MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

HUNTER COVE BRIDGE
MINGO LOOP ROAD OVER HUNTER COVE
RANGELEY, MAINE

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1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and provide geotechnical design recommendations for the replacement of Hunter Cove Bridge which carries Mingo Loop Road over Hunter Cove in Rangeley, Maine. This report presents the subsurface information obtained at the site during the subsurface investigations, geotechnical recommendations, and geotechnical design parameters for the design of the new bridge substructures.

The existing bridge structure was constructed in 1940. The existing steel girder bridge has a span length of 71 feet. The substructures consist of timber backwalls and bridge seats founded on timber piles. According to the 2016 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the deck and superstructure are considered to be in Fair Condition and rated as a 5. The substructure is considered to be in Serious Condition and rated as a 3 due to advanced deterioration and decay of the timber piles. The existing structure's Sufficiency Rating is 38.7 and classified as Structurally Deficient.

The proposed replacement structure consists of a 78-foot single span bridge with a galvanized steel plate girder superstructure on H-pile supported integral abutments. The existing structure will be removed and 2:1 riprap slopes will be placed in front of the new integral abutments. The new Hunter Cove Bridge will be designed to match the existing horizontal alignment. The vertical profile will increase approximately 1.5 feet to improve vertical clearance under the bridge. Traffic will be rerouted through a detour allowing for a bridge closure during construction.

2.0 GEOLOGIC SETTING

The existing structure carries Mingo Loop Road over Hunter Cove approximately 1.3 miles west of Route 4 as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of the Rangeley Quadrangle, Maine, Open-file No. 75-14 (1974), indicates the surficial soils in the vicinity of the bridge project consist of glacial till. Glacial till is a heterogeneous mixture of sand, silt, clay and stones. Glacial till includes two varieties; basil till and ablation till. Basal till is typically fine grained and very compact with low permeability and poor drainage. Ablation till is typically loose, sandy, and stony with moderate permeability and fair to good drainage. These soils generally overly bedrock, but may overlie, or include, sand and gravel.

The Quimby and Greenvale Cove Formations in Western Maine, Geological Survey Bulletin 1274-L by Robert Moench (1969), describes and names the bedrock at the project site as the Quimby Formation consisting of metagraywacke and metashale members. The metagraywacke member contains interbedded conglomeratic metagraywacke, grey to black slate, and felsic metavolcanics rocks. The metashale member contains interlayered cyclically bedded metagraywacke and metashale.

3.0 SUBSURFACE INVESTIGATION

Four test borings explored subsurface conditions at the site. Boring BB-RHC-102 was drilled east of the existing bridge; borings BB-RHC-101, BB-RHC-201, and BB-RHC-202 were drilled west of the existing bridge. The 200-series borings were drilled with the objective of better characterizing the vertical extent of the peat and obtaining an undisturbed sample to determine the compressibility of the peat. The boring locations are shown on Sheet 2 – Boring Location Plan.

The MaineDOT Drill Crew drilled the 100-series test borings in April and May 2016 and the 200-series test borings in August 2017. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 4 and 5 – Boring Logs.

Borings were performed by using a combination of 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, a 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 dill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated per ASTM D4633 "Standard Test Method for Energy Measurement for Dynamic Penetrometers" in October 2014 and recalibrated prior to the 200-series borings in April 2017. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.908 for the 100-series borings and 0.854 for the 200-series borings to the raw field N-values. This hammer efficiency factor (0.908 and 0.854) and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring log.

Bedrock was cored in the 100-series borings using an NQ-2" core barrel and the Rock Quality Designation (RQD) of the core calculated. A Northeast Transportation Technician Certification Program (NETTCP) Certified Subsurface Inspector logged the subsurface conditions encountered. The MaineDOT geotechnical engineer selected the boring location and drilling methods, designated type and depth of sampling techniques, reviewed boring logs, and identified field testing requirements. The borings were located in the field using taped measurements at the completion of the drilling program.

4.0 LABORATORY TESTING

A laboratory testing program was conducted on selected soil samples recovered from the test borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the project site. Laboratory testing consisted of; seven standard grain size analyses with natural water content, six grain size analyses with hydrometer and natural moisture content, four Atterberg limits tests, and four direct estimates of organic matter by loss on ignition. The results of soil tests are included as Appendix B – Laboratory Test Results. Moisture content information and other soil test results are also shown on the boring logs provided in Appendix A – Boring Logs and on Sheet 4 and 5 – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings generally consisted of granular fill, glacial stream sediments, marsh deposits, and glacial till. The fill unit and subsurface soils are underlain by metamorphic bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheet 4 and 5 – Boring Logs. A generalized subsurface profile is shown on Sheet 3 – Interpretive Subsurface Profile. The following paragraphs discuss the subsurface conditions encountered:

5.1 Fill

Encountered in the borings was a layer of granular fill. The thickness of the fill unit was approximately 17.8 to 19.2 feet at the boring locations. The fill encountered generally consisted of:

- Brown, damp to wet, gravelly sand, trace silt, trace relic pavement;
- Brown, damp, sand, some gravel, little silt; and
- Olive, moist, silty sand, little gravel.

Corrected SPT N-values in the fill unit ranged from 15 to 53 blows per foot (bpf), indicating the fill is medium dense to very dense in consistency. One grain size analyses of the unit resulted in the sample being classified as A-1-B under the AASHTO Soil Classification System and SW-SM under the Unified Soil Classification System (USCS). The natural water content of the sample tested was approximately 4 percent.

Wood was encountered in BB-RHC-101 indicating that logs are present within the fill layer. Historical construction records of the existing bridge also indicate the presence of logs in the fill layer near and beneath the proposed Abutment No. 2. See Appendix D – Existing Structure Pile Lengths for historical construction records.

5.2 Marsh Deposit

The borings west of the existing structure encountered a marsh deposit beneath the fill unit. The thickness of the marsh deposit was approximately 4 feet. The encountered deposit generally consisted of dark brown, wet, fibrous peat.

Two undisturbed thin wall tube sampling attempts were performed in BB-RHC-202. One attempt failed to recover a sample. One undisturbed sampling attempt resulted in recovery of a sample of the marsh deposit. An attempt to extract the recovered sample from the thin wall tube for lab testing failed.

Natural water contents of samples from the marsh deposit ranged from approximately 174 to 275 percent. Four loss on ignition tests resulted in the organic content of the samples ranging from approximately 33 to 47 percent. Three grain size analyses on the material remaining after ignition resulted in the samples being classified as A-2-4, A-1-b, and A-1-a under the AASHTO Soil Classification System and SM/OL, SM, and SW-SM under the USCS.

5.3 Glacial Stream Deposit

Boring BB-RHC-101 encountered a glacial stream deposit beneath the marsh deposit. The deposit thickness encountered was approximately 10.5 feet. The encountered deposit generally consisted of grey, wet, fine sand, little silt, trace gravel.

Corrected SPT N-values in the glacial stream deposits ranged from 11 to 14 bpf, indicating the alluvium is medium dense in consistency. Two grain size analyses of samples from the deposit resulted classifications of A-2-4 under the AASHTO Soil Classification System and SM and SP-SM under the USCS. The natural water content of the samples tested ranged from approximately 21 to 24 percent.

5.4 Glacial Till

Glacial till was encountered beneath the fill material east of the existing bridge and beneath the glacial stream deposits west of the existing bridge. The encountered thickness ranged from approximately 37.3 to 54.2 feet. The glacial till deposit generally consisted of:

- Grey to olive, wet, silt, some to trace clay, some to trace sand, trace gravel;
- Grey, wet, silty sand, some gravel;
- Grey, wet, gravelly sand, some silt;
- Olive, wet, fine sandy silt, some clay;
- Grey brown, silty sand;
- Grey, cemented sand, silt, and gravel; and
- Cobbles.

The deepest subunits of glacial till encountered in boring BB-RHC-102 exhibited weak to strong cementation.

Corrected SPT N-values in the glacial till deposit ranged from 5 to greater than 50 bpf, indicating the glacial till is medium stiff to stiff, or very dense, in consistency. Seven grain size analyses and four Atterberg limits tests performed on samples from the deposit resulted classifications of A-4 and A-6 under the AASHTO Soil Classification System and CL and CL-ML under the USCS. The moisture content of the tested samples ranged from approximately 13 to 24 percent. Table 1 summarizes the results of Atterberg limits tests conducted on the fine grained glacial till samples:

Sample No.	Visual Soil Description	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-RHC-101; 9D	SILT, some clay	24.4	Non-Plastic			
BB-RHC-102; 4D	SILT, trace clay	23.8	Non-Plastic			
BB-RHC-102; 6D	SILT, trace clay	23.6	38	21	17	0.15
BB-RHC-102; 9D	SILT, little clay	21.6	26	22	4	-0.1

Table 1 – Summary of Atterberg Limits Tests

Interpretation of the Atterberg limits tests indicates the glacial till has low to medium plasticity. Further, the liquidity indices are near zero indicating the glacial till is overconsolidated.

5.5 Bedrock

Bedrock was encountered and cored in borings BB-RHC-101 and BB-RHC-102. Table 2 summarizes the approximate depths to the initial bedrock cores, the corresponding approximate elevations of the beginning of the core run, and the calculated bedrock RQD. It is likely that the intact bedrock surface at the boring location begins at some elevation above the elevation in Table 2. MaineDOT Drill Crew members observed drilling behavior that indicates the roller cone penetrated bedrock prior to beginning the recovered core run.

Boring	Station	Offset (ft)	Approximate Depth to Bedrock Core Sample (ft)	Approximate Elevation of Bedrock Core Sample (ft)	RQD of Highest Bedrock Core Sample (%)
BB-RHC-101	7+56	9.8 ft Lt.	71.3	1454.1	22
BB-RHC-102	8+44	8.7 ft Lt.	72.2	1453.2	0

Table 2 – Summary of Approximate Bedrock Core Depths, Elevations, and RQD

The bedrock recovered is identified as grey, fine to medium grained, interlaminated metasandstone and metaclaystone, fresh to slightly weathered, moderately hard, breaks along horizontal relic bedding offset by microfaults, breaks are infrequently infilled by silt and gravel, moderate to very close, occasionally interrupted by open, steep, joints. The RQD of the bedrock cores ranged from 0 to 35 percent correlating to a rock mass quality of very poor to poor. Detailed bedrock descriptions and the RQD of bedrock core runs are provided on the boring logs in Appendix A – Boring Logs and on Sheet 4 and 5 – Boring Logs.

5.6 Groundwater

Groundwater depths measured in the test borings ranged from 8.0 to 9.0 ft bgs. The measurements were recorded after completion of the test borings. Note that water was introduced into the borehole during drilling operations. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, river levels, and construction activities.

6.0 FOUNDATION ALTERNATIVES

The March 2017 Preliminary Design Report (PDR) identified pile-supported integral abutments as the most cost effective and preferred substructure type. Spread footings were eliminated early in preliminary design because of the scour risk at the project site. Jointless integral substructure/superstructure connections will allow for faster construction, lower maintenance costs, and a longer service life.

7.0 GEOTECHNICAL DESIGN CONSIDERATIONS AND RECOMMENDATIONS

The following sections provide geotechnical design considerations and recommendations for H-pile supported integral bridge abutments, which are the proposed substructures for the Hunter Cove Bridge replacement project.

7.1 Integral Abutment H-Piles

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

Pile lengths at the proposed abutments may be estimated based on Table 3:

Location	Approximate Bottom Elevation of Proposed Abutment (ft)	Approximate Top of Bedrock Elevation (ft)	Estimated Pile Lengths (ft)
Abutment No. 1	1516.5	1454.1	62.4
Abutment No. 2	1516.5	1453.2	63.3

Table 3 – Estimated Pile Lengths for Integral Abutments No. 1 and No. 2

The estimated pile lengths in Table 3 do not take into account locations where bedrock may be deeper or shallower than that encountered in the test borings, damaged pile, the additional five (5) feet of pile required for dynamic testing instrumentation (per ASTM D4945), additional pile length needed to accommodate leads and driving equipment, or additional pile length needed for embedment in the abutment or pile cap.

7.1.1 Strength Limit State Design

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

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

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

Per AASHTO LRFD Bridge Design Specifications 7th Edition with interim revisions through 2016 (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 design axial load may govern the design. Per LRFD Article 6.5.4.2, at the strength limit state, the axial resistance factor $\phi_c = 0.70$ and the flexural resistance factor $\phi_f = 1.0$ 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). H-piles shall also be analyzed for fixity using LPile® v2016 (LPile) software, or similar.

Abutment H-piles should be analyzed by the geotechnical engineer for determination of unbraced lengths and fixity using LPile. 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 H-pile sections were calculated for upper and lower unbraced pile segments, and for the lower braced pile segment. The controlling resistance shown in Table 4 is for the lower braced pile segment, using a resistance factor, $\phi_c = 0.50$ for severe driving conditions. The factored structural resistances for the approximated upper unbraced segments use an axial resistance factor $\phi_c = 0.70$ for combined axial and flexure are not provided in Table 4 because these did not govern. 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 LPile.

Geotechnical Resistance. The nominal axial geotechnical resistance in the strength limit state was calculated using the guidance in LRFD Article 10.7.3.2.3 which states the nominal bearing resistance of piles driven to point bearing on hard rock shall not exceed the structural pile resistances obtained from LRFD Article 6.9.4.1 with a resistance factor ϕ_c, of 0.50, for severe driving conditions applied. The resulting limiting factored geotechnical compressive resistances for piles driven to rock are provided in Table 4.

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, φ_{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 H-piles for the strength limit states are provided in Table 4. Supporting calculations are provided in Appendix C – Calculations.

Pile Section	Strength Limit State Factored Axial Pile Resistance			
	Structural Resistance[1] ϕ_c=0.50 (kips)	Controlling Geotechnical Resistance[2] ϕ_c=0.50 (kips)	Drivability Resistance[3] φ_{dyn} = 0.65 (kips)	Governing Axial Pile Resistance (kips)
HP 12 x 53	387[4]	387[4]	311	311
HP 12 x 74	545	545	359	359
HP 14 x 73	535[4]	535[4]	389	389
HP 14 x 89	652	652	422	422
HP 14 x 117	860	860	479 (620)[5]	479 (620)[5]

Table 4 – Factored Axial Compressive Resistances for H-Piles at Strength Limit States

[1] Structural resistances were calculated for approximated conditions. 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 LPile results using a resistance factor, ϕ_c=0.70, for combined axial loading and bending.
[2] Based on guidance in LRFD Article 10.7.3.2.3., *Piles Driven to Hard Rock*
[3] Uses a resistance factor, φ_{dyn} = 0.65, assuming the driving criteria is established by dynamic testing, and quality control by dynamic testing of at least two piles per site condition and no less than two percent of production piles.
[4] Does not consider resistance factors of slender elements. 12x53 and 14x73 H-pile sections may require additional reductions based upon structural performance.
[5] Drivability resistance based on a Delmag D19-42. Drivability resistance with a Delmag D36-32 shown in parentheses.

LRFD Article 10.7.3.2.3 states that the nominal axial compressive resistance of piles driven to hard rock is typically controlled by the structural resistance with a resistance factor for severe driving conditions applied. However, the estimated factored axial pile resistances from the drivability analyses for the H-pile sections are less than the controlling factored axial structural resistance per LRFD Article 10.7.3.2.3. Therefore, drivability controls and the recommended governing resistances for pile design are the drivability resistances provided in the rightmost column "Governing Axial Pile Resistance (kips)" in Table 4. The maximum applied factored axial pile load for the strength limit states should not exceed the governing factored pile resistance shown in Table 4 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 $\phi = 1.0$ should be used in accordance with LRFD Article 10.5.5.1. The exception is the overall global stability of the foundation which should be investigated at the Service I load combination and a resistance factor, ϕ, of 0.65.

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

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

Pile Section	Extreme and Service Limit State Factored Axial Pile Resistance			
	Structural Resistance[6] ϕ_c=1.0 (kips)	Controlling Geotechnical Resistance[7] ϕ_c=1.0 (kips)	Drivability Resistance ϕ_c=1.0 (kips)	Governing Axial Pile Resistance (kips)
HP 12 x 53	663[8]	775[8]	479	479
HP 12 x 74	939	1090	552	552
HP 14 x 73	964[8]	1070[8]	599	599
HP 14 x 89	1178	1305	649	649
HP 14 x 117	1558	1720	737 (954)[9]	737 (954)[9]

Table 5 – Factored Axial Compressive Resistances for H-Piles at 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, the estimated factored axial pile resistances from the drivability analyses for the H-pile sections are less than the controlling factored axial structural resistance per LRFD Article 10.7.3.2.3 and the nominal structural resistances. Therefore, drivability controls, and the recommended governing resistances for pile design are the resistances provided in the rightmost column "Governing Axial Pile Resistance (kips)" in Table 5. The maximum applied factored axial pile load for the extreme and service limit states should not exceed the governing factored pile resistance shown in Table 5 above.

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 10.7.3.12. 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 LPile software with pile head deflections, moments, and axial loads supplied by the structural engineer. The designer should utilize the

[6] Nominal structural resistances were calculated for an "unbraced" pile segment using a resistance factor, ϕ = 1.0. Factored structural resistances should be calculated for upper and lower unbraced pile segments determined by LPile 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 may require additional reductions based upon structural performance.
[9] Drivability resistance based on a Delmag D19-42. Drivability resistance with a Delmag D36-32 shown in parentheses.

results of the LPile analyses to recalculate axial compressive structural pile resistances based on unbraced pile segments and verify pile bending stresses do not exceed allowable stresses.

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

Soil Layer	Top Elevation of Layer (ft)	Layer Thickness (ft)	γ_e[10] (pcf)	ϕ[11] (deg) / S_u[12] (psf)	k_s[13] (pci) / ε_{50}[14]
Medium dense, Granular Borrow.	1526.0	9.5	125	32°	90
Medium dense to very dense, SAND.	1516.5	9.0	69	32°	60
Medium stiff to very stiff, SILT.	1507.5	16.0	72	500 psf	ε_{50}=0.10
Very dense, SAND.	1491.5	4.0	84	40°	125
Hard, SILT.	1487.5	10.0	69	4000 psf	ε_{50}=0.04
Very dense or hard, Glacial Till.	1477.5	24.3	85	4000 psf	ε_{50}=0.04

Table 6 – Soil Parameters for Generation of Soil-Resistance (p-y) Curves

7.1.4 Driven Pile Resistance and Pile Quality Control

The contract plans shall require the contractor to perform a wave equation analysis of the proposed pile-hammer system and conduct dynamic pile load tests with signal matching. The first pile driven at each abutment should be dynamically tested to confirm nominal pile resistance and verify the stopping criteria developed by the contractor in the wave equation analysis. Restrikes or additional dynamic tests will be required as part of the pile field quality control program should pile behavior vary radically between adjacent piles, should pile behavior indicate a pile is refusing on a boulder or in a cobble layer above bedrock, should the pile tip be not firmly embedded in bedrock, or if the pile "walk" out of position.

[10] Effective Unit Weight.
[11] Internal angle of friction.
[12] Undrained shear strength.
[13] Soil modulus constant.
[14] Strain at 50 percent of the ultimate stress.

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 may be terminated when the penetration is less than 0.5-inch in 10 consecutive blows.

7.2 Integral Abutment Design

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

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

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

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

Integral abutment sections shall be designed to withstand a lateral earth load equal to the passive pressure state. Calculation of passive earth pressures should assume a Coulomb passive earth pressure coefficient, K_p, of 6.73. Developing full passive pressure assumes that the ratio of lateral abutment movement to abutment height (y/H) exceeds 0.005. If the calculated displacements are significantly less than that required to develop full passive pressure the designer may consider using the Rankine passive earth pressure coefficient of

3.25. A load factor for passive earth pressure is not specified in LRFD. For purposes of the integral abutment backwall reinforcing steel design, use a maximum load factor (γ_{EH}) of 1.50 to calculate factored passive earth pressures.

Additional lateral earth pressure due to live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for abutments if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination of the surcharge load, is permitted per LRFD Article 3.11.6.5. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 7:

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. Drainage behind the structure shall be in accordance with MaineDOT BDG Section 5.4.2.13.

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

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

7.3 In-line Wingwalls

In-line, cantilevered “butterfly” wingwalls may be used in conjunction with the integral abutments. The wingwalls shall be designed for all relevant strength, service, and extreme limit states and load combinations specified in LRFD Articles 3.4.1, 11.5.5 and 11.6. The walls shall be designed to resist lateral earth pressures, vehicular loads, and collision loads, as well as, creep, temperature, and shrinkage deformations. The design of “butterfly” wingwalls shall account for the additional bending stresses resulting from the wingwall being cantilevered off the abutment. These additional bending stresses may require wingwalls longer than 10 feet to be independently supported.

In-line “butterfly” wingwalls shall be designed for passive earth pressure. Calculation of passive earth pressures may assume a Rankine passive earth pressure coefficient, K_p, of 3.25

assuming small wingwall movements. See Appendix C – Calculations for supporting documentation. A load factor for passive earth pressure is not specified in LRFD; use a maximum load factor (γ_{EH}) of 1.50 to calculate factored passive earth pressures.

The wingwalls shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) per LRFD Article 3.11.6.4. An at-rest earth pressure coefficient, K_o, of 0.47 should be used for live load surcharge loads placed upon wingwalls cantilevered off of abutments with the top of the wall restrained from movement. See Appendix C – Calculations for supporting documentation.

7.4 Settlement

Approximately 4 feet of fibrous peat was encountered beneath the fill unit west of the existing bridge. The project calls for an increase in the vertical profile of approximately 1.5 feet. The peat deposit will undergo elastic compression and consolidation in response to the increase of vertical overburden pressure. Elastic settlements resulting from this increase will be immediate and are anticipated to be on the order of 1.5 inches. Consolidation settlement in the peat is estimated to be on the order of 0.4 inches. See Appendix C – Calculations for supporting documentation.

LRFD Article 3.11.8 states that piles can be subject to downdrag when settlement is 0.4 inches or greater. The elastic settlement is anticipated to occur during construction or concurrent with the pile installation. Consolidation of the peat will occur after pile installation, but is not anticipated to result in significant downdrag loads on the piles.

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

7.5 Frost Protection

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

Foundations placed on the native soils should be designed with an appropriate embedment for frost protection. According to MaineDOT BDG Figure 5-1, Maine Design Freezing Index Map, Rangeley has a design freezing index (DFI) of approximately 2200 F-degree days. The anticipated coarse grained fill soils were assigned a water content of 20%. These components correlate to a frost depth of 6.9 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Houlton, Maine has a DFI from the Modberg database of approximately 2189 F-degree days. Houlton was selected because it lies along the same isoline as Rangeley and Rangeley is not available in the Modberg database. A water content of 20% was assumed. These components correlate to a frost depth of approximately 8.3 feet.

Based on the MaineDOT BDG methodology, it is recommended that foundations bearing on soil be designed with an embedment of approximately 6.9 feet for frost protection. See Appendix C – Calculations for supporting calculations.

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

7.6 Scour and Riprap

The PDR notes scour holes upstream and downstream of the existing structure. Elevations of scour hole bottoms are approximately 1506.0 and 1507.0 feet. One grain size analysis was performed on a sample of the glacial till deposit near these elevations to generate grain size curves for determining parameters to be used in scour analyses. Sample BB-RHC-102;4D was assumed to be similar in nature to the soils likely to be exposed to scour conditions. The following streambed grain size parameters may be used in scour analyses:

- Average diameter of particle at 50 percent passing, D_{50} = 0.03 mm (silt).
- Average diameter of particle at 95 percent passing, D_{95} = 0.50 mm (medium sand).

The grain size curves are included in Appendix B – Laboratory Test Results. Note that the provided D_{50} grain size parameter is below the lower bound particle size of 0.2 mm recommended in Hydraulic Engineering Circular No. 18. Sensitivity analyses on particle sizes with a lower bound particle size of 0.2 mm should also be considered to access the risk of scour at this project location.

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

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

Heavy riprap shall conform to MaineDOT Standard Specification 703.28 – Heavy Riprap. The toe of the riprap section shall be constructed at least 1 foot below the streambed elevation. The riprap section shall be underlain by a 1 foot thick layer of bedding material conforming MaineDOT Standard Specification 703.19 and Class 1 nonwoven erosion control geotextile per MaineDOT Standard Details 610(02) and 610(03).

7.7 Seismic Design Considerations

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

Parameter	Design Value
Peak Ground Acceleration (PGA)	0.074g
Acceleration Coefficient (A_S)	0.118g
S_{DS} (Period = 0.2 sec)	0.260g
S_{D1} (Period = 1.0 sec)	0.120g
Site Class	D
Seismic Zone	1

Table 8 – Seismic Design Parameters

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

See Appendix C – Calculations for supporting documentation.

7.8 Construction Considerations

The new integral abutments will be constructed behind the existing abutments to avoid placement of fills in the river. Construction of the abutments will require pile driving. The existing substructures and timber piling, if not removed entirely, may obstruct pile driving operations. The Contractor should excavate the portions of the existing abutments and foundations that conflict with the proposed piles and abutments by conventional excavation methods. Conventional excavation methods include; pre-augering, predrilling, spudding, use of rock chisels, down-hole hammers, or vibratory extraction of existing timber piles. Excavation by these methods shall be incidental to related pay items.

Temporary lateral earth support systems may be required to permit construction of driven pile foundations at the proposed abutments. Boring BB-RHC-101 encountered wood at approximately elevation 1507.0. The inspector's construction notes during construction of the existing bridge indicate one pile refused on what is likely logs and rocks comprising an old corduroy road at the existing Abutment No. 2. See Appendix D – Existing Structure Pile Lengths for historical construction records. The contractor should assume for purposes of bidding and construction that the logs and rocks are present in the entire causeway. The logs and rocks will impede pile driving efforts and may require excavation, pre-augering, predrilling, or spudding prior to driving pile to prevent pile damage. Excavation by these methods shall be incidental to related pay items.

Cobbles were encountered in the very dense glacial till soil layers. Sublayers of cemented sand, silt, and gravel were also encountered in the glacial till deposit. The contractor should assume the use of conventional excavation methods, pre-augering, predrilling, rock chisels, or down-hole hammers is necessary to clear obstructions and allow pile driving activities. The contractor should assume difficult pile driving even after the removal of obstructions. Care should be taken to drive H-piles within allowable tolerances without damaging the H-piles.

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

8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application of the proposed replacement of Hunter Cove Bridge in Rangeley, 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. These analyses and recommendations are based in part upon limited subsurface investigations at discrete exploratory 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 recommended that a geotechnical engineer be provided the opportunity for a review of the design and specifications in order that the earthwork and foundation recommendations and construction considerations presented in this report are be properly interpreted and implemented in the design and specifications.

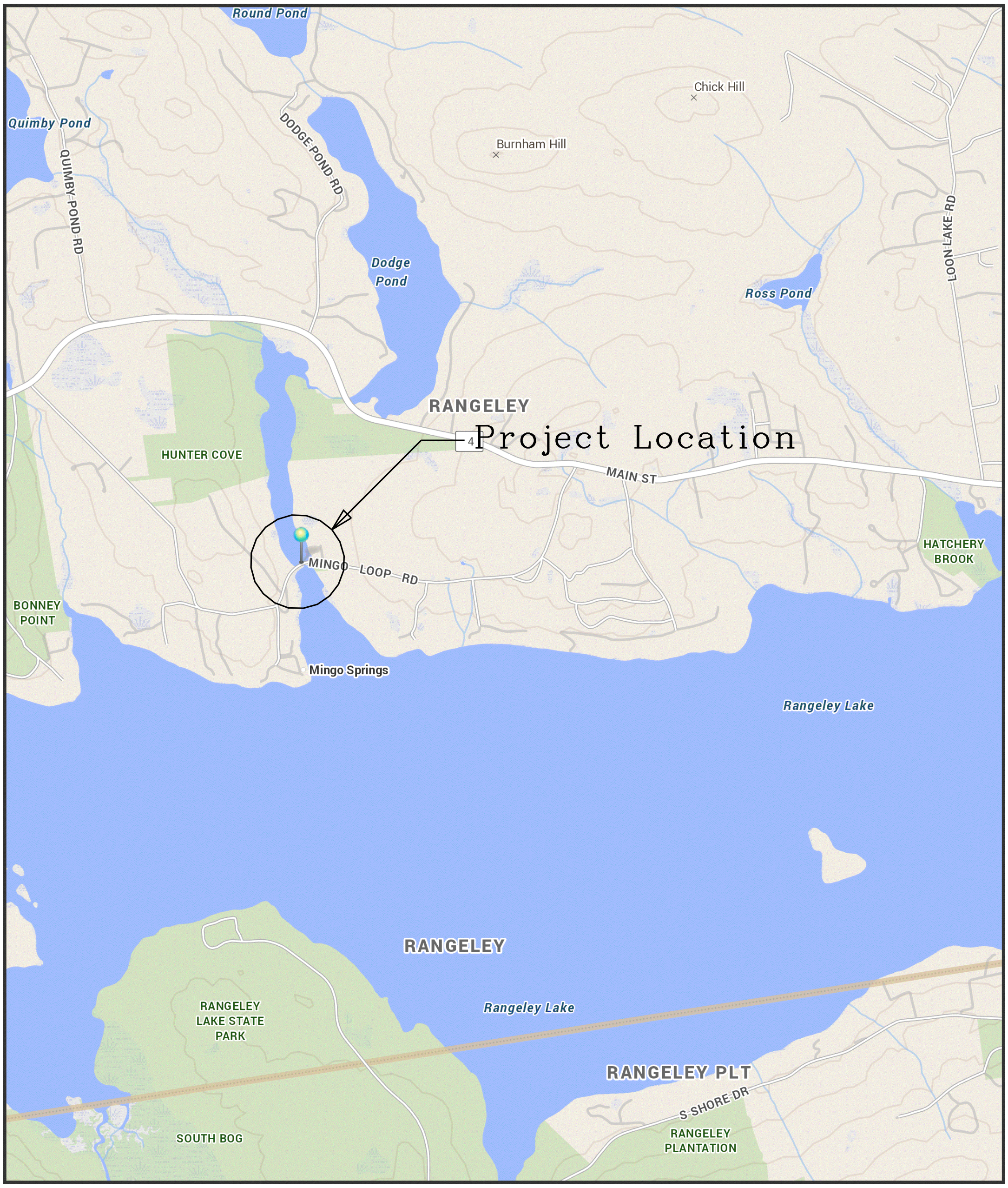
Sheets

Date:11/1/2017

Division: GEOTECH

Username: Terry.White

RANGELEY, MAINE



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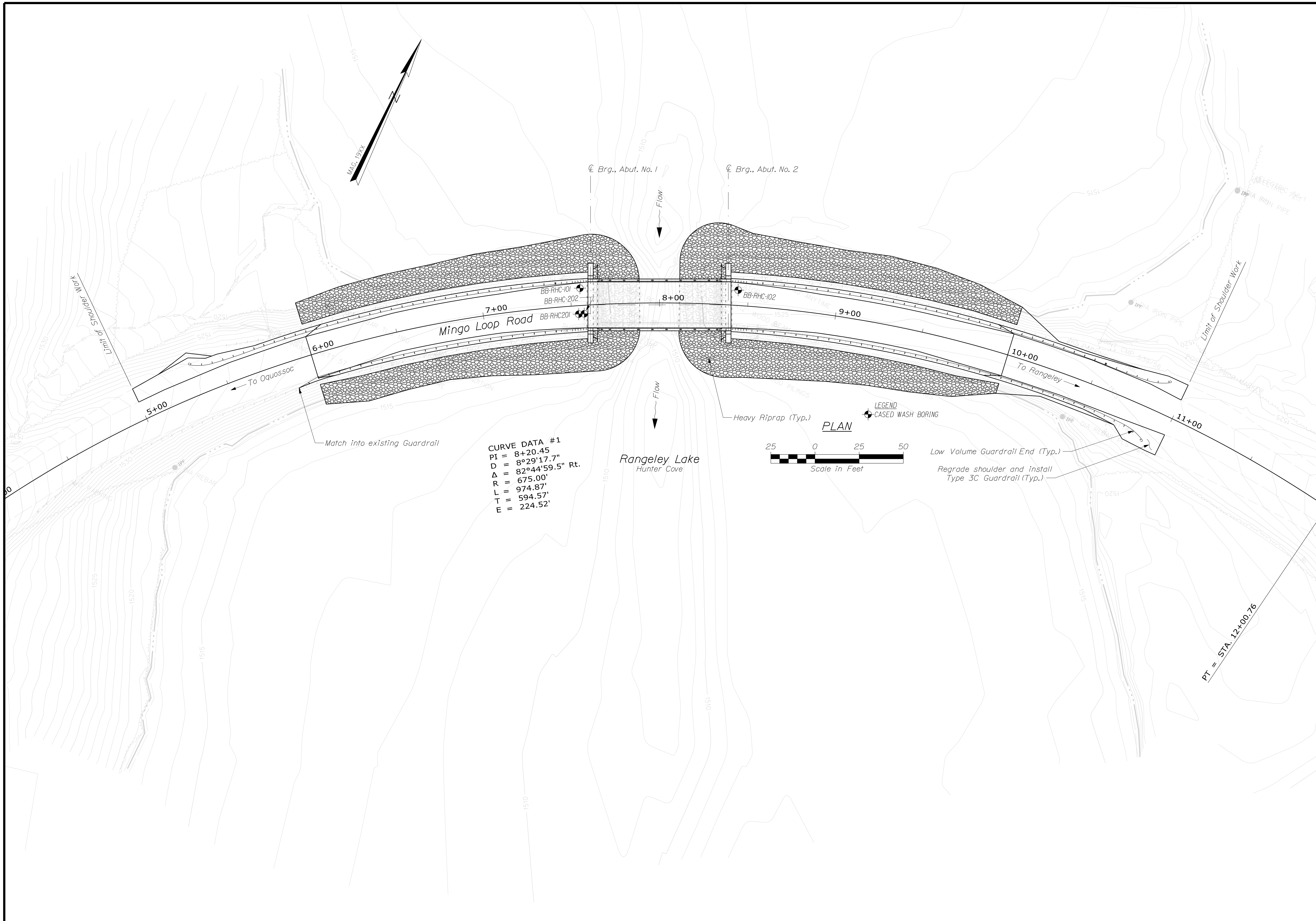
0.7 Miles

1 inch = 0.75 miles

Date: 11/1/2017
Time: 10:39:31 AM

SHEET NUMBER	HUNTER COVE BRIDGE HUNTER COVE	STATE OF MAINE DEPARTMENT OF TRANSPORTATION
1	RANGELEY FRANKLIN COUNTY	018955.00
OF 3	LOCATION MAP	BRIDGE NO. 2384 WIN 18955.00 BRIDGE PLANS



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

018955.00

WIN 18955.00

BRIDGE NO. 2384

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	M. WIGHT	
DESIGN-DETAILED	T. HJELM	
CHECKED-REVIEWED	D. SHAW	
DESIGN2-DETAILED2	B. SLAVEN	JAN 2017
DESIGN3-DETAILED3	T. WHITE	JAN 2017
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

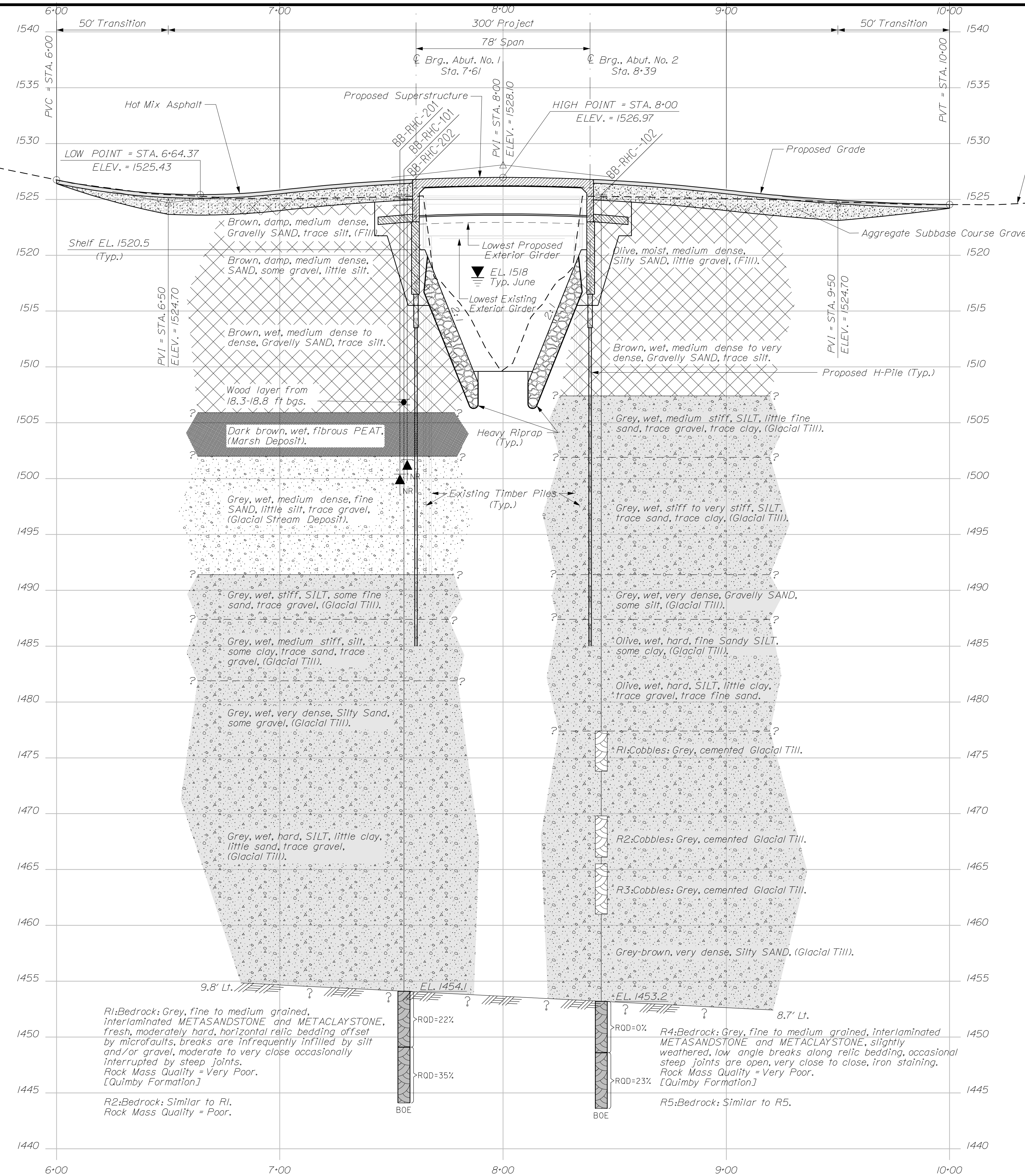
P.E. NUMBER

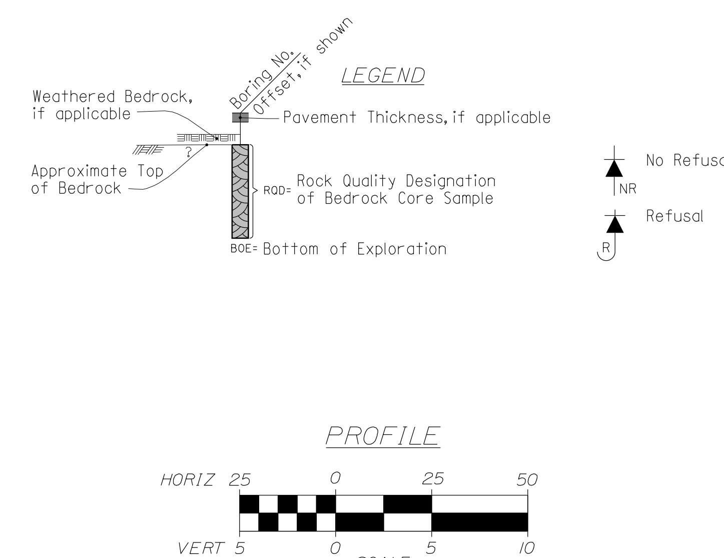
DATE

HUNTER COVE BRIDGE
HUNTER COVE
RANGELEY FRANKLIN COUNTY

BORING LOCATION PLAN

SHEET NUMBER
2
OF 5





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 and bedrock transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

018955.00

WIN 18955.00

BRIDGE PLANS

BRIDGE NO. 2384

		BY	DATE
PROJ. MANAGER	M. WIGHT		
DESIGN-DETAILED	T. HJELM	D. SHAW	
CHECKED-REVIEWED	B. SLAVEN	T. WHITE	JAN 2017
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

HUNTER COVE BRIDGE
HUNTER COVE
RANGELEY FRANKLIN COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
3
OF 5

Filename: ...\OD\GEOTECH\MSTA\007_ISP1.dgn Division: GEOTECH Username: Brandon.Slaven Date:12/5/2017

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

Project: Hunter Cove Bridge #2384 carries Mingo Loop Road over Hunter Cove
Location: Rangeley, Maine; Franklin County

Boring No.: BB-RHC-201
WIN: 18955.00

Driller:	MaineDOT	Elevation (ft.)	1524.9	Auger ID/OD:	5" Solid Stem
Operator:	Travis/James	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/16/2017; 09:30-11:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	7+53.8, 4.9 ft Rt.	Casing ID/OD:	NW-3"	Water Level*:	8.0 ft bgs.
Hammer Efficiency Factor: 0.854		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 = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field 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 = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	1524.5	5" HMA. —0.42	
									Hole caved in at 9.0 ft bgs.	
15							62			
							70			
							102			
	1D	24/12	18.50 - 20.50	4/8/5/4	13	19	a20		a20 blows for 0.5 ft.	
								1505.9	—19.00 Dark brown, wet, fibrous PEAT, (Marsh Deposit).	G#270400 A-1-b, SM WC=173.9% Ignition Loss 33.2%
20	2D/A	24/22	20.50 - 22.50	2/2/2/2	4	6			2D (20.5-22.0 ft bgs) Similar to above.	A-1-a, SW-SM WC=275.1% Ignition Loss 47.1%
								1502.9	—22.00 2D/A (22.0-22.5 ft bgs) Brown, moist, medium stiff, fine Sandy SILT, little organics.	
	3D	24/17	22.50 - 24.50	4/5/8/10	13	19		1501.4	—23.50 Grey, wet, medium dense, Silty fine SAND.	
25								1500.4	—24.50 Bottom of Exploration at 24.50 feet below ground surface. NO REFUSAL	

Remarks:

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

*
than those present at the time measurements were made.

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

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

Project: Hunter Cove Bridge #2384 carries Mingo Loop Road over Hunter Cove
Location: Rangeley, Maine; Franklin County

Boring No.: BB-RHC-101
WIN: 18955.00

Driller:	MaineDOT	Elevation (ft.)	1525.4	Auger ID/OD:	5" Solid Stem
Operator:	Wilder/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/3/2016-5/4/2016	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	7+55.6, 9.8 ft Lt.	Casing ID/OD:	HW-4"/NW-3"	Water Level*:	9.0 ft bgs.
Hammer Efficiency Factor: 0.908		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	1525.1	4" HMA. —0.33	
	1D	24/14	1.50 - 3.50	5/5/12/18	17	26			Brown, damp, medium dense, Gravelly SAND, trace silt, trace relic pavement, (Fill).	
5	2D	24/18	5.00 - 7.00	7/4/6/9	10	15			Brown, damp, medium dense, SAND, some gravel, little silt.	G#270703 A-1-b, SW-SM WC=4.1%
10	3D	24/4	10.00 - 12.00	4/8/3/6	11	17	36		Brown, wet, medium dense, Gravelly SAND, trace silt.	
							57			
							90			
							152			
							66			
15	4D	24/3	15.00 - 17.00	11/19/7/4	26	39	80		Similar to above, except dense.	
							53			
							57			
							172		Wood from 18.3-18.8 ft bgs.	
							94	1505.9	—19.50 Dark brown, wet, fibrous PEAT, (Marsh Deposit).	G#270704 A-2-4, SM/OL WC=197.9% Ignition Loss 45.9%
20	5D	24/22	20.00 - 22.00	2/2/2/4	4	6	31			
							54			
							76			
							87	1501.9	—23.50	
							75			
25	6D	24/14	25.00 - 27.00	5/4/5/8	9	14	80		Grey, wet, medium dense, fine SAND, little silt, trace gravel, (Glacial Stream Deposit).	G#270705 A-2-4, SM WC=21.4%
							88			
							119			
							113			
							116			
30	7D	24/13	30.00 - 32.00	3/2/5/5	7	11	78		Similar to above.	G#270706 A-2-4, SP-SM WC=24.2%
							115			
							147			
							130	1491.4	—34.00	
							133			
35	8D	24/14	35.00 - 37.00	2/4/3/3	7	11	121		Grey, wet, stiff, SILT, some fine sand, trace gravel, (Glacial Till).	G#270707 A-4, CL WC=23.7%
							123			
							136	1487.4	—38.00	
							118			
							129			
40	9D	24/19	40.00 - 42.00	2/1/2/6	3	5	60		Grey, wet, medium stiff, SILT, some clay, trace sand, trace gravel, (Glacial Till).	G#270708 A-4, CL WC=24.4% Non-Plastic
							96			
							112			
							154	1481.9	—43.50	
							181			
45	10D	24/18	45.00 - 47.00	5/11/33/36	44	67	OPEN HOLE		Grey, wet, very dense, Silty SAND, some gravel, (Glacial Till).	
50										

Remarks:

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

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

Page 1 of 2
Boring No.: BB-RHC-101

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

Project: Hunter Cove Bridge #2384 carries Mingo Loop Road over Hunter Cove
Location: Rangeley, Maine; Franklin County

Boring No.: BB-RHC-101
WIN: 18955.00

Driller:	MaineDOT	Elevation (ft.)	1525.4	Auger ID/OD:	5" Solid Stem
Operator:	Wilder/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/3/2016-5/4/2016	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	7+55.6, 9.8 ft Lt.	Casing ID/OD:	HW-4"/NW-3"	Water Level*:	9.0 ft bgs.
Hammer Efficiency Factor: 0.908		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
50	11D	8.4/8.4	50.00 - 50.70	50/55(2.4")	---				Similar to above.	
55	12D	18/16	55.00 - 56.50	31/58/55	113	171			Grey, wet, hard, SILT, little clay, little sand, trace gravel, (Glacial Till).	G#270709 A-4, CL WC=12.6%
60	13D	9.6/9.6	60.00 - 60.80	40/50(3.6")	---				Similar to above.	
65	14D	12/10	65.00 - 66.00	50/55	---		246		Similar to above.	
							53			
							82			
							114			
							285		a300 blows for 0.1 ft.	
70	MD	1.2/0	70.00 - 70.10	30(1.2")	---		a300			
	R1	60/60	71.30 - 76.30	RQD = 22%			NQ-2	1454.1	—71.30 Top of Bedrock at Elev. 1454.1 ft. R1:Bedrock: Grey, fine to medium grained, interlaminated METASANDSTONE and METACLAYSTONE, fresh, moderately hard, horizontal relic bedding offset by microfaults, breaks are infrequently infilled by silt and/or gravel, moderate to very close occasionally interrupted by steep joints. Rock Mass Quality = Very Poor. [Quimby Formation] R1:Core Times (min:sec) 71.3-72.3 ft (1:17) 72.3-73.3 ft (1:09) 73.3-74.3 ft (1:23) 74.3-75.3 ft (1:33) 75.3-76.3 ft (1:33) 100% Recovery	
	R2	60/60	76.30 - 81.30	RQD = 35%				1444.1	R2:Bedrock: Similar to R1. Rock Mass Quality = Poor. R2:Core Times (min:sec) 76.3-77.3 ft (1:25) 77.3-78.3 ft (3:00) 78.3-79.3 ft (2:06) 79.3-80.3 ft (2:22) 80.3-91.3 ft (2:40) 100% Recovery —81.30 Bottom of Exploration at 81.30 feet below ground surface.	
100										

Remarks:

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

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

Page 2 of 2
Boring No.: BB-RHC-101

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

Project: Hunter Cove Bridge #2384 carries Mingo Loop Road over Hunter Cove
Location: Rangeley, Maine; Franklin County

Boring No.: BB-RHC-202
WIN: 18955.00

Driller:	MaineDOT	Elevation (ft.)	1525.0	Auger ID/OD:	5" Solid Stem
Operator:	Travis/James	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/16/2017; 12:00-14:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	7+57.1, 4.8 ft Rt.	Casing ID/OD:	HW-4"	Water Level*:	8.0 ft bgs.
Hammer Efficiency Factor: 0.854		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 = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field 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 = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	1524.6	5" HMA. —0.42	
20								1505.0	—20.00 Set in HW Casing to 20.0 ft bgs.	
	MU	30/0	20.00 - 22.50							
	1U	9.6/6	22.50 - 23.30						Small amount of PEAT.	#271328 Ignition Loss 45.2%
								1501.7	—23.30 Bottom of Exploration at 23.30 feet below ground surface. NO REFUSAL	
25										

Remarks:

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

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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
018955.00
WIN 18955.00
BRIDGE NO. 2384
BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	M. WIGHT		
DESIGN-DETAILED	T. HJELM	D. SHAW	
CHECKED-REVIEWED	-----	-----	-----
DESIGN2-DETAILED2	B.SLAVEN	T.WHITE	DEC 2017
DESIGN3-DETAILED3	-----	-----	-----
REVISIONS 1	-----	-----	-----
REVISIONS 2	-----	-----	-----
REVISIONS 3	-----	-----	-----
REVISIONS 4	-----	-----	-----
FIELD CHANGES	-----	-----	-----

SIGNATURE
P.E. NUMBER
DATE

HUNTER COVE BRIDGE
HUNTER COVE
RANGELEY FRANKLIN COUNTY
BORING LOGS

SHEET NUMBER
4
OF 5

Filename: ...\MSTA\008_BORING LOGS1.dgn   Division: GEOTECH   Username: Brandon.Slaven   Date:12/5/2017

Filename: ...\MSTA\009_BORING LOGS2.dgn   Division: GEOTECH   Username: Brandon.Slaven   Date:12/5/2017

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Hunter Cove Bridge #2384 carries Mingo Loop Road over Hunter Cove
Location: Rangeley, Maine: Franklin County

Boring No.: BB-RHC-102

WIN: 18955.00

Driller:	MaineDOT	Elevation (ft.)	1525.4	Auger ID/OD:	5" Solid Stem
Operator:	Wilder/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	4/27/2016-4/28/2016	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	8+44, 8.7 ft Lt.	Casing ID/OD:	NW-3"	Water Level*:	None Observed
Hammer Efficiency Factor: 0.908		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 = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field 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 = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	1525.2	3" HMA. — 0.25 — Brown, damp, Gravelly SAND, trace silt, (Fill).	
5	1D	24/14	5.00 - 7.00	4/5/8/11	13	20			Olive, moist, medium dense, Silty SAND, little gravel, (Fill).	
10	2D	24/3	10.00 - 12.00	5/9/3/3	12	18	42		Brown, wet, medium dense, Gravelly SAND, trace silt, (Fill).	
							104			
							109			
							84			
							43			
15	3D	24/7	15.00 - 17.00	75/27/8/12	35	53	28		Similar to above, except very dense.	
							30			
							112			
							48	1507.4	18.00	
							13			
20	4D	24/20	20.00 - 22.00	3/2/2/8	4	6	14		Grey, wet, medium stiff, SILT, little fine sand, trace clay, trace gravel, (Glacial Till).	G#270710 A-4, CL WC=23.8% Non-Plastic
							22			
							57			
							48	1501.9	23.50	
							64			
25	5D	24/13	25.00 - 27.00	4/7/6/8	13	20	16		Olive, wet, very stiff, SILT, trace sand, trace clay, (Glacial Till).	G#270711 A-4, CL WC=20.3%
							31			
							42			
							39			
							28			
30	6D	24/18	30.00 - 32.00	3/4/2/8	6	9	34		Similar to above, except stiff.	G#270712 A-6, CL WC=23.6% LL=38 PL=21 PI=17
							46			
							44			
							48			
	7D	24/13	34.00 - 36.00	16/22/19/21	41	62	60	1491.4	34.00 — Grey, wet, very dense, Gravelly SAND, some silt, (Glacial Till).	
35							71			
							58			
							70			
							145	1487.4	38.00	
	8D	24/22	39.00 - 41.00	18/20/18/21	38	58	87		Olive, wet, hard, fine Sandy SILT, some clay, (Glacial Till).	
40							177			
							181			
							232			
							70			
	9D	24/20	44.00 - 46.00	12/19/19/22	38	58	129		Olive, wet, hard, SILT, little clay, trace fine sand, trace gravel, (Glacial Till).	G#270713 A-4, CL-ML WC=21.6% LL=26 PL=22 PI=4
45							123			
							137			
							104			
	R1	43.2/24	48.00 - 51.60				120 NQ-2	1477.4	48.00 — R1: Grey, weakly to strongly cemented, sand, gravel, silt, cobbles, (Glacial Till).	
							116			
50										

Remarks:

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Page 1 of 2

Boring No.: BB-RHC-102

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Hunter Cove Bridge #2384 carries Mingo Loop Road over Hunter Cove
Location: Rangeley, Maine: Franklin County

Boring No.: BB-RHC-102

WIN: 18955.00

Driller: MaineDOT | Elevation (ft.) 1525.4 | Auger ID/OD: 5" Solid Stem
Operator: Wilder/Daggett | Datum: NAVD88 | Sampler: Standard Split Spoon
Logged By: B. Wilder | Rig Type: CME 45C | Hammer Wt./Fall: 140#/30"
Date Start/Finish: 4/27/2016-4/28/2016 | Drilling Method: Cased Wash Boring | Core Barrel: NQ-2"
Boring Location: 8+44, 8.7 ft Lt. | Casing ID/OD: NW-3" | Water Level*: None Observed
Hammer Efficiency Factor: 0.908 | 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 = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field 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 = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
50							83			
							88			
							104			
							136			
							100			
55	R2	44.4/40	55.60 - 59.30				47 NQ-2		R2: Grey, weakly to strongly cemented, sand, gravel, silt, cobbles, (Glacial Till).	
							58			
							58			
							79			
	R3	54/15	59.90 - 64.40				80 NQ-2			
60							66		R3: Grey, weakly to strongly cemented, sand, gravel, silt, cobbles, (Glacial Till).	
							69			
							100			
							150		Roller Coned ahead to 70.0 ft bgs.	
							187			
65							OPEN		Cobble from 64.8-65.3 ft bgs.	
							HOLE			
70	10D	3.6/3.6	70.00 - 70.30	50(3.6")	---				Grey brown, very dense, Silty SAND, (Glacial Till).	
	R4	55.2/44	72.20 - 76.80	RQD = 0%			NQ-2	1453.2	72.20 — Top of Bedrock at Elev. 1453.2 ft. R4:Bedrock: Grey, fine to medium grained, interlaminated METASANDSTONE and METACLAYSTONE, slightly weathered, low angle breaks along relic bedding, occasional steep joints are open, very close to close, iron staining. Rock Mass Quality = Very Poor. [Quimby Formation] R4:Core Times (min:sec) 72.2-73.2 ft (2:00) 73.2-74.2 ft (2:10) 74.2-75.2 ft (2:05) 75.2-76.2 ft (2:05) 76.2-76.8 ft (3:00) 79% Recovery Core Blocked	
75										
	R5	60/60	76.80 - 81.80	RQD = 23%					R5:Bedrock: Similar to R4. Rock Mass Quality = Very Poor. R5:Core Times (min:sec) 76.8-77.8 ft (3:00) 77.8-78.8 ft (2:00) 78.8-79.8 ft (2:30) 79.8-80.8 ft (2:20) 80.8-81.8 ft (2:10) 100% Recovery	
80										
								1443.6	81.80 — Bottom of Exploration at 81.80 feet below ground surface.	
85										
90										
95										
100										

Remarks:

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Page 2 of 2

Boring No.: BB-RHC-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

018955.00

BRIDGE NO. 2384

WIN
18955.00

BRIDGE PLANS

PROJ. MANAGER	M. WIGHT	BY	DATE
DESIGN-DETAILED	T. HJELM	D. SHAW	
CHECKED-REVIEWED	------	------	------
DESIGN2-DETAILED2	B.SLAVEN	T.WHITE	DEC 2017
DESIGN3-DETAILED3	------	------	------
REVISIONS 1	------		------
REVISIONS 2	------		------
REVISIONS 3	------		------
REVISIONS 4	------		------
FIELD CHANGES	------		------

SIGNATURE

P.E. NUMBER

DATE

HUNTER COVE BRIDGE
HUNTER COVE
RANGELEY FRANKLIN COUNTY

BORING LOGS

SHEET NUMBER

5

OF 5

Appendix A

Boring Logs

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Hunter Cove Bridge #2384 carries Mingo Loop Road over Hunter Cove
Location: Rangeley, Maine; Franklin County

Boring No.: BB-RHC-101
WIN: 18955.00

Driller:	MaineDOT	**Elevation (ft.)**	1525.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/3/2016-5/4/2016	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	7+55.6, 9.8 ft Lt.	**Casing ID/OD:**	HW-4"/NW-3"	**Water Level*:**	9.0 ft bgs.
Hammer Efficiency Factor:	0.908	**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 = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane 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	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	1525.1		4" HMA. 0.33	
	1D	24/14	1.50 - 3.50	5/5/12/18	17	26				Brown, damp, medium dense, Gravelly SAND, trace silt, trace relic pavement, (Fill).	
5	2D	24/18	5.00 - 7.00	7/4/6/9	10	15				Brown, damp, medium dense, SAND, some gravel, little silt.	G#270703 A-1-b, SW-SM WC=4.1%
10	3D	24/4	10.00 - 12.00	4/8/3/6	11	17	36			Brown, wet, medium dense, Gravelly SAND, trace silt.	
							57				
							90				
							152				
							66				
15	4D	24/3	15.00 - 17.00	11/19/7/4	26	39	36			Similar to above, except dense.	
							53				
							57				
							172			Wood from 18.3-18.8 ft bgs.	
							94	1505.9		19.50	
20	5D	24/22	20.00 - 22.00	2/2/2/4	4	6	31			Dark brown, wet, fibrous PEAT, (Marsh Deposit).	G#270704 A-2-4, SM/OL WC=197.9% Ignition Loss 45.9%
							54				
							76				
							87	1501.9		23.50	
							75				
25											

Remarks:

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Boring No.: BB-RHC-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Hunter Cove Bridge #2384 carries Mingo Loop Road over Hunter Cove
Location: Rangeley, Maine; Franklin County

Boring No.: BB-RHC-101
WIN: 18955.00

Driller:	MaineDOT	**Elevation (ft.)**	1525.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/3/2016-5/4/2016	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	7+55.6, 9.8 ft Lt.	**Casing ID/OD:**	HW-4"/NW-3"	**Water Level*:**	9.0 ft bgs.
Hammer Efficiency Factor:	0.908	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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SSA = Solid Stem Auger
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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
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C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/14	25.00 - 27.00	5/4/5/8	9	14	80			Grey, wet, medium dense, fine SAND, little silt, trace gravel, (Glacial Stream Deposit).	G#270705 A-2-4, SM WC=21.4%
							88				
							119				
							113				
							116				
30	7D	24/13	30.00 - 32.00	3/2/5/5	7	11	78			Similar to above.	G#270706 A-2-4, SP-SM WC=24.2%
							115				
							147				
							130				
							133	1491.4		34.00	
35	8D	24/14	35.00 - 37.00	2/4/3/3	7	11	121			Grey, wet, stiff, SILT, some fine sand, trace gravel, (Glacial Till).	G#270707 A-4, CL WC=23.7%
							123				
							136				
							118	1487.4		38.00	
							129				
40	9D	24/19	40.00 - 42.00	2/1/2/6	3	5	60			Grey, wet, medium stiff, SILT, some clay, trace sand, trace gravel, (Glacial Till).	G#270708 A-4, CL WC=24.4% Non-Plastic
							96				
							112				
							154	1481.9		43.50	
							181				
45	10D	24/18	45.00 - 47.00	5/11/33/36	44	67	OPEN HOLE			Grey, wet, very dense, Silty SAND, some gravel, (Glacial Till).	
50											

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Hunter Cove Bridge #2384 carries Mingo Loop Road over Hunter Cove Location: Rangeley, Maine; Franklin County	Boring No.: BB-RHC-101 WIN: 18955.00

Driller:	MaineDOT	Elevation (ft.)	1525.4	Auger ID/OD:	5" Solid Stem
Operator:	Wilder/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/3/2016-5/4/2016	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	7+55.6, 9.8 ft Lt.	Casing ID/OD:	HW-4"/NW-3"	Water Level*:	9.0 ft bgs.
Hammer Efficiency Factor:	0.908	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	11D	8.4/8.4	50.00 - 50.70	50/55(2.4")	---					Similar to above.	
55	12D	18/16	55.00 - 56.50	31/58/55	113	171				Grey, wet, hard, SILT, little clay, little sand, trace gravel, (Glacial Till).	G#270709 A-4, CL WC=12.6%
60	13D	9.6/9.6	60.00 - 60.80	40/50(3.6")	---					Similar to above.	
65	14D	12/10	65.00 - 66.00	50/55	---		246			Similar to above.	
							53				
							82				
							114				
							285				
70	MD	1.2/0	70.00 - 70.10	30(1.2")	---		a300			a300 blows for 0.1 ft.	
	R1	60/60	71.30 - 76.30	RQD = 22%			NQ-2	1454.1		71.30 Top of Bedrock at Elev. 1454.1 ft. R1:Bedrock: Grey, fine to medium grained, interlaminated METASANDSTONE and METACLAYSTONE, fresh, moderately hard, horizontal relic bedding offset by microfaults, breaks are infrequently infilled by silt and/or gravel, moderate to very close occasionally interrupted by steep joints. Rock Mass Quality = Very Poor.	
75											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Hunter Cove Bridge #2384 carries Mingo Loop Road over Hunter Cove
Location: Rangeley, Maine; Franklin County

Boring No.: BB-RHC-101
WIN: 18955.00

Driller:	MaineDOT	**Elevation (ft.)**	1525.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/3/2016-5/4/2016	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	7+55.6, 9.8 ft Lt.	**Casing ID/OD:**	HW-4"/NW-3"	**Water Level*:**	9.0 ft bgs.
Hammer Efficiency Factor:	0.908	**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 = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field 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 = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75										[Quimby Formation] R1:Core Times (min:sec) 71.3-72.3 ft (1:17) 72.3-73.3 ft (1:09) 73.3-74.3 ft (1:23) 74.3-75.3 ft (1:33) 75.3-76.3 ft (1:33) 100% Recovery	
	R2	60/60	76.30 - 81.30	RQD = 35%						R2:Bedrock: Similar to R1. Rock Mass Quality = Poor. R2:Core Times (min:sec) 76.3-77.3 ft (1:25) 77.3-78.3 ft (3:00) 78.3-79.3 ft (2:06) 79.3-80.3 ft (2:22) 80.3-91.3 ft (2:40) 100% Recovery	
80											
								1444.1		81.30	
										Bottom of Exploration at 81.30 feet below ground surface.	
85											
90											
95											
100											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Hunter Cove Bridge #2384 carries Mingo Loop Road over Hunter Cove
Location: Rangeley, Maine; Franklin County

Boring No.: BB-RHC-102
WIN: 18955.00

Driller:	MaineDOT	**Elevation (ft.)**	1525.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/27/2016-4/28/2016	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	8+44, 8.7 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.908	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
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RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane 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	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	1525.2		3" HMA. — 0.25 Brown, damp, Gravelly SAND, trace silt, (Fill).	
5	1D	24/14	5.00 - 7.00	4/5/8/11	13	20				Olive, moist, medium dense, Silty SAND, little gravel, (Fill).	
10	2D	24/3	10.00 - 12.00	5/9/3/3	12	18	42			Brown, wet, medium dense, Gravelly SAND, trace silt, (Fill).	
							104				
							109				
							84				
							43				
15	3D	24/7	15.00 - 17.00	75/27/8/12	35	53	28			Similar to above, except very dense.	
							30				
							112	1507.4		18.00	
							48				
							13				
20	4D	24/20	20.00 - 22.00	3/2/2/8	4	6	14			Grey, wet, medium stiff, SILT, little fine sand, trace clay, trace gravel, (Glacial Till).	G#270710 A-4, CL WC=23.8% Non-Plastic
							22				
							57				
							48	1501.9		23.50	
							64				
25											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Hunter Cove Bridge #2384 carries Mingo Loop Road over Hunter Cove
Location: Rangeley, Maine; Franklin County

Boring No.: BB-RHC-102
WIN: 18955.00

Driller:	MaineDOT	**Elevation (ft.)**	1525.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/27/2016-4/28/2016	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	8+44, 8.7 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.908	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/13	25.00 - 27.00	4/7/6/8	13	20	16			Olive, wet, very stiff, SILT, trace sand, trace clay, (Glacial Till).	G#270711 A-4, CL WC=20.3%
							31				
							42				
							39				
							28				
30	6D	24/18	30.00 - 32.00	3/4/2/8	6	9	34			Similar to above, except stiff.	G#270712 A-6, CL WC=23.6% LL=38 PL=21 PI=17
							46				
							44				
							48	1491.4		34.00	
	7D	24/13	34.00 - 36.00	16/22/19/21	41	62	60			Grey, wet, very dense, Gravelly SAND, some silt, (Glacial Till).	
35							71				
							58				
							70	1487.4		38.00	
							145				
	8D	24/22	39.00 - 41.00	18/20/18/21	38	58	87			Olive, wet, hard, fine Sandy SILT, some clay, (Glacial Till).	
40							177				
							181				
							232				
							70				
	9D	24/20	44.00 - 46.00	12/19/19/22	38	58	129			Olive, wet, hard, SILT, little clay, trace fine sand, trace gravel, (Glacial Till).	G#270713 A-4, CL-ML WC=21.6% LL=26 PL=22 PI=4
45							123				
							137				
							104	1477.4		48.00	
	R1	43.2/24	48.00 - 51.60				120 NQ-2			R1: Grey, weakly to strongly cemented, sand, gravel, silt, cobbles, (Glacial Till).	
50							116				

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Hunter Cove Bridge #2384 carries Mingo Loop Road over Hunter Cove Location: Rangeley, Maine; Franklin County	Boring No.: BB-RHC-102 WIN: 18955.00

Driller:	MaineDOT	**Elevation (ft.)**	1525.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/27/2016-4/28/2016	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	8+44, 8.7 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.908	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
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WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
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WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50							83				
							88				
							104				
							136				
							100				
55	R2	44.4/40	55.60 - 59.30				47 NQ-2			R2: Grey, weakly to strongly cemented, sand, gravel, silt, cobbles, (Glacial Till).	
							58				
							58				
							79				
	R3	54/15	59.90 - 64.40				80 NQ-2				
60							66			R3: Grey, weakly to strongly cemented, sand, gravel, silt, cobbles, (Glacial Till).	
							69				
							100				
							150			Roller Coned ahead to 70.0 ft bgs.	
							187				
65							OPEN			Cobble from 64.8-65.3 ft bgs.	
							HOLE				
70	10D	3.6/3.6	70.00 - 70.30	50(3.6")	---					Grey brown, very dense, Silty SAND, (Glacial Till).	
	R4	55.2/44	72.20 - 76.80	RQD = 0%			NQ-2	1453.2		72.20 Top of Bedrock at Elev. 1453.2 ft. R4:Bedrock: Grey, fine to medium grained, interlaminated METASANDSTONE and METACLAYSTONE, slightly weathered, low angle breaks along relic bedding, occasional steep joints are open, very close to close, iron staining.	
75											

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Boring No.: BB-RHC-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Hunter Cove Bridge #2384 carries Mingo Loop Road over Hunter Cove
Location: Rangeley, Maine; Franklin County

Boring No.: BB-RHC-102
WIN: 18955.00

Driller:	MaineDOT	**Elevation (ft.)**	1525.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/27/2016-4/28/2016	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	8+44, 8.7 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.908	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75										Rock Mass Quality = Very Poor. [Quimby Formation] R4:Core Times (min:sec) 72.2-73.2 ft (2:00) 73.2-74.2 ft (2:10) 74.2-75.2 ft (2:05) 75.2-76.2 ft (2:05) 76.2-76.8 ft (3:00) 79% Recovery Core Blocked	
	R5	60/60	76.80 - 81.80	RQD = 23%							
80								1443.6		R5:Bedrock: Similar to R4. Rock Mass Quality = Very Poor. R5:Core Times (min:sec) 76.8-77.8 ft (3:00) 77.8-78.8 ft (2:00) 78.8-79.8 ft (2:30) 79.8-80.8 ft (2:20) 80.8-81.8 ft (2:10) 100% Recovery 81.80 **Bottom of Exploration at 81.80 feet below ground surface.**	
85											
90											
95											
100											

Remarks:

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Boring No.: BB-RHC-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Hunter Cove Bridge #2384 carries Mingo Loop Road over Hunter Cove
Location: Rangeley, Maine; Franklin County

Boring No.: BB-RHC-201
WIN: 18955.00

Driller:	MaineDOT	**Elevation (ft.)**	1524.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Travis/James	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/16/2017; 09:30-11:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	7+53.8, 4.9 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	8.0 ft bgs.
Hammer Efficiency Factor:	0.854	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane 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	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	1524.5		5" HMA. 0.42	
5											
										Hole caved in at 9.0 ft bgs.	
10											
15							62				
							70				
							102				
	1D	24/12	18.50 - 20.50	4/8/5/4	13	19	a20			a20 blows for 0.5 ft.	
								1505.9		19.00 Dark brown, wet, fibrous PEAT, (Marsh Deposit).	G#270400 A-1-b, SM WC=173.9% Ignition Loss 33.2%
20	2D/A	24/22	20.50 - 22.50	2/2/2/2	4	6				2D (20.5-22.0 ft bgs) Similar to above.	A-1-a, SW-SM WC=275.1% Ignition Loss 47.1%
	3D	24/17	22.50 - 24.50	4/5/8/10	13	19		1502.9		22.00 2D/A (22.0-22.5 ft bgs) Brown, moist, medium stiff, fine Sandy SILT, little organics.	
								1501.4		23.50 Grey, wet, medium dense, Silty fine SAND.	
25								1500.4		24.50	

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Hunter Cove Bridge #2384 carries Mingo Loop Road over Hunter Cove Location: Rangeley, Maine; Franklin County	Boring No.: BB-RHC-201 WIN: 18955.00

Driller:	MaineDOT	**Elevation (ft.)**	1524.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Travis/James	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/16/2017; 09:30-11:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	7+53.8, 4.9 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	8.0 ft bgs.
Hammer Efficiency Factor:	0.854	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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HSA = Hollow Stem Auger
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										**Bottom of Exploration at 24.50 feet below ground surface.** NO REFUSAL	
30											
35											
40											
45											
50											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Hunter Cove Bridge #2384 carries Mingo Loop Road over Hunter Cove
Location: Rangeley, Maine; Franklin County

Boring No.: BB-RHC-202
WIN: 18955.00

Driller:	MaineDOT	**Elevation (ft.)**	1525.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Travis/James	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/16/2017; 12:00-14:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	7+57.1, 4.8 ft Rt.	**Casing ID/OD:**	HW-4"	**Water Level*:**	8.0 ft bgs.
Hammer Efficiency Factor:	0.854	**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 = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field 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 = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane 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	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	1524.6		5" HMA. 0.42	
5											
10											
15											
20								1505.0		20.00	
	MU	30/0	20.00 - 22.50							Set in HW Casing to 20.0 ft bgs.	
	1U	9.6/6	22.50 - 23.30							Small amount of PEAT.	#271328 Ignition Loss 45.2%
								1501.7		23.30 **Bottom of Exploration at 23.30 feet below ground surface.** NO REFUSAL	
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.

Boring No.: BB-RHC-202

Appendix B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Rangeley   Work Number: 18955.00

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-RHC-101, 2D	7+55.6	9.8 Lt.	5.0-7.0	270703	1	4.1			SW-SM	A-1-b	0
BB-RHC-101, 5D	7+55.6	9.8 Lt.	20.0-22.0	270704	1	197.9			SM/OL	A-2-4	II
BB-RHC-101, 6D	7+55.6	9.8 Lt.	25.0-27.0	270705	1	21.4			SM	A-2-4	II
BB-RHC-101, 7D	7+55.6	9.8 Lt.	30.0-32.0	270706	1	24.2			SP-SM	A-2-4	0
BB-RHC-101, 8D	7+55.6	9.8 Lt.	35.0-37.0	270707	2	23.7			CL	A-4	IV
BB-RHC-101, 9D	7+55.6	9.8 Lt.	40.0-42.0	270708	2	24.4	-N	P-	CL	A-4	IV
BB-RHC-101, 12D	7+55.6	9.8 Lt.	55.0-56.5	270709	2	12.6			CL	A-4	IV
BB-RHC-102, 4D	8+44	8.7 Lt.	20.0-22.0	270710	3	23.8	-N	P-	CL	A-4	IV
BB-RHC-102, 5D	8+44	8.7 Lt.	25.0-27.0	270711	3	20.3			CL	A-4	IV
BB-RHC-102, 6D	8+44	8.7 Lt.	30.0-32.0	270712	3	23.6	38	17	CL	A-6	III
BB-RHC-102, 9D	8+44	8.7 Lt.	44.0-46.0	270713	3	21.6	26	4	CL-ML	A-4	IV
BB-RHC-201, 1D	7+53.8	4.9 Rt.	19.0-20.5	270400	3	173.9			SM	A-1-b	II
BB-RHC-201, 2D	7+53.8	4.9 Rt.	20.5-22.0	270399	3	275.1			SW-SM	A-1-a	0
							Loss on Ignition (T 267)				
BB-RHC-101, 5D	7+55.6	9.8 Lt.	20.0-22.0	270704	1	197.9	45.9%				
BB-RHC-201, 1D	7+53.8	4.9 Rt.	19.0-20.5	270400	3	173.9	33.2%				
BB-RHC-201, 2D	7+53.8	4.9 Rt.	20.5-22.0	270399	3	275.1	47.1%				
*BB-RHC-202, 1U	7+57.1	4.8 Rt.	20.0-23.3	271328	---	---	45.2%				
*BB-RHC-202, 1U "Consolidation not performed due to tube damage."											

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)

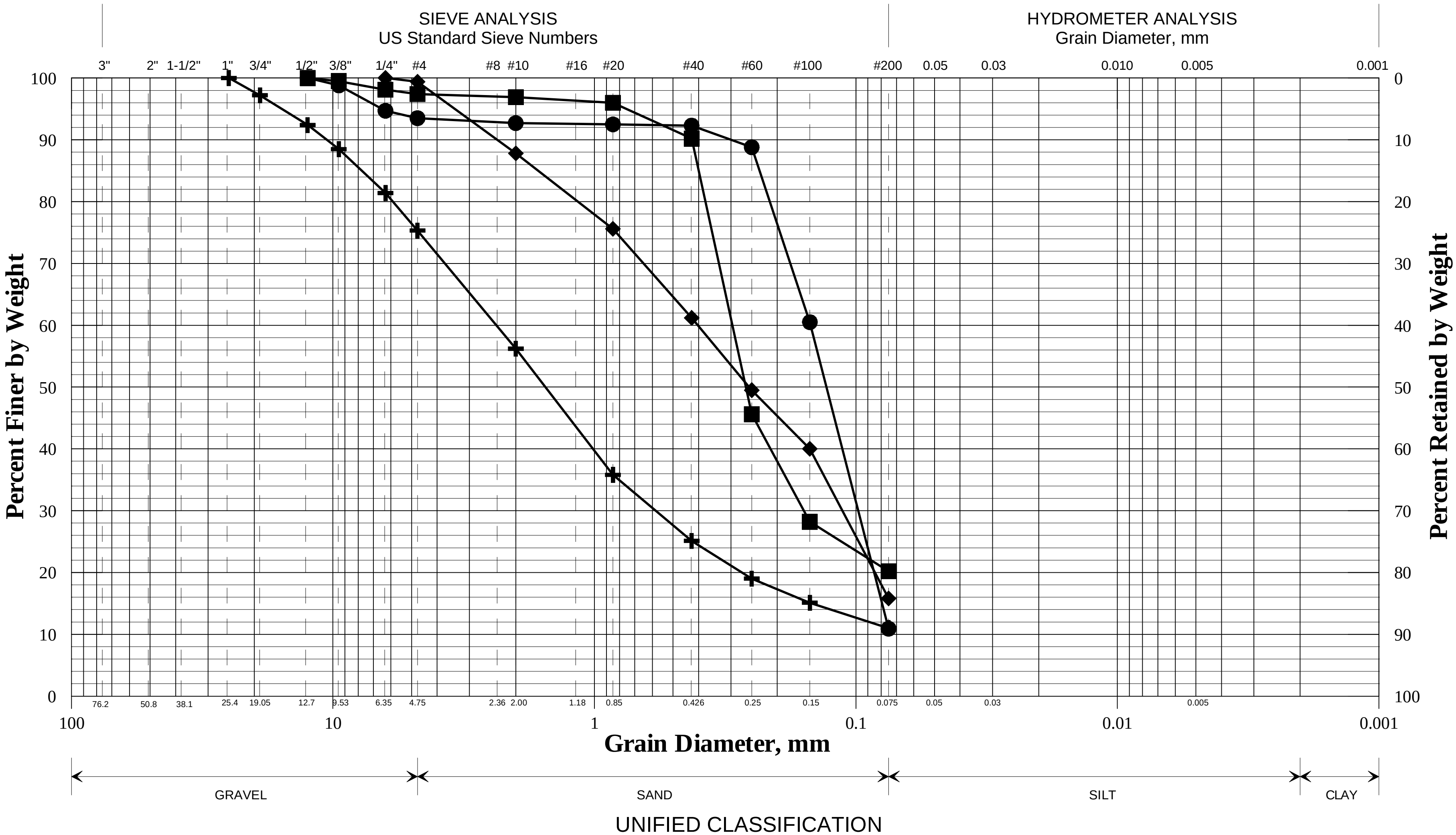
WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

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

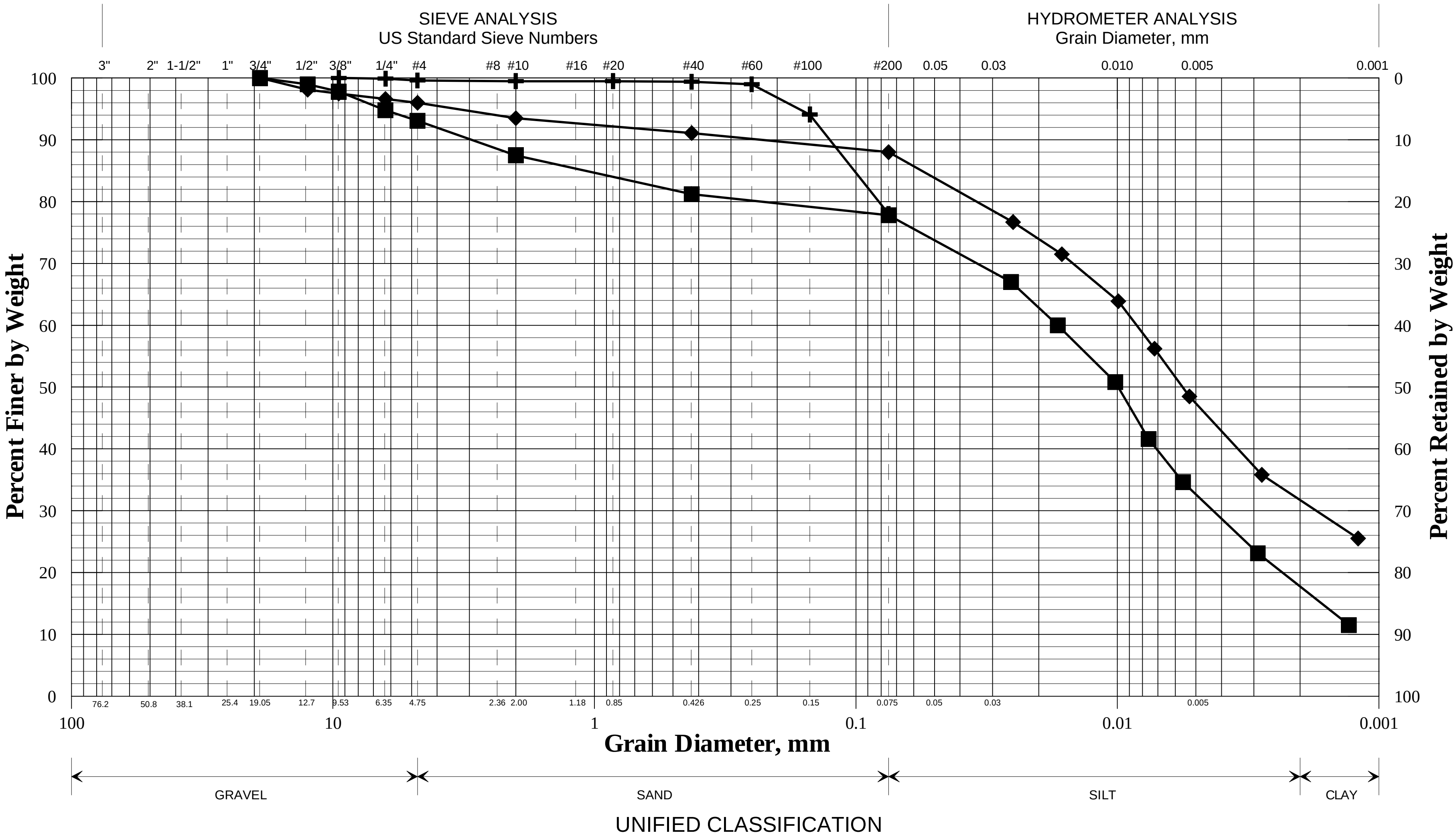


	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-RHC-101/2D	7+55.6	9.8 LT	5.0-7.0	SAND, some gravel, little silt.	4.1			
◆	BB-RHC-101/5D	7+55.6	9.8 LT	20.0-22.0	SAND, little silt, trace gravel. (PEAT)	197.9			
■	BB-RHC-101/6D	7+55.6	9.8 LT	25.0-27.0	SAND, little silt, trace gravel.	21.4			
●	BB-RHC-101/7D	7+55.6	9.8 LT	30.0-32.0	SAND, little silt, trace gravel.	24.2			
▲									
✖									

WIN
018955.00
Town
Rangeley
Reported by/Date
WHITE, TERRY A 1/18/2017

SHEET 1

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

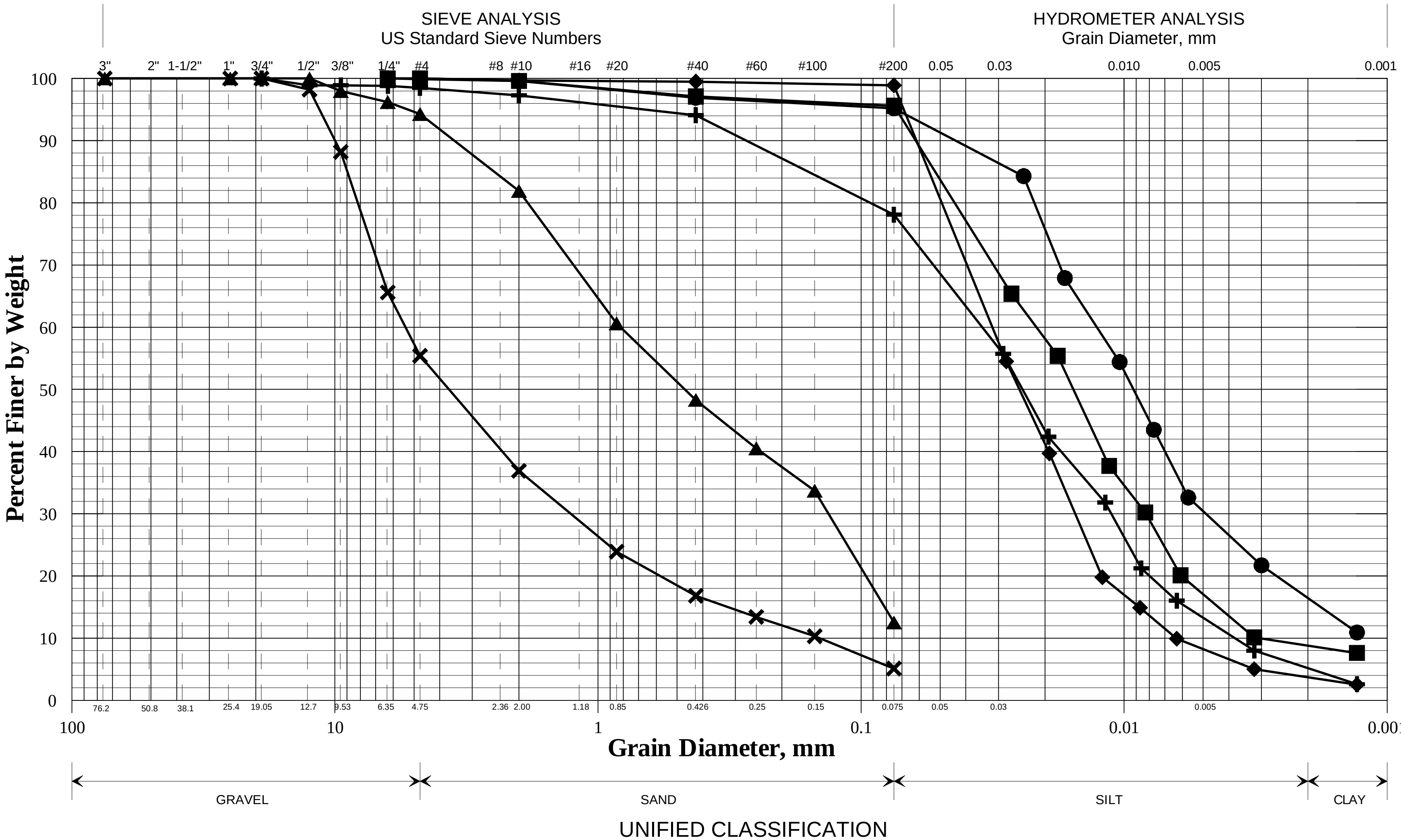


	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-RHC-101/8D	7+55.6	9.8 LT	35.0-37.0	SILT, some sand, trace gravel.	23.7			
◆	BB-RHC-101/9D	7+55.6	9.8 LT	40.0-42.0	SILT, some clay, trace sand, trace gravel.	24.4			NP
■	BB-RHC-101/12D	7+55.6	9.8 LT	55.0-56.5	SILT, little clay, little sand, trace gravel.	12.6			
●									
▲									
✖									

WIN
018955.00
Town
Rangeley
Reported by/Date
WHITE, TERRY A 1/18/2017

SHEET 2

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-RHC-102/4D	8+44	8.7 LT	20.0-22.0	SILT, little sand, trace clay, trace gravel.	23.8			NP
◆	BB-RHC-102/5D	8+44	8.7 LT	25.0-27.0	SILT, trace clay, trace sand.	20.3			
■	BB-RHC-102/6D	8+44	8.7 LT	30.0-32.0	SILT, trace clay, trace sand.	23.6	38	21	17
●	BB-RHC-102/9D	8+44	8.7 LT	44.0-46.0	SILT, little clay, trace sand, trace gravel.	21.6	26	22	4
▲	BB-RHC-201/1D	7+53.8	4.9 RT	19.0-20.5	SAND, little silt, trace gravel.	173.9			
✖	BB-RHC-201/2D	7+53.8	4.9 RT	20.5-22.0	Gravelly SAND, trace silt.	275.1			

WIN	
018955.00	
Town	
Rangeley	
Reported by/Date	
WHITE, TERRY A	10/10/2017

SHEET 3

MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
270708	BB-RHC-101/9D	GEOTECHNICAL (DISTURBED)	5/2/2016	5/20/2016

Sample Type: **GEOTECHNICAL** Location: Station: **7+55.6** Offset, ft: **9.8** **LT** Dbfg, ft: **40.0-42.0**

WIN/Town **018955.00 - RANGELEY** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	100.0
½ in. [12.5 mm]	98.1
⅜ in. [9.5 mm]	97.5
¼ in. [6.3 mm]	96.6
No. 4 [4.75 mm]	96.0
No. 10 [2.00 mm]	93.5
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	91.1
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	88.0
[0.0250 mm]	76.7
[0.0163 mm]	71.5
[0.0099 mm]	63.9
[0.0072 mm]	56.2
[0.0053 mm]	48.5
[0.0028 mm]	35.8
[0.0012 mm]	25.5

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	NP
Specific Gravity, Corrected to 20°C (T 100)	2.79
Loss on Ignition, % (T 267)	
Water Content (T 265), %	24.4

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **6/14/2016**

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MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
270710	BB-RHC-102/4D	GEOTECHNICAL (DISTURBED)	4/27/2016	5/20/2016

Sample Type: **GEOTECHNICAL** Location: Station: **8+44** Offset, ft: **8.7 LT** Dbfg, ft: **20.0-22.0**

WIN/Town **018955.00 - RANGELEY** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	100.0
½ in. [12.5 mm]	98.9
⅜ in. [9.5 mm]	98.9
¼ in. [6.3 mm]	98.8
No. 4 [4.75 mm]	98.5
No. 10 [2.00 mm]	97.3
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	94.1
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	78.1
[0.0288 mm]	55.7
[0.0194 mm]	42.4
[0.0118 mm]	31.8
[0.0086 mm]	21.2
[0.0063 mm]	16.0
[0.0032 mm]	8.0
[0.0013 mm]	2.6

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	NP
Specific Gravity, Corrected to 20°C (T 100)	2.77
Loss on Ignition, % (T 267)	
Water Content (T 265), %	23.8

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **6/14/2016**

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MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
270712	BB-RHC-102/6D	GEOTECHNICAL (DISTURBED)	4/27/2016	5/20/2016

Sample Type: **GEOTECHNICAL** Location: Station: **8+44** Offset, ft: **8.7 LT** Dbfg, ft: **30.0-32.0**

WIN/Town **018955.00 - RANGELEY** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	100.0
No. 4 [4.75 mm]	100.0
No. 10 [2.00 mm]	99.6
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	97.1
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	95.6
[0.0268 mm]	65.4
[0.0179 mm]	55.4
[0.0114 mm]	37.7
[0.0083 mm]	30.2
[0.0061 mm]	20.1
[0.0032 mm]	10.1
[0.0013 mm]	7.6
[mm]	

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	38
Plastic Limit (T 90), %	21
Plasticity Index (T 90), %	17
Specific Gravity, Corrected to 20°C (T 100)	2.76
Loss on Ignition, % (T 267)	
Water Content (T 265), %	23.6

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

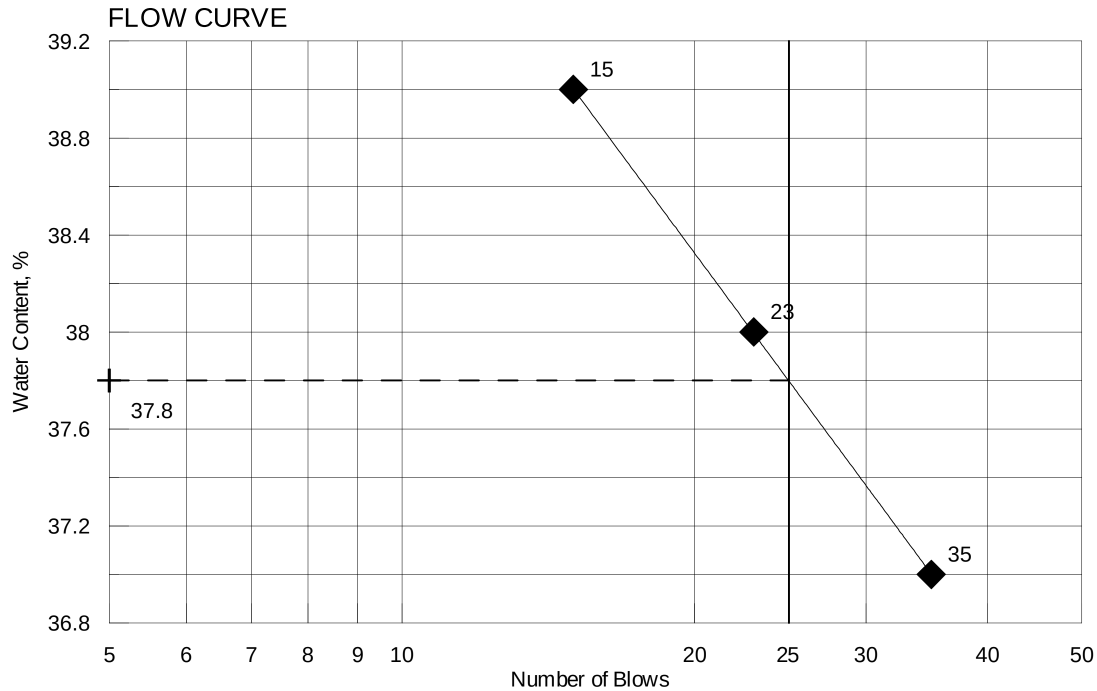
Comments:

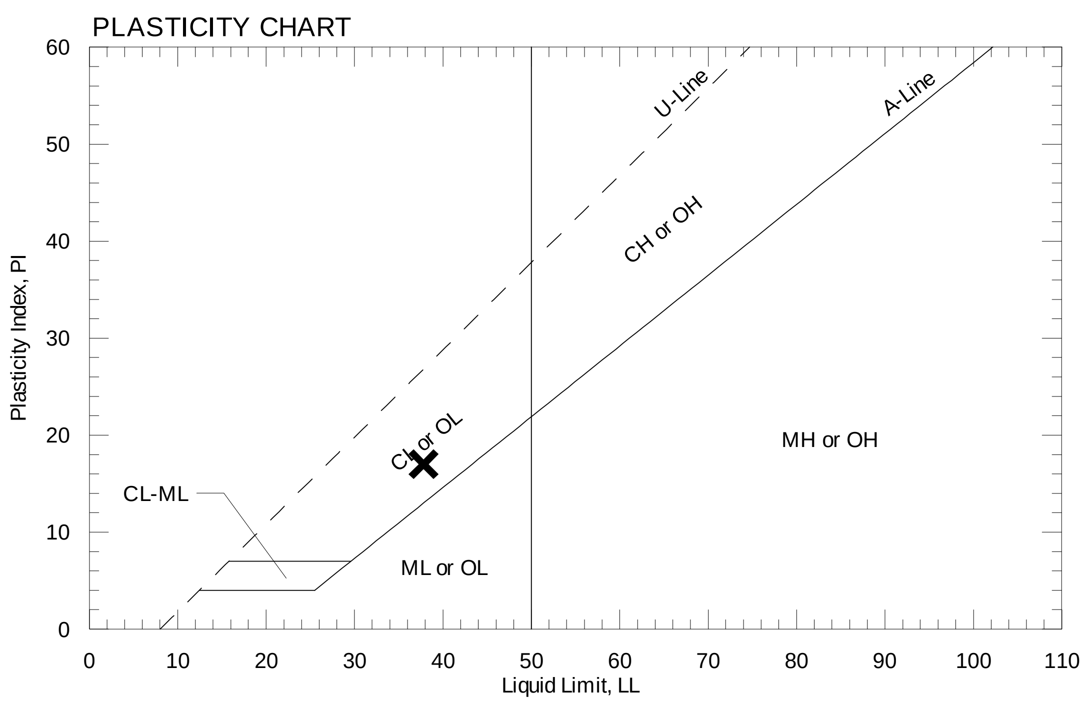
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **6/14/2016**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Rangeley	Reference No.	270712
WIN	018955.00	Water Content, %	23.6
Sampled	4/27/2016	Liquid Limit @ 25 blows (T 89), %	38
Boring No./Sample No.	BB-RHC-102/6D	Plastic Limit (T 90), %	21
Station	8+44	Plasticity Index (T 90), %	17
Depth	30.0-32.0	Tested By	GLIDS





MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
270713	BB-RHC-102/9D	GEOTECHNICAL (DISTURBED)	4/27/2016	5/20/2016

Sample Type: **GEOTECHNICAL** Location: Station: **8+44** Offset, ft: **8.7 LT** Dbfg, ft: **44.0-46.0**

WIN/Town **018955.00 - RANGELEY** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	100.0
No. 4 [4.75 mm]	99.9
No. 10 [2.00 mm]	99.6
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	96.9
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	95.2
[0.0241 mm]	84.3
[0.0168 mm]	67.9
[0.0104 mm]	54.4
[0.0077 mm]	43.5
[0.0057 mm]	32.6
[0.0030 mm]	21.7
[0.0013 mm]	10.9

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	26
Plastic Limit (T 90), %	22
Plasticity Index (T 90), %	4
Specific Gravity, Corrected to 20°C (T 100)	2.79
Loss on Ignition, % (T 267)	
Water Content (T 265), %	21.6

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths

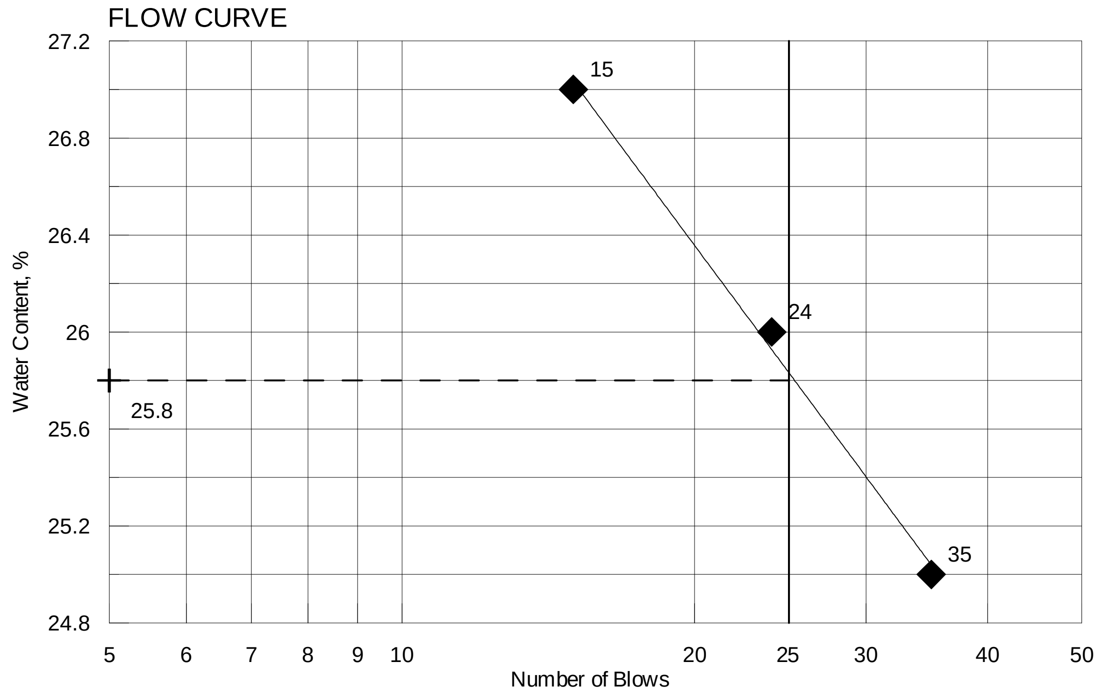
Comments:

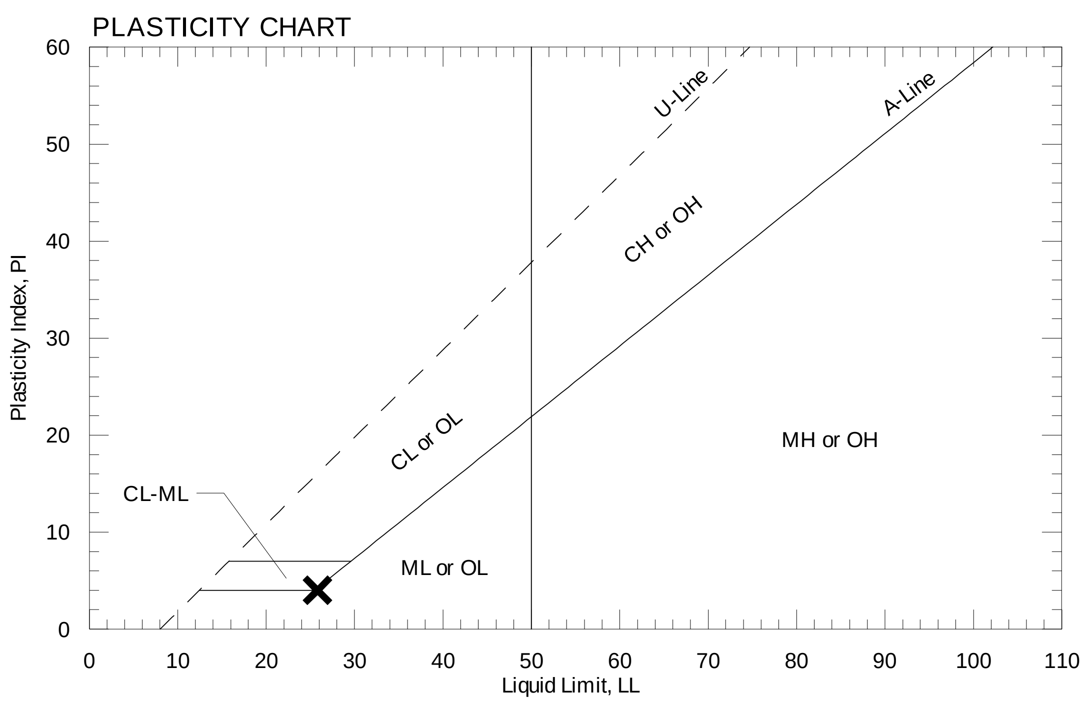
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **6/14/2016**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Rangeley	Reference No.	270713
WIN	018955.00	Water Content, %	21.6
Sampled	4/27/2016	Liquid Limit @ 25 blows (T 89), %	26
Boring No./Sample No.	BB-RHC-102/9D	Plastic Limit (T 90), %	22
Station	8+44	Plasticity Index (T 90), %	4
Depth	44.0-46.0	Tested By	BBURR





Appendix C

Calculations

H Pile Resistance

Design of H-piles

Reference: AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014 with Interim Revisions through 2016

Bedrock Properties

BB-RHC-101, R1 RQD = 22%, RQD = 35%
BB-RHC-102, R4 RQD= 0%, R5 RQD = 23%
Rock Type: Interlaminated metasandstone and metaclaystone.

For Design Purposes, use bedrock data from Kennebunk Bridge in Kennebunk, Maine. Unconfined Compressive Strength test results from a sample of sandstone or phyllite with low angle foliation from BB-KMR-102 with a RQD of 27%.
Design UCT = 16,800 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

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.

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

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

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

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

A. Structural Resistance of upper "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 (Assume plastic hinge develops, K selected per Vtrans Design Procedure)

l = "unbraced" length $l_{unbraced\_top} := 4.0 \cdot ft$ Assumed approximate unbraced top segment

LRFD eq. 6.9.4.1.2-1

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

$$P_e = \begin{pmatrix} 3571 \\ 5236 \\ 7342 \\ 9162 \\ 12489 \end{pmatrix} \cdot kip$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 4.608 \\ 4.804 \\ 6.862 \\ 7.02 \\ 7.261 \end{pmatrix}$$

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:

$$\boxed{P_n = \begin{pmatrix} 708 \\ 999 \\ 1007 \\ 1229 \\ 1624 \end{pmatrix} \cdot kip}$$

this applies to all pile sizes

Factored Axial Structural Resistance at the Strength Limit State

Resistance factor for upper unbraced segments 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

$$\boxed{P_r = \begin{pmatrix} 495 \\ 699 \\ 705 \\ 861 \\ 1137 \end{pmatrix} \cdot kip}$$

B. Structural Resistance of lower "unbraced" segment of pile

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

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

K = effective length factor $K_{eff} := 1.0$ Per Vtrans Integral Abutment Design Guideline, use K=1 for segment pinned at the top and bottom as shown by LPile analysis for 2nd unbraced segment. (see also LRFD Table C4.6.2.5-1)

l = "unbraced" length $l_{unbraced\_mid} := 11.0 \cdot ft$ Assumed middle zone or 2nd unbraced top segment length

LRFD eq. 6.9.4.1.2-1

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

$$P_e = \begin{pmatrix} 2083 \\ 3053 \\ 4282 \\ 5342 \\ 7283 \end{pmatrix} \cdot kip$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 2.687 \\ 2.801 \\ 4.002 \\ 4.094 \\ 4.234 \end{pmatrix}$$

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:

this applies to all pile sizes

$$\boxed{P_n = \begin{pmatrix} 663 \\ 939 \\ 964 \\ 1178 \\ 1558 \end{pmatrix} \cdot kip}$$

Factored Axial Structural Resistance at the Strength Limit State

Resistance factor for middle segment of H-pile in combined compression and flexure: $\phi_{cu} := 0.7$

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

$$P_r = \begin{pmatrix} 464 \\ 657 \\ 675 \\ 825 \\ 1091 \end{pmatrix} \cdot kip$$

C. Structural Resistance of lower "braced" 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 $K_{eff} := 0.65$ LRFD Table C4.6.2.5-1. Use K=0.65 for segment in pure compression. Fixed top and bottom

l = "braced" length $l_{unbraced\_bot} := .1 \cdot ft$ Assume only the very tip is in pure compression

LRFD eq. 6.9.4.1.2-1

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

$$P_e = \begin{pmatrix} 6 \times 10^7 \\ 9 \times 10^7 \\ 1 \times 10^8 \\ 2 \times 10^8 \\ 2 \times 10^8 \end{pmatrix} \cdot kip$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 7.696 \times 10^4 \\ 8.022 \times 10^4 \\ 1.146 \times 10^5 \\ 1.172 \times 10^5 \\ 1.213 \times 10^5 \end{pmatrix}$$

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:

this applies to all pile sizes

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

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

$$P_r = \begin{pmatrix} 387 \\ 545 \\ 535 \\ 652 \\ 860 \end{pmatrix} \cdot kip$$

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:

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

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

$\phi_c := 0.5$

$P_r := \phi_c \cdot P_n$

$$P_r = \begin{pmatrix} 387 \\ 545 \\ 535 \\ 652 \\ 860 \end{pmatrix} \cdot kip$$

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

$$\phi_c := 1.0$$

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

$$P_{r\_ee} = \begin{pmatrix} 775 \\ 1090 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot kip$$

12x53
12x74
14x73
14x89
14x117

Nominal and Factored Axial Geotechnical Resistance of HP piles

Geotechncial axial pile resistance for pile end bearing on rock 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

$$q_{u\_1} := 16800 \cdot psi$$

Geotechnical tip resistance.

$$q_{p\_2} := 2.5 \cdot q_{u\_1} \qquad \boxed{q_{p\_2} = 6048 \cdot ksf}$$

Nominal geotechnical tip resistance, R_p

$$R_p := \overrightarrow{\left(q_{p\_2} \cdot A_s\right)} \qquad R_p = \begin{pmatrix} 651 \\ 916 \\ 899 \\ 1096 \\ 1445 \end{pmatrix} \cdot kip$$

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)

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

$$R_{r\_p2} = \begin{pmatrix} 293 \\ 412 \\ 404 \\ 493 \\ 650 \end{pmatrix} \cdot kip$$

Factored Axial Geotechnical Compressive Resistance - ***Service Limit States***

Resistance factor, end bearing on rock Canadian Geotechnical Society method

$\phi := 1.0$

Factored Geotechnical Tip Resistance (R_r)

$R_{r\_p2} := \phi \cdot R_p$

$$R_{r\_p2} = \begin{pmatrix} 651 \\ 916 \\ 899 \\ 1096 \\ 1445 \end{pmatrix} \cdot kip$$

Factored Axial Geotechnical Compressive Resistance - ***Extreme Limit States***

Resistance factor, end bearing on rock Canadian Geotechnical Society method

$\phi_{ee} := 1.0$

Factored Geotechnical Tip Resistance (R_r)

$R_{r\_p2} := \phi_{ee} \cdot R_p$

$$R_{r\_p2} = \begin{pmatrix} 651 \\ 916 \\ 899 \\ 1096 \\ 1445 \end{pmatrix} \cdot kip$$

Drivability Analyses

Ref: LRFD Article 10.7.8

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

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

$\sigma_{dr} := 0.90 \cdot f_y \cdot \phi_{da}$

$\sigma_{dr} = 45 \cdot ksi$ 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 3 of this Report, estimated pile lengths will be approx. 63.5 ft. Assume contractor drives pile lengths of 70 ft (extra length accommodates for attachment of dynamic testing equipment, embedment into abutment, variation in bedrock surface).

Use proportional shaft resistances so that GRLWeap will assign approx. 20% of the ultimate capacities as skin friction.

Pile Size is 12 x 53

The 12x53 pile can be driven to the resistances below with a D 19-42 hammer at fuel setting -1 (90% of max) and 1.9 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

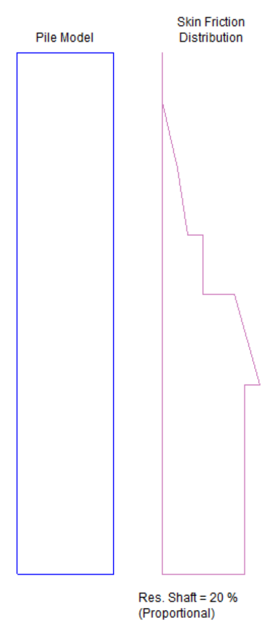
Maine DOT
18955 Rangeley 12x53 H

16-Oct-2017
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	40.95	4.67	8.2	8.40	19.58
420.0	41.76	4.95	9.4	8.50	19.88
440.0	42.45	5.11	10.9	8.59	20.06
460.0	43.12	5.26	12.8	8.67	20.26
470.0	43.42	5.27	13.9	8.71	20.31
479.0	43.66	5.31	15.0	8.74	20.38
480.0	43.66	5.34	15.1	8.74	20.43
500.0	44.19	5.37	18.2	8.80	20.55
520.0	44.62	5.44	22.3	8.87	20.73
540.0	45.00	5.43	28.0	8.91	20.81

DELMAG D 19-42

Ram Weight	4.00	kips
Efficiency	0.800	
Pressure	1440 (90%)	psi
Helmet Weight	1.90	kips
Hammer Cushion	60155	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	70.00	ft
Pile Penetration	63.50	ft
Pile Top Area	15.50	in2



Limit blow counts to 15 bpi

$$R_{ndr} := 479 \cdot kip$$

Strength Limit State

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

$$\boxed{R_{fdr} = 311 \cdot kip}$$

Extreme and Service Limit States

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

$$\boxed{R_{dr} = 479 \cdot kip}$$

Pile Size is 12 x 74

The 12x74 pile can be driven to the resistances below with a D 19-42 hammer at fuel setting -1 (90% of max) and 1.9 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

Maine DOT
18955 Rangeley 12x74 H

16-Oct-2017
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
500.0	37.81	2.90	11.0	8.49	18.38
510.0	38.09	2.93	11.7	8.52	18.42
520.0	38.38	3.00	12.4	8.54	18.50
530.0	38.63	3.06	13.1	8.57	18.56
540.0	38.94	3.11	14.0	8.60	18.57
550.0	39.19	3.21	14.8	8.63	18.69
552.0	39.26	3.20	15.0	8.64	18.67
555.0	39.33	3.25	15.2	8.65	18.72
560.0	39.46	3.23	15.7	8.65	18.71
570.0	39.60	3.27	16.5	8.67	18.74

DELMAG D 19-42

Ram Weight	4.00	kips
Efficiency	0.800	
Pressure	1440 (90%)	psi
Helmet Weight	1.90	kips
Hammer Cushion	60155	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	70.00	ft
Pile Penetration	63.50	ft
Pile Top Area	21.80	in2

Limit blow counts to
15 bpi

$$R_{ndr} := 552 \cdot kip$$

Strength Limit State

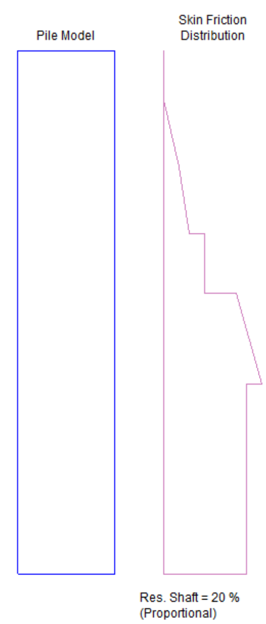
$$R_{fdr} := R_{ndr} \cdot \phi_{dyn}$$

$$\boxed{R_{fdr} = 359 \cdot kip}$$

Extreme and
Service Limit States

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

$$\boxed{R_{dr} = 552 \cdot kip}$$



Pile Size is 14 x 73

The 14x73 pile can be driven to the resistances below with a D 19-42 at max fuel setting hammer and 1.9 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

Maine DOT
18955 Rangeley 14x73 H

16-Oct-2017
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
540.0	41.87	3.39	11.0	9.56	21.69
560.0	42.41	3.52	12.2	9.62	21.86
580.0	42.89	3.65	13.7	9.67	21.98
585.0	43.24	3.65	13.8	9.78	22.24
590.0	43.41	3.66	14.2	9.80	22.26
599.0	43.55	3.73	15.0	9.81	22.31
600.0	43.62	3.72	15.1	9.82	22.31
620.0	44.05	3.83	16.7	9.86	22.45
640.0	44.16	3.80	18.9	9.80	22.22
660.0	44.45	3.92	21.0	9.83	22.31

DELMAG D 19-42

Ram Weight	4.00	kips
Efficiency	0.800	
Pressure	1600 (100%)	psi
Helmet Weight	1.90	kips
Hammer Cushion	60155	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	70.00	ft
Pile Penetration	63.50	ft
Pile Top Area	21.40	in2

Limit blow counts to 15 bpi

$$R_{ndr} := 599 \cdot kip$$

Strength Limit State

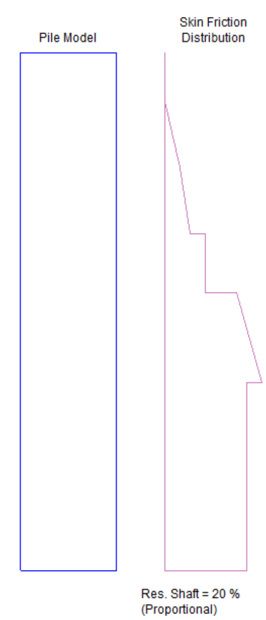
$$R_{fdr} := R_{ndr} \cdot \phi_{dyn}$$

$$\boxed{R_{fdr} = 389 \cdot kip}$$

Extreme and Service Limit States

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

$$\boxed{R_{dr} = 599 \cdot kip}$$



Pile Size is 14 x 89

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

Maine DOT
18955 Rangeley 14x89 H

16-Oct-2017
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
610.0	36.86	5.20	12.3	9.23	21.19
620.0	37.08	5.23	13.0	9.25	21.17
630.0	37.30	5.31	13.7	9.27	21.20
640.0	37.50	5.42	14.4	9.29	21.25
645.0	37.62	5.49	14.7	9.31	21.32
649.0	37.69	5.54	15.0	9.31	21.34
650.0	37.69	5.57	15.1	9.32	21.34
660.0	37.90	5.67	15.9	9.34	21.40
670.0	38.07	5.72	16.8	9.36	21.42
680.0	38.27	5.78	17.7	9.38	21.47

DELMAG D 19-42

Ram Weight	4.00	kips
Efficiency	0.800	
Pressure	1600 (100%)	psi
Helmet Weight	2.70	kips
Hammer Cushion	109975	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	70.00	ft
Pile Penetration	63.50	ft
Pile Top Area	26.10	in2

Limiting driving stress to 45 ksi

$R_{ndr} := 649 \cdot kip$

Strength Limit State

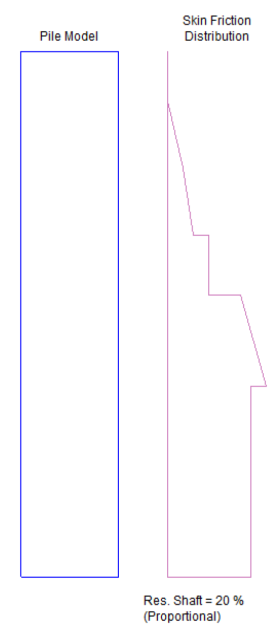
$R_{fdr} := R_{ndr} \cdot \phi_{dyn}$

$\boxed{R_{fdr} = 422 \cdot kip}$

Extreme and Service Limit States

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

$\boxed{R_{dr} = 649 \cdot kip}$



Pile Size is 14 x 117

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

Maine DOT
18955 Rangeley 14x117 H D19-42

16-Oct-2017
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
690.0	33.41	2.75	13.0	9.18	19.78
700.0	33.63	2.79	13.3	9.21	19.86
710.0	33.88	2.83	13.8	9.23	19.91
720.0	34.04	2.86	14.2	9.25	19.96
730.0	34.28	2.89	14.7	9.28	20.07
737.0	34.40	2.91	15.0	9.29	20.11
740.0	34.45	2.92	15.1	9.30	20.12
750.0	34.63	2.94	15.6	9.32	20.17
760.0	34.80	2.98	16.1	9.34	20.22
770.0	35.01	3.03	16.7	9.36	20.23

Limit blows to 15 bpi

DELMAG D 19-42

Ram Weight	4.00 kips
Efficiency	0.800
Pressure	1600 (100%) psi
Helmet Weight	2.70 kips
Hammer Cushion	109975 kips/in
COR of H.C.	0.800
Skin Quake	0.100 in
Toe Quake	0.040 in
Skin Damping	0.050 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	70.00 ft
Pile Penetration	63.50 ft
Pile Top Area	34.40 in2

$R_{ndr} := 737 \cdot kip$

Strength Limit State

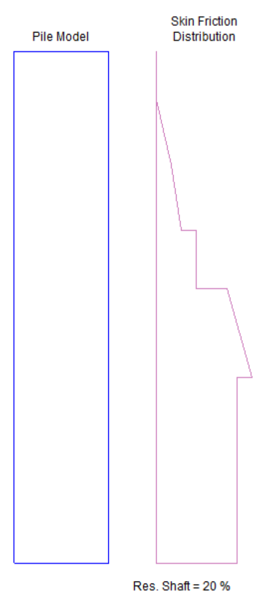
$R_{fdr} := R_{ndr} \cdot \phi_{dyn}$

$\boxed{R_{fdr} = 479 \cdot kip}$

Extreme and Service Limit States

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

$\boxed{R_{dr} = 737 \cdot kip}$



Pile Size is 14 x 117

The 14x117 pile can be driven to the resistances below with a D 36-32 hammer at 73% of max (-2) fuel setting and 2.7 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

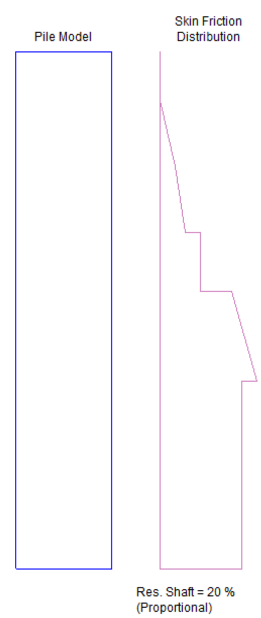
Maine DOT
18955 Rangeley 14x117 H D36-32

16-Oct-2017
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
840.0	40.79	5.02	10.1	7.94	35.63
860.0	41.21	5.12	10.8	7.98	35.88
880.0	41.59	5.19	11.7	8.02	36.01
900.0	41.91	5.20	12.5	8.06	36.16
920.0	42.22	5.23	13.3	8.09	36.34
940.0	42.62	5.24	14.3	8.13	36.50
954.0	42.83	5.23	15.0	8.15	36.58
960.0	42.90	5.24	15.3	8.16	36.66
980.0	43.20	5.24	16.6	8.19	36.74
1000.0	43.48	5.25	17.8	8.22	36.91

DELMAG D 36-32

Ram Weight	7.93 kips
Efficiency	0.800
Pressure	1215 (81%) psi
Helmet Weight	2.70 kips
Hammer Cushion	109975 kips/in
COR of H.C.	0.800
Skin Quake	0.100 in
Toe Quake	0.040 in
Skin Damping	0.050 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	70.00 ft
Pile Penetration	63.50 ft
Pile Top Area	34.40 in2



Limit blows to 15 bpi

$R_{ndr} := 954 \cdot kip$

Strength Limit State

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

$\boxed{R_{fdr} = 620 \cdot kip}$

Extreme and Service Limit States

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

$\boxed{R_{dr} = 954 \cdot kip}$

LABORATORY TESTING DATA SHEET

Project Name Kennebunk Bridge
Project No. 09.0025597.00
Project Manager J. Tooley

Location Kennebunk, ME
Assigned By J. Tooley
Report Date 1/15/2009

Reviewed By [signature]
Date Reviewed 1/15/09

Boring No.	Sample No.	Depth Ft.	Lab No.	Sample Data: Water Content %	Do in.	L in.	(1) Unit Wt. PCF	(2) Wet Density PCF	Bulk Gs.	Compression Tests: (3) Other Tests	(4) Strength KSI	(5) Strain %	(6) Conf. Stress	(7) E sec PSI EE+06	(8) Poisson's Ratio	σт KSI	Is_{50} KSI	Rock Formation or Description or Remarks
BB-KMR-103	R1	25.5-25.9	5		1.966	4.471	172.3			U	17.95	0.32		4.96				Gray fine grained Siltstone
BB-KMR-102	R1	29.9-30.3	6		1.966	4.591	181.6			U	16.80	0.34		5.44				Gray f-m grained Sandstone

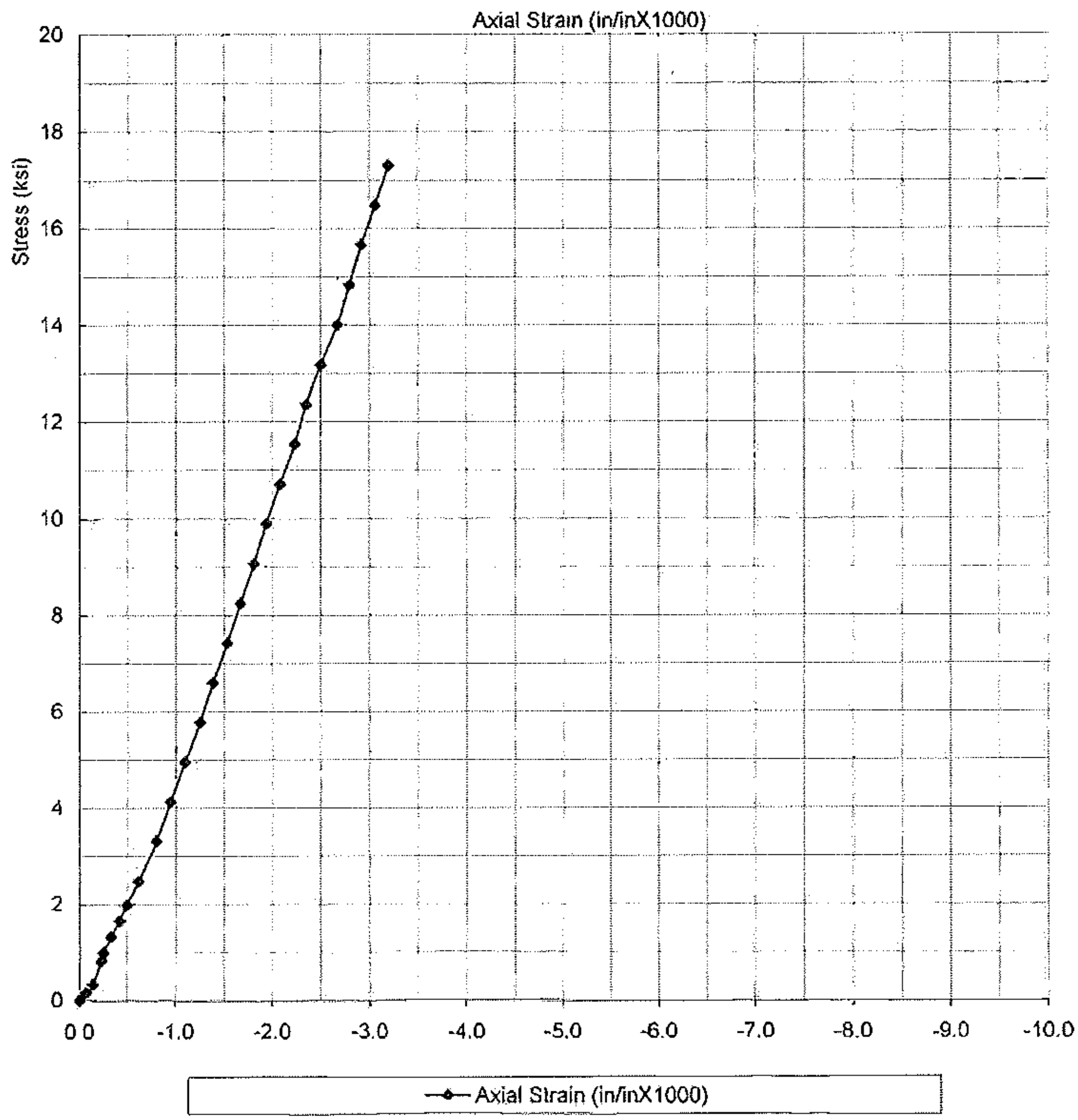
(1) Volume Determined By Measuring Dimensions
(2) Determined by Measuring Dimensions and Weight of Saturated Sample

(3) P=Petrographic PLD=Point Load (diametrical), PLA= Point Load (Axial) RST= Splitting Tensile U= Unconfined Compressive Strength
(4) Taken at Peak Deviator Stress

(5) Strain at Peak Deviator Stress
(6) Represents Confining Stress on Triaxial Tests
(7) Represents Secant Modulus at 50% of Total Failure Stress
(8) Represents Secant Poisson's Ratio at 50% of Total Failure Stress

GZA GeoEnvironmental, Inc.

Kennebunk Bridge
Kennebunk, ME



Rock Testing

Boring No.	BB-KMR-103	File No.	09.0025597.00
Sample No.	R1	Date:	1/14/2009
Depth:	25.5-25.9'	Test No.	U 5

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Kennebunk Bridge Replacement
Location: Kennebunk, ME

Boring No.: BB-KMR-102
PIN: 15098.00

Driller:	New Hampshire Boring	Elevation (ft.)	22.2	Auger ID/OD:	NA
Operator:	Greg/Gerry Michael	Datum:	NAVD 88	Sampler:	Standard Split
Logged By:	Jennifer Tooley	Rig Type:	Truck	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	01/05/09-01/05/09	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ
Boring Location:	St. 15+78, 20.8 L	Casing ID/OD:	4"/4.5"	Water Level*:	
Hammer Efficiency Factor: 0.45		Hammer Type:	Automatic ☐ Hydraulic ☒	Rope & Cathead ☐	

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							RC			Probable boulder.	
	R1	54/54	2.5 - 7.0	RQD = 37%			NQ	19.7		2.5 Hard, fresh, fine to medium grained, dark gray PHYLLITE. Joints are closely spaced, primarily low angle with occasional vertical fractures, planar, smooth to rough, fresh to slightly discolored, and partially open to moderately open. Some silt in filling. Highly fractured zone from 3.25 to 3.75 feet. See Remark 2. R1: Core Times (min) 2.5-3.5 (5) 3.5-4.5 (5) 4.5-5.5 (5) ← U6 16,800 PSI 5.5-6.5 (5) 6.5-7.0 (10)	
5								U6			
	R2	30/30	7.0 - 9.5	RQD = 27%				12.7		Hard, fresh to slightly weathered, fine to medium grained, dark gray PHYLLITE. Joints are very closely spaced, primarily low angle with occasional vertical fractures, planar, smooth to rough, fresh to slightly discolored and partially open. Some Silt in filling. Highly fractured zone from approximately 8.0-9.0 feet. See Remark 2. R2: Core Times (min) 7.0-7.5 (2.5) 7.5-8.5 (5) 8.5-9.5 (5) 9.5	
10										**Bottom of Exploration at 9.50 feet below ground surface.**	
15											
20											
25											

Remarks:

1. Rock at 25 feet from bridge deck; advanced casing 2.0 feet into bedrock; roller cone to 2.5 feet (probable boulder from 0 to 2 feet.)
2. Highly fractured section likely the result of rock coring; the driller had difficulty with rock core and likely caused rock to become fractured.

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.

Parameters for Lateral Pile Resistance

Development of soil model for LPile

OBJECTIVE
Estimate soil parameters for lateral pile analyses

Given:
1) Boring logs and lab data

Assumptions:
1) Groundwater was observed 9.0 ft bgs during the investigation. Assume the groundwater table is at Elevation 1516.5.
2) MaineDOT Bridge Design Guide (BDG) Soil Type 4 will be used for integral abutment backfill.
3) Piles shall be driven to, or within, bedrock.

Abutment Soil Model

1) The design soil layers are delinated as depicted on the attached annotated boring logs, which indicates the top and bottom elevations of the soil layers based on differing engineering properties.

Soil Layer No. 1 (Granular Borrow for Underwater Backfill) El. 1526 - 1516.5

Internal Angle of Friction $\phi_1 := 32\,deg$

Soil Total Unit Weight $\gamma_{1moist} := 125pcf$

MaineDOT BDG Table 3-3

Representative constant giving the variation of soil modulus with depth, k:
Medium dense sand above water table for static loading = 90 pci

Technical Manual
LPile 2016
p. 96

Soil Layer No. 2 (Submerged, Medium Dense, Sand) El. 1516.5 - 1507.5

Internal Angle of Friction

Design N_{60} = 18 bpf $\phi_2 := 32$ Lambe and Whitman, N vs. Phi

Dry Unit Weight

Loose uniform sand = 92 pcf
Dense angular silty sand = 121 pcf
Average Loose and Dense for Medium Dense:
Medium Dense sand: 107 pcf

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 - dry unit weight

$\gamma_{2dry} := 107pcf$

Saturated Unit Weight

Natural water content at saturated state:
Loose uniform sand = 30%
Dense angular silty sand = 15%
Average Loose and Dense for Medium Dense:
Medium Dense angular silty sand: 23%

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 - Natural Moisture Content in a saturated state

$w_{2sat} := .23$

$\gamma_{2saturated} := \gamma_{2dry} \cdot (1 + w_{2sat})$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships

$\gamma_{2saturated} = 132 \cdot pcf$

Effective Unit Weight

weight of water = γ_w =62.4 pcf

$\gamma_w := 62.4\text{pcf}$

$\gamma'_2 := \gamma_{2saturated} - \gamma_w$

$\gamma'_2 = 69 \cdot \text{pcf}$

Representative constant giving the variation of soil modulus with depth, k:
Medium dense sand below water table for static loading = 60 pci

Technical Manual
LPile 2016
p. 96

Soil Layer No. 3 (Submerged, medium stiff, silt) El. 1507.5 - 1491.5

Cohesion

Design N_{60}= 6 $c := 500\text{psf}$

MaineDOT key to Soil and Rock Description

Dry Unit Weight

Dry, Stiff clay = 108 pcf

Das, Principles of Geotechnical Eng. 7th Ed. p. 59:
Table 3.2 Dry, Stiff clay

$\gamma_{3dry} := 108\text{pcf}$

Saturated Unit Weight

Natural water content at saturated state: 24%

Lab test results of samples BB-RHC-102; 4D, 6D

$w_{3sat} := .24$

$\gamma_{3saturated} := \gamma_{3dry} \cdot (1 + w_{3sat})$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59:
Table 3.1 Unit Weight Relationships

$\gamma_{3saturated} = 134 \cdot \text{pcf}$

Effective Unit Weight

$\gamma'_3 := \gamma_{3saturated} - \gamma_w$

$\gamma'_3 = 72 \cdot \text{pcf}$

Representative constant giving the variation of soil modulus with depth, k:
medium stiff silt for static loading = 500 pci

The strain corresponding to a stress of 50 percent of the ultimate stress,
ε_{50}: medium stiff silt = .010

Technical Manual
LPile 2016
p. 77
p. 69

Soil Layer No. 4 (Submerged, Very Dense, Sand) El. 1491.5 -1487.5

Internal Angle of Friction

Design N_{60} Value = 62 $\phi_4 := 40$

Lambe and Whitman, N vs. Phi.
Limit phi to 40 for design

Dry Unit Weight

Dry, Dense Sand = 121 pcf

Das, Principles of Geotechnical Eng. 7th Ed. p. 59:
Table 3.2 - dry unit weight

$\gamma_{4dry} := 121\text{pcf}$

Saturated Unit Weight

Natural water content at saturated state:
Dense angular sand: 21%

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 - Natural Moisture Content in a saturated state

$w_{4sat} := .21$

$\gamma_{4saturated} := \gamma_{4dry} \cdot (1 + w_{4sat})$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships

$\gamma_{4saturated} = 146 \cdot pcf$

Effective Unit Weight

$\gamma'_4 := \gamma_{4saturated} - \gamma_w$

$\gamma'_4 = 84 \cdot pcf$

Representative constant giving the variation of soil modulus with depth, k:
Dense sand below water table for static loading = 125 pci

Technical Manual
LPile 2016
p. 96

Soil Layer No. 5 (Submerged, hard, silt) El. 1487.5 - 1477.5

Cohesion

Design N_{60} = 58 $c := 4000psf$

MaineDOT key to Soil and Rock Description

Dry Unit Weight

Dry, stiff clay = 108 pcf

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 - dry unit weight

$\gamma_{5dry} := 108pcf$

Saturated Unit Weight

Lab results of sample BB-RHC-102; 9D

Natural water content at saturated state:
Sample 9D: 22%

$w_{5sat} := .22$

$\gamma_{5saturated} := \gamma_{5dry} \cdot (1 + w_{5sat})$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships

$\gamma_{5saturated} = 132 \cdot pcf$

Effective Unit Weight

$\gamma'_5 := \gamma_{5saturated} - \gamma_w$

$\gamma'_5 = 69 \cdot pcf$

Representative constant giving the variation of soil modulus with depth, k:
Dense sand below water table for static loading = 2000 pci

Technical Manual
LPile 2016
p. 77

The strain corresponding to a stress of 50 percent of the ultimate stress as presented by Skempton, ε_{50}: Hard silt, .004

Soil Layer No. 6 (Submerged, very dense, Glacial Till) El. 1477.5 - 1453.2

Cohesion

Design N_{60} = -- $c := 4000psf$

MaineDOT key to Soil and Rock Description

Dry Unit Weight

Dry, Glacial Till = 134 pcf

$\gamma_{6dry} := 134pcf$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 - dry unit weight

Saturated Unit Weight

Natural water content at saturated state: 10%

$w_{6sat} := .10$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 - Natural Moisture Content in a saturated state

$\gamma_{6saturated} := \gamma_{6dry} \cdot (1 + w_{6sat})$

$\gamma_{6saturated} = 147 \cdot pcf$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships

Soil Effective Unit Weight

$\gamma'_6 := \gamma_{6saturated} - \gamma_w$

$\gamma'_6 = 85 \cdot pcf$

Representative constant giving the variation of soil modulus with depth, k:
Dense sand below water table for static loading = 2000 pci

Technical Manual
LPile 2016
p. 77

The strain corresponding to a stress of 50 percent of the ultimate stress as presented by Skempton, ε_{50}: Hard silt, .004

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Hunter Cove Bridge #2384 carries Mingo Loop Road over Hunter Cove

Location: Rangeley, Maine; Franklin County

Boring No.: BB-RHC-102

WIN: 18955.00

Driller:	MaineDOT	**Elevation (ft.)**	1525.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/27/2016-4/28/2016	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	8+44, 8.7 ft lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.908	**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 = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field 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 = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_V = Pocket Torvane 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	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	1525.15		3" HMA.	-9
										0.25 Brown, damp, Gravelly SAND, trace silt, (Fill).	
5	1D	24/14	5.00 - 7.00	4/5/8/11	13	20				Olive, moist, medium dense, Silty SAND, little gravel.	
											0
10	2D	24/3	10.00 - 12.00	5/9/3/3	12	18	42			Brown, wet, medium dense, Gravelly SAND, trace silt.	
							104				
							109				
							84				
							43				
15	3D	24/7	15.00 - 17.00	75/27/8/12	35	53	28			Similar to above, except very dense.	
							112				9
							48	1507.40		18.00	
							13				
20	4D	24/20	20.00 - 22.00	3/2/2/8	4	6	14			Grey, wet, medium stiff, SILT, little fine sand, trace clay, trace gravel, (Glacial Till).	G#270710 A-4, CL WC=23.8% Non-Plastic
							22				
							57				
							48	1501.90		23.50	
25											

Layer 1 - Granular Borrow

Layer 1 (El.1525.5 - El. 1516.5)

Layer 2 - Submerged, medium dense to very dense, sand

Layer 2 (El.1516.5 - El. 1507.5)

Layer 3 - Submerged, medium stiff to very stiff, silt

Layer 3 (El. 1507.5 - El. 1491.5)

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:** Hunter Cove Bridge #2384 carries Mingo Loop Road over Hunter Cove **Location:** Rangeley, Maine; Franklin County	**Boring No.:** BB-RHC-102 **WIN:** 18955.00

Driller:	MaineDOT	**Elevation (ft.)**	1525.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/27/2016-4/28/2016	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	8+44, 8.7 ft lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.908	**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 = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field 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 = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane 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	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/13	25.00 - 27.00	4/7/6/8	13	20	16			Olive, wet, very stiff, SILT, trace sand, trace clay, (Glacial Till).	G#270711 A-4, CL WC=20.3%
							31				
							42			Layer 3 - Submerged, medium stiff to very stiff, silt	
							39				
							28				
30	6D	24/18	30.00 - 32.00	3/4/2/8	6	9	34			Similar to above, except stiff.	G#270712 A-6, CL WC=23.6% LL=38 PL=21 PI=17
							46				
				Layer 3 (El. 1507.5 - El. 1491.5)							
							48				25
								1491.40		34.00	
	7D	24/13	34.00 - 36.00	16/22/19/21	41	62	60			Grey, wet, very dense, Gravelly SAND, some silt, (Glacial Till).	
35							71			Layer 4 - Submerged, very dense, sand	
				Layer 4 (El. 1491.5 - El. 1487.5)							
							70				29
								1487.40		38.00	
							145			Olive, wet, hard, fine Sandy SILT, some clay, (Glacial Till).	
	8D	24/22	39.00 - 41.00	18/20/18/21	38	58	87				
40							177				
							181			Layer 5 - Submerged, hard, silt	
							232				
							70				
	9D	24/20	44.00 - 46.00	12/19/19/22	38	58	129			Olive, wet, hard, SILT, little clay, trace fine sand, trace gravel.	G#270713 A-4, CL-ML WC=21.6% LL=26 PL=22 PI=4
45							123				
				Layer 5 (El. 1487.5 - El. 1477.5)							
							104				39
								1477.40		48.00	
	R1	43.2/24	48.00 - 51.60				120 NQ-2			R1: Grey, cemented Glacial Till, cobbles.	
50							116			Layer 6 - Submerged, very dense or hard, Glacial Till	

Layer 6 (El. 1477.5 - El. 1453.2)

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:	Hunter Cove Bridge #2384 carries Mingo Loop Road over Hunter Cove	**Location:**	Rangeley, Maine; Franklin County	**Boring No.:**	BB-RHC-102
				WIN:	18955.00
Driller:	MaineDOT	**Elevation (ft.)**	1525.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/27/2016-4/28/2016	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	8+44, 8.7 ft lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.908	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:

D = Split Spoon Sample	R = Rock Core Sample	S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)	T_v = Pocket Torvane Shear Strength (psf)
MD = Unsuccessful Split Spoon Sample Attempt	SSA = Solid Stem Auger	$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)	WC = Water Content, percent
U = Thin Wall Tube Sample	HSA = Hollow Stem Auger	q_p = Unconfined Compressive Strength (ksf)	LL = Liquid Limit
MU = Unsuccessful Thin Wall Tube Sample Attempt	RC = Roller Cone	N-uncorrected = Raw Field SPT N-value	PL = Plastic Limit
V = Field Vane Shear Test, PP = Pocket Penetrometer	WOH = Weight of 140lb. Hammer	Hammer Efficiency Factor = Rig Specific Annual Calibration Value	PI = Plasticity Index
MV = Unsuccessful Field Vane Shear Test Attempt	WOR/C = Weight of Rods or Casing	N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency	G = Grain Size Analysis
	WO1P = Weight of One Person	N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected	C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50							83				
							88				
							104				
							136				
							100				
55	R2	44.4/40	55.60 - 59.30				47 NQ-2			R2: Grey, cemented Glacial Till, cobbles.	
							58				
							58				
							79				
	R3	54/15	59.90 - 64.40				80 NQ-2				
60							66			R3: Grey, cemented Glacial Till, cobbles.	
							69				
							100				
							150			Roller Coned ahead to 70.0 ft bgs.	
							187			Cobble from 64.8-65.3 ft bgs.	
65							OPEN				
							HOLE			Layer 6 - Submerged, hard or very dense, glacial till	
				Layer 6 (El. 1477.5 - El. 1453.2)							
70	10D	3.6/3.6	70.00 - 70.30	50(3.6")	---					Grey brown, very dense, Silty SAND, (Glacial Till).	
	R4	55.2/44	72.20 - 76.80	RQD = 0%			NQ-2	1453.20		72.20 63.2 Top of Bedrock at Elev. 1453.2 ft. R4:Bedrock: Grey, fine to medium grained, interlaminated METASANDSTONE and METACLAYSTONE, slightly weathered, low angle breaks along foliation, occasional steep joints are open, very close to close, iron staining.	
75											

Remarks:

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

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

3.4 Construction Loads

The construction live load to be used for constructibility checks is 50 psf applied over the entire deck area. Consideration should be given to slab placement sequence for calculation of maximum force effects.

3.5 Railroad Loads

Railroad bridges should be designed according to the latest American Railroad Engineering and Maintenance-of-Way Association specifications (AREMA, 2002), with the Cooper live loading as determined by the railroad company.

3.6 Earth Loads

3.6.1 General

Earth pressures considered for wall and substructure design must use the appropriate soil weight shown in Table 3-3.

Table 3-3 Material Classification

Soil Type	Soil Description	Internal Angle of Friction of Soil, ϕ	Soil Total Unit Weight (pcf)	Coeff. of Friction, tan δ, Concrete to Soil	Interface Friction, Angle, Concrete to Soil δ
1	Very loose to loose silty sand and gravel Very loose to loose sand Very loose to medium density sandy silt Stiff to very stiff clay or clayey silt	29° *	100	0.35	19°
2	Medium density silty sand and gravel Medium density to dense sand Dense to very dense sandy silt	33°	120	0.40	22°
3	Dense to very dense silty sand and gravel Very dense sand	36°	130	0.45	24°
4	Granular underwater backfill Granular borrow	32°	125	0.45	24°
5	Gravel Borrow	36°	135	0.50	27°

* The value given for the internal angle of friction (ϕ) for stiff to very stiff silty clay or clayey silt should be used with caution due to the large possible variation with different moisture contents.

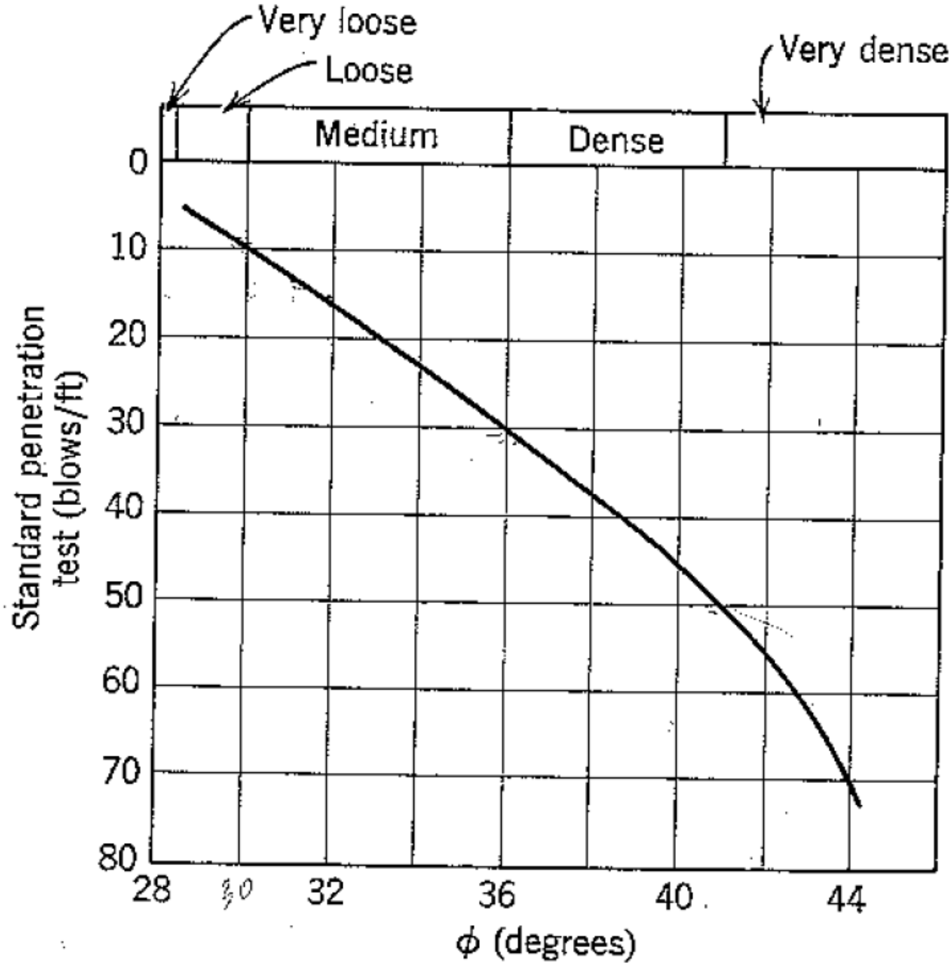


Fig. 11.14 Correlation between friction angle and penetration resistance (From Peck, Hanson, and Thornburn, 1953).

Das, Principles of Geotechnical Engineering 7th ed.

3.4 Various Unit-Weight Relationships

In Sections 3.2 and 3.3, we derived the fundamental relationships for the moist unit weight, dry unit weight, and saturated unit weight of soil. Several other forms of relationships that can be obtained for γ, γ_d, and γ_{sat} are given in Table 3.1. Some typical values of void ratio, moisture content in a saturated condition, and dry unit weight for soils in a natural state are given in Table 3.2.

Table 3.1 Various Forms of Relationships for γ, γ_d, and γ_{sat}

Moist unit weight (γ)		Dry unit weight (γ_d)		Saturated unit weight (γ_{sat})	
Given	**Relationship**	**Given**	**Relationship**	**Given**	**Relationship**
w, G_s, e	$\dfrac{(1+w)G_s\gamma_w}{1+e}$	γ, w	$\dfrac{\gamma}{1+w}$	G_s, e	$\dfrac{(G_s+e)\gamma_w}{1+e}$
S, G_s, e	$\dfrac{(G_s+Se)\gamma_w}{1+e}$	G_s, e	$\dfrac{G_s\gamma_w}{1+e}$	G_s, n	$[(1-n)G_s+n]\gamma_w$
w, G_s, S	$\dfrac{(1+w)G_s\gamma_w}{1+\dfrac{wG_s}{S}}$	G_s, n	$G_s\gamma_w(1-n)$	G_s, w_{sat}	$\left(\dfrac{1+w_{sat}}{1+w_{sat}G_s}\right)G_s\gamma_w$
w, G_s, n	$G_s\gamma_w(1-n)(1+w)$	G_s, w, S	$\dfrac{G_s\gamma_w}{1+\left(\dfrac{wG_s}{S}\right)}$	e, w_{sat}	$\left(\dfrac{e}{w_{sat}}\right)\left(\dfrac{1+w_{sat}}{1+e}\right)\gamma_w$
S, G_s, n	$G_s\gamma_w(1-n)+nS\gamma_w$	e, w, S	$\dfrac{eS\gamma_w}{(1+e)w}$	n, w_{sat}	$n\left(\dfrac{1+w_{sat}}{w_{sat}}\right)\gamma_w$
		γ_{sat}, e	$\gamma_{sat}-\dfrac{e\gamma_w}{1+e}$	γ_d, e	$\gamma_d+\left(\dfrac{e}{1+e}\right)\gamma_w$
		γ_{sat}, n	$\gamma_{sat}-n\gamma_w$	γ_d, n	$\gamma_d+n\gamma_w$
		γ_{sat}, G_s	$\dfrac{(\gamma_{sat}-\gamma_w)G_s}{(G_s-1)}$	γ_d, S	$\left(1-\dfrac{1}{G_s}\right)\gamma_d+\gamma_w$
				γ_d, w_{sat}	$\gamma_d(1+w_{sat})$

Table 3.2 Void Ratio, Moisture Content, and Dry Unit Weight for Some Typical Soils in a Natural State

Type of soil	Void ratio, e	Natural moisture content in a saturated state (%)	Dry unit weight, γ_d lb/ft^3	Dry unit weight, γ_d kN/m^3
Loose uniform sand	0.8	30	92	14.5
Dense uniform sand	0.45	16	115	18
Loose angular-grained silty sand	0.65	25	102	16
Dense angular-grained silty sand	0.4	15	121	19
Stiff clay	0.6	21	108	17
Soft clay	0.9–1.4	30–50	73–93	11.5–14.5
Loess	0.9	25	86	13.5
Soft organic clay	2.5–3.2	90–120	38–51	6–8
Glacial till	0.3	10	134	21

Table 3-6 Representative Values of *k* for Fine Sand Below the Water Table for Static and Cyclic Loading

Recommended *k*	Relative Density		
	Loose	Medium	Dense
MN/m^3 (pci)	5.4 (20.0)	16.3 (60.0)	34 (125.0)

Table 3-7 Representative Values of *k* for Fine Sand Above Water Table for Static and Cyclic Loading

Recommended *k*	Relative Density		
	Loose	Medium	Dense
MN/m^3 (pci)	6.8 (25.0)	24.4 (90.0)	61.0 (225.0)

If the sand profile is coarse or well-graded sand, the user may consider using a higher value of *k* that those suggested in the tables above. While experimental data for *k* in well-graded sands is poorly documented, use of values 10 to 50 percent higher may be appropriate in dense and very dense well-graded sands that do not contain any compressible minerals such as mica.

7. Fit the parabola between point *k* and point *m* as follows:

a. Compute the slope of the *p-y* curve between point *m* and point *u* using

$$m = \frac{p_u - p_m}{y_u - y_m} \quad \text{(3-62)}$$

b. Compute the power of the parabolic section using

$$n = \frac{p_m}{m y_m} \quad \text{(3-63)}$$

c. Compute the coefficient $\bar{C}$ using

$$\bar{C} = \frac{p_m}{y_m^{1/n}} \quad \text{(3-64)}$$

8. Compute the *y* value defining point *k* using

$$y_k = \left(\frac{\bar{C}}{kx}\right)^{\frac{n}{n-1}} \quad \text{(3-65)}$$

Compute the *p* value defining point *k* using

5. Establish the initial linear portion of the *p-y* curve, using the appropriate value of k_s for static loading or k_c for cyclic loading from Table 3-3 for k.

$$p = (kx)y \quad \text{(3-29)}$$

Table 3-3 Representative Values of *k* for Stiff Clays

Average Undrained Shear Strength*	k_s (static loading)	k_c (cyclic loading)
50-100 kPa (1,000-2,000 psf)	135 MN/m^3 (500 pci)	55 MN/m^3 (200 pci)
100-200 kPa (2,000-4,000 psf)	270 MN/m^3 (1,000 pci)	110 MN/m^3 (400 pci)
200-400 kPa (4,000-6,000 psf)	540 MN/m^3 (2,000 pci)	220 MN/m^3 (800 pci)

*The average shear strength should be computed as the average of shear strength of the soil from the ground surface to a depth of 5 pile diameters. It should be defined as one-half the maximum principal stress difference in an unconsolidated-undrained triaxial test. Note: Conversions of stress ranges are approximate in this table.

6. Compute y_{50} as

$$y_{50} = \varepsilon_{50} b \quad \text{(3-30)}$$

using an appropriate value of ε_{50} from results of laboratory tests or, in the absence of laboratory tests, from Table 3-4. Note that the strain values of ε_{50} are dimensionless.

Table 3-4 Representative Values of ε_{50} for Stiff to Hard Clays

Average Undrained Shear Strength	ε_{50}
50-100 kPa (1,000-2,000 psf)	0.007
100-200 kPa (2,000-4,000 psf)	0.005
200-400 kPa (4,000-6,000 psf)	0.004

7. Compute the first parabolic portion of the *p-y* curve using the following equation. The value of p_c is computed using the smaller of the two values computed using Equations 3-26 for shallow wedge failure conditions or Equation 3-27 for deep flow-around failure conditions.

loading. The load was applied in two directions, with the load in the forward direction being more than twice as large as the load in the backward direction. After a significant number of cycles, the deflection at the top of the pile was either stable or creeping slowly, so an equilibrium condition was assumed. The *p-y* curves for cyclic loading are intended to represent a lower-bound condition. Thus, a designer might possibly be computing an overly conservative response of a pile, if the cyclic *p-y* curves are used and if there are only a small number of applications of the design load (the factored load).

3-3-7-2 Procedure for Computing *p-y* Curves in Soft Clay for Static Loading

The following procedure is for short-term static loading and is illustrated by Figure 3-12(a). As noted earlier, the curves for static loading constitute the basis for indicating the influence of cyclic loading and would be rarely used in design if cyclic loading is of concern.

1. Obtain the best possible estimates of the variation of undrained shear strength *c* and effective unit weight with depth. Also, obtain the value of ε_{50}, the strain corresponding to one-half the maximum principal stress difference. If no stress-strain curves are available, typical values of ε_{50} are given in Table 3-2.

Table 3-2 Representative Values of ε_{50} for Soft to Stiff Clays

Consistency of Clay	ε_{50}
Soft	0.020
Medium	0.010
Stiff	0.005

2. Compute the ultimate soil resistance per unit length of pile, using the smaller of the values given by the equations below.

$$p_u = \left[3 + \frac{\gamma'_{avg}}{c} x + \frac{J}{b} x \right] cb \quad (3\text{-}20)$$

$$p_u = 9cb \quad (3\text{-}21)$$

where

γ'_{avg} = average effective unit weight from ground surface to *p-y* curve,[1]

x = depth from the ground surface to *p-y* curve,

c = shear strength at depth x, and

b = width of pile.

[1] Matlock did not specify in his original paper whether the unit weight was total unit weight or effective unit weight. However, API RP2A specifies that effective unit weight be used. Most users have adopted the recommendation by API and this is the implementation chosen for LPile.

Earth Pressure

Earth Pressure:

Backfill engineering strength parameters

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

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

Internal friction angle $\phi' := 32 \cdot deg$

Cohesion $c_1 := 0 \cdot psf$

Integral Abutment - Passive Earth Pressure - Coulomb Theory

α = Angle of fill slope to the horizontal $\alpha := 0 \cdot deg$

$\phi_1 =$ Angle of internal friction $\phi' = 32 \cdot deg$

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

Use Coulomb for cases where interface friction is considered; typically gravity shaped structures, and integral abutments where the ratio of wall height to wall movement is .005 or greater. Coulomb should also be used when the fill slope is greater than horizontal.

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

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

$\delta' := 19.5 \cdot deg$

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

Das, Principles of Foundation Engineering 7th Ed. p. 366 Eq. 7.71

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

Integral Abutment and Wingwall - Passive Earth Pressure - Rankine Theory

Use Rankine only if the ratio of wall height to wall movement is significantly less than .005 and the fill slope is horizontal to the top of the wall. Bowles does not recommend use of Rankine method for K_p when α > 0.

α = Angle of fill slope to the horizontal $\alpha := 0 \cdot deg$

$$K_{p\_rank} := \cos(\alpha) \cdot \frac{\cos(\alpha) + \sqrt{\cos(\alpha)^2 - \cos(\phi')^2}}{\cos(\alpha) - \sqrt{\cos(\alpha)^2 - \cos(\phi')^2}}$$

Das, Principles of Foundation Engineering 7th Ed. p. 363 Eq. 7.67

$\boxed{K_{p\_rank} = 3.25}$

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

Cantilevered Wingwall Live Load Surcharge
At-Rest Earth Pressure - Rankine Theory

$K_o := 1 - \sin(\phi')$

Das, Principles of Geotechnical Engineering 7th Ed. p 427 Eq. 13.5

$\boxed{K_o = 0.47}$

Table 3.11.5.3-1—Friction Angle for Dissimilar Materials (U.S. Department of the Navy, 1982a)

Interface Materials	Friction Angle, δ (degrees)	Coefficient of Friction, tan δ (dim.)
Mass concrete on the following foundation materials:		
• Clean sound rock	35	0.70
• Clean gravel, gravel-sand mixtures, coarse sand	29 to 31	0.55 to 0.60
• Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	24 to 29	0.45 to 0.55
• Clean fine sand, silty or clayey fine to medium sand	19 to 24	0.34 to 0.45
• Fine sandy silt, nonplastic silt	17 to 19	0.31 to 0.34
• Very stiff and hard residual or preconsolidated clay	22 to 26	0.40 to 0.49
• Medium stiff and stiff clay and silty clay	17 to 19	0.31 to 0.34
Masonry on foundation materials has same friction factors.		
Steel sheet piles against the following soils:		
• Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	22	0.40
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17	0.31
• Silty sand, gravel or sand mixed with silt or clay	14	0.25
• Fine sandy silt, nonplastic silt	11	0.19
Formed or precast concrete or concrete sheet piling against the following soils:		
• Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	22 to 26	0.40 to 0.49
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	$\boxed{17 \text{ to } 22}$	0.31 to 0.40
• Silty sand, gravel or sand mixed with silt or clay	17	0.31
• Fine sandy silt, nonplastic silt	14	0.25
Various structural materials:		
• Masonry on masonry, igneous and metamorphic rocks:		
o dressed soft rock on dressed soft rock	35	0.70
o dressed hard rock on dressed soft rock	33	0.65
o dressed hard rock on dressed hard rock	29	0.55
• Masonry on wood in direction of cross grain	26	0.49
• Steel on steel at sheet pile interlocks	17	0.31

3.11.5.4—Passive Lateral Earth Pressure Coefficient, k_p

For noncohesive soils, values of the coefficient of passive lateral earth pressure may be taken from Figure 3.11.5.4-1 for the case of a sloping or vertical wall with a horizontal backfill or from Figure 3.11.5.4-2 for the case of a vertical wall and sloping backfill. For conditions that deviate from those described in Figures 3.11.5.4-1 and 3.11.5.4-2, the passive pressure may be calculated by using a trial procedure based on wedge theory, e.g., see Terzaghi et al. (1996). When wedge theory is used, the limiting value of the wall friction angle should not be taken larger than one-half the angle of internal friction, ϕ_f.

For cohesive soils, passive pressures may be estimated by:

C3.11.5.4

The movement required to mobilize passive pressure is approximately 10.0 times as large as the movement needed to induce earth pressure to the active values. The movement required to mobilize full passive pressure in loose sand is approximately five percent of the height of the face on which the passive pressure acts. For dense sand, the movement required to mobilize full passive pressure is smaller than five percent of the height of the face on which the passive pressure acts, and five percent represents a conservative estimate of the movement required to mobilize the full passive pressure. For poorly compacted cohesive soils, the movement required to mobilize full passive pressure is larger than five percent of the height of the face on which the pressure acts.

Table 7.10 Values of K_p [from Eq. (7.71)] for $\beta = 90°$ and $\alpha = 0°$

	δ' (deg)				
ϕ' (deg)	0	5	10	15	20
15	1.698	1.900	2.130	2.405	2.735
20	2.040	2.313	2.636	3.030	3.525
25	2.464	2.830	3.286	3.855	4.597
30	3.000	3.506	4.143	4.977	6.105
35	3.690	4.390	5.310	6.854	8.324
40	4.600	5.590	6.946	8.870	11.772

Figure 7.25b shows the force triangle at equilibrium for the trial wedge ABC_1. From this force triangle, the value of P_p can be determined, because the direction of all three forces and the magnitude of one force are known.

Similar force triangles for several trial wedges, such as ABC_1, ABC_2, ABC_3, . . . , can be constructed, and the corresponding values of P_p can be determined. The top part of Figure 7.25a shows the nature of variation of the P_p values for different wedges. The *minimum value of* P_p in this diagram is *Coulomb's passive force,* mathematically expressed as

$$P_p = \tfrac{1}{2}\gamma H^2 K_p \tag{7.70}$$

where

$$K_p = \text{Coulomb's passive pressure coefficient}$$

$$= \frac{\sin^2(\beta - \phi')}{\sin^2\beta \sin(\beta + \delta')\left[1 - \sqrt{\dfrac{\sin(\phi' + \delta')\sin(\phi' + \alpha)}{\sin(\beta + \delta')\sin(\beta + \alpha)}}\right]^2} \tag{7.71}$$

The values of the passive pressure coefficient, K_p, for various values of ϕ' and δ' are given in Table 7.10 $(\beta = 90°, \alpha = 0°)$.

Note that the resultant passive force, P_p, will act at a distance $H/3$ from the bottom of the wall and will be inclined at an angle δ' to the normal drawn to the back face of the wall.

7.13 Comments on the Failure Surface Assumption for Coulomb's Pressure Calculations

Coulomb's pressure calculation methods for active and passive pressure have been discussed in Sections 7.5 and 7.12. The fundamental assumption in these analyses is the acceptance of *plane failure surface.* However, for walls with friction, this assumption does not hold in practice. The nature of *actual* failure surface in the soil mass for active and passive pressure is shown in Figure 7.26a and b, respectively (for a vertical wall with a horizontal backfill). Note that the failure surface BC is curved and that the failure surface CD is a plane.

Although the actual failure surface in soil for the case of active pressure is somewhat different from that assumed in the calculation of the Coulomb pressure, the results are not greatly different. However, in the case of passive pressure, as the value of δ' increases, Coulomb's

Table 7.9 (*Continued*)

ϕ' (deg)	α (deg)	$c'/\gamma z$ 0.025	0.050	0.100	0.500
30	0	3.087	3.173	3.346	4.732
	5	3.042	3.129	3.303	4.674
	10	2.907	2.996	3.174	4.579
	15	2.684	2.777	2.961	4.394

7.12 Coulomb's Passive Earth Pressure

Coulomb (1776) also presented an analysis for determining the passive earth pressure (i.e., when the wall moves *into* the soil mass) for walls possessing friction (δ' = angle of wall friction) and retaining a granular backfill material similar to that discussed in Section 7.5.

To understand the determination of Coulomb's passive force, P_p, consider the wall shown in Figure 7.25a. As in the case of active pressure, Coulomb assumed that the potential failure surface in soil is a plane. For a trial failure wedge of soil, such as ABC_1, the forces per unit length of the wall acting on the wedge are

1. The weight of the wedge, W
2. The resultant, R, of the normal and shear forces on the plane BC_1, and
3. The passive force, P_p

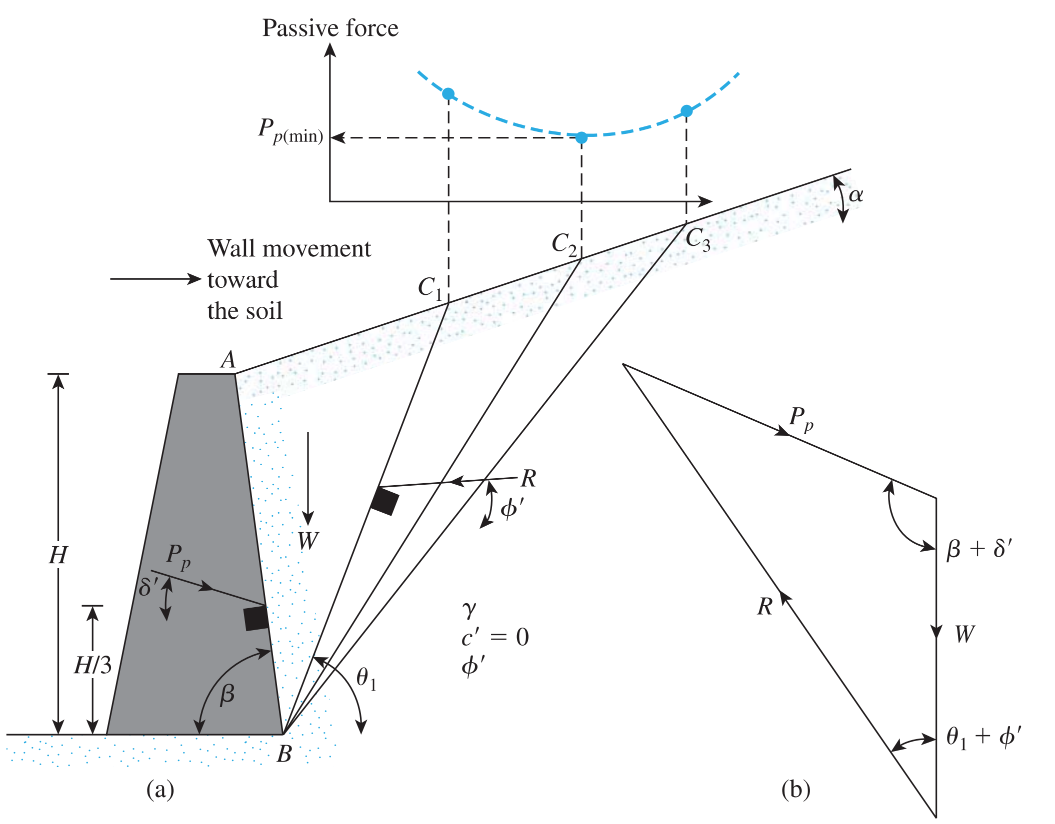


Figure 7.25 Coulomb's passive pressure

At this depth, that is $z = 2$ m, for the bottom soil layer

$$\sigma'_p = \sigma'_o K_{p(2)} + 2c'_2\sqrt{K_{p(2)}} = 31.44(2.56) + 2(10)\sqrt{2.56}$$
$$= 80.49 + 32 = 112.49 \text{ kN/m}^2$$

Again, at $z = 3$ m,

$$\sigma'_o = (15.72)(2) + (\gamma_{sat} - \gamma_w)(1)$$
$$= 31.44 + (18.86 - 9.81)(1) = 40.49 \text{ kN/m}^2$$

Hence,

$$\sigma'_p = \sigma'_o K_{p(2)} + 2c'_2\sqrt{K_{p(2)}} = 40.49(2.56) + (2)(10)(1.6)$$
$$= \mathbf{135.65\ kN/m^2}$$

Note that, because a water table is present, the hydrostatic stress, u, also has to be taken into consideration. For $z = 0$ to 2 m, $u = 0$; $z = 3$ m, $u = (1)(\gamma_w) = 9.81$ kN/m^2.

The passive pressure diagram is plotted in Figure 6.24b. The passive force per unit length of the wall can be determined from the area of the pressure diagram as follows:

Area no.	Area	
1	$(\frac{1}{2})$ (2)(94.32)	= 94.32
2	(112.49)(1)	= 112.49
3	$(\frac{1}{2})$ (1)(135.65 − 112.49)	= 11.58
4	$(\frac{1}{2})$ (9.81)(1)	= 4.905
		$P_P \approx 223.3$ **kN/m**

■

7.11 Rankine Passive Earth Pressure: Vertical Backface and Inclined Backfill

Granular Soil

For a frictionless vertical retaining wall (Figure 7.10) with a *granular backfill* $(c' = 0)$, the Rankine passive pressure at any depth can be determined in a manner similar to that done in the case of active pressure in Section 7.4. The pressure is

$$\sigma'_p = \gamma z K_p \tag{7.65}$$

and the passive force is

$$P_p = \tfrac{1}{2}\gamma H^2 K_p \tag{7.66}$$

where

$$K_p = \cos\alpha \frac{\cos\alpha + \sqrt{\cos^2\alpha - \cos^2\phi'}}{\cos\alpha - \sqrt{\cos^2\alpha - \cos^2\phi'}} \tag{7.67}$$

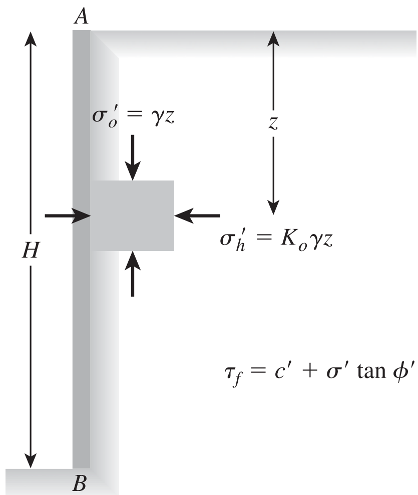


Figure 13.3
Earth pressure at rest

which shows a wall AB retaining a dry soil with a unit weight of γ. The wall is static. At a depth z,

$$\text{Vertical effective stress} = \sigma'_o = \gamma z$$
$$\text{Horizontal effective stress} = \sigma'_h = K_o \gamma z$$

So,

$$K_o = \frac{\sigma'_h}{\sigma'_o} = \text{at-rest earth pressure coefficient}$$

For coarse-grained soils, the coefficient of earth pressure at rest can be estimated by using the empirical relationship (Jaky, 1944)

$$\boxed{K_o = 1 - \sin \phi'} \tag{13.5}$$

where ϕ' = drained friction angle.

While designing a wall that may be subjected to lateral earth pressure at rest, one must take care in evaluating the value of K_o. Sherif, Fang, and Sherif (1984), on the basis of their laboratory tests, showed that Jaky's equation for K_o [Eq. (13.5)] gives good results when the backfill is loose sand. However, for a dense, compacted sand backfill, Eq. (13.5) may grossly underestimate the lateral earth pressure at rest. This underestimation results because of the process of compaction of backfill. For this reason, they recommended the design relationship

$$K_o = (1 - \sin \phi) + \left[\frac{\gamma_d}{\gamma_{d(\min)}} - 1\right] 5.5 \tag{13.6}$$

where γ_d = actual compacted dry unit weight of the sand behind the wall
$\gamma_{d(\min)}$ = dry unit weight of the sand in the loosest state (Chapter 3)

Settlement

Development of embankment soil model for Settle 3D

OBJECTIVE
Estimate soil parameters for Settle analyses.

Given:
1) Boring logs and lab data

Assumptions:
1) Groundwater was observed 9.0 ft bgs during the investigation. Assume the groundwater table is at Elevation 1516.5.
2) MaineDOT Bridge Design Guide (BDG) Soil Type 4 will be used for integral abutment backfill.
3) BDG Soil Type 5 is used to construct the proposed raise in grade.

Embankment Soil Model

Soil Layer No. 1 (Gravel Borrow) El. 1526.5-1525.0

Assume material used to construct 1.5 foot increase in grade is similar to gravel borrow.

Soil Layer Height

$h_1 := 1.5ft$

Soil Total Unit Weight

$\gamma_1 := 135pcf$

MaineDOT BDG Table 3-3
Soil Type 5

Assume the total (moist) unit weight of Soil Type 5 considers placement at the material's optimum moisture content. Assume optimum moisture content of gravel borrow occurs at 8 percent.

Optimum moisture content

$w_{1\_opt} := .08$

Dry unit weight

$$\gamma_{1\_dry} := \frac{\gamma_1}{1 + w_{1\_opt}}$$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59:
Table 3.1 Unit Weight Relationships

$\gamma_{1\_dry} = 125 \cdot pcf$

Saturated Moisture Content

Natural water content at saturated state:
Loose uniform sand = 30%
Dense angular silty sand = 15%
Average Loose and Dense for Medium Dense:
Medium Dense angular silty sand: 23%

Das, Principles of Geotechnical Eng. 7th Ed. p. 59:
Table 3.2 - Natural Moisture Content in a saturated state

$w_{1\_sat} := .23$

Saturated Unit Weight

$$\gamma_{1\_saturated} := \gamma_{1\_dry} \cdot \left(1 + w_{1\_sat}\right)$$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59:
Table 3.1 Unit Weight Relationships

$\gamma_{1\_saturated} = 154 \cdot pcf$

Soil Layer No. 2 (Medium Dense, SAND) El. 1525.0-1516.0

Soil Layer Height

$h_2 := 9\text{ft}$

Dry Unit Weight

$\gamma_{2dry} := 112\text{pcf}$

Hough, Basic Soils Eng. 2nd Ed. p. 34-35
Table 2-3 Clean, fine to coarse SAND (use average of dry weights for medium dense)

Saturated Moisture Content

Natural water content at saturated state:
Loose uniform sand = 30%
Dense uniform sand = 16%

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 - Natural Moisture Content in a saturated state (average loose and dense)

$w_{2sat} := .23$

Saturated Unit Weight

$\gamma_{2saturated} := \gamma_{2dry} \cdot (1 + w_{2sat})$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships

$\gamma_{2saturated} = 138 \cdot \text{pcf}$

Natural Moisture Content

Assume the natural moisture content is half of the saturated moisture content

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 - Natural Moisture Content in a saturated state (average loose and dense)

$w_{2moist} := .5 \cdot w_{2sat} = 0.12$

Moist Unit Weight

$\gamma_{2moist} := \gamma_{2dry} \cdot (1 + w_{2moist})$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships

$\gamma_{2moist} = 125 \cdot \text{pcf}$

Design N-value

BB-RHC-101;2D

$N_{60\_2} := 15$

Young's Modulus

$E_{s\_2} := 500(N_{60\_2} + 15) = 15000$

Bowles Foundation Analysis and Design 5th Ed. p. 316: Table 5-6 for Gravelly SAND with N < 15

$E_{s\_2} := 15000\text{kPa} = 313 \cdot \text{ksf}$

Unload Reload Modulus

$E_{ur2} := 4 \cdot E_{s\_2}$

Mayne and Cox, Constitutive Model Input Parameters (2015), Eq. (5).

$E_{ur2} = 1253 \cdot \text{ksf}$

Soil Layer No. 3 (Medium dense, Gravelly SAND) El. 1516.0-1510.0

Soil Layer Height

$h_3 := 6\text{ft}$

Soil Dry Unit Weight

$\gamma_{3dry} := 112pcf$ — Similar to Soil Layer No. 2

Saturated Moisture Content

$w_{3sat} := .23$ — Similar to Soil Layer No. 2

Saturated Unit Weight

$\gamma_{3saturated} := \gamma_{3dry} \cdot (1 + w_{3sat})$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships

$\gamma_{3saturated} = 138 \cdot pcf$

Design N-value

$N_{60\_3} := 17$ — BB-RHC-101;3D

Young's Modulus

$E_{s\_3} := 600 \cdot (N_{60\_3} + 6) + 2000 = 15800$

Bowles Foundation Analysis and Design 5th Ed. p. 316: Table 5-6 for Gravelly sand (N > 15)

$E_{s\_3} := 15800kPa = 330\,ksf$

Unload Reload Modulus

$E_{ur3} := 4 \cdot E_{s\_3}$

Mayne and Cox, Constitutive Model Input Parameters (2015), Eq. (5).

$E_{ur3} = 1320 \cdot ksf$

Soil Layer No. 4 (Dense, Gravelly SAND) El. 1510.0-1506.0

Soil Layer Height

$h_4 := 4ft$

Soil Total Unit Weight

$\gamma_4 := 125pcf$

MaineDOT BDG Table 3-3
Soil Type 4

Assume the total (moist) unit weight of Soil Type 4 considers placement at the material's optimum moisture content. Assume optimum moisture content of gravel borrow occurs at 8 percent.

Optimum moisture content

$w_{4\_opt} := .08$

Dry unit weight

$$\gamma_{4\_dry} := \frac{\gamma_4}{1 + w_{4\_opt}}$$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships

$\gamma_{4\_dry} = 116 \cdot pcf$

Saturated Moisture Content

Natural water content at saturated state:
Loose uniform sand = 30%
Dense angular silty sand = 15%
Average Loose and Dense for Medium Dense:
Medium Dense angular silty sand: 23%

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 - Natural Moisture Content in a saturated state

$$w_{4\_sat} := .23$$

Saturated Unit Weight

$$\gamma_{4saturated} := \gamma_{4\_dry} \cdot \left(1 + w_{4\_sat}\right)$$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships

$$\gamma_{4saturated} = 142 \cdot \mathrm{pcf}$$

Design N-value

$$N_{60\_4} := 39$$

BB-RHC-101;4D

Young's Modulus

$$E_{s\_4} := 600 \cdot \left(N_{60\_4} + 6\right) + 2000 = 29000$$

Bowles Foundation Analysis and Design 5th Ed. p. 316: Table 5-6 for Gravelly sand (N > 15)

$$E_{s\_4} := 29000\mathrm{kPa} = 606\,\mathrm{ksf}$$

Unload Reload Modulus

$$E_{ur4} := 4 \cdot E_{s\_4}$$

Mayne and Cox, Constitutive Model Input Parameters (2015), Eq. (5).

$$E_{ur4} = 2423 \cdot \mathrm{ksf}$$

Soil Layer No. 5 (Peat)

Unit Weight of Water

$$\gamma_w := 62.4\mathrm{pcf}$$

Existing Effective Overburden Stress at the top of the peat layer

$$\sigma' := \left[\left(h_2 \cdot \gamma_{2moist}\right) + \left[h_3 \cdot \left(\gamma_{3saturated} - \gamma_w\right)\right] + h_4 \cdot \left(\gamma_{4saturated} - \gamma_w\right)\right]$$

$$\sigma' = 1896 \cdot \mathrm{psf} \qquad 1896\mathrm{psf} = 91 \cdot \mathrm{kPa}$$

Void Ratio, e

Mesri and Ajlouni, Engineering Properties of Fibrous Peats, Fig. 11

$$e := 7.5$$

Dry Unit Weight

$$\gamma_{5dry} := 65\mathrm{pcf} = 1041.2 \cdot \frac{\mathrm{kg} \cdot \mathrm{g}}{\mathrm{m}^3}$$

Kazemain, et al. Table 1 - Bulk Density

Saturated Unit Weight

$$\gamma_{5saturated} := \gamma_{5dry} + \left(\frac{e}{1 + e}\right) \cdot \gamma_w$$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships

$$\gamma_{5saturated} = 120 \cdot \mathrm{pcf}$$

Young's Modulus

$E_{s\_5} := 225kPa = 5 \cdot ksf$

Rowe and Soderman, Geotextile Reinforcement of Embankments on Peat Table 1

Unload Reload Modulus

$E_{ur5} := 4 \cdot E_{s\_5}$

$E_{ur5} = 19 \cdot ksf$

Mayne and Cox, Constitutive Model Input Parameters (2015), Eq. (5).

Water Content

$w_5 := 197.9$

Natural moisture content of sample BB-RHC-201;2D

Compression Index

$C_c := 0.0115(w_5)$

$C_c = 2.28$

FHWA GEC No. 6 Table 5-11 p. 83

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Hunter Cove Bridge #2384 carries Mingo Loop Road over Hunter Cove Location: Rangeley, Maine; Franklin County	Boring No.: BB-RHC-101 WIN: 18955.00

Driller:	MaineDOT	**Elevation (ft.)**	1525.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/3/2016-5/4/2016	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	7+55.6, 9.8 ft Lt.	**Casing ID/OD:**	HW-4"/NW-3"	**Water Level*:**	9.0 ft bgs.
Hammer Efficiency Factor:	0.908	**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 = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field 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 = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane 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	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	1525.1		4" HMA. 0.33	
	1D	24/14	1.50 - 3.50	5/5/12/18	17	26				Brown, damp, medium dense, Gravelly SAND, trace silt, trace relic pavement, (Fill).	
5	2D	24/18	5.00 - 7.00	7/4/6/9	10	15				Brown, damp, medium dense, SAND, some gravel, little silt.	G#270703 A-1-b, SW-SM WC=4.1%
10	3D	24/4	10.00 - 12.00	4/8/3/6	11	17	36			Brown, wet, medium dense, Gravelly SAND, trace silt.	
							57				
							90				
							152				
							66				
15	4D	24/3	15.00 - 17.00	11/19/7/4	26	39	36			Similar to above, except dense.	
							53				
							57				
							172			Wood from 18.3-18.8 ft bgs.	
							94	1505.9		19.50	
20	5D	24/22	20.00 - 22.00	2/2/2/4	4	6	31			Dark brown, wet, fibrous PEAT, (Marsh).	G#270704 A-2-4, SM/OL WC=197.9% Ignition Loss 45.9%
							54				
							76				
							87	1501.9		23.50	
25							75				

Soil Layer 1 - Proposed Fill

Soil Layer 2 - Medium dense, SAND

Soil Layer 3 - Medium dense, Gravelly SAND

Soil Layer 4 - Dense, Gravelly SAND

Soil Layer 5 - PEAT

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.

3.4 Construction Loads

The construction live load to be used for constructibility checks is 50 psf applied over the entire deck area. Consideration should be given to slab placement sequence for calculation of maximum force effects.

3.5 Railroad Loads

Railroad bridges should be designed according to the latest American Railroad Engineering and Maintenance-of-Way Association specifications (AREMA, 2002), with the Cooper live loading as determined by the railroad company.

3.6 Earth Loads

3.6.1 General

Earth pressures considered for wall and substructure design must use the appropriate soil weight shown in Table 3-3.

Table 3-3 Material Classification

Soil Type	Soil Description	Internal Angle of Friction of Soil, ϕ	Soil Total Unit Weight (pcf)	Coeff. of Friction, tan δ, Concrete to Soil	Interface Friction, Angle, Concrete to Soil δ
1	Very loose to loose silty sand and gravel Very loose to loose sand Very loose to medium density sandy silt Stiff to very stiff clay or clayey silt	29° *	100	0.35	19°
2	Medium density silty sand and gravel Medium density to dense sand Dense to very dense sandy silt	33°	120	0.40	22°
3	Dense to very dense silty sand and gravel Very dense sand	36°	130	0.45	24°
4	Granular underwater backfill Granular borrow	32°	125	0.45	24°
5	Gravel Borrow	36°	135	0.50	27°

* The value given for the internal angle of friction (ϕ) for stiff to very stiff silty clay or clayey silt should be used with caution due to the large possible variation with different moisture contents.

Das, Principles of Geotechnical Engineering 7th ed.

3.4 Various Unit-Weight Relationships

In Sections 3.2 and 3.3, we derived the fundamental relationships for the moist unit weight, dry unit weight, and saturated unit weight of soil. Several other forms of relationships that can be obtained for γ, γ_d, and γ_{sat} are given in Table 3.1. Some typical values of void ratio, moisture content in a saturated condition, and dry unit weight for soils in a natural state are given in Table 3.2.

Table 3.1 Various Forms of Relationships for γ, γ_d, and γ_{sat}

Moist unit weight (γ)		Dry unit weight (γ_d)		Saturated unit weight (γ_{sat})	
Given	**Relationship**	**Given**	**Relationship**	**Given**	**Relationship**
w, G_s, e	$\dfrac{(1+w)G_s\gamma_w}{1+e}$	γ, w	$\dfrac{\gamma}{1+w}$	G_s, e	$\dfrac{(G_s+e)\gamma_w}{1+e}$
S, G_s, e	$\dfrac{(G_s+Se)\gamma_w}{1+e}$	G_s, e	$\dfrac{G_s\gamma_w}{1+e}$	G_s, n	$[(1-n)G_s+n]\gamma_w$
w, G_s, S	$\dfrac{(1+w)G_s\gamma_w}{1+\dfrac{wG_s}{S}}$	G_s, n	$G_s\gamma_w(1-n)$	G_s, w_{sat}	$\left(\dfrac{1+w_{sat}}{1+w_{sat}G_s}\right)G_s\gamma_w$
w, G_s, n	$G_s\gamma_w(1-n)(1+w)$	G_s, w, S	$\dfrac{G_s\gamma_w}{1+\left(\dfrac{wG_s}{S}\right)}$	e, w_{sat}	$\left(\dfrac{e}{w_{sat}}\right)\left(\dfrac{1+w_{sat}}{1+e}\right)\gamma_w$
S, G_s, n	$G_s\gamma_w(1-n)+nS\gamma_w$	e, w, S	$\dfrac{eS\gamma_w}{(1+e)w}$	n, w_{sat}	$n\left(\dfrac{1+w_{sat}}{w_{sat}}\right)\gamma_w$
		γ_{sat}, e	$\gamma_{sat}-\dfrac{e\gamma_w}{1+e}$	γ_d, e	$\gamma_d+\left(\dfrac{e}{1+e}\right)\gamma_w$
		γ_{sat}, n	$\gamma_{sat}-n\gamma_w$	γ_d, n	$\gamma_d+n\gamma_w$
		γ_{sat}, G_s	$\dfrac{(\gamma_{sat}-\gamma_w)G_s}{(G_s-1)}$	γ_d, S	$\left(1-\dfrac{1}{G_s}\right)\gamma_d+\gamma_w$
				γ_d, w_{sat}	$\gamma_d(1+w_{sat})$

Table 3.2 Void Ratio, Moisture Content, and Dry Unit Weight for Some Typical Soils in a Natural State

Type of soil	Void ratio, e	Natural moisture content in a saturated state (%)	Dry unit weight, γ_d lb/ft^3	Dry unit weight, γ_d kN/m^3
Loose uniform sand	0.8	30	92	14.5
Dense uniform sand	0.45	16	115	18
Loose angular-grained silty sand	0.65	25	102	16
Dense angular-grained silty sand	0.4	15	121	19
Stiff clay	0.6	21	108	17
Soft clay	0.9–1.4	30–50	73–93	11.5–14.5
Loess	0.9	25	86	13.5
Soft organic clay	2.5–3.2	90–120	38–51	6–8
Glacial till	0.3	10	134	21

Hough, 2nd Ed. , 1969

TABLE 2–3. Typical Values of Soil Index Properties

	Part. Size & Gradation				Voids*					Unit Weight† (lb./cu.ft)						
	Approx. Size Range (mm.)		Approx. D_{10} (mm.)	Approx. Range Unif. Coef., C_u	Void Ratio			Porosity (%)		Dry Wt., γ_{dry}			Wet Wt., γ_{wet}		Sub. Wt., γ_{sub}	
	D_{max}	D_{min}			e_{max} (loose)	e_{cr}	e_{min} (dense)	n_{max} (loose)	n_{min} (dense)	Min. (loose)	100% Mod. AASHO	Max. (dense)	Min. (loose)	Max. (dense)	Min. (loose)	Max. (dense)
Granular Materials																
1. Uniform Materials																
a. Equal spheres (theoretical values)	—	—	—	1.0	0.92	—	0.35	47.6	26.0	—	—	—	—	—	—	—
b. Standard Ottawa SAND	0.84	0.59	0.67	1.1	0.80	0.75	0.50	44	33	92	—	110	93	131	57	69
c. Clean, uniform SAND (fine or medium)	—	—	—	1.2 to 2.0	1.0	0.80	0.40	50	29	83	115	118	84	136	52	73
d. Uniform, inorganic SILT	0.05	0.005	0.012	1.2 to 2.0	1.1	—	0.40	52	29	80	—	118	81	136	51	73
2. Well-graded Materials																
a. Silty SAND	2.0	0.005	0.02	5 to 10	0.90	—	0.30	47	23	87	122	127	88	142	54	79
b. Clean, fine to coarse SAND	2.0	0.05	0.09	4 to 6	0.95	0.70	0.20	49	17	85	132	138	86	148	53	86
c. Micaceous SAND	—	—	—	—	1.2	—	0.40	55	29	76	—	120	77	138	48	76
d. Silty SAND & GRAVEL	100	0.005	0.02	15 to 300	0.85	—	0.14	46	12	89	—	146‡	90	155‡	56	92
Mixed Soils																
1. Sandy or silty CLAY	2.0	0.001	0.003	10 to 30	1.8	—	0.25	64	20	60	130	135	100	147	38	85
2. Skip-graded silty CLAY with stones or rk. frag.	250	0.001	—	—	1.0	—	0.20	50	17	84	—	140	115	151	53	89
3. Well-graded GRAVEL, SAND, SILT & CLAY mixture	250	0.001	0.002	25 to 1000	0.70	—	0.13	41	11	100	140	148§	125	156§	62	94
Clay Soils																
1. CLAY (30 to 50% clay sizes)	0.05	0.5μ	0.001	—	2.4	—	0.50	71	33	50	105	112	94	133	31	71
2. Colloidal CLAY (−0.002 mm. $\geq$ 50%)	0.01	10Å	—	—	12	—	0.60	92	37	13	90	106	71	128	8	66
Organic Soils																
1. Organic SILT	—	—	—	—	3.0	—	0.55	75	35	40	—	110	87	131	25	69
2. Organic CLAY (30 to 50% clay sizes)	—	—	—	—	4.4	—	0.70	81	41	30	—	100	81	125	18	62

* Granular materials may reach e_{max} when dry or only slightly moist. Clays can reach e_{max} only when fully saturated.

† Granular materials reach minimum unit weight when at e_{max} and with hygroscopic moisture only. Clays reach minimum unit wet weight when fully saturated at e_{max}. The unit submerged weight of any saturated soil is the unit wet weight minus the unit weight of water.

‡ Applicable for very compact glacial till. Unusually high unit weight values for tills are sometimes due not only to an extremely compact condition but to unusually high specific gravity values.

§ Applicable for hardpan.

General Note: Tabulation is based on $G = 2.65$ for granular soil, $G = 2.7$ for clays, and $G = 2.6$ for organic soils.

the highest porosity, namely, clays, are usually the least pervious since the individual void passages in clays are extremely small though the aggregate void volume is relatively large.

Relative Density. It is indicated in Table 2–3 that each soil type has an individual range of porosity or void ratio. To judge whether a soil at a given void ratio, e, is to be described as dense or loose, it is necessary to establish its existing void ratio with respect to the range of possible void ratios for the particular soil. This is expressed by the term *relative density*, D_d (sometimes, though not advisedly, referred to as *degree of compaction*), defined as,

$$D_d = \frac{e_{max} - e}{e_{max} - e_{min}} \qquad (2\text{–}11)$$

Bowles Foundation Analysis, 5th Ed.

TABLE 5-6
Equations for stress-strain modulus E_s by several test methods

E_s in kPa for SPT and units of q_c for CPT; divide kPa by 50 to obtain ksf. The N values should be estimated as N_{55} and not N_{70}. Refer also to Tables 2-7 and 2-8.

Soil	SPT	CPT
Sand (normally consolidated)	$\boxed{E_s = 500(N + 15)}$ $= 7000\sqrt{N}$ $= 6000N$ — — — ‡$E_s = (15\,000 \text{ to } 22\,000) \cdot \ln N$	$E_s = (2 \text{ to } 4)q_u$ $= 8000\sqrt{q_c}$ — — — $E_s = 1.2(3D_r^2 + 2)q_c$ *$E_s = (1 + D_r^2)q_c$
Sand (saturated)	$E_s = 250(N + 15)$	$E_s = Fq_c$ $e = 1.0 \quad F = 3.5$ $e = 0.6 \quad F = 7.0$
Sands, all (norm. consol.)	¶$E_s = (2600 \text{ to } 2900)N$	
Sand (overconsolidated)	†$E_s = 40\,000 + 1050N$ $E_{s(\mathrm{OCR})} \approx E_{s,\mathrm{nc}}\sqrt{\mathrm{OCR}}$	$E_s = (6 \text{ to } 30)q_c$
Gravelly sand	$E_s = 1200(N + 6)$ $= 600(N + 6) \quad N \le 15$ $\boxed{= 600(N + 6) + 2000 \quad N > 15}$	
Clayey sand	$E_s = 320(N + 15)$	$E_s = (3 \text{ to } 6)q_c$
Silts, sandy silt, or clayey silt	$E_s = 300(N + 6)$	$E_s = (1 \text{ to } 2)q_c$
	If $q_c < 2500$ kPa use $\quad$ §$E_s' = 2.5q_c$ $2500 < q_c < 5000$ use $\quad E_s' = 4q_c + 5000$ where $E_s' = \text{constrained modulus} = \dfrac{E_s(1-\mu)}{(1+\mu)(1-2\mu)} = \dfrac{1}{m_v}$	
Soft clay or clayey silt		$E_s = (3 \text{ to } 8)q_c$

4. It is not easy to determine if a cohesionless deposit is overconsolidated or what the OCR might be. Cementation may be less difficult to discover, particularly if during drilling or excavation sand "lumps" are present. Carefully done consolidation tests will aid in obtaining the OCR of cohesive deposits as noted in Chap. 2.

In general, with an OCR > 1 you should carefully ascertain the site conditions that will prevail at the time settlement becomes the design concern. This evaluation is, of course, true for any site, but particularly so if OCR > 1.

5-9 SIZE EFFECTS ON SETTLEMENTS AND BEARING CAPACITY

5-9.1 Effects on Settlements

A major problem in foundation design is to proportion the footings and/or contact pressure so that settlements between adjacent footings are nearly equal. Figure 5-9 illustrates the problem

Kazemian, et al., A state of art review of peat: Geotechnical engineering perspective
International Journal of the Physical Sciences Vol. 6(8) p. 1974-1981 (2011)

Table 1. Physical and chemical properties of peat.

Peat type	Natural water content (w, %)	Bulk density (Mg/m^3)	Specific gravity (G_s)	Acidity (pH)	Ash content (%)	Reference
Fibrous-woody	484-909	-	-	-	17	Colley (1950)
Fibrous	850	0.95-1.03	1.1-1.8	-	-	Hanrahan (1954), Asadi et al., 2009, 2010
Peat	520	-	-	-	-	Lewis (1956)
	500-1500	0.88-1.22	1.5-1.6	-	-	Lea and Browner (1963)
Amorphous and fibrous	200-600	-	1.62	4.8-6.3	12.2-22.5	Adams (1965)
	355-425	-	1.73	6.7	15.9	
Amorphous to fibrous	850	-	1.5	-	14	Keene and Zawodniak (1968)
Fibrous	605-1290	0.87-1.04	1.41-1.7	-	4.6-15.8	Samson and LaRochell (1972), Moayedi et al.(2011a, b)
Coarse fibrous	613-886	1.04	1.5	4.1	9.4	Berry and Vickers (1975)
Fibrous sedge	350	-	-	4.3	4.8	Levesque et al. (1980)
Fibrous sphagnum	778	-	-	3.3	1	
Coarse fibrous	202-1159	1.05	1.5	4.17	14.3	Berry (1983)
Fine fibrous	660	1.05	1.58	6.9	23.9	NG and Eischen (1983)
Fine fibrous	418	1.05	1.73	6.9	9.4	
Amorphous granular	336	1.05	1.72	7.3	19.5	
Peat portage	600	0.96	1.72	7.3	19.5	Edil and Mochtar (1984)
Peat waupaca	460	0.96	1.68	6.2	15	
Fibrous peat (Middleton)	510	0.91	1.41	7	12	
Fibrous peat (Noblesville)	173-757	0.84	1.56	6.4	6.9-8.4	
Fibrous	660-1590	-	1.53-1.68	-	0.1-32.0	Lefebvre et al. (1984)
Fibrous peat	660-890	0.94-1.15	-	-	-	Olson and Mesri (1970)
Amorphous peat	200-875	1.04-1.23	-	-	-	
Peat	125-375	0	1.55-1.63	5-7	22-45	Yamaguchi et al. (1985)
Peat	419	1	1.61	-	22-45	Jones et al. (1986)
Peat	490-1250	-	1.45	-	20-33	Yamaguchi et al. (1987)
Peat	630-1200	-	1.58-1.71	-	22-35	Nakayama et al. (1990)
Peat	400-1100	0.99-1.1	1.47	4.2	5-15	Yamaguchi 1990
Fibrous	700-800	~1.00	-	-	-	Hansbo (1991)
Peat (Netherlands)	669	0.97	1.52	-	20.8	Termatt and Topolnicki (1994)
Fibrous (Middleton)	510-850	0.99-1.1	1.47-1.64	4.2	5-7	Ajlouni (2000)
Fibrous (James Bay)	1000-1340	0.85-1.02	1.37-1.55	5.3	4.1	

1 Megagram per m^3 = 62.4 lb/ft^3. Density ranges from 52.4-76.8 lb/ft^3. Use 65 lb/ft^3.

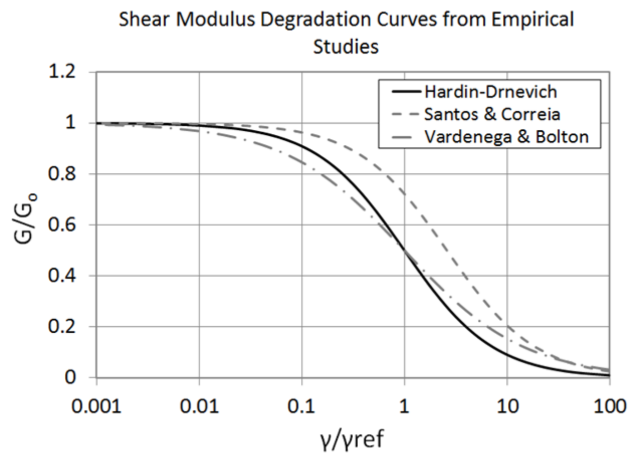


Figure 2. Reduction curves from fitted experimental data studies

$$E_{ur} \cong 4E_{50} \qquad (5)$$

One will note that when viewing the stiffness degradation curve, E_{50} is the smallest of the modulus values discussed. Most numerical programs maintain an elastic stiffness cutoff at E_{ur} (corresponding to G_{ur}), where hardening plasticity accounts for further stiffness reductions.

Advanced hardening models include the values of G_o and $\gamma_{0.7}$ as inputs to define the nonlinearity and small strain stiffness relationships for various geomaterials. Once G_o is determined from seismic shear wave velocity testing, the stiffness degradation curve as shown in Figure 2 can be used to define $\gamma_{0.7}$.

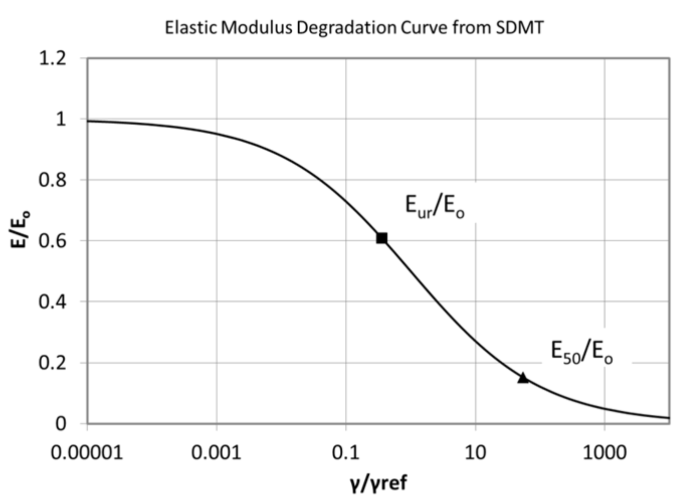


Figure 3. Elastic Modulus reduction curve using SDMT

TABLE 1
Peat Parameters

Parameters	*Value*
Cohesion intercept, c'	1·8 kPa
Angle of internal friction, ϕ'	27°
Dilatancy angle, ψ'	0
Poisson's ratio, ν'	0·15
Coefficient of Earth pressure at rest, K_0'	0·176
Specific gravity, G_s	1·5
Initial void ratio, e_0	9
Unit weight, γ	10·3 kN/m^3
Young's modulus, E' ($0 \leq \sigma'_v \leq 20$ kPa)	85 kPa
E' ($20 < \sigma'_v \leq 40$ kPa	110 kPa
E' ($40 < \sigma'_v \leq 60$ kPa)	140 kPa
E' ($\sigma'_v > 60$ kPa)	225 kPa

above and a soil node below the geotextile. Thus slip could occur independently either above or below the inclusion. As part of the large deformation analysis, the co-ordinates of both the soil and the geotextile were updated during the analysis, thereby allowing the development of 'membrane forces' due to deformation.

4. PREDICTION OF COLLAPSE HEIGHTS

Even though an embankment may be stable, there will often be significant local shear failure within the weak underlying foundation material. Provided that these zones where the shear strength has been reached, referred to as 'plastic zones', are contained or surrounded by soil which is not plastic, that is where the shear stress is less than the shear strength, collapse will not occur. However, the deformation of the embankment may be greatly affected by the development of these contained plastic regions. Thus both deformation and shear strength parameters are required for an elasto-plastic finite element deformation analysis. The collapse height of the embankment is simply the height at which uncontained plastic flow occurs and the shear strain becomes

TABLE 5-11: CORRELATIONS FOR COMPRESSION INDEX, C_C

Correlation	Soil	Source
$C_c = 0.009\ (LL-10)$ (1)	Clay of medium to low sensitivity (S_t<4) (2)	Terzaghi & Peck (1967)
$C_c = 0.0115\ w_n$ (3)	Organic soils, peat	ASCE (1994)
$C_c = 0.04$ to 0.006 (4)	Uniform silts	Hough (1959)
$C_c = 0.015$ to 0.02 (4)	Uniform sand, loose	Hough (1959)
$C_c = 0.004$ to 0.008 (4)	Uniform sand, dense	Hough (1959)

[1] LL=liquid limit
[2] S_t=sensitivity=Undisturbed undrained shear strength/Remolded undrained shear strength
[3] w_n= natural water content
[4] $C_c = 1/C'$ where C' is the bearing capacity index (Figure 5-19). *Note:* These are for cohesionless soils, but are included here for comparison purposes.

where: S_c = primary consolidation due to the change in void ratio of the soil
C_c = compression index of the normally consolidated portion of the e–log σ'_v curve
e_o = initial void ratio
H_o = thickness of layer *n*
σ'_{vo} = initial effective vertical stress at the center of layer *n*
σ'_{vf} = final effective vertical stress at the center of layer *n*

The final effective vertical stress is computed by adding the stress change due to the foundation load to the initial vertical effective stress. The change in the vertical effective stress is computed using the methods discussed in Section 5.3.2. The total settlement will be the sum of the compression in each of the *n* layers of soil.

Normally, the slope, C_c, of the virgin portion of the e-log σ'_v curve is determined from the corrected one-dimensional consolidation curve measured on specimens taken from each relevant soil in the stratigraphic column.

Sometimes the consolidation data are presented in terms of vertical strain (ε_v) instead of void ratio (Figure 5-15). In this case, the slope of the virgin portion of the ε_v versus log σ_v curve is denoted as $C_{c\varepsilon}$, and the settlement is computed using Equation 5-21 for normally consolidated soils.

$$S_c = \sum_{1}^{n} H_o C_{c\varepsilon} \log_{10}(\sigma'_{vf} / \sigma'_{vo}) \tag{5-21}$$

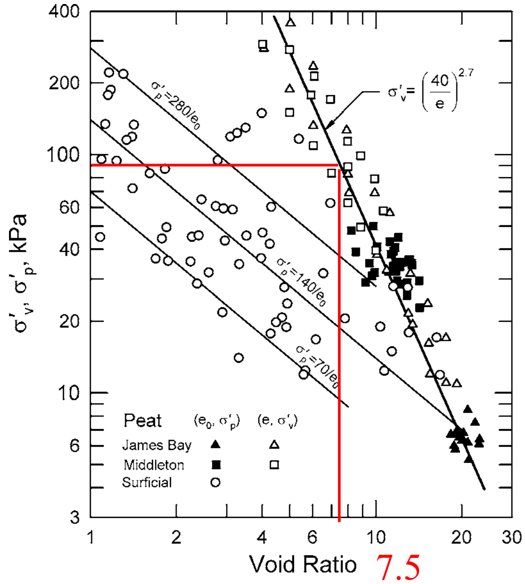


Fig. 11. Relationship between preconsolidation pressure and in situ void ratio e_o for peat deposits (Kogure and Ohira 1977) as well as σ'_v versus e relationship resulting from compression of Middleton and James Bay peats

oped an empirical correlation between σ'_p and in situ void ratio e_o for surficial peat deposits. A correlation between σ'_p and e_o for peats based on compression data of Middleton peat and James Bay peat, together with the Kogure and Ohira (1977) data, is shown in Fig. 11. Note that (e,σ'_v) data from the EOP compression range have been used to extrapolate the (e_o, σ'_p) data of Middleton and James Bay peat.

Therefore, an appropriate equation for EOP settlement of fibrous peats is

$$S = \frac{C_c}{1+e_o}L_o\left[\frac{C_r}{C_c}\log\frac{\sigma'_p}{\sigma'_{vo}} + \log\frac{\sigma'_{vf}}{\sigma'_p}\right] \tag{5}$$

The values of C_r/C_c for fibrous peats are in the range of 0.1 to 0.3. In the absence of undisturbed sampling and oedometer testing, σ'_p may be estimated from an empirical correlation such as those in Fig. 11 or correlations with undrained shear strength described subsequently.

The progress of primary consolidation and associated settlement are determined using a theory of consolidation [e.g., Terzaghi (1923), Mesri et al. (1994a)]. Reliable observations of 1D consolidation in the laboratory and in the field suggest that the time required for primary consolidation of peat is directly proportional to the square of the maximum drainage distance (Hanrahan 1954; Lake 1960; Lea and Brawner 1959, 1963; MacFarlane 1969; Samson and La Rochelle 1972; Hanrahan and Rogers 1981). Computed values for the exponent lower than 2.0 (e.g., 1.5 and 1.6) have been attributed to the contribution of horizontal drainage in the field (Lea and Brawner 1963; Samson and La Rochelle 1972).

Primary consolidation can be speeded up by the use of vertical drains (Mesri and Lo 1991). Because initial permeability of surficial fibrous peat deposits is very high, vertical drains are unnecessary for typical highway embankments and construction schedules, provided that a pervious blanket provides adequate surface drainage. For construction of high embankments in stages, permeability and coefficient of consolidation of peat decrease sufficiently to make the use of vertical drains economical

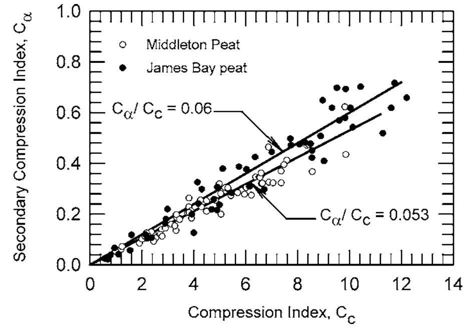


Fig. 12. Secondary compression index, C_α, versus compression index, C_c, for Middleton and James Bay peats

in speeding up excess pore-water pressure dissipation and compression, especially when there is an underlying soft clay layer that is also penetrated by the drains (Hansbo 1982). However, the very large compression commonly associated with fibrous peat layers may lead to crimping of prefabricated wick drains. The reinforcing action of the sand drains may improve the stability of the peat layer (Weber 1969). Because of the high initial horizontal permeability, k_{ho}, of fibrous peat deposits, the vertical drain type must possess high discharge capacity to operate without well resistance (Mesri and Lo 1991).

Secondary Compression

Secondary compression behavior of soils is completely explained and predicted by the C_α/C_c law of compressibility (Mesri and Godlewski 1977, 1979; Mesri and Castro 1987; Mesri 1987; Mesri et al. 1994a, 1997). For any soil a unique interrelationship exists between $C_\alpha=\Delta e/\Delta \log t$ and $C_c=\Delta e/\Delta \log \sigma'_v$ throughout the secondary consolidation stage, and for all pressures in the recompression and compression range.

The values of C_α/C_c for all geotechnical materials are in the range of 0.01 to 0.07. An important aspect of C_α/C_c law of compressibility is the very narrow range of values of C_α/C_c for all geotechnical materials considered together. The magnitude of C_α/C_c appears to depend on the compressibility and deformability of the soil particles. Because peat particles, consisting of a system of cells and baffle walls, are highly compressible, fibrous peat deposits display the highest values of C_α/C_c, whereas the granular materials display the lowest values. Examples of C_α versus C_c are shown in Fig. 12. A significant body of published data indicates values of $C_\alpha/C_c=0.06\pm0.01$ for peat deposits [e.g., Samson and La Rochelle (1972); Berry and Vickers (1975) Lefebvre et al. (1984); Matsuo et al. (1986); Mesri (1986); Mesri et al. (1997)]. Therefore, the *fourth* distinctive characteristic of fibrous peats is that they display the highest values of C_α/C_c among geotechnical materials.

For the following three reasons, secondary compression and associated settlement are often more significant for peat deposits than other geotechnical materials: (1) fibrous peat deposits display the highest values of C_c; (2) fibrous peat deposits have the highest values of C_α/C_c; and (3) primary consolidation of fibrous peat layers in the field is completed commonly within a few weeks or months. Examples of secondary compression of fibrous peats are shown in Fig. 13. The Middleton peat specimen in Fig. 13(a) is loaded to a consolidation pressure near the preconsolidation pressure σ'_p. The coefficient of consolidation in the recompression range is large, and primary consolidation of the 20 mm

The magnitude of the secondary consolidation can be calculated as

Das, Principles of Foundation Eng.7th Ed.

$$S_{c(s)} = C'_\alpha H_c \log(t_2/t_1) \tag{5.92}$$

where

$$C'_\alpha = C_\alpha/(1 + e_p) \tag{5.93}$$

e_p = void ratio at the end of primary consolidation
H_c = thickness of clay layer

Mesri (1973) correlated C'_α with the natural moisture content (w) of several soils, from which it appears that

$$C'_\alpha \approx 0.0001w \tag{5.94}$$

where w = natural moisture content, in percent. For most overconsolidated soils, C'_α varies between 0.0005 to 0.001.

Mesri and Godlewski (1977) compiled the magnitude of C_α/C_c (C_c = compression index) for a number of soils. Based on their compilation, it can be summarized that

- For inorganic clays and silts:

$$C_\alpha/C_c \approx 0.04 \pm 0.01$$

- For organic clays and silts:

$$C_\alpha/C_c \approx 0.05 \pm 0.01$$

- For peats:

$$C_\alpha/C_c \approx 0.075 \pm 0.01$$

No change over range of values, use 0.065

Secondary consolidation settlement is more important in the case of all organic and highly compressible inorganic soils. In overconsolidated inorganic clays, the secondary compression index is very small and of less practical significance.

There are several factors that might affect the magnitude of secondary consolidation, some of which are not yet very clearly understood (Mesri, 1973). The ratio of secondary to primary compression for a given thickness of soil layer is dependent on the ratio of the stress increment, $\Delta\sigma'$, to the initial effective overburden stress, σ'_o. For small $\Delta\sigma'/\sigma'_o$ ratios, the secondary-to-primary compression ratio is larger.

5.18 Field Load Test

The ultimate load-bearing capacity of a foundation, as well as the allowable bearing capacity based on tolerable settlement considerations, can be effectively determined from the field load test, generally referred to as the *plate load test*. The plates that are used for tests in the field are usually made of steel and are 25 mm (1 in.) thick and 150 mm to 762 mm (6 in. to 30 in.) in diameter. Occasionally, square plates that are 305 mm × 305 mm (12 in. × 12 in.) are also used.

To conduct a plate load test, a hole is excavated with a minimum diameter of $4B$ (B is the diameter of the test plate) to a depth of D_f, the depth of the proposed foundation. The plate is placed at the center of the hole, and a load that is about one-fourth to one-fifth of the estimated

Settle3D Analysis Information
18955.00 Rangeley Hunter Cove Br 2384

Project Settings

Document Name	2017.10.19 Embankment rectangle_1.5fill.s3z
Project Title	18955.00 Rangeley Hunter Cove Br 2384
Author	Brandon Slaven
Company	MaineDOT
Date Created	10/19/2017, 12:51:37 PM
Stress Computation Method	Boussinesq
Use average properties to calculate layered stresses	
Improve consolidation accuracy	
Ignore negative effective stresses in settlement calculations	

Results

Time taken to compute: 0.756403 seconds

Stage: Stage 1

Data Type	Minimum	Maximum
Total Settlement [in]	0	1.85479
Consolidation Settlement [in]	0	0.39727
Immediate Settlement [in]	0	1.45752
Loading Stress [ksf]	0.0458037	0.203
Effective Stress [ksf]	0.05075	2.14032
Total Stress [ksf]	0.05075	3.01392
Total Strain	7.86398e-005	0.0389523
Pore Water Pressure [ksf]	0	0.8736
Degree of Consolidation [%]	0	100
Pre-consolidation Stress [ksf]	0.051758	2.1382
Over-consolidation Ratio	1	1
Void Ratio	0	7.39976
Hydroconsolidation Settlement [in]	0	0
Undrained Shear Strength	0	0.407147

Loads

1. Rectangular Load: "1.5 feet New Fill"

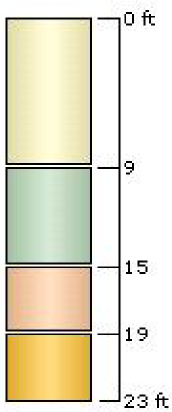
Length	50 ft
Width	35 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	1750 ft^2
Load	0.203 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
-30	7.5
20	7.5
20	42.5
-30	42.5

Soil Layers

Layer #	Type	Thickness [ft]	Depth [ft]
1	2: Medium Dense, SAND	9	0
2	3: Medium dense, Gravelly SAND	6	9
3	4: Dense, Gravelly SAND	4	15
4	5: Peat	4	19



Soil Properties

Property	2: Medium Dense, SAND	3: Medium dense, Gravelly SAND	4: Dense, Gravelly SAND	5: Peat
Color				
Unit Weight [kips/ft^3]	0.112	0.112	0.116	0.065
Saturated Unit Weight [kips/ft^3]	0.138	0.138	0.142	0.12
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	313	330	606	5
Esur [ksf]	1253	1320	2423	19
Primary Consolidation	Disabled	Disabled	Disabled	Enabled
Material Type				Non-Linear
Cc				2.28
Cr				0.1
e0				7.5
OCR	1	1	1	1
Secondary Consolidation	Disabled	Disabled	Disabled	Mesri
Ca/Cc				0.065
Undrained Su A [kips/ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8
Piezo Line ID	1	1	1	1

Groundwater

Groundwater method	Piezometric Lines
Water Unit Weight	0.0624 kips/ft^3

Piezometric Line Entities

ID	Depth (ft)
1	9 ft

Query Lines

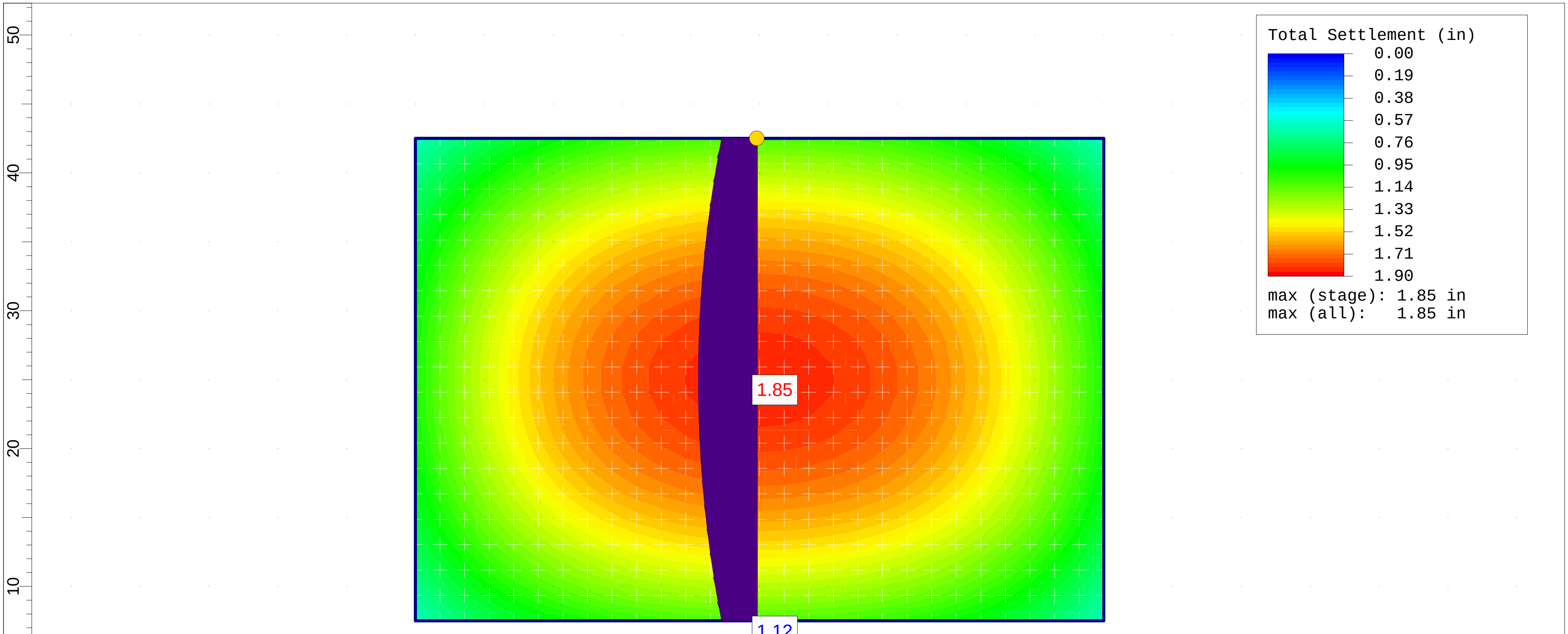
Line #	Start Location	End Location	Horizontal Divisions	Vertical Divisions
1	-5.11734, 42.5	-5.11734, 7.5	20	Auto: 57

Field Point Grid

Number of points	580
Expansion Factor	2.5

Grid Coordinates

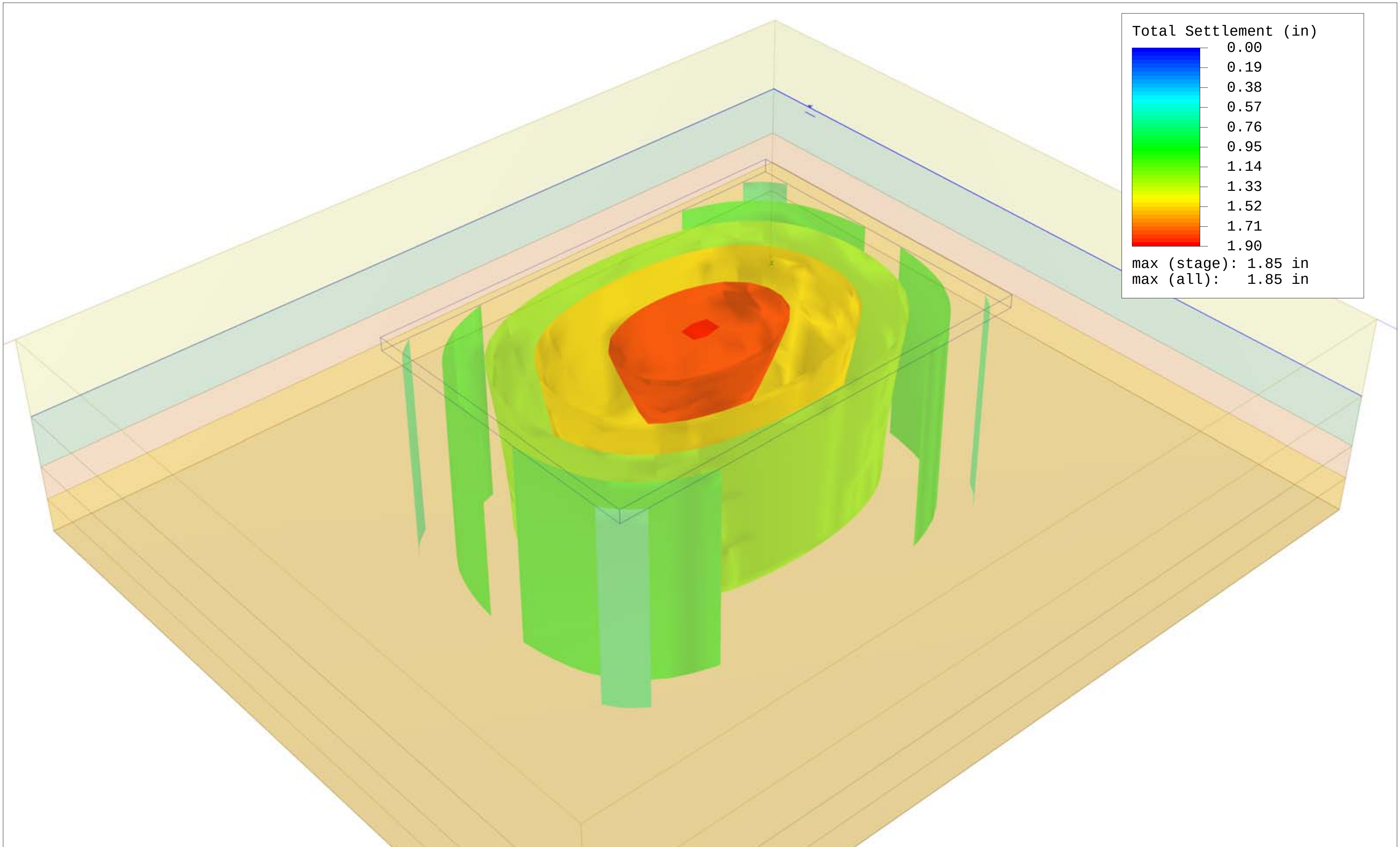
X [ft]	Y [ft]
45	67.5
45	-17.5
-55	-17.5
-55	67.5



Material Name	Color	Unit Weight (kips/ft3)	Sat. Unit Weight (kips/ft3)	Es (ksf)	Eur (ksf)	Material Type	Cc	Cr	OCR	e0	Secondary Consol. Method	Ca/Cc	Mesri Material Type
2: Medium Dense, SAND		0.112	0.138	313	1253								
3: Medium dense, Gravelly SAND		0.112	0.138	330	1320								
4: Dense, Gravelly SAND		0.116	0.142	606	2423								
5: Peat		0.065	0.12	5	19	Non-Linear	2.28	0.1	1	7.5	Mesri	0.065	Middleton Peat

Project	18955.00 Rangeley Hunter Cove Br 2384
Analysis Description	
Drawn By	Brandon Slaven
Company	MaineDOT
Date	10/19/2017, 12:51:37 PM
File Name	2017.10.19 Embankment rectangle_1.5fill.s3z

SETTLE3D 3.020



rocscience	Project	18955.00 Rangeley Hunter Cove Br 2384	
	Analysis Description		
	Drawn By: Brandon Slaven	Company: MaineDOT	
SETTLE3D 3.020	Date: 10/19/2017, 12:51:37 PM	File Name: 2017.10.19 Embankment rectangle_1.5fill.s3z	

Settle3D Analysis Information
18955.00 Rangeley Hunter Cove Br 2384

Project Settings

Document Name	2017.10.19 Embankment Settlement_1.5fill.s3z
Project Title	18955.00 Rangeley Hunter Cove Br 2384
Author	Brandon Slaven
Company	MaineDOT
Date Created	10/19/2017, 12:51:37 PM
Stress Computation Method	Boussinesq
Use average properties to calculate layered stresses	
Improve consolidation accuracy	
Ignore negative effective stresses in settlement calculations	

Stage Settings

Stage #	Name
1	Stage 1

Results

Time taken to compute: 2.85012 seconds

Stage: Stage 1

Data Type	Minimum	Maximum
Total Settlement [in]	0	1.79057
Consolidation Settlement [in]	0	0.38327
Immediate Settlement [in]	0	1.4073
Loading Stress [ksf]	0	0.202608
Effective Stress [ksf]	0	2.13514
Total Stress [ksf]	0	3.00874
Total Strain	3.52457e-007	0.0376688
Pore Water Pressure [ksf]	0	0.8736
Degree of Consolidation [%]	0	100
Pre-consolidation Stress [ksf]	0.00111832	2.13302
Over-consolidation Ratio	1	1
Void Ratio	0	7.40851
Hydroconsolidation Settlement [in]	0	0
Undrained Shear Strength	0	0.40695

Embankments

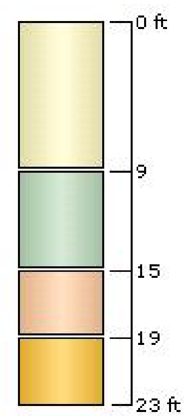
1. Embankment

Center Line	(-5, 0) to (-5, 50)
Number of Layers	1
Near End Angle	90 degrees
Far End Angle	90 degrees
Base Width	35.25

Layer	Stage	Left Bench Width (ft)	Left Angle (deg)	Height (ft)	Unit Weight (kips/ft^3)	Right Angle (deg)	Right Bench Width (ft)
1	Stage 1	0	29.7	1.5	0.135	29.7	0

Soil Layers

Layer #	Type	Thickness [ft]	Depth [ft]
1	2: Medium Dense, SAND	9	0
2	3: Medium dense, Gravelly SAND	6	9
3	4: Dense, Gravelly SAND	4	15
4	5: Peat	4	19



Soil Properties

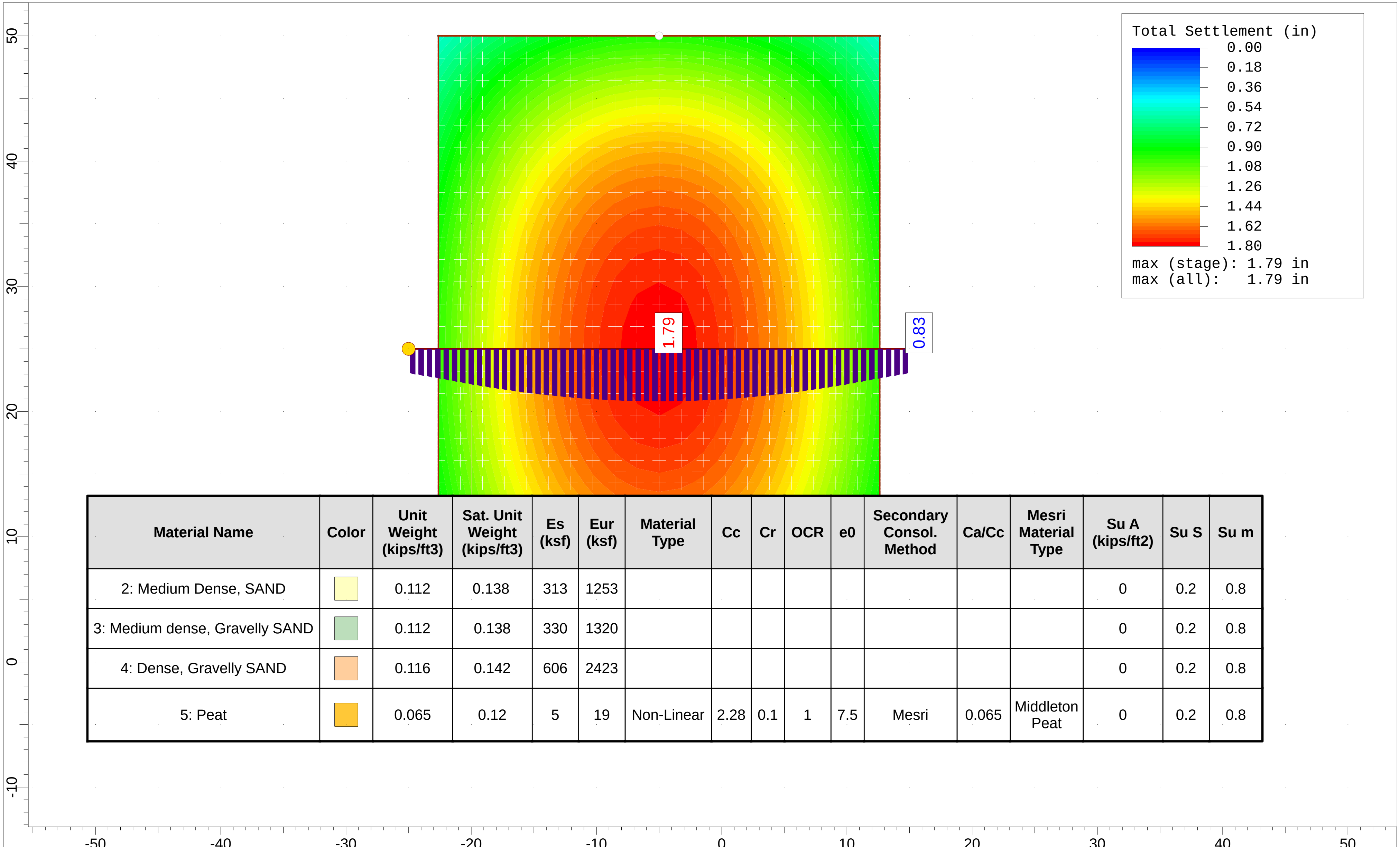
Property	2: Medium Dense, SAND	3: Medium dense, Gravelly SAND	4: Dense, Gravelly SAND	5: Peat
Color				
Unit Weight [kips/ft^3]	0.112	0.112	0.116	0.065
Saturated Unit Weight [kips/ft^3]	0.138	0.138	0.142	0.12
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	313	330	606	5
Esur [ksf]	1253	1320	2423	19
Primary Consolidation	Disabled	Disabled	Disabled	Enabled
Material Type				Non-Linear
Cc				2.28
Cr				0.1
e0				7.5
OCR	1	1	1	1
Secondary Consolidation	Disabled	Disabled	Disabled	Mesri
Ca/Cc				0.065
Undrained Su A [kips/ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8
Piezo Line ID	1	1	1	1

Groundwater

Groundwater method	Piezometric Lines
Water Unit Weight	0.0624 kips/ft^3

Piezometric Line Entities

ID	Depth (ft)
1	9 ft



Material Name	Color	Unit Weight (kips/ft3)	Sat. Unit Weight (kips/ft3)	Es (ksf)	Eur (ksf)	Material Type	Cc	Cr	OCR	e0	Secondary Consol. Method	Ca/Cc	Mesri Material Type	Su A (kips/ft2)	Su S	Su m
2: Medium Dense, SAND		0.112	0.138	313	1253									0	0.2	0.8
3: Medium dense, Gravelly SAND		0.112	0.138	330	1320									0	0.2	0.8
4: Dense, Gravelly SAND		0.116	0.142	606	2423									0	0.2	0.8
5: Peat		0.065	0.12	5	19	Non-Linear	2.28	0.1	1	7.5	Mesri	0.065	Middleton Peat	0	0.2	0.8

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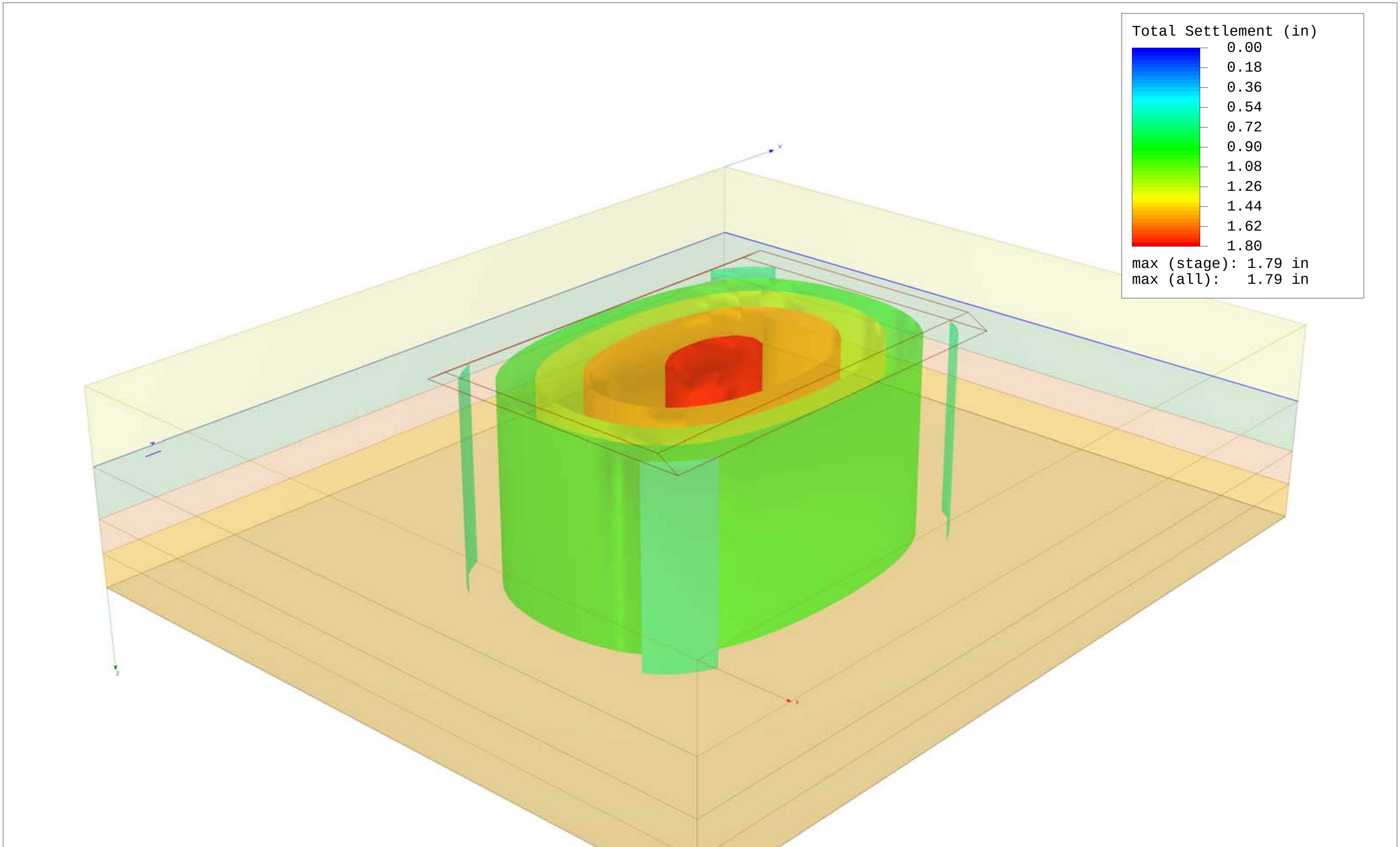
Project	18955.00 Rangeley Hunter Cove Br 2384
Analysis Description	
Drawn By	Brandon Slaven
Company	MaineDOT
Date	10/19/2017, 12:51:37 PM
File Name	2017.10.19 Embankment Settlement_1.5fill.s3z



Project	18955.00 Rangeley Hunter Cove Br 2384
Analysis Description	

Drawn By	Brandon Slaven	*Company*	MaineDOT
Date	10/19/2017, 12:51:37 PM	*File Name*	2017.10.19 Embankment Settlement_1.5fill.s3z (Recovered)

SETTLE3D 3.020

Frost Depth

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

From Design Freezing Index Map: **Rangeley, Maine**
DFI = 2200 degree-days.
Case 1 - coarse grained granular fill soils W=20% (assumed).

For DFI = 2200 $d_1 := 82.6\text{in}$

Depth of Frost Penetration $d_1 = 83 \cdot \text{in}$ $d_1 = 6.9 \cdot \text{ft}$

Method 2 - ModBerg Software

Examine foundations placed on coarse grained fill soils

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

--- ModBerg Results ---

Project Location: Houlton 5 N, Maine
Air Design Freezing Index = 2189 F-days
N-Factor = 0.80
Surface Design Freezing Index = 1751 F-days
Mean Annual Temperature = 40.8 deg F
Design Length of Freezing Season = 148 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	99.4	20.0	125.0	34	46	3.8	1.9	3,600

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

Total Depth of Frost Penetration = 8.28 ft = 99.4 in.

Recommendation: 6.9 feet for design of foundations constructed on coarse grained soils

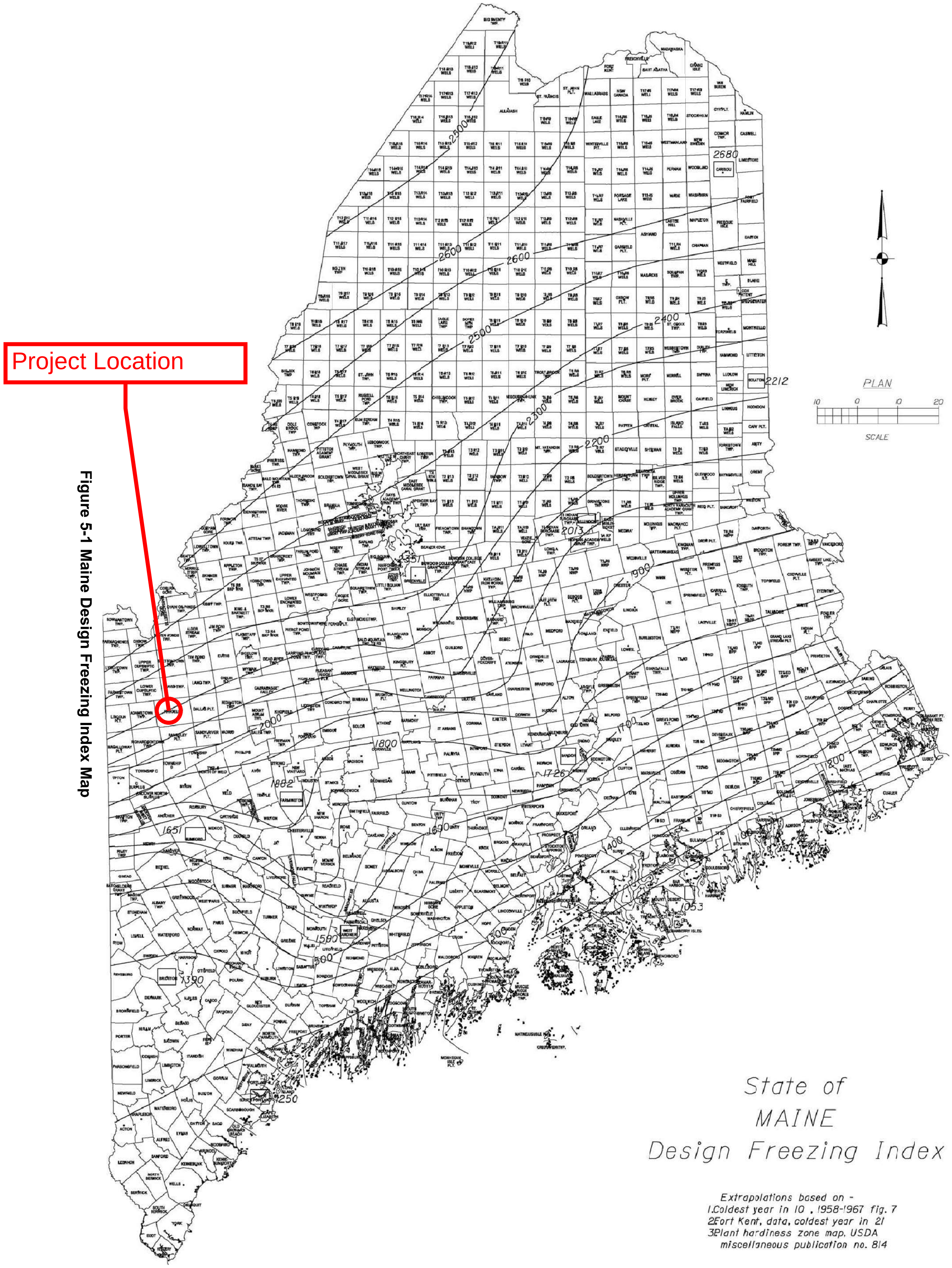


Figure 5-1 Maine Design Freezing Index Map

5.2 General

5.2.1 Frost

Any foundation placed on seasonally frozen soils must be embedded below the depth of frost penetration to provide adequate frost protection and to minimize the potential for freeze/thaw movements. Fine-grained soils with low cohesion tend to be most frost susceptible. Soils containing a high percentage of particles smaller than the No. 200 sieve also tend to promote frost penetration.

In order to estimate the depth of frost penetration at a site, Table 5-1 has been developed using the Modified Berggren equation and Figure 5-1 Maine Design Freezing Index Map. The use of Table 5-1 assumes site specific, uniform soil conditions where the Geotechnical Designer has evaluated subsurface conditions. Coarse-grained soils are defined as soils with sand as the major constituent. Fine-grained soils are those having silt and/or clay as the major constituent. If the make-up of the soil is not easily discerned, consult the Geotechnical Designer for assistance. In the event that specific site soil conditions vary, the depth of frost penetration should be calculated by the Geotechnical Designer.

Table 5-1 Depth of Frost Penetration

Design Freezing Index	Frost Penetration (in)					
	Coarse Grained			Fine Grained		
	w=10%	w=20%	w=30%	w=10%	w=20%	w=30%
1000	66.3	55.0	47.5	47.1	40.7	36.9
1100	69.8	57.8	49.8	49.6	42.7	38.7
1200	73.1	60.4	52.0	51.9	44.7	40.5
1300	76.3	63.0	54.3	54.2	46.6	42.2
1400	79.2	65.5	56.4	56.3	48.5	43.9
1500	82.1	67.9	58.4	58.3	50.2	45.4
1600	84.8	70.2	60.3	60.2	51.9	46.9
1700	87.5	72.4	62.2	62.2	53.5	48.4
1800	90.1	74.5	64.0	64.0	55.1	49.8
1900	92.6	76.6	65.7	65.8	56.7	51.1
2000	95.1	78.7	67.5	67.6	58.2	52.5
2100	97.6	80.7	69.2	69.3	59.7	53.8
2200	100.0	82.6	70.8	71.0	61.1	55.1
2300	102.3	84.5	72.4	72.7	62.5	56.4
2400	104.6	86.4	74.0	74.3	63.9	57.6
2500	106.9	88.2	75.6	75.9	65.2	58.8
2600	109.1	89.9	77.1	77.5	66.5	60.0

Seismic Design

BB-RHC-101				
Depth	**SPT N_{60}**		**di**	**di/N**
2.5	26		2.5	0.10
5	15		2.5	0.17
10	17		5	0.29
15	39		5	0.13
20	6		5	0.83
25	14		5	0.36
30	11		5	0.45
35	11		5	0.45
40	5		5	1.00
45	67		5	0.07
50	100		5	0.05
55	100		5	0.05
60	100		5	0.05
65	100		5	0.05
100	100	**Bedrock**	35	0.35
SUM			**100**	**4.41**

di/di/N **22.68**

BB-RHC-102				
Depth	**SPT N_{60}**		**di**	**di/N**
5	20		5	0.25
10	18		5	0.28
15	53		5	0.09
20	6		5	0.83
25	20		5	0.25
30	9		5	0.56
35	62		5	0.08
40	58		5	0.09
45	58		5	0.09
48	100		3	0.03
70	100		22	0.22
100	100	**Bedrock**	30	0.30
			100	**3.06**

di/di/N **32.64**

SUM	**N_{av}.**	**27.66**

Conclusion: Site Class D

15< N_{av}. <50

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 = 44.96186
Longitude = -070.714420
Site Class B
Data are based on a 0.05 deg grid spacing.

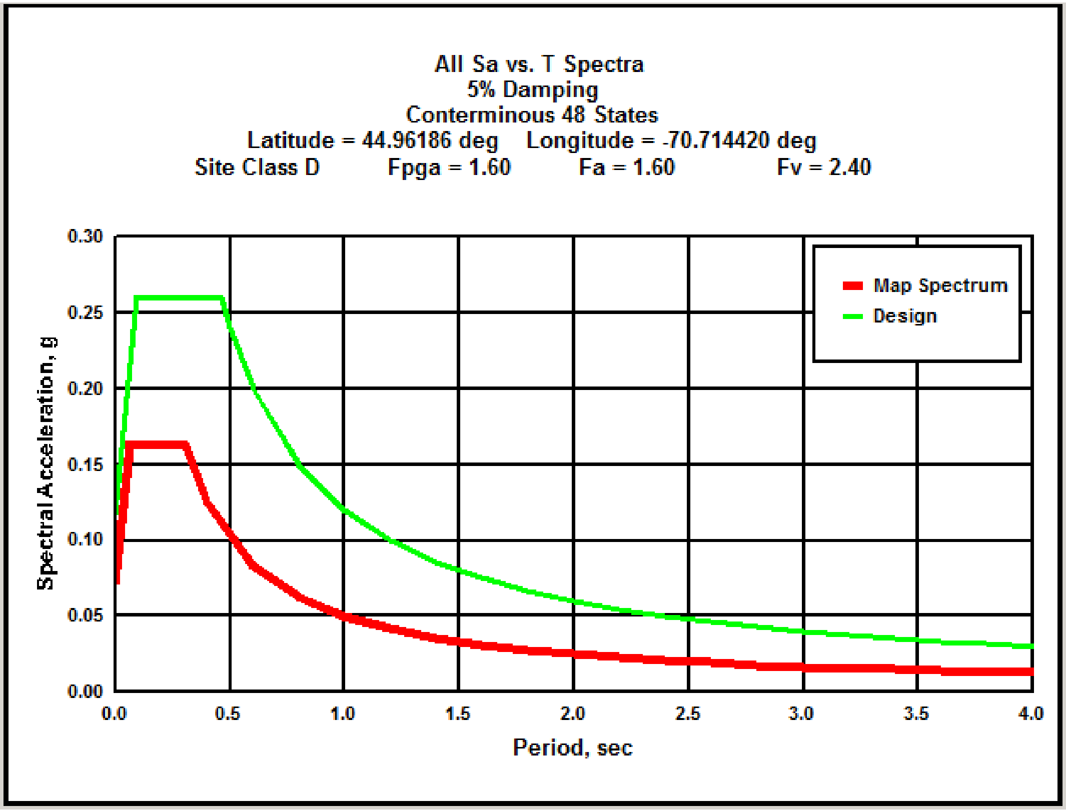
Period (sec)	Sa (g)	
0.0	0.074	PGA - Site Class B
0.2	0.163	Ss - Site Class B
1.0	0.050	S1 - Site Class B

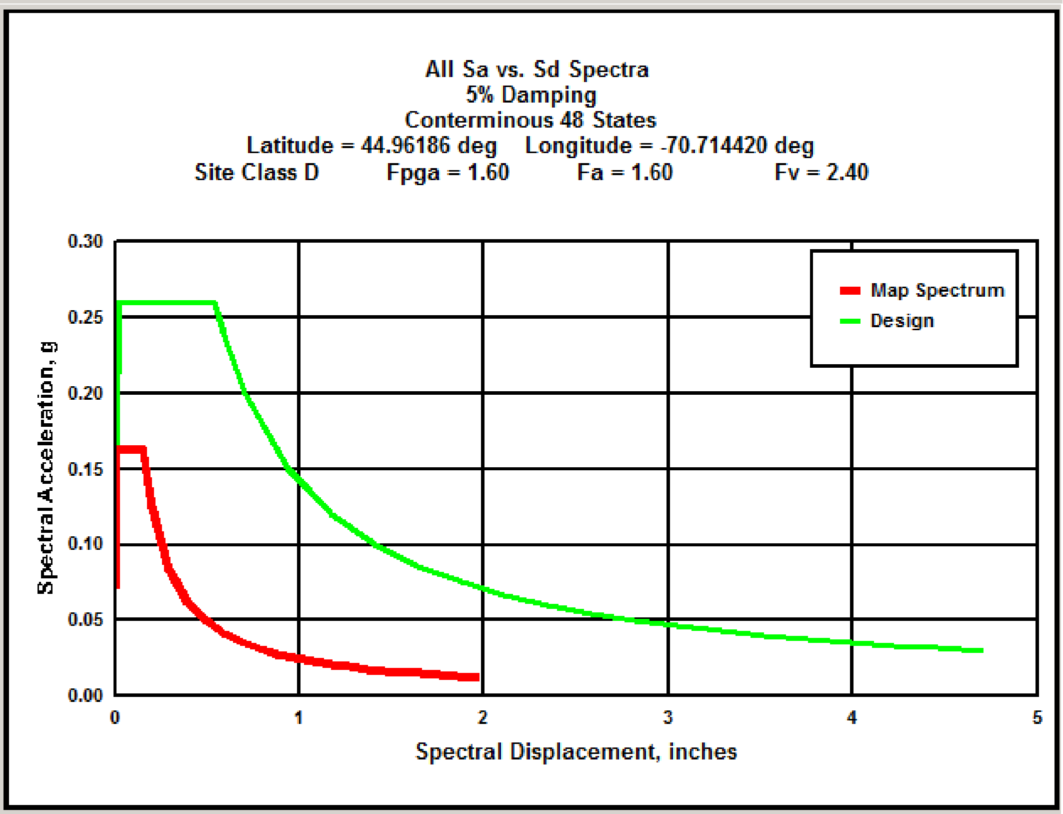
Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 44.96186
Longitude = -070.714420
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.118	As - Site Class D
0.2	0.260	SDs - Site Class D
1.0	0.120	SD1 - Site Class D

Conterminous 48 States
2007 AASHTO Bridge Design
Guidelines Design Response Spectra for
Site Class D Latitude = 44.961860
Longitude = -070.714420
As = FpgaPGA, SDs = FaSs, 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)	Sd in.	
0.000	0.118	0.000	T = 0.0, Sa = As
0.092	0.260	0.022	
0.200	0.260	0.102	T = 0.2, Sa = SDs
0.462	0.260	0.543	T = Ts, Sa = SDs
0.500	0.240	0.587	
0.600	0.200	0.705	
0.800	0.150	0.939	
1.000	0.120	1.174	T = 1.0, Sa = SD1
1.200	0.100	1.409	
1.400	0.086	1.644	
1.600	0.075	1.879	
1.800	0.067	2.114	
2.000	0.060	2.349	
2.200	0.055	2.584	


All Sa vs. T Spectra
5% Damping
Conterminous 48 States
Latitude = 44.96186 deg Longitude = -70.714420 deg
Site Class D Fpga = 1.60 Fa = 1.60 Fv = 2.40
Map Spectrum
Design
Spectral Acceleration, g
0.30
0.25
0.20
0.15
0.10
0.05
0.00
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
Period, sec


All Sa vs. Sd Spectra
5% Damping
Conterminous 48 States
Latitude = 44.96186 deg Longitude = -70.714420 deg
Site Class D Fpga = 1.60 Fa = 1.60 Fv = 2.40
Map Spectrum
Design
Spectral Acceleration, g
0.30
0.25
0.20
0.15
0.10
0.05
0.00
0
1
2
3
4
5
Spectral Displacement, inches

Appendix D

Existing Structure Pile Lengths

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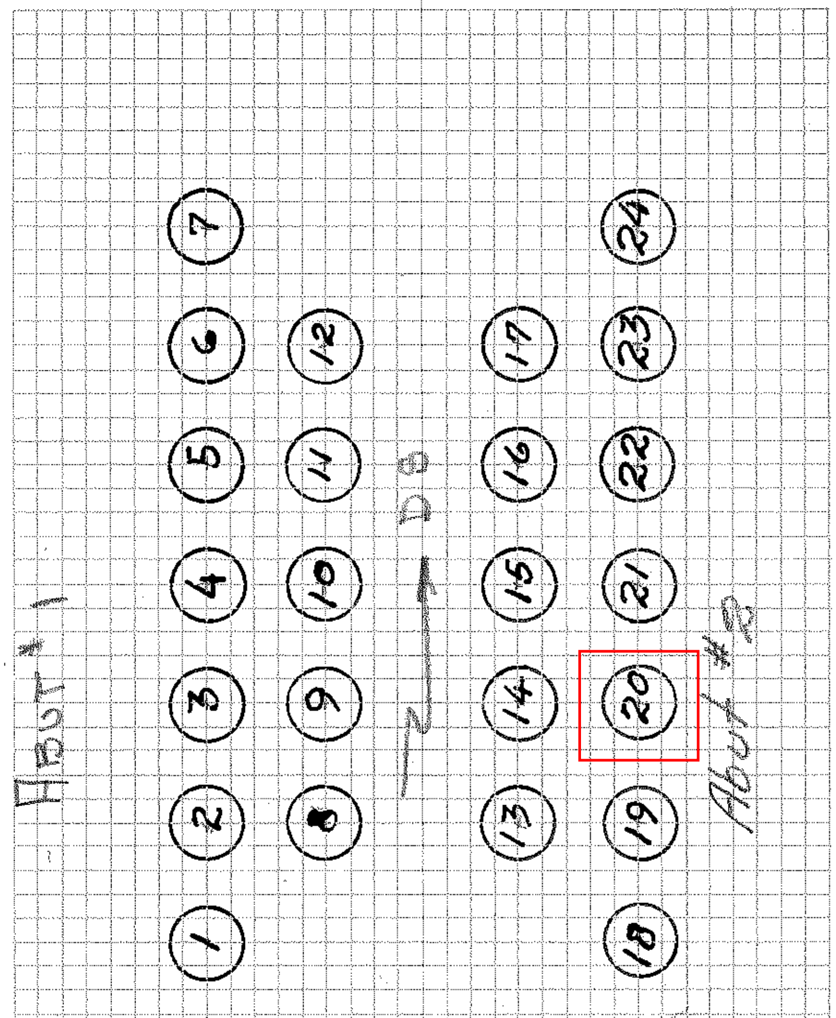
TREATED PILING ABUT# 1 & 2

ITEM 83

Number	Head Length	Cut off	Driven Length		
1	50'-0"	8'-0 1/2"	41'-11 1/2"	✓	
2	50'-4"	8'-0"	42'-4"	✓	
3	50'-6"	10'-4 1/2"	40'-1 1/2"	✓	
4	50'-2"	10'-1 1/2"	40'-0 1/2"	✓	
5	48'-6"	8'-2 3/4"	40'-3 1/4"	✓	
6	50'-3"	7'-4 1/4"	42'-10 1/4"	✓	
7	50'-1"	5'-7 1/2"	44'-5 1/2"	✓	
8	48'-3"	9'-5"	38'-10"	✓	
9	48'-5"	9'-11"	38'-6"	✓	
10	48'-4"	8'-1"	40'-3"	✓	
11	48'-7"	5'-7"	43'-0"	✓	
12	48'-5"	7'-4"	41'-1"	✓	
13	48'-7"	8'-9"	39'-10"	✓	
14	48'-9"	4'-0	44'-9"	✓	
15	48'-7"	6'-4"	42'-3"	✓	
16	50'-2"	7'-5"	42'-9"	✓	
17	48'-2"	6'-7"	41'-7"	✓	
18	50'-6"	2'-8"	47'-10"	✓	
19	50'-2"	5'-0"	48'-2"	✓	
20	50'-6"	32'-8"	17'-10"	✓	Could not penetrate old log corderoy and rocks.
21	50'-7"	4'-7"	46'-0"	✓	
22	50'-6"	8'-1"	42'-5"	✓	
23	50'-5"	8'-6"	41'-11"	✓	
24	50'-4"	8'-6"	41'-10"	✓	
TOTAL	1189'-1"	201'-2"	987'-11"		



Total length driven = 988' ✓

Total length cut-offs over 3' = 199' ✓

988' @ $1.07 = $1,057.40 ✓

Test Piles 89' @ 1.05 = 93.45

1077'

$1,130.85 ✓