
BB-SWSL-103, 14D	12+40.7	6.4 Lt.	69.0-71.0	209908	3	15.5			SW-SM	A-1-b	0
BB-SWSL-103, 15D	12+40.7	6.4 Lt.	74.0-76.0	209909	3	30.9			SP-SM	A-3	0
BB-SWSL-103, 16D	12+40.7	6.4 Lt.	79.0-81.0	209910	3	11.0			SM	A-2-4	II
BB-SWSL-103, 17D	12+40.7	6.4 Lt.	85.0-86.4	209911	3	11.3			SM	A-1-b	II
BB-SWSL-102, 1D	13+24.2	6.6 Lt.	0.0-2.0	207810	4	13.3			SW	A-1-b	0
BB-SWSL-102, 4D	13+24.2	6.6 Lt.	13.0-15.0	207811	4	19.4			SP	A-3	0
BB-SWSL-102, 5D	13+24.2	6.6 Lt.	18.0-20.0	207812	4	21.6			SM	A-2-4	II
BB-SWSL-102, 6D	13+24.2	6.6 Lt.	23.5-25.5	207813	4	12.3			SW-SM	A-1-b	0
BB-SWSL-102, 7D	13+24.2	6.6 Lt.	27.0-29.0	207814	4	30.8	24	3	ML	A-4	IV
BB-SWSL-102, 8D	13+24.2	6.6 Lt.	32.5-34.5	207815	4	27.2	-N	P-	ML	A-4	IV
BB-SWSL-102, 9D	13+24.2	6.6 Lt.	37.0-39.0	207816	5	16.6			SM	A-2-4	II
BB-SWSL-102, 10D	13+24.2	6.6 Lt.	42.0-44.0	207817	5	14.9			SM	A-2-4	II
BB-SWSL-102, 11D	13+24.2	6.6 Lt.	47.0-49.0	207818	5	17.6			SM	A-1-b	II
BB-SWSL-102, 12D	13+24.2	6.6 Lt.	52.0-54.0	207819	5	14.8			SW-SM	A-1-b	0
BB-SWSL-102, 13D	13+24.2	6.6 Lt.	57.0-59.0	207820	5	19.8			SP	A-1-b	0
BB-SWSL-102, 14D	13+24.2	6.6 Lt.	63.0-65.0	207821	5	17.9			SP-SM	A-1-b	0
BB-SWSL-101, 1D	14+06.3	5.8 Rt.	5.0-7.0	207801	6	4.8			SW-SM	A-1-b	0
BB-SWSL-101, 2D	14+06.3	5.8 Rt.	10.0-12.0	207802	6	14.9			SW	A-1-b	0
BB-SWSL-101, 3D	14+06.3	5.8 Rt.	15.0-17.0	207803	6	11.2			SW-SM	A-1-b	0
BB-SWSL-101, 4D	14+06.3	5.8 Rt.	20.0-22.0	207804	6	9.8			SW-SM	A-1-a	0
BB-SWSL-101, 6D	14+06.3	5.8 Rt.	30.0-32.0	207805	6	17.6			SW	A-1-b	0
BB-SWSL-101, 7D	14+06.3	5.8 Rt.	40.0-42.0	207806	6	13.3			SW	A-1-b	0
BB-SWSL-101A, 2D	14+23.4	5.8 Rt.	60.0-62.0	207807	7	15.9			SW	A-1-b	0
BB-SWSL-101A, 3D	14+23.4	5.8 Rt.	70.0-72.0	207808	7	23.6			SP	A-3	0
BB-SWSL-101A, 4D	14+23.4	5.8 Rt.	80.0-81.3	207809	7	12.2			SW-SM	A-1-b	0

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.

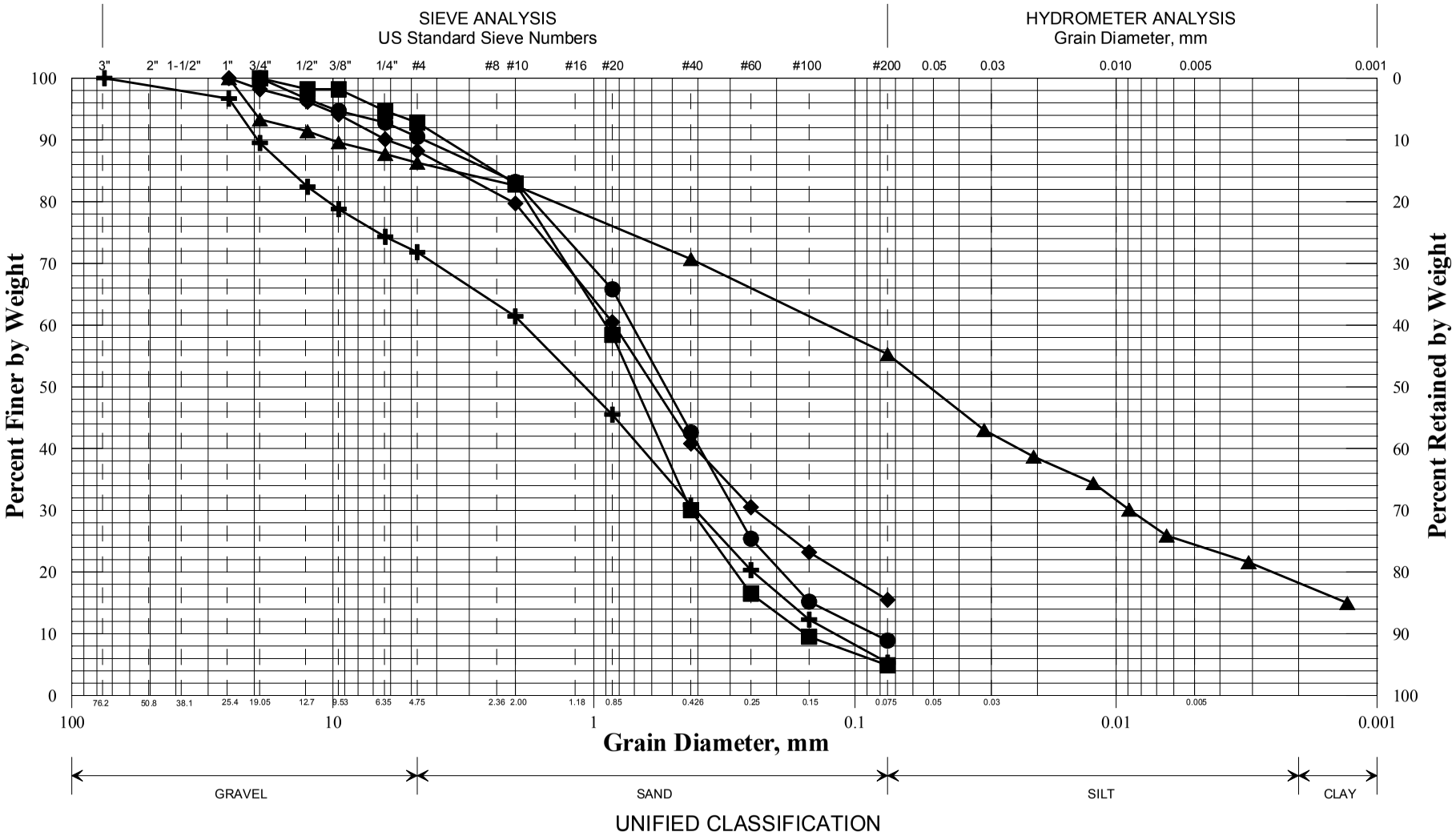
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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-SWSL-103/1D	12+40.7	6.4 LT	2.0-4.0	SAND, some gravel, trace silt.	5.7			
◆	BB-SWSL-103/2D	12+40.7	6.4 LT	5.0-7.0	SAND, little silt, little gravel.	9.9			
■	BB-SWSL-103/3D	12+40.7	6.4 LT	9.0-11.0	SAND, trace gravel, trace silt.	14.4			
●	BB-SWSL-103/4D	12+40.7	6.4 LT	14.0-16.0	SAND, trace gravel, trace silt.	10.1			
▲	BB-SWSL-103/5D	12+40.7	6.4 LT	19.0-20.5	SILT, some sand, little clay, little gravel.	16.5			
✕									

PIN
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Town
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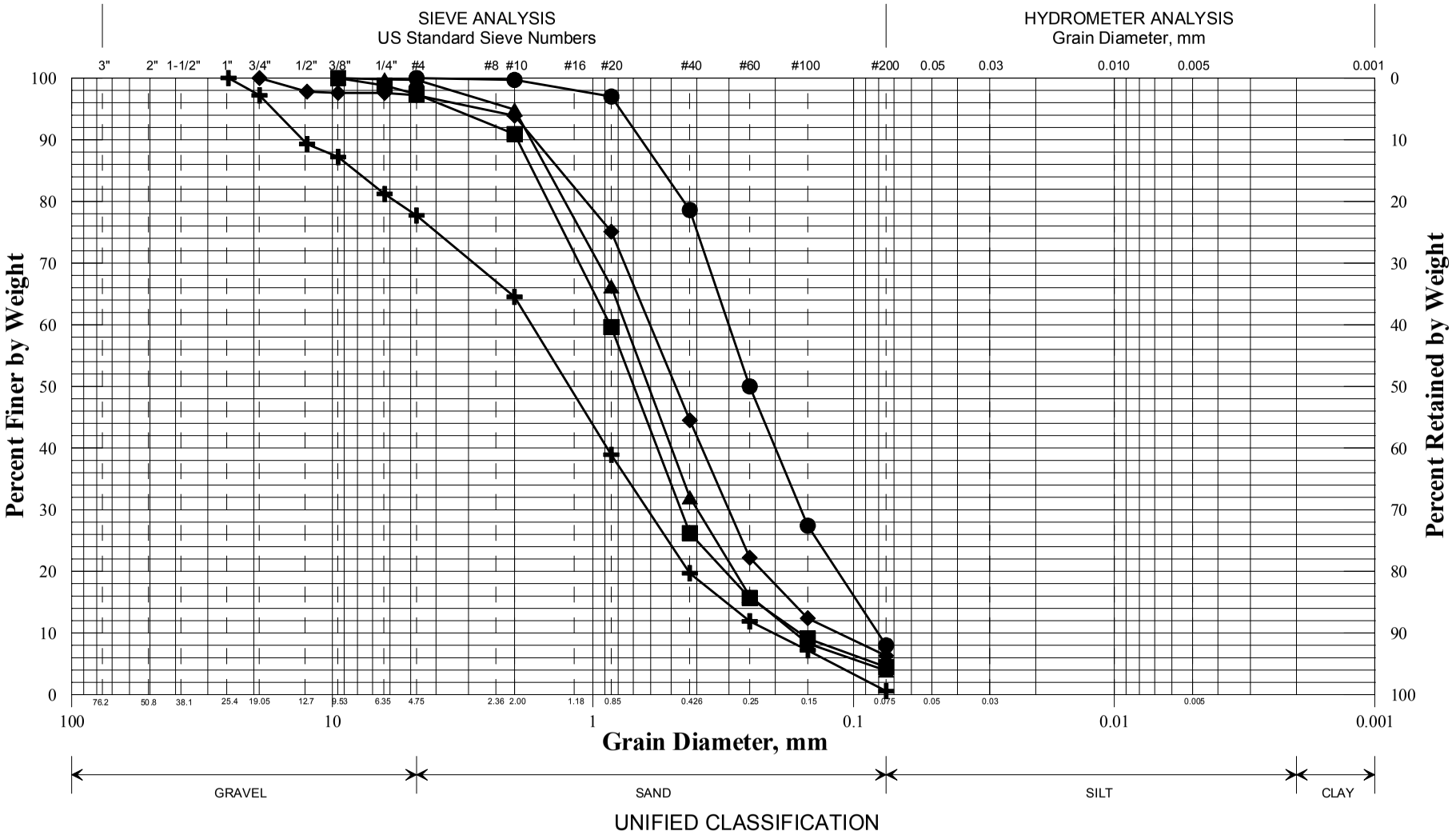
Reported by/Date
WHITE, TERRY A 6/3/2008

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-SWSL-103/7D	12+40.7	6.4 LT	34.0-36.0	SAND, some gravel, trace silt.	13.9			
◆	BB-SWSL-103/8D	12+40.7	6.4 LT	39.0-41.0	SAND, trace silt, trace gravel.	19.5			
■	BB-SWSL-103/9D	12+40.7	6.4 LT	44.0-46.0	SAND, trace silt, trace gravel.	16.3			
●	BB-SWSL-103/10D	12+40.7	6.4 LT	49.0-51.0	SAND, trace silt.	25.0			
▲	BB-SWSL-103/11D	12+40.7	6.4 LT	54.0-56.0	SAND, trace silt, trace gravel.	17.2			
✕									

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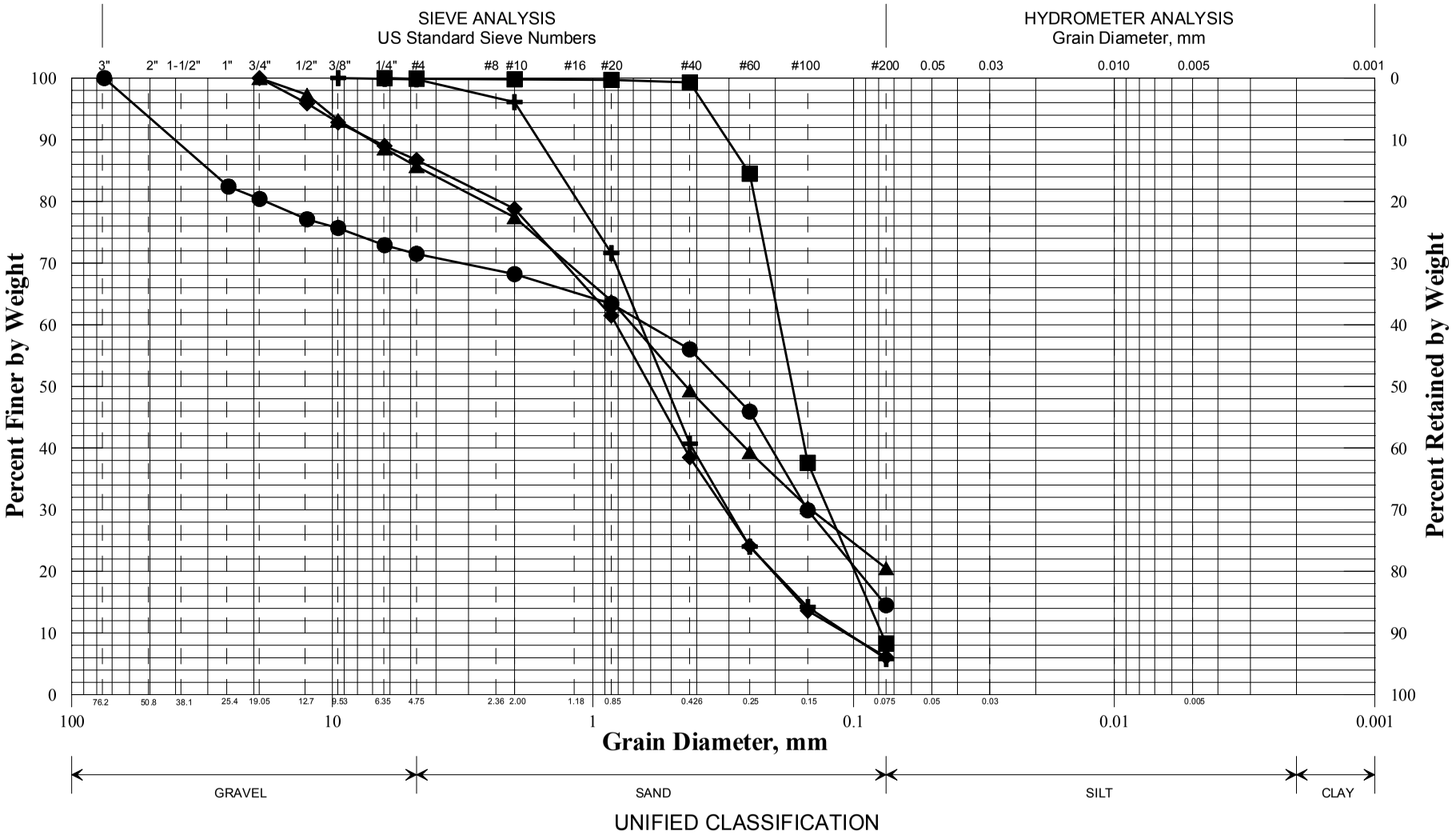
WHITE, TERRY A 5/23/2008

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-SWSL-103/13D	12+40.7	6.4 LT	64.0-66.0	SAND, trace silt, trace gravel.	18.8			
◆	BB-SWSL-103/14D	12+40.7	6.4 LT	69.0-71.0	SAND, little gravel, trace silt.	15.5			
■	BB-SWSL-103/15D	12+40.7	6.4 LT	74.0-76.0	SAND, trace silt.	30.9			
●	BB-SWSL-103/16D	12+40.7	6.4 LT	79.0-81.0	SAND, some gravel, little silt.	11.0			
▲	BB-SWSL-103/17D	12+40.7	6.4 LT	85.0-86.4	SAND, some silt, little gravel.	11.3			
✕									

PIN
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Town
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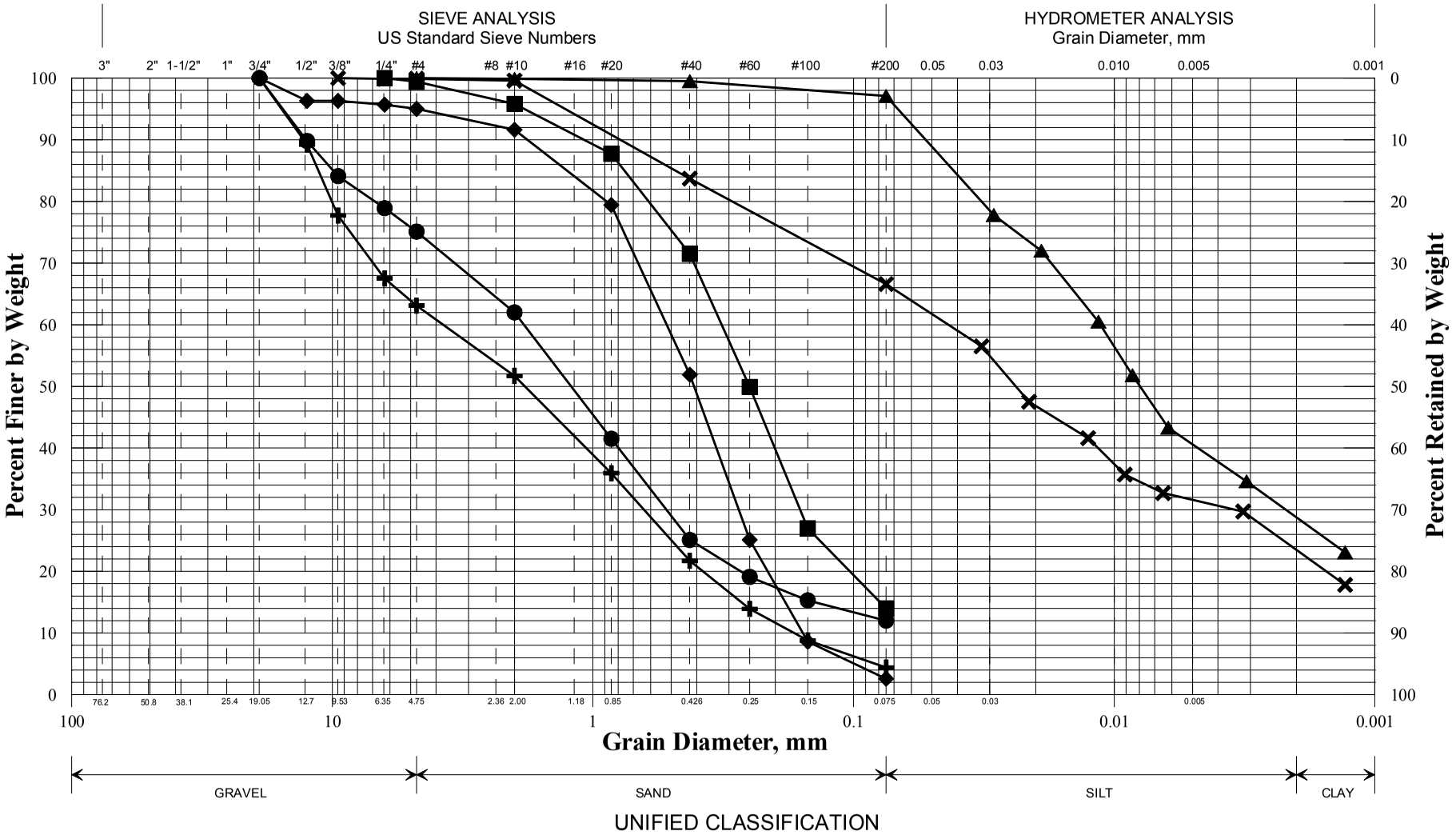
State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-SWSL-102/1D	13+24.2	6.6 LT	0.0-2.0	Gravelly SAND, trace silt.	13.3			
◆	BB-SWSL-102/4D	13+24.2	6.6 LT	13.0-15.0	SAND, trace gravel, trace silt.	19.4			
■	BB-SWSL-102/5D	13+24.2	6.6 LT	18.0-20.0	SAND, little silt, trace gravel.	21.6			
●	BB-SWSL-102/6D	13+24.2	6.6 LT	23.5-25.5	SAND, some gravel, little silt.	12.3			
▲	BB-SWSL-102/7D	13+24.2	6.6 LT	27.0-29.0	SILT, some clay, trace sand.	30.8	24	21	3
✕	BB-SWSL-102/8D	13+24.2	6.6 LT	32.5-34.5	SILT, some sand, some clay, trace gravel.	27.2			NP

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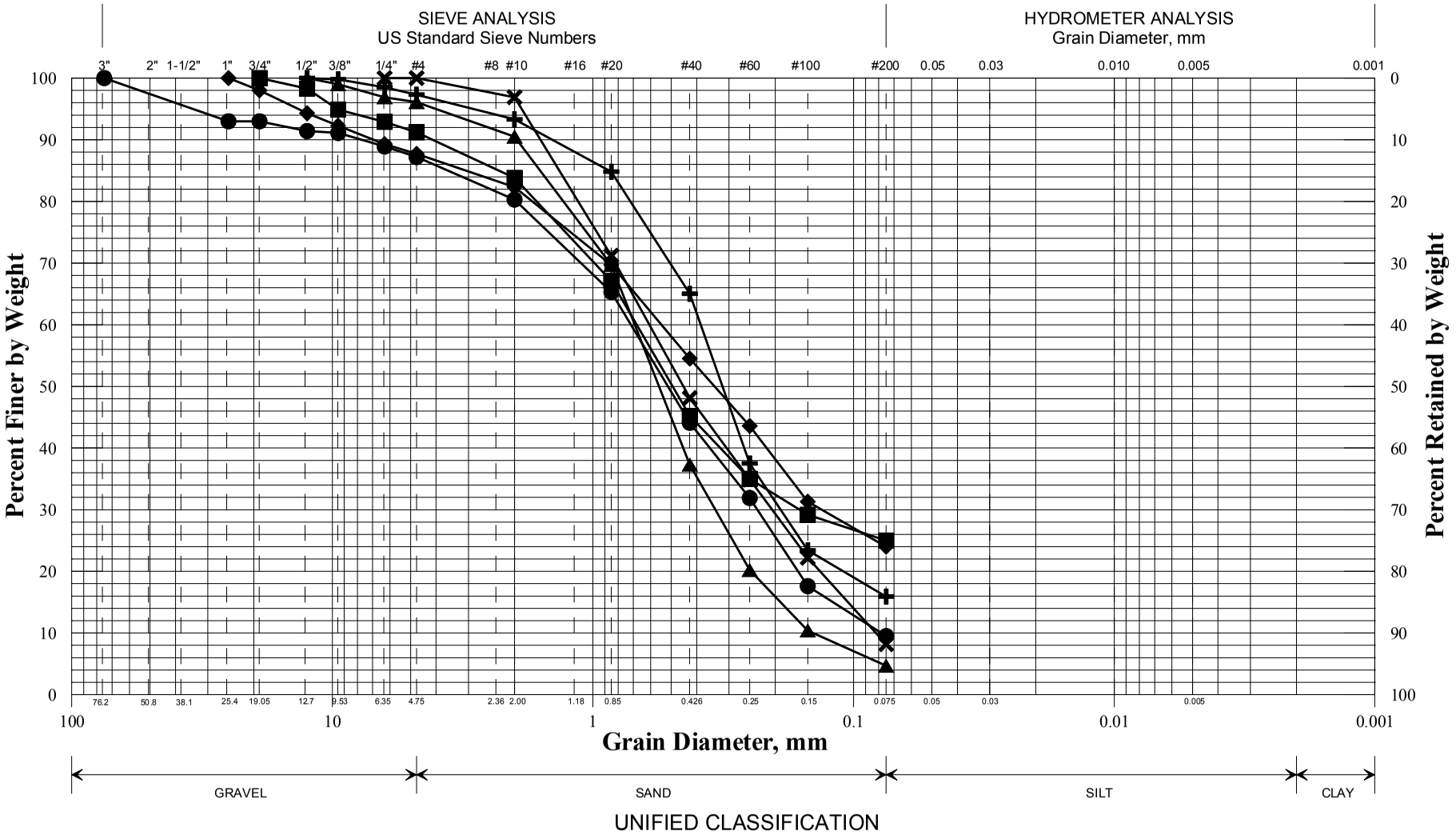
State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-SWSL-102/9D	13+24.2	6.6 LT	37.0-39.0	SAND, little silt, trace gravel.	16.6			
◆	BB-SWSL-102/10D	13+24.2	6.6 LT	42.0-44.0	SAND, some silt, little gravel.	14.9			
■	BB-SWSL-102/11D	13+24.2	6.6 LT	47.0-49.0	SAND, some silt, trace gravel.	17.6			
●	BB-SWSL-102/12D	13+24.2	6.6 LT	52.0-54.0	SAND, little gravel, trace silt.	14.8			
▲	BB-SWSL-102/13D	13+24.2	6.6 LT	57.0-59.0	SAND, trace silt, trace gravel.	19.8			
✕	BB-SWSL-102/14D	13+24.2	6.6 LT	63.0-65.0	SAND, trace silt.	17.9			

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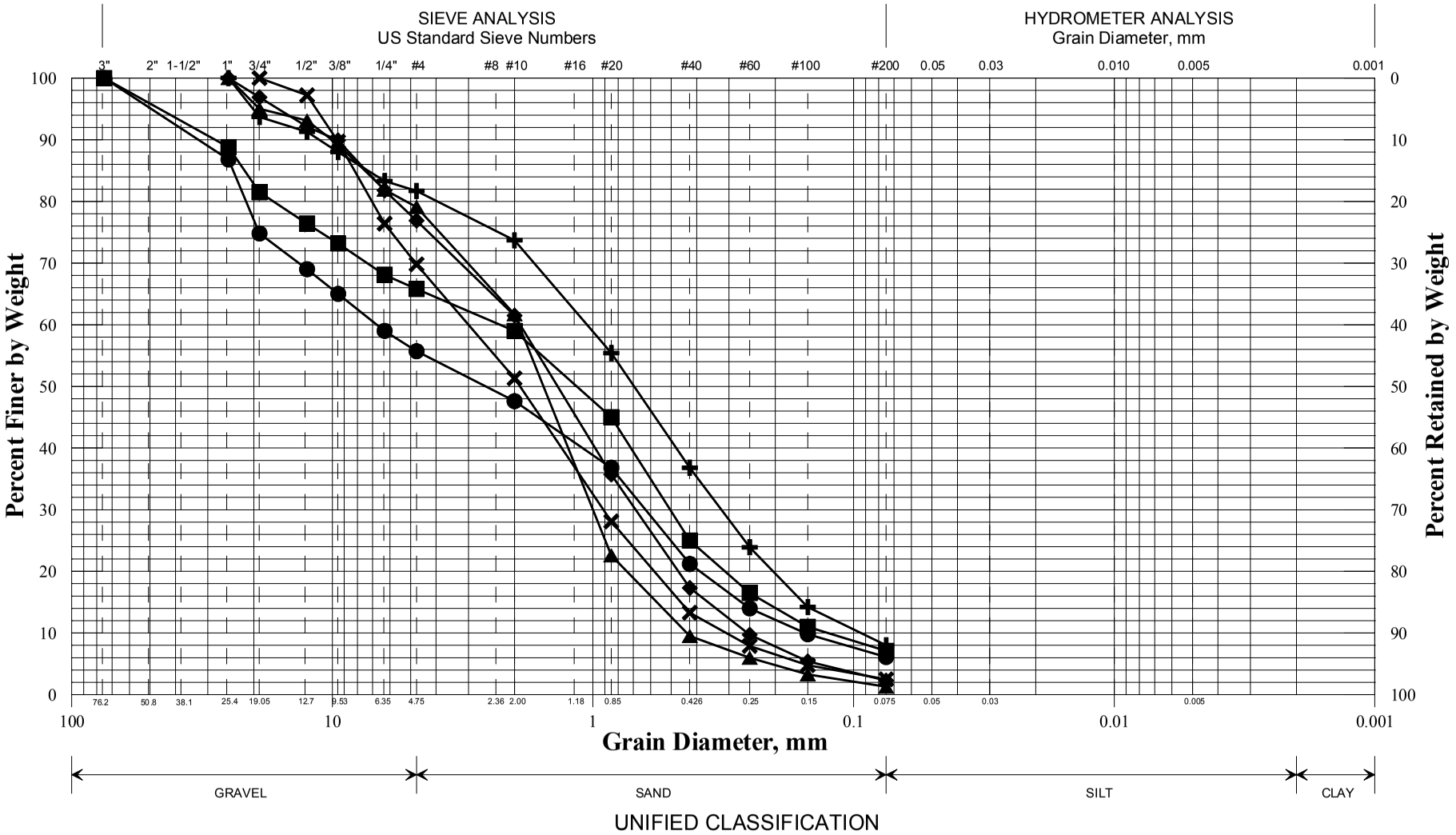
WHITE, TERRY A 5/23/2008

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-SWSL-101/1D	14+06.3	5.8 RT	5.0-7.0	SAND, little gravel, trace silt.	4.8			
◆	BB-SWSL-101/2D	14+06.3	5.8 RT	10.0-12.0	SAND, some gravel, trace silt.	14.9			
■	BB-SWSL-101/3D	14+06.3	5.8 RT	15.0-17.0	SAND, some gravel, trace silt.	11.2			
●	BB-SWSL-101/4D	14+06.3	5.8 RT	20.0-22.0	Gravelly SAND, trace silt.	9.8			
▲	BB-SWSL-101/6D	14+06.3	5.8 RT	30.0-32.0	SAND, some gravel, trace silt.	17.6			
✕	BB-SWSL-101/7D	14+06.3	5.8 RT	40.0-42.0	SAND, some gravel, trace silt.	13.3			

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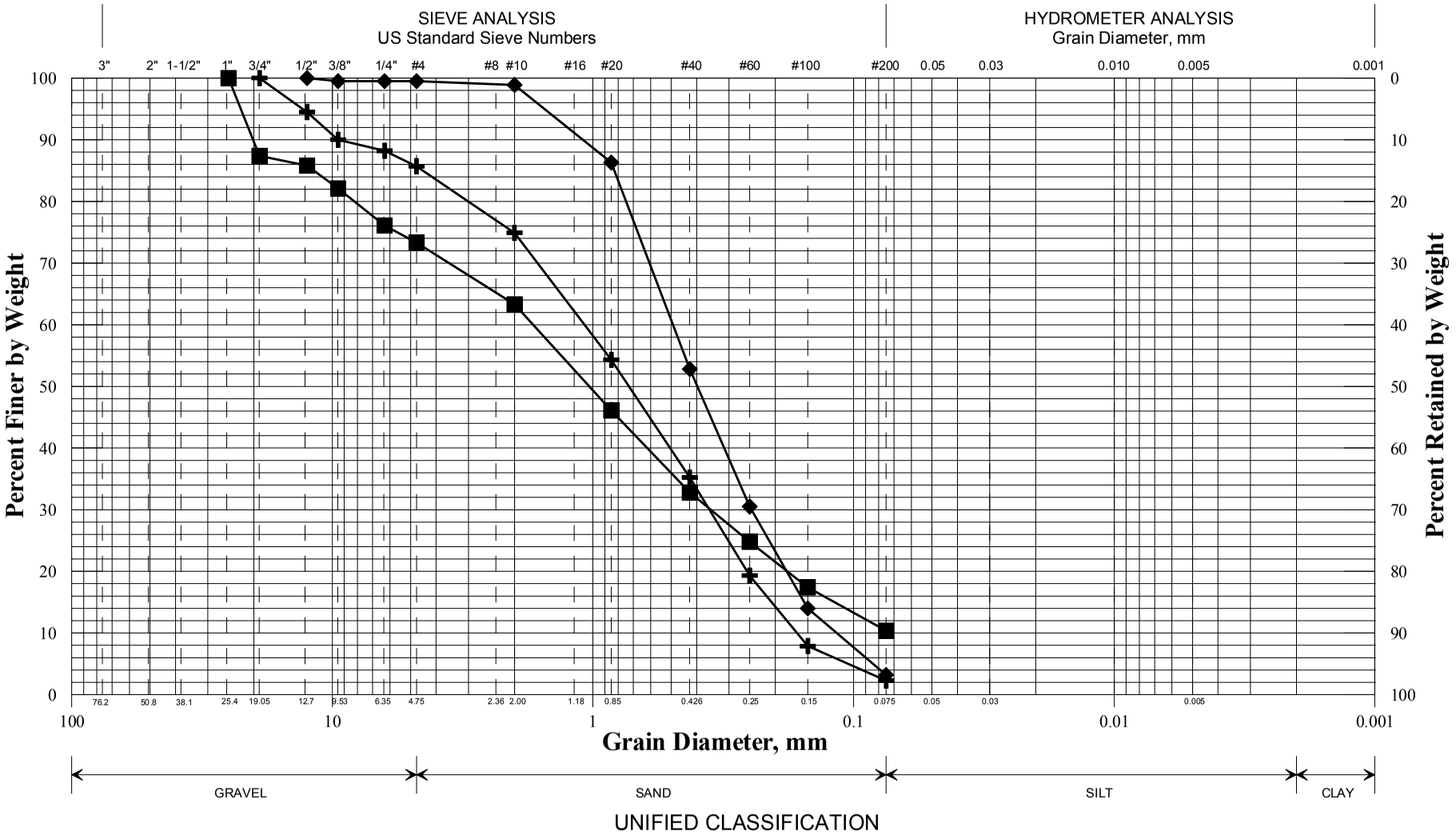
State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-SWSL-101A/2D	14+23.4	5.8 RT	60.0-62.0	SAND, little gravel, trace silt.	15.9			
◆	BB-SWSL-101A/3D	14+23.4	5.8 RT	70.0-72.0	SAND, trace silt, trace gravel.	23.6			
■	BB-SWSL-101A/4D	14+23.4	5.8 RT	80.0-81.3	SAND, some gravel, little silt.	12.2			
●									
▲									
✕									

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015610.00
Town
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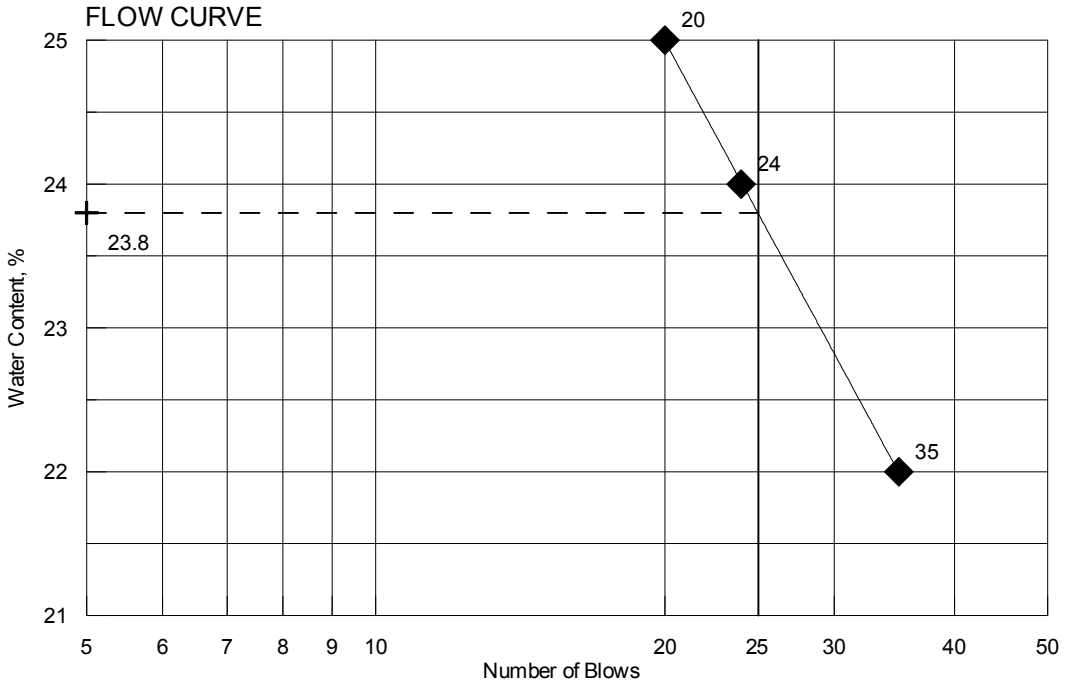
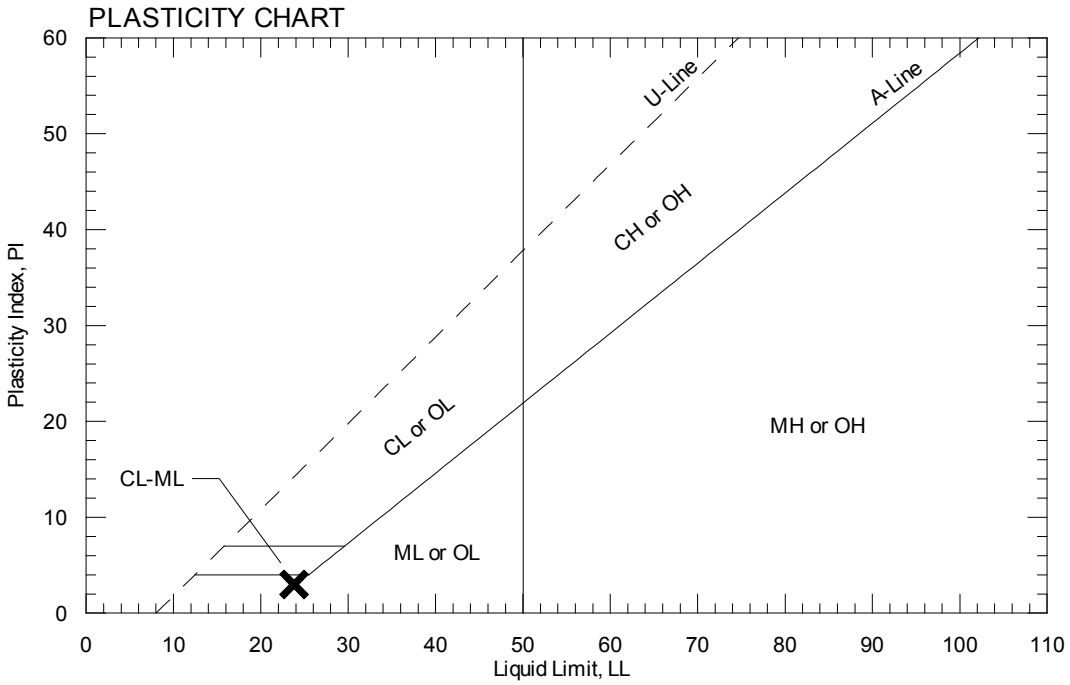
WHITE, TERRY A 5/23/2008

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

TOWN	Standish,Windham	Reference No.	207814
PIN	015610.00	Water Content, %	30.8
Sampled	4/21/2008	Plastic Limit	21
Boring No./Sample No.	BB-SWSL-102/7D	Liquid Limit	24
Station	13+24.2	Plasticity Index	3
Depth	27.0-29.0	Tested By	BBURR





Appendix C

Calculations

Definition of Units:

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $ksf := \frac{kip}{ft^2}$ $tsf := g \cdot \left(\frac{ton}{ft^2}\right)$ $kip := 1000 \cdot lbf$

Abutment Foundations: Integral driven H-piles

Axial Structural Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 4th Edition 2007

Look at the following piles:

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Note: All matrices set up in this order

H-pile Steel area: $A_s := \begin{pmatrix} 15.5 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot in^2$ yield strength: $F_y := 50 \cdot ksi$

Nominal Compressive Resistance $P_n=0.66^{\lambda}*F_y*A_s$: eq. 6.9.4.1-1

Where λ=normalized column slenderness factor

$\lambda=(Kl/r_s\pi)2*F_y/E$ eq. 6.9.4.1-3

$\lambda := 0$ as l = unbraced length = 0

$P_n := 0.66^{\lambda} \cdot F_y \cdot A_s$ $P_n = \begin{pmatrix} 775 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot kip$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

STRENGTH LIMIT STATE:

Factored Resistance:

Driving conditions are assumed "severe" due to the presence of cobbles and boulders.

Strength Limit State Axial Resistance factor for piles in compression under severe driving conditions:

From Article 6.5.4.2 $\phi_c := 0.5$

Factored Compressive Resistance: eq. 6.9.2.1-1

$P_f := \phi_c \cdot P_n$ $P_f = \begin{pmatrix} 388 \\ 535 \\ 653 \\ 860 \end{pmatrix} \cdot kip$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Strength Limit State

SERVICE/EXTREME LIMIT STATES:

Service and Extreme Limit States Axial Resistance

Nominal Compressive Resistance $P_n=0.66^{\lambda}*F_y*A_s$: eq. 6.9.4.1-1

Where λ=normalized column slenderness factor

$\lambda=(Kl/r_s\pi)2*F_y/E$ eq. 6.9.4.1-3

$\lambda := 0$ as l unbraced length is 0

$P_n := 0.66^{\lambda} \cdot F_y \cdot A_s$ $P_n = \begin{pmatrix} 775 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot kip$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Resistance Factors for Service and Extreme Limit States ϕ = 1.0 LRFD 10.5.5.1 and 10.5.8.3

$\phi := 1.0$

Factored Compressive Resistance for Service and Extreme Limit States:

eq. 6.9.2.1-1

$P_f := \phi \cdot P_n$ $P_f = \begin{pmatrix} 775 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot kip$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Service/Extreme Limit States

Geotechnical Resistance

Assume piles will be end bearing on bedrock driven through overlying sand with cobbles and boulders.

Bedrock Type:

Granite RQD ranges from 17 to 100%

Use RQD = 60% and ϕ = 34 to 40 deg (Tomlinson 4th Ed. pg 139)

Axial Geotechnical Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 4th Edition 2007

Look at these piles:

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Note: All matrices set up in this order

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

Pile depth: $d := \begin{pmatrix} 11.78 \\ 13.61 \\ 13.83 \\ 14.21 \end{pmatrix} \cdot in$

Pile width: $b := \begin{pmatrix} 12.045 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot in$

End bearing resistance of piles on bedrock - LRFD code specifies Canadian Geotech Method 1985 (LRFD Table 10.5.5.2.3-1) Canadian Foundation Manual 4th Edition (2006) Section 18.6.3.3.

Average compressive strength of rock core
from AASHTO Standard Spec for Highway Bridges 17 Ed.
Table 4.4.8.1.2B pg 64

q_u for granite compressive strength ranges from 2100 to 49000 psi

use $\sigma_c := 30000 \cdot psi$

Determine K_{sp}: From Canadian Foundation Manual 4th Edition (2006) Section 9.2

Spacing of discontinuities: $c := 36 \cdot in$ Assumed based on rock core

Aperture of discontinuities: $\delta := \frac{1}{64} \cdot in$ joints are tight

Footing width, b: $b = \begin{pmatrix} 12.045 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot in$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

$$K_{sp} := \frac{3 + \frac{c}{b}}{10 \cdot \left(1 + 300 \cdot \frac{\delta}{c}\right)^{0.5}}$$

$$K_{sp} = \begin{pmatrix} 0.5633 \\ 0.5144 \\ 0.5126 \\ 0.5097 \end{pmatrix}$$

K_{sp} includes a factor of safety of 3

Length of rock socket, L_s: $L_s := 0 \cdot in$ Pile is end bearing on rock

Diameter of socket, B_s: $B_s := 1 \cdot ft$

depth factor, d_f: $d_f := 1 + 0.4\left(\frac{L_s}{B_s}\right)$ $d_f = 1$ should be < or = 3 OK

$$q_a := \sigma_c \cdot K_{sp} \cdot d_f \qquad q_a = \begin{pmatrix} 2434 \\ 2222 \\ 2215 \\ 2202 \end{pmatrix} \cdot ksf$$

Nominal Geotechnical Tip Resistance, R_p:

Multiply by 3 to take out FS=3 on K_{sp}

$$R_p := \overrightarrow{\left(3q_a \cdot A_s\right)} \qquad R_p = \begin{pmatrix} 786 \\ 991 \\ 1204 \\ 1578 \end{pmatrix} \cdot kip$$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

STRENGTH LIMIT STATE:

Factored Geotechnical Resistance at Strength Limit State:

Resistance factor, end bearing on rock (Canadian Geotech. Society, 1985 method):

Nominal resistance of Single Pile in Axial Compression - Static Analysis Methods, ϕ_{stat} $\phi_{stat} := 0.45$ LRFD Table 10.5.5.2.3-1

$$R_f := \phi_{stat} \cdot R_p \qquad R_f = \begin{pmatrix} 354 \\ 446 \\ 542 \\ 710 \end{pmatrix} \cdot kip$$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Strength Limit State

SERVICE/EXTREME LIMIT STATES:

Factored Geotechnical Resistance at the Service/Extreme Limit States:

Resistance Factors for Service and Extreme Limit States ϕ = 1.0 LRFD 10.5.5.1 and 10.5.8.3

$$\phi := 1.0$$

$$R_{fse} := \phi \cdot R_p \qquad R_{fse} = \begin{pmatrix} 786 \\ 991 \\ 1204 \\ 1578 \end{pmatrix} \cdot kip$$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Service/Extreme Limit States

DRIVABILITY ANALYSIS

Ref: LRFD Article 10.7.8

For steel piles in compression or tension
$\sigma_{dr} = 0.9 \times \phi_{da} \times f_y$ (eq. 10.7.8-1)

$f_y := 50 \cdot ksi$ yield strength of steel

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

$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y$ $\sigma_{dr} = 45 \cdot ksi$ driving stresses in pile can not exceed 45 ksi

Compute Resistance that can be achieved in a drivability analysis:

The resistance that must be achieved in a drivability analysis will be the maximum applied pile axial load (must be less than the the factored geotechnical resistance from above as this governs) divided by the appropriate resistance factor for wave equation analysis and dynamic test which will be required for construction.

Table 10.5.5.2.3-1 pg 10-38 gives resistance factor for dynamic test, ϕ_{dyn}:

$\phi_{dyn} := 0.65$

There are 5 piles at each abutment. No reduction of Φ_{dyn} is necessary.

Pile Size = 12 x 53

Assume Contractor will use a Delmag D19-42 hammer on highest fuel setting to install 12 x 53 piles

State of Maine Dept. Of Transportation — 23-Oct-2008
Standish- Windham W hite's Bridge — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	35.91	4.48	3.3	8.02	21.07
350.0	39.28	5.84	4.5	8.49	21.99
400.0	42.48	6.44	6.0	9.01	23.31
450.0	44.76	7.06	8.5	9.30	24.10
456.0	45.01	7.11	8.9	9.34	24.17
470.0	45.60	7.25	9.9	9.42	24.43
480.0	45.99	7.33	10.6	9.48	24.57
490.0	46.38	7.36	11.4	9.53	24.70
500.0	46.74	7.44	12.2	9.58	24.84
510.0	47.08	7.52	13.2	9.63	24.99

Limited to driving stress to 45 ksi

Strength Limit State:

$$R_{dr_12x53_factored} := 456 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr_12x53_factored} = 296 \cdot kip$$

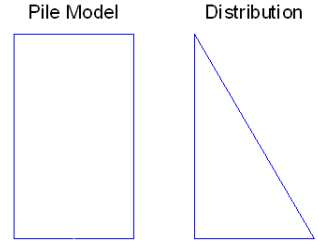
Service and Extreme Limit States: $\phi := 1.0$

$$R_{dr_12x53_servext} := 456 \cdot kip$$

DELMAG D 19-42

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	80.00	ft
Pile Penetration	80.00	ft
Pile Top Area	15.50	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 73

Assume Contractor will use a Delmag D36-32 hammer on third fuel setting to install 14 x 73 piles

State of Maine Dept. Of Transportation — 23-Oct-2008
Standish-Windham White's River Bridge — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
450.0	41.05	4.33	2.5	7.08	39.91
460.0	41.63	4.53	2.6	7.14	40.12
470.0	42.26	4.74	2.7	7.22	40.53
480.0	42.81	4.92	2.8	7.28	40.74
490.0	43.42	5.10	2.9	7.35	41.14
500.0	43.99	5.25	3.0	7.43	41.41
510.0	44.31	5.40	3.1	7.43	41.42
522.0	45.06	5.53	3.2	7.54	41.90
530.0	45.55	5.62	3.3	7.61	42.30
540.0	46.19	5.72	3.4	7.70	42.73

Limit to driving stress to 45 ksi

Strength Limit State:

$$R_{dr_14x73_factored} := 522 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr_14x73_factored} = 339 \cdot kip$$

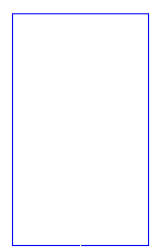
Service and Extreme Limit States: $\phi := 1.0$

$$R_{dr_14x73_servext} := 522 \cdot kip$$

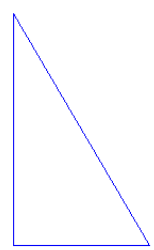
DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	80.00	ft
Pile Penetration	80.00	ft
Pile Top Area	21.40	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 89

Assume Contractor will use a Delmag D36-32 hammer on third fuel setting to install 14 x 73 piles

State of Maine Dept. Of Transportation — 23-Oct-2008
Standish-Windham White's Bridge — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
600.0	44.00	4.52	4.1	7.83	40.87
610.0	44.47	4.58	4.3	7.88	41.21
620.0	44.90	4.63	4.4	7.94	41.51
624.0	45.03	4.65	4.5	7.96	41.54
640.0	45.72	4.74	4.8	8.03	41.97
650.0	46.10	4.79	5.0	8.08	42.16
660.0	46.52	4.92	5.1	8.13	42.51
670.0	46.90	5.05	5.3	8.18	42.71
680.0	47.28	5.17	5.5	8.23	42.93
690.0	47.66	5.31	5.7	8.28	43.25

Limit to driving stress to 45 ksi

Strength Limit State:

$$R_{dr_14x89_factored} := 624 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr_14x89_factored} = 406 \cdot kip$$

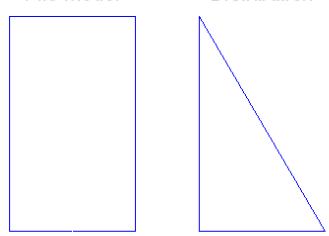
Service and Extreme Limit States: $\phi := 1.0$

$$R_{dr_14x89_servext} := 624 \cdot kip$$

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	80.00	ft
Pile Penetration	80.00	ft
Pile Top Area	26.10	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 117

Assume Contractor will use a Delmag D36-32 hammer on highest fuel setting to install 14 x 73 piles

State of Maine Dept. Of Transportation — 27-Oct-2008
Standish-Windham White's Bridge — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
690.0	43.07	4.74	4.2	8.79	45.66
700.0	43.44	4.85	4.3	8.83	45.89
710.0	43.89	4.96	4.4	8.89	46.26
720.0	44.15	5.05	4.5	8.93	46.34
730.0	44.60	5.15	4.7	8.98	46.73
740.0	44.88	5.24	4.8	9.02	46.83
743.0	45.07	5.27	4.8	9.04	46.99
750.0	45.31	5.33	4.9	9.07	47.19
760.0	45.66	5.39	5.0	9.12	47.42
770.0	46.01	5.43	5.2	9.16	47.66

Limit to driving stress to 45 ksi

Strength Limit State:

$R_{dr_14x117_factored} := 743 \cdot kip \cdot \phi_{dyn}$

$R_{dr_14x117_factored} = 483 \cdot kip$

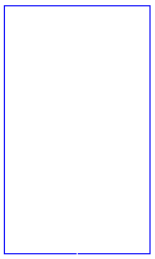
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr_14x117_servext} := 743 \cdot kip$

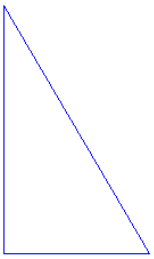
DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	80.00	ft
Pile Penetration	80.00	ft
Pile Top Area	34.40	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Bearing Resistance - Native Soils:

Nominal and factored Bearing Resistance - spread footing on fill soils

Presumptive Bearing Resistance for Service Limit State ONLY

Reference: AASHTO LRFD Bridge Design Specifications Third Edition
Table C10.6.2.6.1-1 "Presumptive Bearing Resistances for Spread Footings at the Service Limit State Modified after US Department of Navy *(1982)*"

Type of Bearing Material:	Consistency In Place:	Bearing Resistance Ordinary Range (ksf)	Recommended Value of Use (ksf)
Coarse to medium sand, with little gravel (SW, SP)	Very Dense	8 to 12	8
	Medium dense to dense	4 to 8	6
	Loose	2 to 6	3

Based on corrected N-values ranging from 5 to 19 - Soils are loose to medium dense

Recommended Value of Use: $6 \cdot ksf = 3 \cdot tsf$

Therefore: $q_{nom} := 3 \cdot tsf$

Resistance factor at the **service limit state** = 1.0 (LRFD Article 10.5.5.1)

$q_{factored_bc} := 3 \cdot tsf$ or $q_{factored_bc} = 6 \cdot ksf$

Nominal and factored Bearing Resistance - spread footing on native soils at the Strength Limit State

Assumptions:

1. Footings will be embedded 6.5 feet for frost protection. $D_f := 6.5 \cdot ft$
2. Assumed parameters for fill soils: (Ref: Bowles 5th Ed Table 3-4)

   Saturated unit weight: $\gamma_s := 125 \cdot pcf$

   Dry unit weight: $\gamma_d := 120 \cdot pcf$

   Internal friction angle: $\phi_{ns} := 32 \cdot deg$

   Undrained shear strength: $c_{ns} := 0 \cdot psf$
3. Use Terzaghi strip equations as L>B
4. Effective stress analysis footing on ϕ-c soil (Bowles 5th Ed. Example 4-1 pg 231)

Depth to Groundwater table: $D_w := 10 \cdot ft$ Based on boring logs

Unit Weight of water: $\gamma_w := 62.4 \cdot pcf$

Look at several footing widths

$$B := \begin{pmatrix} 5 \\ 8 \\ 10 \\ 12 \\ 15 \end{pmatrix} \cdot ft$$

Terzaghi Shape factors from Table 4-1

For a strip footing: $s_c := 1.0$ $s_\gamma := 1.0$

Meyerhof Bearing Capacity Factors - Bowles 5th Ed. table 4-4 pg 223

For ϕ=32 deg

$N_c := 35.47$ $N_q := 23.2$ $N_\gamma := 22.0$

Nominal Bearing Resistance per Terzaghi equation (Bowles 5th Ed. Table 4-1 pg 220)

$$q := D_w \cdot \gamma_d + (D_f - D_w) \cdot (\gamma_s - \gamma_w) \qquad q = 0.4905 \cdot tsf$$

$$q_{nominal} := c_{ns} \cdot N_c \cdot s_c + q \cdot N_q + 0.5(\gamma_s - \gamma_w) B \cdot N_\gamma \cdot s_\gamma$$

$$q_{nominal} = \begin{pmatrix} 13 \\ 14 \\ 15 \\ 16 \\ 17 \end{pmatrix} \cdot tsf$$

Resistance Factor: $\phi_b := 0.45$ AASHTO LRFD Table 10.5.5.2.2-1

$$q_{factored} := q_{nominal} \cdot \phi_b$$

$$q_{factored} = \begin{pmatrix} 6 \\ 6 \\ 7 \\ 7 \\ 7 \end{pmatrix} \cdot tsf$$

Based on these footing widths

$$B := \begin{pmatrix} 5 \\ 8 \\ 10 \\ 12 \\ 15 \end{pmatrix} \cdot ft$$

$$q_{factored} = \begin{pmatrix} 11.8 \\ 12.7 \\ 13.3 \\ 14 \\ 14.9 \end{pmatrix} \cdot ksf$$

At Strength Limit State:

Recommend a limiting factored bearing resistance of 7 tsf or 14 ksf for footings 10 to 15 feet wide on native soils
Recommend a limiting factored bearing resistance of 6 tsf or 12 ksf for footings 9 feet or less wide on native soils

Pipe Pile Pier Bent

Calculate Depth to Fixity for pipe piles:

Soil conditions at boring BB-GWR-102:
68 ft of fill sand, gravel, cobbles and boulders over bedrock

Consider Pile sizes:

24 in diameter 1/2 in wall
26 in diameter 1/2 in wall
28 in diameter 1/2 in wall
30 in diameter 1/2 in wall

24 in diameter 5/8 in wall
26 in diameter 5/8 in wall
28 in diameter 5/8 in wall
30 in diameter 5/8 in wall

Diameter of piles:

$$dia_{steel} := \begin{pmatrix} 24 \\ 26 \\ 28 \\ 30 \end{pmatrix} \cdot in$$

Pipe pile wall thickness:

$$wall_t := \begin{pmatrix} \frac{1}{2} \\ \frac{5}{8} \end{pmatrix} \cdot in$$

Corrosion loss per MaineDOT BDG:

$$cor := \frac{1}{8} in$$

$$dia_{steelcor} := dia_{steel} - 2 \cdot cor \qquad dia_{steelcor} = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} \cdot in \qquad wall_{cor} := wall_t - cor \qquad wall_{cor} = \begin{pmatrix} 0.375 \\ 0.5 \end{pmatrix} \cdot in$$

$$dia_{conccore_0.5} := dia_{steel} - 2 \cdot \frac{1}{2} \cdot in \qquad dia_{conccore_0.5} = \begin{pmatrix} 23 \\ 25 \\ 27 \\ 29 \end{pmatrix} \cdot in$$

Diameter concrete core for 1/2" thick wall

$$dia_{conccore_0.625} := dia_{steel} - 2 \cdot \frac{5}{8} \cdot in \qquad dia_{conccore_0.625} = \begin{pmatrix} 22.75 \\ 24.75 \\ 26.75 \\ 28.75 \end{pmatrix} \cdot in$$

Diameter concrete core for 5/8" thick wall

$$A_{0.5} := \pi \cdot \left(\frac{dia_{steelcor}}{2}\right)^2 - \pi \cdot \left(\frac{dia_{conccore_0.5}}{2}\right)^2 \qquad A_{0.5} = \begin{pmatrix} 27.54 \\ 29.89 \\ 32.25 \\ 34.61 \end{pmatrix} \cdot in^2$$

STEEL AREA FOR 1/2" PILES with 1/8" corrosion loss

$$A_{0.625} := \pi \cdot \left(\frac{dia_{steelcor}}{2}\right)^2 - \pi \cdot \left(\frac{dia_{conccore_0.625}}{2}\right)^2 \qquad A_{0.625} = \begin{pmatrix} 36.52 \\ 39.66 \\ 42.8 \\ 45.95 \end{pmatrix} \cdot in^2$$

STEEL AREA FOR 5/8" PILES with 1/8" corrosion loss

Transformed pile properties of 1/2 inch wall pile:

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

compressive strength of concrete: $f_c := 4.45$ in ksi

Modulus of elasticity of concrete: $E_c := 33000 \cdot wc^{1.5} \cdot \sqrt{f_c} \cdot 1000 \cdot psi$ $\quad E_c = 4044 \cdot ksi$

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

$$n := \frac{E_{steel}}{E_c} \qquad n = 7.17$$

MaineDOT Structural engineers routinely use:

$n := 7.6$

Moment of inertia of concrete core:

$$I_{c_0.5} := \frac{\pi \cdot dia_{conccore_0.5}^{\ 4}}{64} \qquad I_{c_0.5} = \begin{pmatrix} 0.662 \\ 0.925 \\ 1.258 \\ 1.674 \end{pmatrix} ft^4$$

Moment of inertia of steel pipe:

$$I_{s_0.5} := \frac{\pi \cdot \overrightarrow{\left(dia_{steelcor}^{\ 4} - dia_{conccore_0.5}^{\ 4}\right)}}{64} \qquad I_{s_0.5} = \begin{pmatrix} 0.091 \\ 0.116 \\ 0.146 \\ 0.18 \end{pmatrix} ft^4$$

Composite Moment of Inertia:

$$I_{t_0.5} := \overrightarrow{\left(\frac{I_{c_0.5}}{n} + I_{s_0.5}\right)} \qquad I_{t_0.5} = \begin{pmatrix} 0.178 \\ 0.238 \\ 0.311 \\ 0.4 \end{pmatrix} ft^4$$

Transformed Area:

$$A_{conc_0.5} := \pi \cdot \frac{dia_{conccore_0.5}^{\ 2}}{4} \qquad A_{conc_0.5} = \begin{pmatrix} 415.48 \\ 490.87 \\ 572.56 \\ 660.52 \end{pmatrix} \cdot in^2$$

$$A_{t_0.5} := A_{0.5} + \frac{A_{conc_0.5}}{n} \qquad A_{t_0.5} = \begin{pmatrix} 0.571 \\ 0.656 \\ 0.747 \\ 0.844 \end{pmatrix} \cdot ft^2$$

LRFD Eq.10.7.3.13.4-2 for fixity in feet: $1.8*(E_p I_w/n_h)^{0.2}$ (in sands)

E_p Young's modulus of pile in ksi

I_w moment of inertia of pile in ft^4

n_h= rate of increase of soil modulus with depth for sands as specified in Table C10.4.6.3-2 in ksi/ft

Use equation for sands in NCHRP#343 pg 61:

$L_{eq}=L_u+1.8T$ where:

L_{eq} = equivalent free standing length of pile

L_u = unsupported length of pile extending above ground

$T=(E_p*I_p/n_h)^{0.2}$

Rate of increase of soil modulus with depth: for submerged medium dense sand

$$n_h := 0.556 \cdot \frac{ksi}{ft}$$

T parameter:

$$T_{0.5} := \left(\frac{E_{steel} \cdot I_{t_0.5}}{n_h}\right)^{0.2} \qquad T_{0.5} = \begin{pmatrix} 6.22 \\ 6.59 \\ 6.95 \\ 7.31 \end{pmatrix} ft$$

Depth of Fixity:

$$D_{fix_0.5} := 1.8 \cdot T_{0.5}$$

$$D_{fix_0.5} = \begin{pmatrix} 11 \\ 12 \\ 13 \\ 13 \end{pmatrix} ft$$

Depth to fixity for 1/2" wall pipe piles

Check with LRFD Eq. 10.7.3.13.4-2

$$E_{steel} = 29000 \cdot ksi$$

$$I_{t_0.5} = \begin{pmatrix} 0.1779 \\ 0.2377 \\ 0.3113 \\ 0.4003 \end{pmatrix} ft^4 \qquad Check := 1.8 \cdot \left(\frac{E_{steel} \cdot I_{t_0.5}}{n_h}\right)^{0.2} \qquad Check = \begin{pmatrix} 11.19 \\ 11.86 \\ 12.51 \\ 13.16 \end{pmatrix} ft$$

OK

Transformed pile properties of 5/8 inch wall pile:

$$n = 7.6$$

Diameter of concrete core:

$$dia_{conccore_0.625} = \begin{pmatrix} 22.75 \\ 24.75 \\ 26.75 \\ 28.75 \end{pmatrix} \cdot in$$

Diameter concrete core for 5/8" thick wall

Diameter of steel pipe

$$dia_{steelcor} = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} \cdot in$$

Moment of inertia of concrete core:

$$I_{c_0.625} := \frac{\pi \cdot dia_{conccore_0.625}^{4}}{64} \qquad I_{c_0.625} = \begin{pmatrix} 0.634 \\ 0.888 \\ 1.212 \\ 1.617 \end{pmatrix} ft^4$$

Moment of inertia of steel pipe:

$$I_{s_0.625} := \frac{\pi \cdot \overrightarrow{\left(dia_{steelcor}^{4} - dia_{conccore_0.625}^{4}\right)}}{64} \qquad I_{s_0.625} = \begin{pmatrix} 0.119 \\ 0.152 \\ 0.192 \\ 0.237 \end{pmatrix} ft^4$$

Composite Moment of Inertia:

$$I_{t_0.625} := \frac{I_{c_0.625}}{n} + I_{s_0.625} \qquad I_{t_0.625} = \begin{pmatrix} 0.202 \\ 0.269 \\ 0.351 \\ 0.45 \end{pmatrix} ft^4$$

Transformed Area:

$$A_{conc_0.625} := \pi \cdot \frac{dia_{conccore_0.625}^{2}}{4} \qquad A_{conc_0.625} = \begin{pmatrix} 406.49 \\ 481.11 \\ 562 \\ 649.18 \end{pmatrix} \cdot in^2$$

$$A_{t_0.625} := A_{0.625} + \frac{A_{conc_0.625}}{n} \qquad A_{t_0.625} = \begin{pmatrix} 0.625 \\ 0.715 \\ 0.811 \\ 0.912 \end{pmatrix} \cdot ft^2$$

LRFD Eq.10.7.3.13.4-2 for fixity in feet: $1.8*(E_p I_w/n_h)^{0.2}$ (in sands)

E_p Young's modulus of pile in ksi

I_w moment of inertia of pile in ft^4

n_h= rate of increase of soil modulus with depth for sands as specified in Table C10.4.6.3-2 in ksi/ft

Use equation for sands in NCHRP#343 pg 61:

$L_{eq}=L_u+1.8T$ where:

L_{eq} = equivalent free standing length of pile

L_u = unsupported length of pile extending above ground

$T=(E_p*I_p/n_h)^{0.2}$

Rate of increase of soil modulus with depth: for submerged medium dense sand $n_h := 0.556 \cdot \frac{ksi}{ft}$

T parameter:

$$T_{0.625} := \left(\frac{E_{steel} \cdot I_{t_0.625}}{n_h}\right)^{0.2} \qquad T_{0.625} = \begin{pmatrix} 6.38 \\ 6.75 \\ 7.12 \\ 7.48 \end{pmatrix} ft$$

Depth of Fixity:

$$D_{fix_0.625} := 1.8 \cdot T_{0.625}$$

$$D_{fix_0.625} = \begin{pmatrix} 11 \\ 12 \\ 13 \\ 13 \end{pmatrix} ft$$

Depth to fixity for 5/8" wall pipe piles

Check with LRFD Eq. 10.7.3.13.4-2 $\qquad E_{steel} = 29000 \cdot ksi$

$$I_{t_0.625} = \begin{pmatrix} 0.2025 \\ 0.2694 \\ 0.3512 \\ 0.4498 \end{pmatrix} ft^4 \qquad Check := 1.8 \cdot \left(\frac{E_{steel} \cdot I_{t_0.625}}{n_h}\right)^{0.2} \qquad Check = \begin{pmatrix} 11.48 \\ 12.16 \\ 12.82 \\ 13.47 \end{pmatrix} ft$$

OK

Nominal Axial Structural Resistance of pipe piles

Ref: AASHTO LRFD Bridge Design Specifications 4th Edition 2007

Pier - Pipe Pile driven to bedrock, assume driven through cohesionless soils to bedrock (refusal)

Axial pile resistance may be controlled by structural resistance if piles are driven to bedrock.
Check concurrent axial loading and moments with LRFD Equation 6.9.2.2-1 or 6.9.2.2-2.
Use LRFD Equation 6.9.5.1-1 or 6.9.5.1-2 to compute the nominal compressive structural resistance for pipe pile sections.

λ in Equation 6.9.5.1-2 has to be computed for the pipe piles since they 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 reinforcement: $F_{yr} := 60 \cdot ksi$

Assume unsupported length is from bottom of pile cap plus depth to fixity.

Compute λ per 6.9.5.1-3 for composite members:

Effective length factor per LRFD Article 4.6.2.5:

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

$K := 1.0$ Because piles are fixed at the end

Exposed length of pile:

Use exposed pile length - approximately 15 feet

$L_{ex} := 15 \cdot ft$

Unbraced length of column = exposed pile + depth to fixity:

$$L_{UB_0.5} := L_{ex} + D_{fix_0.5} \qquad L_{UB_0.5} = \begin{pmatrix} 26.19 \\ 26.86 \\ 27.51 \\ 28.16 \end{pmatrix} ft$$

$$L_{UB_0.625} := L_{ex} + D_{fix_0.625} \qquad L_{UB_0.625} = \begin{pmatrix} 26.48 \\ 27.16 \\ 27.82 \\ 28.47 \end{pmatrix} ft$$

Longitudinal reinforcement:

Assume longitudinal reinforcement of 12 - #8 bars (1-inch) bars equally spaced for all pile sections.

$$A_r := 12 \cdot \frac{\pi \cdot (1 \cdot in)^2}{4} \qquad A_r = 9.42 \cdot in^2$$

Composite Column Constant per Table 6.9.5.1.1

for tube filled sections: $C1 := 1.0$ $C2 := 0.85$ $C3 := 0.40$

Variable Fe:

$$F_{e_0.5} := F_y + C1 \cdot F_{yr} \cdot \frac{A_r}{A_{0.5}} + C2 \cdot f_c \cdot \frac{A_{conc_0.5}}{A_{0.5}} \qquad F_{e_0.5} = \begin{pmatrix} 116.83 \\ 119.75 \\ 122.9 \\ 126.23 \end{pmatrix} \cdot ksi$$

for 1/2" walls

$$F_{e_0.625} := F_y + C1 \cdot F_{yr} \cdot \frac{A_r}{A_{0.625}} + C2 \cdot f_c \cdot \frac{A_{conc_0.625}}{A_{0.625}} \qquad F_{e_0.625} = \begin{pmatrix} 98.33 \\ 100.5 \\ 102.85 \\ 105.35 \end{pmatrix} \cdot ksi$$

for 5/8" walls

Radius of gyration of both sets of steel sections:

$$r_{s_0.5} := \overrightarrow{\sqrt{\frac{I_{s_0.5}}{A_{0.5}}}} \qquad r_{s_0.5} = \begin{pmatrix} 0.6888 \\ 0.7477 \\ 0.8066 \\ 0.8655 \end{pmatrix} ft$$

for 1/2" walls

$$r_{s_0.625} := \overrightarrow{\sqrt{\frac{I_{s_0.625}}{A_{0.625}}}} \qquad r_{s_0.625} = \begin{pmatrix} 0.6852 \\ 0.7441 \\ 0.803 \\ 0.8619 \end{pmatrix} ft$$

for 5/8" walls

E_e term:

$$E_{e_0.5} := E_{steel} \cdot \left(1 + \frac{C3}{n} \cdot \overrightarrow{\frac{A_{conc_0.5}}{A_{0.5}}}\right) \qquad E_{e_0.5} = \begin{pmatrix} 52028 \\ 54063 \\ 56097 \\ 58132 \end{pmatrix} \cdot ksi$$

for 1/2" walls

$$E_{e_0.625} := E_{steel} \cdot \left(1 + \frac{C3}{n} \cdot \overrightarrow{\frac{A_{conc_0.625}}{A_{0.625}}}\right) \qquad E_{e_0.625} = \begin{pmatrix} 45988 \\ 47514 \\ 49040 \\ 50566 \end{pmatrix} \cdot ksi$$

for 5/8" walls

Lamda (λ) term for composite members LRFD Eq. 6.9.5.1-3

$$\lambda_{0.5} := \overrightarrow{\left[\left(\frac{K \cdot L_{UB_0.5}}{r_{s_0.5} \cdot \pi}\right)^2 \cdot \frac{F_{e_0.5}}{E_{e_0.5}}\right]} \qquad \lambda_{0.5} = \begin{pmatrix} 0.3289 \\ 0.2895 \\ 0.2582 \\ 0.2329 \end{pmatrix}$$

for 1/2" walls

$$\lambda_{0.625} := \overrightarrow{\left[\left(\frac{K \cdot L_{UB_0.625}}{r_{s_0.625} \cdot \pi}\right)^2 \cdot \frac{F_{e_0.625}}{E_{e_0.625}}\right]} \qquad \lambda_{0.625} = \begin{pmatrix} 0.3236 \\ 0.2855 \\ 0.255 \\ 0.2303 \end{pmatrix}$$

for 5/8" walls

Lamda (λ) term for non composite members LRFD Eq. 6.9.4.1-3

$$\lambda_{0.5_tip} := \overrightarrow{\left[\left(\frac{K \cdot L_{UB_0.5}}{r_{s_0.5} \cdot \pi}\right)^2 \cdot \frac{F_y}{E_{steel}}\right]} \qquad \lambda_{0.5_tip} = \begin{pmatrix} 0.2273 \\ 0.2028 \\ 0.1829 \\ 0.1664 \end{pmatrix}$$

for 1/2" walls

$$\lambda_{0.625_tip} := \overrightarrow{\left[\left(\frac{K \cdot L_{UB_0.625}}{r_{s_0.625} \cdot \pi}\right)^2 \cdot \frac{F_y}{E_{steel}}\right]} \qquad \lambda_{0.625_tip} = \begin{pmatrix} 0.2349 \\ 0.2094 \\ 0.1887 \\ 0.1715 \end{pmatrix}$$

for 5/8" walls

Nominal Axial Structural Resistance of 1/2-inch wall

Since λ<2.25 use LRFD Eq. 6.9.5.1-1

$$P_{n_0.5} := \overrightarrow{\left(0.66^{\lambda_{0.5}} \cdot F_{e_0.5} \cdot A_{0.5}\right)} \qquad P_{n_0.5} = \begin{pmatrix} 2806 \\ 3174 \\ 3560 \\ 3966 \end{pmatrix} \cdot kip$$

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

$$P_{n_0.5tip} := \overrightarrow{\left(0.66^{\lambda_{0.5_tip}} \cdot F_y \cdot A_{0.5}\right)} \qquad P_{n_0.5tip} = \begin{pmatrix} 1128 \\ 1237 \\ 1345 \\ 1453 \end{pmatrix} \cdot kip$$

USE THIS FOR DESIGN

for 1/2" walls

Nominal Axial Structural Resistance of 5/8-inch wall

Since λ<2.25 use LRFD Eq. 6.9.5.1-1

$$P_{n_0.625} := \overrightarrow{\left(0.66^{\lambda_{0.625}} \cdot F_{e_0.625} \cdot A_{0.625}\right)} \qquad P_{n_0.625} = \begin{pmatrix} 3139 \\ 3540 \\ 3960 \\ 4399 \end{pmatrix} \cdot kip$$

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

$$P_{n_0.625tip} := \overrightarrow{\left(0.66^{\lambda_{0.625_tip}} \cdot F_y \cdot A_{0.625}\right)} \qquad P_{n_0.625tip} = \begin{pmatrix} 1491 \\ 1636 \\ 1781 \\ 1925 \end{pmatrix} \cdot kip$$

USE THIS FOR DESIGN

for 5/8" walls

<u>Factored Axial Structural Resistance of a single Pipe Pile:</u>

Strength limit state resistance factor for pipe piles in compression, severe driving conditions - LRFD 6.5.4.2 $\qquad \phi_c := 0.6$

Factored Structural Resistance (Pr):

$$P_{r_0.5} := \phi_c \cdot P_{n_0.5} \qquad P_{r_0.5} = \begin{pmatrix} 1684 \\ 1904 \\ 2136 \\ 2379 \end{pmatrix} \cdot kip$$

for 1/2" walls

$$P_{r_0.625} := \phi_c \cdot P_{n_0.625} \qquad P_{r_0.625} = \begin{pmatrix} 1884 \\ 2124 \\ 2376 \\ 2639 \end{pmatrix} \cdot kip$$

for 5/8" walls

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

$$P_{r_0.5tip} := \phi_c \cdot P_{n_0.5tip} \qquad P_{r_0.5tip} = \begin{pmatrix} 677 \\ 742 \\ 807 \\ 872 \end{pmatrix} \cdot kip$$

for 1/2" walls

$$P_{r_0.625tip} := \phi_c \cdot P_{n_0.625tip} \qquad P_{r_0.625tip} = \begin{pmatrix} 894 \\ 982 \\ 1069 \\ 1155 \end{pmatrix} \cdot kip$$

for 5/8" walls

Service and Extreme Limit States Axial Structural Resistance

Resistance Factors for Service and Extreme Limit States ϕ = 1.0 LRFD 10.5.5.1 and 10.5.8.3

$$\phi := 1.0$$

Factored Compressive Resistance for Service and Extreme Limit States:

$$P_{_0.5tipf} := \phi \cdot P_{n_0.5tip} \qquad P_{_0.5tipf} = \begin{pmatrix} 1128 \\ 1237 \\ 1345 \\ 1453 \end{pmatrix} \cdot kip$$

for 1/2" walls

$$P_{_0.625tipf} := \phi \cdot P_{n_0.625tip} \qquad P_{_0.625tipf} = \begin{pmatrix} 1491 \\ 1636 \\ 1781 \\ 1925 \end{pmatrix} \cdot kip$$

for 5/8" walls

COMPUTE GEOTECHNICAL RESISTANCE OF PIPE PILES

Pipe pile capacity based on steel shell end bearing on bedrock - driven through sand, gravel, cobbles and boulders.

Pipe piles evaluated:
24 in diameter 1/2 in wall
26 in diameter 1/2 in wall
28 in diameter 1/2 in wall
30 in diameter 1/2 in wall

RQD of bedrock in channel center location= 63 to 100%.
Bedrock is identified as: GRANITE

24 in diameter 5/8 in wall
26 in diameter 5/8 in wall
28 in diameter 5/8 in wall
30 in diameter 5/8 in wall

Uniaxial Compressive Strength of GRANITE from AASHTO Standard Spec for Highway Bridges 17th Ed. Table 4.4.8.1.2B pg 64
Granite 2100 - 49000 psi Use 30000 psi

$Q_{uc} := 30000 \cdot psi$

Reference: Pile Design and Construction Practice, M.J. Tomlinson, Fourth Edition pg 139
Friction angle = 34 to 40 degrees

$\phi_1 := 38 \cdot deg$

Diameter of piles:

$$dia_{steel} := \begin{pmatrix} 24 \\ 26 \\ 28 \\ 30 \end{pmatrix} \cdot in$$

Pipe pile wall thickness:

$$wall_t := \begin{pmatrix} \frac{1}{2} \\ \frac{5}{8} \end{pmatrix} \cdot in$$

Corrosion loss per MaineDOT BDG:

$$cor := \frac{1}{8} in$$

$$A_{0.5} = \begin{pmatrix} 27.54 \\ 29.89 \\ 32.25 \\ 34.61 \end{pmatrix} \cdot in^2$$

STEEL AREA FOR 1/2" PILES with 1/8" corrosion loss

$$A_{0.625} = \begin{pmatrix} 36.52 \\ 39.66 \\ 42.8 \\ 45.95 \end{pmatrix} \cdot in^2$$

STEEL AREA FOR 5/8" PILES with 1/8" corrosion loss

LRFD Code specifies Canadian Geotechnical Society Method 1985 for resistance determination of end bearing piles on bedrock. (LRFD Table 10.5.5.2.3-1)
Use Canadian Foundation Manual 4th Edition 2006 Section 18.6.3.3.

Determine K_{sp}: From Canadian Foundation Manual 4th Edition (2006) Section 9.2

Spacing of discontinuities: $c := 36 \cdot in$ Assumed based on rock core

Aperture of discontinuities: $\delta := \frac{1}{64} \cdot in$ joints are tight

Footing width, b:

$$b := dia_{steelcor} \qquad b = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} \cdot in$$

$$K_{sp} := \frac{3 + \frac{c}{b}}{10 \cdot \left(1 + 300 \cdot \frac{\delta}{c}\right)^{0.5}}$$

K_{sp} includes a factor of safety of 3

Length of rock socket, L_s: $L_s := 0 \cdot in$ Pile is end bearing on rock

Diameter of socket, B_s: $B_s := 0 \cdot ft$

depth factor, d_f: $d_f := 1 + 0.4\left(\frac{L_s}{B_s}\right)$ $d_f = 1$ should be < or = 3 OK

$$q_{aA} := Q_{uc} \cdot K_{sp} \cdot d_f \qquad q_{aA} = \begin{pmatrix} 1835 \\ 1787 \\ 1746 \\ 1711 \end{pmatrix} \cdot ksf$$

Nominal Geotechnical Tip Resistance, R_p:

Multiply by 3 to take out FS=3 on K_{sp}

$$R_{pA0.5} := \overrightarrow{\left(3q_{aA} \cdot A_{0.5}\right)} \qquad R_{pA0.5} = \begin{pmatrix} 1053 \\ 1113 \\ 1173 \\ 1233 \end{pmatrix} \cdot kip$$

for 1/2" walls

$$R_{pA0.625} := \overrightarrow{\left(3q_{aA} \cdot A_{0.625}\right)} \qquad R_{pA0.625} = \begin{pmatrix} 1396 \\ 1477 \\ 1557 \\ 1638 \end{pmatrix} \cdot kip$$

for 5/8" walls

STRENGTH LIMIT STATE:

Factored Geotechnical Resistance at Strength Limit State:

Resistance factor, end bearing on rock (Canadian Geotech. Society, 1985 method):

Nominal resistance of Single Pile in Axial Compression - Static Analysis Methods, ϕ_{stat} $\phi_{stat} := 0.45$ LRFD Table 10.5.5.2.3-1

$R_{f0.5} := \phi_{stat} \cdot R_{pA0.5}$ $R_{f0.5} = \begin{pmatrix} 474 \\ 501 \\ 528 \\ 555 \end{pmatrix} \cdot kip$ Strength Limit State for 1/2" walls

$R_{f0.625} := \phi_{stat} \cdot R_{pA0.625}$ $R_{f0.625} = \begin{pmatrix} 628 \\ 665 \\ 701 \\ 737 \end{pmatrix} \cdot kip$ Strength Limit State for 5/8" walls

SERVICE/EXTREME LIMIT STATES:

Factored Geotechnical Resistance at the Service/Extreme Limit States:

Resistance Factors for Service and Extreme Limit States ϕ = 1.0 LRFD 10.5.5.1 and 10.5.8.3

$\phi := 1.0$

$R_{fse0.5} := \phi \cdot R_{pA0.5}$ $R_{fse0.5} = \begin{pmatrix} 1053 \\ 1113 \\ 1173 \\ 1233 \end{pmatrix} \cdot kip$ Service/Extreme Limit States for 1/2" walls

$R_{fse0.625} := \phi \cdot R_{pA0.625}$ $R_{fse0.625} = \begin{pmatrix} 1396 \\ 1477 \\ 1557 \\ 1638 \end{pmatrix} \cdot kip$ Service/Extreme Limit States for 5/8" walls

DRIVABILITY ANALYSIS

Ref: LRFD Article 10.7.8

For steel piles in compression or tension
$\sigma_{dr} = 0.9 \times \phi_{da} \times f_y$ (eq. 10.7.8-1)

$f_y := 45 \cdot ksi$ yield strength of steel

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

$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y$ $\sigma_{dr} = 40.5 \cdot ksi$ driving stresses in pile cannot exceed 40 ksi

Compute Resistance that can be achieved in a drivability analysis:

The resistance that must be achieved in a drivability analysis will be the maximum applied pile axial load (must be less than the the factored geotechnical resistance from above as this governs) divided by the appropriate resistance factor for wave equation analysis and dynamic test which will be required for construction.

Table 10.5.5.2.3-1 pg 10-38 gives resistance factor for dynamic test, ϕ_{dyn}:

$\phi_{dyn} := 0.65$

5 piles in pier, no reduction to Φ_{dyn} necessary.

Assume Contractor will use a Delmag D 36-32 hammer on the third fuel setting to install:
24-in Dia. pile with 1/2-in wall thickness

Pile Size = 24"D x 1/2"W

Unsupported length = unbraced length 15 ft + depth to fixity 11 ft = 26 ft.

State of Maine Dept. Of Transportation 27-Oct-2008
Standish-Windham White's Bridge PipePile GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
500.0	38.71	3.53	2.8	7.32	37.14
505.0	39.01	3.59	2.8	7.35	37.17
510.0	39.26	3.65	2.9	7.39	37.37
515.0	39.50	3.70	2.9	7.41	37.45
520.0	39.78	3.75	2.9	7.44	37.51
525.0	40.04	3.79	3.0	7.47	37.71
530.0	40.29	3.82	3.0	7.50	37.77
535.0	40.50	3.86	3.1	7.53	37.85
540.0	40.77	3.88	3.1	7.56	38.07
545.0	41.03	3.92	3.2	7.59	38.14

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	68.00	ft
Pile Penetration	42.00	ft
Pile Top Area	27.54	in2

Limit driving stress to 40 ksi

Strength Limit State:

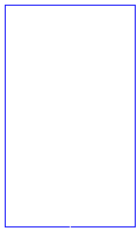
$$R_{dr_24x0.5_factored} := 525 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr_24x0.5_factored} = 341 \cdot kip$$

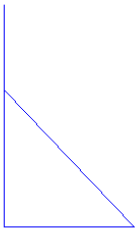
Service and Extreme Limit States: $\phi := 1.0$

$$R_{dr_24x0.5_servext} := 525 \cdot kip$$

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Assume Contractor will use a Delmag D 36-32 hammer on the third fuel setting to install:
26-in Dia. pile with 1/2-in wall thickness

Pile Size = 26"D x 1/2"W

Unsupported length = unbraced length 15 ft + depth to fixity 12 ft = 27 ft.

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Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
550.0	39.35	3.20	3.2	7.56	37.30
555.0	39.54	3.23	3.3	7.60	37.41
560.0	39.84	3.27	3.3	7.63	37.53
565.0	40.05	3.30	3.4	7.66	37.54
570.0	40.27	3.35	3.4	7.68	37.71
575.0	40.26	3.40	3.5	7.63	37.38
580.0	40.49	3.44	3.5	7.66	37.51
585.0	40.73	3.48	3.6	7.70	37.60
590.0	41.04	3.53	3.6	7.74	37.81
595.0	41.28	3.58	3.7	7.77	37.95

Limit driving stress to 40 ksi

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	68.00	ft
Pile Penetration	41.00	ft
Pile Top Area	29.89	in2

Strength Limit State:

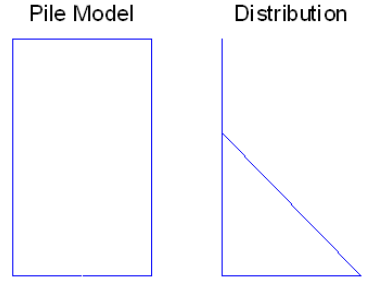
$$R_{dr_26x0.5_factored} := 565 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr_26x0.5_factored} = 367 \cdot kip$$

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

$$R_{dr_26x0.5_servext} := 565 \cdot kip$$

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Assume Contractor will use a Delmag D 36-32 hammer on the third fuel setting to install:
28-in Dia. pile with 1/2-in wall thickness

Pile Size = 28"D x 1/2"W

Unsupported length = unbraced length 15 ft + depth to fixity 13 ft = 28 ft.

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Standish-Windham White's Bridge PipePile — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
600.0	39.59	3.10	3.8	7.74	37.06
605.0	39.84	3.15	3.8	7.77	37.33
609.0	40.08	3.19	3.8	7.80	37.47
615.0	40.38	3.24	3.9	7.85	37.63
620.0	40.62	3.30	3.9	7.88	37.90
625.0	40.83	3.35	4.0	7.91	38.02
630.0	41.10	3.40	4.0	7.95	38.28
635.0	41.31	3.44	4.1	7.99	38.35
640.0	41.54	3.49	4.2	8.02	38.60
645.0	41.77	3.53	4.2	8.05	38.71

Limit driving stress to 40 ksi

Strength Limit State:

$$R_{dr_28x0.5_factored} := 609 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr_28x0.5_factored} = 396 \cdot kip$$

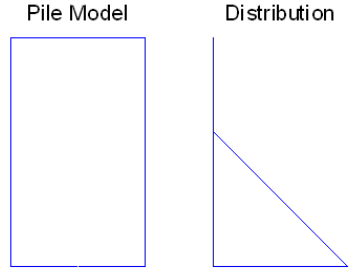
Service and Extreme Limit States: $\phi := 1.0$

$$R_{dr_28x0.5_servext} := 609 \cdot kip$$

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	68.00	ft
Pile Penetration	40.00	ft
Pile Top Area	32.25	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Assume Contractor will use a Delmag D 36-32 hammer on the third fuel setting to install:
30-in Dia. pile with 1/2-in wall thickness

Pile Size = 30"D x 1/2"W

Unsupported length = unbraced length 15 ft + depth to fixity 13 ft = 28 ft.

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Standish-Windham White's Bridge PipePile GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
640.0	39.70	3.13	4.2	7.95	37.68
646.0	40.00	3.18	4.2	7.99	37.99
650.0	40.21	3.21	4.3	8.01	37.95
655.0	40.39	3.24	4.3	8.05	38.14
660.0	40.63	3.29	4.4	8.08	38.40
665.0	40.90	3.36	4.4	8.11	38.59
670.0	41.07	3.41	4.5	8.15	38.66
675.0	41.30	3.52	4.6	8.18	38.89
680.0	41.53	3.60	4.6	8.21	39.11
685.0	41.77	3.65	4.7	8.24	39.16

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	68.00	ft
Pile Penetration	40.00	ft
Pile Top Area	34.61	in2

Limit driving stress to 40 ksi

Strength Limit State:

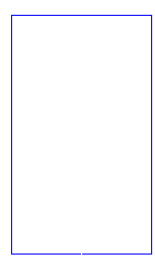
$$R_{dr_30x0.5_factored} := 646 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr_30x0.5_factored} = 420 \cdot kip$$

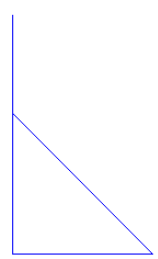
Service and Extreme Limit States: $\phi := 1.0$

$$R_{dr_30x0.5_servext} := 646 \cdot kip$$

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Assume Contractor will use a Delmag D 36-32 hammer on the third fuel setting to install:
24-in Dia. pile with 5/8-in wall thickness

Pile Size = 24"D x 5/8"W

Unsupported length = unbraced length 15 ft + depth to fixity 11 ft = 26 ft.

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Standish-Windham White's Bridge PipePile — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
650.0	38.85	3.08	4.3	7.96	37.29
655.0	39.06	3.16	4.3	7.99	37.47
660.0	39.24	3.21	4.4	8.02	37.54
665.0	39.45	3.28	4.5	8.05	37.71
670.0	39.63	3.34	4.5	8.08	37.89
675.0	39.91	3.41	4.6	8.12	38.06
680.0	40.09	3.49	4.6	8.14	38.30
685.0	40.28	3.53	4.7	8.17	38.33
690.0	40.52	3.59	4.7	8.21	38.49
695.0	40.69	3.67	4.8	8.23	38.74

Limit driving stress to 40 ksi

Strength Limit State:

$$R_{dr_24x0.625_factored} := 680 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr_24x0.625_factored} = 442 \cdot kip$$

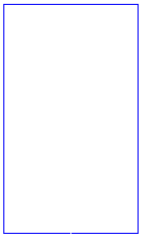
Service and Extreme Limit States: $\phi := 1.0$

$$R_{dr_24x0.625_servext} := 680 \cdot kip$$

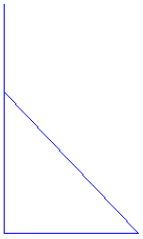
DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	68.00	ft
Pile Penetration	42.00	ft
Pile Top Area	36.52	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Assume Contractor will use a Delmag D 36-32 hammer on the third fuel setting to install:
26-in Dia. pile with 5/8-in wall thickness

Pile Size = 26"D x 5/8"W

Unsupported length = unbraced length 15 ft + depth to fixity 12 ft = 27 ft.

State of Maine Dept. Of Transportation 27-Oct-2008
Standish-Windham White's Bridge PipePile GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
720.0	39.52	3.52	5.0	8.27	38.25
725.0	39.70	3.55	5.1	8.30	38.28
730.0	39.65	3.60	5.2	8.23	38.03
735.0	39.84	3.64	5.3	8.26	38.20
743.0	40.03	3.69	5.4	8.29	38.23
745.0	40.14	3.69	5.4	8.29	38.28
750.0	40.32	3.73	5.5	8.31	38.47
755.0	40.47	3.76	5.5	8.33	38.53
760.0	40.61	3.78	5.6	8.35	38.55
765.0	40.70	3.81	5.7	8.36	38.64

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	68.00	ft
Pile Penetration	41.00	ft
Pile Top Area	39.66	in2

Limit driving stress to 40 ksi

Strength Limit State:

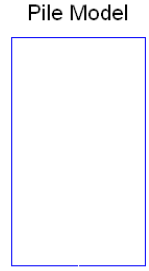
$$R_{dr_26x0.625_factored} := 743 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr_26x0.625_factored} = 483 \cdot kip$$

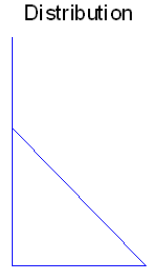
Service and Extreme Limit States: $\phi := 1.0$

$$R_{dr_26x0.625_servext} := 743 \cdot kip$$

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Assume Contractor will use a Delmag D 36-32 hammer on the third fuel setting to install:
28-in Dia. pile with 5/8-in wall thickness

Pile Size = 28"D x 5/8"W

Unsupported length = unbraced length 15 ft + depth to fixity 13 ft = 28 ft.

State of Maine Dept. Of Transportation — 27-Oct-2008
Standish-Windham White's Bridge PipePile — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
800.0	39.87	3.33	6.1	8.41	38.23
805.0	39.98	3.34	6.1	8.42	38.24
808.0	40.06	3.34	6.2	8.43	38.23
815.0	40.23	3.36	6.3	8.46	38.39
820.0	40.42	3.38	6.3	8.47	38.57
825.0	40.53	3.39	6.4	8.48	38.60
830.0	40.64	3.40	6.5	8.49	38.62
835.0	40.82	3.41	6.6	8.51	38.76
840.0	40.96	3.41	6.7	8.53	38.77
845.0	41.10	3.43	6.7	8.54	38.94

Limit driving stress to 40 ksi

Strength Limit State:

$$R_{dr_28x0.625_factored} := 808 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr_28x0.625_factored} = 525 \cdot kip$$

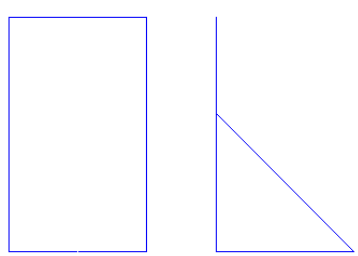
Service and Extreme Limit States: $\phi := 1.0$

$$R_{dr_28x0.625_servext} := 808 \cdot kip$$

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	68.00	ft
Pile Penetration	40.00	ft
Pile Top Area	42.80	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Assume Contractor will use a Delmag D 36-32 hammer on the third fuel setting to install:
30-in Dia. pile with 5/8-in wall thickness

Pile Size = 30"D x 5/8"W

Unsupported length = unbraced length 15 ft + depth to fixity 13 ft = 28 ft.

State of Maine Dept. Of Transportation 27-Oct-2008
Standish-Windham White's Bridge PipePile GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
860.0	39.64	2.80	6.8	8.49	38.09
865.0	39.73	2.81	6.9	8.51	38.05
870.0	39.87	2.81	7.0	8.53	38.17
877.0	40.07	2.81	7.1	8.55	38.31
880.0	40.15	2.82	7.1	8.56	38.41
885.0	40.27	2.83	7.2	8.58	38.37
890.0	40.37	2.85	7.3	8.59	38.52
895.0	40.52	2.88	7.4	8.61	38.63
900.0	40.64	2.89	7.5	8.62	38.63
905.0	40.78	2.92	7.5	8.64	38.74

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	68.00	ft
Pile Penetration	40.00	ft
Pile Top Area	45.95	in2

Limit driving stress to 40 ksi

Strength Limit State:

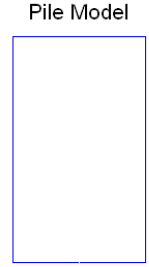
$$R_{dr_30x0.625_factored} := 877 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr_30x0.625_factored} = 570 \cdot kip$$

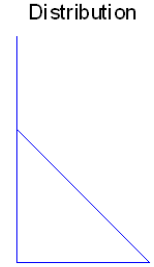
Service and Extreme Limit States: $\phi := 1.0$

$$R_{dr_30x0.625_servext} := 877 \cdot kip$$

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Abutment and Wingwall Passive and Active Earth Pressure:

For cases where interface friction is considered (for gravity structures) use Coulomb Theory

Coulomb Theory - Passive Earth Pressure from Maine DOT Bridge Design Guide Section 3.6.6 pg 3-8

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

Angle of internal soil friction: $\phi := 32 \cdot deg$

Friction angle between fill and wall:
From LRFD Table 3.11.5.3-1 range from 17 to 22 $\delta := 20 \cdot deg$

Angle of backfill to the horizontal $\beta := 0 \cdot deg$

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

$K_p = 6.89$

Rankine Theory - Passive Earth Pressure from Bowles 5th Edition Section 11-5 pg 602

Angle of backfill to the horizontal $\beta := 0 \cdot deg$

Angle of internal soil friction: $\phi := 32 \cdot deg$

$$K_{p_rank} := \frac{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}$$

$K_{p_rank} = 3.25$

Bowles does not recommend the use of the Rankine Method for K_p when β>0.

Rankine Theory - Active Earth Pressure from Maine DOT Bridge Design Guide Section 3.6.5.2 pg 3-7

For a horizontal backfill surface:

$\phi := 32 \cdot deg$

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

$K_a = 0.307$

Frost Protection:

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

From the Design Freezing Index Map:
Standish - Windham, Maine
DFI = 1330 degree-days

From the lab testing: soils are coarse grained assume a water content = ~10%

From Table 5-1 MaineDOT BDG for Design Freezing Index of 1330 frost penetration = 77.2 inches

$Frost_depth := 77.2in$ $\quad$ $Frost_depth = 6.4333 \cdot ft$

Note: The final depth of footing embedment may be controlled by the scour susceptibility of the foundation material and may, in fact, be deeper than the depth required for frost protection.

Method 2 - Check Frost Depth using Modberg Software

Closest Station is Bridgton

--- ModBerg Results ---

Project Location: Bridgton 3 NW, Maine

Air Design Freezing Index = 1600 F-days
N-Factor = 0.80
Surface Design Freezing Index = 1280 F-days
Mean Annual Temperature = 43.9 deg F
Design Length of Freezing Season = 133 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	77.7	10.0	125.0	28	34	2.0	1.6	1,800

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

****
Total Depth of Frost Penetration = 6.47 ft = 77.7 in.
****

$Frost_depth_{modberg} := 77.7 \cdot in$ $\quad$ $Frost_depth_{modberg} = 6.475\,ft$

Use Frost Depth = 6.5 feet for design

Seismic:

Standish-Windham White's Bridge PIN 15610.00
Date and Time: 10/28/2008 4:31:32 PM

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
State - Maine
Zip Code - 04084
Zip Code Latitude = 43.787000
Zip Code Longitude = -070.547600
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.095	PGA - Site Class B
0.2	0.186	Ss - Site Class B
1.0	0.047	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
State - Maine
Zip Code - 04084
Zip Code Latitude = 43.787000
Zip Code Longitude = -070.547600
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class C - Fpga = 1.20, Fa = 1.20, Fv = 1.70
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.114	As - Site Class C
0.2	0.223	SDs - Site Class C
1.0	0.079	SD1 - Site Class C

Seismic Design Parameters for
2007 AASHTO Seismic Design Guidelines

Purpose - The ground motion parameters obtained in this analysis are for use with the design procedures described in AASHTO Guidelines for the Seismic Design of Highway Bridges (2007) The user may calculate seismic design parameters and response spectra (both for period and displacement), for Site Class A through E.

Description - This program allows the user to obtain seismic design parameters for sites in the 50 states of the United States, Puerto Rico and the U.S. Virgin Islands. In most cases the user may perform an analysis for a site by specifying location by either latitude-longitude (recommended) or zip code. However, locations in Puerto and the Virgin Islands may only be specified by latitude-longitude.

Ground motion maps are included in PDF format. These maps may be opened using a map viewer that is part of the software package.

Data - The 2007 AASHTO maps are based on 5% in 50 year probabilistic data from the U.S. Geological Survey data sets for the following regions: 48 conterminous states (2002), Alaska (2006), Hawaii (1998), Puerto Rico and the Virgin Islands (2003). These were the most recent data available at the time of preparation of the AASHTO maps. The AASHTO maps are labelled with a probability of exceedance of 7% in 75 years which is approximately equal to the 5% in 50 year data.

Disclaimer - Correct application of the data obtained from the use of this program and/or maps is the responsibility of the user. This software is not a substitute for technical knowledge of seismic design and/or analysis.