MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

MADAWASKA BRIDGE
MAIN STREET OVER LITTLE MADAWASKA RIVER
STOCKHOLM, MAINE

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GEOTECHNICAL DESIGN SUMMARY

The purpose of this Geotechnical Design Report is to present subsurface information and provide geotechnical design recommendations for the replacement of Madawaska Bridge which carries Main Street over Little Madawaska River in Stockholm, Maine. The proposed replacement structure will be a 100-foot single-span weathering steel beam superstructure founded on H-pile supported integral abutments. The new bridge alignment will closely match the horizontal alignment of the existing bridge. The following design recommendations are discussed in detail in this report:

Integral Abutment H-piles – H-piles for support of the integral abutments should be end bearing and driven to the required resistance on or within bedrock. H-piles should be 50 ksi, Grade A572 steel and fitted with pile tips. The H-piles shall be designed for all relevant strength, service, and extreme limit state load groups. The structural resistance checks should include checking axial, lateral, and flexural resistance. The resistance of the piles should be evaluated for compliance with the interaction equation for combined axial load and flexure. It is recommended that final design include lateral pile resistance analyses using L-Pile® Plus 5.0 (L-Pile) software. Recommended geotechnical parameters for generation of soil-resistance (p-y) curves in lateral pile analyses are provided in Section 7.1.3 of this report.

The contractor is required to perform a wave equation analysis. The first pile driven at each abutment shall be dynamically tested to confirm nominal pile resistance and verify the stopping criteria developed by the contractor in the wave equation analysis. Minimum 24-hour restrike tests will be required due to poor quality bedrock. With this level of quality control, the pile should be driven to a nominal resistance equal to the factored axial pile load divided by a resistance factor, φ_{dyn}, of 0.65. Additional pile tests may be required as part of the pile field quality control program should pile behavior vary radically between adjacent piles or should pile behavior indicate a pile is refusing on a boulder or cobble above bedrock.

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

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

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

Settlement – The fill unit and native soils encountered in the test borings are loose to dense in consistency. These coarse-grained materials are cohesionless and undergo elastic, immediate, compression in response to an increase in the vertical overburden pressure. The project calls for a 14-inch grade increase in the vertical alignment. Elastic settlement due to the proposed grade increase is anticipated to be small and occur relatively quickly. Construction loads could introduce elastic settlements and these settlements are also anticipated to be small and occur relatively quickly. Post construction settlement should be negligible. Any settlement of the bridge abutments will be due to axial compression of the foundation piles and is anticipated to be minimal.

Frost Protection – Pile supported integral abutments shall be embedded a minimum of 4 feet for frost protection. Foundations placed on or in granular soils should be founded a minimum of 9.0 feet below finished exterior grade for frost protection. Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection.

Scour and Riprap – For scour protection and protection of pile supported integral abutments the bridge approach slopes and slopes at abutments shall be armored with plain riprap. The riprap shall be underlain by Class 1 nonwoven erosion control geotextile and 1 foot thick layer of bedding material.

Seismic Design Considerations – Madawaska Bridge is in Seismic Zone 1, therefore no consideration for seismic forces is required except that superstructure connections and minimum support length requirements shall be satisfied.

Construction Considerations – Construction of the abutments will require pile driving. Temporary lateral earth support systems may be required to permit construction of driven pile foundations at the proposed abutments.

The new integral abutments will be constructed behind the existing abutments avoiding placement of fills or cofferdams in the river. There is a potential that the existing substructures, if not removed entirely, may obstruct pile driving operations. The contractor shall be responsible for excavating those portions of the existing abutments and footings that conflict with piles by conventional excavation methods, pre-augering, predrilling, spudding, use of rock chisels, or down-hole hammers. Excavation by these methods shall be made incidental to related pay items. It is assumed that the existing substructures will be removed

to the streambed or slightly below. Care should be taken to ensure suitable materials are not disturbed unnecessarily.

Cobbles and boulders were encountered in the native sand, gravel, and silt soils (glacial till) underlying the bridge approaches. Cobbles may also be encountered in the fill unit based on recovered broken rock fragments. A cemented layer, possibly the "floor" of an abandoned retaining basin, was encountered at Abutment No. 2. These natural obstructions and man-made subsurface features may impact construction activities. Impacts include but are not limited to impeding the driving of sheet piles for temporary earth support systems and driving H-piles for abutment foundations. Obstructions may be cleared by conventional excavation methods, pre-augering, predrilling, spudding, use of rock chisels, or down-hole hammers. Alternative methods to clear obstructions may be used as approved by the Resident. Care should take to drive piles within allowable tolerances.

Excavations for the proposed abutments will expose soils that may become saturated and water seepage may occur during construction. There may be localized sloughing and instability in some excavations and cut slopes. The contractor should control groundwater, surface water infiltration and soil erosion. Water should be controlled by pumping from sumps.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and provide geotechnical design recommendations for the replacement of Madawaska Bridge which carries Main Street over Little Madawaska River in Stockholm, Maine. This report presents subsurface information obtained at the site during the subsurface investigation, foundation recommendations, and geotechnical design parameters for design of the new bridge substructures.

The existing Madawaska Bridge was constructed in 1923 and is a two-span cast-in-place concrete T-beam superstructure. Each span length is 35 feet. The center pier is a mass concrete pier founded on a spread footing on native soils. The existing mass concrete abutments are also founded on spread footings on native granular soils.

The existing structure is in overall "poor" condition with the deck, superstructure, and substructure rated as 4. The entire bridge displays signs of moderate to advanced deterioration. Spalling, scaling, and shear cracks have formed on the concrete elements. The bridge is currently reduced to one lane due to the barriers placed in front of the failed monolithic concrete bridge rails. Five full height cracks in the abutments require ongoing monitoring. The existing bridge has a sufficiency rating of 49 out of a possible 100.

The replacement structure will be a 100-foot single-span weathering steel I-girder superstructure founded on H-pile supported integral abutments. The existing pier will be removed to one foot below the stream bed and the existing abutments will be removed to the extent necessary to allow for construction of the new structure. The new integral abutments will be constructed behind the removed existing abutments and slopes armored with riprap. Riprap should be constructed in a manner that minimizes property and environmental impacts along project limits while providing slope protection and prevention of soil loss.

The new Madawaska Bridge will be located on nearly the same horizontal alignment. An average increase in grade of 14 inches is expected to achieve a desired half percent slope for the length of the bridge. This vertical profile adjustment will require approximately 300 feet of approach work. The new bridge will accommodate two 11-foot lanes travel lanes with an additional 9-feet 4-inches provided for shoulders and curbs. The total width is to be 31-feet 4-inches. A temporary bridge will be located upstream of the existing bridge to maintain one lane of alternating traffic in each direction.

2.0 GEOLOGIC SETTING

Madawaska Bridge in Stockholm crosses the Little Madawaska River as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of the Stockholm Quadrangle, Maine, Open-file No. 78-8 (1978), indicates the surficial soils in the vicinity of the bridge

project consist of glacial till. Glacial till is a heterogeneous mixture of sand, silt, clay, and stones. Glacial till includes two varieties: basil till and ablation till. Basal till is typically fine grained and very compact with low permeability and poor drainage. Ablation till is typically loose, sandy, and stony with moderate permeability and fair to good drainage. These soils generally overly bedrock, but may overlie, or include, sand and gravel.

The Bedrock Geologic Map of Maine, MGS (1985), cites the bedrock at the proposed bridge site as interbedded pelite and sandstone of the Fogelin Hill Formation, a sequenced shale interlayered with siltstone and fine-grained sandstone that overlies the Jemtland Formation in the Stockholm Mountain syncline.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling two (2) test borings terminating with bedrock cores. Test boring BB-SLMR-101 was drilled south of the existing southerly abutment and test boring BB-SLMR-102 was drilled north of the existing northerly abutment. Borings BB-SLMR-101 and BB-SLMR-102 were located to explore subsurface conditions for an on alignment replacement bridge with abutments placed behind the existing structure. The test boring locations are shown on Sheet 2 – Boring Location Plan.

Test boring BB-SLMR-101 was drilled on April 13 and 18, 2012, and BB-SLMR-102 was drilled on April 11 and 12, 2012, by the MaineDOT Drill Crew. Details and sampling methods used, field data obtained, and soil and conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs.

All borings were performed using solid stem auger, cased wash boring, and rock coring techniques. Soil samples were typically obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the split spoon sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The MaineDOT drill rig is equipped with an automatic hammer to drive the split spoon sampler. The automatic hammer was calibrated per ASTM D4633 “Standard Test Method for Energy Measurement for Dynamic Penetrometers” in September of 2011. All N-values discussed in this report are corrected values computed by applying the corresponding average energy transfer factor of 0.783 to the raw field N-values. The hammer efficiency factor (0.783) and both the raw field N-values and the corrected N-values (N_{60}) are shown on the boring logs.

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

4.0 LABORATORY TESTING

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

Soil laboratory testing consisted of two (2) standard grain size analyses with natural water content and eight (8) grain size analyses with hydrometer and natural water content. The results of soil laboratory tests are included as Appendix B – Laboratory Test Results. Laboratory test information is also shown on the boring logs provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings generally consisted of granular fill and glacial till underlain by metamorphic sedimentary bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs. A generalized subsurface profile is shown on Sheet 3 – Interpretive Subsurface Profile. The following paragraphs discuss the subsurface conditions encountered:

5.1 Fill

A layer of granular fill was encountered in both test borings (BB-SLMR-101 and BB-SLMR-102). The fill unit encountered is approximately 14.3 feet thick at proposed Abutment No. 1 and approximately 19.6 feet thick at proposed Abutment No. 2. The fill soils encountered generally consisted of:

- Brown, moist, gravel, some fine to coarse sand, little silt;
- Brown, moist, fine to coarse sand, some gravel, some silt; and
- Brown and grey, moist, gravelly fine to coarse sand, little to trace silt.

Broken rock fragments were recovered in two samples of the fill unit indicating the presence of cobbles.

Corrected SPT N-values in the fills ranged from 9 to 37 blows per foot (bpf) indicating that the fill soils are loose to dense in consistency.

Two (2) grain size analyses of the fill soils resulted in the soil being classified as A-1-b or A-2-4 under the AASHTO Soil Classification System and SM, or GM, under the Unified Soil Classification System (USCS). The natural water content of the samples tested ranged from approximately 7 to 9 percent.

5.2 Cemented Material

A layer consisting of a boulder and cemented material was encountered in boring BB-SLMR-102. The thickness of the layer encountered was approximately 3.3 feet at the boring location. The cement-like material is possibly remnants of the "floor" of an abandoned tannery hot water pond.

5.3 Glacial Till

A layer of glacial till was encountered in both test borings (BB-SLMR-101 and BB-SLMR-102). The glacial till deposit is approximately 47.4 to 47.7 feet thick at the boring locations. The glacial till generally consisted of:

- Grey-brown, mottled, moist, gravel, some fine to coarse sand, little silt, little clay;
- Grey, moist, gravelly silt, some sand, little clay;
- Grey, moist, gravel, some sand, some silt, little clay;
- Grey, moist, sandy silt, little gravel, little clay;
- Dark grey and grey, wet to moist, sand, some to trace gravel, some to little silt, little gravel, little to trace clay;
- Grey, saturated, silty sand, little to trace gravel, little to trace clay;
- Grey, wet, gravelly sand, some to little silt;
- Grey-brown, mottled, fine to medium sand, some gravel, little silt.

Fragments of rock were recovered in some samples of the glacial till indicating the presence of cobbles. Occasional bolders were also encountered in the till deposit.

Corrected SPT N-values in the coarse grained glacial till layers ranged from 13 to 76 bpf indicating the coarse grained layers are medium dense to very dense in consistency. Corrected SPT N-values in the fine grained glacial till layers ranged from 38 to 82 bpf indicating the fine grained layers are medium stiff to stiff in consistency.

Eight (8) grain size analyses of the glacial till samples resulted in the soil being classified as A-2-4 and A-4 under the AASHTO Soil Classification System and GC-GM, SC-SM, or CL-ML under the USCS. The moisture contents of the tested samples ranged from approximately 7 to 21 percent.

5.4 Bedrock

Bedrock was encountered and cored in both test borings. The top of bedrock surface encountered approximately ranged from Elev. 486.9 below proposed Abutment No. 1 to Elev. 489.2 below proposed Abutment No. 2.

The bedrock at the site is identified as dark grey, very fine to fine grained, limestone and shale, hard, fresh to moderately weathered, moderate to steep breaks along bedding, open

joints with calcite and dolomite infilling. The RQD of the bedrock ranges from 8 to 37 percent correlating to a rock mass quality of very poor to poor. The low RQD is caused in large part by breaks along the bedding throughout the cores. Detailed descriptions of specific core segments and the RQD of each core section are provided on Sheet 3 – Boring Logs and in Appendix A – Boring Logs.

Table 1 summarizes approximate depths to bedrock and corresponding approximate top of bedrock elevations.

Proposed Substructure	Boring	Station	Offset (feet)	Approximate Depth to Bedrock (feet)	Approximate Elevation of Bedrock Surface (feet)
Abutment No. 1	BB-SLMR-101	5+48.1	5.9 Ft Rt	67.0	486.9
Abutment No. 2	BB-SLMR-102	6+51.6	7.5 Ft Lt	65.3	489.2

Table 1 – Summary of Approximate Bedrock Depths and Elevations

5.5 Groundwater

Groundwater was measured in test boring BB-SLMR-102 to be approximately 12 feet below ground surface (bgs). The water level measured upon completion of drilling is indicated on the boring logs found in Appendix A. Note that water was introduced into the boreholes during the drilling operations. Therefore, the water level indicated on the boring log may not represent stabilized groundwater conditions. Groundwater levels will fluctuate with changes in the water levels in the river, seasonal changes, precipitation, runoff, and construction activities.

6.0 FOUNDATION ALTERNATIVES

During preliminary design, pile-supported integral abutments were identified at the most cost effective and preferred substructure type. According to the July 2014 Preliminary Design Report, the streambed is considered “somewhat scour susceptible”; therefore, spread footings founded on soil were not considered as a viable foundation alternative. Jointless integral substructure/superstructure connections will allow for faster construction, lower maintenance costs, and longer service life.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

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

7.1 Integral Abutment H-Piles

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

Pile lengths at the proposed abutments may be estimated based on Table 2:

Location (Boring)	Approximate Bottom Elevation of Proposed Abutment (feet)	Approximate Top of Bedrock Elevation (feet)	Estimated Pile Lengths (feet)
Abutment No. 1 BB-SLMR-101	545.5	486.9	58.6
Abutment No. 2 BB-SLMR-102	544.5	489.2	55.3

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

The estimated pile lengths in Table 2 do not take into account locations where bedrock may be deeper or shallower than that encountered in the test borings, damaged pile, the additional five (5) feet of pile required for dynamic testing instrumentation (per ASTM D4945), additional pile length needed to accommodate leads and driving equipment, or additional pile length needed for embedment in the abutment or pile cap.

7.1.1 Strength Limit State Design

The design of pile foundations bearing on bedrock at the strength limit state shall consider;

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

The pile groups should be designed to resist all lateral earth loads, vehicular loads, dead and live loads, and lateral forces transferred through the pile caps. The pile group resistance after scour due to the design flood shall provide adequate foundation resistance using the resistance factors given in this section.

Per LRFD Article 6.5.4.2, at the strength limit state, the axial resistance factor ϕ_c = 0.50 (severe driving conditions) shall be applied to the structural compressive resistance of the pile. Since the H-piles will be subjected to lateral loading, the piles shall also be checked for resistance against combined axial compression and flexure as prescribed in LRFD Articles 6.9.2.2 and 6.15.2. This design axial load may govern the design. Per LRFD Article 6.5.4.2, at the strength limit state, the axial resistance factor ϕ_c = 0.70 and the flexural resistance factor ϕ_f = 1.0 shall be applied to the combined axial and flexural resistance of the pile in the interaction equation (LRFD Eq. 6.9.2.2-1 or -2).

Abutment H-piles should be analyzed by the geotechnical engineer for determination of unbraced lengths and fixity using L-Pile® Plus 5.0 (L-Pile) software. The calculated unbraced lengths should be used to analyze the piles in combined axial compression and flexure resistance as prescribed in LRFD Articles 6.9.2.2 and 6.15.2.

Structural Resistance. The nominal axial compressive structural resistance (P_n) for piles loaded in compression shall be as specified in LRFD Article 6.9.4.1. Preliminary estimates of the structural axial resistance of five (5) H-pile sections were calculated using a resistance factor, ϕ_c = 0.50 for severe driving conditions. The unbraced pile lengths (ℓ) and effective length factors (K) in these evaluations have been assumed. It is the responsibility of the structural engineer to calculate the nominal axial structural compressive resistance (P_n) based on unbraced lengths (ℓ) and effective length factors (K) determined from L-Pile.

Geotechnical Resistance. The nominal axial geotechnical resistance in the strength limit state was calculated using the guidance in LRFD Article 10.7.3.2.1 which states the nominal bearing resistance of piles driven to point bearing on hard rock shall not exceed the structural pile resistances obtained from LRFD Article 6.9.4.1 with a resistance factor ϕ_c, of 0.50, for severe driving conditions applied. The resulting limiting factored geotechnical compressive resistances for piles driven to rock are provided in Table 3.

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

A summary of the calculated factored axial compressive structural, geotechnical, and drivability resistances of five (5) H-piles for the strength limit states are provided in Table 3. Supporting calculations are provided in Appendix C – Calculations.

Pile Section	Strength Limit State Factored Axial Pile Resistance			
	Structural Resistance[1] ϕ_c=0.50 (kips)	Controlling Geotechnical Resistance[2] (kips)	Drivability Resistance φ_{dyn} = 0.65 (kips)	Governing Axial Pile Resistance (kips)
HP 12 x 53	387[3]	387[3]	328	328
HP 12 x 74	544	544	377	377
HP 14 x 73	535[3]	534[3]	374	374
HP 14 x 89	652	652	409 (491)[4]	491
HP 14 x 117	859	859	618	618

Table 3 – Factored Axial Compressive Resistances for H-Piles at Strength Limit States

LRFD Article 10.7.3.2.3 states that the nominal axial compressive resistance of piles driven to hard rock is typically controlled by the structural resistance with a resistance factor for severe driving conditions applied. However, the estimated factored axial pile resistances from the drivability analyses for the H-pile sections are less than the controlling factored axial structural resistance per LRFD Article 10.7.3.2.3. Therefore, drivability controls, and the recommended governing resistances for pile design are the drivability resistances provided in the rightmost column "Governing Axial Pile Resistance (kips)" in Table 3. The maximum applied factored axial pile load for the strength limit states should not exceed the governing factored pile resistance shown in Table 3 above.

7.1.2 Service and Extreme Limit State Design

The design of H-piles at the service limit state shall consider tolerable transverse and longitudinal movement of the piles and pile group movements/stability considering changes in soil conditions due to scour due to the design flood (Q_{100}). For the service limit state, resistance factors of ϕ = 1.0 should be used in accordance with LRFD Article 10.5.5.1. The exception is the overall global stability of the foundation which should be investigated at the Service I load combination and a resistance factor, ϕ, of 0.65.

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

[1] Structural resistances were calculated for approximated normal conditions (no scour). Controlling value shown here is for a segment in pure compression using a resistance factor, ϕ_c=0.50, for severe driving conditions. Factored structural resistances should be calculated for upper and lower unbraced segments based on L-Pile results using a resistance factor, ϕ_c=0.70, for combined axial loading and bending.
[2] Based on guidance in LRFD Article 10.7.3.2.3., *Piles Driven to Hard Rock*
[3] Does not consider resistance factors of slender elements. 12x53 and 14x73 H-pile sections may require additional reductions based upon structural performance.
[4] Estimated resistances obtained by driving with a Delmag 19-42. Estimated resistance obtained by driving with a Delmag D36-32 shown in parentheses.

structural failure. The extreme event load combinations are those related to seismic forces, ice loads, debris loads, and certain hydraulic events. Extreme limit state design shall also check that the nominal pile foundation resistance remaining after scour due to the check flood (Q_{500}) can support the extreme limit state loads. Resistance factors for extreme limit states, per LRFD Article 10.5.5.3, shall be taken as $\phi = 1.0$ with the exception of uplift of piles, for which the resistance factor, φ_{up}, shall be 0.80 or less per LRFD Article 10.5.5.3.2.

The nominal axial geotechnical pile resistance at the service and extreme limit state was calculated using the guidance in LRFD Article 10.7.3.2.3. The calculated factored axial structural, geotechnical, and drivability resistances of five (5) H-pile sections for the service and extreme limit states are provided in Table 4. Supporting documentation is provided in Appendix C – Calculations.

Pile Section	Extreme and Service Limit State Factored Axial Pile Resistance			
	Structural Resistance[5] (kips)	Controlling Geotechnical Resistance[6] (kips)	Drivability Resistance (kips)	Governing Axial Pile Resistance (kips)
HP 12 x 53	774[7]	774[7]	505	505
HP 12 x 74	1088	1088	580	580
HP 14 x 73	1069[7]	1069[7]	575	575
HP 14 x 89	1303	1303	630 (755)[8]	755
HP 14 x 117	1718	1718	950	950

Table 4 – Factored Axial Compressive Resistances for H-Piles at Service and Extreme Limit States

LRFD Article 10.7.3.2.3 states that the nominal axial compressive resistance of piles driven to hard rock is typically controlled by the structural resistance with a resistance factor for severe driving conditions applied. However, the estimated factored axial pile resistances from the drivability analyses for the H-pile sections are less than the controlling factored axial structural resistance per LRFD Article 10.7.3.2.3 and the nominal structural resistances. Therefore, drivability controls, and the recommended governing resistances for pile design are the resistances provided in the rightmost column "Governing Axial Pile Resistance (kips)" in Table 4. The maximum applied factored axial pile load for the extreme and service limit states should not exceed the governing factored pile resistance shown in Table 4 above.

[5] Normal conditions consider no soil loss due to scour. Nominal structural resistances were calculated for a braced pile segment using a resistance factor, $\phi = 1.0$. Factored structural resistances should be calculated for upper and lower unbraced pile segments determined by L-Pile analyses.
[6] Based on guidance in LRFD Article 10.7.3.2.3., *Piles Driven to Hard Rock.*
[7] Does not consider resistance factors of slender elements. 12x53 and 14x73 H-pile sections may require additional reductions based upon structural performance.
[8] Estimated resistances obtained by driving with a Delmag 19-42. Estimated resistance obtained by driving with a Delmag D36-32 shown in parentheses.

7.1.3 Lateral Pile Resistance/Behavior

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

A series of lateral pile resistance analyses should be performed by the geotechnical engineer to evaluate pile behavior at both abutments using L-Pile software with pile head deflections, moments, and axial loads supplied by the structural engineer. The designer should utilize the results of the L-Pile analyses to recalculate axial compressive structural pile resistances based on unbraced pile segments and verify pile bending stresses do not exceed allowable stresses.

Geotechnical parameters for generation of soil-resistance (p-y) curves in lateral pile analyses are provided in Tables 5 and 6. In general, the model developed should emulate the soil at the site by using the soil layers (referenced in Tables 5 and 6 by elevations) and using appropriate structural parameters and pile-head boundary conditions for the pile section being analyzed.

Soil Layer	Approx. Elevation of Soil Layer (feet)	Water Table Condition	Effective Unit Weight lbs/in^3 (lbs/ft^3)	k_s (lb/in^3)	Cohesion lb/in^2 (lb/ft^2)	ε_{50}	Internal Angle of Friction
Loose to Dense, SAND and Gravelly SAND, (Fill).	554 - 542	Above	0.0694 (120)	90	-	-	32°
Loose, Gravelly SAND, (Fill).	542 - 534	Below	0. 0336 (58)	20	-	-	30°
Very Dense, GRAVEL, some sand, (Glacial Till).	534 - 525.5	Below	0.0388 (67)	125	-	-	34°
Layered dense SAND, GRAVEL, and hard SILT, (Glacial Till).	525.5 - 506.5	Below	0.0336 (58)	1325	27.8 (4000)	.005	-
Dense, Silty SAND, Gravelly SAND, and SAND, (Glacial Till).	506.5 - 487.5	Below	0.0365 (63)	125	-	-	34°

Table 5 – Soil Parameters for Generation of Soil-Resistance (p-y) Curves at Abutment No. 1

Soil Layer	Approx. Elevation of Soil Layer (feet)	Water Table Condition	Effective Unit Weight lbs/in^3 (lbs/ft^3)	k_s (lb/in^3)	Cohesion lb/in^2 (lb/ft^2)	ε_{50}	Internal Angle of Friction
Medium dense, GRAVEL, and Gravelly SAND, (Fill).	554 – 542.5	Above	0.0694 (120)	90	-	-	32°
Medium dense, GRAVEL, and Gravelly SAND, (Fill).	542.5 - 537	Below	0. 0365 (63)	60	-	-	32°
Medium dense to dense, SAND, Silty SAND, Boulders, (Glacial Till).	537 - 499	Below	0.0376 (65)	125	-	-	34°
Medium dense, SAND, (Glacial Till).	499 - 489	Below	0.0336 (58)	60	-	-	34°

Table 6 – Soil Parameters for Generation of Soil-Resistance (p-y) Curves at Abutment No. 2

7.1.4 Driven Pile Resistance and Pile Quality Control

The contract plans shall require the contractor to perform a wave equation analysis of the proposed pile-hammer system and conduct dynamic pile load tests with signal matching. The first pile driven at each abutment should be dynamically tested to confirm nominal pile resistance and verify the stopping criteria developed by the contractor in the wave equation analysis. The pile driving acceptance criteria developed shall prevent pile damage. Relaxation of poor quality bedrock around pile tips may occur and result in a reduction of pile resistance. Minimum 24-hour restrikes of test piles will be required in order to monitor the anticipated relaxation and ensure final required nominal pile resistances are achieved. Additional dynamic tests may be required as part of the pile field quality control program should pile behavior vary radically between adjacent piles, or should pile behavior indicate a pile is refusing on a boulder or in a cobble layer above bedrock.

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

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

stresses in the pile determined in the drivability analysis shall be less than 45 ksi, in accordance with LRFD Article 10.7.8. A hammer should be selected which provides the required pile resistance when the penetration resistance for the final 3 to 6 inches is 3 to 15 blows per inch (bpi). If an abrupt increase in driving resistance is encountered, the driving could be terminated when the penetration is less than 0.5-inch in 10 consecutive blows.

7.2 Integral Abutment Design

Integral abutment sections shall be designed for all relevant strength, service, and extreme limit states and load combinations specified in LRFD Articles 3.4.1 and 11.5.5. Stub abutments shall be designed to resist all lateral earth loads, vehicular loads, dead and live loads, and lateral forces transferred through the integral superstructure. The design of the integral abutment at the strength limit state shall consider reinforced-concrete structural design.

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

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

The designer may assume Soil Type 4 (MaineDOT Bridge Design Guide (BDG) Section 3.6.1) for abutment backfill material soil properties. The backfill properties are as follows: angle of internal friction (ϕ) of 32 degrees, total unit weight (γ) of 125 pcf, and a soil-concrete interface friction angle (δ) of 20 degrees.

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

Additional lateral earth pressure due to live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for abutments if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination of the surcharge load, is permitted per

LRFD Article 3.11.6.5. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 7 below:

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

Table 7 – Equivalent Height of Soil for Estimating Live Load Surcharge on Abutments

The abutment design shall include a drainage system behind the abutment to intercept any groundwater. Weep holes, if required, should be constructed approximately 6 inches above the riprap shelf. Drainage behind the structure shall be in accordance with MaineDOT BDG Section 5.4.2.13.

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

Slopes in front of the pile supported integral abutments should be set back from the riverbank and should be constructed with riprap and erosion control geotextile. The slopes should not exceed 1.75H:1V in accordance with MaineDOT Standard Detail 610(03).

7.3 In-line Wingwalls

In-line, cantilevered "butterfly" wingwalls may be used in conjunction with the integral abutments. The wingwalls shall be designed for all relevant strength, service, and extreme limit states and load combinations specified in LRFD Articles 3.4.1, 11.5.5 and 11.6. The walls shall be designed to resist lateral earth pressures, vehicular loads, and collision loads, as well as, creep, temperature, and shrinkage deformations. The design of "butterfly" wingwalls shall account for the additional bending stresses resulting from the wingwall being cantilevered off the abutment. The use of independently supported wingwalls should be considered when construction of butterfly wingwalls is no longer economical.

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

There are no bearing resistance considerations or special foundation supports needed for wingwalls that are cantilevered off the abutment. However, it is recommended that the geotechnical engineer be consulted should other earth retaining systems not discussed within this report be considered for design. Independently supported wingwalls that are not pile supported are required to meet the embedment requirements of Section 7.5 of this report.

7.4 Settlement

The approximately 16 to 20-foot thick fill unit encountered in the test borings is loose to dense in consistency. These coarse grained materials undergo elastic, immediate, compression in response to an increase of vertical overburden pressure. The project calls for a 14-inch grade increase in the vertical alignment. Elastic settlement due to the proposed grade increase is anticipated to be small and occur relatively quickly. Construction loads could introduce elastic settlements and these settlements are also anticipated to be small and occur relatively quickly. Post construction settlement should be negligible.

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

7.5 Frost Protection

Pile-supported integral abutments shall be embedded a minimum of 4.0 feet for frost protection per MaineDOT BDG Section 5.2.1.

Foundations placed on fill side slopes should be designed with an appropriate embedment for frost protection. According to BDG Figure 5-1, Maine Design Freezing Index Map, Stockholm has a design freezing index of approximately 2600 F-degree days. An assumed water content of 10% was used for coarse grained soils. These components correlate to a frost depth of approximately 9.1 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Stockholm was assigned a design freezing index of approximately 2462 F-degree days, for Caribou, the closest location in the Modberg database. An assumed water content of 10% was used for coarse grained fill soils above the water table. These components correlate to a frost depth of approximately 8.8 feet. Based on an average of these results, it is recommended foundations be designed with an embedment of approximately 9.0 feet for frost protection. See Appendix C – Calculations for supporting documentation.

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

7.6 Scour and Riprap

Grain size analyses were performed on samples taken at the approximate streambed elevation to generate grain size curves for determining parameters to be used in scour analyses. The sample was assumed to be similar in nature to the soils likely to be exposed to scour conditions. The following streambed grain size parameters can be used in scour analyses:

- Average diameter of particle at 50 percent passing, D_{50} = 1.55 mm (medium sand)
- Average diameter of particle at 95 percent passing, D_{95} = 35 mm (coarse gravel)
- Soil Classification AASHTO Soil Type A-2-4.

The grain size curves are included in Appendix B – Laboratory Test Results.

The consequences of changes in foundation conditions resulting from the design (Q_{100}) and check (Q_{500}) floods for scour shall be considered at the strength and extreme limit states, respectively. Design at the strength limit state should consider loss of lateral and vertical support due to scour. Design at the extreme limit state should check that the nominal foundation resistance due to the check flood (Q_{500}) event is no less that the extreme limit state loads. At the service limit state, the design shall limit movements and ensure overall stability considering scour at the design load.

For scour protection of the pile supported abutments, the PDR indicates the bridge approach slopes and the abutment slopes will be armored with riprap. Refer to MaineDOT BDG Section 2.3.11.3 for information regarding scour design. Typically the top of the riprap is located at, or above, the Q_{50} elevation.

Plain riprap shall conform to MaineDOT Standard Specification 703.26 – Plain and Hand Laid Riprap. 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 MaineDOT Standard Specification 703.19 and Class 1 nonwoven erosion control geotextile per MaineDOT Standard Details 610(02) and 610(03).

7.7 Seismic Design Considerations

The United States Geological Survey Seismic Design CD (Version 2.1) provided with the LRFD Manual, and LRFD Articles 3.10.3.1 and 3.10.6 were used to develop parameters for seismic design. Based on site coordinates, the software provided the recommended AASHTO Response Spectra for a 7 percent probability of exceedance in 75 years. These results are summarized in Table 8:

Parameter	Design Value
Peak Ground Acceleration (PGA)	0.085g
Acceleration Coefficient (A_S)	0.102g
S_{DS} (Period = 0.2 sec)	0.233g
S_{D1} (Period = 1.0 sec)	0.096g
Site Class	C
Seismic Zone	1

Table 8 – Seismic Design Parameters

In conformance with LRFD Article 4.7.4 seismic analysis is not required for bridges in Seismic Zone 1 or single-span bridges regardless of seismic zone. However, superstructure connections and minimum support length requirements shall be designed per LRFD Articles 3.10.9.2 and 4.7.4.4, respectively.

See Appendix C- Calculations for supporting documentation.

7.8 Construction Considerations

Construction of the abutments will require pile driving. Temporary lateral earth support systems may be required to permit construction of driven pile foundations at the proposed abutments.

The new integral abutments will be constructed behind the existing abutments avoiding placement of fills or cofferdams in the river. There is a potential that the existing substructures, if not removed entirely, may obstruct pile driving operations. The contractor shall be responsible for excavating those portions of the existing abutments and footings that conflict with piles by conventional excavation methods, pre-augering, predrilling, spudding, use of rock chisels, or down-hole hammers. Excavation by these methods shall be made incidental to related pay items. It is assumed that the existing substructures will be removed to the streambed or slightly below. Care should be taken to ensure suitable materials are not disturbed unnecessarily.

Cobbles and boulders were encountered in the native sand, gravel, and silt soils (glacial till) underlying the bridge approaches. Cobbles may also be encountered in the fill unit based on broken rock fragments recovered in the borings. A cemented layer, possibly the “floor” of an abandoned retaining basin, was encountered at Abutment No. 2. These natural and man-made subsurface features may impact construction activities. Impacts include but are not limited to impeding the driving of sheet piles for temporary earth support systems and driving H-piles for abutment foundations. Obstructions may be cleared by conventional excavation methods, pre-augering, predrilling, spudding, use of rock chisels, or down-hole hammers. Alternative methods to clear obstructions may be used as approved by the Resident. Care should take to drive piles within allowable tolerances.

Excavations for the proposed abutments will expose soils that may become saturated and water seepage may occur during construction. There may be localized sloughing and instability in some excavations and cut slopes. The contractor should control groundwater, surface water infiltration and soil erosion. Water should be controlled by pumping from sumps.

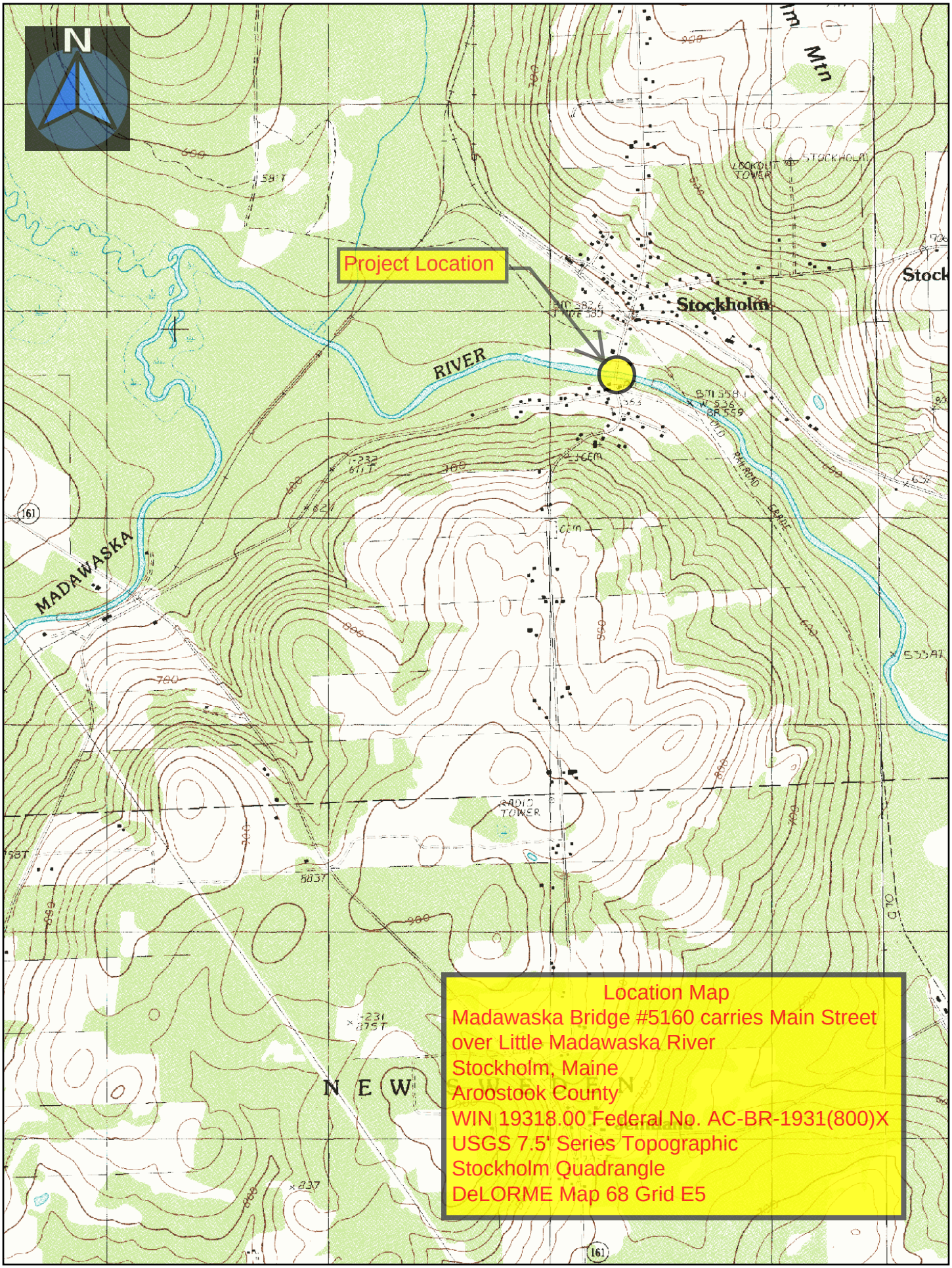
8.0 CLOSURE

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

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

It is also recommend that the geotechnical engineer be provided an 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



Map Scale 1:24000

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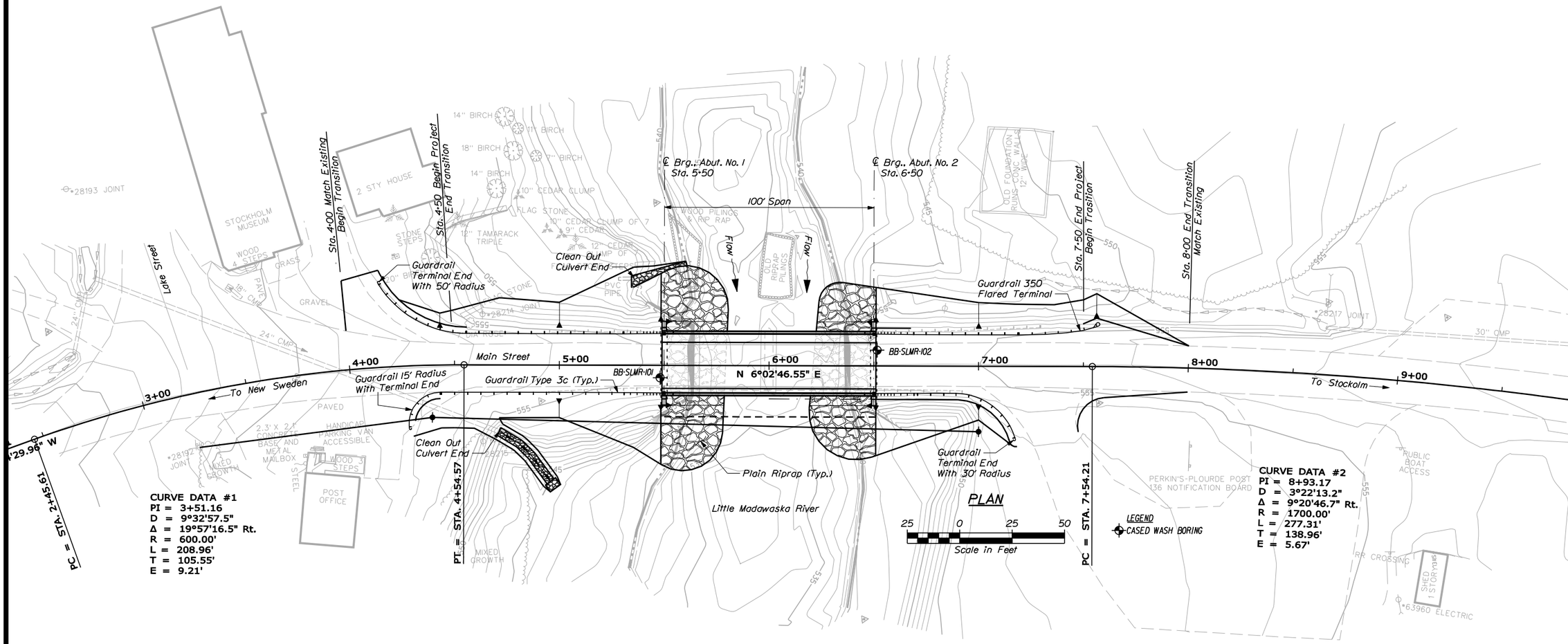


STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

AC-BR-1931(800)X

WIN 19318.00

BRIDGE NO. 5160

BRIDGE PLANS

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

SIGNATURE

P.E. NUMBER

DATE

MADAWASKA BRIDGE
LITTLE MADAWASKA RIVER
STOCKHOLM AROOSTOOK COUNTY

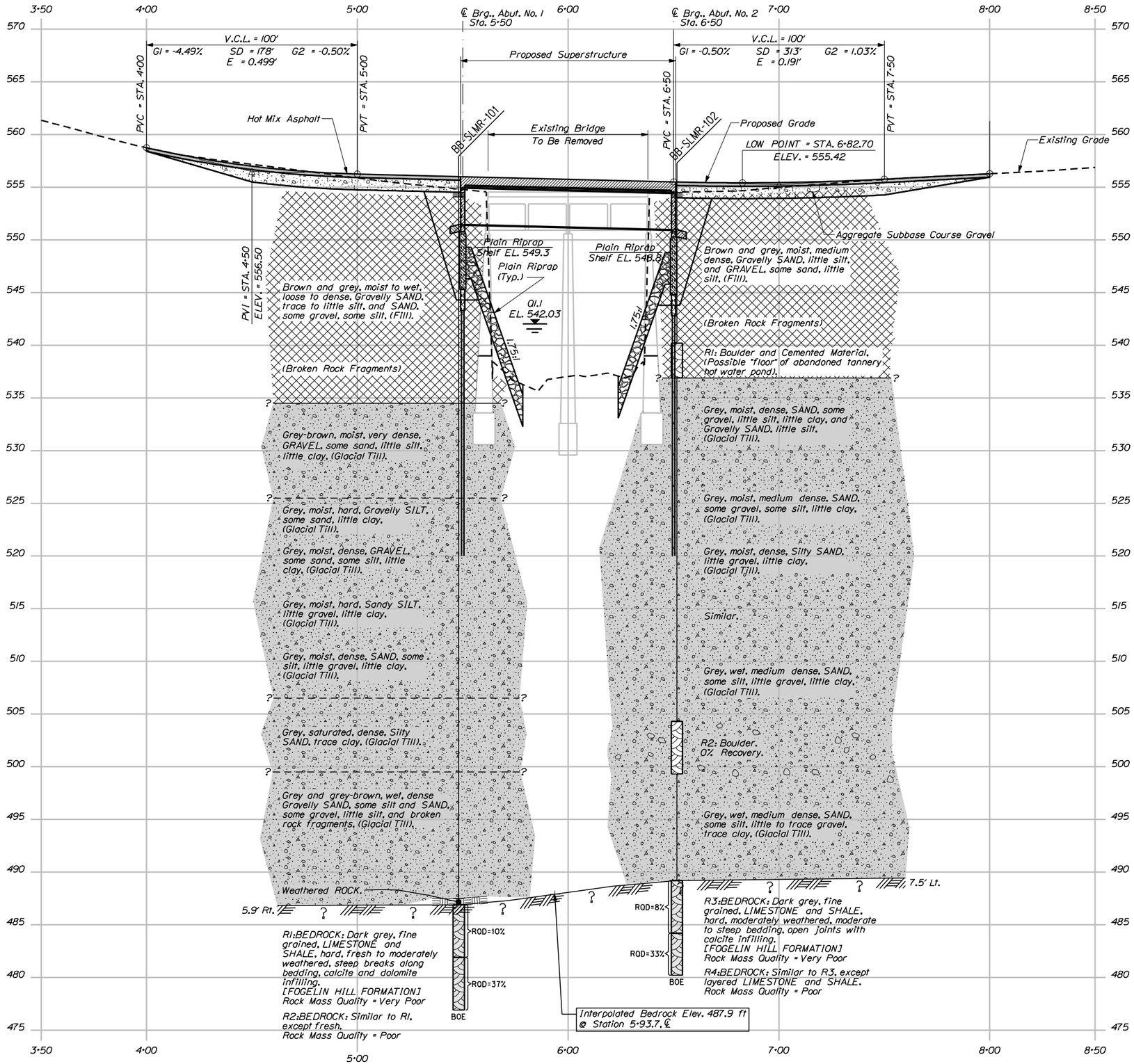
BORING LOCATION PLAN

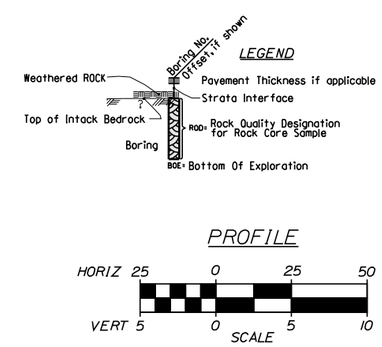
SHEET NUMBER

2

OF 4

Filename: ...\00\geotech\msta\006_BLP1.dgn Division: GEOTECH Username: terry.white Date:8/27/2015





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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

AC-BR-1931(800)X

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BRIDGE PLANS

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PROJ. MANAGER		
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FIELD CHANGES		

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DATE

MADAWASKA BRIDGE
LITTLE MADAWASKA RIVER
STOCKHOLM AROOSTOOK COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

3

OF 4

Filename: ...\00\geotech\msta\007_ISP1.dgn Division: GEOTECH Username: terry.white Date:8/27/2015

Filename: ...\msta\008_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:8/27/2015

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

Project: Madawaska Bridge #5160 carries Main Street over Little Madawaska
Location: Stockholm, Maine
Boring No.: BB-SLMR-101
WIN: 19318.00

Driller:	MaineDOT	Elevation (ft.)	554.5	Auger ID/OD:	5" Solid Stem
Operator:	Wilder/Giles/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	4/13,18/2012	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	5+48.1, 5.9 ft Rt.	Casing ID/OD:	HW & NW	Water Level*:	None Observed
Hammer Efficiency Factor: 0.783		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

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	554.08	5" PAVEMENT. —0.42	
	S1		1.00 - 4.00						Brown, moist, fine to coarse SAND, some gravel, some silt, (Fill).	G#237522 A-2-4, SM WC=9.1%
5	1D	24/12	5.00 - 7.00	13/18/10/5	28	37			Brown, moist, dense, Gravelly fine to coarse SAND, little silt, (Fill).	
10	2D	24/7	10.00 - 12.00	9/4/3/6	7	9	19		Grey, wet, loose, Gravelly (broken rock fragments) fine to coarse SAND, trace silt, (Fill). Roller Coned ahead to 15.0 ft bgs.	
							13			
							25			
							23			
							39			
15	3D	24/4	15.00 - 17.00	2/4/4/7	8	10	30		Broken rock fragments.	
							56			
							52			
							47			
							55	534.50	—20.00	
20	4D	24/17	20.00 - 22.00	9/21/37/43	58	76	6		Grey-brown, mottled, moist, very dense, GRAVEL, some fine to coarse sand, little silt, little clay, blocky, non-plastic, (Glacial Till). Changed to NW Casing at 20.0 ft bgs. Roller Coned ahead to 30.0 ft bgs.	G#237523 A-2-4, GC-GM WC=8.5%
							5			
							7			
							11			
							14			
25							4			
							5			
							11			
							19	525.50	—29.00	
							89			
30	5D	24/16	30.50 - 32.50	35/36/27/23	63	82	75		Grey, moist, hard, Gravelly SILT, some sand, little clay, (Glacial Till).	
							161			
							159			
							156			
							196			
35	6D	24/17	35.00 - 37.00	12/21/15/23	36	47	169		Grey, moist, dense, GRAVEL, some sand, some silt, little clay, (Glacial Till).	G#237524 A-4, GC-GM WC=7.4%
							121			
							186			
							208			
							275			
40	7D	24/13	40.00 - 42.00	14/15/14/21	29	38	44		Grey, moist, hard, Sandy SILT, little gravel, little clay, (Glacial Till). Roller Coned ahead to 45.0 ft bgs.	
							55			
							72			
							87			
							132			
45	8D	24/14	45.00 - 47.00	13/15/16/17	31	40	91		Grey, moist, dense, SAND, some silt, little gravel, little clay, (Glacial Till).	G#237525 A-4, SC-SM WC=8.5%
							104			
							111			
							212	506.50	—48.00	
50							175			

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-SLMR-101

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Date Start/Finish:	4/13,18/2012	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	5+48.1, 5.9 ft Rt.	Casing ID/OD:	HW & NW	Water Level*:	None Observed
Hammer Efficiency Factor: 0.783		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
50	9D	24/19	50.00 - 52.00	9/11/13/9	24	31	122		Grey, saturated, dense, Silty fine to medium SAND, trace clay, (Glacial Till).	G#261880 A-4, CL-ML WC=20.4%
							136			
							155			
							146			
							162			
55	10D	24/14	55.00 - 57.00	19/12/14/16	26	34	81	499.00	—55.50 Grey, wet, dense, Gravelly (broken rock fragments) fine to coarse SAND, some silt, (Glacial Till).	
							110			
							128			
							137			
							161			
60	11D	24/14	60.70 - 62.70	13/16/19/15	35	46	105		Broken rock fragments.	
							93			
							92			
							113			
							112			
65	12D	24/14	65.00 - 67.00	5/14/14/38	28	37	113		Grey-brown, mottled, dense, fine to medium SAND, some gravel (rock fragments), little silt, (Glacial Till).	
							122	487.50	—67.00	
	R1	60/55	67.60 - 72.60	RQD = 10%			190 NQ-2	486.90	Weathered ROCK in tip of spoon. Roller Coned ahead to 67.6 ft bgs. —67.60 Top of Bedrock at Elev. 486.9 ft. R1:Bedrock: Dark grey, fine grained, LIMESTONE and SHALE, hard, moderately weathered, steep breaks along bedding, calcite and dolomite infilling. Rock Mass Quality = Very Poor. Fogelin Hill Formation. R1:Core Times (min:sec) 67.6-68.6 ft (4:00) 68.6-69.6 ft (2:10) 69.6-70.6 ft (2:00) 70.6-71.6 ft (4:20) 71.6-72.6 ft (3:40) 92% Recovery	
70										
	R2	60/60	72.60 - 77.60	RQD = 37%					R2: Bedrock: Similar to R1 except fresh. Rock Mass Quality = Poor. R2:Core Times (min:sec) 72.6-73.6 ft (3:25) 73.6-74.6 ft (2:35) 74.6-75.6 ft (2:50) 75.6-76.6 ft (2:45) 76.6-77.6 ft (2:50) 100% Recovery	
75										
								476.90	—77.60 Bottom of Exploration at 77.60 feet below ground surface.	
80										
85										
90										
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-SLMR-101

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

Project: Madawaska Bridge #5160 carries Main Street over Little Madawaska
Location: Stockholm, Maine
Boring No.: BB-SLMR-102
WIN: 19318.00

Driller:	MaineDOT	Elevation (ft.)	554.5	Auger ID/OD:	5" Solid Stem
Operator:	Wilder/Giles/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	4/11/12-4/12/12	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	6+51.6, 7.5 ft Lt.	Casing ID/OD:	HW & NW	Water Level*:	12.0 ft bgs.
Hammer Efficiency Factor: 0.783		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

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	554.04	5.5" PAVEMENT. —0.46	
	1D	24/16	1.00 - 3.00	11/12/8/9	20	26			Brown, moist, medium dense, GRAVEL, some fine to coarse sand, little silt, (Fill).	G#261881 A-1-b, GM WC=7.4%
5	2D	24/4	5.00 - 7.00	6/6/7/7	13	17			Similar to above.	
10	3D	24/5	10.00 - 12.00	11/7/4/5	11	14	28		Recovered broken rock fragments. Based on drill water: Grey, medium dense, Gravelly fine to coarse SAND, little silt, (Fill).	
							29			
							44			
							47			
	R1	42/30	14.30 - 17.80				a50	540.20	a50 blows for 0.3 ft. —14.30 R1: Boulder (Sandstone) and cemented material. R1:Core Times (min:sec) 14.3-15.3 ft (6:00) 15.3-16.3 ft (6:51) 16.3-17.3 ft (1:30) 17.3-17.8 ft (0:45) Roller Coned through Boulder with 3 7/8" dia. roller cone.	
15							OPEN HOLE			
								536.90	Weathered cemented bed from 16.5-17.1 ft bgs. Possible "floor" of abandoned tannery hot water pond. Roller Coned ahead to 25.0 ft bgs. —17.60	
20	4D	24/16	20.00 - 22.00	8/13/22/22	35	46			Grey, moist, dense, fine to coarse SAND, some gravel, little silt, little clay, (Glacial Till).	G#261882 A-2-4, SC-SM WC=7.2%
25	5D	24/12	25.00 - 27.00	8/16/15/17	31	40	18		Recovered broken rock fragments. Based on wash water: Grey, dense, Gravelly fine to coarse SAND, little silt. Changed to NW Casing at 25.0 ft bgs.	
							35			
							51			
							49			
							43			
30	6D	24/20	30.00 - 32.00	7/10/6/11	16	21	38		Grey, moist, medium dense, fine to coarse SAND, some gravel, some silt, little clay, (Glacial Till).	G#261883 A-2-4, SC-SM WC=8.1%
							41			
							36			
							43			
							57			
35	7D	24/14	35.00 - 37.00	10/14/14/13	28	37	62		Grey, moist, dense, Silty SAND, little gravel, little clay, (Glacial Till).	
							67			
							73			
							106			
							121			
40	8D	24/18	40.00 - 42.00	7/11/13/19	24	31	93		Similar to above.	
							139			
							167			
							198			
							189			
45	9D	24/18	45.00 - 47.00	5/7/13/19	20	26	33		Grey, wet, medium dense, SAND, some silt, little gravel, little clay, (Glacial Till).	G#261884 A-4, SC-SM WC=13.2%
							62			
							77			
							78			
50							135		Roller Coned ahead to 50.2 ft bgs. Cobble from 49.5-50.2 ft bgs.	

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-SLMR-102

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

Project: Madawaska Bridge #5160 carries Main Street over Little Madawaska
Location: Stockholm, Maine
Boring No.: BB-SLMR-102
WIN: 19318.00

Driller:	MaineDOT	Elevation (ft.)	554.5	Auger ID/OD:	5" Solid Stem
Operator:	Wilder/Giles/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	4/11/12-4/12/12	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	6+51.6, 7.5 ft Lt.	Casing ID/OD:	HW & NW	Water Level*:	12.0 ft bgs.
Hammer Efficiency Factor: 0.783		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

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	R2	60/0	50.20 - 55.20				17	504.30	—50.20 R2:BOULDER. R2:Core Times (min:sec) 50.2-51.2 ft (0:45) 51.2-52.2 ft (0:10) 52.2-53.2 ft (0:10) 53.2-54.2 ft (0:10) 54.2-55.2 ft (0:15) 0% Recovery	
							23			
							34			
							36			
							37			
55	10D	24/20	55.50 - 57.50	3/5/5/10	10	13	47	499.00	—55.50 Grey, wet, medium dense, fine to medium SAND, some silt, trace gravel, trace clay, (Glacial Till).	G#261885 A-2-4, SC-SM WC=20.7%
							66			
							70			
							87			
							156			
60	11D	24/19	60.00 - 62.00	8/10/12/18	22	29	60		Grey, wet, medium dense, fine to coarse SAND, some silt, little gravel, trace clay, (Glacial Till).	
							58			
							112			
							129			
							163		Roller Coned ahead to 65.3 ft bgs.	
65	R3	60/51	65.30 - 70.30	RQD = 8%			NQ-2	489.20	—65.30 Top of Bedrock at Elev. 489.2 ft. R3: Bedrock: Dark grey, very fine grained, LIMESTONE and SHALE, hard, moderately weathered, moderate to steep breaks along bedding, open joints with calcite infilling. Rock Mass Quality = Very Poor. Fogelin Hill Formation. R3:Core Times (min:sec) 65.3-66.3 ft (3:05) 66.3-67.3 ft (4:00) 67.3-68.3 ft (3:40) 68.3-69.3 ft (2:50) 69.3-70.3 ft (2:25) 85% Recovery	
70	R4	48/48	70.30 - 74.30	RQD = 33%					R4: Bedrock: Similar to R3. Layered LIMESTONE and SHALE. Shale is platey. RockMassQuality = Poor. R4:Core Times (min:sec) 70.3-71.3 ft (2:45) 71.3-72.3 ft (2:00) 72.3-73.3 ft (2:00) 73.3-74.3 ft (4:00) 85% Recovery Core Blocked	
								480.20	—74.30 Bottom of Exploration at 74.30 feet below ground surface.	
75										

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-SLMR-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
AC-BR-1931(800)X
WIN 19318.00
BRIDGE PLANS
BRIDGE NO. 5160

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

SIGNATURE
P.E. NUMBER
DATE

MADAWASKA BRIDGE
LITTLE MADAWASKA RIVER
STOCKHOLM AROOSTOOK COUNTY

BORING LOGS

SHEET NUMBER
4
OF 4

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Modified Burmister System

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations: (in this order)

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

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Madawaska Bridge #5160 carries Main Street over Little Madawaska River
Location: Stockholm, Maine

Boring No.: BB-SLMR-101
PIN: 19318.00

Driller:	MaineDOT	**Elevation (ft.)**	554.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/13,18/2012	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+48.1, 5.9 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.783	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
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T_v = Pocket Torvane Shear Strength (psf)
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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	554.08	5" PAVEMENT. 0.42	
	S1		1.00 - 4.00						Brown, moist, fine to coarse SAND, some gravel, some silt, (Fill).	G#237522 A-2-4, SM WC=9.1%
5	1D	24/12	5.00 - 7.00	13/18/10/5	28	37			Brown, moist, dense, Gravelly fine to coarse SAND, little silt, (Fill).	
10	2D	24/7	10.00 - 12.00	9/4/3/6	7	9	19		Grey, wet, loose, Gravelly (broken rock fragments) fine to coarse SAND, trace silt, (Fill). Roller Coned ahead to 15.0 ft bgs.	
							13			
							25			
							23			
							39			
15	3D	24/4	15.00 - 17.00	2/4/4/7	8	10	30		Broken rock fragments.	
							56			
							52			
							47			
							55			
20	4D	24/17	20.00 - 22.00	9/21/37/43	58	76	6	534.50	20.00 Grey-brown, mottled, moist, very dense, GRAVEL, some fine to coarse sand, little silt, little clay, blocky, non-plastic, (Glacial Till). Changed to NW Casing at 20.0 ft bgs. Roller Coned ahead to 30.0 ft bgs.	G#237523 A-2-4, GC-GM WC=8.5%
							5			
							7			
							11			
25							14			

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Madawaska Bridge #5160 carries Main Street over Little Madawaska River
Location: Stockholm, Maine

Boring No.: BB-SLMR-101
PIN: 19318.00

Driller:	MaineDOT	**Elevation (ft.)**	554.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/13,18/2012	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+48.1, 5.9 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.783	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							4				
							5				
							11				
							19				
							89	525.50		29.00	
30	5D	24/16	30.50 - 32.50	35/36/27/23	63	82	75			Grey, moist, hard, Gravelly SILT, some sand, little clay, (Glacial Till).	
							161				
							159				
							156				
							196				
35	6D	24/17	35.00 - 37.00	12/21/15/23	36	47	169			Grey, moist, dense, GRAVEL, some sand, some silt, little clay, (Glacial Till).	G#237524 A-4, GC-GM WC=7.4%
							121				
							186				
							208				
							275				
40	7D	24/13	40.00 - 42.00	14/15/14/21	29	38	44			Grey, moist, hard, Sandy SILT, little gravel, little clay, (Glacial Till). Roller Coned ahead to 45.0 ft bgs.	
							55				
							72				
							87				
							132				
45	8D	24/14	45.00 - 47.00	13/15/16/17	31	40	91			Grey, moist, dense, SAND, some silt, little gravel, little clay, (Glacial Till).	G#237525 A-4, SC-SM WC=8.5%
							104				
							111	506.50		48.00	
							212				
50							175				

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Madawaska Bridge #5160 carries Main Street over Little Madawaska River
Location: Stockholm, Maine

Boring No.: BB-SLMR-101
PIN: 19318.00

Driller:	MaineDOT	**Elevation (ft.)**	554.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/13,18/2012	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+48.1, 5.9 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.783	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	9D	24/19	50.00 - 52.00	9/11/13/9	24	31	122		Grey, saturated, dense, Silty fine to medium SAND, trace clay, (Glacial Till).	G#261880 A-4, CL-ML WC=20.4%
							136			
							155			
							146			
							162			
55	10D	24/14	55.00 - 57.00	19/12/14/16	26	34	81	499.00	55.50	
							110		Grey, wet, dense, Gravelly (broken rock fragments) fine to coarse SAND, some silt, (Glacial Till).	
							128			
							137			
							161			
60	11D	24/14	60.70 - 62.70	13/16/19/15	35	46	105		Broken rock fragments.	
							93			
							92			
							113			
							112			
65	12D	24/14	65.00 - 67.00	5/14/14/38	28	37	113		Grey-brown, mottled, dense, fine to medium SAND, some gravel (rock fragments), little silt, (Glacial Till).	
							122			
	R1	60/55	67.60 - 72.60	RQD = 10%			190 NQ-2	487.50	67.00	
								486.90	Weathered ROCK in tip of spoon. Roller Coned ahead to 67.6 ft bgs.	
									67.60	
									Top of Bedrock at Elev. 486.9 ft. R1:Bedrock: Dark grey, fine grained, LIMESTONE and SHALE, hard, moderately weathered, steep breaks along bedding, calcite and dolomite infilling. Rock Mass Quality = Very Poor. Fogelin Hill Formation. R1:Core Times (min:sec) 67.6-68.6 ft (4:00) 68.6-69.6 ft (2:10) 69.6-70.6 ft (2:00) 70.6-71.6 ft (4:20) 71.6-72.6 ft (3:40) 92% Recovery	
70										
	R2	60/60	72.60 - 77.60	RQD = 37%						
									R2: Bedrock: Similar to R1 except fresh. Rock Mass Quality = Poor. R2:Core Times (min:sec) 72.6-73.6 ft (3:25)	
75										

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Madawaska Bridge #5160 carries Main Street over Little Madawaska River
Location: Stockholm, Maine

Boring No.: BB-SLMR-101
PIN: 19318.00

Driller:	MaineDOT	**Elevation (ft.)**	554.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/13,18/2012	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+48.1, 5.9 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.783	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75										73.6-74.6 ft (2:35) 74.6-75.6 ft (2:50) 75.6-76.6 ft (2:45) 76.6-77.6 ft (2:50) 100% Recovery	
								476.90		77.60 **Bottom of Exploration at 77.60 feet below ground surface.**	
80											
85											
90											
95											
100											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Madawaska Bridge #5160 carries Main Street over Little Madawaska River
Location: Stockholm, Maine

Boring No.: BB-SLMR-102
PIN: 19318.00

Driller:	MaineDOT	**Elevation (ft.)**	554.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/11/12-4/12/12	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	6+51.6, 7.5 ft Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	12.0 ft bgs.
Hammer Efficiency Factor:	0.783	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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MD = Unsuccessful Split Spoon Sample attempt
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MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
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	554.04	5½" PAVEMENT. 0.46	
	1D	24/16	1.00 - 3.00	11/12/8/9	20	26			Brown, moist, medium dense, GRAVEL, some fine to coarse sand, little silt, (Fill).	G#261881 A-1-b, GM WC=7.4%
5	2D	24/4	5.00 - 7.00	6/6/7/7	13	17			Similar to above.	
10	3D	24/5	10.00 - 12.00	11/7/4/5	11	14	28		Recovered broken rock fragments. Based on drill water: Grey, medium dense, Gravelly fine to coarse SAND, little silt, (Fill).	
							29			
							44			
							47			
	R1	42/30	14.30 - 17.80				[a]50	540.20	[a]50 blows for 0.3 ft. 14.30	
15							OPEN HOLE		R1: Boulder (Sandstone) and cemented material. R1:Core Times (min:sec) 14.3-15.3 ft (6:00) 15.3-16.3 ft (6:51) 16.3-17.3 ft (1:30) 17.3-17.8 ft (0:45)	
								536.90	Roller Coned through Boulder with 3 7/8" dia. roller cone. Weathered cemented bed from 16.5-17.1 ft bgs. Possible "floor" of abandoned tannery hot water pond. Roller Coned ahead to 25.0 ft bgs. 17.60	
20	4D	24/16	20.00 - 22.00	8/13/22/22	35	46			Grey, moist, dense, fine to coarse SAND, some gravel, little silt, little clay, (Glacial Till).	G#261882 A-2-4, SC-SM WC=7.2%
25										

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Boring No.: BB-SLMR-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Madawaska Bridge #5160 carries Main Street over Little Madawaska River
Location: Stockholm, Maine

Boring No.: BB-SLMR-102
PIN: 19318.00

Driller:	MaineDOT	**Elevation (ft.)**	554.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/11/12-4/12/12	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	6+51.6, 7.5 ft Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	12.0 ft bgs.
Hammer Efficiency Factor:	0.783	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/12	25.00 - 27.00	8/16/15/17	31	40	18			Recovered broken rock fragments. Based on wash water: Grey, dense, Gravelly fine to coarse SAND, little silt. Changed to NW Casing at 25.0 ft bgs.	
							35				
							51				
							49				
							43				
30	6D	24/20	30.00 - 32.00	7/10/6/11	16	21	38			Grey, moist, medium dense, fine to coarse SAND, some gravel, some silt, little clay, (Glacial Till).	G#261883 A-2-4, SC-SM WC=8.1%
							41				
							36				
							43				
							57				
35	7D	24/14	35.00 - 37.00	10/14/14/13	28	37	62			Grey, moist, dense, Silty SAND, little gravel, little clay, (Glacial Till).	
							67				
							73				
							106				
							121				
40	8D	24/18	40.00 - 42.00	7/11/13/19	24	31	93			Similar to above.	
							139				
							167				
							198				
							189				
45	9D	24/18	45.00 - 47.00	5/7/13/19	20	26	33			Grey, wet, medium dense, SAND, some silt, little gravel, little clay, (Glacial Till).	G#261884 A-4, SC-SM WC=13.2%
							62				
							77				
							78				
50							135			Roller Coned ahead to 50.2 ft bgs. Cobble from 49.5-50.2 ft bgs.	

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Madawaska Bridge #5160 carries Main Street over Little Madawaska River
Location: Stockholm, Maine

Boring No.: BB-SLMR-102
PIN: 19318.00

Driller:	MaineDOT	**Elevation (ft.)**	554.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/11/12-4/12/12	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	6+51.6, 7.5 ft Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	12.0 ft bgs.
Hammer Efficiency Factor:	0.783	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
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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	R2	60/0	50.20 - 55.20				17	504.30	50.20 R2:BOULDER. R2:Core Times (min:sec) 50.2-51.2 ft (0:45) 51.2-52.2 ft (0:10) 52.2-53.2 ft (0:10) 53.2-54.2 ft (0:10) 54.2-55.2 ft (0:15) 0% Recovery	
							23			
							34			
							36			
							37			
55	10D	24/20	55.50 - 57.50	3/5/5/10	10	13	47	499.00	55.50 Grey, wet, medium dense, fine to medium SAND, some silt, trace gravel, trace clay, (Glacial TIll).	G#261885 A-2-4, SC-SM WC=20.7%
							66			
							70			
							87			
							156			
60	11D	24/19	60.00 - 62.00	8/10/12/18	22	29	60		Grey, wet, medium dense, fine to coarse SAND, some silt, little gravel, trace clay, (Glacial TIll).	
							58			
							112			
							129			
							163			
65	R3	60/51	65.30 - 70.30	RQD = 8%			NQ-2	489.20	Roller Coned ahead to 65.3 ft bgs. 65.30 Top of Bedrock at Elev. 489.2 ft. R3: Bedrock: Dark grey, very fine grained, LIMESTONE and SHALE, hard, moderately weathered, moderate to steep breaks along bedding, open joints with calcite infilling. Rock Mass Quality = Very Poor. Fogelin Hill Formation. R3:Core Times (min:sec) 65.3-66.3 ft (3:05) 66.3-67.3 ft (4:00) 67.3-68.3 ft (3:40) 68.3-69.3 ft (2:50) 69.3-70.3 ft (2:25) 85% Recovery	
70	R4	48/48	70.30 - 74.30	RQD = 33%					R4: Bedrock: Similar to R3. Layered LIMESTONE and SHALE. Shale is platey. RockMassQuality = Poor. R4:Core Times (min:sec) 70.3-71.3 ft (2:45) 71.3-72.3 ft (2:00) 72.3-73.3 ft (2:00) 73.3-74.3 ft (4:00) 85% Recovery	
75								480.20		

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Madawaska Bridge #5160 carries Main Street over Little Madawaska River
Location: Stockholm, Maine

Boring No.: BB-SLMR-102
PIN: 19318.00

Driller:	MaineDOT	**Elevation (ft.)**	554.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/11/12-4/12/12	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	6+51.6, 7.5 ft Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	12.0 ft bgs.
Hammer Efficiency Factor:	0.783	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75										Core Blocked	
										74.30	
										Bottom of Exploration at 74.30 feet below ground surface.	
80											
85											
90											
95											
100											

Remarks:

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Appendix B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Stockholm Work Number: 19318.00

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-SLMR-101, S1	5+48.1	5.9 Rt.	1.0-4.00	237522	1	9.1			SM	A-2-4	II
BB-SLMR-101, 4D	5+48.1	5.9 Rt.	20.0-22.0	237523	1	8.5			GC-GM	A-2-4	III
BB-SLMR-101, 6D	5+48.1	5.9 Rt.	35.0-37.0	237524	1	7.4			GC-GM	A-4	III
BB-SLMR-101, 8D	5+48.1	5.9 Rt.	45.0-47.0	237525	1	8.5			SC-SM	A-4	III
BB-SLMR-101, 9D	5+48.1	5.9 Rt.	50.0-52.0	261880	1	20.4			CL-ML	A-4	IV
BB-SLMR-102, 1D	6+51.6	7.5 Lt.	1.0-3.0	261881	2	7.4			GM	A-1-b	I
BB-SLMR-102, 4D	6+51.6	7.5 Lt.	20.0-22.0	261882	2	7.2			SC-SM	A-2-4	III
BB-SLMR-102, 6D	6+51.6	7.5 Lt.	30.0-32.0	261883	2	8.1			SC-SM	A-2-4	III
BB-SLMR-102, 9D	6+51.6	7.5 Lt.	45.0-47.0	261884	2	13.2			SC-SM	A-4	III
BB-SLMR-102, 10D	6+51.6	7.5 Lt.	55.5-57.5	261885	2	20.7			SC-SM	A-2-4	II

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

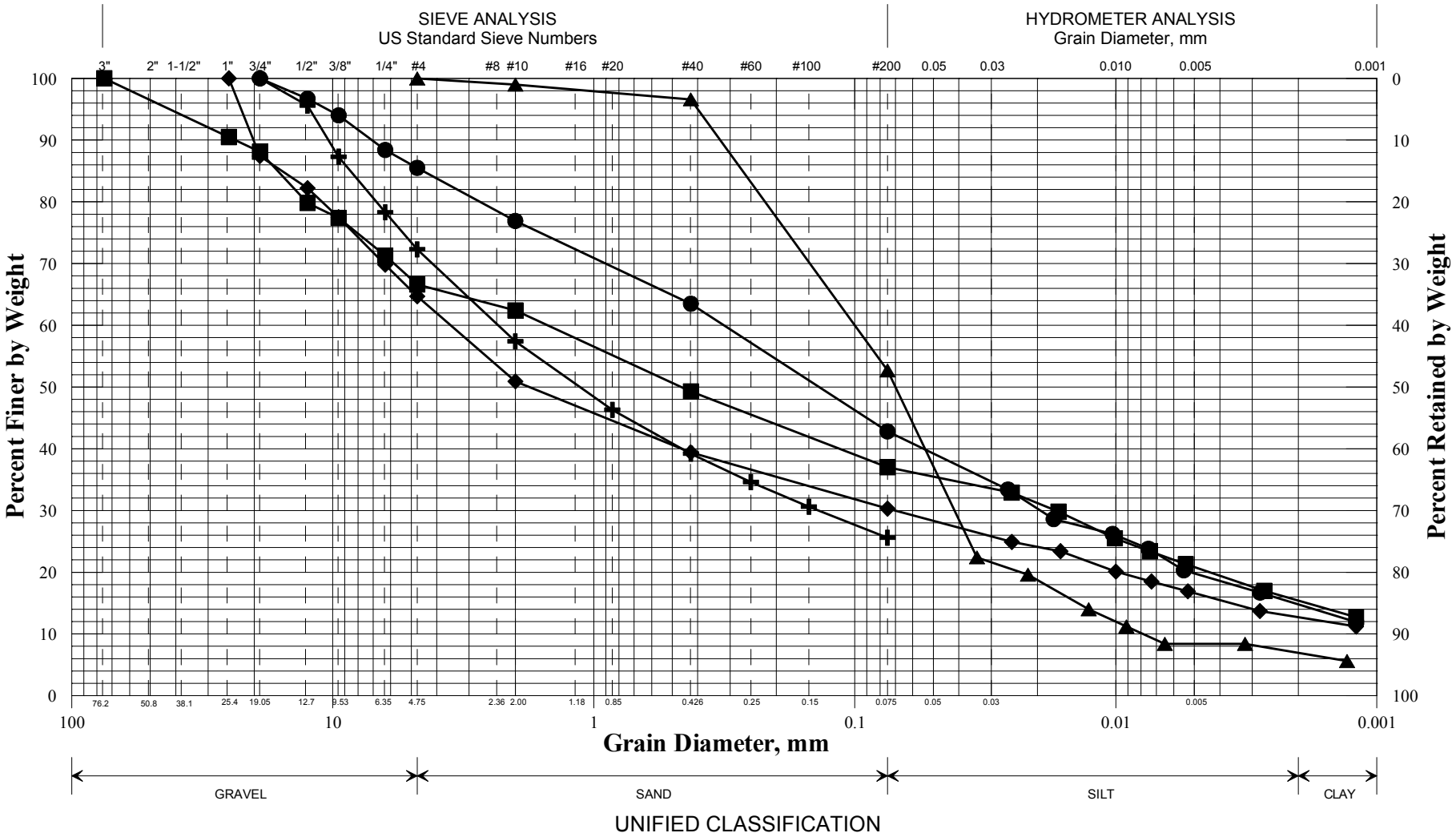
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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-SLMR-101/S1	5+48.1	5.9 RT	1.0-4.0	SAND, some gravel, some silt.	9.1			
◆	BB-SLMR-101/4D	5+48.1	5.9 RT	20.0-22.0	GRAVEL, some sand, little silt, little clay.	8.5			
■	BB-SLMR-101/6D	5+48.1	5.9 RT	35.0-37.0	GRAVEL, some sand, some silt, little clay.	7.4			
●	BB-SLMR-101/8D	5+48.1	5.9 RT	45.0-47.0	SAND, some silt, little gravel, little clay.	8.5			
▲	BB-SLMR-101/9D	5+48.1	5.9 RT	50.0-52.0	Silty SAND, trace clay.	20.4			
✖									

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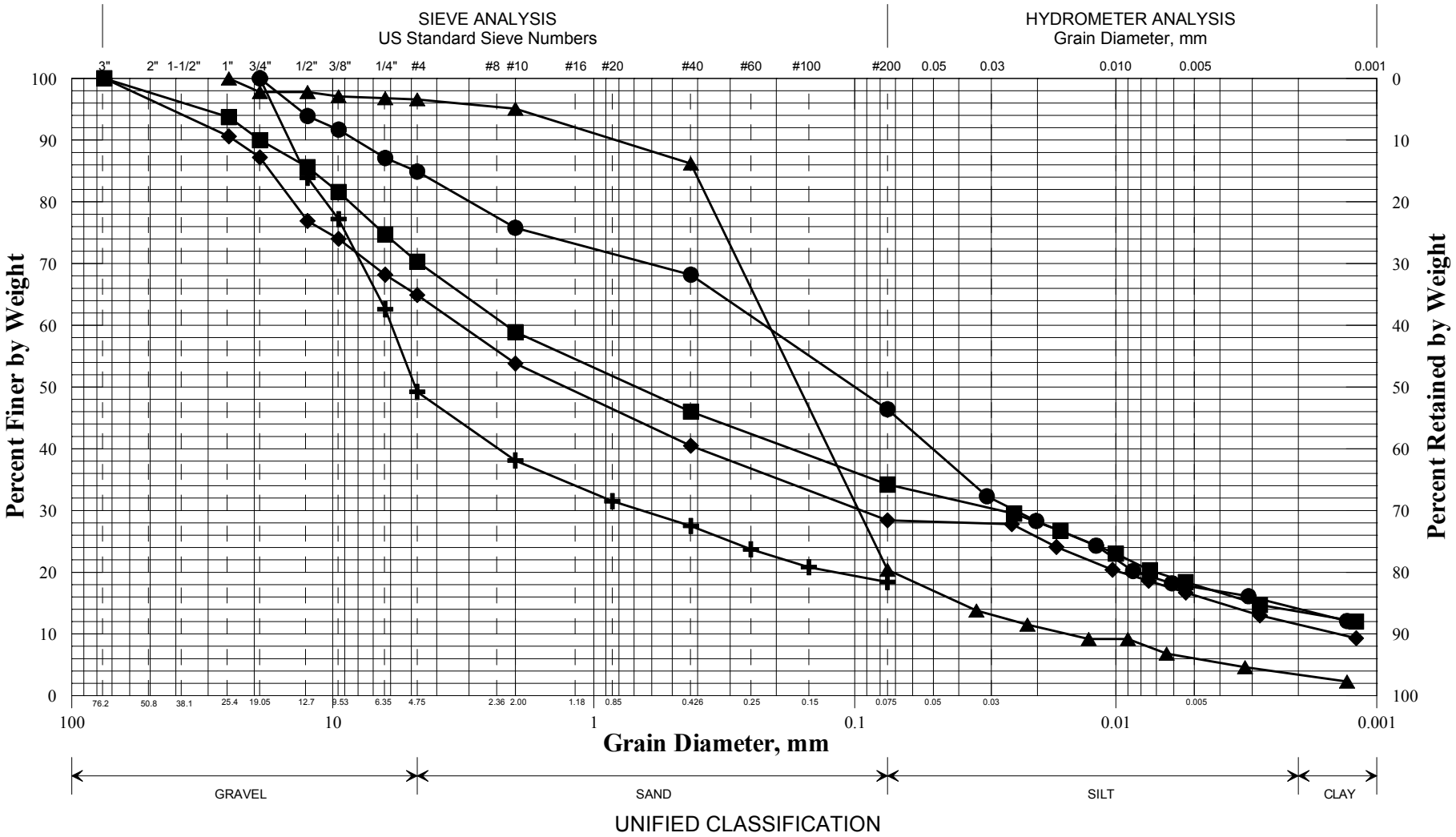
WHITE, TERRY A 5/22/2012

SHEET 1

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-SLMR-102/1D	6+51.6	7.5 LT	1.0-3.0	GRAVEL, some sand, little silt.	7.4			
◆	BB-SLMR-102/4D	6+51.6	7.5 LT	20.0-22.0	SAND, some gravel, little silt, little clay.	7.2			
■	BB-SLMR-102/6D	6+51.6	7.5 LT	30.0-32.0	SAND, some gravel, some silt, little clay.	8.1			
●	BB-SLMR-102/9D	6+51.6	7.5 LT	45.0-47.0	SAND, some silt, little gravel, little clay.	13.2			
▲	BB-SLMR-102/10D	6+51.6	7.5 LT	55.5-57.5	SAND, some silt, trace gravel, trace clay.	20.7			
✖									

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Appendix C

Calculations

Design of H-piles

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

Generalized Bedrock Properties

BB-SLMR-101, R1 RQD= 10%, R2 RQD= 37%
BB-SLMR-102. R1 RQD= 8%, R2 RQD= 33%
Rock Type: Limestone and Shale, hard, fresh to moderately weathered, breaks along moderate to steep bedding

AASHTO LRFD Table C.10.4.6.4-1;
Shale ϕ = 20-27
Limestone ϕ = 34-40

AASHTO Standard Specifications for Bridges 17th Edition, Table 4.4.8.1.2B;
Shale C_o = 1,000-5,100 psi
Limestone C_o = 3,500-42,000 psi

For Design Purposes, use bedrock data from BB-SLMR-102: RQD = 8% and an Unconfined Compressive Strength of 3,500 psi.

Pile Properties

Use the following piles:

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

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

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

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

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

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

Nominal Axial Structural Resistance

Determine equivalent yield resistance

$Q := 1.0$ LRFD Article 6.9.4.2

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

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

Slender element reduction factor, Q, may be required to reduce resistance for 12x53 and 14x73 H-pile sections per LRFD 6.9.4.2.

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

A. Structural Resistance of unbraced segment of pile

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

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

K = effective length factor $K_{eff} := 1.2$

LRFD Table C4.6.2.5-1
Design value for ideal conditions when one end fixed and one end subject to translation.

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

r_s = radius of gyration

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

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

LRFD eq. 6.9.4.1.2-1

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

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

LRFD Article 6.9.4.1.1

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

If Pe/Po > or = 0.44, then:

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

LRFD Eq. 6.9.4.1.1-1

then:

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

12x53
12x74
14x73
14x89
14x117

Factored Axial Structural Resistance at the Strength Limit State

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

$$\phi_{cu} := 0.5$$

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

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

Factored structural compressive resistance, P_r

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

12x53
12x74
14x73
14x89
14x117

LRFD 10.7.3.2.3 - Piles Driven to Hard Rock -

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

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

Nominal Structural Resistance Previously Calculated (lower braced segment):

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

12x53
12x74
14x73
14x89
14x117

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

$$\phi_c := 0.5$$

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

12x53
12x74
14x73
14x89
14x117

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

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

$$\phi_c := 1.0$$

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

12x53
12x74
14x73
14x89
14x117

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

Nominal and Factored Axial Geotechnical Resistance of HP piles

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

Nominal unit bearing resistance of pile point, q_p

Design value of compressive strength of rock core $q_{u\_1} := 3500 \cdot psi$

Spacing of discontinuities $s_d := 3 \cdot in$

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

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

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

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

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

$dd = 1$ OK

K_{sp}

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

$$K_{sp} = \begin{pmatrix} 0.203 \\ 0.203 \\ 0.2 \\ 0.2 \\ 0.2 \end{pmatrix}$$

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

Geotechnical tip resistance.

$$q_{p\_1} := 3 \cdot q_{u\_1} \cdot K_{sp} \cdot dd \qquad q_{p\_1} = \begin{pmatrix} 307 \\ 307 \\ 303 \\ 303 \\ 302 \end{pmatrix} \cdot ksf$$

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

Case I

$$R_{p\_1} := \overrightarrow{\left(q_{p\_1} \cdot A_s\right)} \qquad R_{p\_1} = \begin{pmatrix} 33 \\ 46 \\ 45 \\ 55 \\ 72 \end{pmatrix} \cdot kip$$

ϕ = 1.0

Factored Axial Geotechnical Compressive Resistance - *Strength Limit States*

Resistance factor, end bearing on rock Canadian Geotechnical Society method

$$\phi_{stat} := 0.45$$

LRFD Table 10.5.5.2.3-1

Factored Geotechnical Tip Resistance (R_r)

$$R_{r\_p1} := \phi_{stat} \cdot R_{p\_1} \qquad R_{r\_p1} = \begin{pmatrix} 15 \\ 21 \\ 20 \\ 25 \\ 33 \end{pmatrix} \cdot kip$$

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

Nominal and Factored Axial Geotechnical Resistance of HP piles

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

Nominal unit bearing resistance of pile point, q_p

Design value of compressive strength of rock core

$$q_{u\_1} := 3500 \cdot psi$$

Geotechnical tip resistance.

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

$$q_{p\_1} = \begin{pmatrix} 307 \\ 307 \\ 303 \\ 303 \\ 302 \end{pmatrix} \cdot ksf$$

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

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

$$R_{p\_2} = \begin{pmatrix} 136 \\ 191 \\ 187 \\ 228 \\ 301 \end{pmatrix} \cdot kip$$

ϕ = 1.0

Factored Axial Geotechnical Compressive Resistance - *Strength Limit States*

Resistance factor, end bearing on rock Canadian Geotechnical Society method

$$\phi_{stat} := 0.65$$ LRFD Table 10.5.5.2.3-1

Factored Geotechnical Tip Resistance (R_r)

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

$$R_{r\_p2} = \begin{pmatrix} 88 \\ 124 \\ 122 \\ 148 \\ 196 \end{pmatrix} \cdot kip$$

Drivability Analyses

Ref: LRFD Article 10.7.8

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

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

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

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

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

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

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

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

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

GRLWeap Soil and Pile Model Assumptions

Based on Table 7-1 of this Report, estimated pile lengths at Abutment 1 will be approx. 59 feet.

Minimum length of pile embedment in soil is 58.5 ft.

Assume the Contractor drives pile lengths of 70 ft.

Use constant shaft resistance at 40% - use min. ultimate capacity of 400 kips, so GRLWeap will assign 160 kip shaft resistance (triangular distribution for coarse grain soils, acting on approx. 59' embeded pile portion) to all ultimate capacities analyzed.

Pile Size is 12 x 53

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

State of Maine Dept. Of Transportation
19318 Madawaska Br 12x53 Delmag 19-42

25-Aug-2015
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
400.0	32.51	6.63	6.4	8.76	21.98
420.0	33.88	6.92	7.3	8.98	22.59
440.0	35.16	7.15	8.4	9.17	23.05
460.0	36.28	7.32	9.8	9.33	23.51
480.0	37.47	7.45	11.9	9.39	23.62
490.0	38.09	7.54	13.0	9.45	23.80
500.0	38.71	7.64	14.0	9.51	24.01
505.0	39.01	7.63	14.7	9.54	24.02
510.0	39.31	7.67	15.3	9.56	24.11
520.0	39.88	7.68	16.7	9.62	24.27

Limit blow count to 15 bpi.

$R_{ndr} := 505 \cdot kip$

Strength Limit State

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

$R_{fdr} = 328 \cdot kip$

Extreme and Service Limit States

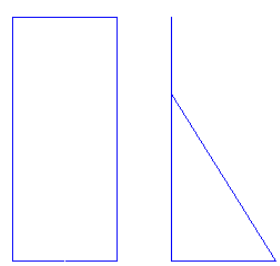
$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 505 \cdot kip$

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer C	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Dampin	0.050	sec/ft
Toe Dampin	0.150	sec/ft
Pile Length	70.00	ft
Pile Penetrat	59.00	ft
Pile Top Are	15.50	in2

Pile Model | Skin Friction Distribution



Res. Shaft = 40 %
(Constant Res. Shaft)

Pile Size is 12 x 74

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

State of Maine Dept. Of Transportation
19318 Madawaska Br 12x74 Delmag 19-42

25-Aug-2015
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
400.0	27.35	4.74	5.4	8.31	19.57
450.0	29.78	5.75	7.2	8.56	20.20
500.0	32.37	6.50	9.3	8.84	20.96
520.0	33.29	6.68	10.4	8.93	21.17
540.0	34.24	6.86	11.6	9.02	21.41
560.0	35.11	7.06	13.0	9.10	21.60
580.0	35.91	7.25	14.6	9.17	21.80
585.0	35.91	7.29	15.5	9.10	21.62
590.0	36.11	7.31	16.0	9.11	21.62
600.0	36.48	7.39	17.1	9.14	21.71

DELMAG D 19-42

Limit Blow counts to 15 bpi

$R_{ndr} := 580 \cdot kip$

Strength Limit State

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

$R_{fdr} = 377 \cdot kip$

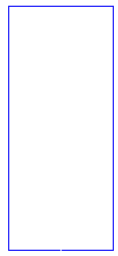
Extreme and Service Limit States

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

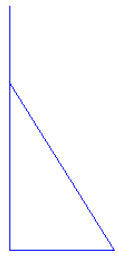
$R_{dr} = 580 \cdot kip$

Efficiency	0.800	
Helmet	2.70	kips
Hammer C	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Dampin	0.050	sec/ft
Toe Dampin	0.150	sec/ft
Pile Length	70.00	ft
Pile Penetrat	59.00	ft
Pile Top Are	21.80	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 40 %
(Constant Res. Shaft)

Pile Size is 14 x 73

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

State of Maine Dept. Of Transportation
19318 Madawaska Br 14x73 Delmag 19-42

25-Aug-2015
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
400.0	27.43	4.89	5.5	8.25	19.49
450.0	30.09	5.95	7.2	8.59	20.36
500.0	32.66	6.70	9.5	8.87	21.05
520.0	33.65	6.93	10.6	8.97	21.36
540.0	34.56	7.12	11.8	9.06	21.57
560.0	35.43	7.35	13.2	9.13	21.81
570.0	35.83	7.45	14.1	9.17	21.90
575.0	36.05	7.48	14.6	9.18	21.92
580.0	36.01	7.54	15.4	9.11	21.75
590.0	36.40	7.59	16.6	9.14	21.78

Limit blow count to 15 bpi

$R_{ndr} := 575 \cdot kip$

Strength Limit State

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

$R_{fdr} = 374 \cdot kip$

Extreme and Service Limit States

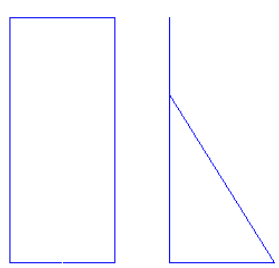
$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 575 \cdot kip$

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer C	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Dampin	0.050	sec/ft
Toe Dampin	0.150	sec/ft
Pile Length	70.00	ft
Pile Penetrat	59.00	ft
Pile Top Are	21.40	in2

Pile Model | Skin Friction Distribution



Res. Shaft = 40 %
(Constant Res. Shaft)

Pile Size is 14 x 89

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

State of Maine Dept. Of Transportation
19318 Madawaska Br 14x89 Delmag 19-42

25-Aug-2015
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
400.0	25.39	3.30	5.1	8.17	18.49
450.0	27.04	3.85	6.6	8.35	18.93
500.0	29.54	4.21	8.1	8.61	19.61
550.0	31.68	4.66	10.1	8.80	20.10
600.0	33.62	5.04	12.8	8.97	20.50
610.0	33.95	5.15	13.4	9.01	20.65
620.0	34.33	5.21	14.2	9.03	20.69
630.0	34.69	5.28	14.9	9.06	20.77
640.0	35.03	5.38	15.6	9.09	20.87
650.0	35.35	5.43	16.6	9.11	20.91

DELMAG D 19-42

Limit to 15 bpi.

$R_{ndr} := 630 \cdot kip$

Strength Limit State

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

$R_{fdr} = 409 \cdot kip$

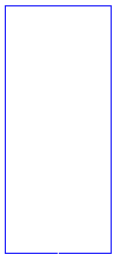
Extreme and Service Limit States

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

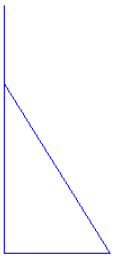
$R_{dr} = 630 \cdot kip$

Efficiency	0.800	
Helmet	2.70	kips
Hammer C	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Dampin	0.050	sec/ft
Toe Dampin	0.150	sec/ft
Pile Length	70.00	ft
Pile Penetrat	59.00	ft
Pile Top Are	26.10	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 40 %
(Constant Res. Shaft)

Pile Size is 14 x 89

The 14 x 89 pile can be driven to the resistances below with a Delmag D 36-32 at a reasonable blow count and level of driving stress. See GRLWEAP results below:
Fuel Setting: Max

State of Maine Dept. Of Transportation
19318 Madawaska Br 14x89 Delmag 36-32

25-Aug-2015
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
400.0	25.98	1.77	3.0	5.66	24.62
600.0	34.58	4.32	7.1	6.64	29.17
700.0	38.92	4.65	11.2	7.03	31.31
750.0	40.88	5.00	14.4	7.21	32.26
755.0	41.02	5.03	14.9	7.23	32.34
760.0	41.24	5.05	15.3	7.24	32.41
770.0	41.31	5.09	16.8	7.20	32.17
780.0	41.61	5.14	17.6	7.23	32.33
790.0	41.90	5.18	18.6	7.26	32.49
800.0	42.22	5.18	19.7	7.28	32.57

DELMAG D 36-32

Limit stress to < 45 ksi. Blow counts > 10 will overstress pile

$R_{ndr} := 755 \cdot kip$

Strength Limit State

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

$R_{fdr} = 491 \cdot kip$

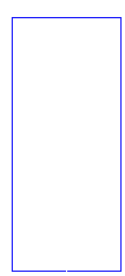
Extreme and Service Limit States

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

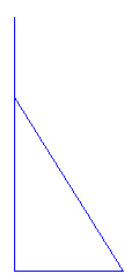
$R_{dr} = 755 \cdot kip$

Efficiency	0.800	
Helmet	2.70	kips
Hammer C	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Dampin	0.050	sec/ft
Toe Dampin	0.150	sec/ft
Pile Length	70.00	ft
Pile Penetrat	59.00	ft
Pile Top Are	26.10	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 40 %
(Constant Res. Shaft)

Pile Size is 14 x 117

The 14 x 117 pile can be driven to the resistances below with a Delmag D 36-32 at a reasonable blow count and level of driving stress. See GRLWEAP results below:
Fuel Setting: 73 % of Max

State of Maine Dept. Of Transportation
19318 Madawaska Br 14x117 Delmag 36-32

25-Aug-2015
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
400.0	26.06	0.69	2.5	6.19	27.51
600.0	31.37	3.25	4.9	7.06	31.17
800.0	39.31	4.69	9.3	7.76	34.73
850.0	40.94	4.95	11.0	7.91	35.52
900.0	42.46	5.04	12.7	8.05	36.34
925.0	43.15	5.08	13.8	8.11	36.60
950.0	43.85	5.11	14.9	8.17	36.86
955.0	43.97	5.13	15.1	8.18	36.97
960.0	44.11	5.13	15.3	8.19	37.06
975.0	44.51	5.15	16.1	8.23	37.22

Limit blow counts to 15 per inch. Increasing fuel overstresses pile at lower capacity

$R_{ndr} := 950 \cdot kip$

Strength Limit State

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

$R_{fdr} = 618 \cdot kip$

Extreme and Service Limit States

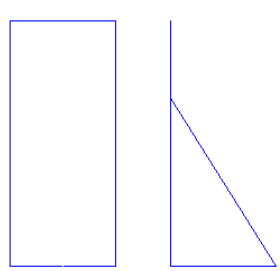
$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 950 \cdot kip$

DELMAG D 36-32

Efficiency	0.800	
Helmet	2.70	kips
Hammer C	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Dampin	0.050	sec/ft
Toe Dampin	0.150	sec/ft
Pile Length	70.00	ft
Pile Penetrat	59.00	ft
Pile Top Are	34.40	in2

Pile Model | Skin Friction Distribution



Res. Shaft = 40 %
(Constant Res. Shaft)

Earth Pressure:

Backfill engineering strength parameters

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

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

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

Cohesion $c_1 := 0 \cdot psf$

Integral Abutment and Wingwall - Passive Earth Pressure - Coulomb Theory

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

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

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

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

For precast IAB abutment against clean sand, silty sand-gravel mixture use δ = 17 - 22, per LRFD Table 3.11.5.3-1

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

$\delta := 19.5 \cdot deg$

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

$K_{p\_coul} = 6.73$

Integral Abutment and Wingwall - Passive Earth Pressure - Rankine Theory

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

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

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

$K_{p\_rank} = 3.25$

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

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

From Design Freezing Index Map: **Stockholm, Maine**
DFI = 2680 degree-days.
Case 1 - coarse grained granular fill soils W=10% (assumed).

Table 5-1 provides a max DFI of 2600 resulting in a frost depth of 109.1 in.

For DFI = 2600 $d1 := 109.1$

$d := 109.1 \cdot in$

Depth of Frost Penetration $d = 109 \cdot in$ $d = 9.1 \cdot ft$

Method 2 - ModBerg Software

Examine foundations placed on coarse grained fill soils

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

--- ModBerg Results ---

Project Location: Caribou, Maine
Air Design Freezing Index = 2462 F-days
N-Factor = 0.80
Surface Design Freezing Index = 1970 F-days
Mean Annual Temperature = 39.2 deg F
Design Length of Freezing Season = 156 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	105.0	10.0	125.0	28	34	2.0	1.6	1,800

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

Total Depth of Frost Penetration = 8.8 ft = 105.0 in.

Recommendation: 9.0 feet for design of foundations constructed on coarse grained soils

BB-101					BB-102				
Depth	**SPT N**		**di**	**di/N**	**Depth**	**SPT N**		**di**	**di/N**
5	37		5	0.14	2	26		2	0.08
10	9		5	0.56	5	17		3	0.18
15	10		5	0.50	10	14		7	0.50
20	76		5	0.07	15	100		1	0.01
30	82		10	0.12	20	46		7	0.15
35	47		5	0.11	25	40		5	0.13
40	38		5	0.13	30	21		5	0.24
45	40		5	0.13	35	37		5	0.14
50	31		5	0.16	40	31		5	0.16
55	34		5	0.15	45	26		5	0.19
60	46		5	0.11	50	13		5	0.38
65	37		2	0.05	55	29		5	0.17
67	100	**Bedrock**	38	0.38	60	100	**Bedrock**	45	0.45
SUM			**100**	**2.08**				**100**	**1.69**

di/di/N **48.14** **di/di/N** **59.16**

SUM	**Nav.**	**53.65**

Conclusion: Site Class C

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

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years

Latitude = 47.040000
Longitude = -068.140000
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)		
0.0	0.085	PGA	- Site Class B
0.2	0.194	Ss	- Site Class B
1.0	0.056	S1	- Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1

Latitude = 47.040000
Longitude = -068.140000
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	0.102	As	- Site Class C
0.2	0.233	SDs	- Site Class C
1.0	0.096	SD1	- Site Class C