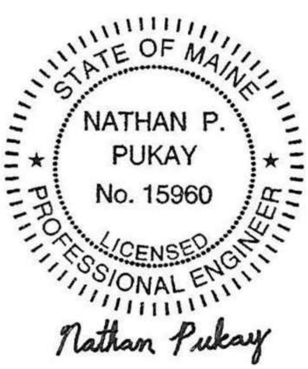
MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

MOOSEHORN BRIDGE
STATION ROAD OVER MOOSEHORN BROOK
CHARLOTTE, MAINE



Prepared by:

Nathan Pukay, P.E.
Transportation Engineer II

Reviewed by:

Laura Krusinski, P.E.
Senior Geotechnical Engineer

Washington County
WIN 21686.10

Soils Report 2025-17
Bridge No. 3332

Federal Project No. 2168610
April 25, 2025

Table of Contents

Sheets

Appendices

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and provide geotechnical design recommendations for the replacement of Moosehorn Bridge which carries Station Road over Moosehorn Brook in Charlotte, Maine. This report presents the subsurface information obtained at the site during the subsurface investigations, geotechnical design recommendations, and construction recommendations for the new substructures.

The existing Moosehorn Bridge was constructed in 1952 and consists of a pair of 13-foot diameter steel pipe culverts. According to the 2022 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the culverts are in poor condition, with heavy pitting and holes. Both culverts have begun unzipping, show signs of distortion, and the west culvert has a 5-foot-long hole with backfill spilling through.

Available as-built drawings indicate a previous structure at the bridge consisted of a timber bridge comprised of rock-filled timber cribbing and four timber pile bent piers.

The proposed replacement structure consists of a 74-foot, single-span, precast concrete Northeast Extreme Tee (NEXT) F-beam bridge founded on pile-supported integral abutments with cantilevered, in-line wingwalls. Piles will be driven to bedrock. 1.75H:1V (horizontal:vertical) riprap slopes will be constructed in front of the new integral abutments. The new bridge will be located on a horizontal alignment that will approximately match the existing. To increase the freeboard of the superstructure, the vertical alignment will be raised approximately 12 inches at Abutment No. 1 and approximately 19 inches at Abutment No. 2.

Traffic will be maintained with an off-site detour using State and local roads.

2.0 GEOLOGIC SETTING

Moosehorn Bridge carries Station Road over Moosehorn Brook as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geologic Map of Maine (1985) indicates the surficial soils in the vicinity of the bridge project consist of glaciomarine deposits with both fine-grained and coarse-grained facies, swamp, marsh, and bog deposits (wetland deposits), and glacial till. The fine-grained glaciomarine deposits consist of silt, clay, sand, and minor amounts of gravel. The coarse-grained glaciomarine deposits consist of sand, gravel, and minor amounts of silt. The wetland deposits consist of peat, muck, clay, silt, and sand. Glacial till is a heterogeneous mixture of sand, silt, clay, and stones deposited by glacial ice.

The MGS Bedrock Geology of the Calais Quandrangle, Maine, Open-File No. 03-97 (2003) maps the bedrock at the site as granite of the Charlotte Pluton.

3.0 SUBSURFACE INVESTIGATION

Five test borings were drilled to explore subsurface conditions at the site. Borings BB-CHAR-101, BB-CHAR-201, and BB-CHAR-201A were drilled at or near the location of proposed Abutment No. 1. Borings BB-CHAR-102 and BB-CHAR-202 were drilled at or near the location of proposed Abutment No. 2. The boring locations are shown on Sheet 2 – Boring Location Plan.

Borings BB-CHAR-101 and BB-CHAR-102 were drilled in June 2017 by New England Boring Contractors (NEBC) under the direction of Golder Associates. Borings BB-CHAR-201 and BB-CHAR-201A were drilled in March 2024 by Seaboard Drilling LLC under the direction of MaineDOT. The remaining boring, BB-CHAR-202, was drilled in March 2024 by the MaineDOT Drill Crew. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheets 4 and 5 – Boring Logs.

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, the 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 drill rig used by NEBC for BB-CHAR-102 performed SPT sampling using a 140-lb safety hammer with a rope and cathead. The drill rigs used for the remaining borings were equipped with automatic hammers to drive the split spoon. The hammers were calibrated per ASTM D 4633 "Standard Test Method for Energy Measurement for Dynamic Penetrometers" to establish hammer efficiency factors. All N-values discussed in this report are corrected N-values computed by applying the hammer efficiency factors. The hammer efficiency factors and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring logs.

Bedrock was cored in three of the borings using NQ-2" core barrels and the Rock Quality Designation (RQD) of the cores calculated. Boring BB-CHAR-102 was terminated within weathered bedrock upon reaching refusal. BB-CHAR-201 was also terminated in weathered bedrock, but due to drilling and tooling difficulties. Geotechnical engineers from Golder Associates (2017) and MaineDOT (2024) selected the boring locations and drilling methods, designated the type and depth of sampling techniques, and identified field-testing requirements. Geotechnical engineers from Golder Associates (2017) and MaineDOT (2024), along with a MaineDOT NETTCP Certified Subsurface Inspector (2024) logged the subsurface conditions encountered in the borings. The borings were located in the field using taped measurements at the completion of the drilling program and then located by MaineDOT Survey.

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 on soil samples consisted of six standard grain size analyses with natural water content, three grain size analyses with hydrometer and natural water content, two Atterberg limits tests, and one test for organic content (loss on ignition).

Soil laboratory testing was performed by GeoTesting Express of Acton, Massachusetts, and the MaineDOT Lab in Bangor, Maine. The results of soil tests are included in Appendix C – Laboratory Test Results. Moisture content information and other soil test results are also presented on the boring logs provided in Appendix A – Boring Logs and on Sheets 4 and 5 – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings generally consisted of Fill, Wetland Deposits, Glaciomarine Deposits, and Glacial Till overlying Bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheets 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

A layer of Fill was encountered in the test borings. The thickness of the Fill unit encountered was approximately 9 to 16 feet. The fill materials encountered consisted of:

- Brown, SAND, trace to some gravel, trace to little silt;
- Brown to blue-grey, Gravelly SAND, trace silt, trace wood fragments;
- Brown, GRAVEL, little sand, little silt; and
- Brown, Gravelly SILT, little sand.

One corrected SPT N-value in the fine-grained Fill unit was 6 blows per foot (bpf) indicating the fine-grained fill is medium stiff in consistency.

Corrected SPT N-values in the coarse-grained Fill unit ranged from 3 to 33 bpf indicating the coarse-grained fill is very loose to dense in consistency.

Three grain size analyses performed on samples recovered from the Fill unit indicated the material is classified as A-1-a under the AASHTO Soil Classification System and SP and SW-SM under the Unified Soil Classification System (USCS). The natural water contents of the samples tested ranged from 4 to 11 percent.

5.2 Wetland Deposit

A Wetland Deposit was encountered in BB-CHAR-202 beneath the fill layer. The encountered thickness was approximately 5 feet. The deposit consisted of:

- Grey-brown SILT and CLAY; and
- Brown, soft, SILT, little peat, little sand.

One corrected SPT N-value within the Wetland Deposit was 3 bpf, indicating the deposit is soft.

A loss on ignition test performed on a sample of the deposit measured an organic content of 18 percent. The natural water content of the tested sample was 113 percent.

5.3 Glaciomarine Deposit

A predominately coarse-grained Glaciomarine Deposit was encountered in the borings underlying the Wetland Deposit in BB-CHAR-202 and the Fill in the remaining borings. The thickness of the Glaciomarine Deposit encountered was approximately 10 to 22 feet. The Glaciomarine Deposits varied from:

- Grey to brown, SAND, trace to some silt, trace to little clay;
- Grey, Silty SAND, trace to little gravel, trace clay;
- Grey, Sandy SILT, trace clay
- Grey, SILT, some clay, little sand
- Grey, Clayey SILT; and
- Grey, Silty CLAY, trace gravel.

Two corrected SPT N-values within the fine-grained subunit were 6 and 11 bpf indicating the fine-grained subunit is medium stiff to stiff in consistency.

Corrected SPT N-values within the coarse-grained Glaciomarine Deposit ranged from 6 to 29 bpf indicating the deposit is loose to medium dense in consistency.

Four grain size analyses performed on samples recovered from the deposit resulted in the material being classified as A-4 under the AASHTO Soil Classification System and SM, ML, and CL under the USCS.

Atterberg limits tests were conducted on two samples of the fine-grained subunit, and are summarized below:

Boring No. and Sample No.	Soil Description	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-CHAR-102, 6D	Silty CLAY	25	31	18	13	0.54
BB-CHAR-202, 4D	SILT, some clay	24	Non-Plastic			

The plasticity index of sample BB-CHAR-102, 6D indicates that the fine-grained subunit is medium in plasticity (Burmister, 1949). The natural water content of the same sample measured 25 percent. With a liquid limit of 31 and a plastic limit of 18, the resulting liquidity index for the sample was less than 1.0, indicating that the deposit is lightly preconsolidated.

5.4 Glacial Till

Glacial Till was encountered in the borings underlying the Glaciomarine Deposits. The thickness of the Glacial Till deposit encountered was approximately 22 to 31 feet. The Glacial Till varied from:

- Grey to dark grey, SILT, little to some sand, trace to some gravel, trace to little clay;
- Grey to dark grey, Silty SAND, trace to some gravel, trace to little clay;
- Grey, Sandy SILT, some gravel;
- Grey, SAND, some silt, trace to little gravel, trace to little clay;
- Grey, GRAVEL, little sand, little silt, trace clay; and
- Cobbles.

Corrected SPT N-value within the fine-grained Glacial Till ranged from 16 to greater than 50 bpf indicating the fine-grained Glacial Till is very stiff to hard in consistency.

Corrected SPT N-values within the coarse-grained Glacial Till ranged from 21 to greater than 50 bpf indicating the deposit is medium dense to very dense in consistency.

Two grain size analysis performed on samples recovered from the deposit resulted in the material being classified as A-1-a and A-4 under the AASHTO Soil Classification System and GM and CL under the USCS. The natural water contents of the samples tested were 4 and 12 percent.

5.5 Bedrock

Bedrock was encountered and cored in three of the borings. The table below summarizes the borings in which bedrock was cored, the depth to bedrock, corresponding top of bedrock elevations and RQD’s.

Boring	Station	Offset (feet)	Approximate Depth to Bedrock (feet)	Approximate Elevation of Bedrock Surface (feet)	RQD (%) (R1, R2, R3, R4)
BB-CHAR-101	4+71.3	7.3 Rt	66.4	16.8	6, 68
BB-CHAR-201A	4+55.9	6.1 Lt	64.0	19.2	7, 24, 55
BB-CHAR-202	5+31.9	9.5 Rt	57.8	24.7	0, 0, 38

Bedrock at the site generally consisted of pink, grey, brown and chalky white, medium to coarse-grained, BIOTITE-HORNBLENDE GRANITE, soft to hard, slightly to severely weathered, with zones decomposed to sand, joints dipping at horizontal to vertical angles, spaced very close to close. The RQD of the bedrock cores ranged from 0 to 68 percent, corresponding to a Rock Quality of very poor to fair.

Detailed bedrock descriptions and RQD's are provided in Appendix A – Boring Logs and on Sheets 4 and 5 – Boring Logs. Rock core photographs are provided in Appendix B – Rock Core Photographs.

Interpreted elevations of severely weathered bedrock have been identified in the Boring Logs and on Sheet 3 – Interpretive Subsurface Profile.

5.6 Groundwater

Groundwater was measured at depths ranging from 6 to 10 feet below the roadway surface during or upon completion of the borings. Note that water was introduced into the boreholes during drilling operations and the measured levels may not represent stabilized groundwater elevations. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, river levels and construction activities.

6.0 FOUNDATION ALTERNATIVES

In 2017, during Golder Associates' preliminary subsurface investigation, the project scope was to replace the existing twin steel culverts with twin butted precast concrete box culverts, each with an 18-foot span and a 14-foot rise. The project was reinitiated in January 2023, with an integral abutment bridge identified as the preferred structure due to cost and ease of construction.

The MaineDOT Preliminary Design Report (PDR), dated April 26, 2024, recommended a detail-build superstructure. However, during final design, MaineDOT removed the detail-build option and instead opted to provide a fully designed NEXT F-beam superstructure. The PDR also recommended a single-lane bridge; however, additional public outreach and input during the final design process identified the need for a two-lane structure.

7.0 GEOTECHNICAL DESIGN CONSIDERATIONS AND RECOMMENDATIONS

The following sections provide geotechnical design considerations and recommendations for H-pile supported integral abutments for the Moosehorn Bridge replacement project.

7.1 Integral Abutment H-Piles

Abutments No. 1 and 2 will be integral abutments founded on a single row of H-piles. Piles will be driven to the required nominal resistance on or within bedrock.

Piles will be HP 14x89, or larger, and shall be 50 ksi, Grade A572 steel. The piles shall be fitted with driving pile points conforming to MaineDOT Standard Specification 711.10 to protect pile tips and improve penetration into bedrock.

Pile lengths at the proposed abutments may be estimated based on the following table.

Abutment	Approximate Bottom Elevation of Proposed Abutment (feet)	Approximate Top of Competent Bedrock Elevation[1] (feet)	Estimated In-Place Pile Lengths[2] (feet)
Abutment No. 1	73.2	9.9	66
Abutment No. 2	73.2	14.5	61

The estimated pile lengths in the table above consider the depth of severely weathered bedrock, but do not take into account damaged pile, the additional five feet of pile required for dynamic testing instrumentation (per ASTM D4945), additional pile length needed to accommodate leads and driving equipment or variations in the bedrock surface.

The design of piles at the strength limit state shall consider;

- compressive axial geotechnical resistance of piles,
- drivability resistance of piles,
- structural resistance of piles in axial compression, and
- structural resistance of 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.

[1] Refer to Appendix A – Boring Logs and Appendix B – Rock Core Photographs.
[2] Estimated pile lengths include 2-foot embedment into the pile cap, (rounded up to foot increments).

Per AASHTO LRFD Bridge Design Specifications 9th Edition (LRFD) Article 6.5.4.2, at the strength limit state, the axial resistance factor $\phi_c = 0.50$ (severe driving conditions) shall be applied to the structural compressive resistance of the pile. Since the H-piles will be subjected to lateral loading, the piles shall also be checked for 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.

7.1.1 Axial Pile Resistance – Strength Limit State

Structural Resistance. Preliminary estimate of the factored structural axial resistance of a HP 14x89 pile section was calculated for the lower braced pile segment in pure axial compression. The factored structural axial resistance shown in the table below is for the lower braced pile segment, using a resistance factor, $\phi_c = 0.50$, for severe driving conditions. It is the responsibility of the structural engineer to calculate the factored axial structural compressive resistance based on the lengths of the upper and lower unbraced pile segments, as determined from LPile, using a resistance factor of $\phi_c = 0.70$ for combined axial and bending and appropriate effective length factors (K). This resistance may be the controlling value.

Geotechnical Resistance. The nominal axial geotechnical resistance of driven piles at 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 nominal 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 axial compressive resistances are provided in the table below.

Drivability Analyses. Drivability analyses were performed for HP 14x89 and HP 14x117 pile sections to determine the pile resistance that might be achieved considering available diesel hammers. LRFD 10.7.8 limits driving stresses to $0.90f_y$, which for 50 ksi steel piles is 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 driven HP 14x89 piles at the strength limit states are summarized in the following table. Drivability resistances for the HP 14x117 pile section are provided in a GRLWEAP summary table in Appendix D – Calculations.

Strength Limit State Factored Axial Pile Resistance					
Pile Section	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[6] (kips)
HP 14 x 89	652	652	436[4]	474[5]	436[4]

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

The maximum applied factored axial pile load should not exceed the governing factored axial pile resistance shown in the previous table.

---

[1] Structural resistances were calculated for a braced pile segment in pure axial compression, using a resistance factor, ϕ_c, for severe driving conditions. Factored structural resistances should be calculated for upper and lower unbraced pile segments based upon L-Pile results using a resistance factor of $\phi_c = 0.70$ for combined axial loading and bending. These resistances may be the controlling values.

[2] Geotechnical axial pile resistance evaluations assumed piles penetrate weathered bedrock and terminate on, or in, competent bedrock. Based on guidance in LRFD Article 10.7.3.2.3., *Piles Driven to Hard Rock.* 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 nominal structural resistance values obtained from LRFD Article 6.9.4.1 with a resistance factor ϕ_c, of 0.50, for severe driving conditions applied when computing the factored resistance.

[3] Drivability analyses were performed to determine the pile resistance that might be achieved considering available diesel hammers. Nominal drivability resistances were determined based on a limiting driving criteria of 15 bpi and a maximum driving stress of 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.

[4] Drivability resistance based on a APE D19-42 pile hammer at Fuel Setting 4; Abutments 1 and 2 are the same.

[5] Drivability resistance based on a APE D25-42 pile hammer at Fuel Setting 4, Abutment 1 pile controls.

[6] Drivability evaluations were performed for both Abutments No.1 and 2 piles. Resistances for the 14x89 pile sections based on a APE D19-42 pile hammer govern.

7.1.2 Axial Pile Resistance – Service and Extreme Limit State

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. 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 driven 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 and vehicle collision. 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 calculated factored axial structural, geotechnical and drivability resistances of a HP 14x89 pile section for the service and extreme limit states are summarized below.

Service and Extreme Limit State Factored Axial Pile Resistance					
Pile Section	Structural Resistance[1] $\phi = 1.0$ (kips)	Controlling Geotechnical Resistance[2] $\phi = 1.0$ (kips)	Drivability Resistance[3] $\phi = 1.0$ (kips)		Governing Axial Pile Resistance[6] (kips)
HP 14 x 89	1,305	1,305	670[4]	730[5]	670[4]

[1] Nominal structural resistances were calculated for the lower, braced pile segment in pure axial compression. Factored structural resistances should be calculated for upper and lower unbraced pile segments in combined axial loading and bending, based on LPile results. These resistances may be the controlling values.

[2] Geotechnical axial pile resistance evaluations assumed piles penetrate weathered bedrock and terminate on, or in, competent bedrock. Based on guidance in LRFD Article 10.7.3.2.3., *Piles Driven to Hard Rock.* 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 nominal structural resistance values obtained from LRFD Article 6.9.4.1.

[3] Drivability analyses were performed to determine the pile resistance that might be achieved considering available diesel hammers. Nominal drivability resistances were determined based on a limiting driving criteria of 15 bpi and a maximum driving stress of 45 ksi.

[4] Drivability resistance based on a APE D19-42 pile hammer at Fuel Setting 4; Abutments 1 and 2 are the same.

[5] Drivability resistance based on a APE D25-42 pile hammer at Fuel Setting 4, Abutment 1 pile controls.

[6] Drivability evaluations were performed for both Abutments No.1 and 2 piles. Resistances for the 14x89 pile sections based on a APE D19-42 pile hammer govern.

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. However, the estimated factored axial pile resistances from the drivability analyses for the H-pile sections are less than the controlling factored axial geotechnical resistance and the structural resistance calculated for a braced pile segment. 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 the table above.

The maximum applied factored axial pile load for the service and extreme limit states shall not exceed the governing factored axial pile resistance shown in the table 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 will be performed to evaluate pile behavior at the abutments using LPile software. The designer should utilize the lateral pile analyses to evaluate the associated pile stresses, bending moments, and fixity due to factored pile head loads and displacements.

Geotechnical parameters for generation of soil-resistance (p-y) curves in lateral pile analyses are provided in the tables below. The models developed should emulate appropriate structural parameters and pile-head boundary conditions for the pile section(s) being analyzed.

LPile Input Parameters Abutment No. 1						
Soil Layer	Soil/Rock Model	Top Elevation of Layer (ft)	Layer Thickness (ft)	γ_e[1] (pcf)	ϕ'[2] (deg)	k_s[3] (pci)
Granular Borrow	Reese Sand	84	11	125	32	90
Submerged Fill	Reese Sand	73	3	58	30	20
Glaciomarine Deposit	Reese Sand	70	15	63	30	50
Glacial Till	Reese Sand	55	31	78	38	125

LPile Input Parameters Abutment No. 2						
Soil Layer	Soil/Rock Model	Top Elevation of Layer (ft)	Layer Thickness (ft)	γ_e[1] (pcf)	ϕ'[2] (deg)	k_s[3] (pci)
Granular Borrow	Reese Sand	84	11	125	32	90
Wetland Deposit	Reese Sand	73	4	53	26	20
Glaciomarine Deposit	Reese Sand	69	21	63	30	60
Glacial Till	Reese Sand	48	23	78	38	125

7.1.4 Scour and Pile Buckling Evaluation and Pile Lateral Resistance

In consideration of LRFD Article 3.7.5, it is recommended that the bridge designer evaluate the potential for buckling of the piles due to scour effects. The design shall consider the maximum anticipated depth of scour as per the site-specific scour analysis. The assessment should account for the reduction in lateral support to the pile provided by the surrounding soil as a result of scour.

The design should ensure that the piles remain stable under the combined effects of axial and lateral loads and the loss of lateral support caused by scour. The bridge designer should refer to LRFD Article 10.7.3.13.1 for guidance on pile buckling analysis.

The effect of scour should also be considered in the determination of minimum pile embedment to ensure fixity is satisfied after the design scour event; refer to LRFD 10.7.3.6.

[1] Effective unit weight.
[2] Effective internal angle of friction.
[3] Soil modulus constant.

7.1.5 Driven 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. Minimum 24-hour restrike tests will be required to verify time-dependent loss of pile resistance does not occur. If a loss in pile resistance does occur, the driving criteria shall be adjusted. Restrikes or additional dynamic tests may be required as part of the pile field quality control program should pile behavior vary radically between adjacent piles, should the pile tip be not firmly embedded in bedrock, or if piles “walk” out of position.

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

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

7.1.6 Corrosion Mitigation

At Abutment No. 2, subsurface exploration encountered approximately 4 feet of Wetland Deposits. Laboratory testing of a representative sample indicated an organic content of approximately 18 percent. According to Table 4-9 of Geotechnical Engineering Circular No. 5 (GEC No. 5), this classifies the material as organic soil. Section 4.10.5 of GEC No. 5 notes that soils with substantial organic content may exhibit high corrosion potential. Based on local experience, soils with high organic content are commonly associated with acidic conditions and low electrical resistivity, both of which are indicators of a potentially corrosive environment as outlined in LRFD Article 10.7.5. It is recommended that the bridge designer considers upsizing the pile section at Abutment No. 2 to HP 14x117 to provide allowance of sacrificial steel for corrosion resistance.

7.2 Integral Abutment and Wingwall 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. A resistance factor (ϕ) of 1.0 shall be used to assess abutment design at the service limit state, including: settlement and excessive horizontal movement. The overall stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, ϕ, of 0.65. Resistance factors for extreme limit state shall be taken as 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:

- Internal Friction Angle (ϕ) = 32°
- Total Unit Weight (γ) = 125 pcf
- Soil-Concrete Interface Friction Angle (δ) = 17° (ref: LRFD Table 3.11.5.3-1)

Integral abutments and in-line wingwalls shall be designed to withstand a lateral earth load equal to the passive pressure state. Estimation of passive earth pressure should consider LRFD C3.11.5.4, which states that the relative wall movement to induce full passive pressure is approximately 0.05 for dense backfill, and FHWA NHI-06-089 Figure 10-4 which supports a K_p of 6.0 and greater for dense backfills and wall rotations equal to or greater than 0.02. In general, when the calculated ratio of lateral movement to wall height exceeds 0.004, a passive earth pressure coefficient can be estimated using MassDOT LRFD Bridge Design Manual Figure 3.10.8-1 (reproduced in Appendix D – Calculations). Assuming a 74-foot span, concrete beam superstructure, the thermal movement at each abutment was estimated to be 0.21 inch, resulting in an estimated ratio of thermal expansion to abutment height (δ/H) of 0.0016. Therefore, Rankine Theory is recommended to determine the passive earth pressure coefficient. Using Rankine Theory, a lateral earth pressure coefficient of 3.25 is recommended assuming a δ/H of 0.0016 and a level backfill (see Appendix D – Calculations).

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 the table below:

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

In-line wingwalls shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to an equivalent height of soil of 2.0 feet. 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.

7.3 Abutment Sections

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. Conventional French Drains are the preferred system compared to other systems.

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

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).

7.4 Settlement

The project calls for a grade raise of approximately 13 inches at Abutment No. 1 and 19 inches at Abutment No. 2. Beneath the approach fills at Abutment No. 2, an approximately 4-foot-thick wetland deposit consisting of soft silt with peat was encountered. Borings at both bridge approaches encountered a marine deposit consisting of variable medium stiff silt and clay, and loose to medium dense sands. These soil deposits will undergo immediate elastic and consolidation settlement in response to the increase in vertical overburden pressure due to the grade raise.

Settlement calculations were performed using Rocscience Settle3D Version 3.0. The resulting, estimated total settlement is on the order of 1.7 inches over the design life of the structure, with 0.5 inches of that being immediate elastic settlement which will occur during the initial stages of construction. The remaining settlement (1.2 inches) is attributed to consolidation and it is anticipated that most of this will occur during the overall duration of the construction project, assuming base pavement is installed in the fall and surface pavement is installed in the spring.

See Appendix D for supporting calculations.

7.5 Frost Protection

Foundations placed on soil should be designed with an appropriate embedment for frost protection. According to MaineDOT BDG Figure 5-1, Maine Design Freezing Index Map, Charlotte has a design freezing index (DFI) of approximately 1350 F-degree days. The anticipated coarse-grained fill soil was assigned a water content of 10%. These components correlate to a frost depth of 6.5 feet. Any foundation bearing on soils shall be embedded 6.5 feet for frost protection.

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

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

7.6 Seismic Design Considerations

The United States Geological Survey Seismic Design CD (Version 2.1) provided with the 2014 LRFD Code (7^{th} Edition), 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 the following table:

Parameter	Design Value
Peak Ground Acceleration (PGA)	0.085g
Acceleration Coefficient (A_S)	0.136g
S_{DS} (Period = 0.2 sec)	0.263g
S_{D1} (Period = 1.0 sec)	0.099g
Site Class	D
Seismic Zone	1

In conformance with LRFD Table 4.7.4.3-1 seismic analysis is not required for 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.

8.0 CONSTRUCTION RECOMMENDATIONS AND CONSIDERATIONS

Any soft or unsuitable soil encountered at the abutment subgrade elevations shall be excavated in its entirety and replaced with Granular Borrow – Material for Underwater Backfill and the exposed subgrade then thoroughly compacted. Any loose, coarse-grained soils encountered at the subgrade level shall be proof compacted.

Excavation for the abutments is anticipated to be accomplished using sloped open cut methods in accordance with MaineDOT and OSHA requirements. Excavations 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.

Cobbles were encounter in the glacial till deposit. There is potential for these obstructions to cause difficulties during pile driving operations. If obstructions are encountered prior to reaching the maximum required penetration resistance on bedrock, then they may be cleared by conventional excavation methods, pre-augering, predrilling, spudding, use of rock chisels, or down-hole hammers.

Driven H-pile may reach the required nominal capacity within the glacial till or weathered bedrock. If this occurs, the pile driving criteria should be carried out for 6 consecutive inches. The geotechnical engineer will review the pile logs to confirm the depth of penetration is acceptable. If the depth of penetration is not acceptable, the contractor will be responsible to advance the pile further, which may include, but is not limited to, modifying the pile driving equipment, excavation, or predrilling.

Based on the hydrology report, the typical water elevation is El. 75 ft. A cofferdam will likely be necessary to successfully dewater and construct the abutments. Wood chips were noted in BB-CHAR-101 within the existing fill. Wood chips indicate the presence of wood debris or timber which may obstruct the installation of a cofferdam. Additionally, a previous structure at the bridge was supported on stone-filled log crib abutments. Wood or stone obstructions may need to be removed by conventional excavation methods.

9.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Moosehorn Bridge in Charlotte, 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 final design and specifications in order that the earthwork and foundation recommendations and construction considerations presented in this report are properly interpreted and implemented in the design and specifications.

Sheets

Username: Nathan.P.Pukay Division: GEOTECH Date:4/11/2025

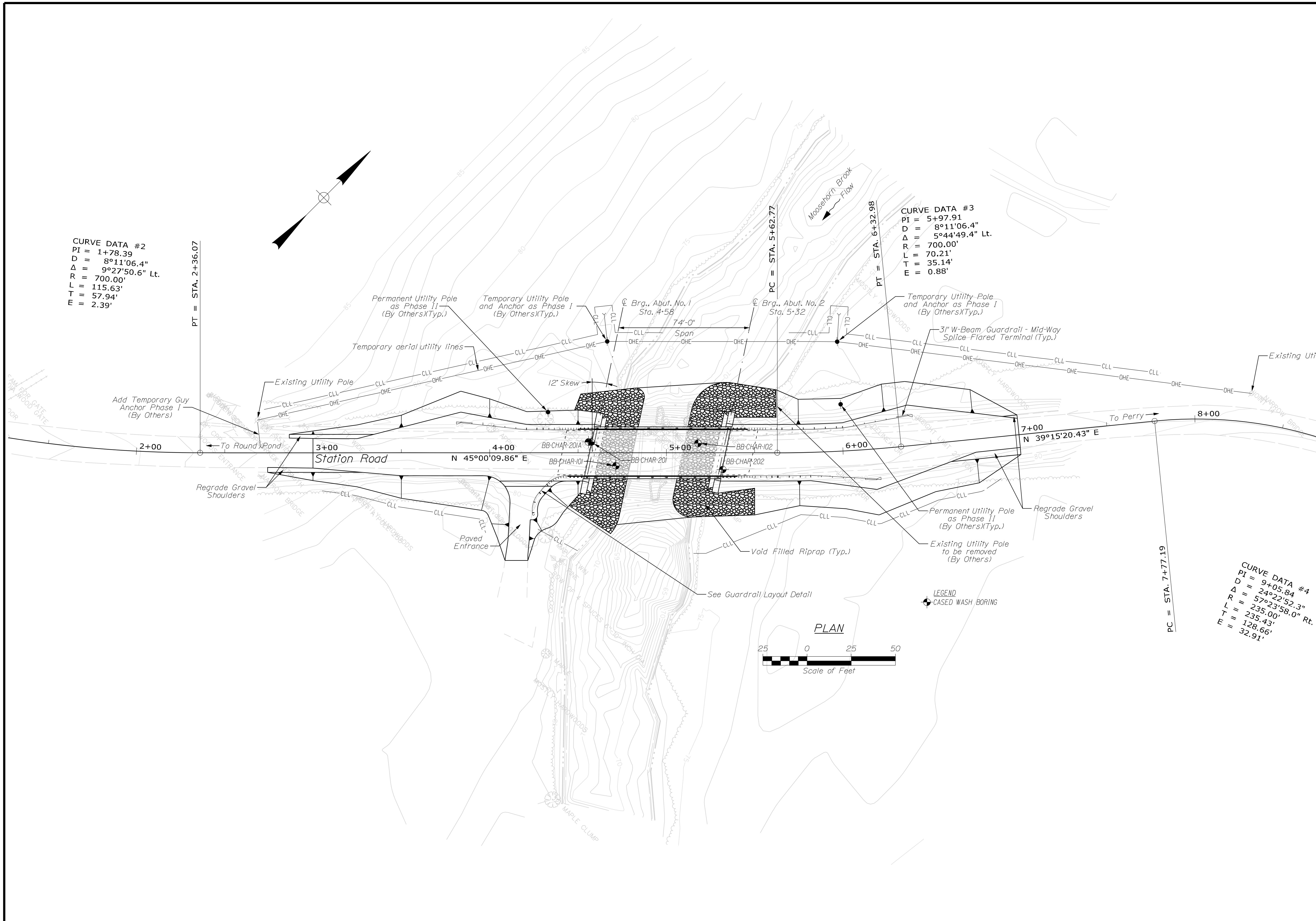
CHARLOTTE, MAINE

0.25 Miles
1 inch = 0.28 miles

Date: 4/10/2025
Time: 7:17:03 AM

SHEET NUMBER	MOOSEHORN BRIDGE MOOSEHORN BROOK CHARLOTTE WASHINGTON CTY.	STATE OF MAINE DEPARTMENT OF TRANSPORTATION
1		02168610
OF 5	LOCATION MAP	WIN BRIDGE NO. 3332 021686.10 BRIDGE PLANS

Filename: ...\10\GEOTECH\MSTA\005_BLP1.dgn Division: GEOTECH Username: Nathan.P.Pukay Date:6/2/2025



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

02168610

WIN 021686.10

BRIDGE NO. 3332

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	M. PARLIN		
DESIGN-DETAILED	B. BARTLETT	D. SHAW	
CHECKED-REVIEWED	------	------	------
DESIGN2-DETAILED2	N.PUKAY	T.WHITE	APR 2025
DESIGN3-DETAILED3	------	------	------
REVISIONS 1	------		------
REVISIONS 2	------		------
REVISIONS 3	------		------
REVISIONS 4	------		------
FIELD CHANGES	------		------

SIGNATURE

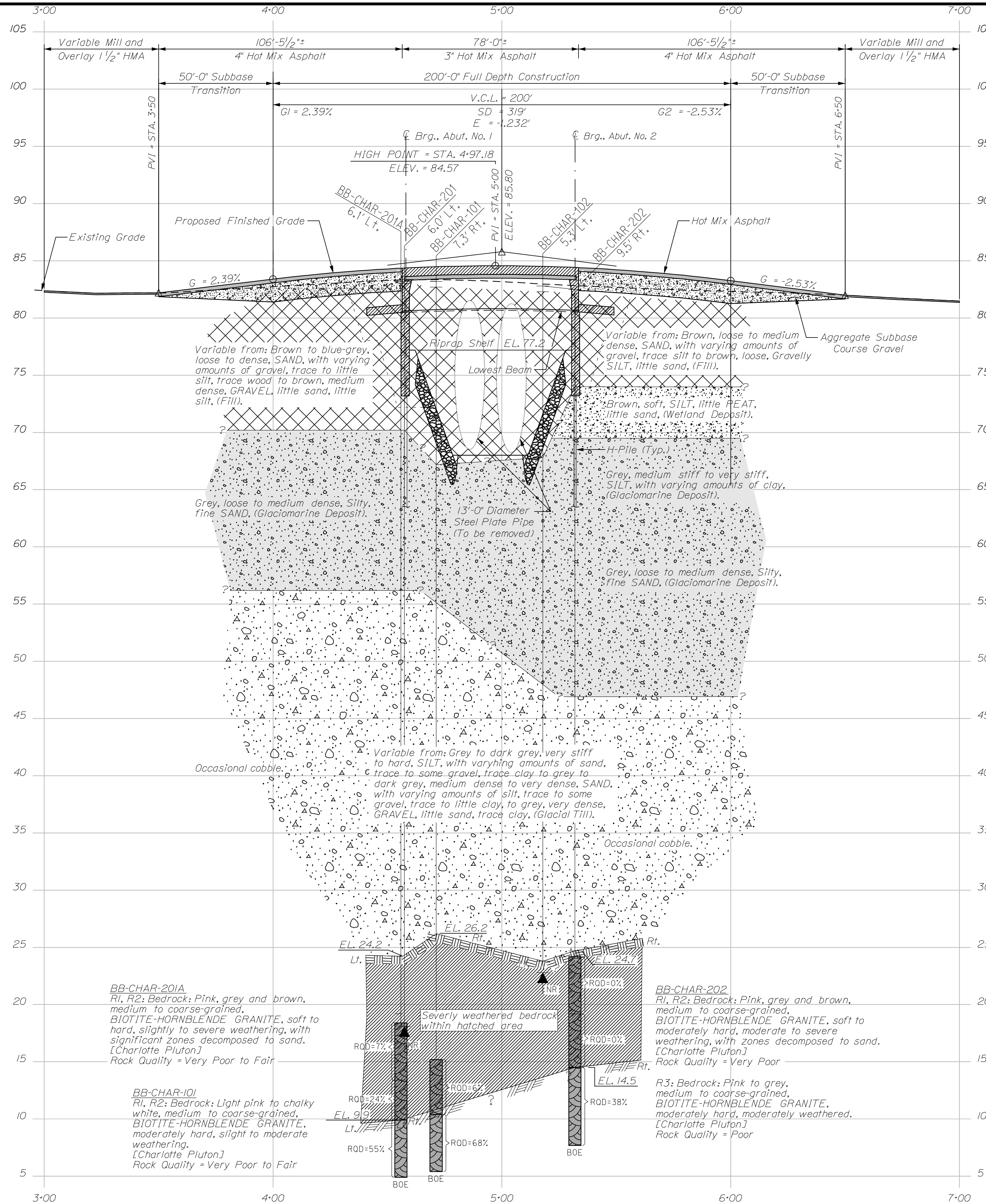
P.E. NUMBER

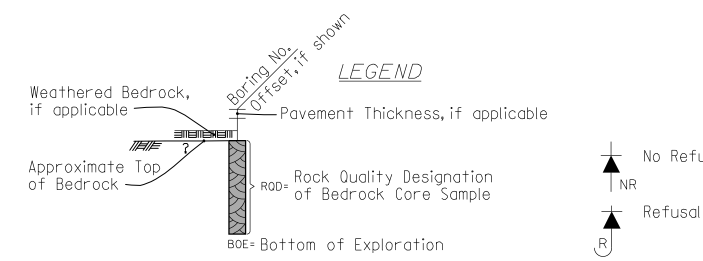
DATE

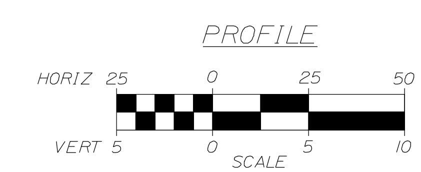
MOOSEHORN BRIDGE
MOOSEHORN BROOK
CHARLOTTE WASHINGTON COUNTY

BORING LOCATION PLAN

SHEET NUMBER
2
OF 5







Notes: 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.

"Varying Amounts" term = Portion is 0 to 50 percent of total.

Filename: ...\10\GEOTECH\MSTA\006_ISP1.dgn Division: GEOTECH Username: Nathan.P.Pukay Date:4/28/2025

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
02168610
WIN 021686.10
BRIDGE NO. 3332
BRIDGE PLANS

PROJ. MANAGER	M. PARLIN	BY	DATE
DESIGN-DETAILED	B. BARTLETT	D. SHAW	
CHECKED-REVIEWED	-----	-----	-----
DESIGN2-DETAILED2	N.PUKAY	T.WHITE	APR 2025
DESIGN3-DETAILED3	-----	-----	-----
REVISIONS 1	-----	-----	-----
REVISIONS 2	-----	-----	-----
REVISIONS 3	-----	-----	-----
REVISIONS 4	-----	-----	-----
FIELD CHANGES	-----	-----	-----

SIGNATURE
P.E. NUMBER
DATE

MOOSEHORN BRIDGE
MOOSEHORN BROOK
CHARLOTTE WASHINGTON COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
3
OF 5

Filename: ...\MSTAV007_BORING LOGS1.dgn   Division: GEOTECH   Username: Nathan.P.Pukay   Date:4/28/2025

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

Project: Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook
Location: Charlotte, Maine

Boring No.: BB-CHAR-101
WIN: 21686.10

Driller:	New England Boring Contractors	Elevation (ft.)	83.2	Auger ID/OD:	4" SSA
Operator:	Mike Porter	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	TRM	Rig Type:	Truck Mounted Mobile #83	Hammer Wt./Fall:	140 lbs./30"
Date Start/Finish:	9:45AM 6/7/17_10:17AM 6/9/2017	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	4+71.3, 7.3 ft Rt.	Casing ID/OD:	HW(4"/4.5")	Water Level*:	6.1' bgs @ 7:15 on 6/8/17
Hammer Efficiency Factor: 0.869		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	82.7	6" of pavement. 0.5	
	1D	24/12	1.00 - 3.00	13/13/10/9	23	33			Brown, dry, dense, SAND, some gravel and fractured rock, little silt, (Fill).	
	2D	24/6	3.00 - 5.00	5/5/4/4	9	13	14		Brown, damp, medium dense, GRAVEL, little sand, little silt, (Fill).	
							23			
5							28			
							27			
							34			
							23			
							17			
10	3D	24/3.6	10.00 - 12.00	5/3/2/3	5	7	22		Brown, damp, loose, SAND, trace fine gravel, trace silt, (Fill).	
							17			
	MD	24/0	12.00 - 14.00	3/3/1/3	4	6	19			
							25			
	4D	24/10.8	14.00 - 16.00	4/4/2/3	6[3]	9	11		Blue-grey, damp, loose, Gravelly SAND, trace silt, trace wood fragments, (Fill).	G#414662 A-1-a, SP WC=10.7%
15							73			
	MD	24/0	16.00 - 18.00	15/5/3/3	8	12	49	67.2	16.0	
							22			
	MV1		18.60 - 19.00				24			
	5D	24/15.6	19.00 - 21.00	3/4/4/3	8[3]	12	25		Grey, wet, medium dense, Silty SAND, (Glaciomarine Deposit).	
20							30			
	6D	24/20.4	21.00 - 23.00	4/3/4/7	7	10	35		Grey, moist, loose, Silty SAND, (Glaciomarine Deposit).	
							41			
							65			
							78			
25	7D	24/9.6	25.00 - 27.00	3/3/5/5	8	12	70		Grey, moist, medium dense, Silty SAND, little gravel, (Glaciomarine Deposit).	G#414665 A-4, SM WC=15.9%
							75	57.2	26.0	
							113			
							126			
							110		Cobbles.2	
30	8D	24/2.4	30.00 - 32.00	5/5/6/8	11	16	90		Grey, damp, very stiff, SILT, some sand, little clay, little gravel and rock fragments, (Glacial Till).	
							98			
							144			
							153		Cobbles.2	
							227			
35	9D	24/19.2	35.00 - 37.00	6/9/43/50	52[3]	75	102		Grey, damp, hard, SILT, some sand, little clay, little gravel and rock fragments, (Glacial Till).	
							146		Roller Coned ahead from 35.0-40.0 ft bgs.	
							276			
							311			
							471			
40	10D	24/16.8	40.00 - 42.00	25/45/92/63	137[3]	198	36		Grey, damp, very dense, GRAVEL, little sand, little silt, trace clay, (Glacial Till). Cobble at 40.0'.	G#414666 A-1-a, GM WC=4.2%
							50			
							37			
							91			
							114			
45	11D	15.6/8.4	45.00 - 46.30	20/31/(50/3.6")	81	117	72		Grey, damp, hard, SILT, some gravel, some sand, little clay, (Glacial Till).	
							159			
							130		Cobbles.2	
							102		Cobble from 48.7' to 49.1'.	
							243			
50	12D	24/20.4	50.00 - 52.00	29/41/33/42	74	107	71		Grey, damp, very dense, SAND, some silt, little gravel, little clay, (Glacial Till).	
							57			
							63			
							176			
							169			
55	13D	20.4/14.4	55.00 - 56.70	10/35/51/(50/2")	86	125	171		Grey, damp, hard, SILT, little gravel, little sand, little clay; 1" piece of pink granite, (Glacial Till).	
							275	26.2	57.0	
							130			
							558			
	14D	2.4/1.2	59.00 - 59.20	50/2.4"			480		Fractured, pink, GRANITE, (Weathered Bedrock).	
60							RC			
65	15D	6/4.8	65.00 - 65.50	117/6"					Grey, SILT, some sand, some pink, fractured rock fragments, (Weathered Bedrock).	
							NQ	16.8	66.4 Top of Bedrock at Elev. 16.8 ft.	
	R1	58/17	68.00 - 72.83	RQD = 6%					R1: Bedrock: Light pink, medium to coarse-grained, BIOTITE-HORNBLENDE GRANITE, moderately hard, moderately weathered, fractures are horizontal to moderately dipping, very close to close spacing, planar to curved discontinuities, rough to very rough surfaces. [Charlotte Pluton]. Rock Quality = Very Poor. R1 Core Times: (min:sec) 68.0-69.0 ft (1:05) 69.0-70.0 ft (1:34) 70.0-71.0 ft (1:11) 71.0-72.0 ft (0:52) 72.0-72.8 ft (1:25) 29% Recovery	
70										
	R2	60/54	72.80 - 77.80	RQD = 68%					R2: Bedrock: Light pink to chalky white, medium to	
75										

Remarks:

1. 300 lb hammer for 4" casing. Auto hammer starting at 10' for 4" casing.
2. Presence of cobbles is assumed based on drilling behavior.
3. Uncorrected N-value is computed from the blow counts of a 3" spoon sample. The resulting N_{60} value may overestimate the soil density.

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-CHAR-101

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS
Project: Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook
Location: Charlotte, Maine
Boring No.: BB-CHAR-101
WIN: 21686.10
Driller: New England Boring Contractors | Elevation (ft.) 83.2 | Auger ID/OD: 4" SSA
Operator: Mike Porter | Datum: NAVD88 | Sampler: Standard Split Spoon
Logged By: TRM | Rig Type: Truck Mounted Mobile #83 | Hammer Wt./Fall: 140 lbs./30"
Date Start/Finish: 9:45AM 6/7/17_10:17AM 6/9/2017 | Drilling Method: Cased Wash Boring | Core Barrel: NQ-2"
Boring Location: 4+71.3, 7.3 ft Rt. | Casing ID/OD: HW(4"/4.5") | Water Level*: 6.1' bgs @ 7:15 on 6/8/17
Hammer Efficiency Factor: 0.869 | Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐

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
75									coarse-grained, BIOTITE-HORNBLENDE GRANITE, moderately hard, slightly weathered, fractures are moderately dipping (35 to 55 degrees), closely spaced (2" to 10"), planar to stepped discontinuities, rough to very rough surfaces with manganese oxide coatings. [Charlotte Pluton]. Rock Quality = Fair. R2 Core Times: (min:sec) 72.8-73.8 ft (1:46) 73.8-74.8 ft (1:23) 74.8-75.8 ft (1:04) 75.8-76.8 ft (1:06) 76.8-77.8 ft (1:22) 90% Recovery	
								5.4	77.8 Bottom of Exploration at 77.8 feet below ground surface.	
80										
85										
90										
95										
100										

Remarks:

1. 300 lb hammer for 4" casing. Auto hammer starting at 10' for 4" casing.
2. Presence of cobbles is assumed based on drilling behavior.
3. Uncorrected N-value is computed from the blow counts of a 3" spoon sample. The resulting N_{60} value may overestimate the soil density.

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-CHAR-101

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

Project: Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook
Location: Charlotte, Maine

Boring No.: BB-CHAR-102
WIN: 21686.10

Driller:	New England Boring Contractors	Elevation (ft.)	82.8	Auger ID/OD:	4" SSA
Operator:	Tom Schaefer	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	TRM	Rig Type:	Mobile B53	Hammer Wt./Fall:	140 lbs./30"
Date Start/Finish:	1:45pm 6/12/17_12:45pm 6/12/17	Drilling Method:	SSA & D+W	Core Barrel:	NQ-2"
Boring Location:	5+17.9, 5.3 ft Lt.	Casing ID/OD:	HW(4"/4.5")	Water Level*:	See Remarks
Hammer Efficiency Factor: 0.6		Hammer Type:	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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	82.3	6" of pavement. 0.5	
	1D	24/13.2	1.00 - 3.00	6/8/6/4	14	14			Brown, dry, medium dense, SAND, little fine gravel, trace silt, (Fill).	
	2D	24/12	3.00 - 5.00	5/3/2/2	5	5			Brown, dry, loose, medium to coarse SAND, some gravel, trace silt, (Fill).	G#414663 A-1-a, SW-SM WC=4.0%
5	3D	24/9.6	5.00 - 7.00	1/2/3/2	5	5			Brown, dry, loose, medium to coarse SAND, some gravel, trace silt, trace white quartz fragments, (Fill).	
	4D	24/8.4	7.00 - 9.00	1/4/2/3	6	6			Brown, dry, medium stiff, Gravelly SILT, little sand, (Fill).	
							15			
10							16			
							15			
							14			
							14			
							13			
15	5D	24/12	15.00 - 17.00	2/2/4/3	6	6	2	67.8	15.0 5D: Layered, brown, moist, loose, fine to medium SAND with organics, trace silt, odor; grey, moist, loose, fine to medium SAND, trace silt; grey, moist, medium stiff, Clayey SILT, (Glaciomarine Deposit).	
							3			
	6D	24/21.6	17.00 - 19.00	2/2/4/3	6	6	11		6D: Layered, grey, moist, medium stiff, Silty CLAY, trace fine gravel, PP: 0.75, 1.0; TV: 0.325, 0.300; grey, damp, loose, fine SAND, little clay, trace silt, (Glaciomarine Deposit).	G#414668 WC=25.0% LL=31% PL=18% PI=13%
							20			
	7D	24/15.6	19.00 - 21.00	3/3/4/4	7	7	28		Grey, damp, loose, SAND, some silt, (Glaciomarine Deposit).	
20							14			
	8D	24/13.2	21.00 - 23.00	2/5/6/8	11	11	24		Grey, moist, stiff, Sandy SILT, trace clay, (Glaciomarine Deposit).	G#414664 A-4, ML WC=24.9%
							31			
	9D	24/12	23.00 - 25.00	4/4/4/5	8	8	24		Grey, moist, loose, SAND, some silt, (Glaciomarine Deposit).	
							35			
25	10D	24/20.4	25.00 - 27.00	2/3/2/2	5	5			Grey, moist, loose, Silty, fine SAND, trace clay, (Glaciomarine Deposit).	
30	11D	24/13.2	30.00 - 32.00	5/9/9/11	18	18	15		Grey, moist, medium dense, Silty, fine SAND, (Glaciomarine Deposit).	
							19			
							29			
							42			
							49			
35	12D	24/14.4	35.00 - 37.00	5/5/24/26	29	29			Grey, moist, medium dense, Silty, fine SAND, trace clay, (Glaciomarine Deposit).	
							21			
							26	45.8	37.0	
							34			
							57			
40	13D	24/13.2	40.00 - 42.00	15/19/19/23	38[2]	38	54		Grey, moist, hard, SILT, some sand, little clay, little gravel, (Glacial Till). Cobbles from 40' to 45'.2	
							64			
							120			
							130			
							89			
45	14D	24/13.2	45.00 - 47.00	14/19/23/28	42	42	OPEN		Dark grey, moist, hard, SILT, some sand, little clay, little gravel, (Glacial Till).	G#414667 A-4, CL WC=11.7%
50	15D	6/4.8	50.00 - 50.50	(50/6")					Dark grey, moist, very dense, Silty SAND, little clay, little gravel, (Glacial Till).	
55	16D	11/10	55.00 - 55.92	44/(50/5")					Dark grey, moist, hard, SILT, some sand, little clay, little gravel and rock fragments, (Glacial Till).	
								23.8	59.0 Pale red sand/fractured rock from 59' to 60' in wash.	
	MD	0/0	59.90 - 59.90	(50/0")				22.8	No penetration with 2" split spoon. 60.0 Bottom of Exploration at 60.0 feet below ground surface.	
60										
65										
70										
75										

Remarks:

1. Water levels: 7.4' bgs at 17:05 on 06/12/17; 6.7' bgs at 7:10 06/13/17; No water in hole after casing removed.
2. Presence of cobbles is assumed based on drilling behavior.
3. Boring collapsed back to 8.1' when casing removed.
4. 300 lb hammer used for 4" casing installation.

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

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

Page 1 of 1
Boring No.: BB-CHAR-102

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

Project: Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook
Location: Charlotte, Maine

Boring No.: BB-CHAR-201
WIN: 21686.10

Driller:	Seaboard Drilling LLC	Elevation (ft.)	83.2	Auger ID/OD:	5" Solid Stem
Operator:	Hanscom/Wall	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/12-14/2024	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	4+57.4, 6.0 ft Lt.	Casing ID/OD:	NW(3"/3.5"), HW(4"/4.5")	Water Level*:	9.0 ft bgs.
Hammer Efficiency Factor: 1.066		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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	83.0	2" HMA. 0.2	
5	1D	24/11	5.00 - 7.00	3/3/2/2	5	9			Brown, moist, loose, SAND, some gravel, little silt, (Fill).	
10	MD	24/0	10.00 - 12.00	5/2/3/2	5	9	16		Loose. No sample recovered. Soil similar to 1D on auger flights.	
							13			
							30			
							23	70.2	13.0	
							28			
15	2D	24/18	15.00 - 17.00	2/2/3/5	5	9	29		Grey, wet, loose, Silty, fine SAND, little clay, (Glaciomarine Deposit).	
							34			
							42			
							46			
							55			
20	3D	24/20	20.00 - 22.00	3/3/4/4	7	12	51		Grey, wet, medium dense, Silty, fine SAND, (Glaciomarine Deposit).	G#380946 A-4, SM WC=21.7%
							43			
							48			
							48			
							51			
25	4D	24/17	25.00 - 27.00	4/4/4/6	8	14	52		Similar to 3D.	
							96			
							60			
							72	55.2	28.0	
							87			
30	5D	24/16	30.00 - 32.00	5/5/7/7	12	21	81		Grey, wet, medium dense, Silty SAND, little clay, trace gravel, (Glacial Till).	
							163			
							146			
							170			
							212			
35	6D	24/7	35.00 - 37.00	9/10/17/20	27	48	OPEN HOLE		Grey, wet, dense, Silty SAND, little gravel, trace clay, (Glacial Till).	
40	7D	9.6/9.6	40.00 - 40.80	21/58(3.6")	---				Grey, wet, very dense, Silty SAND, some gravel, (Glacial Till). Cobble from 40.8-41.3 ft bgs. Cobble from 42.5-43.2 ft bgs.	
45	8D	12/6	45.00 - 46.00	33/55	---				Similar to 7D.	
50									Cobble from 49.9-50.3 ft bgs. Set in NW Casing at 50.0 ft bgs.	
	9D	24/17	51.00 - 53.00	19/28/29/40	57	101			Grey, wet, very dense, SAND, some silt, trace gravel, (Glacial Till).	
55	10D	4.8/4.8	55.00 - 55.40	70(4.8")	---				Grey, wet, very dense, Silty SAND, some gravel, (Glacial Till).	
								24.2	59.0	
60	11D	6/5	60.00 - 60.50	90	---				Pink to brown, wet, very dense, SAND, some gravel, trace silt, (Weathered Bedrock).	
65	12D	1.2/1.2	65.00 - 65.10	50(1.2")	---			18.1	Pink to grey, wet, very dense, GRAVEL, some sand, trace silt (Weathered Bedrock). 65.1 Bottom of Exploration at 65.1 feet below ground surface. Broke NW(3") Casing, moved to BB-CHAR-201A. 20.0 ft of NW(3") broken casing abandoned in hole from 65.1 ft bgs (El. 18.1) to 45.1 ft bgs (El. 38.1) .	
70										
75										

Remarks:

1) Auto Hammer #367
2) 20.0 ft of NW(3") broken casing abandoned in hole from 65.1 ft bgs (El. 18.1) to 45.1 ft bgs (El. 38.1).

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

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

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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

02168610

WIN
021686.10

BRIDGE NO. 3332

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	M. PARLIN		
DESIGN-DETAILED	B. BARTLETT	D. SHAW	
CHECKED-REVIEWED	-----	-----	-----
DESIGN2-DETAILED2	N.PUKAY	T.WHITE	APR 2025
DESIGN3-DETAILED3			
REVISIONS 1	-----	-----	-----
REVISIONS 2	-----	-----	-----
REVISIONS 3	-----	-----	-----
REVISIONS 4	-----	-----	-----
FIELD CHANGES	-----	-----	-----

SIGNATURE

P.E. NUMBER

DATE

MOOSEHORN BRIDGE
MOOSEHORN BROOK
CHARLOTTE WASHINGTON COUNTY

BORING LOGS

SHEET NUMBER
4
OF 5

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

Project: Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook
Location: Charlotte, Maine
Boring No.: BB-CHAR-201A
WIN: 21686.10

Driller:	Seaboard Drilling LLC	Elevation (ft.)	83.2	Auger ID/OD:	5" Solid Stem
Operator:	Kevin/Jason	Datum:	NAVD88	Sampler:	N/A
Logged By:	B. Wilder	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	N/A
Date Start/Finish:	3/13/2024-3/14/2024	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	4+55.9, 6.1 ft Lt.	Casing ID/OD:	NW-3" & HW-4"	Water Level*:	9.5 ft bgs.
Hammer Efficiency Factor: 1.066		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 140 lb. 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							DRIVE HW		Drove HW Casing to 35.0 ft bgs. See BB-CHAR-201 for soil descriptions.	
35							OPEN HOLE			
64.0								19.2	Top of Bedrock at Elev. 19.2 ft. Roller Coned ahead to 64.8 ft bgs. Set in NW casing to 64.8 ft bgs.	
65	R1	60/42	64.80 - 69.80	RQD = 7%			NQ-2		R1: Bedrock: Pink, grey and brown, medium to coarse-grained, BIOTITE-HORNBLENDE GRANITE, soft to moderately hard, moderately to severely weathered, with significant zones decomposed to sand, joints dipping at horizontal to moderate angles, spaced very close. [Charlotte Pluton] Rock Quality = Very Poor. R1: Core Times (min:sec) 64.8-65.8 ft (1:43) 65.8-66.8 ft (2:21) 66.8-67.8 ft (2:07) 67.8-68.8 ft (2:06) 68.8-69.8 ft (2:55) 70% Recovery	
70	R2	42/34	69.80 - 73.30	RQD = 24%					R2: Bedrock: Pink to grey, medium to coarse-grained, BIOTITE-HORNBLENDE GRANITE, moderately hard, moderately weathered, joints dipping at horizontal to low angles, closely spaced, then pink, grey and brown, medium to coarse-grained, BIOTITE-HORNBLENDE GRANITE, severely weathered and decomposed to sand. [Charlotte Pluton] Rock Quality = Very Poor.	
	R3	60/60	73.30 - 78.30	RQD = 55%						

Remarks:
1) Auto Hammer #367
2) 20.0 ft of NW(3") broken casing abandoned in hole from 64.8 ft bgs (El. 19.2) to 44.8 ft bgs (El. 39.2).

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-CHAR-201A

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

Project: Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook
Location: Charlotte, Maine
Boring No.: BB-CHAR-201A
WIN: 21686.10

Driller:	Seaboard Drilling LLC	Elevation (ft.)	83.2	Auger ID/OD:	5" Solid Stem
Operator:	Kevin/Jason	Datum:	NAVD88	Sampler:	N/A
Logged By:	B. Wilder	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	N/A
Date Start/Finish:	3/13/2024-3/14/2024	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	4+55.9, 6.1 ft Lt.	Casing ID/OD:	NW-3" & HW-4"	Water Level*:	9.5 ft bgs.
Hammer Efficiency Factor: 1.066		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Depth (ft.)	Elevation (ft.)	Visual Description and Remarks
75		R2: Core Times (min:sec) 69.8-70.8 ft (1:53) 70.8-71.8 ft (1:53) 71.8-72.8 ft (2:00) 72.8-73.3 ft (3:00) Core Blocked 81% Recovery
		R3: Bedrock: Pink to grey, medium to coarse-grained, BIOTITE-HORNBLENDE GRANITE, moderately hard to hard, slightly weathered, joints dipping at horizontal to moderate angles, closely spaced, then pink, grey and brown, medium to coarse-grained, GRANITE, soft to moderately hard, severely weathered, with a zone decomposed to sand, joints dipping at horizontal to low angles, spaced very close. [Charlotte Pluton] Rock Quality = Fair. R3: Core Times (min:sec) 73.3-74.3 ft (1:36) 74.3-75.3 ft (2:02) 75.3-76.3 ft (2:42) 76.3-77.3 ft (2:41) 77.3-78.3 ft (2:50) 100% Recovery
78.3	4.9	Bottom of Exploration at 78.3 feet below ground surface. NW(3") casing broke while pulling. 20.0 ft of NW(3") broken casing abandoned in hole from 64.8 ft bgs (El. 19.2) to 44.8 ft bgs (El. 39.2).

Remarks:
1) Auto Hammer #367
2) 20.0 ft of NW(3") broken casing abandoned in hole from 64.8 ft bgs (El. 19.2) to 44.8 ft bgs (El. 39.2).

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-CHAR-201A

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

Project: Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook
Location: Charlotte, Maine
Boring No.: BB-CHAR-202
WIN: 21686.10

Driller:	MaineDOT	Elevation (ft.)	82.5	Auger ID/OD:	5" Solid Stem
Operator:	Doggett/Andrie	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	N. Pukay	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/12/2024-3/13/2024	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	5+31.9, 9.5 ft Rt.	Casing ID/OD:	HW-4"	Water Level*:	
Hammer Efficiency Factor: 0.962		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/14	0.00 - 2.00	4/6/6/7	12	19	SSA		Brown, moist, medium dense, SAND, little gravel, trace silt, (Fill).	
5	2D	24/5	5.00 - 7.00	1/1/1/1	2	3			Brown, moist, very loose, Gravelly SAND, trace silt, (Fill).	G#380947 A-1-a, SW-SM WC=7.3%
8.5								74.0	Grey-brown silt and clay on auger flights from 8.5 to 10.0 ft bgs.	
10	3D	24/24	10.00 - 12.00	1/1/1/1	2	3	6		Brown, wet, soft, SILT, little peat, little sand, (Wetland Deposit).	#380948 Organic Content 18.4% WC=112.9%
							5			
							10		Silt, peat, and sand observed in wash from 10.0 to 13.0 ft bgs.	
13.0							20	69.5		
							28			
15	4D/A	24/24	15.00 - 17.00	2/7/9/8	16	26	18		4D (15.0-16.0 ft bgs.) Grey, wet, very stiff, SILT, some clay, little fine sand, (Glaciomarine Deposit). 4D/A (16.0-17.0 ft bgs.) Grey, wet, medium dense, Silty, fine SAND, trace clay, (Glaciomarine Deposit).	G#380949 A-4, CL WC=23.5% Non-Plastic
							25			
							20			
							38			
							101			
20	5D	24/14	20.00 - 22.00	4/4/3/6	7	11	52		Grey, wet, medium dense, fine SAND, some silt, trace clay, (Glaciomarine Deposit).	
							45			
							50			
							35			
							38			
25	6D	24/20	25.00 - 27.00	4/4/6/6	10	16	44		Grey, wet, medium dense, Silty, fine SAND, (Glaciomarine Deposit).	
							42			
							45			
							51			
							46			
30	7D	24/14	30.00 - 32.00	2/3/4/3	7	11	47		Similar to 6D.	
							45			
							50			
							65			
34.5							83	48.0		
35	8D	24/20	35.00 - 37.00	6/10/13/13	23	37	OPEN HOLE		Grey, wet, hard, SILT, some sand, trace gravel, trace clay, (Glacial Till).	
40	9D	24/6	40.00 - 42.00	10/12/11/61	23	37			Grey, wet, hard, Sandy SILT, some gravel, (Glacial Till).	
45	10D	24/9	45.00 - 47.00	7/9/12/18	21	34			Similar to 9D.	
50	11D	24/13	50.00 - 52.00	20/17/28/31	45	72			Grey, wet, very dense, Silty SAND, some gravel, (Glacial Till).	
55	12D	24/5	55.00 - 57.00	26/33/43/56	76	122			Similar to 11D.	
57.8								24.7	Top of Bedrock at Elev. 24.7 ft. Pink granite sand in wash at 57.8 ft bgs.	
	R1 MD	60/20 1.2/0	58.30 - 63.30 58.30 - 58.40	RQD = 0% 50(1.2")	---		NQ-2		R1: Bedrock: Pink, grey and brown, medium to coarse-grained, BIOTITE-HORNBLENDE GRANITE, soft to moderately hard, moderately to severely weathered, with zones decomposed to sand, joints dipping at horizontal to vertical angles, spaced very close. [Charlotte Pluton] Rock Quality = Very Poor. R1: Core Times (min:sec) 58.3-59.3 ft (1:13) 59.3-60.3 ft (1:28) 60.3-61.3 ft (1:22) 61.3-62.3 ft (3:38) 62.3-63.3 ft (1:31) 33% Recovery	
	R2	56.4/28	63.30 - 68.00	RQD = 0%					R2: Bedrock: Similar to R1. [Charlotte Pluton] Rock Quality = Very Poor. R2: Core Times (min:sec) 63.3-64.3 ft (1:59) 64.3-65.3 ft (1:37) 65.3-66.3 ft (2:36) 66.3-68.0 ft (3:24) Core Blocked 49% Recovery	
	R3	81.6/62	68.00 - 74.80	RQD = 38%				7.7	R3: Bedrock: Pink to grey, medium to coarse-grained, BIOTITE-HORNBLENDE GRANITE, moderately hard, moderately weathered, joints dipping at horizontal to moderate angles, closely spaced. [Charlotte Pluton] Rock Quality = Poor. R3: Core Times (min:sec) 68.0-69.0 ft (1:55) 69.0-70.0 ft (0:47) 70.0-71.0 ft (1:53) 71.0-72.0 ft (2:29) 72.0-73.0 ft (2:39) 73.0-74.0 ft (2:53) 74.0-74.8 ft (2:58)	

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-CHAR-202

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

Project: Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook
Location: Charlotte, Maine
Boring No.: BB-CHAR-202
WIN: 21686.10

Driller:	MaineDOT	Elevation (ft.)	82.5	Auger ID/OD:	5" Solid Stem
Operator:	Doggett/Andrie	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	N. Pukay	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/12/2024-3/13/2024	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	5+31.9, 9.5 ft Rt.	Casing ID/OD:	HW-4"	Water Level*:	
Hammer Efficiency Factor: 0.962		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Depth (ft.)	Visual Description and Remarks
75	76% Recovery
74.8	Bottom of Exploration at 74.8 feet below ground surface.

Remarks:

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 2 of 2
Boring No.: BB-CHAR-202

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
02168610
WIN 021686.10
BRIDGE NO. 3332
BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	M. PARLIN		
DESIGN-DETAILED	B. BARTLETT	D. SHAW	
CHECKED-REVIEWED			
DESIGN2-DETAILED2	N.PUKAY	T.WHITE	APR 2025
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE
P.E. NUMBER
DATE

MOOSEHORN BRIDGE
MOOSEHORN BROOK
CHARLOTTE WASHINGTON COUNTY

BORING LOGS

SHEET NUMBER
5
OF 5

Appendix A

Boring Logs

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook
Location: Charlotte, Maine

Boring No.: BB-CHAR-101
WIN: 21686.10

Driller:	New England Boring Contractors	**Elevation (ft.)**	83.2	**Auger ID/OD:**	4" SSA
Operator:	Mike Porter	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	TRM	**Rig Type:**	Truck Mounted Mobile #83	**Hammer Wt./Fall:**	140 lbs./30"
Date Start/Finish:	9:45AM 6/7/17_10:17AM 6/9/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	4+71.3, 7.3 ft Rt.	**Casing ID/OD:**	HW(4"/4.5")	**Water Level*:**	6.1' bgs @ 7:15 on 6/8/17
Hammer Efficiency Factor:	0.869	**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.
0							SSA	82.7		6" of pavement 0.5	
	1D	24/12	1.00 - 3.00	13/13/10/9	23	33				Brown, dry, dense, SAND, some gravel and fractured rock, little silt, (Fill).	
	2D	24/6	3.00 - 5.00	5/5/4/4	9	13	14			Brown, damp, medium dense, GRAVEL, little sand, little silt, (Fill).	
							23				
5							28				
							27				
							34				
							23				
							17				
10	3D	24/3.6	10.00 - 12.00	5/3/2/3	5	7	22			Brown, damp, loose, SAND, trace fine gravel, trace silt, (Fill).	
							17				
	MD	24/0	12.00 - 14.00	3/3/1/3	4	6	19				
							25				
	4D	24/10.8	14.00 - 16.00	4/4/2/3	6[3]	9	11			Blue-grey, damp, loose, Gravelly SAND, trace silt, trace wood fragments, (Fill).	G#414662 A-1-a, SP WC=10.7%
15							73	67.2		16.0	
	MD	24/0	16.00 - 18.00	15/5/3/3	8	12	49				
							22				
	MV1		18.60 - 19.00				24				
	5D	24/15.6	19.00 - 21.00	3/4/4/3	8[3]	12	25			Grey, wet, medium dense, Silty SAND, (Glaciomarine Deposit).	
20							30				
	6D	24/20.4	21.00 - 23.00	4/3/4/7	7	10	35			Grey, moist, loose, Silty SAND, (Glaciomarine Deposit).	
							41				
							65				
							78				
25											

Remarks:

1. 300 lb hammer for 4" casing. Auto hammer starting at 10' for 4" casing.
2. Presence of cobbles is assumed based on drilling behavior.
3. Uncorrected N-value is computed from the blow counts of a 3" spoon sample. The resulting N_{60} value may overestimate the soil density.

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-CHAR-101

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook **Location:** Charlotte, Maine	**Boring No.:** BB-CHAR-101 **WIN:** 21686.10

Driller:	New England Boring Contractors	**Elevation (ft.)**	83.2	**Auger ID/OD:**	4" SSA
Operator:	Mike Porter	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	TRM	**Rig Type:**	Truck Mounted Mobile #83	**Hammer Wt./Fall:**	140 lbs./30"
Date Start/Finish:	9:45AM 6/7/17_10:17AM 6/9/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	4+71.3, 7.3 ft Rt.	**Casing ID/OD:**	HW(4"/4.5")	**Water Level*:**	6.1' bgs @ 7:15 on 6/8/17
Hammer Efficiency Factor:	0.869	**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 140 lb. 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.
25	7D	24/9.6	25.00 - 27.00	3/3/5/5	8	12	70			Grey, moist, medium dense, Silty SAND, little gravel, (Glaciomarine Deposit).	G#414665 A-4, SM WC=15.9%
							75	57.2		26.0	
							113				
							126				
							110			Cobbles.[2]	
30	8D	24/2.4	30.00 - 32.00	5/5/6/8	11	16	90			Grey, damp, very stiff, SILT, some sand, little clay, little gravel and rock fragments, (Glacial Till).	
							98				
							144				
							153			Cobbles.[2]	
							227				
35	9D	24/19.2	35.00 - 37.00	6/9/43/50	52[3]	75	102			Grey, damp, hard, SILT, some sand, little clay, little gravel and rock fragments, (Glacial Till).	
							146				
							276			Roller Coned ahead from 35.0-40.0 ft bgs.	
							311				
							471				
40	10D	24/16.8	40.00 - 42.00	25/45/92/63	137[3]	198	36			Grey, damp, very dense, GRAVEL, little sand, little silt, trace clay, (Glacial Till). Cobble at 40.0'.	G#414666 A-1-a, GM WC=4.2%
							50				
							37				
							91				
							114				
45	11D	15.6/8.4	45.00 - 46.30	20/31/(50/3.6")	81	117	72			Grey, damp, hard, SILT, some gravel, some sand, little clay, (Glacial Till).	
							159				
							130			Cobbles.[2]	
							102			Cobble from 48.7' to 49.1'.	
							243				
50											

Remarks:

1. 300 lb hammer for 4" casing. Auto hammer starting at 10' for 4" casing.
2. Presence of cobbles is assumed based on drilling behavior.
3. Uncorrected N-value is computed from the blow counts of a 3" spoon sample. The resulting N_{60} value may overestimate the soil density.

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: Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook Location: Charlotte, Maine	Boring No.: BB-CHAR-101 WIN: 21686.10

Driller:	New England Boring Contractors	**Elevation (ft.)**	83.2	**Auger ID/OD:**	4" SSA
Operator:	Mike Porter	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	TRM	**Rig Type:**	Truck Mounted Mobile #83	**Hammer Wt./Fall:**	140 lbs./30"
Date Start/Finish:	9:45AM 6/7/17_10:17AM 6/9/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	4+71.3, 7.3 ft Rt.	**Casing ID/OD:**	HW(4"/4.5")	**Water Level*:**	6.1' bgs @ 7:15 on 6/8/17
Hammer Efficiency Factor:	0.869	**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 140 lb. 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.
50	12D	24/20.4	50.00 - 52.00	29/41/33/42	74	107	71			Grey, damp, very dense, SAND, some silt, little gravel, little clay, (Glacial Till).	
							57				
							63				
							176				
							169				
55	13D	20.4/14.4	55.00 - 56.70	10/35/51/(50/2")	86	125	171			Grey, damp, hard, SILT, little gravel, little sand, little clay; 1" piece of pink granite, (Glacial Till).	
							275				
							130	26.2		57.0	
							558				
	14D	2.4/1.2	59.00 - 59.20	50/2.4"			480			Fractured, pink, GRANITE, (Weathered Bedrock).	
60							RC				
65	15D	6/4.8	65.00 - 65.50	117/6"						Grey, SILT, some sand, some pink, fractured rock fragments, (Weathered Bedrock).	
							NQ	16.8		66.4 Top of Bedrock at Elev. 16.8 ft.	
	R1	58/17	68.00 - 72.83	RQD = 6%						R1: Bedrock: Light pink, medium to coarse-grained, BIOTITE-HORNBLENDE GRANITE, moderately hard, moderately weathered, fractures are horizontal to moderately dipping, very close to close spacing, planar to curved discontinuities, rough to very rough surfaces. [Charlotte Pluton]. Rock Quality = Very Poor. R1 Core Times: (min:sec) 68.0-69.0 ft (1:05) 69.0-70.0 ft (1:34) 70.0-71.0 ft (1:11) 71.0-72.0 ft (0:52) 72.0-72.8 ft (1:25) 29% Recovery	
70											
	R2	60/54	72.80 - 77.80	RQD = 68%							
75											

Remarks:

1. 300 lb hammer for 4" casing. Auto hammer starting at 10' for 4" casing.
2. Presence of cobbles is assumed based on drilling behavior.
3. Uncorrected N-value is computed from the blow counts of a 3" spoon sample. The resulting N_{60} value may overestimate the soil density.

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: Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook
Location: Charlotte, Maine

Boring No.: BB-CHAR-101
WIN: 21686.10

Driller:	New England Boring Contractors	**Elevation (ft.)**	83.2	**Auger ID/OD:**	4" SSA
Operator:	Mike Porter	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	TRM	**Rig Type:**	Truck Mounted Mobile #83	**Hammer Wt./Fall:**	140 lbs./30"
Date Start/Finish:	9:45AM 6/7/17_10:17AM 6/9/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	4+71.3, 7.3 ft Rt.	**Casing ID/OD:**	HW(4"/4.5")	**Water Level*:**	6.1' bgs @ 7:15 on 6/8/17
Hammer Efficiency Factor:	0.869	**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 140 lb. 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								5.4		R2: Bedrock: Light pink to chalky white, medium to coarse-grained, BIOTITE-HORNBLENDE GRANITE, moderately hard, slightly weathered, fractures are moderately dipping (35 to 55 degrees), closely spaced (2" to 10"), planar to stepped discontinuities, rough to very rough surfaces with manganese oxide coatings. [Charlotte Pluton]. Rock Quality = Fair. R2 Core Times: (min:sec) 72.8-73.8 ft (1:46) 73.8-74.8 ft (1:23) 74.8-75.8 ft (1:04) 75.8-76.8 ft (1:06) 76.8-77.8 ft (1:22) 90% Recovery 77.8	
										Bottom of Exploration at 77.8 feet below ground surface.	
80											
85											
90											
95											
100											

Remarks:

1. 300 lb hammer for 4" casing. Auto hammer starting at 10' for 4" casing.
2. Presence of cobbles is assumed based on drilling behavior.
3. Uncorrected N-value is computed from the blow counts of a 3" spoon sample. The resulting N_{60} value may overestimate the soil density.

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:	Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook	**Boring No.:**	BB-CHAR-102
Location:	Charlotte, Maine	**WIN:**	21686.10

Driller:	New England Boring Contractors	**Elevation (ft.)**	82.8	**Auger ID/OD:**	4" SSA
Operator:	Tom Schaefer	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs./30"
Date Start/Finish:	1:45pm 6/12/17_12:45pm 6/12/17	**Drilling Method:**	SSA & D+W	**Core Barrel:**	NQ-2"
Boring Location:	5+17.9, 5.3 ft Lt.	**Casing ID/OD:**	HW(4"/4.5")	**Water Level*:**	See Remarks
Hammer Efficiency Factor:	0.6	**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.
0							SSA	82.3		6" of pavement. 0.5	
	1D	24/13.2	1.00 - 3.00	6/8/6/4	14	14				Brown, dry, medium dense, SAND, little fine gravel, trace silt, (Fill).	
	2D	24/12	3.00 - 5.00	5/3/2/2	5	5				Brown, dry, loose, medium to coarse SAND, some gravel, trace silt, (Fill).	G#414663 A-1-a, SW-SM WC=4.0%
5	3D	24/9.6	5.00 - 7.00	1/2/3/2	5	5				Brown, dry, loose, medium to coarse SAND, some gravel, trace silt, trace white quartz fragments, (Fill).	
	4D	24/8.4	7.00 - 9.00	1/4/2/3	6	6				Brown, dry, medium stiff, Gravelly SILT, little sand, (Fill).	
							15				
10							16				
							15				
							14				
							14				
							13				
15	5D	24/12	15.00 - 17.00	2/2/4/3	6	6	2	67.8		15.0 5D: Layered, brown, moist, loose, fine to medium SAND with organics, trace silt, odor; grey, moist, loose, fine to medium SAND, trace silt; grey, moist, medium stiff, Clayey SILT, (Glaciomarine Deposit).	
							3				
	6D	24/21.6	17.00 - 19.00	2/2/4/3	6	6	11			6D: Layered, grey, moist, medium stiff, Silty CLAY, trace fine gravel, PP: 0.75, 1.0; TV: 0.325, 0.300; grey, damp, loose, fine SAND, little clay, trace silt, (Glaciomarine Deposit).	G#414668 WC=25.0% LL=31% PL=18% PI=13%
							20				
	7D	24/15.6	19.00 - 21.00	3/3/4/4	7	7	28			Grey, damp, loose, SAND, some silt, (Glaciomarine Deposit).	
20							14				
	8D	24/13.2	21.00 - 23.00	2/5/6/8	11	11	24			Grey, moist, stiff, Sandy SILT, trace clay, (Glaciomarine Deposit).	G#414664 A-4, ML WC=24.9%
							31				
	9D	24/12	23.00 - 25.00	4/4/4/5	8	8	24			Grey, moist, loose, SAND, some silt, (Glaciomarine Deposit).	
							35				
25											

Remarks:

1. Water levels: 7.4' bgs at 17:05 on 06/12/17; 6.7' bgs at 7:10 06/13/17; No water in hole after casing removed.
2. Presence of cobbles is assumed based on drilling behavior.
3. Boring collapsed back to 8.1' when casing removed.
4. 300 lb hammer used for 4" casing installation.

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:** Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook **Location:** Charlotte, Maine	**Boring No.:** BB-CHAR-102 **WIN:** 21686.10

Driller:	New England Boring Contractors	**Elevation (ft.)**	82.8	**Auger ID/OD:**	4" SSA
Operator:	Tom Schaefer	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs./30"
Date Start/Finish:	1:45pm 6/12/17_12:45pm 6/12/17	**Drilling Method:**	SSA & D+W	**Core Barrel:**	NQ-2"
Boring Location:	5+17.9, 5.3 ft Lt.	**Casing ID/OD:**	HW(4"/4.5")	**Water Level*:**	See Remarks
Hammer Efficiency Factor:	0.6	**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 140 lb. 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.
25	10D	24/20.4	25.00 - 27.00	2/3/2/2	5	5				Grey, moist, loose, Silty, fine SAND, trace clay, (Glaciomarine Deposit).	
30	11D	24/13.2	30.00 - 32.00	5/9/9/11	18	18	15			Grey, moist, medium dense, Silty, fine SAND, (Glaciomarine Deposit).	
							19				
							29				
							42				
							49				
35	12D	24/14.4	35.00 - 37.00	5/5/24/26	29	29				Grey, moist, medium dense, Silty, fine SAND, trace clay, (Glaciomarine Deposit).	
							21				
							26	45.8		37.0	
							34				
							57				
40	13D	24/13.2	40.00 - 42.00	15/19/19/23	38[2]	38	54			Grey, moist, hard, SILT, some sand, little clay, little gravel, (Glacial Till). Cobbles from 40' to 45'.[2]	
							64				
							120				
							130				
							89				
45	14D	24/13.2	45.00 - 47.00	14/19/23/28	42	42	OPEN			Dark grey, moist, hard, SILT, some sand, little clay, little gravel, (Glacial Till).	G#414667 A-4, CL WC=11.7%
50											

Remarks:

1. Water levels: 7.4' bgs at 17:05 on 06/12/17; 6.7' bgs at 7:10 06/13/17; No water in hole after casing removed.
2. Presence of cobbles is assumed based on drilling behavior.
3. Boring collapsed back to 8.1' when casing removed.
4. 300 lb hammer used for 4" casing installation.

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: Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook
Location: Charlotte, Maine

Boring No.: BB-CHAR-102
WIN: 21686.10

Driller:	New England Boring Contractors	**Elevation (ft.)**	82.8	**Auger ID/OD:**	4" SSA
Operator:	Tom Schaefer	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs./30"
Date Start/Finish:	1:45pm 6/12/17_12:45pm 6/12/17	**Drilling Method:**	SSA & D+W	**Core Barrel:**	NQ-2"
Boring Location:	5+17.9, 5.3 ft Lt.	**Casing ID/OD:**	HW(4"/4.5")	**Water Level*:**	See Remarks
Hammer Efficiency Factor:	0.6	**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 140 lb. 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	15D	6/4.8	50.00 - 50.50	(50/6")					Dark grey, moist, very dense, Silty SAND, little clay, little gravel, (Glacial Till).	
55	16D	11/10	55.00 - 55.92	44/(50/5")					Dark grey, moist, hard, SILT, some sand, little clay, little gravel and rock fragments, (Glacial Till).	
								23.8	59.0	
	MD	0/0	59.90 - 59.90	(50/0")					Pale red sand/fractured rock from 59' to 60' in wash.	
60								22.8	No penetration with 2" split spoon.	
									60.0	
									Bottom of Exploration at 60.0 feet below ground surface.	
65										
70										
75										

Remarks:

1. Water levels: 7.4' bgs at 17:05 on 06/12/17; 6.7' bgs at 7:10 06/13/17; No water in hole after casing removed.
2. Presence of cobbles is assumed based on drilling behavior.
3. Boring collapsed back to 8.1' when casing removed.
4. 300 lb hammer used for 4" casing installation.

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: Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook
Location: Charlotte, Maine

Boring No.: BB-CHAR-201
WIN: 21686.10

Driller:	Seaboard Drilling LLC	**Elevation (ft.)**	83.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Hanscom/Wall	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/12-14/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	4+57.4, 6.0 ft Lt.	**Casing ID/OD:**	NW(3"/3.5"), HW(4"/4.5")	**Water Level*:**	9.0 ft bgs.
Hammer Efficiency Factor:	1.066	**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.
0							SSA	83.0		2" HMA. 0.2	
5	1D	24/11	5.00 - 7.00	3/3/2/2	5	9				Brown, moist, loose, SAND, some gravel, little silt, (Fill).	
10	MD	24/0	10.00 - 12.00	5/2/3/2	5	9	16			Loose. No sample recovered. Soil similar to 1D on auger flights.	
							13				
							30				
							23	70.2		13.0	
							28				
15	2D	24/18	15.00 - 17.00	2/2/3/5	5	9	29			Grey, wet, loose, Silty, fine SAND, little clay, (Glaciomarine Deposit).	
							34				
							42				
							46				
							55				
20	3D	24/20	20.00 - 22.00	3/3/4/4	7	12	51			Grey, wet, medium dense, Silty, fine SAND, (Glaciomarine Deposit).	G#380946 A-4, SM WC=21.7%
							43				
							48				
							48				
							51				
25											

Remarks:

1) Auto Hammer #367
2) 20.0 ft of NW(3") broken casing abandoned in hole from 65.1 ft bgs (El. 18.1) to 45.1 ft bgs (El. 38.1).

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: Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook
Location: Charlotte, Maine

Boring No.: BB-CHAR-201
WIN: 21686.10

Driller:	Seaboard Drilling LLC	**Elevation (ft.)**	83.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Hanscom/Wall	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/12-14/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	4+57.4, 6.0 ft Lt.	**Casing ID/OD:**	NW(3"/3.5"), HW(4"/4.5")	**Water Level*:**	9.0 ft bgs.
Hammer Efficiency Factor:	1.066	**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 140 lb. 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.
25	4D	24/17	25.00 - 27.00	4/4/4/6	8	14	52			Similar to 3D.	
							96				
							60				
							72	55.2		28.0	
							87				
30	5D	24/16	30.00 - 32.00	5/5/7/7	12	21	81			Grey, wet, medium dense, Silty SAND, little clay, trace gravel, (Glacial Till).	
							163				
							146				
							170				
							212				
35	6D	24/7	35.00 - 37.00	9/10/17/20	27	48	OPEN HOLE			Grey, wet, dense, Silty SAND, little gravel, trace clay, (Glacial Till).	
40	7D	9.6/9.6	40.00 - 40.80	21/58(3.6")	---					Grey, wet, very dense, Silty SAND, some gravel, (Glacial Till). Cobble from 40.8-41.3 ft bgs.	
										Cobble from 42.5-43.2 ft bgs.	
45	8D	12/6	45.00 - 46.00	33/55	---					Similar to 7D.	
50											

Remarks:

1) Auto Hammer #367
2) 20.0 ft of NW(3") broken casing abandoned in hole from 65.1 ft bgs (El. 18.1) to 45.1 ft bgs (El. 38.1).

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: Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook
Location: Charlotte, Maine

Boring No.: BB-CHAR-201
WIN: 21686.10

Driller:	Seaboard Drilling LLC	**Elevation (ft.)**	83.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Hanscom/Wall	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/12-14/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	4+57.4, 6.0 ft Lt.	**Casing ID/OD:**	NW(3"/3.5"), HW(4"/4.5")	**Water Level*:**	9.0 ft bgs.
Hammer Efficiency Factor:	1.066	**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 140 lb. 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.
50										Cobble from 49.9-50.3 ft bgs. Set in NW Casing at 50.0 ft bgs.	
	9D	24/17	51.00 - 53.00	19/28/29/40	57	101				Grey, wet, very dense, SAND, some silt, trace gravel, (Glacial Till).	
55	10D	4.8/4.8	55.00 - 55.40	70(4.8")	---					Grey, wet, very dense, Silty SAND, some gravel, (Glacial Till).	
								24.2		59.0	
60	11D	6/5	60.00 - 60.50	90	---					Pink to brown, wet, very dense, SAND, some gravel, trace silt, (Weathered Bedrock).	
65	12D	1.2/1.2	65.00 - 65.10	50(1.2")	---			18.1		Pink to grey, wet, very dense, GRAVEL, some sand, trace silt (Weathered Bedrock).	
										65.1	
										Bottom of Exploration at 65.1 feet below ground surface. Broke NW(3") Casing, moved to BB-CHAR-201A. 20.0 ft of NW(3") broken casing abandoned in hole from 65.1 ft bgs (El. 18.1) to 45.1 ft bgs (El. 38.1) .	
70											
75											

Remarks:

1) Auto Hammer #367
2) 20.0 ft of NW(3") broken casing abandoned in hole from 65.1 ft bgs (El. 18.1) to 45.1 ft bgs (El. 38.1).

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: Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook Location: Charlotte, Maine	Boring No.: BB-CHAR-201A WIN: 21686.10

Driller:	Seaboard Drilling LLC	**Elevation (ft.)**	83.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Jason	**Datum:**	NAVD88	**Sampler:**	N/A
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	3/13/2024-3/14/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	4+55.9, 6.1 ft Lt.	**Casing ID/OD:**	NW-3" & HW-4"	**Water Level*:**	9.5 ft bgs.
Hammer Efficiency Factor:	1.066	**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.
0							DRIVE HW			Drove HW Casing to 35.0 ft bgs. See BB-CHAR-201 for soil descriptions.	
5											
10											
15											
20											
25											

Remarks:

1) Auto Hammer #367
2) 20.0 ft of NW(3") broken casing abandoned in hole from 64.8 ft bgs (El. 19.2) to 44.8 ft bgs (El. 39.2).

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:	Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook			**Boring No.:**	BB-CHAR-201A
Location:	Charlotte, Maine			**WIN:**	21686.10
Driller:	Seaboard Drilling LLC	**Elevation (ft.)**	83.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Jason	**Datum:**	NAVD88	**Sampler:**	N/A
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	3/13/2024-3/14/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	4+55.9, 6.1 ft Lt.	**Casing ID/OD:**	NW-3" & HW-4"	**Water Level*:**	9.5 ft bgs.
Hammer Efficiency Factor:	1.066	**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 140 lb. 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.
25											
30											
35							OPEN HOLE				
40											
45											
50											

Remarks:

1) Auto Hammer #367
2) 20.0 ft of NW(3") broken casing abandoned in hole from 64.8 ft bgs (El. 19.2) to 44.8 ft bgs (El. 39.2).

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:** Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook **Location:** Charlotte, Maine	**Boring No.:** BB-CHAR-201A **WIN:** 21686.10

Driller:	Seaboard Drilling LLC	**Elevation (ft.)**	83.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Jason	**Datum:**	NAVD88	**Sampler:**	N/A
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	3/13/2024-3/14/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	4+55.9, 6.1 ft Lt.	**Casing ID/OD:**	NW-3" & HW-4"	**Water Level*:**	9.5 ft bgs.
Hammer Efficiency Factor:	1.066	**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 140 lb. 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.
50											
55											
60											
								19.2		64.0	
	R1	60/42	64.80 - 69.80	RQD = 7%			NQ-2			Top of Bedrock at Elev. 19.2 ft. Roller Coned ahead to 64.8 ft bgs. Set in NW casing to 64.8 ft bgs. R1: Bedrock: Pink, grey and brown, medium to coarse-grained, BIOTITE-HORNBLENDE GRANITE, soft to moderately hard, moderately to severely weathered, with significant zones decomposed to sand, joints dipping at horizontal to moderate angles, spaced very close. [Charlotte Pluton] Rock Quality = Very Poor. R1: Core Times (min:sec) 64.8-65.8 ft (1:43) 65.8-66.8 ft (2:21) 66.8-67.8 ft (2:07) 67.8-68.8 ft (2:06) 68.8-69.8 ft (2:55) 70% Recovery R2: Bedrock: Pink to grey, medium to coarse-grained, BIOTITE-HORNBLENDE GRANITE, moderately hard, moderately weathered, joints dipping at horizontal to low angles, closely spaced, then pink, grey and brown, medium to coarse-grained, BIOTITE-	
65											
	R2	42/34	69.80 - 73.30	RQD = 24%							
70											
	R3	60/60	73.30 - 78.30	RQD = 55%							
75											

Remarks:

1) Auto Hammer #367
2) 20.0 ft of NW(3") broken casing abandoned in hole from 64.8 ft bgs (El. 19.2) to 44.8 ft bgs (El. 39.2).

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: Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook
Location: Charlotte, Maine

Boring No.: BB-CHAR-201A

WIN: 21686.10

Driller:	Seaboard Drilling LLC	**Elevation (ft.)**	83.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Jason	**Datum:**	NAVD88	**Sampler:**	N/A
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	3/13/2024-3/14/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	4+55.9, 6.1 ft Lt.	**Casing ID/OD:**	NW-3" & HW-4"	**Water Level*:**	9.5 ft bgs.
Hammer Efficiency Factor:	1.066	**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 140 lb. 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										HORNBLENDE GRANITE, severely weathered and decomposed to sand. [Charlotte Pluton] Rock Quality = Very Poor. R2: Core Times (min:sec) 69.8-70.8 ft (1:53) 70.8-71.8 ft (1:53) 71.8-72.8 ft (2:00) 72.89-73.3 ft (3:00) Core Blocked 81% Recovery	
								4.9			
80										R3: Bedrock: Pink to grey, medium to coarse-grained, BIOTITE-HORNBLENDE GRANITE, moderately hard to hard, slightly weathered, joints dipping at horizontal to moderate angles, closely spaced, then pink, grey and brown, medium to coarse-grained, GRANITE, soft to moderately hard, severly weathered, with a zone decomposed to sand, joints dipping at horizontal to low angles, spaced very close. [Charlotte Pluton] Rock Quality = Fair. R3: Core Times (min:sec) 73.3-74.3 ft (1:36) 74.3-75.3 ft (2:02) 75.3-76.3 ft (2:42) 76.3-77.3 ft (2:41) 77.3-78.3 ft (2:50) 100% Recovery	
85										78.3	
										Bottom of Exploration at 78.3 feet below ground surface. NW(3") Casing broke while pulling. 20.0 ft of NW(3") broken casing abandoned in hole from 64.8 ft bgs (El. 19.2) to 44.8 ft bgs (El. 39.2).	
90											
95											
100											

Remarks:

1) Auto Hammer #367
2) 20.0 ft of NW(3") broken casing abandoned in hole from 64.8 ft bgs (El. 19.2) to 44.8 ft bgs (El. 39.2).

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: Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook
Location: Charlotte, Maine

Boring No.: BB-CHAR-202
WIN: 21686.10

Driller:	MaineDOT	**Elevation (ft.)**	82.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/12/2024-3/13/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+31.9, 9.5 ft Rt.	**Casing ID/OD:**	HW-4"	**Water Level*:**	
Hammer Efficiency Factor: 0.962		**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.
0	1D	24/14	0.00 - 2.00	4/6/6/7	12	19	SSA			Brown, moist, medium dense, SAND, little gravel, trace silt, (Fill).	
5	2D	24/5	5.00 - 7.00	1/1/1/1	2	3				Brown, moist, very loose, Gravelly SAND, trace silt, (Fill).	G#380947 A-1-a, SW-SM WC=7.3%
								74.0		8.5 Grey-brown silt and clay on auger flights from 8.5 to 10.0 ft bgs.	
10	3D	24/24	10.00 - 12.00	1/1/1/1	2	3	6			Brown, wet, soft, SILT, little peat, little sand, (Wetland Deposit).	#380948 Organic Content 18.4% WC=112.9%
							5				
							10			Silt, peat, and sand observed in wash from 10.0 to 13.0 ft bgs.	
							20	69.5		13.0	
							28				
15	4D/A	24/24	15.00 - 17.00	2/7/9/8	16	26	18			4D (15.0-16.0 ft bgs.) Grey, wet, very stiff, SILT, some clay, little fine sand, (Glaciomarine Deposit). 4D/A (16.0-17.0 ft bgs.) Grey, wet, medium dense, Silty, fine SAND, trace clay, (Glaciomarine Deposit).	G#380949 A-4, CL WC=23.5% Non-Plastic
							25				
							20				
							38				
							101				
20	5D	24/14	20.00 - 22.00	4/4/3/6	7	11	52			Grey, wet, medium dense, fine SAND, some silt, trace clay, (Glaciomarine Deposit).	
							45				
							50				
							35				
25							38				

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: Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook
Location: Charlotte, Maine

Boring No.: BB-CHAR-202
WIN: 21686.10

Driller:	MaineDOT	**Elevation (ft.)**	82.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/12/2024-3/13/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+31.9, 9.5 ft Rt.	**Casing ID/OD:**	HW-4"	**Water Level*:**	
Hammer Efficiency Factor:	0.962	**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 140 lb. 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.
25	6D	24/20	25.00 - 27.00	4/4/6/6	10	16	44			Grey, wet, medium dense, Silty, fine SAND, (Glaciomarine Deposit).	
							42				
							45				
							51				
							46				
30	7D	24/14	30.00 - 32.00	2/3/4/3	7	11	47			Similar to 6D.	
							45				
							50				
							65				
							83	48.0		34.5	
35	8D	24/20	35.00 - 37.00	6/10/13/13	23	37	OPEN HOLE			Grey, wet, hard, SILT, some sand, trace gravel, trace clay, (Glacial Till).	
40	9D	24/6	40.00 - 42.00	10/12/11/61	23	37				Grey, wet, hard, Sandy SILT, some gravel, (Glacial Till).	
45	10D	24/9	45.00 - 47.00	7/9/12/18	21	34				Similar to 9D.	
50											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook
Location: Charlotte, Maine

Boring No.: BB-CHAR-202

WIN: 21686.10

Driller:	MaineDOT	**Elevation (ft.)**	82.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/12/2024-3/13/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+31.9, 9.5 ft Rt.	**Casing ID/OD:**	HW-4"	**Water Level*:**	
Hammer Efficiency Factor:	0.962	**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 140 lb. 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.
50	11D	24/13	50.00 - 52.00	20/17/28/31	45	72				Grey, wet, very dense, Silty SAND, some gravel, (Glacial Till).	
55	12D	24/5	55.00 - 57.00	26/33/43/56	76	122				Similar to 11D.	
								24.7		57.8	
	R1	60/20	58.30 - 63.30	RQD = 0%	---		NQ-2			Top of Bedrock at Elev. 24.7 ft. Pink granite sand in wash at 57.8 ft bgs. R1: Bedrock: Pink, grey and brown, medium to coarse-grained, BIOTITE-HORNBLENDE GRANITE, soft to moderately hard, moderately to severely weathered, with zones decomposed to sand, joints dipping at horizontal to vertical angles, spaced very close. [Charlotte Pluton] Rock Quality = Very Poor. R1: Core Times (min:sec) 58.3-59.3 ft (1:13) 59.3-60.3 ft (1:28) 60.3-61.3 ft (1:22) 61.3-62.3 ft (3:38) 62.3-63.3 ft (1:31) 33% Recovery	
	MD	1.2/0	58.30 - 58.40	50(1.2")							
60											
	R2	56.4/28	63.30 - 68.00	RQD = 0%						R2: Bedrock: Similar to R1. [Charlotte Pluton] Rock Quality = Very Poor. R2: Core Times (min:sec) 63.3-64.3 ft (1:59) 64.3-65.3 ft (1:37) 65.3-66.3 ft (2:36) 66.3-68.0 ft (3:24) Core Blocked 49% Recovery	
65											
	R3	81.6/62	68.00 - 74.80	RQD = 38%						R3: Bedrock: Pink to grey, medium to coarse-grained, BIOTITE-HORNBLENDE GRANITE, moderately hard, moderately weathered, joints dipping at horizontal to moderate angles, closely spaced. [Charlotte Pluton] Rock Quality = Poor. R3: Core Times (min:sec) 68.0-69.0 ft (1:55) 69.0-70.0 ft (0:47) 70.0-71.0 ft (1:53)	
70											
75								7.7			

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: Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook Location: Charlotte, Maine	Boring No.: BB-CHAR-202 WIN: 21686.10

Driller:	MaineDOT	**Elevation (ft.)**	82.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/12/2024-3/13/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+31.9, 9.5 ft Rt.	**Casing ID/OD:**	HW-4"	**Water Level*:**	
Hammer Efficiency Factor:	0.962	**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 140 lb. 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										71.0-72.0 ft (2:29) 72.0-73.0 ft (2:39) 73.0-74.0 ft (2:53) 74.0-74.8 ft (2:58) 76% Recovery	
										74.8	
										Bottom of Exploration at 74.8 feet below ground surface.	
80											
85											
90											
95											
100											

Remarks:

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

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

Appendix B

Rock Core Photographs

MaineDOT
Moosehorn Bridge #3332 Carries Station Road Over Moosehorn Brook
Charlotte, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-CHAR-101	R1	68.0-72.8	58	17	4	6	GRANITE	1
BB-CHAR-101	R2	72.8-77.8	60	54	41	68	GRANITE	1+2

Notes: 1. “Box row” indicates the section of the box where the core run is contained: 1 = top, 4 = bottom.
2. Top of each core run is on the left and increases with depth to the right.
3. Transition between core runs is marked by wooden blocks.

MaineDOT

Moosehorn Bridge #3332 Carries Station Road Over Moosehorn Brook
Charlotte, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-CHAR-201A	R1	64.8-69.8	60	42	4	7	GRANITE	1
BB-CHAR-201A	R2	69.8-73.3	42	34	10	24	GRANITE	2
BB-CHAR-201A	R3	73.3-78.3	60	60	33	55	GRANITE	3

Notes: 1. "Box row" indicates the section of the box where the core run is contained: 1 = top, 4 = bottom.
2. Top of each core run is on the left and increases with depth to the right.

MaineDOT
Moosehorn Bridge #3332 Carries Station Road Over Moosehorn Brook
Charlotte, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-CHAR-202	R1	58.3-63.3	60	20	0	0	GRANITE	1
BB-CHAR-202	R2	63.3-68.0	56.4	28	0	0	GRANITE	2
BB-CHAR-202	R3	68.0-74.8	81.6	62	31	38	GRANITE	3

Notes: 1. “Box row” indicates the section of the box where the core run is contained: 1 = top, 4 = bottom.
2. Top of each core run is on the left and increases with depth to the right.

Appendix C

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Charlotte   Work Number: 21686.10

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-CHAR-201, 3D	4+57.4	6.0 Lt.	20.0-22.0	380946	1	21.7			SM	A-4	III
BB-CHAR-202, 2D	5+31.9	9.5 Rt.	5.0-7.0	380947	1	7.3			SW-SM	A-1-a	0
BB-CHAR-202, 3D	5+31.9	9.5 Rt.	10.0-12.0	380948	---	112.9	Loss on Ignition, (T 267) 18.4%				
BB-CHAR-202, 4D	5+31.9	9.5 Rt.	15.0-17.0	380949	1	23.5	-N	P-	CL	A-4	IV
BB-CHAR-101, 4D	4+71.3	7.3 Rt.	14.0-16.0	414662	2	10.7			SP	A-1-a	0
BB-CHAR-101, 7D	4+71.3	7.3 Rt.	25.0-27.0	414665	3	15.9			SM	A-4	III
BB-CHAR-101, 10D	4+71.3	7.3 Rt.	40.0-42.0	414666	4	4.2			GM	A-1-a	0
BB-CHAR-102, 2D	5+17.9	5.3 Lt.	3.0-5.0	414663	5	4.0			SW-SM	A-1-a	0
BB-CHAR-102, 6DA	5+17.9	5.3 Lt.	17.0-19.0	414668	---	25	31	13	CL	A-4	III
BB-CHAR-102, 8D	5+17.9	5.3 Lt.	21.0-23.0	414664	6	24.9			ML	A-4	IV
BB-CHAR-102, 14D	5+17.9	5.3 Lt.	45.0-47.0	414667	7	11.7			CL	A-4	IV

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

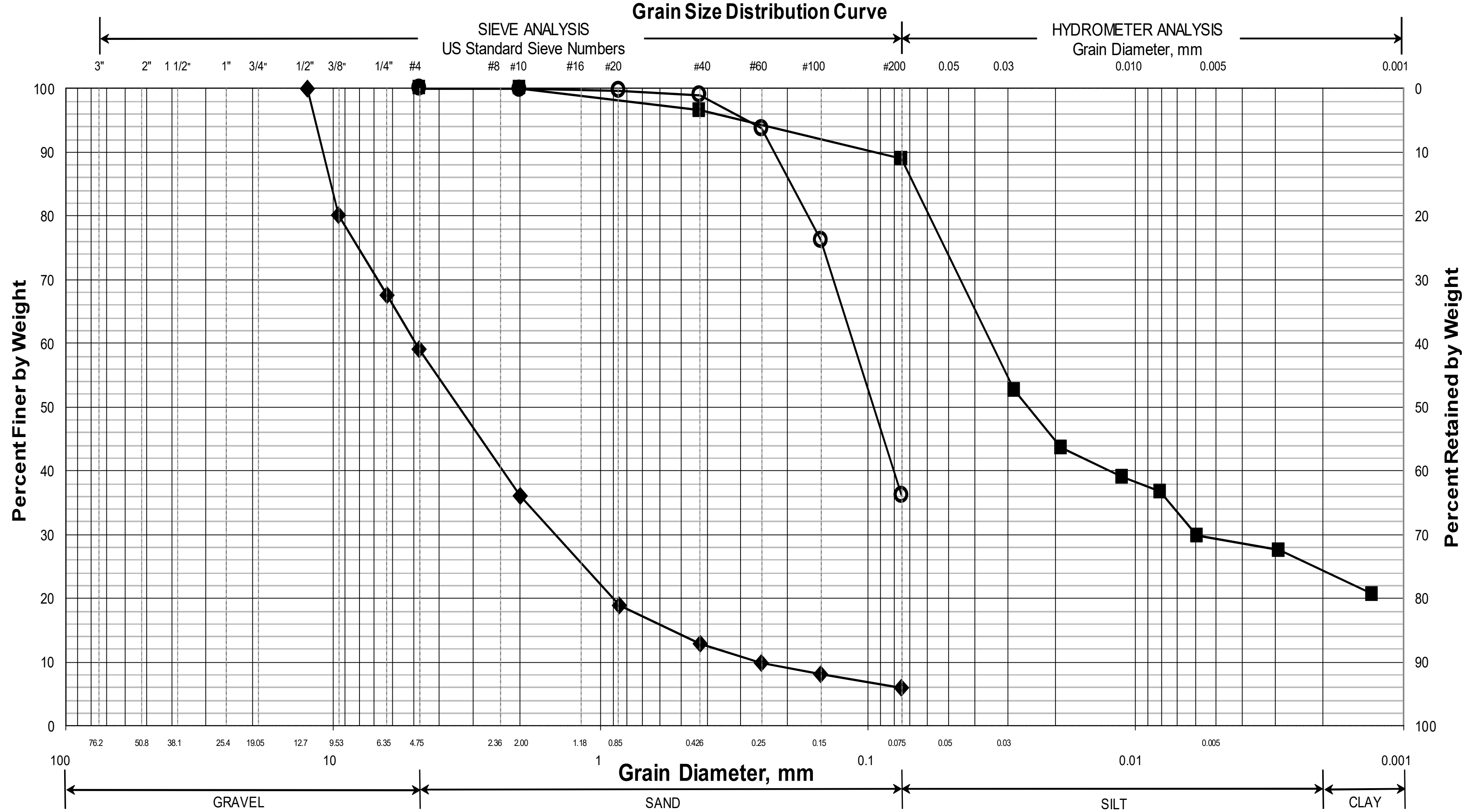
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

Maine Department of Transportation
Grain Size Distribution Curve



UNIFIED CLASSIFICATION

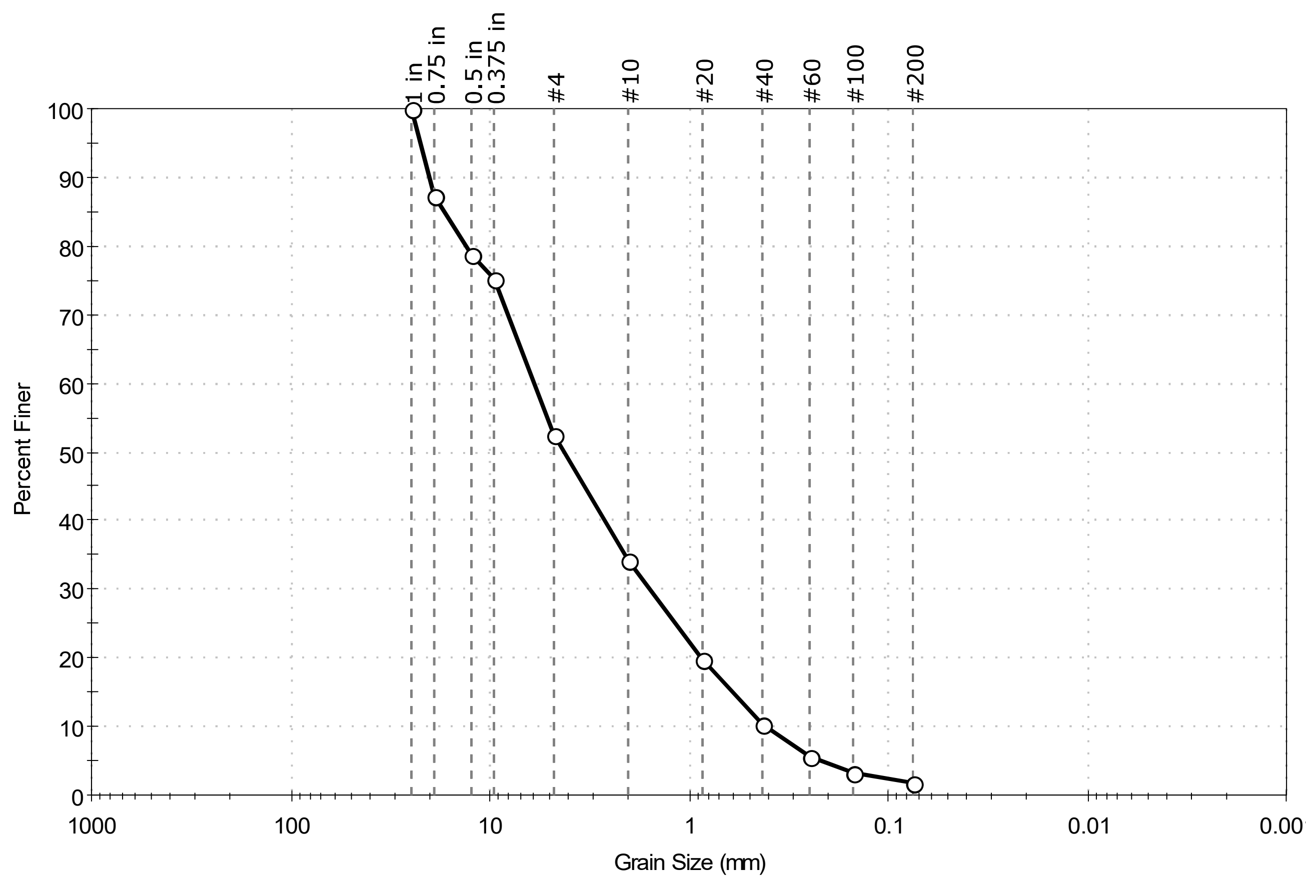
	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-CHAR-201/3D	4+57.4	6.0 LT	20.0-22.0	Silty SAND.	21.7			
◆	BB-CHAR-202/2D	5+31.9	9.5 RT	5.0-7.0	Gravelly SAND, trace silt.	7.3			
■	BB-CHAR-202/4D	5+31.9	9.5 RT	15.0-17.0	SILT, some clay, little sand.	23.5			NP
●									
▲									
×									

WIN	
021686.10	
Town	
Charlotte	
Reported by/Date	
WHITE, TERRY A	4/19/2024

SHEET 1

Client: Golder Associates			
Project: Station Road Culvert Replacement			
Location: Charlotte, ME		Project No: GTX-306601	
Boring ID: BB-CHAR-101	Sample Type: jar	Tested By: jbr	
Sample ID: 4D	Test Date: 06/26/17	Checked By: emm	
Depth : 14-16 ft	Test Id: 414662		
Test Comment: ---			
Visual Description: Moist, dark gray sand with gravel			
Sample Comment: ---			

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	47.6	50.7	1.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	87		
0.5 in	12.50	79		
0.375 in	9.50	75		
#4	4.75	52		
#10	2.00	34		
#20	0.85	20		
#40	0.42	10		
#60	0.25	6		
#100	0.15	3		
#200	0.075	1.7		

Coefficients

D_{85} =16.9377 mm D_{30} =1.5593 mm
D_{60} =5.9824 mm D_{15} =0.5961 mm
D_{50} =4.2337 mm D_{10} =0.4095 mm
C_u =14.609 C_c =0.992

Classification

ASTM Poorly graded sand with gravel (SP)

AASHTO Stone Fragments, Gravel and Sand (A-1-a (1))

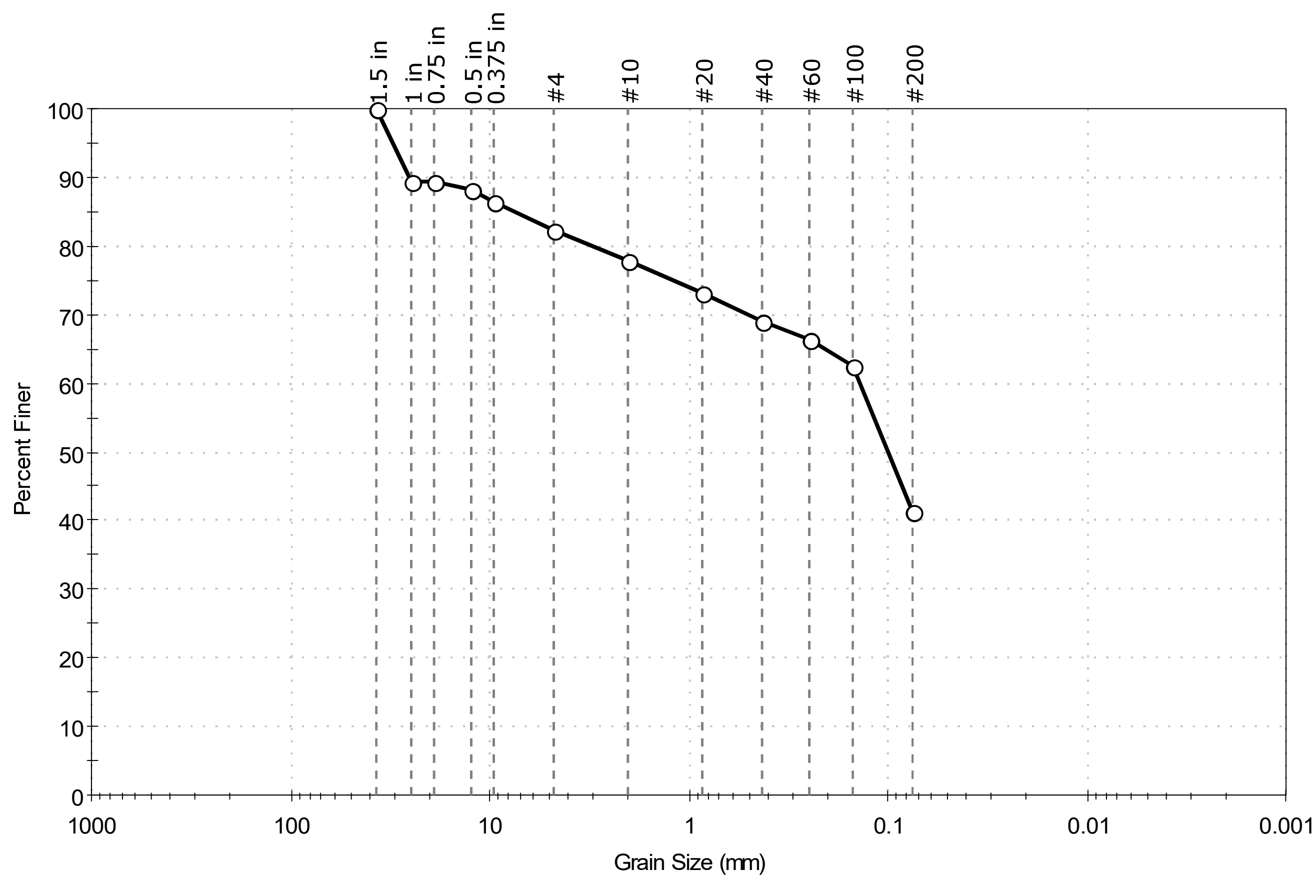
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Golder Associates		
Project:	Station Road Culvert Replacement		
Location:	Charlotte, ME	Project No:	GTX-306601
Boring ID: BB-CHAR-101	Sample Type: jar	Tested By:	jbr
Sample ID: 7D	Test Date: 06/26/17	Checked By:	emm
Depth : 25-27 ft	Test Id: 414665		
Test Comment:	---		
Visual Description:	Moist, olive silty sand with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	17.6	41.0	41.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	89		
0.75 in	19.00	89		
0.5 in	12.50	88		
0.375 in	9.50	86		
#4	4.75	82		
#10	2.00	78		
#20	0.85	73		
#40	0.42	69		
#60	0.25	66		
#100	0.15	62		
#200	0.075	41		

Coefficients

D_{85} =7.5402 mm | D_{30} =N/A
D_{60} =0.1384 mm | D_{15} =N/A
D_{50} =0.0995 mm | D_{10} =N/A
C_u =N/A | C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

printed 6/28/2017 4:20:11 PM

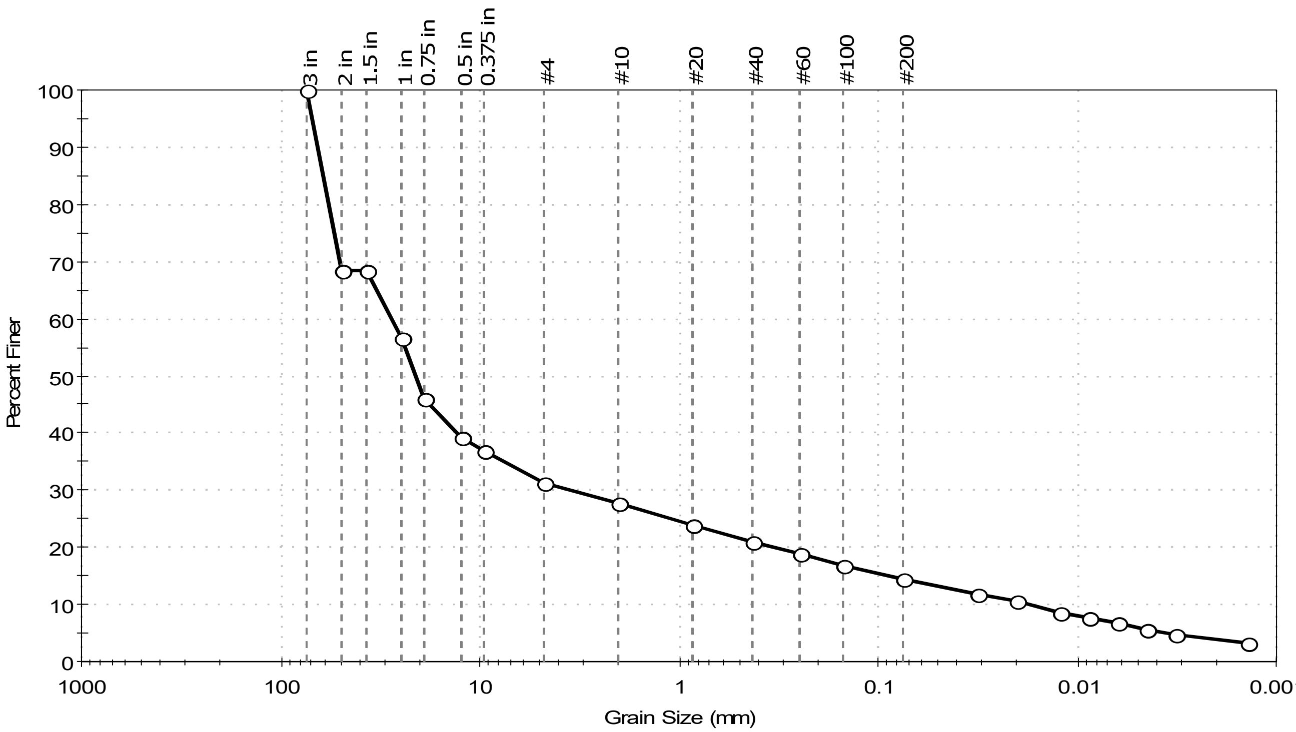
Client:	Golder Associates		
Project:	Station Road Culvert Replacement		
Location:	Charlotte, ME	Project No:	GTX-306601
Boring ID: BB-CHAR-101	Sample Type: jar	Tested By:	jbr
Sample ID: 10D	Test Date: 06/27/17	Checked By:	emm
Depth : 40-42 ft	Test Id: 414666		

Test Comment: ---

Visual Description: Moist, olive clayey gravel with sand

Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	68.8	16.7	14.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
3 in	75.00	100		
2 in	50.00	68		
1.5 in	37.50	68		
1 in	25.00	57		
0.75 in	19.00	46		
0.5 in	12.50	39		
0.375 in	9.50	37		
#4	4.75	31		
#10	2.00	28		
#20	0.85	24		
#40	0.42	21		
#60	0.25	19		
#100	0.15	17		
#200	0.075	14		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0315	12		
---	0.0204	10		
---	0.0123	9		
---	0.0088	8		
---	0.0063	7		
---	0.0045	6		
---	0.0032	5		
---	0.0014	3		

Coefficients

D_{85} =61.8853 mm D_{30} =3.5615 mm

D_{60} =28.1010 mm D_{15} =0.0874 mm

D_{50} =21.0464 mm D_{10} =0.0179 mm

C_u =1569.888 C_c =25.217

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-a (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Dispersion Device : Apparatus A - Mech Mixer

Dispersion Period : 1 minute

Specific Gravity : 2.65

Separation of Sample: #200 Sieve

printed 6/28/2017 4:20:12 PM

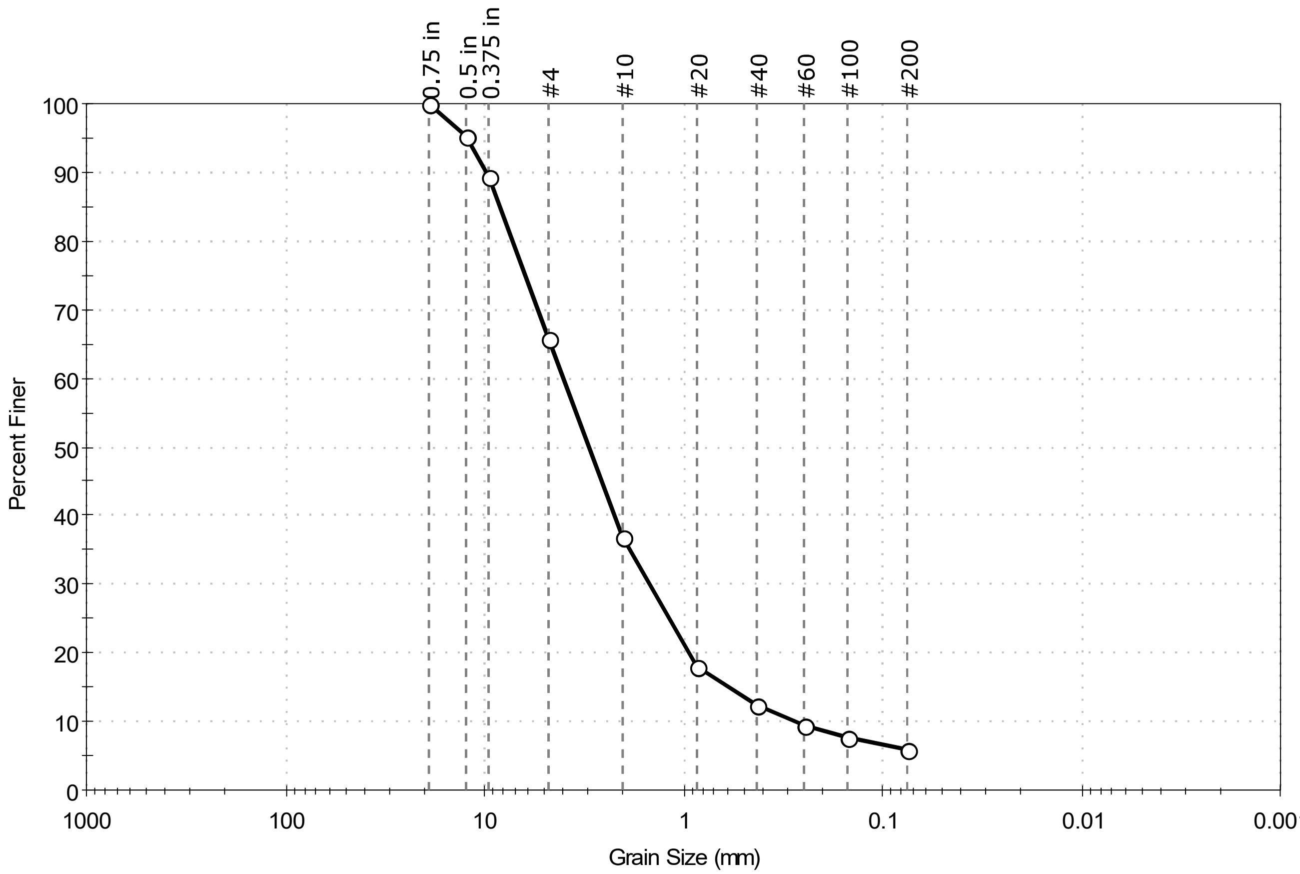
Client:	Golder Associates		
Project:	Station Road Culvert Replacement		
Location:	Charlotte, ME	Project No:	GTX-306601
Boring ID: BB-CHAR-102	Sample Type: jar	Tested By:	jbr
Sample ID: 2D	Test Date: 06/26/17	Checked By:	emm
Depth : 3-5 ft	Test Id: 414663		

Test Comment: ---

Visual Description: Moist, grayish brown sand with silt and gravel

Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	34.1	60.1	5.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	95		
0.375 in	9.50	89		
#4	4.75	66		
#10	2.00	37		
#20	0.85	18		
#40	0.42	12		
#60	0.25	10		
#100	0.15	8		
#200	0.075	5.8		

Coefficients

D_{85} =8.3398 mm D_{30} =1.4650 mm

D_{60} =3.9837 mm D_{15} =0.5911 mm

D_{50} =2.9581 mm D_{10} =0.2719 mm

C_u =14.651 C_c =1.981

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-a (1))

Sample/Test Description

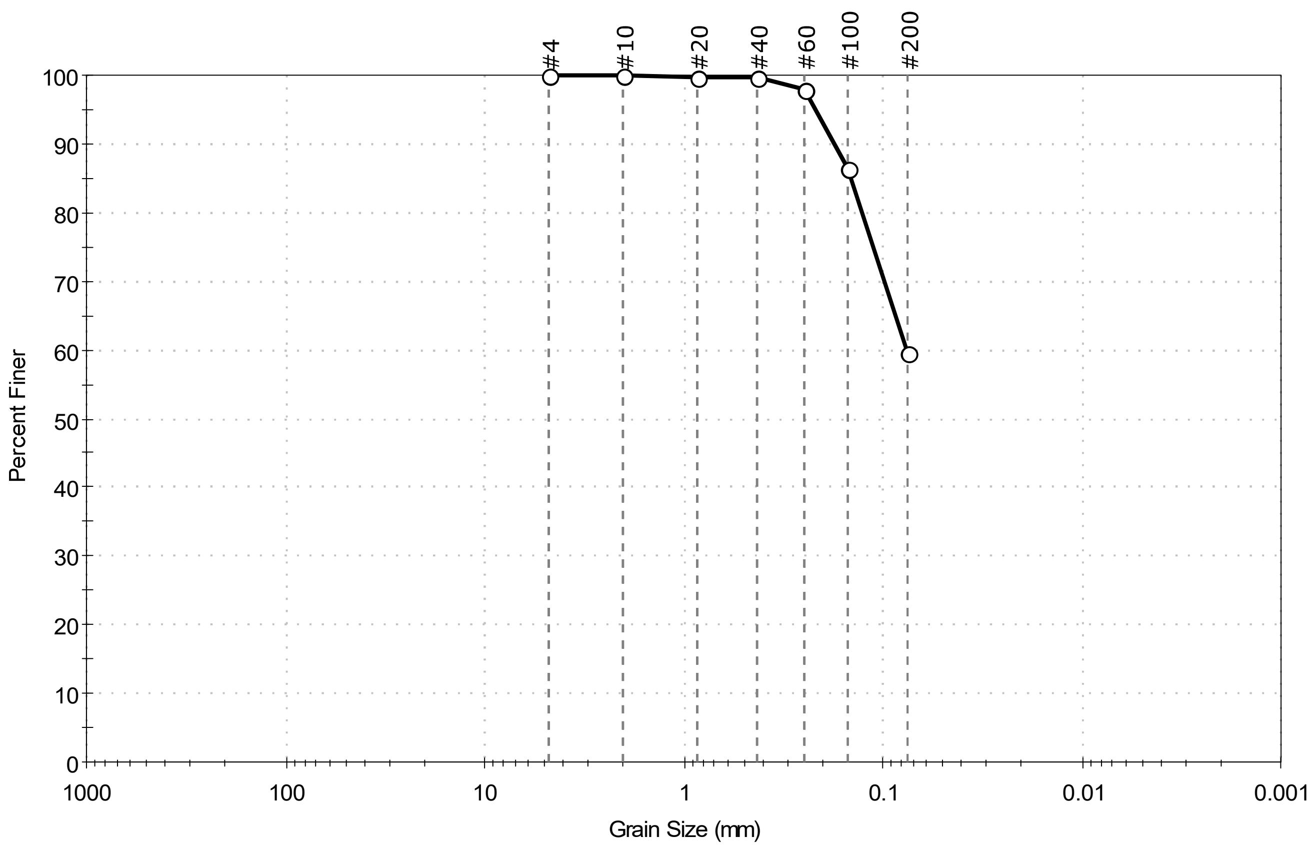
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

printed 6/28/2017 4:20:13 PM

Client:	Golder Associates			
Project:	Station Road Culvert Replacement			
Location:	Charlotte, ME		Project No:	GTX-306601
Boring ID:	BB-CHAR-102	Sample Type: jar	Tested By:	jbr
Sample ID:	8D	Test Date: 06/26/17	Checked By:	emm
Depth :	21-23 ft	Test Id: 414664		
Test Comment:	---			
Visual Description:	Moist, olive sandy silt			
Sample Comment:	---			

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	40.3	59.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	98		
#100	0.15	86		
#200	0.075	60		

Coefficients

D_{85} =0.1447 mm
D_{30} =N/A
D_{60} =0.0756 mm
D_{15} =N/A
D_{50} =N/A
D_{10} =N/A
C_u =N/A
C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

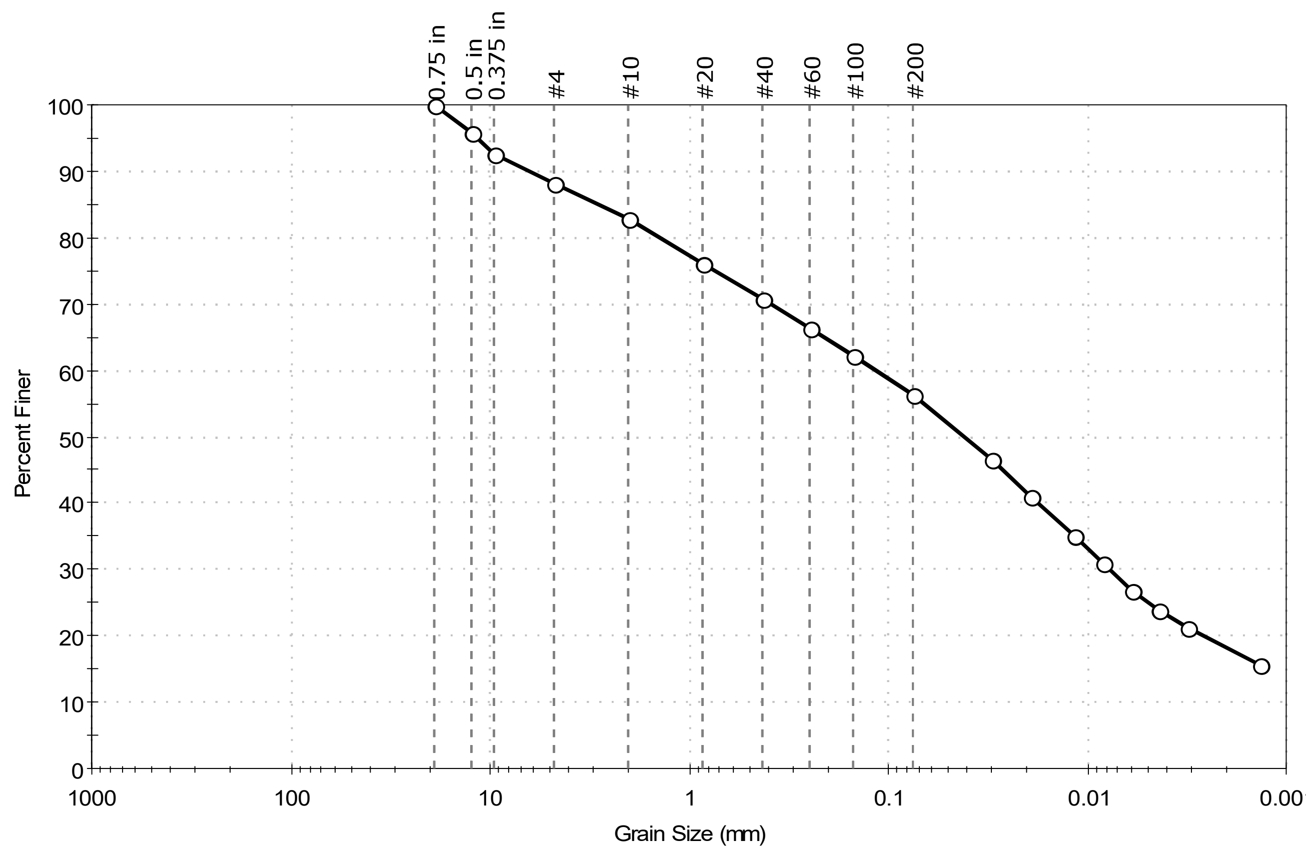
Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

printed 6/28/2017 4:20:14 PM

Client: Golder Associates			
Project: Station Road Culvert Replacement			
Location: Charlotte, ME		Project No:	GTX-306601
Boring ID: BB-CHAR-102	Sample Type: jar	Tested By:	jbr
Sample ID: 14D	Test Date: 06/27/17	Checked By:	emm
Depth : 45-47 ft	Test Id: 414667		
Test Comment: ---			
Visual Description: Moist, olive sandy clay			
Sample Comment: ----			

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	11.7	31.9	56.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	96		
0.375 in	9.50	93		
#4	4.75	88		
#10	2.00	83		
#20	0.85	76		
#40	0.42	71		
#60	0.25	66		
#100	0.15	62		
#200	0.075	56		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0301	47		
---	0.0191	41		
---	0.0117	35		
---	0.0084	31		
---	0.0060	27		
---	0.0044	24		
---	0.0031	21		
---	0.0014	16		

Coefficients

D_{85} = 2.8100 mm D_{30} = 0.0078 mm
D_{60} = 0.1156 mm D_{15} = N/A
D_{50} = 0.0415 mm D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Dispersion Device : Apparatus A - Mech Mixer

Dispersion Period : 1 minute

Specific Gravity : 2.65

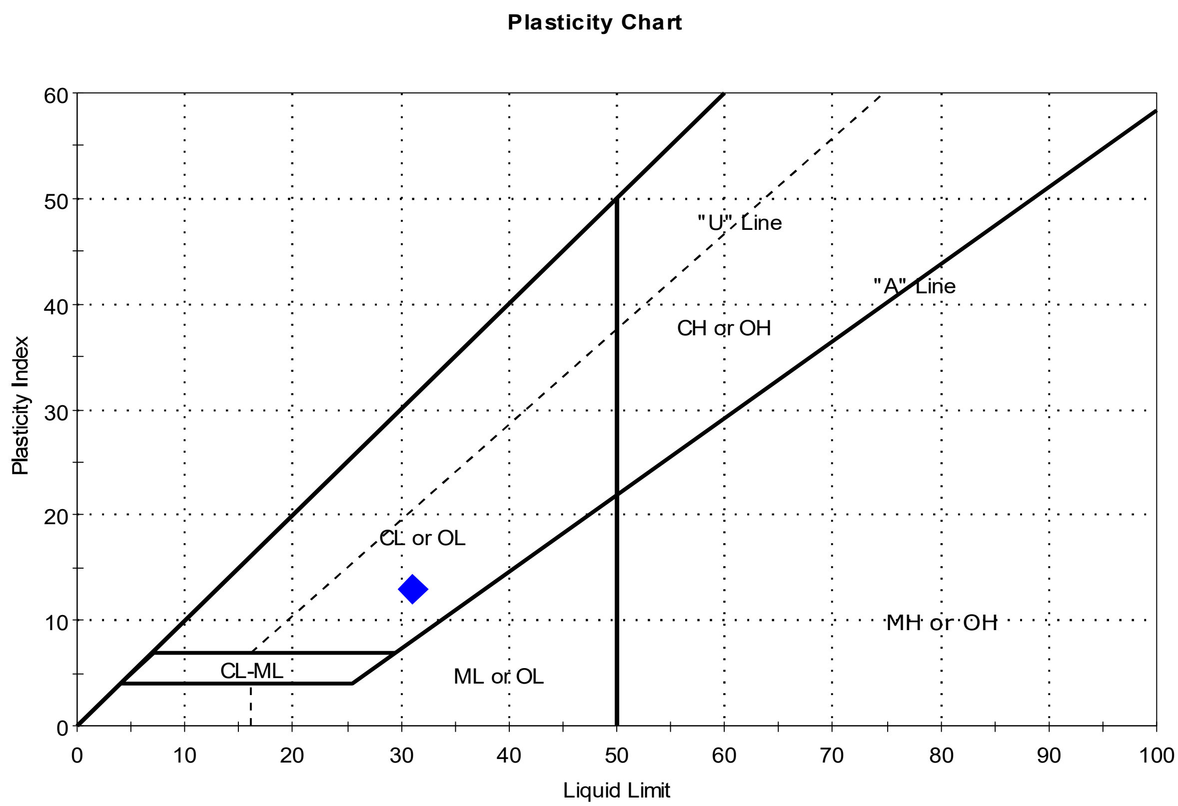
Separation of Sample: #200 Sieve

printed 6/28/2017 4:20:14 PM

Client:	Golder Associates				
Project:	Station Road Culvert Replacement				
Location:	Charlotte, ME			Project No:	GTX-306601
Boring ID:	BB-CHAR-102	Sample Type:	jar	Tested By:	cam
Sample ID:	6DA	Test Date:	06/28/17	Checked By:	emm
Depth :	17-19 ft	Test Id:	414668		

Test Comment: ---
Visual Description: Moist, olive clay
Sample Comment: ---

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	6DA	;-CHAR-1(	17-19 ft	25	31	18	13	0.5	

Sample Prepared using the WET method

Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

printed 6/28/2017 3:27:20 PM

Appendix D

Calculations

Liquidity Index and Sensitivity

Liquidity Index

$$LI := \frac{WC - PL}{LL - PL}$$

Das, Principles of Engineering, 7th Edition, Equation 4.16

BB-CHAR-102, 6D

$WC := 25.0$

$LL := 31$

$PL := 18$

$$LI := \frac{WC - PL}{LL - PL} = 0.54$$

Driven H-Pile Resistance

Design of H-piles

Reference: AASHTO LRFD Bridge Design Specifications, 9th Edition, 2020.

Bedrock Properties

BB-CHAR-101, R1 RQD = 6%, R2 RQD = 68%
Rock Type: GRANITE, moderately hard, moderately weathered to slightly weathered

BB-CHAR-201A, R1 RQD = 7%, R2 RQD = 24%, R3 RQD = 55%
Rock Type: GRANITE, soft to hard, severely weathered to slightly weathered

BB-CHAR-202, R1 RQD = 0%%, R2 RQD = 0%, R3 RQD = 38%
Rock Type: GRANITE, soft to moderately hard, severely weathered to slightly weathered

Granite Co = 2,100-49,000 psi

(AASHTO Standard Specifications for Bridges 17th Edition, Table 4.4.8.1.2B)

For Design Purposes: Assume pile tip bears on competent bedrock.
For Drivability Analysis: Assume pile tip ends on weathered bedrock.

Pile Properties

Use the following piles: 14x89

$A_g := 26.1\cdot in^2$ $\quad d := 13.8\cdot in$ $\quad b := 14.7\cdot in$ $\quad t_f := 0.615in$ $\quad t_w := t_f$

$A_{box} := d\cdot b$ $\quad A_{box} = 202.86\cdot in^2$

r_s= radius of gyration $\quad r_s := 3.53in$ $\quad$ radius of gyration about the Y-Y or weak axis per LRFD Article C6.9.4.1.2.

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

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

Check For Slender Members

Check that pile selections are composed of nonslender elements per LRFD 6.9.4.2

LRFD eq. 6.9.4.2.1-1

$$\frac{b}{t} \le \lambda_r$$

From Table 6.9.4.2.1-1:

For flanges: $\lambda_{rf} := 0.56 \cdot \sqrt{\frac{E}{F_y}}$ where b_f = Half-flange width

$\lambda_{rf} = 13.487$ $b_f := 0.5 \cdot b$ $b_f = 7.35 \cdot in$

$\frac{b_f}{t_f} = 11.951$ H-pile size is nonslender for flange members

For webs: $\lambda_{rw} := 1.09 \sqrt{\frac{E}{F_y}}$ where b_w = Web height/distance between flanges

$\lambda_{rw} = 26.251$ $b_w := d - 2 \cdot t_f$ $b_w = 12.57 \cdot in$

$\frac{b_w}{t_w} = 20.439$ H-Pile size is nonslender for web members

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

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

Nominal Axial Structural Resistance

Determine equivalent yield resistance $P_o := F_y \cdot A_g$ LRFD Article 6.9.4.1.1.

$P_o = 1305 \cdot kip$

Per VTrans Integral Abutment Design Guideline, the controlling SPR (Structural Pile Resistance) will be the lowest axial capacity (P_r) of the top segment or the second segment of the upper zone or the lower zone of the pile. The SPR will be compared with the applied axial load.

A. Structural Resistance of lower "braced" segment of pile

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

K = effective length factor $K_{eff} := 0.65$ LRFD Table C4.6.2.5-1. Use K=0.65 for assumed segment in pure compression. Fixed top and bottom

l = "unbraced" length $l_{unbraced\_bot} := 0.1 \cdot ft$ Assume in pure compression

LRFD eq. 6.9.4.1.2-1

$$P_e := \frac{\pi^2 \cdot E}{\left(\frac{K_{eff} \cdot l_{unbraced\_bot}}{r_s}\right)^2} \cdot A_g \qquad P_e = 1.53 \times 10^8 \cdot kip$$

LRFD Article 6.9.4.1.1 For compressive members with nonslender element cross-sections:

$$\frac{P_o}{P_e} = 8.529 \times 10^{-6}$$

If Po/Pe < or = 2.25, 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 $\boxed{P_n = 1305 \cdot kip}$

Factored Axial Structural Resistance for the Strength Limit State

Resistance factor for H-pile in pure compression, severe driving conditions, per LRFD 6.5.4.2 for the case where pile tip is necessary $\phi_c := 0.5$

The Factored Structural Resistance (Pr) per LRFD 6.9.2.1-1 is $P_r := \phi_c \cdot P_n$

Factored structural compressive resistance, P_r $\boxed{P_r = 652 \cdot kip}$

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:

$$P_n = 1305 \cdot kip$$

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

$$\phi_c := 0.5$$

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

$$\boxed{P_r = 652 \cdot kip}$$ 14x89

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

$$\phi_c := 1.0$$ LRFD 6.5.5

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

$$\boxed{P_{r\_ee} = 1305 \cdot kip}$$ 14x89

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

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

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

Limit driving stress to 45 ksi or limit blow count to 15 blows per inch (bpi).

Compute the resistance that can be achieved in a drivability 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

Abutment #1:
Based on proposed bottom of footing of elevation 73.2 at abutment #1, the estimated pile length will be approx. 49 feet. Assume the contractor drives pile lengths of 70 ft (extra length accommodates for additional length to reach competent bedrock, attachment of dynamic testing equipment, embedment into abutment, variation in bedrock surface).

Use constant shaft resistances so that GRLWeap will assign approx. 216 kips as skin friction based on shaft resistance calculations and local experience in similar deposits.

Abutment #2:
Based on proposed bottom of footing of elevation 73.2 at abutment #2, the estimated pile length will be approx. 48.5 feet. Assume the contractor drives pile lengths of 65 ft (extra length accommodates for additional length to reach competent bedrock, attachment of dynamic testing equipment, embedment into abutment, variation in bedrock surface).

Use constant shaft resistances so that GRLWeap will assign approx. 191 kips as skin friction based on shaft resistance calculations and local experience in similar deposits.

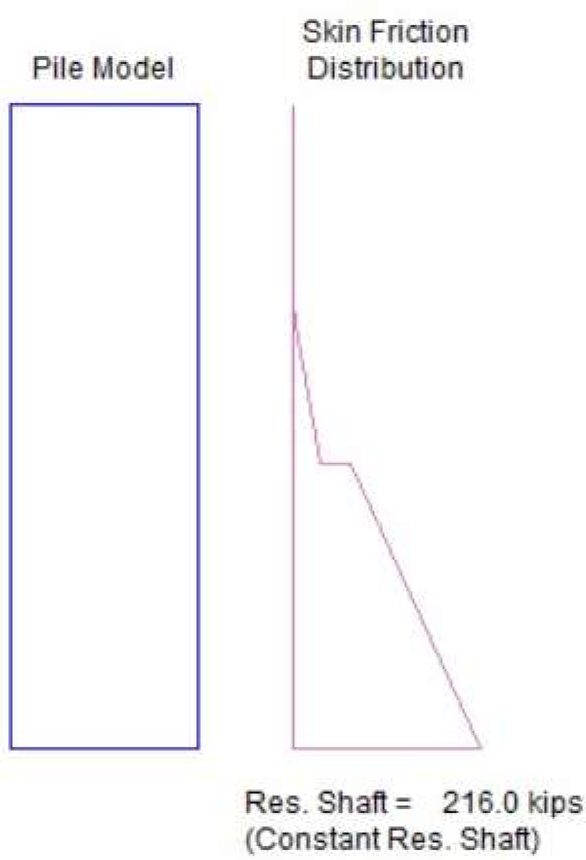
Use the GRLWEAP simple resistance distribution with varying soil layer consistencies added to the soil profile input. This assumes that the amount of skin friction developed between layers may vary and is influenced by soil consistency.

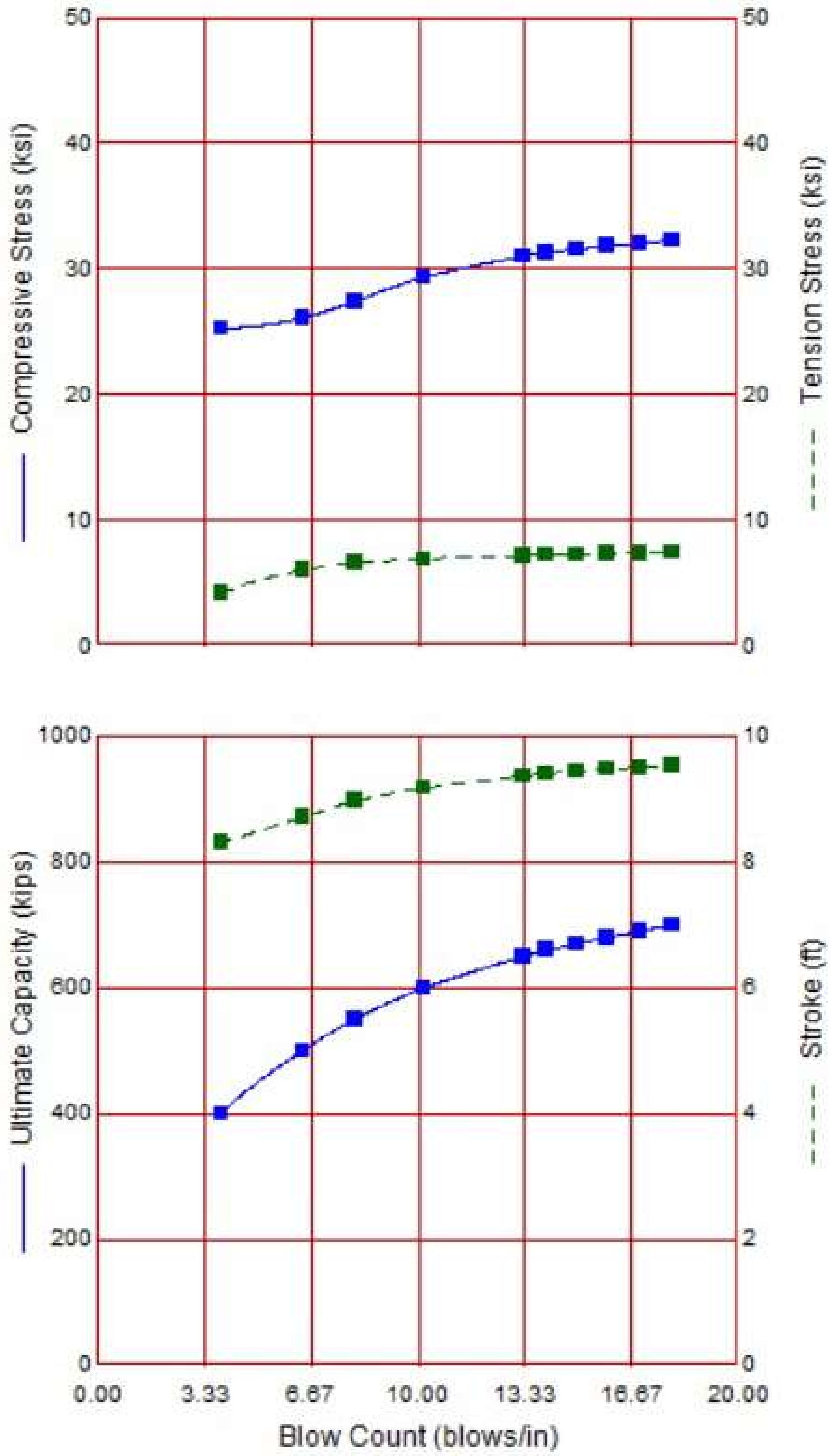
Abutment 1, Pile Size is 14 x 89, APE D19-42 Hammer

The 14x89 pile can be driven to the resistances below with an APE D19-42 hammer at fuel setting 4 (100% of Max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE D 19-42

Ram Weight	4.19	kips
Efficiency	0.800	
Pressure	1710 (100%)	psi
Helmet Weight	3.00	kips
Hammer Cushion	34825	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	70.00	ft
Pile Penetration	49.00	ft
Pile Top Area	26.10	in2




Compressive Stress (ksi)
Tension Stress (ksi)
Ultimate Capacity (kips)
Stroke (ft)
Blow Count (blows/in)
0.00
3.33
6.67
10.00
13.33
16.67
20.00

Maine DOT
21686 Charlotte 14x89 ABT #1 D19-42

12-Mar-2025
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	25.26	4.24	3.8	8.31	22.03
500.0	26.12	6.07	6.4	8.71	22.92
550.0	27.43	6.63	8.0	8.97	23.55
600.0	29.36	6.97	10.2	9.18	24.16
650.0	31.00	7.23	13.3	9.37	24.64
660.0	31.30	7.28	14.0	9.41	24.77
670.0	31.56	7.33	14.9	9.45	24.82
680.0	31.83	7.39	15.9	9.48	24.88
690.0	32.03	7.44	16.9	9.51	24.95
700.0	32.31	7.50	17.9	9.54	25.09

Limit to 15 bpi

$$R_{ndr} := 670 \cdot kip$$

Strength Limit State

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

$$\boxed{R_{fdr} = 436 \cdot kip}$$

Extreme and
Service Limit States

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

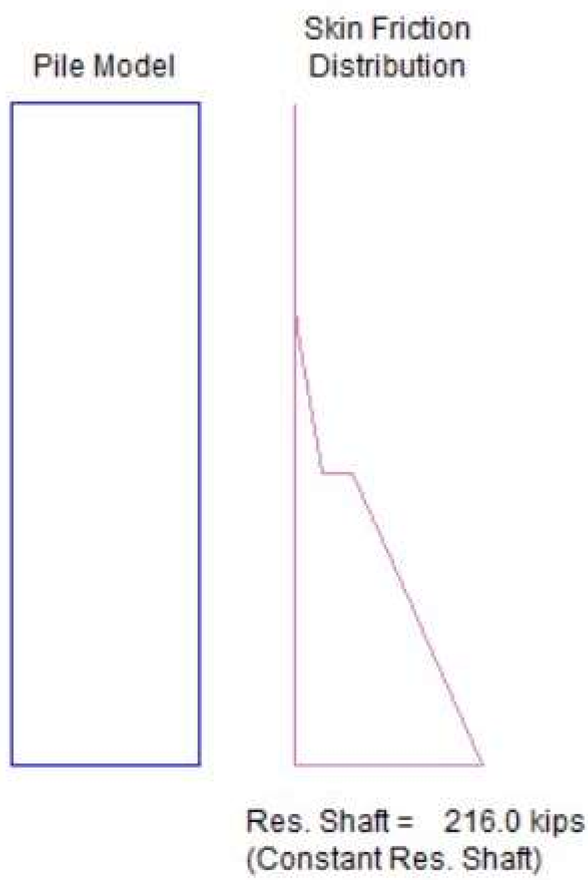
$$\boxed{R_{dr} = 670 \cdot kip}$$

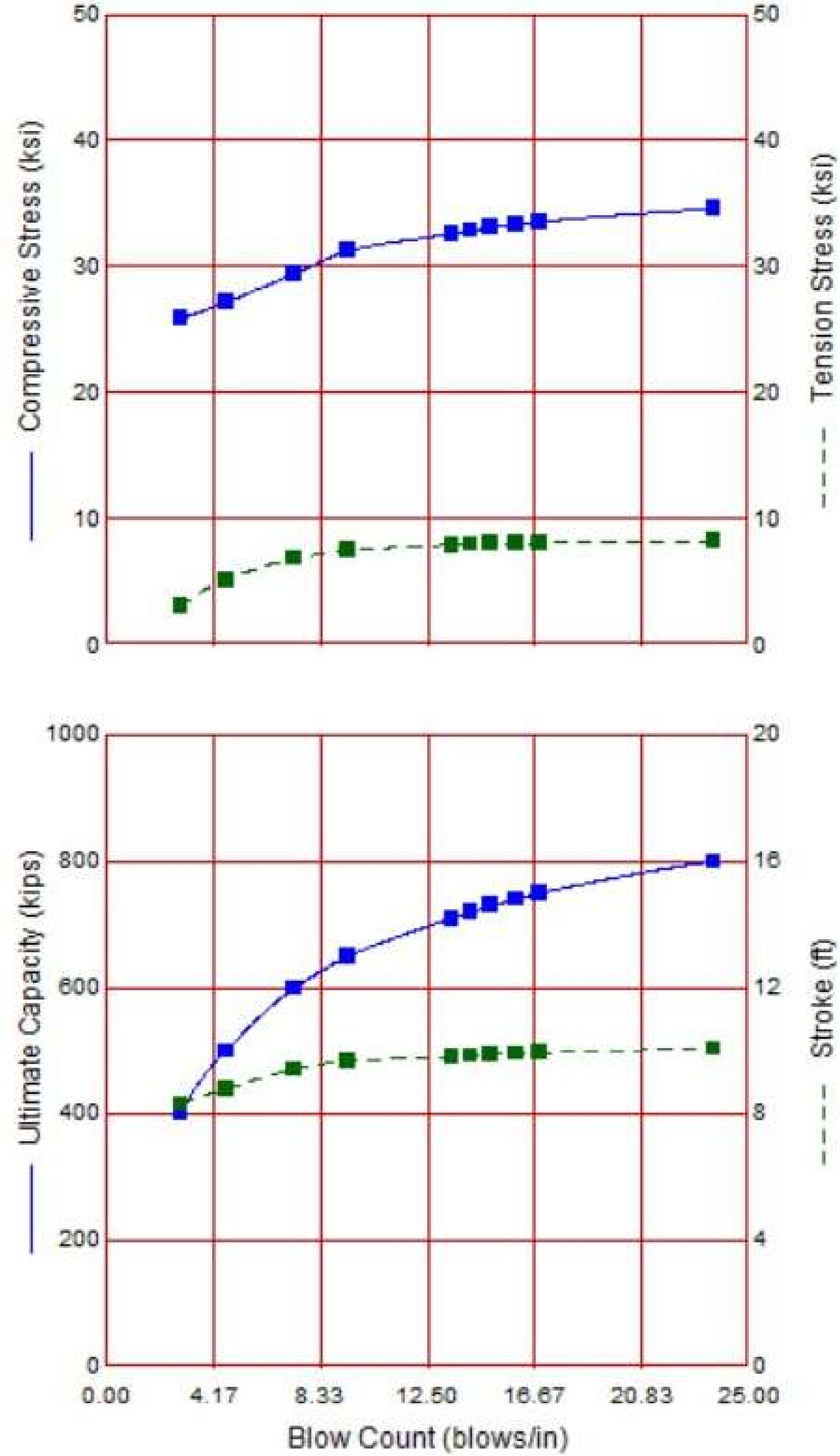
Abutment 1, Pile Size is 14 x 89, APE D25-42 Hammer

The 14x89 pile can be driven to the resistances below with a APE D25-42 hammer at fuel setting 4 (100% of max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1425 (100%)	psi
Helmet Weight	3.00	kips
Hammer Cushion	34825	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	70.00	ft
Pile Penetration	49.00	ft
Pile Top Area	26.10	in2




Compressive Stress (ksi)
Tension Stress (ksi)
Ultimate Capacity (kips)
Stroke (ft)
Blow Count (blows/in)
0.00
4.17
8.33
12.50
16.67
20.83
25.00

Maine DOT
21686 Charlotte 14x89 ABT #1 D25-42

12-Mar-2025
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	25.90	3.08	2.9	8.30	25.61
500.0	27.18	5.20	4.6	8.78	27.03
600.0	29.42	6.94	7.3	9.41	29.20
650.0	31.25	7.56	9.4	9.67	30.07
710.0	32.60	7.99	13.4	9.81	30.44
720.0	32.84	8.05	14.2	9.84	30.58
730.0	33.11	8.08	15.0	9.88	30.71
740.0	33.33	8.11	15.9	9.91	30.78
750.0	33.56	8.15	16.8	9.94	30.92
800.0	34.59	8.28	23.6	10.08	31.36

Limit to 15 bpi

$$R_{ndr} := 730 \cdot kip$$

Strength Limit State

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

$$\boxed{R_{fdr} = 474 \cdot kip}$$

Extreme and
Service Limit States

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

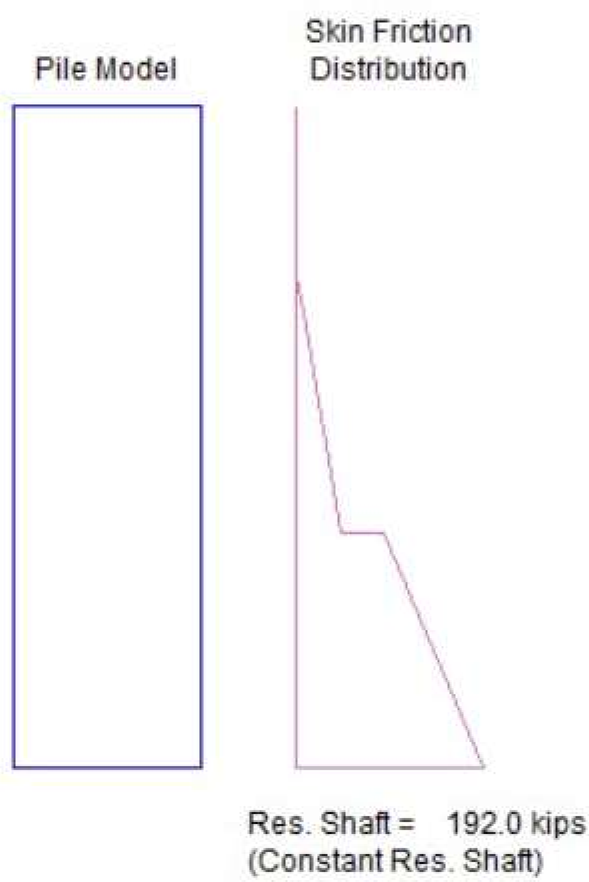
$$\boxed{R_{dr} = 730 \cdot kip}$$

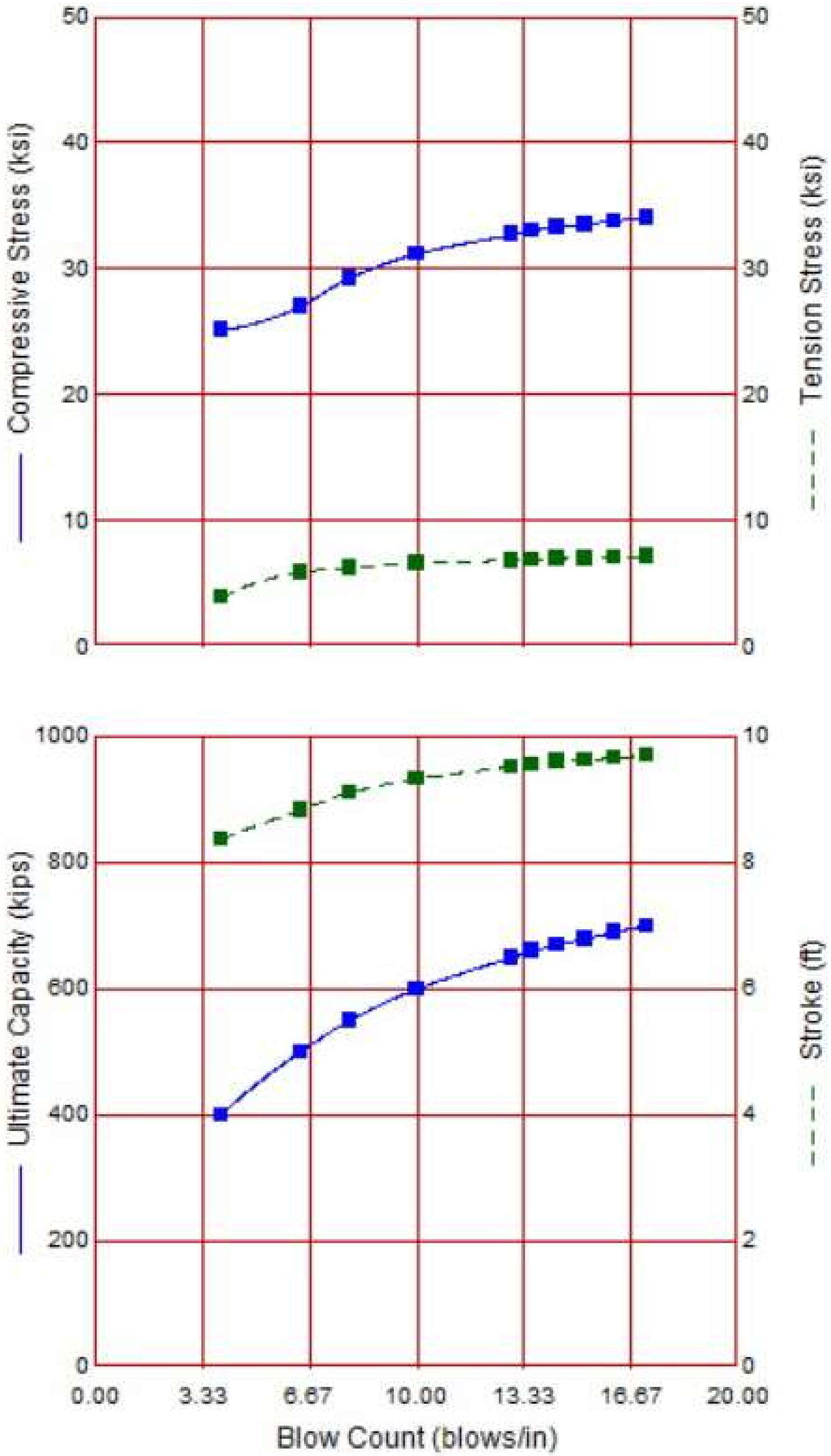
Abutment 2, Pile Size is 14 x 89, APE D19-42 Hammer

The 14x89 pile can be driven to the resistances below with a APE D19-42 hammer at fuel setting 4 (100% of max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE D 19-42

Ram Weight	4.19	kips
Efficiency	0.800	
Pressure	1710 (100%)	psi
Helmet Weight	3.00	kips
Hammer Cushion	34825	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	65.00	ft
Pile Penetration	48.50	ft
Pile Top Area	26.10	in2




Compressive Stress (ksi)
Tension Stress (ksi)
Ultimate Capacity (kips)
Stroke (ft)
Blow Count (blows/in)
0.00
3.33
6.67
10.00
13.33
16.67
20.00

Maine DOT
21686 Charlotte 14x89 ABT #2 D19-42

12-Mar-2025
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	25.14	3.97	3.9	8.37	21.91
500.0	27.03	5.95	6.4	8.84	23.06
550.0	29.28	6.32	7.9	9.11	23.76
600.0	31.20	6.63	10.0	9.33	24.31
650.0	32.72	6.90	12.9	9.52	24.76
660.0	33.06	6.96	13.6	9.56	24.92
670.0	33.31	7.01	14.4	9.60	24.99
680.0	33.58	7.06	15.2	9.63	25.06
690.0	33.83	7.12	16.1	9.66	25.16
700.0	34.09	7.18	17.2	9.70	25.23

Limit to 15 bpi

$$R_{ndr} := 670 \cdot kip$$

Strength Limit State

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

$$\boxed{R_{fdr} = 436 \cdot kip}$$

Extreme and
Service Limit States

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

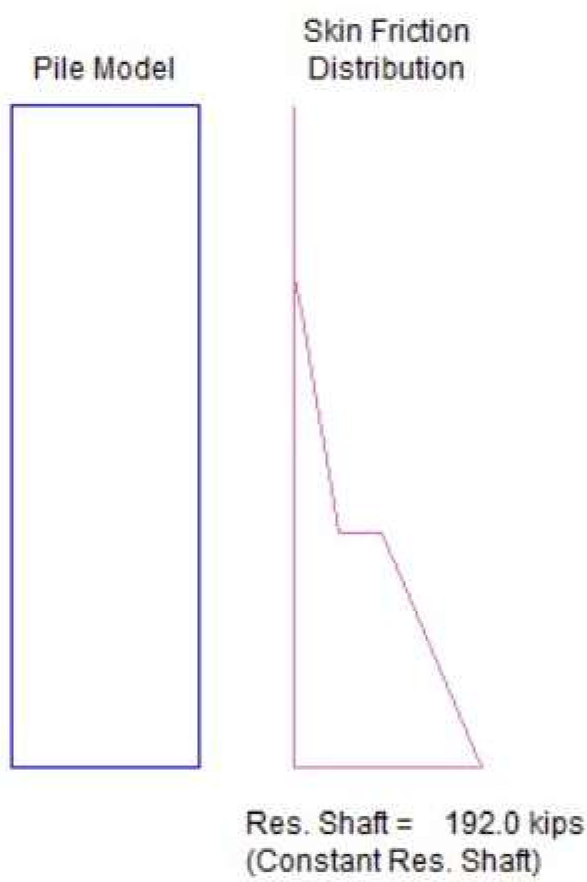
$$\boxed{R_{dr} = 670 \cdot kip}$$

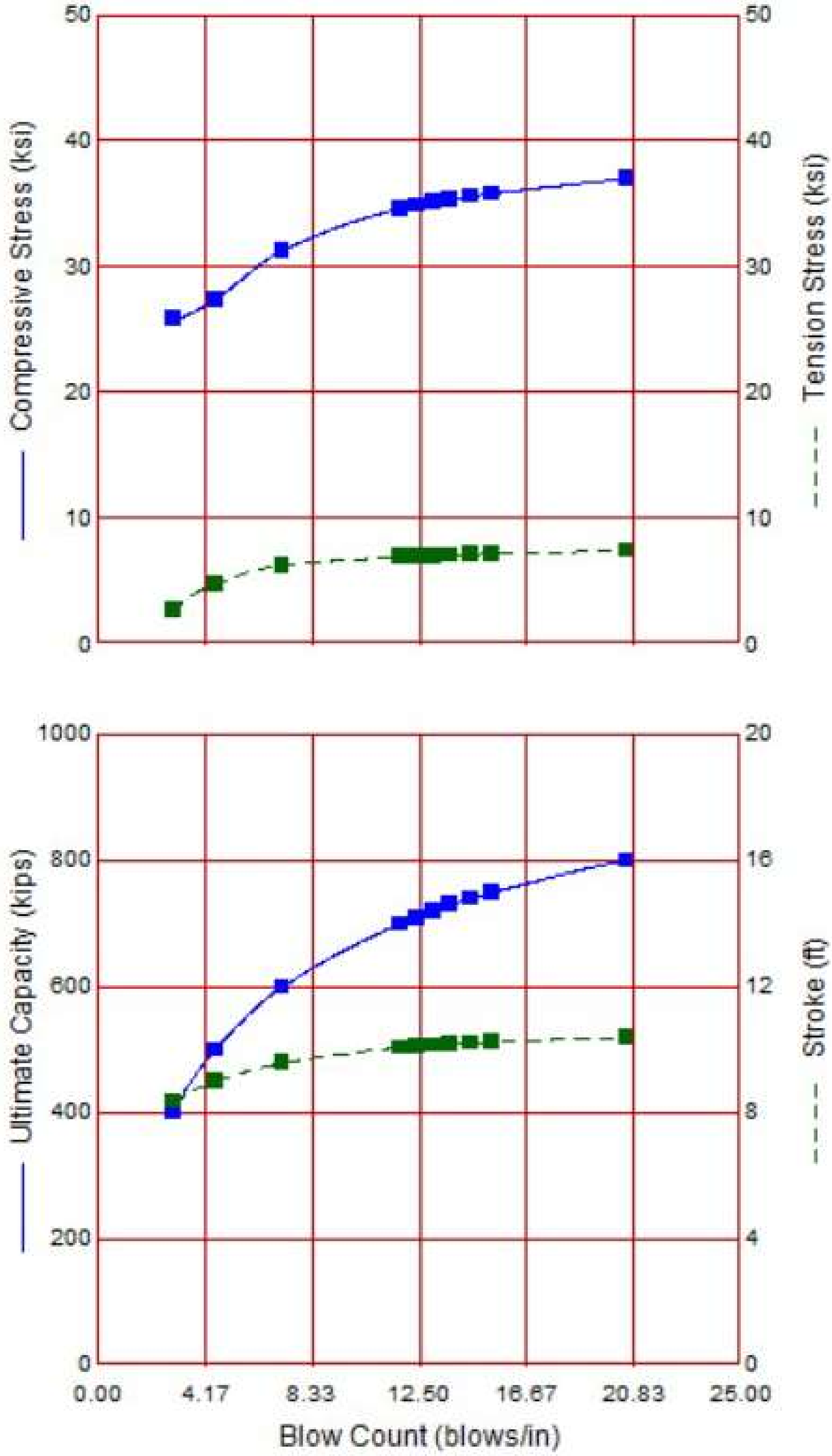
Abutment 2, Pile Size is 14 x 89, APE D25-42 Hammer

The 14x89 pile can be driven to the resistances below with a APE D25-42 hammer at fuel setting 4 (100% of max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1425 (100%)	psi
Helmet Weight	3.00	kips
Hammer Cushion	34825	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	65.00	ft
Pile Penetration	48.50	ft
Pile Top Area	26.10	in2




Compressive Stress (ksi)
Tension Stress (ksi)
50
40
30
20
10
0
Ultimate Capacity (kips)
Stroke (ft)
1000
800
600
400
200
0
20
16
12
8
4
0
0.00
4.17
8.33
12.50
16.67
20.83
25.00
Blow Count (blows/in)

Maine DOT | 12-Mar-2025
21686 Charlotte 14x89 ABT #2 D25-42 | 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	25.88	2.79	2.9	8.36	25.61
500.0	27.46	4.85	4.6	8.98	27.27
600.0	31.32	6.31	7.2	9.57	29.19
700.0	34.67	6.97	11.7	10.07	30.91
710.0	34.91	7.02	12.4	10.11	30.98
720.0	35.15	7.08	13.0	10.15	31.13
730.0	35.42	7.14	13.7	10.19	31.27
740.0	35.64	7.20	14.5	10.22	31.35
750.0	35.85	7.24	15.3	10.25	31.42
800.0	37.00	7.48	20.5	10.41	31.99

Limit to 15 bpi

$R_{ndr} := 740 \cdot kip$

Strength Limit State

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

$\boxed{R_{fdr} = 481 \cdot kip}$

Extreme and
Service Limit States

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

$\boxed{R_{dr} = 740 \cdot kip}$

21686.10 Charlotte - Moosehorn Bridge #3332

GRL WEAP INPUT + RESULT SUMMARY

Created By: NPP 5/14/25

Checked By: LK 5/21/25

	Abutment	Pile Size	Pile Length	Pile Penetration	Hammer	Fuel Setting	Shaft Quake	Toe Quake	Shaft Damping	Toe Damping	Skin Friction	Ultimate Capacity	Max Comp Stress	Max Tension Stress	Blows/In	Stroke	Energy
Abutment #1 14x89 APE D19-42	1	HP 14x89	70	49	APE D19-42	3	0.10	0.10	0.05	0.15	216	610	27.58	6.65	14.3	8.36	21.10
	1	HP 14x89	70	49	APE D19-42	4	0.10	0.10	0.05	0.15	216	670	31.56	7.33	14.9	9.45	24.82
Abutment #1 14x89 APE D25-42	1	HP 14x89	70	49	APE D25-42	3	0.10	0.10	0.05	0.15	216	660	28.34	7.27	15.0	8.67	25.07
	1	HP 14x89	70	49	APE D25-42	4	0.10	0.10	0.05	0.15	216	730	33.11	8.08	15.0	9.88	30.71
Abutment #2 14x89 APE D19-42	2	HP 14x89	65	48.5	APE D19-42	3	0.10	0.10	0.05	0.15	192	620	29.56	6.41	14.7	8.53	21.33
	2	HP 14x89	65	48.5	APE D19-42	4	0.10	0.10	0.05	0.15	192	670	33.31	7.01	14.4	9.60	24.99
Abutment #2 14x89 APE D25-42	2	HP 14x89	65	48.5	APE D25-42	3	0.10	0.10	0.05	0.15	192	660	30.07	6.46	14.2	8.83	25.24
	2	HP 14x89	65	48.5	APE D25-42	4	0.10	0.10	0.05	0.15	192	740	35.64	7.20	14.5	10.22	31.35
Abutment #1 14x117 APE D19-42	1	HP 14x117	70	49	APE D19-42	3	0.10	0.10	0.05	0.15	216	680	25.63	4.01	14.9	8.29	19.74
	1	HP 14x117	70	49	APE D19-42	4	0.10	0.10	0.05	0.15	216	740	29.26	4.30	14.7	9.34	23.25
Abutment #1 14x117 APE D25-42	1	HP 14x117	70	49	APE D25-42	3	0.10	0.10	0.05	0.15	216	730	26.25	4.78	14.8	8.59	23.12
	1	HP 14x117	70	49	APE D25-42	4	0.10	0.10	0.05	0.15	216	810	31.14	5.32	14.4	9.84	28.65
Abutment #2 14x117 APE D19-42	2	HP 14x117	65	48.5	APE D19-42	3	0.10	0.10	0.05	0.15	192	680	26.88	3.81	14.7	8.37	19.86
	2	HP 14x117	65	48.5	APE D19-42	4	0.10	0.10	0.05	0.15	192	740	30.52	4.06	14.5	9.44	23.36
Abutment #2 14x117 APE D25-42	2	HP 14x117	65	48.5	APE D25-42	3	0.10	0.10	0.05	0.15	192	730	27.68	4.45	14.3	8.70	23.31
	2	HP 14x117	65	48.5	APE D25-42	4	0.10	0.10	0.05	0.15	192	820	32.80	4.89	14.6	10.01	28.84

Hammer Information:

APE D19-42	Fuel Setting #3	39,119 ft-lbs
APE D19-42	Fuel Setting #4	47,132 ft-lbs
APE D25-42	Fuel Setting #3	55,814 ft-lbs
APE D25-42	Fuel Setting #4	62,016 ft-lbs

APE D19-42	APE D25-42
#1 1247 psi	#1 1040 psi
#2 1385 psi	#2 1155 psi
#3 1539 psi	#3 1280 psi
#4 1710 psi	#4 1425 psi

TABLE 4.4.8.1.2B Typical Range of Uniaxial Compressive Strength (C_o) as a Function of Rock Category and Rock Type

Rock Category	General Description	Rock Type	C_o[1] (ksf)	C_o[1] (psi)
A	Carbonate rocks with well-developed crystal cleavage	Dolostone	700- 6,500	4,800-45,000
		Limestone	500- 6,000	3,500-42,000
		Carbonatite	800- 1,500	5,500-10,000
		Marble	800- 5,000	5,500-35,000
		Tactite-Skarn	2,700- 7,000	19,000-49,000
B	Lithified argillaceous rock	Argillite	600- 3,000	4,200-21,000
		Claystone	30- 170	200- 1,200
		Marlstone	1,000- 4,000	7,600-28,000
		Phyllite	500- 5,000	3,500-35,000
		Siltstone	200- 2,500	1,400-17,000
		Shale[2]	150- 740	1,000- 5,100
		Slate	3,000- 4,400	21,000-30,000
C	Arenaceous rocks with strong crystals and poor cleavage	Conglomerate	700- 4,600	4,800-32,000
		Sandstone	1,400- 3,600	9,700-25,000
		Quartzite	1,300- 8,000	9,000-55,000
D	Fine-grained igneous crystalline rock	Andesite	2,100- 3,800	14,000-26,000
		Diabase	450-12,000	3,100-83,000
E	Coarse-grained igneous and metamorphic crystalline rock	Amphibolite	2,500- 5,800	17,000-40,000
		Gabbro	2,600- 6,500	18,000-45,000
		Gneiss	500- 6,500	3,500-45,000
		Granite	300- 7,000	2,100-49,000
		Quartzdiorite	200- 2,100	1,400-14,000
		Quartzmonzonite	2,700- 3,300	19,000-23,000
		Schist	200- 3,000	1,400-21,000
		Syenite	3,800- 9,000	26,000-62,000

[1]Range of Uniaxial Compressive Strength values reported by various investigations.
[2]Not including oil shale.

$$\rho = q_o(1 - \nu^2)BI_p/E_m, \text{ with } I_p = (L/B)^{1/2}/\beta_z \qquad (4.4.8.2.2\text{-}2)$$

Values of I_p may be computed using the β_z values presented in Table 4.4.7.2.2B from Article 4.4.7.2.2 for rigid footings. Values of Poisson's ratio (υ) for typical rock types are presented in Table 4.4.8.2.2A. Determination of the rock mass modulus (E_m) should be based on the results of in-situ and laboratory tests. Alternatively, values of E_m may be estimated by multiplying the intact rock modulus (E_o) obtained from uniaxial compression tests by a reduction factor (α_E) which accounts for frequency of discontinuities by the rock quality designation (RQD), using the following relationships (Gardner, 1987):

$$E_m = \alpha_E E_o \qquad (4.4.8.2.2\text{-}3)$$

$$\alpha_E = 0.0231(RQD) - 1.32 \geq 0.15 \qquad (4.4.8.2.2\text{-}4)$$

For preliminary design or when site-specific test data cannot be obtained, guidelines for estimating values of E_o (such as presented in Table 4.4.8.2.2B or Figure 4.4.8.2.2A) may be used. For preliminary analyses or for final design when in-situ test results are not available, a value of $\alpha_E = 0.15$ should be used to estimate E_m.

4.4.8.2.3 Tolerable Movement

Refer to Article 4.4.7.2.3.

4.4.9 Overall Stability

The overall stability of footings, slopes, and foundation soil or rock shall be evaluated for footings located on

Earth Pressure

Earth Pressure:

Backfill engineering strength parameters

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

Unit weight $\gamma_1 := 125 \cdot \text{pcf}$

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

Cohesion $c_1 := 0 \cdot \text{psf}$

Abutment Backfill Angles

α = Angle of fill slope to the horizontal

Angles computed based on proposed roadway elevation change 25 feet behind the centerline of the abutments

$\text{Rise}_{ABT1} := -0.3\text{ft}$ $\text{Rise}_{ABT2} := -0.3\text{ft}$ $\text{Run} := 25\text{ft}$

$$\alpha_{ABT1} := \text{atan}\left(\frac{\text{Rise}_{ABT1}}{\text{Run}}\right) = -0.69 \cdot \text{deg}$$ Abutment No. 1

$$\alpha_{ABT2} := \text{atan}\left(\frac{\text{Rise}_{ABT2}}{\text{Run}}\right) = -0.69 \cdot \text{deg}$$ Abutment No. 2

Integral Abutment - Passive Earth Pressure - Coulomb Theory

α = Angle of fill slope to the horizontal $\alpha := -0.69\text{deg}$

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

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

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 .020 or greater. Coulomb should also be used when the fill slope is greater than horizontal.

For formed concrete 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)

$\delta' := 17 \cdot \text{deg}$

$$K_{p\_coulomb} := \frac{\sin(\beta - \phi')^2}{\sin(\beta)^2 \cdot \sin(\beta + \delta') \cdot \left(1 - \sqrt{\frac{\sin(\phi' + \delta') \cdot \sin(\phi' + \alpha_{ABT1})}{\sin(\beta + \delta') \cdot \sin(\beta + \alpha_{ABT1})}}\right)^2}$$

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

$\boxed{K_{p\_coulomb} = 5.82}$

Integral Abutment and Wingwall - Passive Earth Pressure - Rankine Theory

Per the BDG, use Rankine only if the ratio of wall height to wall movement is 0.005 or less 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 $\alpha = -0.69\cdot\text{deg}$

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

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

Integral Abutment - Passive Pressure Coefficient per MassDOT LRFD Bridge Manual Part 1

$$K = 0.43 + 5.7[1 - e^{-190(\delta_T/H)}]$$

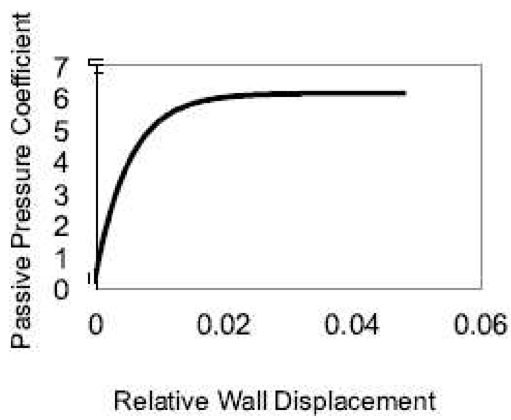


Figure 3.10.8-1: Plot of Passive Pressure Coefficient, K, vs. Relative Wall Displacement, δ_T/H.

Estimate Thermal Movement

$\Delta := \alpha\cdot L\cdot(T_{MaxDesign} - T_{MinDesign})$ LRFD Eq. 3.12.2

where:
L = expansion length (in.)
α = coefficient of thermal expansion (in./in./°F)

Bridge Span (Approximate Expansion Length)

$L := 74\text{ft}$ $L = 888\cdot\text{in}$

Coefficient of thermal expansion (in./in./°F) per Vtrans Integral Abutment Design Guidelines 4.5.1.1

$$\alpha_{steel} := 0.0000065$$

$$\alpha_{concrete} := 0.0000060$$

Bridge superstructure will consist of concrete beams, choose $\alpha_{concrete}$

Choose thermal movement range (°F) from LRFD Table 3.12.2.1-1

Table 3.12.2.1-1—Procedure A Temperature Ranges

Climate	Steel or Aluminum	Concrete	Wood
Moderate	0° to 120°F	10° to 80°F	10° to 75°F
Cold	−30° to 120°F	0° to 80°F	0° to 75°F

$$T_{Max} := 80$$

$$T_{Min} := 0$$

$\Delta := \alpha_{concrete} \cdot L \cdot (T_{Max} - T_{Min})$

$\Delta = 0.43 \cdot in$ Total movement from thermal displacement

$\delta := 0.5 \cdot \Delta$ Thermal displacement at each abutment

$$\delta = 0.21 \cdot in$$

Compute Relative Wall Displacement

Abutment height: $h := 11.2ft$ $h = 134.4 \cdot in$

Relative wall displacement: $x := \frac{\delta}{h}$ $x = 0.0016$

$K := 0.43 + 5.7 \cdot [1 - \exp[-190(x)]]$

$\boxed{K = 1.91}$ **< K_{p_rank} of 3.25, therefore recommend K=3.25 for both Abutments**

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	17 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 7.9 (*Continued*)

		$c'/\gamma z$			
ϕ' (deg)	α (deg)	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

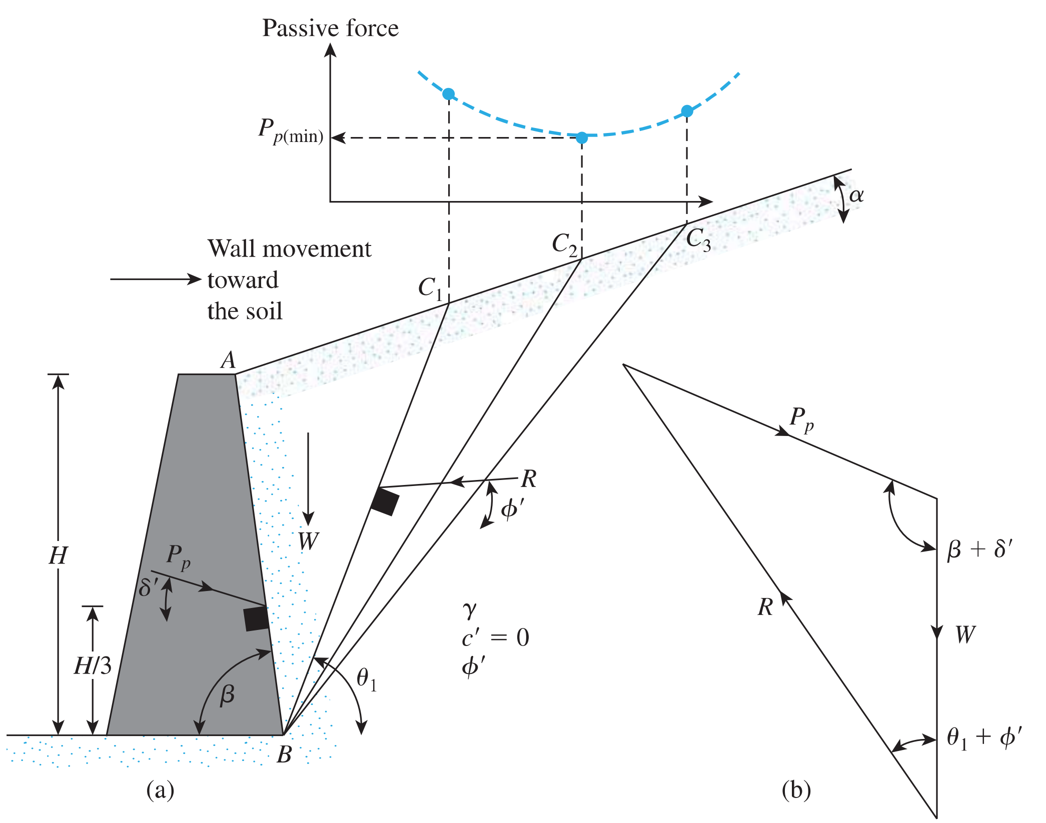


Figure 7.25 Coulomb's passive pressure

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

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

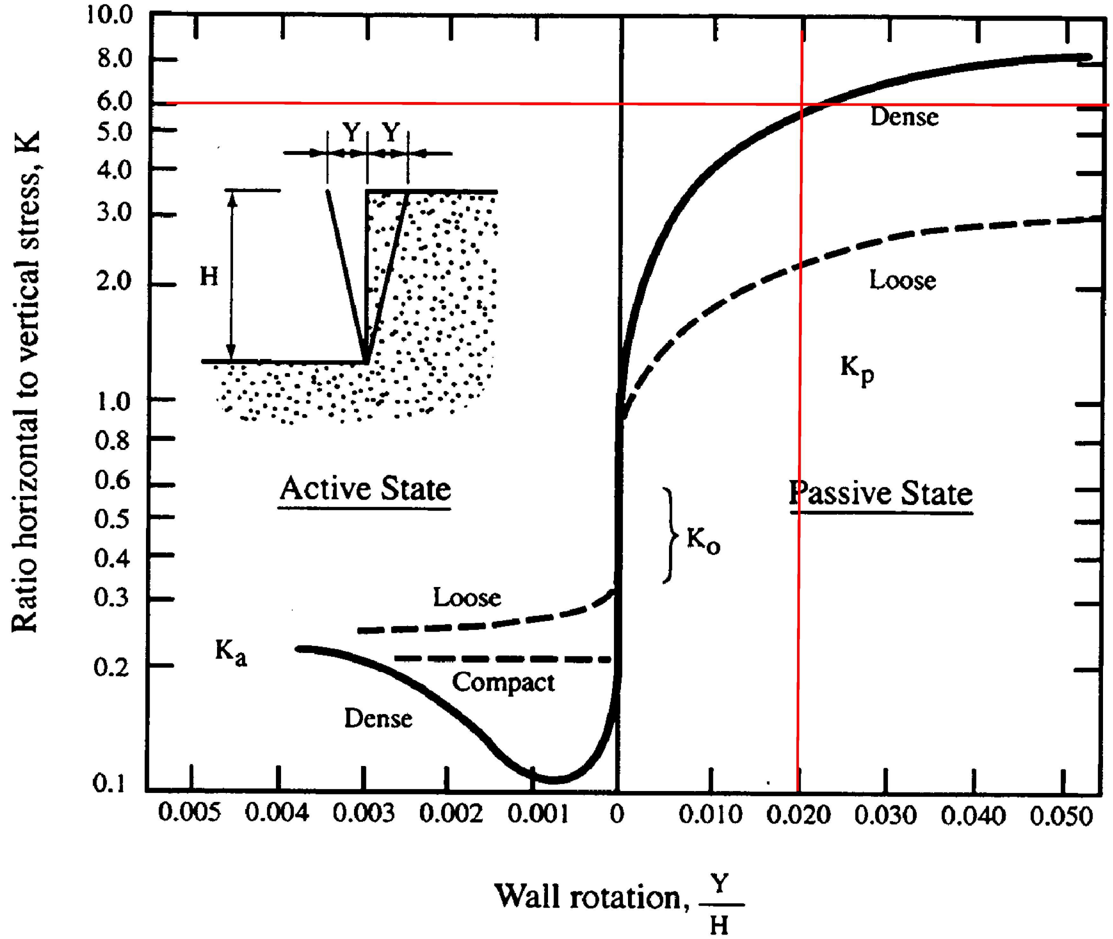
$$\sigma'_p = \gamma z K_p \tag{7.65}$$

and the passive force is

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

where

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



Magnitude of Wall Rotation to Reach Failure

Soil type and condition	Rotation, Y/H	
	Active	Passive
Dense cohesionless	0.001	0.02
Loose cohesionless	0.004	0.06
Stiff cohesive	0.010	0.02
Soft cohesive	0.020	0.04

Figure 10-4. Effect of wall movement on wall pressures (after Canadian Geotechnical Society, 1992).

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: Charlotte, Maine
DFI = 1350 degree-days.
Coarse-Grained Fill w=10% (BB-CHAR-101 4D, BB-CHAR-102 2D, BB-CHAR-202 2D)

Coarse-Grained Fill

For DFI = 1300, Coarse-Grained Soil, w=10%

$DFI_1 := 1300 \quad d_1 := 76.3in$

d=Depth of Frost Penetration

For DFI = 1400, Coarse-Grained Soil, w=10%

$DFI_2 := 1400 \quad d_2 := 79.2in$

Interpolate for DFI = 1350, Coarse-Grained Soil, w=10%

$DFI_3 := 1350$

$$d_{coarse} := d_1 + \left(DFI_3 - DFI_1\right) \cdot \frac{\left(d_2 - d_1\right)}{\left(DFI_2 - DFI_1\right)}$$

$\boxed{d_{coarse} = 77.8 \cdot in}$ $\boxed{d_{coarse} = 6.5 \cdot ft}$

Recommend any foundation bearing on soil be embedded **6.5 feet** for frost protection.

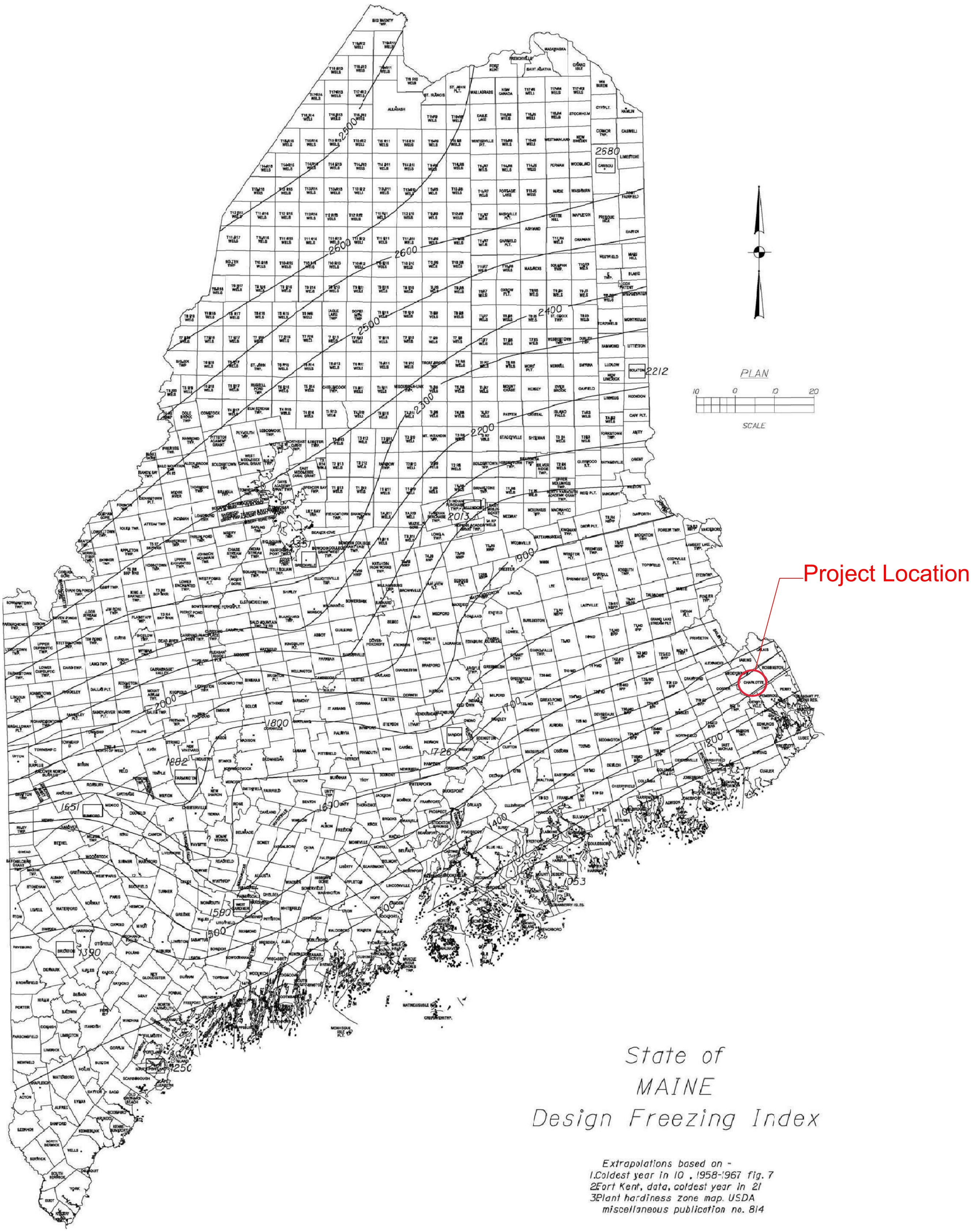


Figure 5-1 Maine Design Freezing Index Map

5.2 General

MaineDOT Bridge Design Guide

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 Parameters

BB-CHAR-101			
Depth	**N_{60}**	**di**	**di/N**
1	33	3	0.09
3	13	5	0.38
10	7	4	0.57
12	6	2	0.33
14	9	2	0.22
16	12	3	0.25
19	12	2	0.17
21	10	4	0.40
25	12	1	0.08
30	16	6	0.38
35	75	4	0.05
40	100	9	0.09
45	100	5	0.05
50	100	5	0.05
55	100	2	0.02
59	100	8	0.08
65	100	1	0.01
66	100	34	0.34
SUM		**100**	**3.57**
		di/di/N	**28.00**

BB-CHAR-102			
Depth	**N_{60}**	**di**	**di/N**
1	14	3	0.21
3	5	2	0.40
5	5	2	0.40
7	6	8	1.33
15	6	2	0.33
17	6	2	0.33
19	7	2	0.29
21	11	2	0.18
23	8	2	0.25
25	5	5	1.00
30	18	5	0.28
35	29	5	0.17
40	38	5	0.13
45	42	5	0.12
50	100	5	0.05
55	100	5	0.05
60	100	40	0.40
SUM		**100**	**5.93**
		di/di/N	**16.86**

BB-CHAR-201/201A			
Depth	**N_{60}**	**di**	**di/N**
5	9	10	1.11
10	9	3	0.33
15	9	7	0.78
20	12	5	0.42
25	14	3	0.21
30	21	7	0.33
35	48	5	0.10
40	100	5	0.05
45	100	5	0.05
51	100	5	0.05
55	100	5	0.05
60	100	4	0.04
64	100	36	0.36
SUM		**100**	**3.89**
		di/di/N	**25.70**

BB-CHAR-202			
Depth	**N_{60}**	**di**	**di/N**
0	19	5	0.26
5	3	3	1.00
10	3	5	1.67
15	26	3	0.12
20	11	9	0.82
25	16	5	0.31
30	11	5	0.45
35	37	5	0.14
40	37	5	0.14
45	34	5	0.15
50	72	5	0.07
55	100	3	0.03
58	100	42	0.42
SUM		**100**	**5.57**
		di/di/N	**17.96**

SUM	**Nav.**	**22.43**

15 < Nav. < 50 bpf

Conclusion: Site Class D

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

Charlotte, Moosehorn Bridge #3332
WIN 21686.10
June 13, 2024

Abutment No. 1 and 2 Seismic Parameters

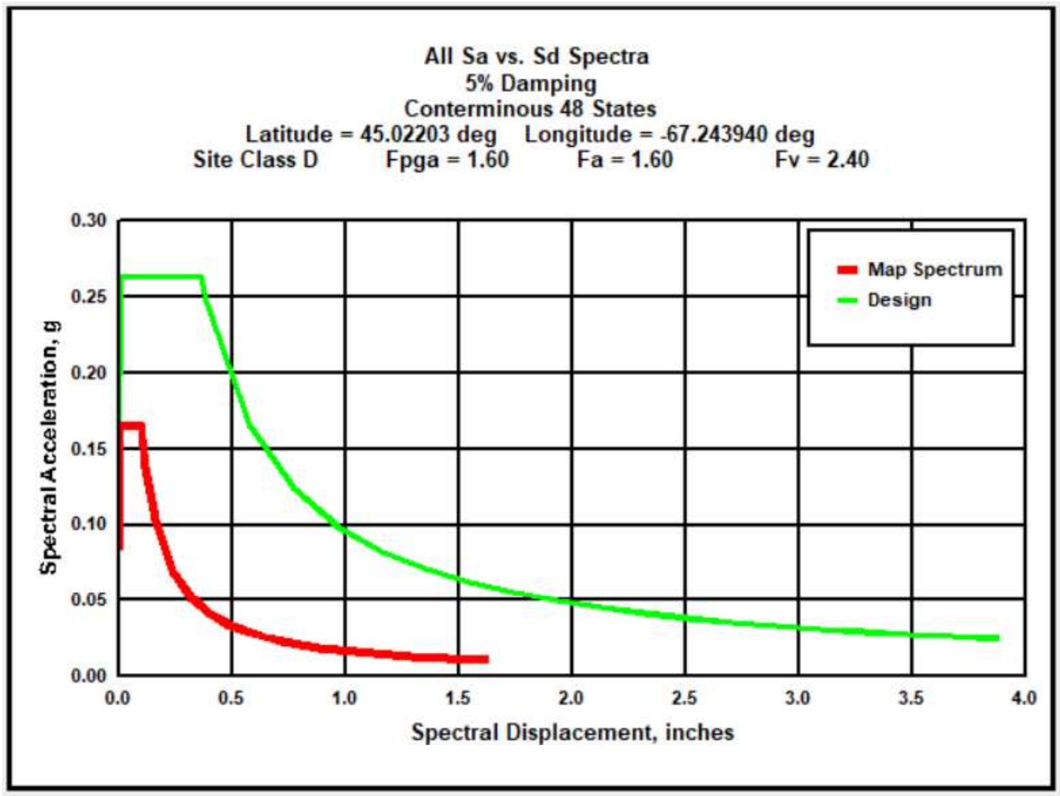
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 45.022028
Longitude = -067.243944
Site Class B
Data are based on a 0.05 deg grid spacing.

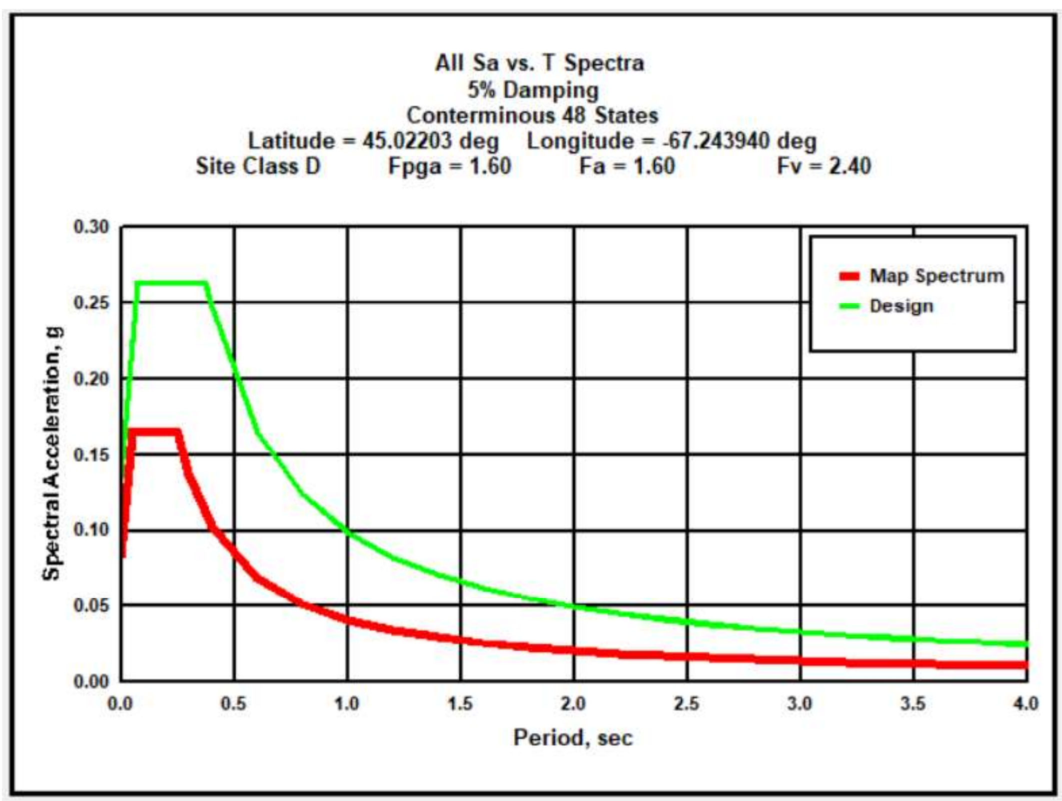
Period (sec)	Sa (g)	
0.0	0.085	PGA - Site Class B
0.2	0.164	Ss - Site Class B
1.0	0.041	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
Latitude = 45.022028
Longitude = -067.243944
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	0.136	As - Site Class D
0.2	0.263	SDs - Site Class D
1.0	0.099	SD1 - Site Class D

Charlotte, Moosehorn Bridge #3332
WIN 21686.10
June 13, 2024





Settlement

Settle3D Analysis Information
21686 Charlotte Moosehorn Bridge - Abutment No. 2 Approach

Project Settings

Document Name	21686 Charlotte - Abutment No. 2 Approach Settlement d3.s3z
Project Title	21686 Charlotte Moosehorn Bridge - Abutment No. 2 Approach
Analysis	Time Dependent Analysis
Author	N. Pukay
Company	MaineDOT
Date Created	4/8/2025, 9:07:01 AM
Stress Computation Method	Boussinesq
Time-dependent Consolidation Analysis	
Time Units	years
Permeability Units	feet/year
Use average properties to calculate layered stresses	

Stage Settings

Stage #	Name	Time [years]
1	Stage 1	0
2	Stage 2	0.1
3	Stage 3	0.25
4	Stage 4	0.5
5	Stage 5	1
6	Stage 6	2
7	Stage 7	75

Results

Time taken to compute: 0 seconds

Stage: Stage 1 = 0 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	0.424618
Consolidation Settlement [in]	0	0
Immediate Settlement [in]	0	0.424618
Secondary Settlement [in]	0	0
Loading Stress [ksf]	0.0360269	0.2
Effective Stress [ksf]	0.2	4.39692
Total Stress [ksf]	0.2	7.57167
Total Strain	0.000111053	0.00303173
Pore Water Pressure [ksf]	0	3.17475
Excess Pore Water Pressure [ksf]	0	0.177335
Degree of Consolidation [%]	0	0
Pre-consolidation Stress [ksf]	0.20975	4.39048
Over-consolidation Ratio	1	1.5
Void Ratio	0	1.49328
Permeability [ft/y]	0	0.081624
Coefficient of Consolidation [ft^2/y]	0	18.25
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.879384

Stage: Stage 2 = 0.1 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	0.785581
Consolidation Settlement [in]	-0.000796364	0.360963
Immediate Settlement [in]	0	0.424618
Secondary Settlement [in]	0	0
Loading Stress [ksf]	0.0360269	0.2
Effective Stress [ksf]	0.2	4.29778
Total Stress [ksf]	0.2	7.57167
Total Strain	0.000111053	0.0266756
Pore Water Pressure [ksf]	0	3.27389
Excess Pore Water Pressure [ksf]	0	0.146583
Degree of Consolidation [%]	0	28.9629
Pre-consolidation Stress [ksf]	0.20975	4.39048
Over-consolidation Ratio	1	1.50805
Void Ratio	0	1.49332
Permeability [ft/y]	0	0.081624
Coefficient of Consolidation [ft^2/y]	0	18.25
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.875382

Stage: Stage 3 = 0.25 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	0.969793
Consolidation Settlement [in]	-0.000640192	0.545175
Immediate Settlement [in]	0	0.424618
Secondary Settlement [in]	0	0
Loading Stress [ksf]	0.0360269	0.2
Effective Stress [ksf]	0.2	4.29295
Total Stress [ksf]	0.2	7.57167
Total Strain	0.000111053	0.0297691
Pore Water Pressure [ksf]	0	3.27872
Excess Pore Water Pressure [ksf]	0	0.139999
Degree of Consolidation [%]	0	43.7438
Pre-consolidation Stress [ksf]	0.20975	4.39048
Over-consolidation Ratio	1	1.51302
Void Ratio	0	1.48813
Permeability [ft/y]	0	0.081624
Coefficient of Consolidation [ft^2/y]	0	18.25
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.875185

Stage: Stage 4 = 0.5 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	1.16198
Consolidation Settlement [in]	-1.37031e-006	0.737361
Immediate Settlement [in]	0	0.424618
Secondary Settlement [in]	0	0
Loading Stress [ksf]	0.0360269	0.2
Effective Stress [ksf]	0.2	4.30567
Total Stress [ksf]	0.2	7.57167
Total Strain	0.000111053	0.0311915
Pore Water Pressure [ksf]	0	3.266
Excess Pore Water Pressure [ksf]	0	0.127277
Degree of Consolidation [%]	0	59.1644
Pre-consolidation Stress [ksf]	0.20975	4.39048
Over-consolidation Ratio	1	1.49906
Void Ratio	0	1.47451
Permeability [ft/y]	0	0.081624
Coefficient of Consolidation [ft^2/y]	0	18.25
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.875703

Stage: Stage 5 = 1 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	1.3924
Consolidation Settlement [in]	0	0.967786
Immediate Settlement [in]	0	0.424618
Secondary Settlement [in]	0	0
Loading Stress [ksf]	0.0360269	0.2
Effective Stress [ksf]	0.2	4.35419
Total Stress [ksf]	0.2	7.57167
Total Strain	0.000111053	0.0324501
Pore Water Pressure [ksf]	0	3.21747
Excess Pore Water Pressure [ksf]	0	0.0787545
Degree of Consolidation [%]	0	77.6532
Pre-consolidation Stress [ksf]	0.20975	4.39048
Over-consolidation Ratio	1	1.44934
Void Ratio	0	1.45511
Permeability [ft/y]	0	0.081624
Coefficient of Consolidation [ft^2/y]	0	18.25
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.877668

Stage: Stage 6 = 2 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	1.58558
Consolidation Settlement [in]	0	1.16096
Immediate Settlement [in]	0	0.424618
Secondary Settlement [in]	0	0
Loading Stress [ksf]	0.0360269	0.2
Effective Stress [ksf]	0.2	4.40812
Total Stress [ksf]	0.2	7.57167
Total Strain	0.000111053	0.0334809
Pore Water Pressure [ksf]	0	3.16354
Excess Pore Water Pressure [ksf]	0	0.0248242
Degree of Consolidation [%]	0	93.1533
Pre-consolidation Stress [ksf]	0.20975	4.40175
Over-consolidation Ratio	1	1.39801
Void Ratio	0	1.43868
Permeability [ft/y]	0	0.081624
Coefficient of Consolidation [ft^2/y]	0	18.25
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.879832

Stage: Stage 7 = 75 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	1.67091
Consolidation Settlement [in]	0	1.24629
Immediate Settlement [in]	0	0.424618
Secondary Settlement [in]	0	0
Loading Stress [ksf]	0.0360269	0.2
Effective Stress [ksf]	0.2	4.43295
Total Stress [ksf]	0.2	7.57167
Total Strain	0.000111053	0.0339412
Pore Water Pressure [ksf]	0	3.13872
Excess Pore Water Pressure [ksf]	-5.76458e-021	7.773e-021
Degree of Consolidation [%]	0	100
Pre-consolidation Stress [ksf]	0.20975	4.42657
Over-consolidation Ratio	1	1.37559
Void Ratio	0	1.43145
Permeability [ft/y]	0	0.081624
Coefficient of Consolidation [ft^2/y]	0	18.25
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.88082

Loads

1. Rectangular Load

Length	100 ft
Width	20 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	2000 ft^2
Load	0.2 ksf
Depth	0 ft
Installation Stage	Stage 1 = 0 y

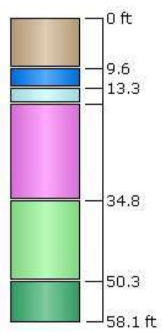
Coordinates

X [ft]	Y [ft]
-50	-10
50	-10
50	10
-50	10

Soil Layers

Ground Surface Drained: Yes

Layer #	Type	Thickness [ft]	Depth [ft]	Drained at Bottom
1	1) Fill: Granular Borrow	9.6	0	No
2	2) Wetland Deposit: Soft SILT	3.7	9.6	No
3	3) Glaciomarine Deposit: Medium Stiff SILT	3	13.3	No
4	4) Glaciomarine Deposit: Med Silty SAND	18.5	16.3	No
5	5) Glacial Till: Dense, Sandy SILT	15.5	34.8	No
6	6) Glacial Till: Very Dense, Silty SAND	7.8	50.3	No



Soil Properties

Property	1) Fill: Granular Borrow	2) Wetland Deposit: Soft SILT	3) Glaciomarine Deposit: Medium Stiff SILT	4) Glaciomarine Deposit: Med Silty SAND	5) Glacial Till: Dense, Sandy SILT	6) Glacial Till: Very Dense, Silty SAND
Color						
Unit Weight [kips/ft^3]	0.125	0.051	0.093	0.105	0.12	0.13
Saturated Unit Weight [kips/ft^3]	0.1313	0.109	0.115	0.128	0.134	0.145
Immediate Settlement	Enabled	Enabled	Enabled	Enabled	Enabled	Enabled
Es [ksf]	450	54	72	155	252	325
Esur [ksf]	1800	216	288	620	1008	1300
Primary Consolidation	Disabled	Enabled	Enabled	Disabled	Disabled	Disabled
Material Type		Non-Linear	Non-Linear			
Cc		1.3	0.13			
Cr		0.13	0.02			
e0		1.5	0.955			
OCR	1	1	1.5	1	1	1
Cv [ft^2/y]		6.458	18.25			
B-bar		1	1			
Undrained Su A [kips/ft2]	0	0	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8	0.8	0.8
Piezo Line ID	1	1	1	1	1	1

Groundwater

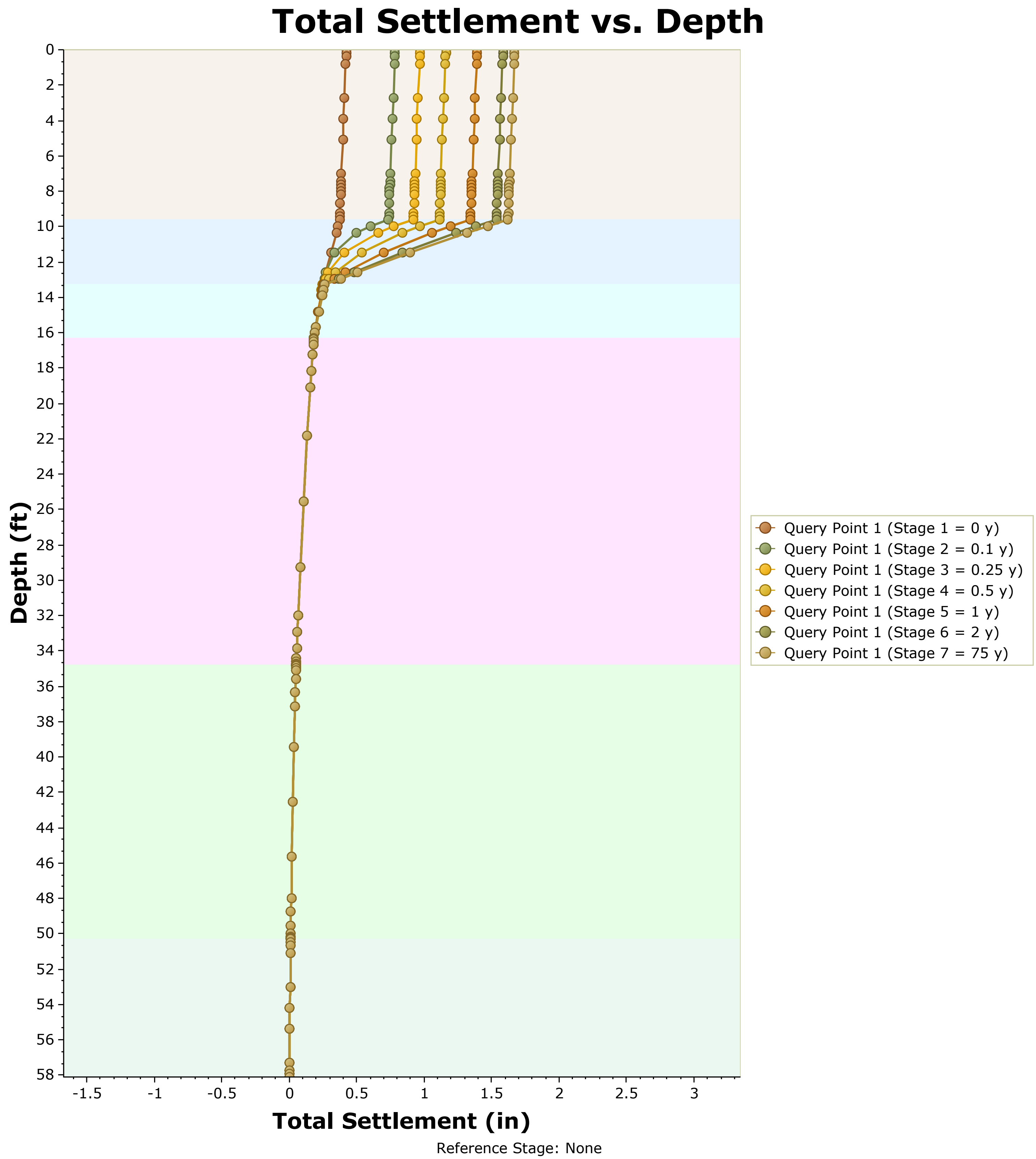
Groundwater method	Piezometric Lines
Water Unit Weight	0.0624 kips/ft^3

Piezometric Line Entities

ID	Depth (ft)
1	7.8 ft

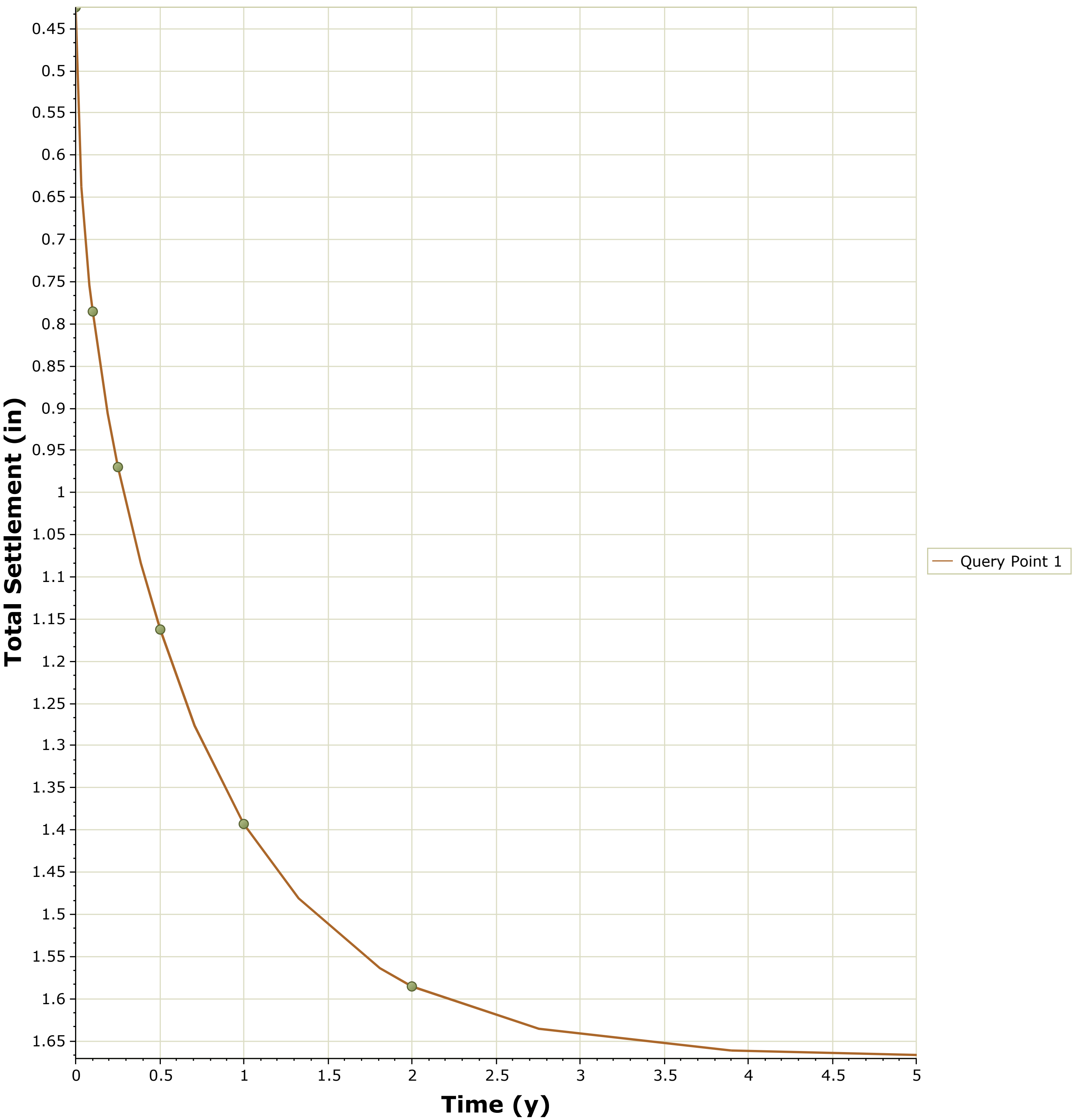
Query Points

Point #	(X,Y) Location	Number of Divisions
1	-4.61853e-014, -8.88178e-016	Auto: 75



rocscience SETTLE3D 3.015	Project: 21686 Charlotte Moosehorn Bridge - Abutment No. 2 Approach		
	Analysis Description: Time Dependent Analysis		
	Drawn By: N. Pukay	Company: MaineDOT	
	Date: 4/8/2025, 9:07:01 AM	File Name: 21686 Charlotte - Abutment No. 2 Approach Settlement d3.s3z	

Time vs. Total Settlement



Reference Stage: None

Total Settlement at Depth = 0 ft

rocscience	*Project* 21686 Charlotte Moosehorn Bridge - Abutment No. 2 Approach		
	Analysis Description Time Dependent Analysis		
	Drawn By N. Pukay	*Company* MaineDOT	
SETTLE3D 3.015	*Date* 4/8/2025, 9:07:01 AM	*File Name* 21686 Charlotte - Abutment No. 2 Approach Settlement d3.s3z	

Objective:

1) To estimate soil parameters for Settle 3D analysis at Abutment No. 2. approach

Given:

1) Boring Logs BB-CHAR-101, -102, -201, -202 and lab test data.

Assumptions:

1) Groundwater is at Q1.1 water elevation or El. 75.0
2) MaineDOT Bridge Design Guide (BDG) Soil Type 4 is used to construct the proposed raise roadway grade (approximately 19 inches).
3) Unless otherwise noted, BB-CHAR-202 will be used to determine strata elevations and consistencies for the Abutment No. 2 approach

References:

1) Hough, B. K. (1969). Basic soils engineering
2) Holtz, R. D., & Kovacs, W. D. (1981). An introduction to geotechnical engineering (1st ed.)
3) Das, B. M. (2014). Principles of geotechnical engineering (7th ed.)
4) Bowles, J. E. (2016). Foundations analysis and design (5th ed.)
5) Cox, C., & Mayne, P. W. Constitutive model input parameters for numerical analyses of geotechnical problems: An in-situ testing case study
6) Andrews, D. W. (1986). The engineering aspects of the Presumpscot formation.
7) Edmunds TWP - Washington County Soils Report 54-21: Hobart Stream Bridge, Route US I. Maine Department of Transportation, 1954.

Calculations for approach embankment behind Abutment No. 2

Surcharge Load

Maximum depth of new fill = 19 inches

$H_{fill} := 19in = 1.583 \cdot ft$

$\gamma_{fill} := 125pcf$ BDG Table 3-3, Soil Type 4, Granular Borrow

$\sigma_{z\_induced} := \gamma_{fill} \cdot H_{fill}$

$\boxed{\sigma_{z\_induced} = 0.2 \cdot ksf}$

Existing Ground Elevation = El. 82.8 ft

Soil Layer 1 (Elev. 82.8 - 73.2) Fill: Granular Borrow, with drainage system

$N_1 := 30$ Assumed

$$E_{s1} := \frac{500(N_1 + 15)}{50} ksf$$

Bowles Table 5-6, Equation for stress-strain modulus E_s for Sand (normally consolidated)

$\boxed{E_{s1} = 450 \cdot ksf}$

$E_{ur1} := 4 \cdot E_{s1}$ Mayne and Cox, Eq. 5 Constitutive Model Input Parameters, $E_s = E_{50}$

$\boxed{E_{ur1} = 1800 \cdot ksf}$

$\boxed{\gamma_{dry1} := 125pcf}$ BDG Table 3-3, Soil Type 4, Granular Borrow

$w_{sat1} := 5\%$ Assumed

$\gamma_{sat1} := \gamma_{dry1} \cdot (1 + w_{sat1})$

$\boxed{\gamma_{sat1} = 131.3 \cdot pcf}$

Soil Layer 2 (Elev. 73.2 - 69.5) Wetland Deposit: Soft, SILT, little peat, little sand

$N_{60\_2} := 3$

$$E_{s2} := \frac{300(N_{60\_2} + 6)}{50} ksf$$

Bowles Table 5-6, Equation for stress-strain modulus E_s for Silt

$\boxed{E_{s2} = 54 \cdot ksf}$

$E_{ur2} := 4 \cdot E_{s2}$

Mayne and Cox, Eq. 5 Constitutive Model Input Parameters, $E_s = E_{50}$

$\boxed{E_{ur2} = 216 \cdot ksf}$

$\boxed{\gamma_{dry2} := 51 \cdot pcf}$ Das, Table 3.2: Dry Unit Weights, Soft Organic Clay 38-51 pcf

$w_{sat2} := 112.9\%$ BB-CHAR-202, 3D Natural Water Content

$\gamma_{sat2} := \gamma_{dry2} \cdot (1 + w_{sat2})$

$\boxed{\gamma_{sat2} = 109 \cdot pcf}$

$Cc_2 := 0.0115 w_{sat2} \cdot 100$

Das, Table 11.6: Correlations for Compression Index, Organic soils, peats, organic silt, and clay

$\boxed{Cc_2 = 1.3}$

$\boxed{C_{r2} := 0.13}$ Assume 10% of Cc

$\boxed{OCR_2 := 1.0}$ Conservatively assume normally consolidated

$\boxed{e_2 := 1.5}$ Assumed

$\boxed{C_{v2} := 6.458 \frac{ft^2}{yr}}$ Settle3D recommended values for organic silt - lower bound

Soil Layer 3 (69.5 - 66.5) Glaciomarine Deposits: SILT, some clay, little sand

$N_{60\_3} := 6$

Note: Material was recovered as part of a split sample (4D/A). Spoon blow counts were 2-7-9-8 with a resulting N60 of 26. Conservatively reduce N60 to 6 (medium stiff) in consideration of BB-CHAR-102 samples at a similar depth.

$E_{s3} := \frac{300(N_{60\_3} + 6)}{50}\,ksf$

Bowles Table 5-6, Equation for stress-strain modulus E_s for Silt

$\boxed{E_{s3} = 72 \cdot ksf}$

$E_{ur3} := 4 \cdot E_{s3}$

Mayne and Cox, Eq. 5 Constitutive Model Input Parameters, $E_s = E_{50}$

$\boxed{E_{ur3} = 288 \cdot ksf}$

$\boxed{\gamma_{dry3} := 93 \cdot pcf}$

Das, Table 3.2: Dry Unit Weights, Soft Clay 73-93 pcf

$w_{sat3} := 23.5\%$

BB-CHAR-202, 4D Natural Water Content

$\gamma_{sat3} := \gamma_{dry3} \cdot (1 + w_{sat3})$

$\boxed{\gamma_{sat3} = 115 \cdot pcf}$

$\boxed{C_{c3} := 0.13}$

Edmunds Township, Soils Report 54-21

$\boxed{C_{r3} := 0.02}$

Edmunds Township, Soils Report 54-21

$d_3 := 82.8ft - 69.5ft = 13.3 \cdot ft$

Depth to top of soil layer 3

$\boxed{OCR_3 := 1.5}$

Andrews, Table IV, OCR at varying depths
At depth=10, OCR=2.25; At depth=15, OCR=1.47

$\boxed{e_3 := 0.955}$

Edmunds Township, Soils Report 54-21

$C_{v3} := 0.05 \frac{ft^2}{day}$

Andrews, pg. 11, Cv range from 0.05-0.15 square feet per day. Choose lower bound.

$\boxed{C_{v3} = 18.262 \frac{ft^2}{yr}}$

Soil Layer 4 (66.5 - 48.0) Glaciomarine Deposits: Medium dense, Silty, fine SAND

$N_{60\_4} := 16$

$$E_{s4} := \frac{250 \cdot (N_{60\_4} + 15)}{50} ksf$$

Bowles Table 5-6, Equation for stress-strain modulus E_s for Sand (saturated)

$\boxed{E_{s4} = 155 \cdot ksf}$

$E_{ur4} := 4 \cdot E_{s4}$

Mayne and Cox, Eq. 5 Constitutive Model Input Parameters, $E_s = E_{50}$

$\boxed{E_{ur4} = 620 \cdot ksf}$

$\boxed{\gamma_{dry4} := 105 \cdot pcf}$

Das, Table 3.2: Dry Unit Weights, Angular-Grained Silty Sand 102-121 pcf. Sand component is poorly graded.

$w_{sat4} := 21.7\%$

BB-CHAR-201, 3D Natural Water Content

$\gamma_{sat4} := \gamma_{dry4} \cdot (1 + w_{sat4})$

$\boxed{\gamma_{sat4} = 128 \cdot pcf}$

Soil Layer 5 (48.0 - 32.5) Glacial Till: Dense, Sandy SILT, some gravel

$N_{60\_5} := 36$

$$E_{s5} := \frac{300 \cdot (N_{60\_5} + 6)}{50} ksf$$

Bowles Table 5-6, Equation for stress-strain modulus E_s, Sandy Silt

$\boxed{E_{s5} = 252 \cdot ksf}$

$E_{ur5} := 4 \cdot E_{s5}$

Mayne and Cox, Eq. 5 Constitutive Model Input Parameters, $E_s = E_{50}$

$\boxed{E_{ur5} = 1008 \cdot ksf}$

$\boxed{\gamma_{dry5} := 120 \cdot pcf}$

Holtz & Kovacs, Table 2-1: Dry Unit Weights, Glacial Till 106-144 pcf.

$w_{sat5} := 11.7\%$

BB-CHAR-102, 14D Natural Water Content

$\gamma_{sat5} := \gamma_{dry5} \cdot (1 + w_{sat5})$

$\boxed{\gamma_{sat5} = 134 \cdot pcf}$

Soil Layer 6 (32.5 - 24.7) Glacial Till: Very Dense, Silty SAND, some gravel

$N_{60\_6} := 50$

$E_{s6} := \dfrac{250 \cdot (N_{60\_6} + 15)}{50} ksf$

Bowles Table 5-6, Equation for stress-strain modulus E_s, Sand (saturated)

$\boxed{E_{s6} = 325 \cdot ksf}$

$E_{ur6} := 4 \cdot E_{s6}$

Mayne and Cox, Eq. 5 Constitutive Model Input Parameters, $E_s = E_{50}$

$\boxed{E_{ur6} = 1300 \cdot ksf}$

$\boxed{\gamma_{dry6} := 130 \cdot pcf}$

Holtz & Kovacs, Table 2-1: Dry Unit Weights, Glacial Till 106-144 pcf.

$w_{sat6} := 11.7\%$

BB-CHAR-102, 14D Natural Water Content

$\gamma_{sat6} := \gamma_{dry6} \cdot (1 + w_{sat6})$

$\boxed{\gamma_{sat6} = 145 \cdot pcf}$

NO. 340-L410 DIETZGEN GRAPH PAPER
SEMI-LOGARITHMIC
4 CYCLES X 10 DIVISIONS PER INCH

EUGENE DIETZGEN CO.
MADE IN U. S. A.

Edmunds TWP - Washington County Soils Report 54-21: Hobart Stream Bridge, Route US I. Maine Department of Transportation, 1954.

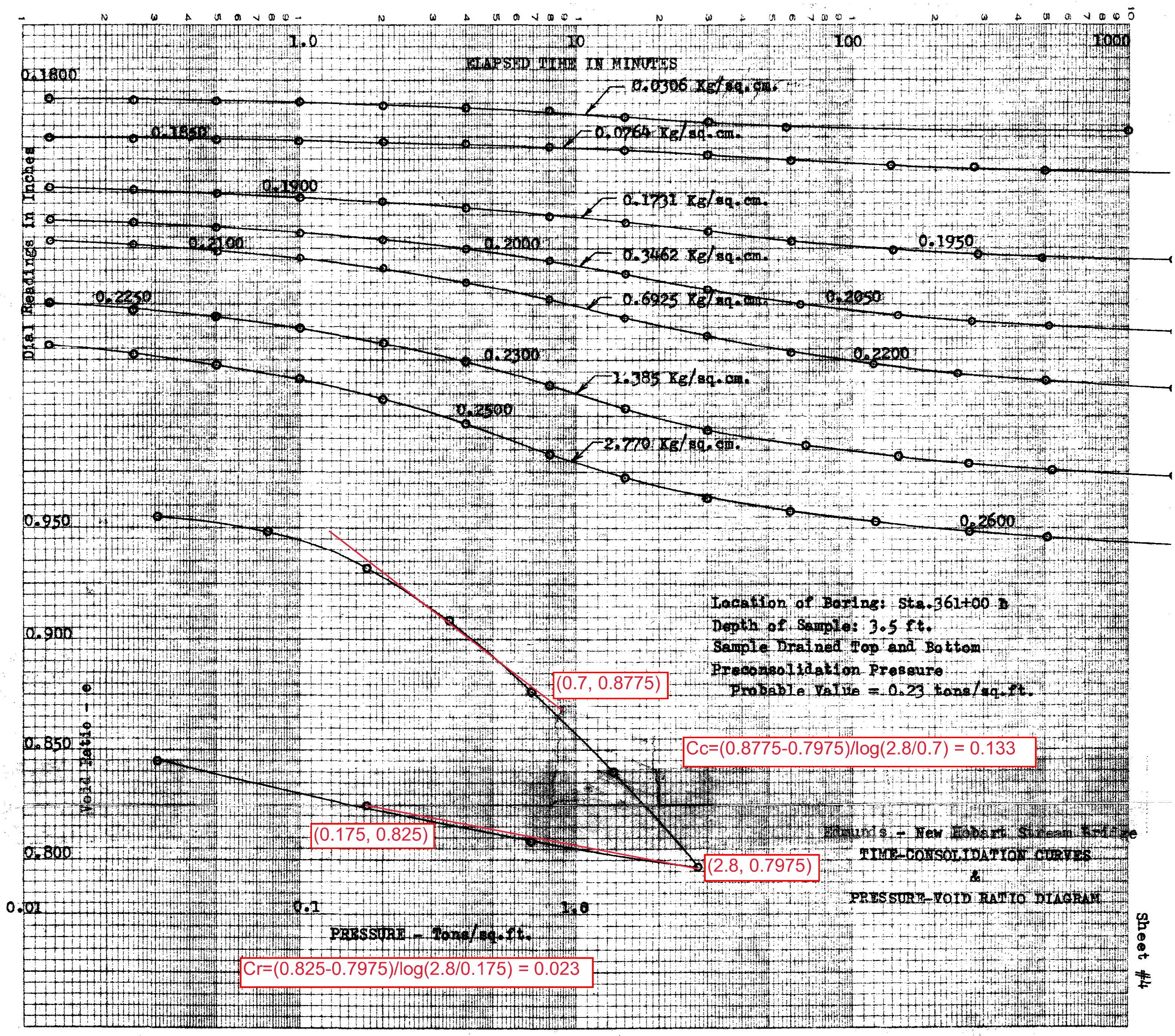


Table 11.6 Correlations for Compression Index, C_c*

Equation	Reference	Region of applicability
$C_c = 0.007(LL - 7)$	Skempton (1944)	Remolded clays
$C_c = 0.01w_N$		Chicago clays
$C_c = 1.15(e_O - 0.27)$	Nishida (1956)	All clays
$C_c = 0.30(e_O - 0.27)$	Hough (1957)	Inorganic cohesive soil: silt, silty clay, clay
$C_c = 0.0115w_N$		Organic soils, peats, organic silt, and clay
$C_c = 0.0046(LL - 9)$		Brazilian clays
$C_c = 0.75(e_O - 0.5)$		Soils with low plasticity
$C_c = 0.208e_O + 0.0083$		Chicago clays
$C_c = 0.156e_O + 0.0107$		All clays

After Rendon-Herrero, 1980. With permission from ASCE.
Note: e_O = *in situ* void ratio; w_N = *in situ* water content.

Nagaraj and Murty (1985) expressed the compression index as

$$C_c = 0.2343\left[\frac{LL\,(\%)}{100}\right]G_s \tag{11.37}$$

Based on the modified Cam clay model, Wroth and Wood (1978) have shown that

$$C_c \approx 0.5G_S\frac{[PI(\%)]}{100} \tag{11.38}$$

where PI = plasticity index.

If an average value of G_s is taken to be about 2.7 (Kulhawy and Mayne, 1990)

$$C_c \approx \frac{PI}{74} \tag{11.39}$$

More recently, Park and Koumoto (2004) expressed the compression index by the following relationship.

$$C_c = \frac{n_o}{371.747 - 4.275n_o} \tag{11.40}$$

where n_o = *in situ* porosity of the soil

11.10 Swell Index (C_s)

The swell index is appreciably smaller in magnitude than the compression index and generally can be determined from laboratory tests. In most cases,

$$C_s \simeq \tfrac{1}{5} \text{ to } \tfrac{1}{10}C_c$$

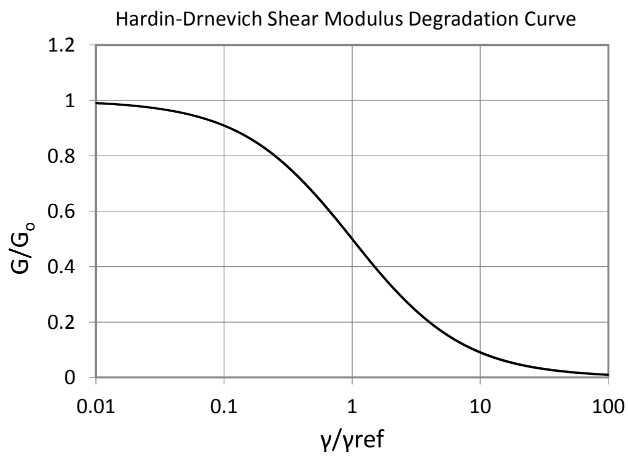


Figure 1. Shear modulus reduction curve
(after Hardin and Drnevich 1972)

$$\frac{G}{G_o} = \frac{1}{1 + \left(\frac{\gamma}{\gamma_{ref}}\right)^{\alpha}} \tag{2}$$

$$\frac{G}{G_o} = \frac{1}{1 + a\left(\frac{\gamma}{\gamma_{ref}}\right)} \tag{3}$$

To construct the site specific G-γ modulus degradation curve, the working shear strain γ_{DMT} corresponding with G_{DMT} must be determined (Cox & Mayne, 2015).

Once the G-γ modulus degradation curve is determined using in-situ testing, a corresponding E-γ modulus degradation curve can be constructed using Hooke's law and elastic theory as shown in Figure 3.

Then, the secant modulus in triaxial testing at 50 percent strength E_{50} can also be determined using values obtained from SDMT testing. Where according to Vermeer (2001),

$$E_{50} \cong M_{DMT} \tag{4}$$

The unloading/reloading modulus in the drained/undrained triaxial test, $E_{ur,}$ cannot readily be determined using data obtained from DMT testing and must be calculated using accepted relationships if not using laboratory testing such as that given by Vermeer (2001),

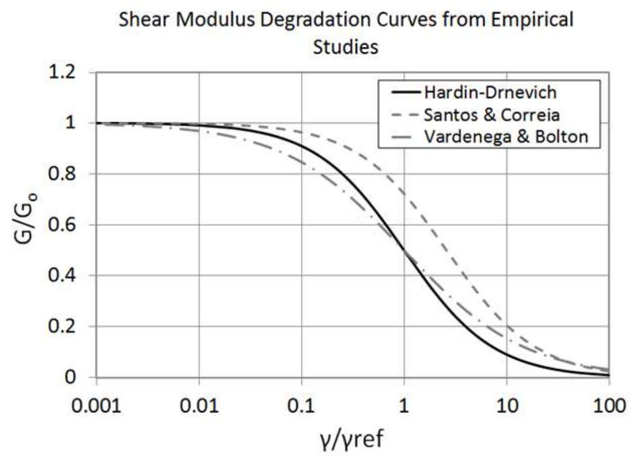


Figure 2. Reduction curves from fitted experimental data studies

$$E_{ur} \cong 4E_{50} \tag{5}$$

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 $\gamma_{0.7}$.

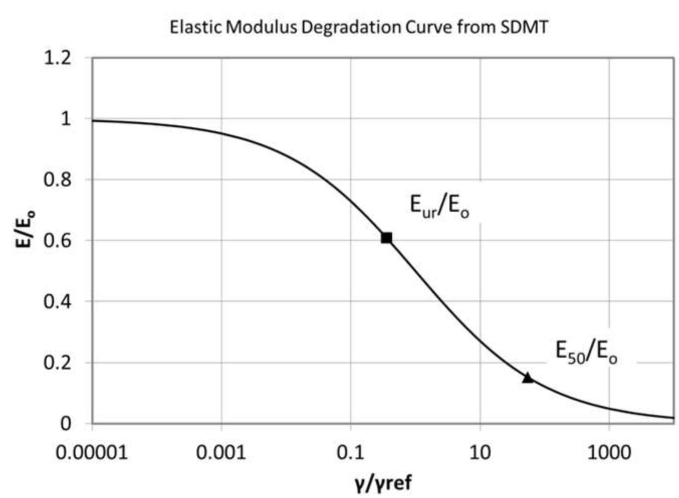


Figure 3. Elastic Modulus reduction curve using SDMT

The coefficient of compression varies with orientation within the soil mass. The vertical coefficient is measured by conventional testing methods. The horizontal coefficient takes special devices. The co-efficients are also a function of testing procedures.

Vertical coefficients for the Presumpscot Formation in the Portland area are reported in the 0.05 to 0.15 square feet per day range. One report gives the ratio of horizontal to vertical coefficient of 1.2 to 1.5.

With information on the coefficient of consolidation in hand, the settlement rate can be predicted if the drainage characteristics of the deposit, such as sand layer spacings and overlying soil permeability, are known.

Over-Consolidation Ratio, OCR

The Presumpscot Formation is an over-consolidated deposit. The upper crust has been significantly over-consolidated due, probably to the combined forces of dessication, drying and wetting in the presence of certain salts (Bowles, 1979), and chemical bonding. The soft, deeper deposit is also slightly over-consolidated as the result of secondary compression.

The amount of over-consolidation can be expressed as the Over-Consolidation Ratio, OCR:

$$OCR = \frac{\text{Apparent past vertical pressure, Pc}}{\text{Existing vertical pressure, P}}$$

Table IV, Over-Consolidation Ratio, presents OCR data for a well-documented site in Portland, Maine.

TABLE IV

Over-Consolidation Ratio

Depth (Feet)	OCR
5	4.4
10	2.25
15	1.47
20	1.20
30	1.14
40	1.13
50	1.12
60	1.12

Permeability

The coefficient of permeability, k, of the Presumpscot Formation varies with the deposit's void ratio and is different in the horizontal and vertical directions because of natural stratification. The permeability of the silty clay is on the order of 5×10^{-8} to 1×10^{-7} centimeters per second. The ratio of horizontal to vertical permeability of the silty clay is estimated to be 1.2 to 1.5. It is important to note that these values are for small laboratory samples of the silty clay. The permeability of a Presumpscot Formation deposit, as a whole, would probably be higher because of sand layers.

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)	$E_s = 500(N + 15)$ $= 7000\sqrt{N}$ $= 6000N$ — — — ‡$E_s = (15\,000 \text{ to } 22\,000) \cdot \ln N$	$E_s = (2 \text{ to } 4)q_u$ $= 8000\sqrt{q_c}$ — — — $E_s = 1.2(3D_r^2 + 2)q_c$ *$E_s = (1 + D_r^2)q_c$
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	$E_s = 1200(N + 6)$ $= 600(N + 6) \quad N \le 15$ $= 600(N + 6) + 2000 \quad N > 15$	
Clayey sand	$E_s = 320(N + 15)$	$E_s = (3 \text{ to } 6)q_c$
Silts, sandy silt, or clayey silt	$E_s = 300(N + 6)$ If $q_c < 2500$ kPa use $\quad$ $^{\$}E_s' = 2.5q_c$ $2500 < q_c < 5000$ use $\quad$ $E_s' = 4q_c + 5000$ where $E_s' = \text{constrained modulus} = \dfrac{E_s(1-\mu)}{(1+\mu)(1-2\mu)} = \dfrac{1}{m_v}$	$E_s = (1 \text{ to } 2)q_c$
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

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/ft³	Dry unit weight, γ_d kN/m³
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

FAY, SPOFFORD & THORNDIKE, INC.
ENGINEERS
BOSTON

PROJECT ____________________

FILE NUMBER ____________________
SHEET NUMBER ____________________

SUBJECT ____________________

DATE ____________________
COMPUTED BY ____________________
CHECKED BY ____________________

Table 11.3 Summary of Friction Angle Data for Use in Preliminary Design

	Friction Angles							
	Slope Angle of Repose		At Ultimate Strength		At Peak Strength			
					Medium Dense		Dense	
Classification	$i(°)$	Slope (vert. to hor.)	$\phi_{cv}(°)$	$\tan \phi_{cv}$	$\phi(°)$	$\tan \phi$	$\phi(°)$	$\tan \phi$
Silt (nonplastic)	26 to 30	1 on 2 / 1 on 1.75	26 to 30	0.488 / 0.577	28 to 32	0.532 / 0.625	30 to 34	0.577 / 0.675
Uniform fine to medium sand	26 to 30	1 on 2 / 1 on 1.75	26 to 30	0.488 / 0.577	30 to 34	0.577 / 0.675	32 to 36	0.675 / 0.726
Well-graded sand	30 to 34	1 on 1.75 / 1 on 1.50	30 to 34	0.577 / 0.675	34 to 40	0.675 / 0.839	38 to 46	0.839 / 1.030
Sand and gravel	32 to 36	1 on 1.60 / 1 on 1.40	32 to 36	0.625 / 0.726	36 to 42	0.726 / 0.900	40 to 48	0.900 / 1.110

LOOSE 28 ≤ φ ≤ 30
SEE FIG. 11.14 LAMBE/WHIT.

31°

35°

GLACIAL TILL

From B. K. Hough, *Basic Soils Engineering*. Copyright © 1957, The Ronald Press Company, New York.
Note. Within each range, assign lower values if particles are well rounded or if there is significant soft shale or mica content, higher values for hard, angular particles. Use lower values for high normal pressures than for moderate normal pressure.

Table 1.4 Porosity, Void Ratio, and Unit Weight of Typical Soils in Natural State

Description	Porosity (n)	Void Ratio (e)	Water Content (w)[a]	Unit Weight g/cu cm γ_d[b]	g/cu cm γ_{sat}[c]	lb/cu ft γ_d	lb/cu ft γ_{sat}
1. Uniform sand, loose	0.46	0.85	32	1.43	1.89	90	118
2. Uniform sand, dense	0.34	0.51	19	1.75	2.09	109	130
3. Mixed-grained sand, loose	0.40	0.67	25	1.59	1.99	99	124
4. Mixed-grained sand, dense	0.30	0.43	16	1.86	2.16	116	135
5. Windblown silt (loess)	0.50	0.99	21	1.36	1.86	85	116
6. Glacial till, very mixed-grained	0.20	0.25	9	2.12	2.32	132	145
7. Soft glacial clay	0.55	1.2	45	1.22	1.77	76	110
8. Stiff glacial clay	0.37	0.6	22	1.70	2.07	106	129
9. Soft slightly organic clay	0.66	1.9	70	0.93	1.58	58	98
10. Soft very organic clay	0.75	3.0	110	0.68	1.43	43	89
11. Soft montmorillonitic clay (calcium bentonite)	0.84	5.2	194	0.43	1.27	27	80

med. DENSE SAND
$\gamma = \frac{118+130}{2} = 124$

$\gamma \cong 122$

$\frac{124+135}{2}$

$\gamma = 130$ PCF

med. Glacial Clay
$\gamma = 120$

PEAT 90-100 PCF

[a] w = water content when saturated, in per cent of dry weight.
[b] γ_d = dry unit weight.
[c] γ_{sat} = saturated unit weight.

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}	Approx. Range Unif. Coef.,	Void Ratio			Porosity (%)		Dry Wt., γ_{dry}			Wet Wt., γ_{wet}		Sub. Wt., γ_{sub}	
	D_{max}	D_{min}	(mm.)	C_u	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. ≥ 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,

$$D_d = \frac{e_{max} - e}{e_{max} - e_{min}} \tag{2-11}$$

l.

ne; fill in the "given" or measured
he calculations as indicated for (c),

dish $= 462$ g
dish $= 364$ g
$- b) = 98$ g
dish $= 39$ g
$- d) = 325$ g
$100\% = 30.2\%$

y determined in grams (g) on an

geotechnical engineering is density.
is mass per unit volume, so its units
corresponding units in the cgs and
ensity is the ratio that connects the
with the mass side. There are several
nical engineering practice. First, we
ρ, the density of the particles, solid
ρ_w. Or, in terms of the basic masses

$$\frac{M_s + M_w}{V_t} \tag{2-6}$$

(2-7)

(2-8)

of the total density ρ will depend on
he voids as well as the density of the
ould range from slightly above 1000
) to 2.4 Mg/m^3). Typical values of ρ_s
800 kg/m^3 (2.5 to 2.8 Mg/m^3). Most
6 and 2.7 Mg/m^3. For example, a

common mineral in sands is quartz; its $\rho_s = 2.65$ Mg/m^3. Most clay soils have a value of ρ_s between 2.65 and 2.80 Mg/m^3, depending on the predominant mineral in the soil, whereas organic soils may have a ρ_s as low as 2.5 Mg/m^3. Consequently, it is usually close enough for geotechnical work to *assume* a ρ_s of 2.65 or 2.70 Mg/m^3 for most phase problems, unless a specific value of ρ_s is given.

The density of water varies slightly, depending on the temperature. At 4°C, when water is at its densest, ρ_w exactly equals 1000 kg/m^3 (1 g/cm^3), and this density is sometimes designated by the symbol ρ_o. For ordinary engineering work, it is sufficiently accurate to take $\rho_w \approx \rho_o = 1000$ kg/m^3 = 1 Mg/m^3.

There are three other useful densities in soils engineering. They are the dry density ρ_d, the saturated density ρ_{sat}, and the submerged or buoyant density ρ'.

$$\rho_d = \frac{M_s}{V_t} \tag{2-9}$$

$$\rho_{sat} = \frac{M_s + M_w}{V_t} (V_a = 0,\ S = 100\%) \tag{2-10}$$

$$\rho' = \rho_{sat} - \rho_w \tag{2-11}$$

Strictly speaking, total ρ should be used instead of ρ_{sat} in Eq. 2-11, but in most cases completely submerged soils are also completely saturated, or at least it is reasonable to assume they are saturated. The dry density ρ_d is a common basis for judging the degree of compaction of earth embankments (Chapter 5). A typical range of values of ρ_d, ρ_{sat}, and ρ' for several soil types is shown in Table 2-1.

From the basic definitions provided in this section, other useful relationships can be derived, as we show in the examples in the next section.

TABLE 2-1 Some Typical Values for Different Densities of Some Common Soil Materials*

Soil Type	Density (Mg/m^3)		
	ρ_{sat}	ρ_d	ρ'
Sands and gravels	119 – 150 1.9–2.4	93 - 144 1.5–2.3	1.0–1.3 62 – 81
Silts and clays	87 - 131 1.4–2.1	37 - 112 0.6–1.8	0.4–1.1 25 – 69
Glacial tills	131 - 150 2.1–2.4	106 - 144 1.7–2.3	1.1–1.4 69 – 87
Crushed rock	119 - 137 1.9–2.2	93 - 125 1.5–2.0	0.9–1.2 56 – 75
Peats	62 - 69 1.0–1.1	6 - 19 0.1–0.3	0.0–0.1 0 – 6
Organic silts and clays	81 - 112 1.3–1.8	0.5–1.5 31 - 93	0.3–0.8 19 - 50

*Modified after Hansbo (1975).