MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

CAIN BRIDGE
ROUTES 11 AND 100 OVER TWELVE MILE BROOK
CLINTON, MAINE

Prepared by:
Brandon Slaven
Assistant Geotechnical Engineer

STATE OF MAINE
Laura
Krusinski
8046
LICENSED
PROFESSIONAL ENGINEER

Laura Krusinski, P.E.
Senior Geotechnical Engineer

Kennebec County
WIN 20480.00

Soils Report No. 2015-20
Bridge No. 2117

Fed No. STP-2048(000)
July 30, 2015

Table of Contents

Tables

Sheets

Appendices

GEOTECHNICAL DESIGN SUMMARY

The purpose of this Geotechnical Design Report is to present subsurface information and provide geotechnical design recommendations for the replacement of Cain Bridge which carries Route 11 and 100 over the Twelve Mile Brook in Clinton, Maine. The proposed replacement structure will be a 59-foot single-span, precast, prestressed concrete voided slab superstructure founded on H-pile supported integral abutments. The proposed alignment of the new bridge will closely match the 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. The H-piles shall be designed for all relevant strength, service and extreme limit state load groups. It is recommended that during final design a series of lateral pile resistance analyses using L-Pile® Plus 5.0 (L-Pile) be conducted by the geotechnical engineer to determine if the pile length provided is sufficient to prevent translation of the pile and to evaluate the soil-pile interaction for combined axial and flexure loads and thermal displacements. The resulting bending moment in the pile should be evaluated by the structural engineer. The structural resistance of the piles should then be evaluated for structural compliance with the interaction equation.

If the results of L-Pile analyses indicate that the H-pile design does not achieve fixity or requires a pinned condition at the pile tip, piles may require installation of the pile tips in bedrock sockets or special pile points to improve penetration and friction at the pile tips.

The contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at each abutment. The first pile driven at each abutment shall be dynamically tested to confirm nominal pile resistance and verify preliminary stopping criteria developed by the contractor in the wave equation analysis. 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. Minimum 48-hour restrike tests will be required as part of the pile field quality control program to monitor relaxation within the friable, vertically foliated, bedrock. Final stopping criteria will not be established until the completion of restrike testing.

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

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

Wingwalls – In-line "butterfly" or return "U-shaped", wingwalls may be constructed monolithically with the integral abutments. The walls shall be designed to resist lateral earth pressures, vehicular loads, collision loads, creep and temperature shrinkage deformations, and the additional bending stresses resulting from the wingwall being cantilevered off the abutment. The design of the "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. The design of U-shaped monolithic wingwalls shall, at a minimum, also consider a second load case where the wingwall is subjected to active pressure and to collision loads on wall mounted bridge rail under the extreme limit state.

There are no bearing resistance considerations or special foundation support needed for wingwalls that are cantilevered off the abutment. However, independently supported wall sections shall be embedded a minimum of 6.9 feet for frost protection.

Settlement – The fill soils and native sand and silt deposit encountered in the test borings are loose to medium dense or medium stiff to stiff in consistency. The coarse grained materials are cohesionless and undergo elastic compression when a load greater than the existing overburden pressure is being applied. No significant fills are expected but it is anticipated some modifications to the existing vertical profile will be required. Elastic settlements due to these modifications are 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 will be minimal. Any settlement of the bridge abutments should be due to axial compression of the foundation piles and is anticipated to be less than 0.5 inch.

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 6.9 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 riprap. The top of the riprap will be located at a minimum elevation of 4 feet below bottom of beam. The riprap shall be underlain by Class 1 nonwoven erosion control geotextile and 1 foot thick layer of bedding material.

Seismic Design Considerations – Cain Bridge is a single span structure 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.

Occasional cobbles were encountered in the native sand and silt soils underlying the bridge approaches. Cobbles may also be encountered in the fill soils. There is potential for these obstructions to 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 be taken 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.

Experience with friable, near vertically foliated. phyllite indicates the piles may not encounter abrupt refusal on bedrock. Minimum 48-hour restrike tests are required due to anticipated relaxation of the pile tips in bedrock. Driven piles should not be accepted until the conclusion of 48-hour restrike testing and verification of the achieved nominal resistances. Piles should not be cut-off until after acceptance to avoid splicing and allow for any necessary redriving of test or production piles.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and provide geotechnical design recommendations for the replacement of Cain Bridge which carries Routes 11 and 100 over Twelve Mile Brook in Clinton, Maine. This report presents the 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 Cain Bridge was constructed in 1927 and is a two-span, cast-in-place concrete slab superstructure with a cantilevered sidewalk. Each span length is 19.5 feet. The center pier is a mass concrete pier founded on timber piles. As part of the 1927 construction the granite block abutments of the pre-existing bridge, which bear on soil, were underpinned at the toe with unreinforced concrete, and widened with short concrete abutment sections and new concrete wingwalls on timber piles. The exception is the upstream wingwall of the south abutment which is not on piles and terminates into the pre-existing granite block wingwall. One course of granite stone was removed and a concrete bridge seat constructed. The result is the current mass concrete and granite block abutments.

The bridge is in overall poor condition according to the 2014 Maine Department of Transportation (MaineDOT) Bridge Inspection Report. Significant loss of pointing and shifting of stones is noted at both abutments. Erosion from behind the upstream, stone wingwall has resulted in granite blocks toppling into the stream. The substructures, concrete deck and superstructure are all rated as 4 for “poor”. The bridge has a Sufficiency Rating of 36. The bridge is listed as scour critical and a scour Plan of Action (POA) report was completed in 2011. High water events have required the bridge to be monitored 3 to 5 times a year. MaineDOT bridge inspection photos indicate large amounts of debris is historically caught on the pier nose.

The replacement structure will be a 59-foot single-span, integral abutment bridge. The integral abutments will be founded on H-piles driven to, or within, bedrock. Several replacement options were identified as feasible during preliminary design and are detailed in the Preliminary Design Report (PDR) dated December 5, 2014. The replacement options were constructing both integral abutments behind the existing abutments, constructing one integral abutment in front and one behind the existing abutments, and constructing both integral abutments in front of the existing abutments. Hydraulic considerations dictated a voided concrete slab superstructure with integral abutments placed behind the existing abutments as the selected alternative.

The new Cain Bridge will be located on nearly the same horizontal and vertical alignment as the existing bridge. The replacement bridge will require approximately 235 feet of approach work to match existing conditions. The new bridge will accommodate two (2) 11-foot lanes with a 5-foot sidewalk and a total width of 38 feet. Staged construction will be utilized to maintain a one-lane travelway with the need to accommodate pedestrian traffic being evaluated during final design.

2.0 GEOLOGIC SETTING

Cain Bridge in Clinton crosses Twelve Mile Brook as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of the Waterville Quadrangle, Maine, Open-file No. 85-51 (1986), indicates the surficial soils in the vicinity of the bridge project consist of glaciomarine deposits with nearby contacts to swamp tidal marsh deposits. Glaciomarine deposits, known locally as the Presumpscot Formation, generally consist of clay and silt that washed out of the Late Wisconsinan glacier and accumulated on the ocean floor when the relative sea level was higher than at present. Tidal marsh deposits generally consist of peat, silt, clay, and sand that accumulate in depressions and other poorly drained areas.

The Bedrock Geologic Map of Maine, MGS (1985), cites the bedrock at the proposed bridge site as interbedded pelite and sandstone of the Waterville Formation.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling two (2) test borings terminating with bedrock cores. Boring BB-CTM-101 was drilled south of the existing south abutment and boring BB-CTM-102 was drilled north of the existing north abutment. The test boring locations are shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile.

Test borings BB-CTM-101 and BB-CTM-102 were drilled on June 23 and 24, 2014 by the MaineDOT Drill Crew. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 3 – 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 July of 2013. All N-values discussed in this report are corrected values computed by applying the corresponding average energy transfer factor of 0.867 to the raw field N-values. The hammer efficiency factor (0.867) and both the raw field N-values and the corrected N-values 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 six (6) standard grain size analyses with natural water content and one (1) Atterberg Limits test. 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 3 – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the borings generally consisted of granular fill and native sand and silt underlain by metamorphic sedimentary bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs. A generalized subsurface profile is shown on Sheet 2 – Boring Location Plan and 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. The fill unit is approximately 12.1 feet thick at the boring locations. The fill soils encountered generally consisted of:

- Brown, damp, sand, little to some gravel, little to some silt;
- Brown, moist, sandy silt, trace gravel; and
- Brown, moist, silt, some fine to medium sand.

Corrected SPT N-values in the coarse-grained fill soils ranged from 10 to 20 blows per foot (bpf) indicating that the coarse-grained fill strata is loose to medium dense in consistency. Corrected SPT N-values in the fine-grained fill soils ranged from 6 to 10 bpf indicating the fine-grained fill strata is medium stiff to stiff in consistency.

Two (2) grain size analyses of the fill soils resulted in the soil being classified as A-2-4 or A-4 under the AASHTO Soil Classification System and SM or ML under the Unified Soil

Classification System (USCS). The natural water content of the samples tested ranged from approximately 13 to 15 percent.

5.2 Native Sands and Silts

A layer of native sands and silts was encountered below the fill unit in the test borings. The thickness of the native sand and silt deposit ranged from approximately 15.1 feet in boring BB-CTM-102 to approximately 21.3 feet in boring BB-CTM-102. The deposit encountered generally consisted of:

- Grey, wet, gravelly sand, little to trace silt, occasional cobbles;
- Grey, wet, sand, some to trace silt, little to trace gravel, trace wood fragments;
- Grey, wet, fine sand; and
- Grey, wet, silt, little sand.

Corrected SPT N-values in the native sand layers ranged from 6 to 19 bpf indicating the native sands are loose to medium dense in consistency. One (1) SPT N-value in a silt layer was 9 bpf indicating that subunit is stiff in consistency.

Four (4) grain size analyses of the native sands and silts resulted in the soils being classified as A-1-b, A-2-4, A-3, or A-4 under the AASHTO Soil Classification System and SP-SM, ML, or SM, under the USCS. The moisture contents of the tested samples ranged from approximately 15 to 31 percent. One (1) Atterberg Limits test on a sample of the silt soils resulted in the sample being classified as non-plastic.

5.3 Bedrock

Bedrock was encountered and cored in both borings. Table 1 summarizes approximate depths to bedrock, corresponding approximate top of bedrock elevations and RQD.

Boring	Station	Offset (feet)	Approximate Depth to Bedrock (feet)	Approximate Elevation of Bedrock Surface (feet)	RQD (R1,R2) (%)
BB-CTM-101	14+12.9	8.3 ft Lt	28.7	88.7	0,0
BB-CTM-102	14+79.4	8.9 ft Rt	34.8	81.5	0,0

Table 1 – Summary of Approximate Bedrock Depths and Elevations

The bedrock at the site is identified as grey, aphanitic to fine grained, phyllite, hard, very slightly to slightly weathered, breaks along steeply dipping to near vertical foliation, very close, tight to healed with occasional silty or sandy infilling and calcite stringers. The RQD of the bedrock was determined to be 0 percent correlating to a rock mass quality of very poor. The low RQD is caused in large part by the core breaks along near vertical foliation. Detailed bedrock descriptions and the RQD of each core run are provided on the boring logs on Sheet 3 – Boring Logs and in Appendix A – Boring Logs.

5.4 Groundwater

Groundwater was measured in boring BB-CTM-102 to be approximately 18.0 feet below ground surface (bgs). The water levels measured upon completion of drilling are indicated on the boring logs found in Appendix A. Note that water was introduced into the boreholes during the drilling operations. Therefore, the water levels indicated on the boring logs may not represent stabilized groundwater conditions. Groundwater levels will fluctuate with changes in water levels in the river, seasonal changes, precipitation, runoff, and construction activities.

6.0 FOUNDATION ALTERNATIVES

During preliminary design pile-supported integral abutments were identified as the most cost effective and preferred substructure type. Alternatives to replacement, which included rehabilitation and placement of scour countermeasures, were investigated at a concept level; however, advanced deterioration of the existing substructures precluded rehabilitation or improvement options.

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 Cain Bridge replacement project.

7.1 Integral Abutment H-Piles

Abutments No. 1 and No. 2 will be integral abutments founded on a single row of H-piles. The piles should be end bearing and driven to the required resistance on bedrock or within bedrock. Piles may be HP 12x53, 12x74, 14x73, 14x89, or 14x117 depending on the factored design axial loads. H-piles should be 50 ksi, Grade A572 steel. The piles should be oriented for weak axis bending. Piles should be fitted with pile tips conforming to MaineDOT Standard Specification 711.10 to protect section ends, improve friction, and increase bearing area at the pile tip. If the results of a L-Pile® Plus 5.0 (L-Pile) analyses indicate that the H-pile design does not achieve fixity or requires a pinned condition at the pile tip, piles may require installation of the pile tips in bedrock sockets or special pile points to improve penetration and friction at the pile tips.

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

Location	Estimated Bottom Elevation of Proposed Abutment (feet)	Interpolated Top of Bedrock Elevation at Proposed Centerline (feet)	Estimated Pile Lengths[1] (feet)
Abutment No. 1	109.1	88.7	20.4
Abutment No. 2	108.5	81.5	27.0

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

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

7.1.1 Strength Limit State Design

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

- compressive axial geotechnical resistance of individual piles bearing on bedrock,
- drivability resistance of individual piles driven to bedrock,
- structural resistance of individual piles in axial compression, 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 AASHTO LRFD Bridge Design Specifications 7th Edition (LRFD) Article 6.5.4.2, at the strength limit state, the axial resistance factor $\phi_c = 0.50$ (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

[1] Pile lengths do not include embedment into the pile cap or potential penetration into fractured bedrock.

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 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 sections for the strength limit state is provided in Table 3. Supporting calculations are provided in Appendix C – Calculations.

Pile Section	Strength Limit State Factored Axial Pile Resistance			
	Structural Resistance[2] ϕ_c=0.50 (kips)	Controlling Geotechnical Resistance[3] ϕ_c=0.50 (kips)	Drivability Resistance φ_{dyn} = 0.65 (kips)	Governing Axial Pile Resistance (kips)
HP 12 x 53	385[4]	385[4]	399	385[4]
HP 12 x 74	542	542	618	542
HP 14 x 73	533[4]	533[4]	585	533[4]
HP 14 x 89	650	650	702	650
HP 14 x 117	857	857	777 (996)[5]	857

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. The estimated factored axial pile resistances from the drivability analyses for the H-pile shapes driven with a Delmag D19-42 hammer are greater than the controlling factored axial structural resistance per LRFD Article 10.7.3.2.3. Therefore, the recommended governing resistances for pile design are the structural compressive resistances provided in the rightmost column "Governing Axial Pile Resistance (kips)" in Table 3, above. The maximum applied factored axial pile load 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.

---

[2]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.
[3]Based on guidance in LRFD Article 10.7.3.2.3., *Piles Driven to Hard Rock*
[4] Does not consider resistance factors of slender elements. 12x53 and 14x73 H-pile sections require additional reductions based upon structural performance considerations.
[5] Estimated resistances obtained by driving with a Delmag D19-42. Estimated resistance obtained by driving with a Delmag D36-32 shown in parentheses.

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 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 ϕ = 1.0, with the exception of uplift of piles, for which the resistance factor, φ_{up}, shall be 0.80 or less per LRFD Article 10.5.5.3.2.

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

Pile Section	Extreme and Service Limit State Factored Axial Pile Resistance			
	Structural Resistance (normal conditions)[6] ϕ_c=1.0 (kips)	Controlling Geotechnical Resistance ϕ_c=1.0[7] (kips)	Drivability Resistance φ = 1.0 (kips)	Governing Axial Pile Resistance (kips)
HP 12 x 53	771[8]	771[8]	614	614
HP 12 x 74	1084	1084	950	950
HP 14 x 73	1066[8]	1066[8]	900	900
HP 14 x 89	1300	1300	1080	1080
HP 14 x 117	1714	1714	1195 (1532)[9]	1532

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

LRFD Article 10.7.3.2.3 states that the nominal axial compressive resistance of piles driven to hard rock is typically controlled by the structural resistance with a resistance factor for severe driving conditions applied. However, for the site conditions at both abutments, the estimated factored axial pile resistances from the drivability analyses for the H-pile sections are less than the nominal structural resistances and controlling factored axial structural

[6] Normal conditions consider no soil loss due to scour. Nominal structural resistances were calculated for a braced pile segment using a resistance factor, ϕ = 1.0. Factored structural resistances should be calculated for upper and lower unbraced pile segments determined by L-Pile analyses.
[7] Based on guidance in LRFD Article 10.7.3.2.3., *Piles Driven to Hard Rock.*
[8] Does not consider resistance factors of slender elements. 12x53 and 14x73 H-pile sections require additional reductions based upon structural performance considerations.
[9] Estimated resistances obtained by driving with a Delmag D19-42. Estimated resistance obtained by driving with a Delmag D36-32 shown in parentheses.

resistance per LRFD Article 10.7.3.2.3. 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 should not exceed the governing factored pile resistance shown in Table 4.

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

Soil Layer	Approx. thickness of Soil Layer (feet)	Water Table Condition	Effective Unit Weight lbs/in^3 (lbs/ft^3)	k_s (lb/in^3)	Internal Angle of Friction
Sand Fill (medium dense)	12.1	Above	0.0723 (125)	90	34°
Native Sands (loose)	4.4	Above	.0665 (115)	25	30°
Native Sands (loose)	1.5	Below	0.0365 (63)	20	30°
Native Sands (medium dense)	8.2	Below	0.0365 (63)	60	32°

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

Soil Layer	Approx. Thickness of Soil Layer (feet)	Water Table Condition	Effective Unit Weight lbs/in^3 (lbs/ft^3)	k_s (lb/in^3)	Cohesion psi (psf)	E_{50}	Internal Angle of Friction
Sand Fill (loose)	2	Above	0.0665 (115)	25	-	-	30°
Silt Fill (medium stiff to stiff)	10.1	Above	0.0694 (120)	240	6.95 (1000)	0.009	-
Native Sands (loose)	4.5	Above	0.0665 (115)	25	-	-	30°
Native Sands (loose)	5.5	Below	0.0307 (53)	20	-	-	30°
Native Silt (stiff)	3	Below	0.0336 (58)	500	10.42 (1500)	0.007	-
Native Sands (loose)	8.3	Below	0.0307 (53)	20	-	-	32°

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 dynamic pile tests with signal matching. The first pile driven at each abutment should be dynamically tested to confirm nominal pile resistance and verify preliminary stopping criteria developed by the contractor in the wave equation analysis. The pile driving acceptance criteria developed shall prevent pile damage. Minimum 48 hour restrike tests will be required for test piles due to the anticipated relaxation of the friable, vertically foliated, bedrock near the pile tip. Production piles may be driven to the verified preliminary stopping criteria, but should not be accepted or cut-off until completion of restrike testing and establishment of a final stopping criteria. Care should be taken to ensure test and production piles are of sufficient length and condition to be driven to the final stopping criteria. The contractor may choose to not install production piles until a final driving criterion is established from dynamic test pile results and restrike test results. Additional dynamic tests may be required if pile behavior indicates the pile is not seated firmly on bedrock or if piles “walk” out of position.

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

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

7.2 Integral Abutment Design

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

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

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

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

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

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

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

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

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

7.3 Wingwalls

In-line "butterfly" wingwalls, or return "U-shape" wingwalls, may be constructed monolithically with the integral abutments. The monolithic 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, collision loads, and creep and temperature and shrinkage deformations. The design of monolithic wingwalls shall account for the additional bending stresses resulting from the wingwall being cantilevered off the abutment. For monolithic U-shaped wingwalls a chamfer, typically 1 foot, should be used between the abutment and the wingwalls to minimize concrete shrinkage cracking caused by the abrupt change in thickness at the connection.

The design of the "butterfly" wingwalls shall at a minimum consider a load case at the service limit state where the wingwall is subjected to passive earth pressure to account for the bridge moving laterally and pushing the wingwall into the fill. Calculation of passive earth pressures may assume a Rankine passive earth pressure coefficient, K_p, of 3.25 assuming small wingwall movements. A load factor for passive earth pressure is not specified in LRFD; use a maximum load factor (γ_{EH}) of 1.50 to calculate factored passive earth pressures.

The design of U-shaped monolithic wingwalls shall at a minimum also consider a second load case where the wingwall is subjected to active pressure and to collision loads on wall mounted bridge rail under the extreme limit state. Calculation of active earth pressure shall use the Rankine active earth pressure coefficient, K_a, of 0.31 assuming a level backslope and 0.52 for a 2H:1V backslope. See Appendix C – Calculations for supporting documentation.

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 provided 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 fill soils and native sand and silt deposit encountered in the test borings are loose to medium dense or stiff in consistency. The coarse grained materials are cohesionless and undergo elastic compression where a load greater than the existing overburden pressure is being applied. No significant fills are expected but it is anticipated some modifications to the existing vertical profile will be required. Elastic settlements due to these modifications are 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 minimal.

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

7.5 Frost Protection

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

Foundations placed on fill side slopes should be designed with an appropriate embedment for frost protection. According to BDG Figure 5-1, Maine Design Freezing Index Map, Clinton has a design freezing index of approximately 1650 F-degree days. An assumed water content of 10% was used for coarse grained soils. These components correlate to a frost depth of approximately 7.1 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Clinton was assigned a design freezing index of approximately 1395 F-degree days, for Waterville, 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 6.7 feet. Based on an average of these results, it is recommended foundations be designed with an embedment of approximately 6.9 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 soil samples of the native sand deposit 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} = 0.54 mm (medium sand)
- Average diameter of particle at 95 percent passing, D_{95} = 5.16 mm (fine gravel)
- Soil Classification AASHTO Soil Type A-3.

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.

Preliminary scour depths for the design flood (Q_{100}) event for the existing structure indicate scour will leave 9 feet of soil[10] overlying bedrock at Abutment No. 1 destabilizing the pile group if left unprotected. The PDR indicates the bridge approach slopes and the abutment slopes should be armored with riprap. Refer to MaineDOT BDG Section 2.3.11.3 for information regarding riprap design.

Plain riprap shall conform to MaineDOT Standard Specification 703.26. The toe of the riprap section shall be constructed 1 foot below the streambed elevation. The riprap section shall be underlain by a 1 foot thick layer of bedding material conforming to item number 703.19 of the Standard Specification and Class 1 nonwoven erosion control geotextile per Standard Details 610(02) through 610(03). Typically the top of the riprap is located at the Q_{50} elevation. To minimize stream and property impacts at this project site, the top of the riprap may be located 4 feet below the bottom of the beams.

[10] Scour Plan of Action prepared by CHA Consulting, 8/30/2011.

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)	0.074g
Acceleration Coefficient (A_S)	0.118g
S_{DS} (Period = 0.2 sec)	0.251g
S_{D1} (Period = 1.0 sec)	0.110g
Site Class	D
Seismic Zone	1

Table 8 – Seismic Design Parameters

In conformance with LRFD Articles 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.

Occasional cobbles were encountered in the native sand and silt soils underlying the bridge approaches. Cobbles may also be encountered in the fill soils. There is potential for these obstructions to 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.

Experience with friable, nearly vertically foliated phyllite indicates the piles may not encounter abrupt refusal on bedrock. Minimum 48-hour restrike tests are required due to anticipated relaxation of the pile tips in bedrock. Driven piles should not be accepted until the conclusion of 48-hour restrike testing and verification of the achieved nominal resistances. Piles should not be cut-off until after acceptance to avoid splicing and allow for any necessary redriving of test or production piles.

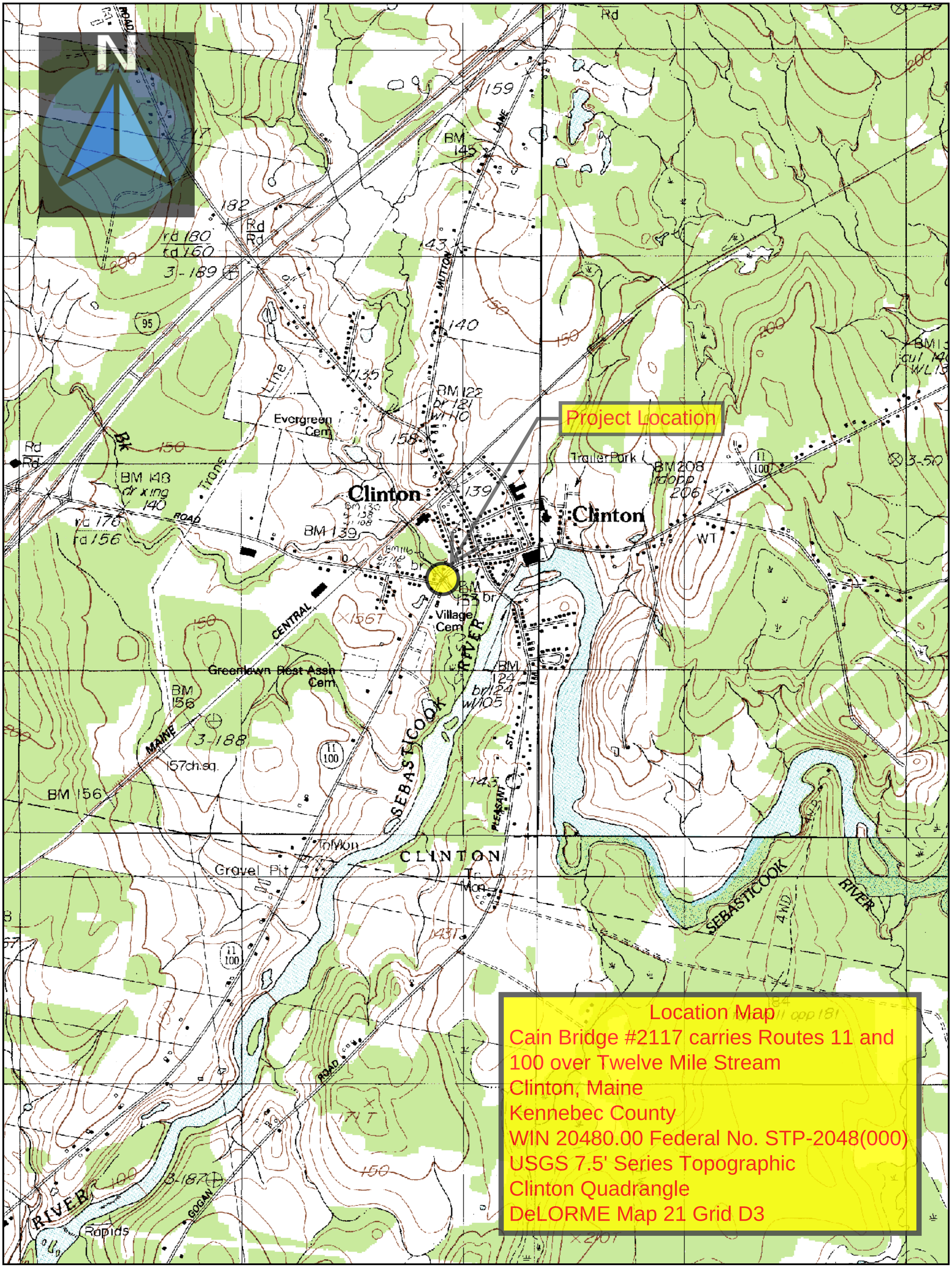
8.0 CLOSURE

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

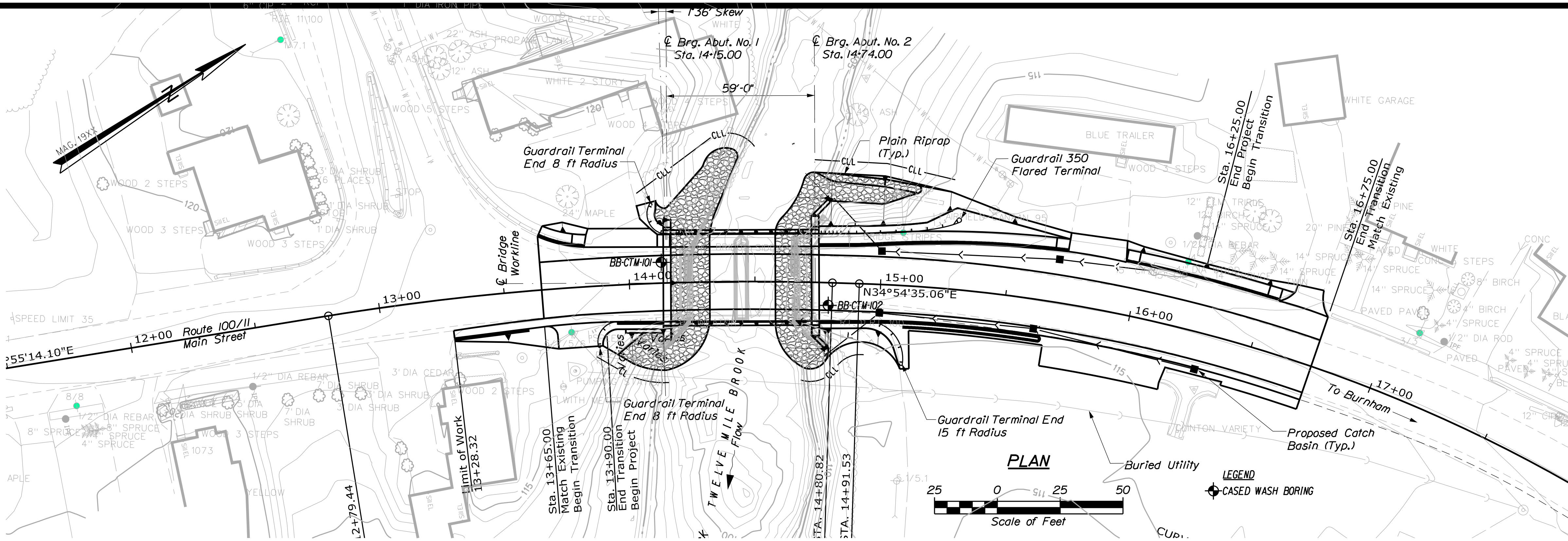
In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes 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.

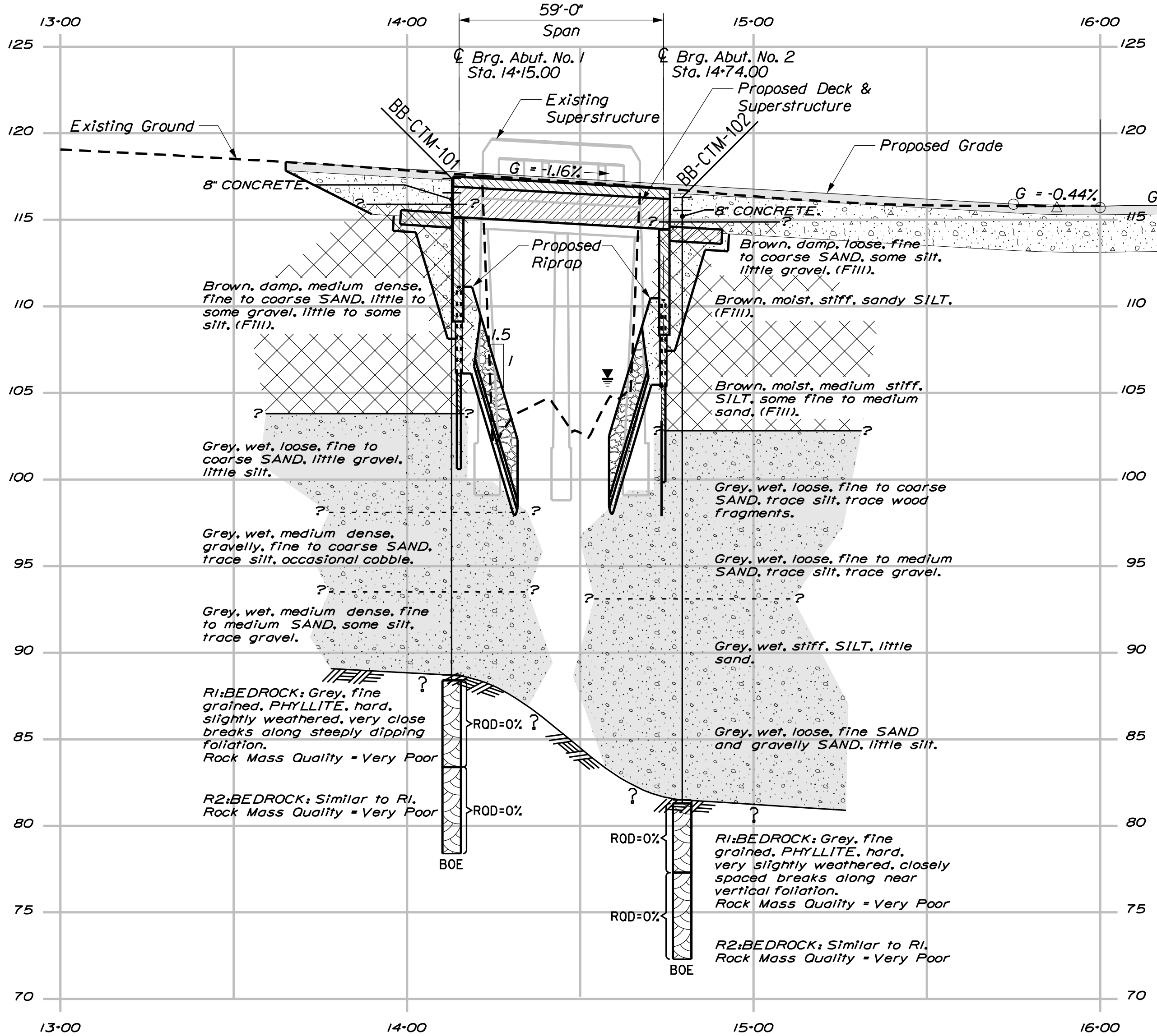
It is also recommend that the geotechnical engineer be provided the opportunity for a general review of the final design and specifications in order that the earthwork and foundation recommendations may be properly interpreted and implemented in the design.

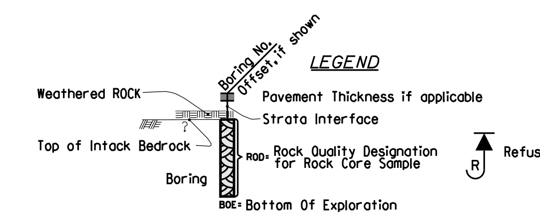
Sheets



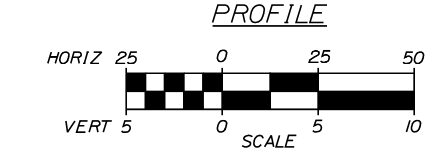
Map Scale 1:24000







PROFILE



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

STP-2048(000)

WIN 20480.00

BRIDGE PLANS

BRIDGE NO. 2117

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

SIGNATURE

P.E. NUMBER

DATE

CLINTON
CAIN BRIDGE
TWELVE MILE BROOK
KENNEBEC COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

2

OF 3

Filename: ...\MSTA\007_BORING LOGS1.dgn Division: GEOTECH Username: Brandon.Slaven Date:7/31/2015

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Cain Bridge #2117 carries Routes 11 & 100 over Twelve Mile Stream
Location: Clinton, Maine

Boring No.: BB-CTM-101

WIN: 20480.00

Driller:	MaineDOT	Elevation (ft.)	117.4	Auger ID/OD:	5" Solid Stem
Operator:	Giles/Daggett/Giles	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	6/24/2014; 07:30-13:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	14+12.9, 8.3 ft Lt.	Casing ID/OD:	NW	Water Level*:	None Observed
Hammer Efficiency Factor: 0.867		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA			10" PAVEMENT.	
								116.57		0.83 — 8" CONCRETE	
								115.90		1.50	
	1D	24/15	2.00 - 4.00	3/6/4/4	10	14				Brown, damp, medium dense, fine to coarse SAND, little gravel, little silt, (Fill).	
5	2D	24/16	5.00 - 7.00	2/4/4/4	8	12				Brown, damp, medium dense, fine to coarse SAND, some gravel, little silt, (Fill).	
10	3D	24/17	10.00 - 12.00	4/6/8/8	14	20				Brown, damp, medium dense, fine to coarse SAND, some silt, some gravel, (Fill).	G#243180 A-2-4, SM WC=14.5%
								103.80		13.60	
15	4D	24/18	15.00 - 17.00	2/3/4/5	7	10	10			Grey, wet, loose, fine to coarse SAND, little gravel, little silt.	G#243181 A-1-b, SM WC=15.4%
							9				
							17				
							16				
							17	97.90		19.50	
20	5D	24/13	20.00 - 22.00	5/6/7/7	13	19	17			Grey, wet, medium dense, gravelly, fine to coarse SAND, trace silt, occasional cobble.	
							27				
							38				
							36				
							35	93.40		24.00	
25	6D	24/18	25.00 - 27.00	3/3/5/6	8	12	21			Grey, wet, medium dense, fine to medium SAND, some silt, trace gravel.	G#243182 A-2-4, SM WC=22.4%
							25				
							31				
							a150			a150 blows for 0.7 ft.	
								88.70		28.70	
	R1	60/60	29.00 - 34.00	RQD = 0%			NQ-2			Top of Bedrock at Elev. 88.7 ft. Roller Coned ahead to 29.0 ft bgs. R1:Bedrock: Grey, fine grained to aphanitic, PHYLLITE, hard, slightly weathered, breaks are steeply dipping along foliation, very close, tight to healed, some joints with silty or sandy infilling, calcite stringers. Waterville Formation. Rock Mass Quality = Very Poor. R1:Core Times (min:sec) 29.0-30.0 ft (6:15) 30.0-31.0 ft (6:48) 31.0-32.0 ft (6:34) 32.0-33.0 ft (7:05) 33.0-34.0 ft (8:30) 100% Recovery	
30											
	R2	60/60	34.00 - 39.00	RQD = 0%						R2:Bedrock: Similar to R1, except 6 inch quartz vein in end of R2. Rock Mass Quality = Very Poor. R2:Core Times (min:sec) 34.0-35.0 ft (6:00) 35.0-36.0 ft (5:20) 36.0-37.0 ft (7:10) 37.0-38.0 ft (13:00) 38.0-39.0 ft (14:00) 100% Recovery Core Blocked	
35											
								78.40		39.00 — Bottom of Exploration at 39.00 feet below ground surface.	
40											
45											
50											

Remarks:

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

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

Page 1 of 1

Boring No.: BB-CTM-101

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Cain Bridge #2117 carries Routes 11 & 100 over Twelve Mile Stream
Location: Clinton, Maine

Boring No.: BB-CTM-102

WIN: 20480.00

Driller:	MaineDOT	Elevation (ft.)	116.3	Auger ID/OD:	5" Solid Stem
Operator:	Giles/Daggett/Giles	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	6/23/2014; 10:30-15:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	14+79.4, 8.9 ft Rt.	Casing ID/OD:	NW	Water Level*:	18.0 ft bgs.
Hammer Efficiency Factor: 0.867		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA			9" PAVEMENT.	
								115.55		0.75 — 8" CONCRETE.	
								114.88		1.42	
	1D	24/16	2.00 - 4.00	4/3/4/4	7	10				Brown, damp, loose, fine to coarse SAND, some silt, little gravel, (Fill).	
5	2D	24/14	5.00 - 7.00	3/3/4/4	7	10				Brown, moist, stiff, sandy SILT, trace gravel, (Fill).	G#243183 A-4, ML WC=13.1%
10	3D	24/13	10.00 - 12.00	2/2/2/2	4	6				Brown, moist, medium stiff, SILT, some fine to medium sand, (Fill).	
								102.80		13.50	
15	4D	24/16	15.00 - 17.00	1/2/2/3	4	6	12			Grey, wet, loose, fine to coarse SAND, trace silt, trace wood fragments.	
							11				
							16				
							18				
							17				
20	5D	24/14	20.00 - 22.00	2/2/2/4	4	6	7			Grey, wet, loose, fine to medium SAND, trace silt, trace gravel.	G#243184 A-3, SP-SM WC=22.3%
							11				
							14				
							15	92.80		23.50	
							21				
25	6D	24/13	25.00 - 27.00	5/3/3/4	6	9	10			Grey, wet, stiff, SILT, little sand.	G#243185 A-4, ML WC=31.3% Non-Plastic
							15				
							23				
							27				
							29				
30	7D/A	24/18	30.00 - 32.00	2/2/4/7	6	9	21			7D (30.0-31.5 ft) Grey, wet, loose, fine SAND.	
							31			7D/A (31.5-32.0 ft) Grey, wet, loose, gravelly, fine to coarse SAND, little silt.	
							36				
							127				
							a300			a300 blows for 0.8 ft.	
								81.50		34.80	
35	R1	48/46	35.00 - 39.00	RQD = 0%			NQ-2			Top of Bedrock at Elev. 81.5 ft. Roller Coned ahead to 35.0 ft bgs. R1:Bedrock: Grey, fine grained, PHYLLITE, hard, very slightly weathered, breaks along near vertical foliation, very close, tight, a few surfaces oxidized/ discolored. Waterville Formation. Rock Mass Quality = Very Poor. R1:Core Times 5:38) 36.0-37.0 ft (4:33) 37.0-38.0 ft (4:20) 38.0-39.0 ft (6:00) 100% Recovery Core Blocked	
	R2	60/18	39.00 - 44.00	RQD = 0%						R2: Bedrock: Similar to R1. Rock Mass Quality = Very Poor. R2:Core Times (min:sec) 39.0-40.0 ft (7:35) 40.0-41.0 ft (5:50) 41.0-42.0 ft (14:00) 42.0-43.0 ft (6:33) 43.0-44.0 ft (6:30) 30% Recovery	
40											
								72.30		44.00 — Bottom of Exploration at 44.00 feet below ground surface.	
45											
50											

Remarks:

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

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

Page 1 of 1

Boring No.: BB-CTM-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2048(000)

WIN 20480.00

BRIDGE PLANS

BRIDGE NO. 2117

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

SIGNATURE

P.E. NUMBER

DATE

CLINTON

CAIN BRIDGE
TWELVE MILE BROOK
KENNEBEC COUNTY

BORING LOGS

SHEET NUMBER

3

OF 3

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 thumbnail
Hard	>30	over 4000	Indented by thumbnail with difficulty

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

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

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

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Cain Bridge #2117 carries Routes 11 & 100 over Twelve Mile Stream
Location: Clinton, Maine

Boring No.: BB-CTM-101
PIN: 20480.00

Driller:	MaineDOT	**Elevation (ft.)**	117.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Giles/Daggett/Giles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/24/2014; 07:30-13:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+12.9, 8.3 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.867	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	116.57		10" PAVEMENT. 0.83	
								115.90		8" CONCRETE 1.50	
	1D	24/15	2.00 - 4.00	3/6/4/4	10	14				Brown, damp, medium dense, fine to coarse SAND, little gravel, little silt, (Fill).	
5	2D	24/16	5.00 - 7.00	2/4/4/4	8	12				Brown, damp, medium dense, fine to coarse SAND, some gravel, little silt, (Fill).	
10	3D	24/17	10.00 - 12.00	4/6/8/8	14	20				Brown, damp, medium dense, fine to coarse SAND, some silt, some gravel, (Fill).	G#243180 A-2-4, SM WC=14.5%
								103.80		13.60	
15	4D	24/18	15.00 - 17.00	2/3/4/5	7	10	10			Grey, wet, loose, fine to coarse SAND, little gravel, little silt.	G#243181 A-1-b, SM WC=15.4%
							9				
							17				
							16				
							17	97.90		19.50	
20	5D	24/13	20.00 - 22.00	5/6/7/7	13	19	17			Grey, wet, medium dense, gravelly, fine to coarse SAND, trace silt, occasional cobble.	
							27				
							38				
							36	93.40		24.00	
25							35				

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.

Boring No.: BB-CTM-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Cain Bridge #2117 carries Routes 11 & 100 over Twelve Mile Stream
Location: Clinton, Maine

Boring No.: BB-CTM-101
PIN: 20480.00

Driller:	MaineDOT	**Elevation (ft.)**	117.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Giles/Daggett/Giles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/24/2014; 07:30-13:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+12.9, 8.3 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.867	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/18	25.00 - 27.00	3/3/5/6	8	12	21			Grey, wet, medium dense, fine to medium SAND, some silt, trace gravel.	G#243182 A-2-4, SM WC=22.4%
							25				
							31				
							a150	88.70		a150 blows for 0.7 ft. —28.70	
	R1	60/60	29.00 - 34.00	RQD = 0%			NQ-2			Top of Bedrock at Elev. 88.7 ft. Roller Coned ahead to 29.0 ft bgs. R1:Bedrock: Grey, fine grained to aphanitic, PHYLLITE, hard, slightly weathered, breaks are steeply dipping along foliation, very close, tight to healed, some joints with silty or sandy infilling, calcite stringers. Waterville Formation. Rock Mass Quality = Very Poor R1:Core Times (min:sec) 29.0-30.0 ft (6:15) 30.0-31.0 ft (6:48) 31.0-32.0 ft (6:34) 32.0-33.0 ft (7:05) 33.0-34.0 ft (8:30) 100% Recovery	
30											
	R2	60/60	34.00 - 39.00	RQD = 0%						R2:Bedrock: Similar to R1, except 6 inch quartz vein in end of R2. Rock Mass Quality = Very Poor. R2:Core Times (min:sec) 34.0-35.0 ft (6:00) 35.0-36.0 ft (5:20) 36.0-37.0 ft (7;10) 37.0-38.0 ft (13;00) 38.0-39.0 ft (14:00) 100% Recovery	
35											
								78.40		Core Blocked —39.00	
40										**Bottom of Exploration at 39.00 feet below ground surface.**	
45											
50											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Cain Bridge #2117 carries Routes 11 & 100 over Twelve Mile Stream

Location: Clinton, Maine

Boring No.: BB-CTM-102

PIN: 20480.00

Driller:	MaineDOT	**Elevation (ft.)**	116.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Giles/Daggett/Giles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/23/2014; 10:30-15:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+79.4, 8.9 ft Rt.	**Casing ID/OD:**	NW	**Water Level*:**	18.0 ft bgs.
Hammer Efficiency Factor:	0.867	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	115.55		9" PAVEMENT. 0.75	
								114.88		8" CONCRETE. 1.42	
	1D	24/16	2.00 - 4.00	4/3/4/4	7	10				Brown, damp, loose, fine to coarse SAND, some silt, little gravel, (Fill).	
5	2D	24/14	5.00 - 7.00	3/3/4/4	7	10				Brown, moist, stiff, sandy SILT, trace gravel, (Fill).	G#243183 A-4, ML WC=13.1%
10	3D	24/13	10.00 - 12.00	2/2/2/2	4	6				Brown, moist, medium stiff, SILT, some fine to medium sand, (Fill).	
								102.80		13.50	
15	4D	24/16	15.00 - 17.00	1/2/2/3	4	6	12			Grey, wet, loose, fine to coarse SAND, trace silt, trace wood fragments.	
							11				
							16				
							18				
							17				
20	5D	24/14	20.00 - 22.00	2/2/2/4	4	6	7			Grey, wet, loose, fine to medium SAND, trace silt, trace gravel.	G#243184 A-3, SP-SM WC=22.3%
							11				
							14				
							15	92.80		23.50	
							21				
25											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Cain Bridge #2117 carries Routes 11 & 100 over Twelve Mile Stream
Location: Clinton, Maine

Boring No.: BB-CTM-102
PIN: 20480.00

Driller:	MaineDOT	**Elevation (ft.)**	116.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Giles/Daggett/Giles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/23/2014; 10:30-15:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+79.4, 8.9 ft Rt.	**Casing ID/OD:**	NW	**Water Level*:**	18.0 ft bgs.
Hammer Efficiency Factor:	0.867	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/13	25.00 - 27.00	5/3/3/4	6	9	10			Grey, wet, stiff, SILT, little sand.	G#243185 A-4, ML WC=31.3% Non-Plastic
							15				
							23				
							27				
							29				
30	7D/A	24/18	30.00 - 32.00	2/2/4/7	6	9	21			7D (30.0-31.5 ft) Grey, wet, loose, fine SAND.	
							31			7D/A (31.5-32.0 ft) Grey, wet, loose, gravelly, fine to coarse SAND, little silt.	
							36				
							127				
							a300	81.50		a300 blows for 0.8 ft. 34.80	
35	R1	48/46	35.00 - 39.00	RQD = 0%			NQ-2			Top of Bedrock at Elev. 81.5 ft. Roller Coned ahead to 35.0 ft bgs. R1:Bedrock: Grey, fine grained, PHYLLITE, hard, very slightly weathered, breaks along near vertical foliation, very close, tight, a few surfaces oxidized/discolored. Waterville Formation. Rock Mass Quality = Very Poor R1:Core Times 5:38) 36.0-37.0 ft (4:33) 37.0-38.0 ft (4:20) 38.0-39.0 ft (6:00) 100% Recovery Core Blocked	
	R2	60/18	39.00 - 44.00	RQD = 0%						R2: Bedrock: Similar to R1. Rock Mass Quality = Very Poor. R2:Core Times (min:sec) 39.0-40.0 ft (7:35) 40.0-41.0 ft (5:50) 41.0-42.0 ft (14:00) 42.0-43.0 ft (6:33) 43.0-44.0 ft (6:30) 30% Recovery	
40											
								72.30		44.00 **Bottom of Exploration at 44.00 feet below ground surface.**	
45											
50											

Remarks:

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

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

Boring No.: BB-CTM-102

Appendix B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Clinton Work Number: 20480.00

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-CTM-101, 3D	14+12.9	8.3 Lt.	10.0-12.0	243180	1	14.5			SM	A-2-4	II
BB-CTM-101, 4D	14+12.9	8.3 Lt.	15.0-17.0	243181	1	15.4			SM	A-1-b	II
BB-CTM-101, 6D	14+12.9	8.3 Lt.	25.0-27.0	243182	1	22.4			SM	A-2-4	II
BB-CTM-102, 2D	14+79.4	8.9 Rt.	5.0-7.0	243183	1	13.1			ML	A-4	IV
BB-CTM-102, 5D	14+79.4	8.9 Rt.	20.0-22.0	243184	1	22.3			SP-SM	A-3	0
BB-CTM-102, 6D	14+79.4	8.9 Rt.	25.0-27.0	243185	1	31.3	-N	P-	ML	A-4	IV

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.

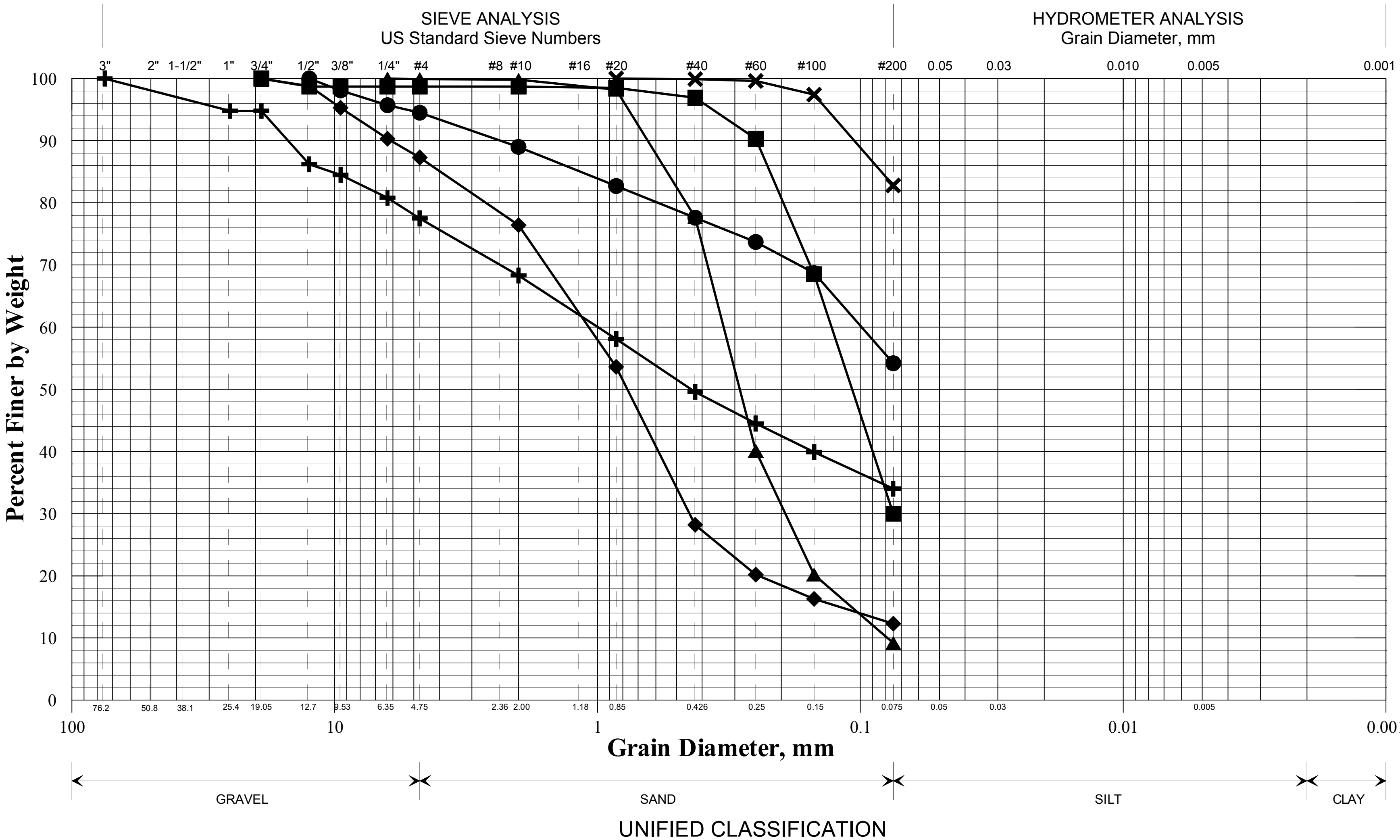
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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-CTM-101/3D	14+12.9	8.3 LT	10.0-12.0	SAND, some silt, some gravel.	14.5			
◆	BB-CTM-101/4D	14+12.9	8.3 LT	15.0-17.0	SAND, little gravel, little silt.	15.4			
■	BB-CTM-101/6D	14+12.9	8.3 LT	25.0-27.0	SAND, some silt, trace gravel.	22.4			
●	BB-CTM-102/2D	14+79.4	8.9 RT	5.0-7.0	Sandy SILT, trace gravel.	13.1			
▲	BB-CTM-102/5D	14+79.4	8.9 RT	20.0-22.0	SAND, trace silt, trace gravel.	22.3			
✖	BB-CTM-102/6D	14+79.4	8.9 RT	25.0-27.0	SILT, little sand.	31.3			NP

WIN
020480.00
Town
Clinton
Reported by/Date
WHITE, TERRY A 7/17/2014

SHEET 1

Appendix C

Calculations

Design of H-piles

Reference: AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014

Bedrock Properties at Abutment 1

BB-CTM-101, R_1=0%, R_2=0%
Rock Type: Phyllite (Metamorphic) fine grained, hard, slightly weathered, breaks along near vertical foiliation.

ϕ = 27-34 (AASHTO LRFD Table C.10.4.6.4-1);
Co = 2,100 - 49,000 psi (AASHTO Standard Specifications for Bridges 17th Edition, Table 4.4.8.1.2B)

For Design Purposes, use bedrock data from BB-CTM-101: RQD = 0% and an assumed Unconfined Compressive Strength of 3500 psi.

Pile Properties

Use the following piles: 12x53, 12x74, 14x73, 14x89, 14x117

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

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

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

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 Pr = ϕ Pn

Nominal Axial Structural Resistance

Determine equivalent yield resistance $P_o = QF_yA_s$ (LRFD 6.9.4.1.1)

$Q := 1.0$ LRFD Article 6.9.4.2

$P_o := Q \cdot F_y \cdot A_s$

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

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

Assume a 1 foot unbraced section of pile due to settlement or scour, L=1. Assume dowel connection creates plastic hinge; one end fixed and one free to rotate, K=2.1. 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 $K_{eff} := 2.1$

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

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

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

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

$$P_e = \begin{pmatrix} 57143 \\ 83776 \\ 117479 \\ 146584 \\ 199822 \end{pmatrix} \cdot kip$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 73.732 \\ 76.859 \\ 109.794 \\ 112.325 \\ 116.176 \end{pmatrix}$$

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:

$$\boxed{P_n = \begin{pmatrix} 771 \\ 1084 \\ 1066 \\ 1300 \\ 1714 \end{pmatrix} \cdot kip}$$

this applies to all pile sizes

Factored Axial Structural Resistance at the Strength Limit State

Resistance factor for un damaged H-pile in combined compression and flexure per LRFD 6.5.4.2:

$$\phi_{cu} := 0.7$$

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

this applies to all pile sizes

$$\boxed{P_r = \begin{pmatrix} 539 \\ 759 \\ 746 \\ 910 \\ 1200 \end{pmatrix} \cdot kip}$$

Factored Axial Structural Resistance for the Strength Limit State

Resistance factor for H-pile in pure compression, severe driving conditions, per LRFD 6.5.4.2 for the case where pile tip is necessary

$$\phi_c := 0.5$$

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

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

Factored structural compressive resistance, P_r

$$\boxed{P_r = \begin{pmatrix} 385 \\ 542 \\ 533 \\ 650 \\ 857 \end{pmatrix} \cdot kip}$$

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:

$$P_n = \begin{pmatrix} 771 \\ 1084 \\ 1066 \\ 1300 \\ 1714 \end{pmatrix} \cdot kip$$

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

$$\phi_c := 0.5$$

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

$$\boxed{P_r = \begin{pmatrix} 385 \\ 542 \\ 533 \\ 650 \\ 857 \end{pmatrix} \cdot kip}$$

12x53
12x74
14x73
14x89
14x117

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

$$\phi_c := 1.0$$

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

$$\boxed{P_{r\_ee} = \begin{pmatrix} 771 \\ 1084 \\ 1066 \\ 1300 \\ 1714 \end{pmatrix} \cdot kip}$$

12x53
12x74
14x73
14x89
14x117

Nominal and Factored Axial Geotechnical Resistance of HP piles

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

Nominal unit bearing resistance of pile point, qp

Design value of compressive strength of rock core

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

Spacing of discontinuities $s_d := 2 \cdot in$

Width of discontinuities. Joints are tight to healed per boring logs $t_d := \frac{1}{128} \cdot in$

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

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

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

Depth factor

$$dd := 1 + 0.4 \cdot \frac{H_s}{D_s}$$

$dd = 1$ and dd < 3

OK

Ksp

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

$$K_{sp} = \begin{pmatrix} 0.215 \\ 0.215 \\ 0.213 \\ 0.213 \\ 0.213 \end{pmatrix}$$

Ksp has a factor of safety of 3.0 in the CGS method. Remove in calculation of pile tip resistance, below.

Geotechnical tip resistance.

$$q_{p\_1} := 3 \cdot q_{u\_1} \cdot K_{sp} \cdot dd$$

$$q_{p\_1} = \begin{pmatrix} 325 \\ 325 \\ 322 \\ 322 \\ 322 \end{pmatrix} \cdot ksf$$

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

Case I

$$R_{p\_1} := \overrightarrow{\left(q_{p\_1} \cdot A_s\right)}$$

$$R_{p\_1} = \begin{pmatrix} 35 \\ 49 \\ 48 \\ 58 \\ 77 \end{pmatrix} \cdot kip$$

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)

$$R_{r\_p1} := \phi_{stat} \cdot R_{p\_1}$$

$$R_{r\_p1} = \begin{pmatrix} 16 \\ 22 \\ 22 \\ 26 \\ 35 \end{pmatrix} \cdot kip$$

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.

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

$$q_{p\_1} = \begin{pmatrix} 325 \\ 325 \\ 322 \\ 322 \\ 322 \end{pmatrix} \cdot ksf$$

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

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

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

Factored Axial Geotechnical Compressive Resistance - *Strength Limit States*

Resistance factor, end bearing on rock Canadian Geotechnical Society method

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

Factored Geotechnical Tip Resistance (R_r)

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

$$R_{r\_p2} = \begin{pmatrix} 61 \\ 86 \\ 84 \\ 103 \\ 135 \end{pmatrix} \cdot kip$$

Note: Factored resistances of IRM and LFRD converge at q_u=22,250 psi

Drivability Analyses

Ref: LRFD Article 10.7.8

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

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

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

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

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

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

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

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

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

GRLWeap Soil and Pile Model Assumptions

Based on Table 2 of this Report, estmated pile lengths at Abutment 1 will be approx. 20.4 ft.

Assume contractor drives pile lengths of 30.5 ft. (5' testing + 3' rock embedment +2' pile into pile.)

Use constant shaft resistances so that GRLWeap will
assign 30 kip shaft resistance to all ultimate capacities analyzed.

Pile Size is 12 x 53

The 12x53 pile can be driven to the resistances below with a D 19-42 hammer and 2.7 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below: Fuel Setting: Max

State of Maine Dept. Of Transportation
20480 Cain Abut 1 12x53 Delmag 19-42

08-Jun-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
300.0	26.40	1.78	1.9	7.14	18.00
350.0	27.48	2.77	2.4	7.54	18.30
400.0	29.02	3.08	2.8	7.92	18.75
450.0	31.93	2.97	3.3	8.34	19.51
500.0	35.25	2.81	3.9	8.88	20.49
550.0	38.66	4.18	4.6	9.33	21.07
600.0	42.53	4.98	5.6	9.81	22.00
614.0	43.60	5.25	5.9	9.93	22.24
615.0	43.73	5.27	6.1	9.84	21.94
650.0	46.32	5.50	6.6	10.20	22.70

Rounding up blow counts >6 to 7 will overstress the piles

$R_{ndr} := 614 \cdot kip$

Strength Limit State

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

$\boxed{R_{fdr} = 399 \cdot kip}$

Extreme and Service Limit States

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

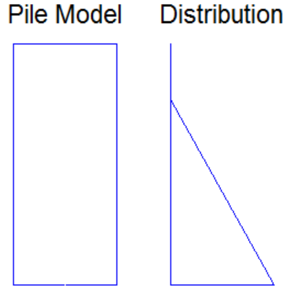
$\boxed{R_{dr} = 614 \cdot kip}$

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer C	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Dampin	0.050	
Toe Dampin	0.150	
Pile Length	30.50	ft
Pile Penetrat	23.50	ft
Pile Top Are	15.50	in2

Pile Model

Skin Friction Distribution



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

Pile Size is 12 x 74

The 12x74 pile can be driven to the resistances below with a D 19-42 hammer and 2.7 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below: Fuel Setting: Max

State of Maine Dept. Of Transportation
20480 Cain Abut 1 12x74 Delmag 19-42

08-Jun-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
300.0	24.38	1.06	2.0	7.22	17.38
500.0	27.90	3.78	3.9	8.67	19.34
700.0	34.61	4.46	6.7	9.78	21.03
900.0	42.43	5.10	10.6	10.81	22.82
925.0	43.51	5.58	11.2	10.97	23.11
930.0	43.72	5.67	11.3	10.99	23.14
935.0	43.91	5.74	11.5	11.01	23.17
940.0	44.11	5.81	11.6	11.04	23.20
950.0	44.46	5.93	12.0	11.08	23.26
975.0	45.69	6.16	12.7	11.23	23.48

Limit driving stress < 45 ksi stress

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

Strength Limit State

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

$\boxed{R_{fdr} = 618 \cdot kip}$

Extreme and Service Limit States

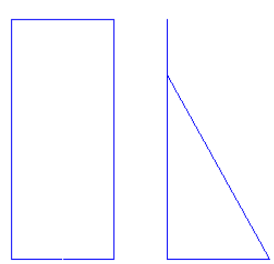
$R_{dr} := R_{ndr} \cdot \phi$

$\boxed{R_{dr} = 950 \cdot kip}$

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer C	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Dampin	0.050	
Toe Damping	0.150	
Pile Length	30.50	ft
Pile Penetrat	23.50	ft
Pile Top Are	21.80	in2

Pile Model   Skin Friction Distribution



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

Pile Size is 14 x 73

The 14x73 pile can be driven to the resistances below with a D 19-42 hammer and 2.7 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:
Fuel Setting: Max

State of Maine Dept. Of Transportation
20480 Cain Abut 1 14x73 Delmag 19-42

08-Jun-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
300.0	24.48	1.06	2.0	7.21	17.38
500.0	28.08	3.69	3.9	8.69	19.40
700.0	35.16	4.75	6.7	9.84	21.20
900.0	43.15	5.35	11.0	10.81	22.80
925.0	44.17	5.67	11.4	11.03	23.20
930.0	44.51	5.77	11.6	11.06	23.32
935.0	44.77	5.86	11.8	11.08	23.35
940.0	45.03	5.94	11.9	11.11	23.38
950.0	45.52	6.09	12.2	11.17	23.44
975.0	46.66	6.33	12.9	11.32	23.65

Blow counts >11 bpi round up to 12 bpi will result in a driving stresses that exceed 45 ksi. Use bpi of 11.

$R_{ndr} := 900 \cdot kip$

Strength Limit State

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

$\boxed{R_{fdr} = 585 \cdot kip}$

Extreme and Service Limit States

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

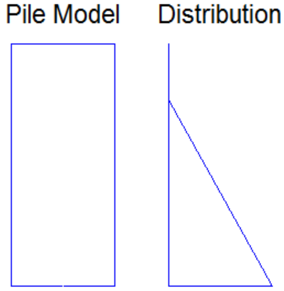
$\boxed{R_{dr} = 900 \cdot kip}$

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer C	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Dampin	0.050	
Toe Damping	0.150	
Pile Length	30.50	ft
Pile Penetrat	23.50	ft
Pile Top Are	21.40	in2

Pile Model

Skin Friction Distribution



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

Pile Size is 14 x 89

The 14x89 pile can be driven to the resistances below with a D 19-42 hammer and 2.7 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:
Fuel Setting: Max

State of Maine Dept. Of Transportation
20480 Cain Abut 1 14x89 Delmag 19-42

08-Jun-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
300.0	23.37	1.08	2.1	7.27	17.09
600.0	27.29	3.21	5.0	8.93	18.68
900.0	35.91	6.26	9.3	10.20	21.21
1000.0	39.12	6.48	11.3	10.62	21.80
1100.0	42.30	7.50	14.0	11.02	22.69
1115.0	42.83	7.68	14.5	11.09	22.86
1130.0	43.33	7.81	15.0	11.16	23.03
1145.0	43.73	7.91	15.4	11.24	23.19
1160.0	44.17	8.00	15.9	11.32	23.35
1175.0	44.68	8.07	16.4	11.40	23.53

DELMAG D 19-42

Limiting blows to 15 bpi

$R_{ndr} := 1130 \cdot kip$

Strength Limit State

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

$R_{fdr} = 735 \cdot kip$

Extreme and Service Limit States

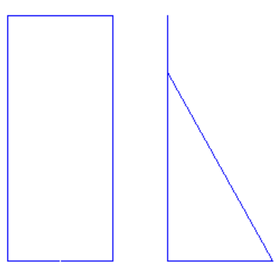
$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 1130 \cdot kip$

Efficiency	0.800	
Helmet	2.70	kips
Hammer C	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Dampin	0.050	
Toe Dampin	0.150	
Pile Length	30.50	ft
Pile Penetrat	23.50	ft
Pile Top Are	26.10	in2

Pile Model

Skin Friction Distribution



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

Pile Size is 14 x 117

The 14x117 pile can be driven to the resistances below with a D 19-42 hammer and 2.7 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:
Fuel Setting: Max

State of Maine Dept. Of Transportation
20480 Cain Abut 1 14x117 Delmag 19-42

08-Jun-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
300.0	22.10	1.28	2.3	7.49	16.91
600.0	24.73	3.22	5.2	8.62	17.37
900.0	27.47	5.53	8.7	9.41	18.98
1000.0	29.81	5.95	10.2	9.68	19.55
1100.0	32.20	7.07	11.9	9.96	20.25
1200.0	34.53	7.62	14.3	10.23	20.82
1215.0	34.88	7.63	14.7	10.27	20.92
1230.0	35.22	7.67	15.0	10.31	21.02
1245.0	35.57	7.72	15.5	10.35	21.12
1260.0	35.92	7.74	15.9	10.39	21.22

DELMAG D 19-42

Limit blows to 15 bpi

$R_{ndr} := 1230 \cdot kip$

Strength Limit State

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

$\boxed{R_{fdr} = 800 \cdot kip}$

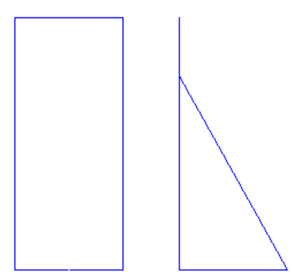
Extreme and Service Limit States

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

$\boxed{R_{dr} = 1230 \cdot kip}$

Efficiency	0.800	
Helmet	2.70	kips
Hammer C	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Dampin	0.050	
Toe Dampin	0.150	
Pile Length	30.50	ft
Pile Penetrat	23.50	ft
Pile Top Are	34.40	in2

Pile Model   Skin Friction Distribution



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

Pile Size is 14 x 117

The 14x117 pile can be driven to the resistances below with a D 36-32 hammer and 2.7 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below: Fuel Setting: 81% (Max -2)

State of Maine Dept. Of Transportation
20480 Cain Abut 1 14x117 Delmag 36-32

08-Jun-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
300.0	23.29	0.66	1.2	5.36	28.41
600.0	28.25	1.81	2.8	6.56	27.61
900.0	32.03	4.17	4.8	7.67	30.76
1200.0	36.24	2.06	7.8	8.76	34.01
1500.0	43.07	7.35	13.2	9.44	35.91
1515.0	43.54	7.95	13.6	9.45	35.73
1530.0	43.95	8.17	13.9	9.48	35.77
1532.0	44.02	8.14	14.0	9.47	35.79
1550.0	44.39	8.51	14.4	9.49	35.89
1575.0	45.13	10.13	15.0	9.53	35.91

DELMAG D 36-32

Limit stess in pile to 45 ksi

$R_{ndr} := 1532 \cdot kip$

Efficiency	0.800	
Helmet	2.70	kips
Hammer C	109975	kips/ir
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Dampin	0.050	
Toe Dampin	0.150	
Pile Length	30.50	ft
Pile Penetrat	23.50	ft
Pile Top Are	34.40	in2

Strength Limit State

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

$\boxed{R_{fdr} = 996 \cdot kip}$

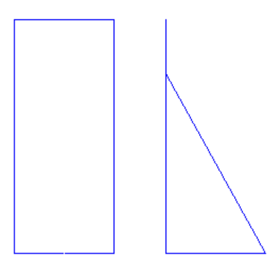
Extreme and Service Limit States

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

$\boxed{R_{dr} = 1532 \cdot kip}$

Pile Model

Skin Friction Distribution



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

Summary of factored axial pile resistances for strenght, extreme, and service limt states at Abutment No. 1:

Pile Section	Strength Limit State Factored Axial Pile Resistance			
	Structural Resistance ϕ_c=0.50 (kips)	Controlling Geotechnical Resistance ϕ_c=0.50 (kips)	Drivability Resistance φ_{dyn} = 0.65 (kips)	Governing Axial Pile Resistance (kips)
Abutment No. 1				
HP 12 x 53	385	385	399	385
HP 12 x 74	542	542	618	542
HP 14 x 73	533	533	585	533
HP 14 x 89	650	650	735	650
HP 14 x 117	857	857	800 (996)	800 (857)

Pile Section	Extreme and Service Limit State Factored Axial Pile Resistance			
	Structural Resistance (normal conditions) ϕ_c=1.0 (kips)	Controlling Geotechnical Resistance ϕ_c=1.0 (kips)	Drivability Resistance φ = 1.0 (kips)	Governing Axial Pile Resistance (kips)
Abutment No. 1				
HP 12 x 53	771	771	614	614
HP 12 x 74	1084	1084	950	950
HP 14 x 73	1066	1066	900	900
HP 14 x 89	1300	1300	1130	1130
HP 14 x 117	1714	1714	1230 (1532)	1230 (1532)

Design of H-piles

Reference: AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014

Bedrock Properties at Abutment 2

BB-CTM-101, R_1=0%, R_2=0%

Rock Type: Phyllite (Metamorphic) fine grained, hard, slightly weathered, breaks along near vertical foiliation.

ϕ = 27-34 (AASHTO LRFD Table C.10.4.6.4-1);

Co = 2,100 - 49,000 psi (AASHTO Standard Specifications for Bridges 17th Edition, Table 4.4.8.1.2B)

For Design Purposes, use bedrock data from BB-CTM-102: RQD = 0% and an assumed Unconfined Compressive Strength of 3500 psi.

Pile Properties

Use the following piles: 12x53, 12x74, 14x73, 14x89, 14x117

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

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

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

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 Pr = ϕ Pn

Nominal Axial Structural Resistance

Determine equivalent yield resistance Po=QF_yA_s (LRFD 6.9.4.1.1)

$Q := 1.0$ LRFD Article 6.9.4.2

$P_o := Q \cdot F_y \cdot A_s$

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

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

Assume a 1 foot unbraced section of pile due to settlement or scour, L=1. Assume dowel connection creates plastic hinge; one end fixed and one free to rotate, K=2.1. 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 $K_{eff} := 2.1$ LRFD Table C4.6.2.5-1
Design value for ideal conditions when one end fixed and one free to rotate.

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

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

$$P_e := \overrightarrow{\left[\frac{\pi^2 \cdot E}{\left(\frac{K_{eff} \cdot l_{unbraced\_top}}{r_s}\right)^2} \cdot A_s\right]} \qquad P_e = \begin{pmatrix} 57143 \\ 83776 \\ 117479 \\ 146584 \\ 199822 \end{pmatrix} \cdot kip$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 73.732 \\ 76.859 \\ 109.794 \\ 112.325 \\ 116.176 \end{pmatrix}$$

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:

$$\boxed{P_n = \begin{pmatrix} 771 \\ 1084 \\ 1066 \\ 1300 \\ 1714 \end{pmatrix} \cdot kip}$$

this applies to all pile sizes

Factored Axial Structural Resistance at the Strength Limit State

Resistance factor for unbraced segements of H-pile in combined compression and flexure per LRFD 6.5.4.2:

$$\phi_{cu} := 0.7$$

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

this applies to all pile sizes

$$\boxed{P_r = \begin{pmatrix} 539 \\ 759 \\ 746 \\ 910 \\ 1200 \end{pmatrix} \cdot kip}$$

Factored Axial Structural Resistance for the Strength Limit State

Resistance factor for lower portion of H-pile in pure compression, severe driving conditions, per LRFD 6.5.4.2 for the case where pile tip is necessary

$$\phi_c := 0.5$$

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

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

Factored structural compressive resistance, P_r

$$P_r = \begin{pmatrix} 385 \\ 542 \\ 533 \\ 650 \\ 857 \end{pmatrix} \cdot kip$$

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:

$$P_n = \begin{pmatrix} 771 \\ 1084 \\ 1066 \\ 1300 \\ 1714 \end{pmatrix} \cdot kip$$

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

$$\phi_c := 0.5$$

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

$$\boxed{P_r = \begin{pmatrix} 385 \\ 542 \\ 533 \\ 650 \\ 857 \end{pmatrix} \cdot kip}$$

12x53
12x74
14x73
14x89
14x117

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

$$\phi_c := 1.0$$

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

$$\boxed{P_{r\_ee} = \begin{pmatrix} 771 \\ 1084 \\ 1066 \\ 1300 \\ 1714 \end{pmatrix} \cdot kip}$$

12x53
12x74
14x73
14x89
14x117

Nominal and Factored Axial Geotechnical Resistance of HP piles

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

Nominal unit bearing resistance of pile point, q_p

Design value of compressive strength of rock core

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

Spacing of discontinuities $\quad s_d := 2 \cdot in$

Width of discontinuities. Joints are open to tight per boring logs $\quad t_d := \frac{1}{128} \cdot in$

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

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

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

Depth factor

$$dd := 1 + 0.4 \cdot \frac{H_s}{D_s}$$

$dd = 1$ OK

Ksp

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

$$K_{sp} = \begin{pmatrix} 0.215 \\ 0.215 \\ 0.213 \\ 0.213 \\ 0.213 \end{pmatrix}$$

Ksp has a factor of safety of 3.0 in the CGS method. Remove in calculation of pile tip resistance, below.

Geotechnical tip resistance.

$q_{p\_1} := 3 \cdot q_{u\_1} \cdot K_{sp} \cdot dd$

$$\boxed{q_{p\_1} = \begin{pmatrix} 325 \\ 325 \\ 322 \\ 322 \\ 322 \end{pmatrix} \cdot ksf}$$

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

Case I $R_{p\_1} := \overrightarrow{\left(q_{p\_1} \cdot A_s\right)}$

$$\boxed{R_{p\_1} = \begin{pmatrix} 35 \\ 49 \\ 48 \\ 58 \\ 77 \end{pmatrix} \cdot kip}$$

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 (Rr)

$R_{r\_p1} := \phi_{stat} \cdot R_{p\_1}$

$$\boxed{R_{r\_p1} = \begin{pmatrix} 16 \\ 22 \\ 22 \\ 26 \\ 35 \end{pmatrix} \cdot kip}$$

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, qp

Design value of compressive strength of rock core

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

Geotechnical tip resistance.

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

$$q_{p\_1} = \begin{pmatrix} 325 \\ 325 \\ 322 \\ 322 \\ 322 \end{pmatrix} \cdot ksf$$

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

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

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

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 (Rr)

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

$$R_{r\_p2} = \begin{pmatrix} 61 \\ 86 \\ 84 \\ 103 \\ 135 \end{pmatrix} \cdot kip$$

Drivability Analyses

Ref: LRFD Article 10.7.8

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

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

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

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

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

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

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

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

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

GRLWeap Soil and Pile Model Assumptions

Based on Table 2 of this Report, estmated pile lengths at Abutment 1 will be approx. 27 ft.

Assume contractor drives pile lengths of 40 ft. (5' testing + 3' rock embedment +2' cap + 3' misc.)

Use constant shaft resistances so that GRLWeap will
assign 30 kip shaft resistance to all ultimate capacities analyzed.

Pile Size is 12 x 53

The 12x53 pile can be driven to the resistances below with a D 19-42 hammer and 2.7 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:
Fuel Setting: Max

State of Maine Dept. Of Transportation
20480 Cain Abut 2 12x53 Delmag 19-42

08-Jun-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
300.0	26.19	2.42	2.0	7.16	18.53
400.0	28.74	4.04	3.0	8.03	19.80
500.0	34.75	6.09	4.4	9.05	21.66
600.0	40.50	7.42	6.7	9.92	23.25
650.0	43.38	8.67	7.7	10.36	24.02
660.0	44.05	8.89	7.9	10.43	24.23
664.0	44.23	8.91	8.0	10.46	24.24
670.0	44.64	9.01	8.2	10.50	24.34
675.0	44.80	9.00	8.3	10.54	24.38
680.0	45.10	9.09	8.5	10.57	24.47

Rounding up blow counts >8 to 9 will overstress the piles

$R_{ndr} := 664 \cdot kip$

Strength Limit State

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

$\boxed{R_{fdr} = 432 \cdot kip}$

Extreme and Service Limit States

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

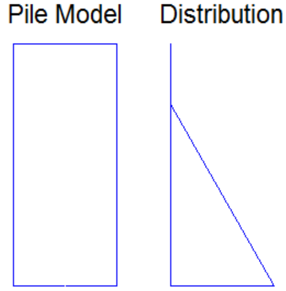
$\boxed{R_{dr} = 664 \cdot kip}$

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer C	109975	kips/in
Skin Quake	0.040	in
Toe Quake	0.100	in
Skin Dampin	0.050	
Toe Dampin	0.150	
Pile Length	40.00	ft
Pile Penetrat	30.00	ft
Pile Top Are	15.50	in2

Pile Model

Skin Friction Distribution



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

Pile Size is 12 x 74

The 12x74 pile can be driven to the resistances below with a D 19-42 hammer and 2.7 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below: Fuel Setting: Max

State of Maine Dept. Of Transportation
20480 Cain Abut 2 12x74 Delmag 19-42

08-Jun-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
300.0	24.28	1.06	2.1	7.23	17.68
600.0	30.38	3.85	5.3	9.12	20.40
900.0	41.88	5.91	11.4	10.56	23.11
920.0	42.57	6.09	12.0	10.66	23.31
940.0	43.49	6.26	12.5	10.81	23.70
960.0	44.13	6.36	13.6	10.81	23.67
967.0	44.35	6.38	14.0	10.81	23.66
970.0	44.45	6.40	14.2	10.81	23.66
975.0	44.61	6.42	14.5	10.81	23.65
985.0	45.10	6.49	14.5	10.95	24.00

Limit driving stress < 45 ksi stress

$R_{ndr} := 967 \cdot kip$

Strength Limit State

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

$R_{fdr} = 629 \cdot kip$

Extreme and Service Limit States

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

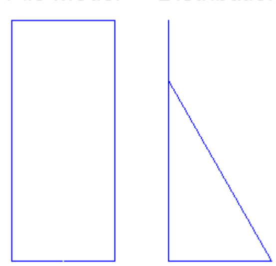
$R_{dr} = 967 \cdot kip$

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer C	109975	kips/in
Skin Quake	0.040	in
Toe Quake	0.100	in
Skin Dampin	0.050	
Toe Damping	0.150	
Pile Length	40.00	ft
Pile Penetrat	30.00	ft
Pile Top Are	21.80	in2

Pile Model

Skin Friction Distribution



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

Pile Size is 14 x 73

The 14x73 pile can be driven to the resistances below with a D 19-42 hammer and 2.7 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:
Fuel Setting: Max

State of Maine Dept. Of Transportation
20480 Cain Abut 2 14x73 Delmag 19-42

09-Jun-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
300.0	24.37	1.03	2.1	7.23	17.75
600.0	30.78	4.00	5.3	9.16	20.50
900.0	42.54	5.71	11.6	10.64	23.26
920.0	43.49	5.92	12.0	10.81	23.70
930.0	43.82	6.03	12.5	10.81	23.69
939.0	44.11	6.10	13.0	10.81	23.68
960.0	44.79	6.24	14.3	10.81	23.65
965.0	45.02	6.29	14.0	10.94	23.94
970.0	45.22	6.32	14.2	10.96	24.00
980.0	45.69	6.40	14.7	11.00	24.11

Blow counts >13 bpi result in a driving stresses that exceed 45 ksi. Use bpi of 13.

$R_{ndr} := 939 \cdot kip$

Strength Limit State

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

$\boxed{R_{fdr} = 610 \cdot kip}$

Extreme and Service Limit States

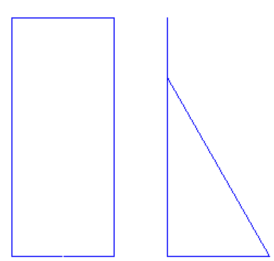
$R_{dr} := R_{ndr} \cdot \phi$

$\boxed{R_{dr} = 939 \cdot kip}$

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer C	109975	kips/in
Skin Quake	0.040	in
Toe Quake	0.100	in
Skin Dampin	0.050	
Toe Dampin	0.150	
Pile Length	40.00	ft
Pile Penetrat	30.00	ft
Pile Top Are	21.40	in2

Pile Model   Skin Friction Distribution



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

Pile Size is 14 x 89

The 14x89 pile can be driven to the resistances below with a D 19-42 hammer and 2.7 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below: Fuel Setting: Max

State of Maine Dept. Of Transportation
20480 Cain Abut 2 14x89 Delmag 19-42

08-Jun-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
300.0	23.28	1.92	2.2	7.27	17.32
600.0	26.99	4.32	5.1	8.84	19.19
900.0	35.41	6.40	9.9	9.93	21.46
1000.0	38.53	7.24	12.3	10.30	22.34
1020.0	39.14	7.56	13.0	10.37	22.55
1040.0	39.77	7.73	13.6	10.44	22.69
1060.0	40.39	7.88	14.3	10.51	22.85
1070.0	40.68	7.91	14.7	10.55	22.89
1080.0	41.01	8.01	15.0	10.58	23.01
1100.0	41.62	8.05	15.8	10.65	23.17

DELMAG D 19-42

Limiting blows to 15 bpi

$R_{ndr} := 1080 \cdot kip$

Strength Limit State

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

$\boxed{R_{fdr} = 702 \cdot kip}$

Extreme and Service Limit States

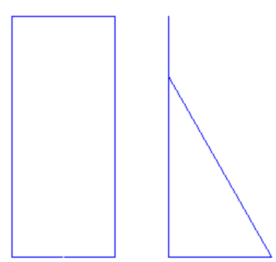
$R_{dr} := R_{ndr} \cdot \phi$

$\boxed{R_{dr} = 1080 \cdot kip}$

Efficiency	0.800	
Helmet	2.70	kips
Hammer C	109975	kips/in
Skin Quake	0.040	in
Toe Quake	0.100	in
Skin Dampin	0.050	
Toe Dampin	0.150	
Pile Length	40.00	ft
Pile Penetrat	30.00	ft
Pile Top Are	26.10	in2

Pile Model

Skin Friction Distribution



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

Pile Size is 14 x 117

The 14x117 pile can be driven to the resistances below with a D 19-42 hammer and 2.7 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:
Fuel Setting: Max

State of Maine Dept. Of Transportation
20480 Cain Abut 2 14x117 Delmag 19-42

08-Jun-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
300.0	22.04	2.80	2.3	7.47	17.09
600.0	24.54	5.02	5.3	8.54	17.70
900.0	27.30	5.05	9.0	9.25	19.35
1000.0	29.66	5.77	10.6	9.51	19.93
1100.0	31.91	6.24	12.6	9.77	20.57
1120.0	32.34	6.23	13.0	9.82	20.66
1140.0	32.78	6.25	13.5	9.87	20.80
1160.0	33.27	6.21	14.1	9.92	20.89
1180.0	33.70	6.19	14.6	9.97	21.04
1195.0	34.07	6.23	15.0	10.01	21.13

DELMAG D 19-42

Limit blows to 15 bpi

$R_{ndr} := 1195 \cdot kip$

Strength Limit State

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

$R_{fdr} = 777 \cdot kip$

Extreme and Service Limit States

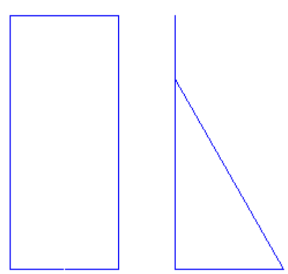
$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 1195 \cdot kip$

Efficiency	0.800	
Helmet	2.70	kips
Hammer C	109975	kips/in
Skin Quake	0.040	in
Toe Quake	0.100	in
Skin Dampin	0.050	
Toe Damping	0.150	
Pile Length	40.00	ft
Pile Penetrat	30.00	ft
Pile Top Are	34.40	in2

Pile Model

Skin Friction Distribution



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

Pile Size is 14 x 117

The 14x117 pile can be driven to the resistances below with a D 36-32 hammer and 2.7 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:
Fuel Setting: 81% (Max -2)

State of Maine Dept. Of Transportation
20480 Cain Abut 2 14x117 Delmag 36-32

08-Jun-2015
GRLWEAP (TM) Version 2003

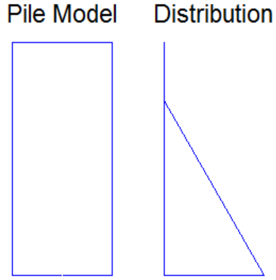
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	22.90	0.90	1.2	5.32	28.34
600.0	27.82	2.16	2.9	6.53	28.51
900.0	31.61	3.10	4.9	7.66	31.95
1200.0	36.12	4.06	8.3	8.70	35.84
1400.0	40.95	4.39	11.4	9.34	38.55
1500.0	43.00	4.58	13.2	9.50	39.33
1550.0	43.92	5.75	14.3	9.56	39.43
1570.0	44.28	6.36	14.8	9.59	39.53
1580.0	44.53	6.36	15.0	9.60	39.72
1590.0	44.59	6.91	15.3	9.61	39.64

DELMAG D 36-32

Efficiency	0.800	
Helmet	2.70	kips
Hammer C	109975	kips/in
Skin Quake	0.040	in
Toe Quake	0.100	in
Skin Dampin	0.050	
Toe Dampin	0.150	
Pile Length	40.00	ft
Pile Penetrat	30.00	ft
Pile Top Are	34.40	in2

Pile Model

Skin Friction Distribution



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

Limit blow counts to 15 per inch

$$R_{ndr} := 1580 \cdot kip$$

Strength Limit State

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

$$\boxed{R_{fdr} = 1027 \cdot kip}$$

Extreme and Service Limit States

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

$$\boxed{R_{dr} = 1580 \cdot kip}$$

Summary of factored axial pile resistances for strenght, extreme, and service limt states at Abutment No. 2:

Pile Section	Strength Limit State Factored Axial Pile Resistance			
	Structural Resistance ϕ_c=0.50 (kips)	Controlling Geotechnical Resistance ϕ_c=0.50 (kips)	Drivability Resistance φ_{dyn} = 0.65 (kips)	Governing Axial Pile Resistance (kips)
Abutment No. 2				
HP 12 x 53	385	385	432	385
HP 12 x 74	542	542	629	542
HP 14 x 73	533	533	610	533
HP 14 x 89	650	650	702	650
HP 14 x 117	857	857	777 (1027)	777 (857)

Pile Section	Extreme and Service Limit State Factored Axial Pile Resistance			
	Structural Resistance (normal conditions) ϕ_c=1.0 (kips)	Controlling Geotechnical Resistance ϕ_c=1.0 (kips)	Drivability Resistance φ = 1.0 (kips)	Governing Axial Pile Resistance (kips)
Abutment No. 2				
HP 12 x 53	771	771	664	664
HP 12 x 74	1084	1084	967	967
HP 14 x 73	1066	1066	939	939
HP 14 x 89	1300	1300	1080	1080
HP 14 x 117	1714	1714	1195 (1580)	1195 (1580)

Backfill engineering strength parameters

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

Unit weight	$\gamma_1 := 125 \cdot \text{pcf}$
Internal friction angle	$\phi_1 := 32 \cdot \text{deg}$
Cohesion	$c_1 := 0 \cdot \text{psf}$

U-shaped and Butterfly Wingwalls - At-Rest

At-Rest Earth Pressure - Rankine Theory

Reference: LRFD 3.11.5.2 For walls less than 5 feet, or braced stem walls that prevent rotation.

$$K_o := 1 - \sin(\phi_1)$$

$$K_o = 0.47$$

U-shaped Wingwalls - Active Earth Pressure

Active pressure acting parallel to the travelway is assumed to be resisted by the superstructure and can be neglected for butterfly walls. Design of U-shaped wingwalls shall consider active pressure acting perpendicular to the travelway.

Active Earth Pressure - Rankine Theory

Rankine shall be used for **long heeled** cantilever walls (See LFRD C3.11.d.3-1), where the failure surface is uninterrupted by the top of the wall stem. The earth pressure is applied to a plane extending vertically up from the heel of the wall base, and the weight of the soil on the inside of the vertical plane is considered as part of the wall weight. The failure sliding surface is not restricted by the top of the wall or back face of wall.

- For cantilever walls with horizontal backslope

$$K_a := \tan\left(45 \cdot \text{deg} - \frac{\phi_1}{2}\right)^2 \qquad K_a = 0.307$$

- For a sloped backfill (2H:1V)

  β = Angle of fill slope to the horizontal

  $\beta := 26.6 \cdot \text{deg}$

$$K_{aslope} := \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}} \qquad K_{aslope} = 0.519$$

- Pa is oriented at an angle of β to the vertical plane

U-shaped and Butterfly Wingwall - Passive Earth Pressure - Coulomb Theory

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

$\phi_1 := 32 \cdot \text{deg}$

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

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

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

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

$\delta := 19.5 \cdot \text{deg}$

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

$\boxed{K_{p\_coul} = 6.73}$

U-shaped and Butterfly Wingwall - Passive Earth Pressure - Rankine Theory

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

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

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

$\boxed{K_{p\_rank} = 3.255}$

Pp 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: **Clinton, Maine**
DFI = 1650 degree-days.
Case 1 - coarse grained granular fill soils W=10% (assumed).

For DFI = 1600 $d1 := 84.8$

For DFI = 1700 $d2 := 87.5$

$$d := in\cdot\left(\frac{d2 - d1}{10}\cdot 2 + d1\right)$$

Depth of Frost Penetration $d = 85\cdot in$ $d = 7.1\cdot ft$

Method 2 - ModBerg Software

Examine foundations placed on coarse grained fill soils

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

--- ModBerg Results ---

Project Location: Waterville, Maine
Air Design Freezing Index = 1395 F-days
N-Factor = 0.80
Surface Design Freezing Index = 116 F-days
Mean Annual Temperature = 45.3 deg F
Design Length of Freezing Season = 122 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	80.6	10.0	135.0	30	36	2.9	2.1	1,944

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 = 6.72 ft = 80.6 in.

Recommendation: 6.9 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**
2	14		2	7	2	10		2	0.20
5	12		3	0.25	5	10		3	0.30
10	20		5	0.25	10	6		5	0.83
15	10		5	0.50	15	6		5	0.83
20	19		5	0.26	20	6		5	0.83
25	12		5	0.42	25	9		5	0.56
30	100	**Bedrock**	75	0.75	30	9		5	0.56
					35	100	**Bedrock**	70	0.70
SUM			**100**	**2.43**				**100**	**4.61**

di/di/N **41.16** **di/di/N** **21.69**

SUM	**Nav.**	**31.42**

Conclusion: Site Class D

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

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 44.636300 Longitude = -069.504900
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.074	PGA - Site Class B
0.2	0.157	Ss - Site Class B
1.0	0.046	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
Latitude = 44.636300 Longitude = -069.504900
As = Fpga PGA, SDs = FaSs, and SD1 = FvS1
Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.118	As - Site Class D
0.2	0.251	SDs - Site Class D
1.0	0.110	SD1 - Site Class D