MAINE DEPARTMENT OF TRANSPORTATION BRIDGE PROGRAM GEOTECHNICAL SECTION AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

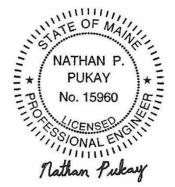
MOOSEHORN BRIDGE STATION ROAD OVER MOOSEHORN BROOK CHARLOTTE, MAINE



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

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1.0 Introduction

The purpose of this Geotechnical Design Report is to present subsurface information and provide geotechnical design recommendations for the replacement of 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 (N₆₀) 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		No	n-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 9^{th} 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.

<u>Drivability Analyses</u>. 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)	Resis φ _{dyn} =	ubility tance ³ = 0.65 ps)	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)	Resis φ =	ability tance ³ 1.0 ps)	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 $\left(\begin{array}{c c} Soil/Rock \\ Model \end{array}\right)$ $\left(\begin{array}{c c} Top \\ Elevation \\ of Layer \\ (ft) \end{array}\right)$ $\left(\begin{array}{c c} Layer \\ Thickness \\ (ft) \end{array}\right)$ $\left(\begin{array}{c c} \gamma_{e^1}(pcf) \end{array}\right)$ $\left(\begin{array}{c c} \phi'^2(deg) \end{array}\right)$ $\left(\begin{array}{c c} k_s^3(pci) \end{array}\right)$									
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 $\left \begin{array}{c} Soil/Rock \\ Model \end{array} \right \left \begin{array}{c} Top \\ Elevation \\ of Layer \\ (ft) \end{array} \right \left \begin{array}{c} Layer \\ Thickness \\ (ft) \end{array} \right \left \begin{array}{c} \phi^{1} \left(pcf \right) \end{array} \right \left \begin{array}{c} \phi^{2} \left(deg \right) \end{array} \right \left \begin{array}{c} k_{s}^{3} \left(pc \right) \end{array} \right \left \begin{array}{c} \phi^{2} \left(deg \right) \end{array} \left \begin{array}{c} \phi^{2} \left(deg \right) \end{aligned} \left \begin{array}{c} \phi^{2} \left(deg \right) \right \left \begin{array}{c} \phi^{2} \left(deg \right) \end{aligned} \left \begin{array}{c} \phi^{2} \left(deg \right) \right \left \begin{array}{c} \phi^{2} \left(deg \right) \end{aligned} \left \begin{array}{c} \phi^{2} $									
Granular Borrow	Reese Sand	84	11	125	32	90			
Wetland Deposit	Reese Sand	73	4	53	26	20			
Glaciomarine Deposit	Glaciomarine Deposit Reese Sand			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 $(\gamma) = 125 \text{ pcf}$
- 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 (heq) taken from the table below:

Abutment Height	h _{eq}
(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_0 , 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 (7th 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.

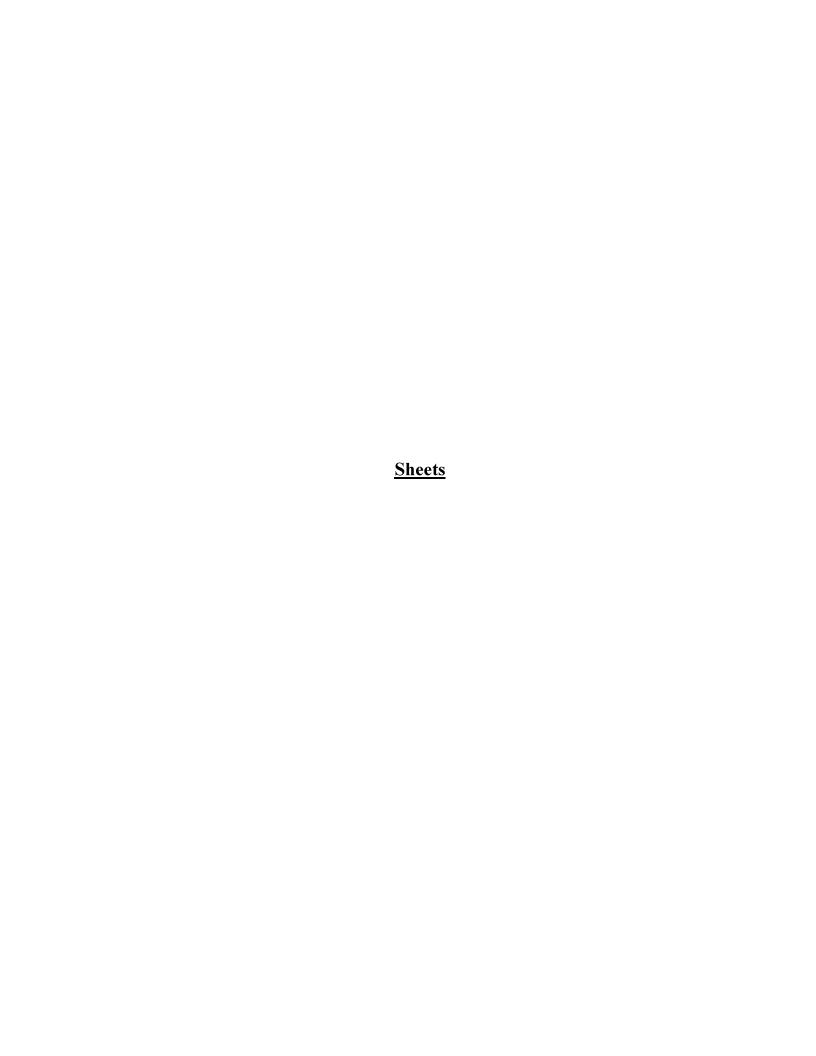
Based on the hydrology report, the typical water elevation is El. 75 ft. A cofferdam will likely be necessary to successfully dewater and construct the abutments. Wood chips were noted in BB-CHAR-101 within the existing fill. Wood chips indicate the presence of wood debris or timber which may obstruct 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

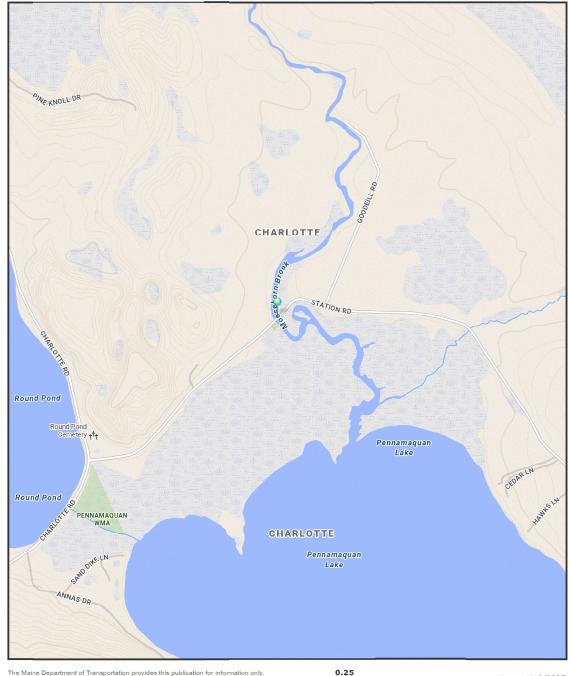
This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Moosehorn Bridge in Charlotte, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed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



CHARLOTTE, MAINE

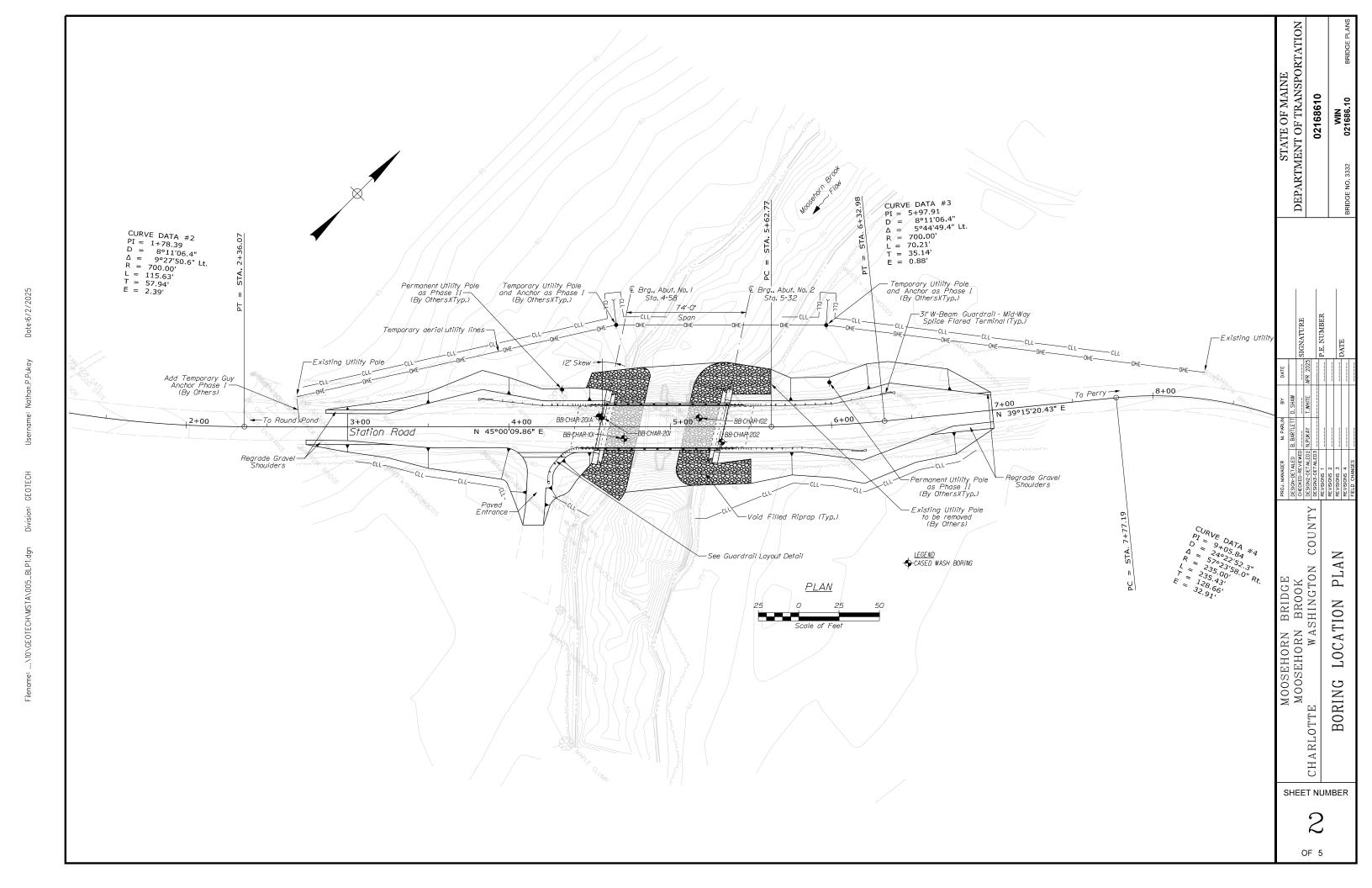


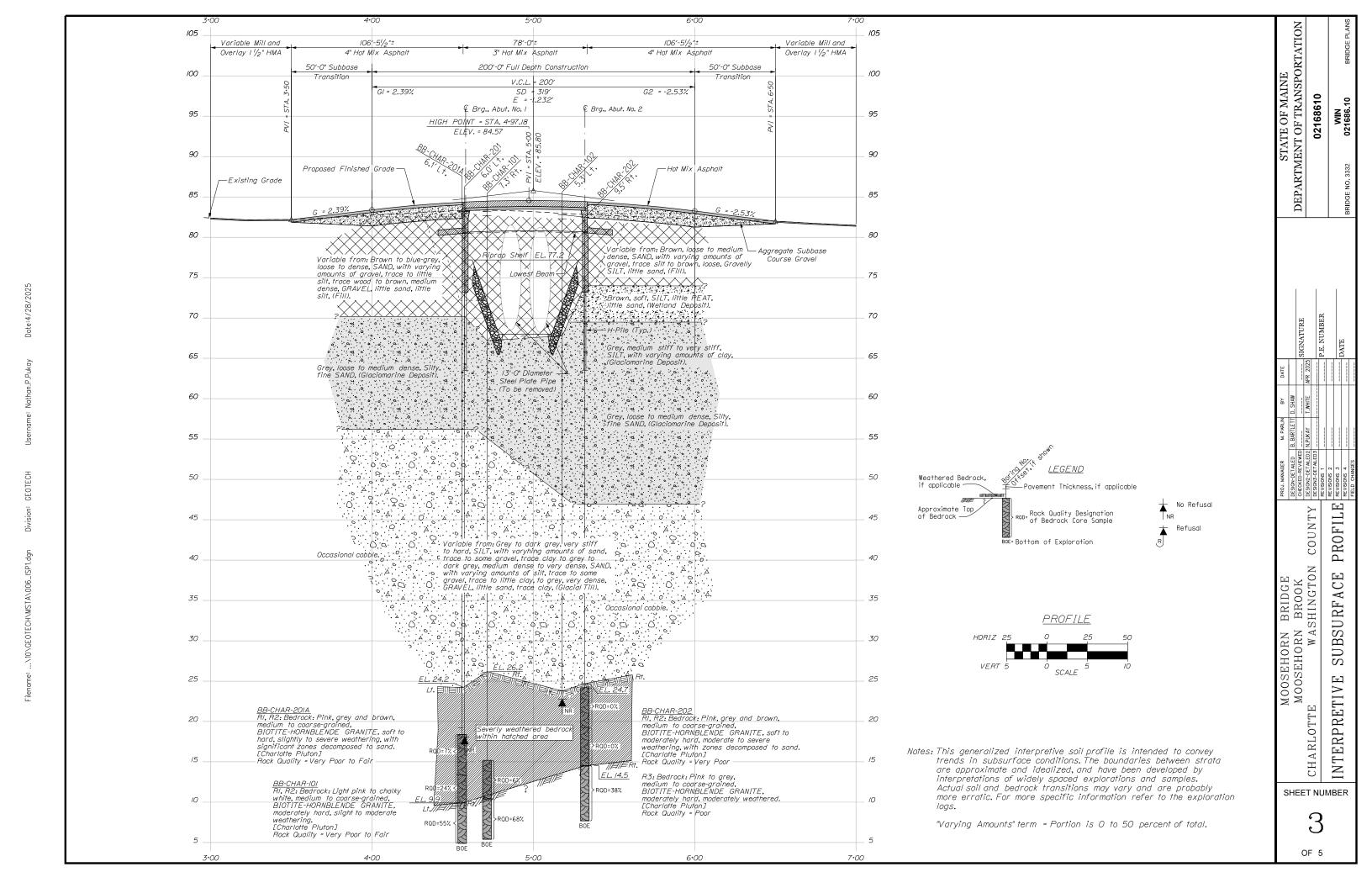
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0.25
Miles
1 inch = 0.28 miles

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

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





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	2D	24/6	3.00 - 5.00	5/5/4/4	9	13	14	1	₩	Brown, domp, medfum dense, QRAVEL, Ifftia send, Ifftia \$11+, (FTII).
5 -							23			
							28	-	₩	
							34	ł	₩	
							23			
10 -	3D	24/3.6	10.00 -	5/3/2/3	5	7	17	ł	₩	Brown, domp, loose, SAND, trace fine gravel, trace afit, (FTII).
		2.000		3/3/2/3		Ė	17			
	ИО	24/0	12.00 - 14.00	3/3/1/3	4	6	19		₩	
	4D	24/10.8	14.00 -	4/4/2/3	63	9	25	-	▓	Blue-grey, damp. loose, Gravelly SAND, troce stit. G#4146 trace wood fragments, (Ftil).
15 -			16.00				73	67.2		trace wood frogments, (FTII). A-I-a, WC=10.
	М	24/0	16.00 - 18.00	15/5/3/3	8	12	49	""		
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20 -	5D	24/15.6	19.00 -	3/4/4/3	83	12	25			Orey, wet, medium dense, STITY SAND, (Gloatemarine Deposit).
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							65			
25 -			25.00 -				78	-		Grey, moist, medium dense, Stity SAND, Ifttle gravel, GM4146 (Glactomarine Deposit).
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							126			Cobbles.2
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			32.00		-		98			
							144			Cobbles.2
							153	-		
35 -	9D	24/19.2	35.00 - 37.00	6/9/43/50	523	75	102	ł		Crey, domp, hard, SILT, some send, liftile alay, liftile gravel and rock fragments, (Glocial TTII).
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							276 311	-		
							471	1		
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							91	1		
45 -			45.00 -				114			Grey+ damp. hard. SILT+ same gravel+ same sand+ lfttle alay+ (Glasfal Tfll).
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							130	i		Cabbles, 2
							102			Cobble from 48.7' to 49.1'.
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55 -	130	20.4/	55.00 - 56.70	10/35/51/(50/2")	86	125	171			Grey, damp, hard, SILT, little gravel, little sand, little slay: 1" piece of pink granite, (Glasial IIII).
							275	26.2		57.0-
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	140	2.4/1.2	59.00 - 59.20	50/2.4"			480	1		Fractured, přínk, GRANITE, (Meathered Bedrock).
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	15D	6/4.8	65.00 - 65.50	117/6"			\forall	16.8		Grey, SILT. some sond, some pink, frostured rock frogments, (Weathered Bedrock).
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	sation i	Thes repre	seant approxim	mile boundorfes between	seri ty	est fro	nsf†fens (may be gr	90JB1 -		Poge 2 of 2		

ANDOURNATION OF THE PROPERTY O	Station Road over Mossehern Brook	Boring No.: <u>BB-CHAR-102</u>	Maine Department of Transportatio	ON Project: Mosemen Bridge #3332 corrise Sketten Road over Mosemen Broak	IAR-201	NOI
Company Comp	SBI 1/2004 1/20	Auger 1D/00: 4" SSA Sampler: Standard Split Spoon	US_CUSTOMARY_UNITS Driller: Seaboard Drilling_LLC Elev Operator: Hansoom/Mail Datu	#1N: 216 ret1en (ft.) 83.2 Auger ID/00: 5" Sel1d St m: NAVD88 Sompler: Standard Sp	· orr	AT
	Dote Start/Finfsh: 1:45pm 6/12/17_12:45pm 6/12/17 DrTilfing Method: SSA & D+W	Core Barrel: NO-2" Water Level*: See Remarks Rose & Cathead EE	Date Start/Finish: 3/12-14/2024 Dril			
	Definitions I in the Core Service I in the	near Strength (pef Typ Robust Torvins Share Strength (pef I in (pef) UC = Noter Contents percent I LL = Litual Litet I LL = Litual Litet I LL = Litual Litet I	Definition: D = Solid Topern Somple SSI = Selid Sitem I, MD = Unsuccessful Solid Speep Somple Atheron U = Thin Yell Tube Semple MSI = Unsuccessful Toth Yell Tube Somple Atheron MSI = Unsuccessful Tinth Yell Tube Somple Atheron MSI = Walfort of 144 V = Final (Vers Short Tube I) Tube Somple Atheron MSI = Walfort of 144 V = Final (Vers Short Tube I) Tube Somple Atheron MSI = Walfort of 144 V = Final (Vers Short Tube I) Tube Somple Atheron MSI = Walfort of 144 V = Final (Vers Short Tube I) Tube Somple Atheron MSI = Walfort of 144 V = Final (Vers Short Tube I) Tube II Tube Somple Atheron MSI = Walfort of 144 MSI = Wal	uger Su(leb) = Leb Vene Undrefred Shear Strength (pef) VC = Noter Centent, p	aroant	MAIN SANS 310
	2 4 9 0 0 9 £ 2 5 5 7 Visual Deer	Laberatory Testing Results/ AASTO	Sample Information	§ VTsual Description and Remarks	Loberatory Testing Results/ AASHTD	
AND STATE OF THE PROPERTY OF T	SSA 82.5 66 of povement.	0.5	1-0-00 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0		Uniffied Class	TATI SNT (
A SACRETING TO SACRET THE SACRET STATE OF THE SACRET STATE S		tfum to coarsa SAND, same gravel, 0#414663 A-1-a, SM-SM NC=4.0%				
### 10 10 10 10 10 10 10 10 10 10 10 10 10	30 24/9.6 7.00 1/2/3/2 5 5 rroce at 11, troce with	le quartz frogmente: (FTII).	- 5 - 1D 24/11 5.00 - 3/3/2/2 5	Brown, motat, loose, SMO, some grovel, Iffile stit.		EPAI
	9.00 1/4/2/3 6 6 (FTII).					
	15		MO 24/0 10.00 - 5/2/3/2 5	9 16 fl Tghta.		
	13	olat. Josep, fine to medium SAM	. 15	23 70.2		
V	50 24/12 17.00 27/2/4/3 6 8 2 with originals rroad a fine to medium Subs. 1 series of the series of		20 24/18 17.00 2/2/3/5 5	34		
Column C	70 24/15.6 19.00 3/3/4/4 7 7 28 Cope.11.0 14	(D. some stit. (Glocfomorthe	20 30 24/20 20.00 - 3/3/4/4 7	55	A-4. SM	RE
# W MAN MAN WAS NOT WA	31		22.00	48	WC=21.7%	3NATUI 3. NUMI
1	35		- 25 - 40 24/17 25.00 - 4/4/4/6 8	51 14 52 Staffler to 30.		
1				60 55.2	-	
10 10 10 10 10 10 10 10	32.00	anse. STity, fine SAND.	- 30 - 50 24/16 30.00 - 5/5/7/7 12	21 81 Crey, wet, medium dense, STIHy SAMD, ITHHis clay, troop gravel, (Glosfal TIII).		D. S.
10 10 10 10 10 10 10 10	29 42			146		표정 [발 [시]] 다 다 다
10 10 10 10 10 10 10 10	25 120 24/14.4 35.00 5/5/24/26 29 29 clcy. (Glocieror fine De		- 35 - 60 24/T 35.00 - 9/10/17/20 27	212 48 OPEN HGE. HGE.		0 VED 102
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Column C)·		Cooble from 40.8-41.5 ft bgs.		7
10	89					NU
10	140 24/13.2 47.00 14/19/23/28 42 42 9F(N 1111the grown). (Gloof	A-4- C. RC=11.7%	8D 12/6 45:00 - 33/55	3.11.00 10.00		ט
## 150 11/10 150-00 150-	50 (50) 6/4.8 50.00 - (50)/6*1 Dark gray, motat, very	/ dense. Stity SAND. Ifftle cley.	. 50 -	Scoto in From 401-9-50-3, fri bette		GE OK TON
## 150 11/10 150-00 150-	50,50 Source / Iffile growth (Globia	51 TT(1).	90 24/17 51.00 - 19/28/29/40 57 55.00	Orey, vet, very dense. SANO, some stift froce gravel. IClocfol Tfill).		RIDO BROC IING
10 10 15 15 15 15 15 15		d. SILT. some sond. Ifttle clay. c fragments. (Glosfal Tfil).	- 55 - 100 4.8/4.8 55.00 - 70(4.8")			
10 10 10 10 10 10 10 10						HOR W ING
EMBOCALS. 170 181 181 181 181 181 181 181	60 1 22.8 No penetration with 2	split speen.	- 60 - 11D 6/5 60.00 - 90			\mathbf{I}_{Ω} Ω
Elementary To Shell and the second control of the second control						MO MC
SHEET NUMBER To	65		- 65 - 120 1.2/1.2 65.00 - 50(1.2")	18.1 Pfnk to grey, set, very dense, GRAYEL, some send, troe after (Recthered Bedrock). 65, 56 tom of Exploration of 65, 56 tom of Exploration of 65, 56 tom of 68 to 65	- f	TT
Shemania. 1. Stere (leoials 7.4" logs of 1105 on 06/12/11 6.1" logs of 7.110 06/13/17; Ne water in hele offer coating removed. 2. Presence of cootine is consumed board on off litting period. 2. Presence of cootine is consumed board on off litting period. 3. Stere (leoials 7.4" logs of 1105 on 06/12/11 6.1" logs of 7.110 06/13/17; Ne water in hele offer coating removed. 3. Stere (leoials 7.4" logs of 1105 on 06/12/11 6.1" logs of 7.110 06/13/17; Ne water in hele offer coating removed. 3. Stere (leoials 7.4" logs of 1105 on 06/12/11 6.1" logs of 7.110 06/13/17; Ne water in hele offer coating dendors of the little of 110 offer logs of 1105 on 06/12/11 6.1" logs offer logs of 7.110 06/13/17; Ne water in hele offer coating dendors of the little offer logs of 1105 on 06/12/11 6.1" logs logs offer logs of 1105 on 06/12/11 6.1" logs of 1105 on 06/12/11 6.1" logs offer logs of 1105 on 06/12/11 logs offer logs of 1105 on				WK(5*) braken costing abmotored in hole from 65.1 ft bg (El. 18.1) to 45.1 ft bgs (El. 38.1) .		RL
Eminicial: 1. Institut (localize 1.4.1 logic of 17105 on 06/12/171 6.77 logic of 17110 6/13/171 No vertice for hole offer osaling removed. 2. Presence of accessing consumed board on of 111ing perior for. 2. Presence of accessing consumed consider of 111ing perior for. 2. Properties of accessing consumed consider of 111ing perior for. 2. 2. 20.0 for off 1813 localize for 111ing perior for. 2. 20.0 for off 1813 localize for 111ing perior for. 3. Institution of 11 in the former osalized consideration of 11ing perior for. 3. Institution of 11 in the former osalized consideration of 11ing perior for 11ing perior fo	70		- 70			工
1. Steric (localist 7.4" logs of 17.110 and 19.12/11 is 1.1" log of 17.110 and 19.12/11 is local extern coating removed. 2. Preserve of troopies 15 cossessed board on 47.1117 patrol form. 3. Sering call (localed bods in 6.1.1" removed for 4.1" long heard for 4.1.1 to 45.1 ft logs (Ei, 35.1). 4. 20 (b) The first of t						SHEET NUMBER
StretTfeather lines represent approlites business self-types trendfifters my be greated. Pops 1 of 1 StretTfeather lines represent approlites business self-types trendfifters my be greated. Pops 1 of 1		ng removed.		om 65-1 ft bgs (Ei- 18-1) to 45-1 ft bgs (Ei- 38-1).		4
* storr lead reaching hote been reached the set user conditions storied. Orandester flustuations ray easer due to conditions ray easer due to conditions ray easer due to conditions ray easer due to the time reacurements are read. ** Note: Issue reaction or reactions read and the reactions are read.** ** Note: Issue reaction or reactions read and the reactions ray easer due to the time reacurements are read.** ** Note: Issue reaction or reactions read and the reactions ray easer due to the time reactions ray read.** ** Note: Issue reaction or reactions read and the reactions ray easer due to the time reactions ray read.** ** Note: Issue reaction or reactions ray easer due to the time reactions ray read.** ** Note: Issue reaction or reactions ray read.** ** Note: Issue reaction or reactions ray read.** ** Note: Issue reaction ray read of the time reactions ray read.** ** Note: Issue reaction or reactions ray read.** ** Note: Issue reaction ray read.** ** Note: Issue r	Stretification lines represent approximate boundaries between sell typess transitions may be gradual. * Soter level readings have been mode at times and under conditions atotal. Organization flustuations may assure due to conditions at	Page 1 of 1	* Noter level readings have been made at times and under senditions state	ted. Graundwater flustuations may seem due to conditions other	-201	OF 5

Driller: Operator: Legged By Date Star Boring Lo Hammer Et Definitions S US = Union U = Thir Vo WY = Union C								h Borting Hydroul to molecule Field trans Undershed by view Underland Sheer Strength that Compressing Strength that on high Field 971 H-volum compressing Strength that the Strength Strength that the Strength Strength that Strength Strength that Streng	Boring No.: WIN: Auger 10/00: Sompler: Normer W1./Foll: Core Borrel: Normer 10.00: Note 10.00: Normer 10.00: N	DEPAKTMENT OF TRANSPORTATION	02168610	WIN 021686.10 BRIDGE PLANS				
80	Pen./	Somp1e (44.)	and a	N-M	Neo	Costng	E levat	Graph	I/ISS RECOVER'S Section of Exploration	at 74.8 feet below g	74.8	Testing Results/ ASHTO and Hiffed Close	STATE OF MAINE	DEPAKIMENI C	02	BRIDGE NO. 3332 02
90														SIGNATURE	P.E. NUMBER	DATE
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													MOOSEHORN BRIDGE	MOOSEHORN BROOK	CHARLOTTE WASHINGTON	BORING LOGS
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Appendix A

Boring Logs

Boring No.: BB-CHAR-101 **Maine Department of Transportation** Project: Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook Soil/Rock Exploration Log Location: Charlotte, Maine US CUSTOMARY UNITS WIN: 21686.10 Driller: Elevation (ft.) 4" SSA New England Boring Contractors 83.2 Auger ID/OD: NAVD88 Operator: Mike Porter Datum: Sampler: Standard Split Spoon Logged By: TRM Rig Type: Truck Mounted Mobile #83 Hammer Wt./Fall: 140 lbs./30" Date Start/Finish: NQ-2" 9:45AM 6/7/17_10:17AM 6/9/2017 **Drilling Method:** Cased Wash Boring Core Barrel: **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: Rope & Cathead [Automatic ⊠ Hydraulic 🗆 S_{II} = Peak/Remolded Field Vane Undrained Shear Strength (psf) Definitions R = Rock Core Sample T_v = Pocket Torvane Shear Strength (psf) S_{u(lab)} = Lab Vane Undrained Shear Strength (psf) WC = Water Content, percent D = Split Spoon Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger = Unconfined Compressive Strength (ksf) LL = Liquid Limit MD = Unsuccessful Split Spoon Sample Attempt qp = Unconfined Compressive Colongu., N-uncorrected = Raw Field SPT N-value RC = Roller Cone U = Thin Wall Tube Sample PL = Plastic Limit MU = Unsuccessful Thin Wall Tube Sample Attempt WOH = Weight of 140lb. Hammer Hammer Efficiency Factor = Rig Specific Annual Calibration Value PI = Plasticity Index WOR/C = Weight of Rods or Casing N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency G = Grain Size Analysis V = Field Vane Shear Test. PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt WO1P = Weight of One Person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test Sample Information Laboratory Depth N-uncorrected Testing Ë Log Blows (/6 in.) Results/ S RQD (% Visual Description and Remarks Pen./Rec. Elevation (ft.) Graphic I AASHTO Sample | (ft.) Strength Sample Casing Blows Depth and psf) <mark>ا</mark>60 Jnified Class 6" of pavement SSA 82.7 Brown, dry, dense, SAND, some gravel and fractured rock, little silt. 1D 24/12 13/13/10/9 1.00 - 3.0023 33 Brown, damp, medium dense, GRAVEL, little sand, little silt, (Fill). 9 2D 24/6 3.00 - 5.005/5/4/4 13 14 23 5 28 27 34 23 17 10 Brown, damp, loose, SAND, trace fine gravel, trace silt, (Fill). 3D 24/3 6 10.00 - 12.00 5 7 5/3/2/3 22 17 MD 24/012.00 - 14.00 3/3/1/3 4 6 19 25 Blue-grey, damp, loose, Gravelly SAND, trace silt, trace wood G#414662 63 4D 24/10.8 14.00 - 16.00 4/4/2/3 9 11 fragments, (Fill). A-1-a, SP 15 WC=10.7% 73 67.2 MD 24/0 16.00 - 18.00 15/5/3/3 12 49 22 MV1 18.60 - 19.00 24 Grey, wet, medium dense, Silty SAND, (Glaciomarine Deposit). 83 24/15.6 19.00 - 21.00 12 5D 3/4/4/3 25 20 30 Grey, moist, loose, Silty SAND, (Glaciomarine Deposit). 6D 24/20.4 21.00 - 23.00 4/3/4/7 7 10 35 41 65 78 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 4

BB-CHAR-101 **Boring No.: Maine Department of Transportation Project:** Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook Soil/Rock Exploration Log Location: Charlotte, Maine **US CUSTOMARY UNITS** WIN: 21686.10 Auger ID/OD: Driller: Elevation (ft.) New England Boring Contractors 83.2 4" SSA Operator: Datum: NAVD88 Sampler: Mike Porter Standard Split Spoon Hammer Wt./Fall: TRM 140 lbs./30" Logged By: Rig Type: Truck Mounted Mobile #83 Date Start/Finish: 9:45AM 6/7/17_10:17AM 6/9/2017 NQ-2" **Drilling Method:** Cased Wash Boring Core Barrel: Casing ID/OD: **Boring Location:** HW(4"/4.5") Water Level*: 6.1' bgs @ 7:15 on 6/8/17 4+71.3, 7.3 ft Rt. Hammer Efficiency Factor: 0.869 Hammer Type: Automatic Hvdraulic [Rope & Cathead [S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf) R = Rock Core Sample T_V = Pocket Torvane 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 D = Split Spoon Sample SSA = Solid Stem Auger WC = Water Content, percent MD = Unsuccessful Split Spoon Sample Attempt HSA = Hollow Stem Auger LL = Liquid Limit U = Thin Wall Tube Sample RC = Roller Cone PL = Plastic Limit PI = Plasticity Index MU = Unsuccessful Thin Wall Tube Sample Attempt WOH = Weight of 140 lb. Hammer Hammer Efficiency Factor = Rig Specific Annual Calibration Value V = Field Vane Shear Test. PP = Pocket Penetrometer WOR/C = Weight of Rods or Casing N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency G = Grain Size Analysis MV = Unsuccessful Field Vane Shear Test Attempt N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected WO1P = Weight of One Person C = Consolidation Test Sample Information Laboratory Testing Depth Ë. N-uncorrected Blows (/6 in. Shear Strength (psf) or RQD (%) Results/ Visual Description and Remarks Pen./Rec. Elevation (ft.) AASHTO Sample [(ft.) Graphic Sample Casing Blows Depth and N_{60} Inified Class 25 Grey, moist, medium dense, Silty SAND, little gravel, (Glaciomarine G#414665 7D 24/9.6 25.00 - 27.00 3/3/5/5 12 70 A-4, SM 57.2 WC=15.9% 75 113 126 Cobbles.2 110 30 Grey, damp, very stiff, SILT, some sand, little clay, little gravel and 24/2.4 30.00 - 32.00 5/5/6/8 11 16 rock fragments, (Glacial Till). 98 144 Cobbles.2 153 227 35 Grey, damp, hard, SILT, some sand, little clay, little gravel and rock 9D 24/19.2 | 35.00 - 37.00 6/9/43/50 523 75 102 fragments, (Glacial Till). 146 Roller Coned ahead from 35.0-40.0 ft bgs. 276 311 471 40 G#414666 Grey, damp, very dense, GRAVEL, little sand, little silt, trace clay, 1373 10D 24/16.8 40.00 - 42.00 25/45/92/63 198 36 A-1-a, GM (Glacial Till) WC=4.2% Cobble at 40.0'. 50 37 91 114 45 Grey, damp, hard, SILT, some gravel, some sand, little clay, (Glacial 15.6/8.4 45.00 - 46.30 11D 20/31/(50/3.6") 81 117 72 Till). 159 Cobbles.2 130 Cobble from 48.7' to 49.1'. 102 243

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.

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N	Maine Department of Transportat					On Project: Moosehorn Bridge #3332 carries Station					Boring No.: BB-CH		AR-101
		_	Soil/Rock Exp				Location			osehorn Brook Jaine			
		<u>L</u>	JS CUSTOM/	ARY UNITS			Location	ii. Che			WIN:	2168	36.10
Drille	ır.		New England	Boring Contractors	FI	evation	(ft)	83.2	,		Auger ID/OD:	4" SSA	
	ator:		Mike Porter	Borning Contractors	_	tum:	(16.)		VD88		Sampler:	Standard Split	Snoon
	ed By:		TRM		+	g Type	:			ted Mobile #83	Hammer Wt./Fall:	<u> Броон</u>	
	Start/F			7_10:17AM 6/9/2017	_		lethod:		ed Wash		Core Barrel:	140 lbs./30" NQ-2"	
	ng Loca		4+71.3, 7.3 ft		$\overline{}$	sing IE			(4"/4.5"		Water Level*:	6.1' bgs @ 7:1:	5 on 6/8/17
			actor: 0.869		_	mmer		Autom			Rope & Cathead □		
Definit	ions:			R = Rock C			-	S _u =	Peak/Re	molded Field Vane Undrained She	ear Strength (psf) T	v = Pocket Torvane She	
MD =		sful Split Spc	on Sample Atten		ow Sten			q _p `=	Unconfin	Vane Undrained Shear Strength (p ed Compressive Strength (ksf)	L	VC = Water Content, per L = Liquid Limit	æni
MU =	Unsucces		Il Tube Sample A		eight of 1			Ham	mer Effici	= Raw Field SPT N-value ency Factor = Rig Specific Annual	Calibration Value F	PL = Plastic Limit PI = Plasticity Index	
V = Fid MV =	eld Vane S Unsucces:	Shear Test, sful Field Var	PP = Pocket Pe ne Shear Test Att	netrometer WOR/C = W tempt WO1P = W						uncorrected Corrected for Hamme er Efficiency Factor/60%)*N-uncor		G = Grain Size Analysis C = Consolidation Test	
				Sample Information					1 1				Laboratory
		<u>:</u>	g l	<u> </u>	ted				_D				Testing
ft.)	Sample No.	Pen./Rec. (in.)	Depth	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected				Graphic Log	Visual De	scription and Remar	ks	Results/ AASHTO
Depth (ft.)	nple	, 'R	Sample I (ft.)	ws (ear engt engt f)	DOUT		Casing Blows	Elevation (ft.)	aphic				and
	Sal	Pe	Sal (#.)	8 ch 25 ch 2	ž	N ₆₀	8 8 8	E E	Ğ				Unified Class
50	12D	24/20.4	50.00 - 52.00	29/41/33/42	74	107	71			Grey, damp, very dense, SA (Glacial Till).	AND, some silt, little g	ravel, little clay,	
							57						
							63						
ı							176						
							169						
- 55 -	13D	20.4/14.4	55.00 - 56.70	10/35/51/(50/2")	86	125	171			Grey, damp, hard, SILT, litt pink granite, (Glacial Till).	tle gravel, little sand, l	little clay; 1" piece of	
							275					57.0	
							130	26.2				57.0	
							558			E CDANITE	(W. 4. 1D.1.1)	.	
- 60 -	14D	2.4/1.2	59.00 - 59.20	50/2.4"			480			Fractured, pink, GRANITE	, (Weathered Bedrock).	
							RC						
- 65 -				/						Grey, SILT, some sand, son	ne pink, fractured rock	k fragments,	
	15D	6/4.8	65.00 - 65.50	117/6"			+	166		(Weathered Bedrock).		66.4	
							NQ	16.8		Top of Bedrock at Elev. 16.	8 ft.	— — — — —66.4	
									THE STATE OF	R1: Bedrock: Light pink, m	edium to coarse-grain	ed BIOTITE-	
	R1	58/17	68.00 - 72.83	RQD = 6%						HORNBLENDE GRANITI	E, moderately hard, me	oderately weathered,	
- 70 -									響点	spacing, planar to curved di surfaces.			
, 0									***	[Charlotte Pluton].			
									15 / S	Rock Quality = Very Poor. R1 Core Times: (min:sec)			
									183	68.0-69.0 ft (1:05)			
	R2	60/54	72.80 - 77.80	RQD = 68%					P. C.	69.0-70.0 ft (1:34) 70.0-71.0 ft (1:11)			
								1	 空台	71.0-72.0 ft (0:52)			
									(流)	72.0-72.8 ft (1:25) 29% Recovery			
75								<u> </u>	学孩	2570 Recovery			
	arks:												
1.3	00 lb har	nmer for 4'	' casing. Auto	hammer starting at 10' fo	or 4" c	asing.							

- 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 N₆₀ 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.

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N	Maine Department of Transportation				Project: Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook					Boring No.: BB-CH		AR-101	
			Soil/Rock Exp US CUSTOM				Locatio				WIN:	2168	36.10
Drille	ır.		New England	Boring Contractors	FI	evatior	(ft)	83.2			Auger ID/OD:	4" SSA	
Oper			Mike Porter	Boring Contractors		atum:	. (11.)		/D88		Sampler:	Standard Split	Spoon
	ed By:		TRM			g Type	:			nted Mobile #83	Hammer Wt./Fall:	140 lbs./30"	Брооп
	Start/F	inish:		17 10:17AM 6/9/2017	-		lethod:			Boring	Core Barrel:	NQ-2"	
Borin	ng Loca	ition:	4+71.3, 7.3 ft	_	-	asing II			4"/4.5"		Water Level*:	6.1' bgs @ 7:1:	5 on 6/8/17
Ham	mer Eff	iciency F	actor: 0.869		Ha	ammer	Type:	Automa	ıtic 🛛	Hydraulic □	Rope & Cathead □		
MD = l U = Th MU = l V = Fie	olit Spoon Jnsuccess in Wall Tu Jnsuccess old Vane S	sful Split Sp ube Sample sful Thin Wa Shear Test,	all Tube Sample A PP = Pocket Pe ane Shear Test A	SSA = S mpt	COre Sa olid Stem ollow Ster ller Cone Veight of Weight of Weight of	Auger m Auger 140 lb. H of Rods o	r Casing	S _{u(la} q _p = N-un Hami N ₆₀ :	b) = Lab Unconfin corrected ner Effic = SPT N-	molded Field Vane Undrained She Vane Undrained Shear Strength (ed Compressive Strength (ksf) I = Raw Field SPT N-value ency Factor = Rig Specific Annual uncorrected Corrected for Hamme er Efficiency Factor/60%)*N-uncor	psf)	= Pocket Torvane She = Water Content, pere = Liquid Limit = Plastic Limit = Plasticity Index Grain Size Analysis Consolidation Test	
H		Ι -		Sample Information		1							Laboratory
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		scription and Remarks		Testing Results/ AASHTO and Unified Class.
75									学者	R2: Bedrock: Light pink to BIOTITE-HORNBLENDE			
80 -								5.4	1. 1. 1. 1. 1. 1. 1. 1. 1. 1. 1. 1. 1. 1	weathered, fractures are mo closely spaced (2" to 10"), very rough surfaces with m [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	oderately dipping (35 to planar to stepped discort	55 degrees), tinuities, rough to	
· 85 -													
. 05													
95 -													
Rem	arks:	1	1	1	1		1			<u> </u>			
		nmer for 4	l" casing Auto	hammer starting at 10	for 4" c	asino							
2. P	resence o	of cobbles	is assumed bas	sed on drilling behavio	r.	_	ple. The	resulting	g N ₆₀ v	alue may overestimate the soi	il density.		

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

than those present at the time measurements were made.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other

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BB-CHAR-102 **Boring No.: Maine Department of Transportation** Project: Moosehorn Bridge #3332 carries Station Road over Moosehorn Brook Soil/Rock Exploration Log Location: Charlotte, Maine US CUSTOMARY UNITS WIN: 21686.10 Driller: 4" SSA New England Boring Contractors Elevation (ft.) 82.8 Auger ID/OD: NAVD88 Operator: Tom Schaefer Datum: Sampler: Standard Split Spoon Rig Type: Logged By: TRM 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:** Rope & Cathead ⊠ Automatic Hydraulic 🗆 S_{II} = Peak/Remolded Field Vane Undrained Shear Strength (psf) Definitions R = Rock Core Sample T_v = Pocket Torvane Shear Strength (psf) S_{u(lab)} = Lab Vane Undrained Shear Strength (psf) WC = Water Content, percent D = Split Spoon Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger = Unconfined Compressive Strength (ksf) LL = Liquid Limit MD = Unsuccessful Split Spoon Sample Attempt qp = Unconfined Compressive Strength N-uncorrected = Raw Field SPT N-value RC = Roller Cone U = Thin Wall Tube Sample PL = Plastic Limit MU = Unsuccessful Thin Wall Tube Sample Attempt WOH = Weight of 140lb. Hammer Hammer Efficiency Factor = Rig Specific Annual Calibration Value PI = Plasticity Index WOR/C = Weight of Rods or Casing N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency G = Grain Size Analysis V = Field Vane Shear Test. PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt WO1P = Weight of One Person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test Sample Information Laboratory Depth N-uncorrected Testing Ë Log 3lows (/6 in.) Results/ S RQD (% Visual Description and Remarks Pen./Rec. Elevation (ft.) Strength (psf) Graphic I AASHTO Sample | (ft.) Sample Casing Blows Depth and <mark>ا</mark>60 Jnified Class 6" of payement SSA 82.3 Brown, dry, medium dense, SAND, little fine gravel, trace silt, (Fill). 1D 24/13.2 14 1.00 - 3.006/8/6/4 14 G#414663 Brown, dry, loose, medium to coarse SAND, some gravel, trace silt, 5 5 2D 24/12 3.00 - 5.005/3/2/2 1-1-a, SW-SN (Fill) WC=4.0% 5 Brown, dry, loose, medium to coarse SAND, some gravel, trace silt, 3D 24/9.6 5.00 - 7.00 1/2/3/25 5 trace white quartz fragments, (Fill). Brown, dry, medium stiff, Gravelly SILT, little sand, (Fill). 4D 24/8.4 7.00 - 9.001/4/2/3 6 6 10 16 15 14 14 13 15 67.8 5D: Layered, brown, moist, loose, fine to medium SAND with 5D 24/12 15.00 - 17.00 2/2/4/3 6 6 2 organics, trace silt, odor; grey, moist, loose, fine to medium SAND, trace silt; grey, moist, medium stiff, Clayey SILT, (Glaciomarine 3 Deposit). G#414668 6D: Layered, grey, moist, medium stiff, Silty CLAY, trace fine 6D 24/21.6 17.00 - 19.00 2/2/4/3 6 11 6 WC=25.0% gravel, PP: 0.75, 1.0; TV: 0.325, 0.300; grey, damp, loose, fine LL=31% SAND, little clay, trace silt, (Glaciomarine Deposit). 20 PL=18% PI=13% Grey, damp, loose, SAND, some silt, (Glaciomarine Deposit). 7 7D 24/15.6 19.00 - 21.00 7 3/3/4/4 28 20 14 Grey, moist, stiff, Sandy SILT, trace clay, (Glaciomarine Deposit). G#414664 8D 24/13.2 21.00 - 23.00 2/5/6/8 11 11 24 A-4, ML WC=24.9% 31 Grey, moist, loose, SAND, some silt, (Glaciomarine Deposit). 9D 24/12 23.00 - 25.00 4/4/4/5 8 8 24 35 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.

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Maine Department of Transportation						n	Project:		Bridge #3332 carries Station	Boring No.: BB-CH		AR-102
			Soil/Rock Exp	-			Location	Road over 1: Charlotte	Ioosehorn Brook Maine		• • •	25.40
		<u>[</u>	JS CUSTOM.	ARY UNITS						WIN:	2168	36.10
Drille	r:		New England	Boring Contractors	Ele	evation	ı (ft.)	82.8		Auger ID/OD:	4" SSA	
Oper	ator:		Tom Schaefer		Da	tum:	. ,	NAVD88		Sampler:	Standard Split	Spoon
Logg	ed By:		TRM		Ri	g Type	:	Mobile B	3	Hammer Wt./Fall: 140 lbs./30"		
Date	Start/Fi	nish:	1:45pm 6/12/	17_12:45pm 6/12/17	Dr	illing N	/lethod:	SSA & D	W	Core Barrel: NQ-2"		
Borir	g Loca	tion:	5+17.9, 5.3 ft	Lt.	Ca	sing II	D/OD:	HW(4"/4	")	Water Level*: See Remarks		
Hamı Definiti		ciency F	actor: 0.6	R = Rock		ammer	Type:	Automatic S. = Peak	Hydraulic □ lemolded Field Vane Undrained She	Rope & Cathead ear Strength (nsf) T. = 1	Pocket Torvane She	ar Strength (nsf)
D = Sp MD = U U = Th MU = U V = Fie	lit Spoon Insuccess In Wall Tu Insuccess Id Vane S	sful Split Spo lbe Sample sful Thin Wal Shear Test,	oon Sample Atter II Tube Sample A PP = Pocket Pe ne Shear Test At	SSA = Sol MSA = Ho RC = Rolle Attempt WOH = W enetrometer WOR/C =	lid Stem llow Ster er Cone eight of Weight o	Auger n Auger 140 lb. H of Rods o	r Casing	S _{u(lab)} = L q _p = Unco N-uncorred Hammer E N ₆₀ = SPT	b Vane Undrained Shear Strength (ined Compressive Strength (ksf) sd = Raw Field SPT N-value ciency Factor = Rig Specific Annual N-uncorrected Corrected for Hamme mer Efficiency Factor/60%)*N-uncor	psf) WC = LL = I PL = Calibration Value PI = F er Efficiency G = G	Water Content, perd Liquid Limit Plastic Limit Plasticity Index Grain Size Analysis Consolidation Test	
IVIV – C	msuccess	iui rieiu vai		Sample Information	veignt of	One Per	SOII	N60 - (па	mer Emiciency Pactor/60%) N-uncol	Tected C - C	onsolidation rest	
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 I od	Visual De	scription 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, fir Deposit).	ne SAND, trace clay, (Gla	aciomarine	
ŀ									Deposity.			
ŀ												
Ī												
· 30 🕇	11D	24/12.2	30.00 - 32.00	5/9/9/11	18	18	15		Grey, moist, medium dense	, Silty, fine SAND, (Glaci	omarine	
ŀ	1110	24/13.2	30.00 - 32.00	3/3/3/11	10	10			Deposit).			
-							19					
							29					
							42					
,							49					
35	12D	24/14.4	35.00 - 37.00	5/5/24/26	29	29			Grey, moist, medium dense (Glaciomarine Deposit).	, Silty, fine SAND, trace	clay,	
İ							21					
İ							26	45.8			37.0	
ŀ							34					
ŀ							57					
40	13D	24/13.2	40.00 - 42.00	15/19/19/23	382	38	54		Grey, moist, hard, SILT, so Till).	me sand, little clay, little	gravel, (Glacial	
Ì							64		Cobbles from 40' to 45'.2			
Ì							120					
ŀ							130					
ŀ							89					
45	14D	24/13.2	45.00 - 47.00	14/19/23/28	42	42	OPEN		Dark grey, moist, hard, SIL (Glacial Till).	T, some sand, little clay, l	ittle gravel,	G#414667 A-4, CL
ŀ									(3			WC=11.7%
ł						1						
50												
Rema	ırks:	i				1		17.27	•			

- 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 2 of 3

Maine Department of Transporta					tion	Project			ridge #3332 carries Station	Boring No.: BB-CHA		AR-102
			Soil/Rock Exp US CUSTOM			Location			oosehorn Brook Maine	WIN:	216	86.10
					_					*****		30.10
Drille	er:		New England	Boring Contractors	Elevatio	n (ft.)	82.	3		Auger ID/OD:	4" SSA	
Oper	ator:		Tom Schaefer	:	Datum:		NA	VD88		Sampler:	Standard Split	Spoon
Logg	jed By:		TRM		Rig Type	ə :	Mo	bile B53	}	Hammer Wt./Fall:	140 lbs./30"	
Date	Start/Fi	nish:	1:45pm 6/12/	17_12:45pm 6/12/17	Drilling		SSA	& D+V	W	Core Barrel:	NQ-2"	
Boriı	ng Loca	tion:	5+17.9, 5.3 ft	Lt.	Casing I	D/OD:	HW	(4"/4.5	')	Water Level*:	See Remarks	
		ciency F	actor: 0.6	R = Rock Co	Hammer	Type:	Auton			Rope & Cathead ⊠		
MD = 1 U = Th MU = 1 V = Fie	olit Spoon S Unsuccess in Wall Tu Unsuccess eld Vane S	sful Split Sp be Sample sful Thin Wa Shear Test,	oon Sample Atter	SSA = Solid MPA = Hollo RC = Roller WOH = Wei enetrometer WOR/C = W	Stem Auger w Stem Auger Cone ght of 140 lb. F eight of Rods	lammer or Casing	S _{u(I} q _p = N-u Han N ₆₀	ab) = Lab Unconfir ncorrected mer Effic = SPT N	emolded Field Vane Undrained She Vane Undrained Shear Strength (hed Compressive Strength (ksf) d = Raw Field SPT N-value iency Factor = Rig Specific Annual -uncorrected Corrected for Hamme	psf)	Pocket Torvane She = Water Content, per Liquid Limit Plastic Limit Plasticity Index Grain Size Analysis	
MV =	Jnsuccess	itul Fleid Va	ne Shear Test At	Sample Information	eight of One Pe	erson	N60	= (Hamn	ner Efficiency Factor/60%)*N-uncor	rected C = C	Consolidation Test	1
Depth (ft.)	Sample No.	Pen /Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	Casing Blows	Elevation (ft.)	Graphic Log	Visual De	scription and Remarks		Laboratory Testing Results/ AASHTO and Unified Class.
50	15D	6/4.8	50.00 - 50.50	(50/6")					Dark grey, moist, very dens (Glacial Till).	se, Silty SAND, little clay	, little gravel,	
- 55 -	16D	11/10	55.00 - 55.92	44/(50/5")					Dark grey, moist, hard, SIL rock fragments, (Glacial Ti		little gravel and	
											50.0	
- 60 -	MD	0/0	59.90 - 59.90	(50/0")		$\mathbb{H}^{\mathbb{V}}$	23.8		Pale red sand/fractured rocl No penetration with 2" spli			
							_		Bottom of Exploration	n at 60.0 feet below grou		
- 65 -							_					
- 70 -							_					
_ 75 _	arke:											
Rem		de. 7 11 b -	e at 17:05 on 0	6/12/17: 6 7! has at 7:10	16/12/17· N-	water in	hole of	or cooin-	removed			
				6/12/17; 6.7' bgs at 7:10 (ed on drilling behavior.	ло/13/1/; No	water in	noie afte	a casing	removed.			

- 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.

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Maine Department of Train	nsportation	Project:		ridge #3332 carries Station	Boring No.: BB-CH		AR-201
Soil/Rock Exploration Log US CUSTOMARY UNITS		Location	Road over Mon: Charlotte, N	oosehorn Brook Aaine	WIN:	2168	36.10
Driller: Seaboard Drilling LLC	Elevati	ion (ft.)	83.2		Auger ID/OD:	5" Solid Stem	
Operator: Hanscom/Wall	Datum		NAVD88		Sampler:	Standard Split	Spoon
Logged By: B. Wilder	Rig Ty	pe:	Diedrich D-	-50	Hammer Wt./Fall:	140#/30"	•
Date Start/Finish: 3/12-14/2024	Drilling	g Method:	Cased Wasl	n 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	Hamme	er 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 Sample Inf	R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Aug RC = Roller Cone WOH = Weight of 140lb. WOR/C = Weight of Rod WO1P = Weight of One I	ger . Hammer ds or Casing	S _{u(lab)} = Lab q _p = Unconfir N-uncorrected Hammer Effic N ₆₀ = SPT N	emolded Field Vane Undrained She Vane Undrained Shear Strength (ped Compressive Strength (ksf) d = Raw Field SPT N-value iency Factor = Rig Specific Annual -uncorrected Corrected for Hamme rer Efficiency Factor/60%)*N-uncor	osf) WC LL = PL Calibration Value PI = or Efficiency G =	= Pocket Torvane She: = Water Content, perd = Liquid Limit = Plastic Limit = Plasticity Index Grain Size Analysis Consolidation Test	
	-						Laboratory Testing
Sample No. Sample Denth (ft.) Sample Depth (ft.) Blows (/6 in.) Shear	(psf) or RQD (%)	N60 Casing Blows	Elevation (ft.) Graphic Log		scription and Remarks	3	Results/ AASHTO and Unified Class
		SSA	83.0	2" HMA.		0.2-	
- 5				Brown, moist, loose, SANE), some gravel, little silt	. (Fill).	
1D 24/11 5.00 - 7.00 3/3/2	2/2 5	9			, ,		
MD 24/0 10.00 - 12.00 5/2/	3/2 5	9 16		Loose. No sample recovered	d. Soil similar to 1D on a	auger flights.	
		13					
		30	l				
		23	70.2			13.0-	
		28					
D 24/18 15.00 - 17.00 2/2/3	3/5 5	9 29		Grey, wet, loose, Silty, fine	SAND, little clay, (Glac	ciomarine Deposit).	
		34					
		42					
		46					
		55					
3D 24/20 20.00 - 22.00 3/3/4	4/4 7 1	12 51		Grey, wet, medium dense, S	Silty, fine SAND, (Glaci	omarine Deposit).	G#380946 A-4, SM WC=21.7%
		43					
		48					
		48					
25		51					
Remarks: 1) Auto Hammer #367 2) 20.0 ft of NW(3") broken casing abandoned in last stratification lines represent approximate boundaries between			5.1 ft bgs (El. 38	3.1).	Page 1 of 3		

Boring No.: BB-CHAR-201

*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.

I	Maine Department of Transports					1	Project	Moos	ehorn E	ridge #3332 carries Station	Boring No.:	BB-CH	AR-201
		-	Soil/Rock Exp	-			Locatio			oosehorn Brook			
		<u>l</u>	JS CUSTOM	ARY UNITS			Locatio	II. Cha	none, r	rame	WIN:	2168	36.10
Drille	· r·		Seaboard Dril	ling I I C	Flox	/ation	/ft \	83.2			Auger ID/OD:	5" Solid Stem	
	ator:		Hanscom/Wa		Dati		i (it.)		/D88		Sampler:	Standard Split	Snoon
			B. Wilder	11	+	Туре			rich D	50	Hammer Wt./Fall:	140#/30"	эрооп
	ed By: Start/F		3/12-14/2024		+					n Boring	Core Barrel:	N/A	
	ng Loca		4+57.4, 6.0 ft	T+	$\overline{}$		lethod: D/OD:				Water Level*:	9.0 ft bgs.	
				Lt.	+		Type:), HW(4"/4.5")		9.0 ft bgs.	
Definit		iciency F	actor: 1.066	R = Rock C			Type.	Autom:		Hydraulic □ molded Field Vane Undrained She	Rope & Cathead ar Strength (psf) T	, = Pocket Torvane She	ar Strength (psf)
D = S	olit Spoon		oon Sample Atter	SSA = Solid	l Stem Αι	ıger		Siiria	ام = Lab	Vane Undrained Shear Strength (page Compressive Strength (ksf)	osf) W	C = Water Content, per L = Liquid Limit	
U = Th	in Wall Tu	ube Sample		RC = Roller	Cone	-		N-un	correcte	d = Raw Field SPT N-value	Pl	_ = Plastic Limit	
V = Fi	eld Vane S	Shear Test,	II Tube Sample <i>F</i> PP = Pocket Pe	enetrometer WOR/C = V	eight of l	Rods o	r Casing	N ₆₀	= SPT N	iency Factor = Rig Specific Annual -uncorrected Corrected for Hamme	r Efficiency G	= Plasticity Index = Grain Size Analysis	
MV =	Jnsucces:	sful Field Va	ne Shear Test At	tempt WO1P = Wo Sample Information	eight of C	ne Per	son	N ₆₀	= (Hamn I	ner Efficiency Factor/60%)*N-uncor	rected C	= Consolidation Test	
				·	ъ				1				Laboratory
_	Ö	Pen /Rec. (in.)	Depth	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected				Log	\"			Testing Results/
H.	<u>e</u>	Sec	<u>е</u>	gth ('6	Sorre		D	tion]: 	Visual De	scription and Remark	(S	AASHTO
Depth (ft.)	Sample No.	en./l	Sample I (ft.)	ows near fren rren srf) RC	ŭ	N ₆₀	Casing Blows	Elevation (ft.)	Graphic I				and Unified Class
25	ιχ	<u>a</u>	ığ €	<u> </u>	Ż	z	O M	⊞€	()	Similar to 2D			
23	4D	24/17	25.00 - 27.00	4/4/4/6	8	14	52			Similar to 3D.			
							96	1					
								ł					
							60	55.2				28.0-	
							72	33.2				28.0	
							0.5	1					
- 30 -							87				CIL CAND ENL 1		
	5D	24/16	30.00 - 32.00	5/5/7/7	12	21	81			Grey, wet, medium dense, S (Glacial Till).	silty SAND, little clay	, trace gravei,	
							163	1		,			
					\rightarrow		103	-					
							146						
							170						
								1					
- 35 -							212			Grey, wet, dense, Silty SAN	ID little amovel trace a	slav. (Clasial Till)	
	6D	24/7	35.00 - 37.00	9/10/17/20	27	48	OPEN			Gley, wei, delise, silty SAF	D, fittle graver, trace c	nay, (Giaciai Tili).	
							HOLE	1					
								-					
								1					
- 40 -										G 1 1 671	CANED	(CL : 1 TH)	
	7D	9.6/9.6	40.00 - 40.80	21/58(3.6")						Grey, wet, very dense, Silty		(Glacial Till).	
								1		Cobble from 40.8-41.3 ft bg	gs.		
								4					
										Cobble from 42.5-43.2 ft bg	gs.		
								1					
								-					
- 45 -													
43	8D	12/6	45.00 - 46.00	33/55						Similar to 7D.			
					+		++	1					
					+			1					
50													
	arks:	1					-		V # 7.				
1) A	uto Han	nmer #367											
			oken casing al	pandoned in hole from 65	.1 ft bgs	s (El. 1	(8.1) to 4	5.1 ft bg	s (El. 3	3.1).			

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.

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I	Main	e Depa	artment	of Transport	ation		Projec				ridge #3332 carries Station	Boring No.:	BB-CH	AR-201		
			Soil/Rock Expl				Locat				oosehorn Brook Iaine		21.6	26.10		
		<u>.</u>	US CUSTOMA	ARY UNITS								WIN:	2168	86.10		
Drille	er:		Seaboard Drill	ling LLC	Elev	ation	(ft.)	8	83.2			Auger ID/OD:	5" Solid Stem			
Ope	rator:		Hanscom/Wal	1	Datu	ım:]	NAV	D88		Sampler:	Standard Split	Spoon		
Logg	ged By:		B. Wilder		Rig 1	Type	:]	Died	rich D-	50	Hammer Wt./Fall:	140#/30"			
Date	Start/Fi	nish:	3/12-14/2024		Drilli	ing N	lethod	: (Case	d Wash	n Boring	Core Barrel:	N/A			
Bori	ng Loca	tion:	4+57.4, 6.0 ft	Lt.	Casi	ing IC	O/OD:]	NW(3"/3.5"), HW(4"/4.5")	Water Level*:	9.0 ft bgs.			
Definit D = S _I MD = U = TI MU = V = Fi	tions: olit Spoon : Unsuccess nin Wall Tu Unsuccess eld Vane S	Sample sful Split Spo lbe Sample sful Thin Wa Shear Test,	con Sample Attenual Tube Sample At PP = Pocket Per ne Shear Test Att	RC = Roller WOH = We netrometer	core Sampled Stem Aug ow Stem A r Cone right of 140 Veight of R	le ger Auger O Ib. Ha Rods or	Casing	2 0 1 1	S _u = F S _{u(lak} q _p = l N-unc Hamn N ₆₀ =	o) = Lab Unconfin corrected ner Effic : SPT N-	Hydraulic □ molded Field Vane Undrained She Vane Undrained Shear Strength (ed Compressive Strength (ksf) = Raw Field SPT N-value ency Factor = Rig Specific Annual uncorrected Corrected for Hamme er Efficiency Factor/60%)*N-uncor	osf) W LI P Calibration Value P or Efficiency G	v = Pocket Torvane She // = Water Content, per L = Liquid Limit L = Plastic Limit l = Plasticity Index i = Grain Size Analysis = Consolidation Test			
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (pst) or RQD (%)	N-uncorrected	N ₆₀	Casing	Elevation	(ft.)	Graphic Log		scription and Remarl	ks	Laboratory Testing Results/ AASHTO and Unified Class		
50											Cobble from 49.9-50.3 ft by Set in NW Casing at 50.0 ft					
	9D	24/17	51.00 - 53.00	19/28/29/40	57	101					Grey, wet, very dense, SAN	ID, some silt, trace gra	vel, (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							
- 60 -	11D	6/5	60.00 - 60.50	90			1				Pink to brown, wet, very de (Weathered Bedrock).	nse, SAND, some gra	vel, trace silt,			
- 65 -	12D	1.2/1.2	65.00 - 65.10	50(1.2")			V	<u> </u>	18.1		Pink to grey, wet, very dens (Weathered Bedrock).	se, GRAVEL, some sai	nd, trace silt			
											Bottom of Exploration Broke NW(3") Casing, mov broken casing abandoned in bgs (El. 38.1) .	ed to BB-CHAR-2014	ound surface. A. 20.0 ft of NW(3")			
- 70 -																
75							<u>L</u>									
1) A		nmer #367 NW(3") bi	roken casing ab	andoned in hole from 65	5.1 ft bgs	(El. 1	8.1) to	45.1 f	t bgs	(El. 38	3.1).					

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

than those present at the time measurements were made.

Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other

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Boring No.: BB-CHAR-201

]	Main	_		of Transport	tatio	n	Proje				ridge #3332 carries Station	Boring No.:	BB-CH	AR-201A
			Soil/Rock Exp	•			Loca		: Char		oosehorn Brook Maine		• • •	26.40
			US CUSTOM	IARY UNITS								WIN:	2168	36.10
Drill	ler:		Seaboard Dri	lling LLC	Fle	vation	(ft)		83.2			Auger ID/OD:	5" Solid Stem	
-	rator:		Kevin/Jason	ming EEC		tum:	(11.)		NAV	/D88		Sampler:	N/A	
<u> </u>	ged By:		B. Wilder			Type:				rich D	50	Hammer Wt./Fal		
-	Start/Fi	inish:	3/13/2024-3/2	14/2024		Iling M		d:			n Boring	Core Barrel:	NQ-2"	
-	ing Loca		4+55.9, 6.1 ft		-	sing ID				3" & I	-	Water Level*:	9.5 ft bgs.	
-			actor: 1.066		_	mmer		. /	Automa			Rope & Cathead	3.0 20 080	
Defin	itions:			R = Rock	Core San	nple	71		S ₁₁ = 1	Peak/R	emolded Field Vane Undrained She	ear Strength (psf)	T _V = Pocket Torvane She	
MD =		sful Split Sp	oon Sample Atte		llow Stem				S _{u(lal}	b) = Lar Unconfi	Vane Undrained Shear Strength (led Compressive Strength (ksf)	pst)	WC = Water Content, per LL = Liquid Limit	cent
	hin Wall Tu Unsuccess		all Tube Sample /	RC = Rolle Attempt WOH = W		40lb. Hai	mmer				d = Raw Field SPT N-value iency Factor = Rig Specific Annual	Calibration Value	PL = Plastic Limit PI = Plasticity Index	
			PP = Pocket Pe ane Shear Test A		Weight of	f Rods or One Pers	Casing	9	N ₆₀ =	SPT N	-uncorrected Corrected for Hamme ner Efficiency Factor/60%)*N-uncor	r Efficiency rected	G = Grain Size Analysis C = Consolidation Test	
				Sample Information							,			Laboratory
		. u	l ∉	_	fed									Testing
l ⊋	Š	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected				<u>_</u>	Graphic Log	Visual De	scription and Rem	arks	Results/ AASHTO
Depth (ft.)	Sample No.	%	l be	ws (arengtlengtl	20		ing	ς l	/atic	phic				and
Dep	San	Pen	San (ff.)	Blov Stre (psf	Z-Z	N 60	Casing	윤	Elevation (ft.)	Gra				Unified Class.
0							DRI	\neg			Drove HW Casing to 35.0 f	t bgs.		
							H	v I			See BB-CHAR-201 for soil	descriptions.		
								\dashv						
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- 5														
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- 10														
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							\Box	\neg						
25 Ren	l narks:													
		mar #267												
	Auto Ham 20.0 ft of 1			bandoned in hole from 6	54.8 ft bg	gs (El. 1	9.2) to	44.	8 ft bgs	E1. 3	9.2).			
L														
Strati	fication line	s represent	approximate bou	undaries between soil types	; transitio	ns may b	e gradı	ıal.				Page 1 of 4		

Boring No.: BB-CHAR-201A

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.

]	Main		artment			atior	ո	Proj	ect:			Bridge #3332 carries Station	Boring No.	BB-CH	AR-201A
			Soil/Rock Exp US CUSTOM	_				Loca	atio	Road on: Cha		oosehorn Brook Maine	WIN:	216	86.10
			00 0001011	WALL CIVITO									WIN.	210	80.10
Drill			Seaboard Dri	lling LLC		+	vation	(ft.)		83.2			Auger ID/OD:	5" Solid Stem	
⊢÷	rator:		Kevin/Jason			_	um:				/D88		Sampler:	N/A	
-	ged By:		B. Wilder	1.1/202.1		+	Type:		_		lrich D		Hammer Wt./Fa		
-	Start/F		3/13/2024-3/			_	ling M					h Boring	Core Barrel: Water Level*:	NQ-2"	
_	ing Loca		4+55.9, 6.1 ft actor: 1.066			_	sing ID mmer 1			Automa		IW-4" Hydraulic □	Rope & Cathead	9.5 ft bgs.	
Defin	itions:		actor. 1.000		R = Rock C	ore Sam	ple	урс	•	S _u =	Peak/R	emolded Field Vane Undrained She	ear Strength (psf)	T _V = Pocket Torvane She	
	Split Spoon Unsucces		oon Sample Atte	mpt	SSA = Solid HSA = Holld	w Stem				S _{u(la}	_{b)} = Lal Unconfi	o Vane Undrained Shear Strength (ned Compressive Strength (ksf) d = Raw Field SPT N-value	psf)	WC = Water Content, per LL = Liquid Limit	cent
		ube Sample sful Thin W	all Tube Sample /	Attempt	RC = Roller WOH = We		40 lb. Hai	mmer		N-un Hami	correcte ner Effi	d = Raw Field SPT N-value ciency Factor = Rig Specific Annua	I Calibration Value	PL = Plastic Limit PI = Plasticity Index	
V = F MV =	ield Vane S Unsuccess	Shear Test, sful Field Va	PP = Pocket Pe ane Shear Test A	enetrometer ttempt	WOR/C = V WO1P = W	Veight of eight of C	Rods or One Pers	Casin on	ıg	N ₆₀ :	= SPT N = (Hami	-uncorrected Corrected for Hamme ner Efficiency Factor/60%)*N-unco	er Efficiency rrected	G = Grain Size Analysis C = Consolidation Test	
			1	Sample Info	ormation										Laboratory
	_ ا	Ë.	Sample Depth (ft.)	<u>ج</u>	<u></u>	ted									Testing
(# (#	Sample No.	Pen./Rec. (in.)	De	Blows (/6 in.) Shear Strength	%) (N-uncorrected				LO O	Graphic Log	Visual De	scription and Rem	arks	Results/ AASHTO
Depth (ft.)	l mple	n./R	l mg/m	ws ear	RQI	Jour	0	Casing	S/M	Elevation (ft.)	aphi				and
	Sa	Pe	Sa (#	용양광	g p	ž	N ₆₀	ပ္ပ	Bic	Ele (A.	ō				Unified Class.
25															
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l															
- 30	†														
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	-							-	Н						
- 35	<u> </u>							1							
								OP							
								НО	LE						
40	\vdash														
- 45															
1 43															
	<u> </u>														
50 Ren	l narks:			<u> </u>								I			<u> </u>
		nmer #367	,												
			roken casing a	bandoned in h	ole from 64	.8 ft bg	s (El. 19	9.2) t	o 44	.8 ft bgs	s (El. 3	9.2).			
Strati	fication line	es represen	t approximate bou	undaries betwee	n soil types; t	ransition	s may be	grad	ual.				Page 2 of	4	

Boring No.: BB-CHAR-201A

*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.

N	Iain	e Dep	artment	of Transport	ation	Proje	ect:			ridge #3332 carries Station	Boring No.:	BB-CH	4R-201A
			Soil/Rock Expl US CUSTOM			Loca	tion		over Mo rlotte, M	oosehorn Brook 1aine	WIN:	2168	86.10
Drille	r:		Seaboard Drill	ling LLC	Elevation	n (ft.)		83.2			Auger ID/OD:	5" Solid Stem	
Oper			Kevin/Jason	8	Datum:	(14.)			/D88		Sampler:	N/A	
<u> </u>	ed By:		B. Wilder		Rig Typ	φ.			rich D-	50	Hammer Wt./Fal		
	Start/Fi	inish:	3/13/2024-3/1-	4/2024	Drilling		q٠			Boring	Core Barrel:	NQ-2"	
	g Loca		4+55.9, 6.1 ft		Casing				-3" & H		Water Level*:	9.5 ft bgs.	
_			actor: 1.066	Lt.	Hamme			Automa			Rope & Cathead	7.5 ft 0gs.	
Definit D = Sp MD = U U = Th MU = U V = Fie	ons: lit Spoon S Jnsuccess in Wall Tu Jnsuccess eld Vane S	Sample sful Split Spo lbe Sample sful Thin Wa Shear Test,	oon Sample Atten all Tube Sample A PP = Pocket Pei ine Shear Test Att	SSA = Soli PSA = Hol RC = Rolle WOH = We netrometer WOR/C = WOR/C	Core Sample d Stem Auger ow Stem Auge	r Hammer or Casin		S _u = S _{u(la} q _p = N-un Hami	Peak/Re (b) = Lab Unconfin corrected mer Effici = SPT N-	molded Field Vane Undrained She Vane Undrained Shear Strength (jed Compressive Strength (ksf) = Raw Field SPT N-value ency Factor = Rig Specific Annual uncorrected Corrected for Hamme er Efficiency Factor/60%)'N-uncor	ear Strength (psf) psf) Calibration Value er Efficiency	T _V = Pocket Torvane She WC = Water Content, pen LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test	
		l _		Sample Information			_		1				Laboratory
O Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	Casing	Blows	Elevation (ft.)	Graphic Log	Visual De	scription and Rema	arks	Testing Results/ AASHTO and Unified Class.
- 55 -	RI	60/42	64.80 - 69.80	RQD = 7%		No	D-2	19.2		Top of Bedrock at Elev. 19		64.0	
- 65 - - 70 -	R2	42/34	69.80 - 73.30 73.30 - 78.30	RQD = 24% RQD = 55%					THE STATE OF THE S	Roller Coned ahead to 64.8 Set in NW casing to 64.8 ft R1: Bedrock: Pink, grey an BIOTITE-HORNBLENDE moderately to severely wea to sand, joints dipping at he 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 (2:06) 68.8-69.8 ft (2:55) 70% Recovery R2: Bedrock: Pink to grey, HORNBLENDE GRANITI joints dipping at horizontal grey and brown, medium to	bgs. d brown, medium to GRANITE, soft to n thered, with significate rizontal to moderate medium to coarse-gr E, moderately hard, r to low angles, closel	ained, BIOTITE-noderately weathered, y spaced, then pink,	
Rem	arks:												
		mer #367 NW(3") bi	roken casing ab	andoned in hole from 6	4.8 ft bgs (El	. 19.2) to	o 44.	.8 ft bg:	s (El. 39	2.2).			

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.

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Boring No.: BB-CHAR-201A

Maine Dep	artment	of Transport	ation	Project			ridge #3332 carries Station	Boring No.:	BB-CH	AR-201A
	Soil/Rock Expl US CUSTOMA			Locatio			oosehorn Brook Aaine	WIN:	216	86.10
Driller:	Seaboard Drill	ling LLC	Elevatio	n (ft.)	83.2			Auger ID/OD:	5" Solid Stem	
Operator:	Kevin/Jason		Datum:		NA	VD88		Sampler:	N/A	
Logged By:	B. Wilder		Rig Type	e:	Died	irich D-	50	Hammer Wt./Fall:	N/A	
Date Start/Finish:	3/13/2024-3/1	4/2024	Drilling	Method:	Case	ed Wasł	n Boring	Core Barrel:	NQ-2"	
Boring Location:	4+55.9, 6.1 ft	Lt.	Casing I	D/OD:	NW	-3" & H	[W-4"	Water Level*:	9.5 ft bgs.	
Hammer Efficiency	Factor: 1.066		Hammei	Type:	Autom		Hydraulic □	Rope & Cathead		
Definitions: D = Split Spoon Sample MD = Unsuccessful Split S U = Thin Wall Tube Sample MU = Unsuccessful Thin W V = Field Vane Shear Test, MV = Unsuccessful Field V	e lall Tube Sample A PP = Pocket Per ane Shear Test Att	SSA = Soling	ore Sample d Stem Auger ow Stem Auger r Cone light of 140 lb. H Veight of Rods eight of One Pe	Hammer or Casing	S _{u(la} q _p = N-un Ham N ₆₀	ab) = Lab Unconfir corrected mer Effic = SPT N-	wolded Field Vane Undrained Sh Vane Undrained Shear Strength i led Compressive Strength (ksf) d = Raw Field SPT N-value iency Factor = Rig Specific Annua uncorrected Corrected for Hammer er Efficiency Factor/60%)*N-unco	(psf) WC	Pocket Torvane She Water Content, per Liquid Limit Plastic Limit Plasticity Index Grain Size Analysis Consolidation Test	
 		Sample Information			1	1				Laboratory
Depth (ft.) Sample No. Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	Casing Blows	Elevation (ft.)	Graphic Log		escription and Remarks		Testing Results/ AASHTO and Unified Class
75						がた	HORNBLENDE GRANIT	E, severely weathered and	d decomposed to	
- 80					4.9	ACCOUNTY ACCOUNTY	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 E 81% Recovery R3: Bedrock: Pink to grey, HORNBLENDE GRANIT weathered, joints dipping a spaced, then pink, grey and GRANITE, soft to moderat decomposed to sand, joints very close. [Charlotte Pluton] Rock Quality = Fair.	Blocked medium to coarse-graine E, moderately hard to hat t horizontal to moderate a l brown, medium to coars tely hard, severly weather	rd, slightly angles, closely se-grained, red, with a zone	
85					-		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 **Bottom of Exploration** NW(3") Casing broke whill abandoned in hole from 64		(3") broken casing	
- 90					-					
- 95					-					
100										
	oroken casing ab	andoned in hole from 6-			4.8 ft bg	s (El. 39	2.2).	Page 4 of 4		

Boring No.: BB-CHAR-201A

than those present at the time measurements were made.

Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other

I	Maine	e Dep	artment	of Transporta	ation	1	Project:			Bridge #3332 carries Station	Boring No.:	BB-CH	AR-202
			Soil/Rock Exp JS CUSTOM				Locatio			Moosehorn Brook Maine	WIN:	2168	36.10
<u> </u>			M: DOT		1		(5)				A	511.0 11.1.0	
Drille			MaineDOT			ation	ι (π.)	82.:			Auger ID/OD:	5" Solid Stem	
<u> </u>	rator:		Daggett/Andr	le	Datu				VD8		Sampler:	Standard Split	Spoon
	ged By:		N. Pukay			Type			E 45		Hammer Wt./Fall:	140#/30"	
_	Start/Fi		3/12/2024-3/1				lethod:			sh Boring	Core Barrel:	NQ-2"	
	ng Loca		5+31.9, 9.5 ft	Kt.			D/OD:		/-4"		Water Level*:		
Defini D = S MD = U = TI MU = V = Fi	tions: olit Spoon S Unsuccess nin Wall Tu Unsuccess eld Vane S	Sample sful Split Split libe Sample sful Thin Wa Shear Test,	oon Sample Atter III Tube Sample A PP = Pocket Pe ne Shear Test At	RC = Roller WOH = We netrometer	ore Samp d Stem Au ow Stem A Cone ight of 140 Veight of F	ole iger Auger Olb. Ha Rods ol	mmer r Casing	S _{u(I} q _p = N-ur Han N ₆₀	= Peak ab) = I Unconcorre nmer E	Hydraulic ☐ Remolded Field Vane Undrained She ab Vane Undrained Shear Strength (i ffined Compressive Strength (ksf) ted = Raw Field SPT N-value ficiency Factor = Rig Specific Annual N-uncorrected Corrected for Hamme mer Efficiency Factor/60%)*N-uncor	psf) WC = LL = I PL = I Calibration Value PI = F er Efficiency G = G	Pocket Torvane She Water Content, per Liquid Limit Plastic Limit Plasticity Index Grain Size Analysis Consolidation Test	
		I		Sample Information					4				Laboratory
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.)	raphic Log	Visual De	scription and Remarks		Testing Results/ AASHTO and Unified Class.
0	1D	24/14	0.00 - 2.00	4/6/6/7	12	19	SSA		\bowtie	Brown, moist, medium den	se, SAND, little gravel, tra	ace silt, (Fill).	
- 5 -	2D	24/5	5.00 - 7.00	1/1/1/1	2	3		74.0		Brown, moist, very loose, C	Gravelly SAND, trace silt,	, (Fill). -8,5	G#380947 A-1-a, SW-SM WC=7.3%
- 10 -								/4.0	J	Grey-brown silt and clay or Brown, wet, soft, SILT, litt		10.0 ft bgs.	#380948
	3D	24/24	10.00 - 12.00	1/1/1/1	2	3	5			Brown, wet, son, Sier, nit	te peat, fittle saild, (weilar	id Deposit).	Organic Content 18.4%
							10	69.:	5 1000	Silt, peat, and sand observe	d in wash from 10.0 to 13.	.0 ft bgs.	WC=112.9%
							20	0511				1510	
15							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 sand, (Glaciomarine Depos 4D/A (16.0-17.0 ft bgs.) Gr	it).		G#380949 A-4, CL WC=23.5%
							25			trace clay, (Glaciomarine D		inty, line SAND,	Non-Plastic
							20						
							38						
- 20 -	5D	24/14	20.00 - 22.00	4/4/3/6	7	11	52			Grey, wet, medium dense, f (Glaciomarine Deposit).	fine SAND, some silt, trac	e clay,	
							45						
							50						
							35						
_ 25 _							38						
Rem	arks:												

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.

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Boring No.: BB-CHAR-202

N	Maine	e Dep	artment	of Transporta	atio	n	Project				ridge #3332 carries Station	Boring No.:	BB-CH	AR-202
			Soil/Rock Exp				Locatio				oosehorn Brook Iaine		21.6	26.10
			JS CUSTOM.	ARY UNITS								WIN:	2168	36.10
Drille	er:		MaineDOT		Ele	vation	(ft.)	82	.5			Auger ID/OD:	5" Solid Stem	
Ope	rator:		Daggett/Andr	le	Dat	tum:		N.	٩V	D88		Sampler:	Standard Split	Spoon
Logg	ged By:		N. Pukay		Rig	Туре	:	Cl	ИE	45C		Hammer Wt./Fall:	140#/30"	
-	Start/Fi		3/12/2024-3/1		-		lethod:				Boring	Core Barrel:	NQ-2"	
-	ng Loca		5+31.9, 9.5 ft	Rt.	_	sing IE			W-4			Water Level*:		
Ham Definit		ciency F	actor: 0.962	R = Rock C		mmer	Туре:	Auto			Hydraulic □ molded Field Vane Undrained She	Rope & Cathead ear Strength (psf) Ty =	Pocket Torvane She	ar Strength (psf)
D = S _I MD = U = TI MU = V = Fi	plit Spoon S Unsuccess nin Wall Tu Unsuccess eld Vane S	ful Split Spo be Sample ful Thin Wa hear Test,	oon Sample Atter III Tube Sample A PP = Pocket Pe ne Shear Test At	SSA = Solid	d Stem A ow Stem Cone ight of 14 Veight of	Auger Auger 40 lb. Ha	r Casing	S _u q _p N- Ha Ne	(lab = U unco mm 0 =) = Lab nconfir orrected er Effic SPT N	Vane Undrained Shear Strength (ed Compressive Strength (ksf) I = Raw Field SPT N-value lency Factor = Rig Specific Annual uncorrected Corrected for Hamme er Efficiency Factor/60%)'N-uncor	psf) WC	= Water Content, pen Liquid Limit = Plastic Limit Plasticity Index Grain Size Analysis Consolidation Test	
		<u> </u>		Sample Information	ō				┥					Laboratory
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	(11.)	Graphic Log		scription and Remarks		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, S	Silty, fine SAND, (Glacio	omarine Deposit).	
							42							
							45	1						
							51	1						
							46	1						
- 30 -	7D	24/14	30.00 - 32.00	2/3/4/3	7	11	47	1			Similar to 6D.			
							45	1						
							50	1						
							65							
25							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, som Till).	e sand, trace gravel, trace	e clay, (Glacial	
								-						
- 40 -								-			Grey, wet, hard, Sandy SIL	T some gravel (Glacial	Till)	
	9D	24/6	40.00 - 42.00	10/12/11/61	23	37		-			Grey, wet, hard, sandy Ste	1, some graver, (Glaciar	1111).	
								-						
								-	ļ					
								$\frac{1}{2}$						
- 45 -	10D	24/9	45.00 - 47.00	7/9/12/18	21	34		1			Similar to 9D.			
	100	2117	13.00 17.00	775712710				+	į					
								+	k					
								-	ľ					
								1						
50									į					
	arks:													
Stratif	ication line	s represent	approximate bou	ndaries between soil types; t	transitior	ns may b	e gradual.					Page 2 of 4		
		-	been made at tim	nes and under conditions stat ats were made.	ted. Gro	oundwate	er fluctuati	ons may	OC	cur due	to conditions other	Boring No	.: BB-CHAl	R-202

N	Main	e Depa	artment	of Transporta	tion	Pi	rojec				ridge #3332 carries Station	Boring No.:	BB-CH	AR-202
			Soil/Rock Exp			L	ocati			over Mo lotte, N	oosehorn Brook laine	WIN:	2168	86.10
					1									
Drille			MaineDOT		Elevation		t.)		32.5			Auger ID/OD:	5" Solid Stem	_
Oper			Daggett/Andrl	e	Datum:					'D88		Sampler:	Standard Split	Spoon
	ed By:		N. Pukay		Rig Typ					E 45C		Hammer Wt./Fall:	140#/30"	
_	Start/Fi		3/12/2024-3/1		Drilling						Boring	Core Barrel:	NQ-2"	
	ng Loca		5+31.9, 9.5 ft	Rt.	Casing				∃W-			Water Level*:		
Ham Definit		iciency F	actor: 0.962	R = Rock C	Hamme ore Sample	r Ty	pe:			tic⊠ Peak/Re	Hydraulic □ molded Field Vane Undrained She	Rope & Cathead ear Strength (nsf) T. =	Pocket Torvane She	ar Strength (nsf)
D = Sp	olit Spoon		oon Sample Atten	SSA = Solid	Stem Auger w Stem Auge	_		S	o Pu(lat	_{o)} = Lab	Vane Undrained Shear Strength (μ ed Compressive Strength (ksf)	osf) WC =	- Water Content, per Liquid Limit	
U = Th	in Wall Tu	ibe Sample	•	RC = Roller	Cone			N	√-unc	corrected	= Raw Field SPT N-value	PL =	Plastic Limit	
			II Tube Sample A PP = Pocket Pe		ght of 140 lb. /eight of Rods						ency Factor = Rig Specific Annual uncorrected Corrected for Hamme		Plasticity Index Grain Size Analysis	
MV =	Jnsuccess	sful Field Va	ne Shear Test Att		eight of One P	ersor	1	N	160 =	(Hamm	er Efficiency Factor/60%)*N-uncor	rected C = 0	Consolidation Test	1
				Sample Information	70	Т		Т						Laboratory
	o o	Ë	ept	in.)	cte					og.	١٣. ١٦.			Testing Results/
(#.)	<u>0</u>	Sec	<u>е</u>	(/6 Th	910		ס	ië E		lic L	Visual Des	scription and Remarks		AASHTO
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	3	Casing Blows	Elevation	<u>.</u>	Graphic Log				and Unified Class.
50	ιχ	3,	l % €	ಹರ್ಥ-	ż ž	_	<u> </u>		#)	<u> </u>	0 1 01	CAND 1 (CI	: 1 7731	Orimou Grass.
30	11D	24/13	50.00 - 52.00	20/17/28/31	45 72	2					Grey, wet, very dense, Silty	SAND, some gravel, (G	acial Till).	
								1						
						+	+	-						
								7						
								\dashv						
- 55 -								_			al II			
33	12D	24/5	55.00 - 57.00	26/33/43/56	76 122	2	۱ ۱				Similar to 11D.			
							11							
							11/	4						
							W	1,	4.7				57.8 ⁻	
	R1	60/20	58.30 - 63.30	ROD = 0%			√ NQ-2			N. XX	Top of Bedrock at Elev. 24.		27.0	
	MD	1.2/0	58.30 - 58.40	50(1.2")		+	+	+			Pink granite sand in wash a	t 57.8 ft bgs.		
- 60 -										1.50	R1: Bedrock: Pink, grey and BIOTITE-HORNBLENDE			
00											moderately to severely wear			
								1			joints dipping at horizontal [Charlotte Pluton]	to vertical angles, spaced	very close.	
								4			Rock Quality = Very Poor.			
										19.75 19.75	R1: Core Times (min:sec) 58.3-59.3 ft (1:13)			
	R2	56 4/28	63.30 - 68.00	RQD = 0%				1		がある	59.3-60.3 ft (1:28)			
	112	20.020	00.00	1142 070		-	_	-			60.3-61.3 ft (1:22) 61.3-62.3 ft (3:38)			
- 65 -								╛		1	62.3-63.3 ft (1:31)			
03										ر در از از از از از از از از از از از از از	33% Recovery			
						+	+	┨		1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	R2: Bedrock: Similar to R1			
						_		4		八八八五八十二十二十二十二十二十二十二十二十二十二十二十二十二十二十二十二十二十	[Charlotte Pluton] Rock Quality = Very Poor.			
										大学	R2: Core Times (min:sec)			
	R3	81.6/62	68.00 - 74.80	RQD = 38%							63.3-64.3 ft (1:59) 64.3-65.3 ft (1:37)			
		0110/02	7 1100	1142 2071				-			65.3-66.3 ft (2:36)			
- 70 -											66.3-68.0 ft (3:24) Core Blo 49% Recovery	ocked		
/0											•		1 DIOTITE	
						+	+	1			R3: Bedrock: Pink to grey, HORNBLENDE GRANITE			
								4			joints dipping at horizontal [Charlotte Pluton]	to moderate angles, close	ly spaced.	
							, Ι,				Rock Quality = Poor.			
						\top	$\forall f$				R3: Core Times (min:sec) 68.0-69.0 ft (1:55)			
						+	-\/-	-		學商	68.0-69.0 ft (1:55) 69.0-70.0 ft (0:47)			
75 _							V		<u>7.7</u>	5	70.0-71.0 ft (1:53)			
	arks:													
Ctrotic	ootion lin -	o represe	annrovimete h	adariaa hatusan aail t : t	rancitions ~	, bo -	rodu-			_		Page 3 of 4		

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-202

I	Main	e Dep	artment	of Transport	ation	Pro	ject:			ridge #3332 carries Station	Boring No.:	BB-CH	AR-202
			Soil/Rock Exp			Loc	atio	Road on: Chai		oosehorn Brook Maine	l		
			US CUSTOM	ARY UNITS							WIN:	2168	36.10
Drill	er:		MaineDOT		Elevat	ion (ft.))	82.5			Auger ID/OD:	5" Solid Stem	
Ope	rator:		Daggett/Andi	rle	Datum	:		NAV	/D88		Sampler:	Standard Split	Spoon
Log	ged By:		N. Pukay		Rig Ty	pe:		СМІ	E 45C		Hammer Wt./Fall:	140#/30"	
Date	Start/F	inish:	3/12/2024-3/	13/2024	Drillin	g Metho	od:	Case	d Wasl	n Boring	Core Barrel:	NQ-2"	
Bori	ng Loca	ation:	5+31.9, 9.5 ft	Rt.	_	ID/OD		HW-	4"		Water Level*:		
Ham Defini		iciency F	actor: 0.962		Hamm Core Sample	er Type	e:	Automa		Hydraulic □ emolded Field Vane Undrained She	Rope & Cathead ar Strength (psf) T =	Pocket Torvane She	ar Strength (nef)
D = S MD = U = T MU = V = F	plit Spoon Unsuccess hin Wall Tu Unsuccess ield Vane S	sful Split Sp ube Sample sful Thin Wa Shear Test,	oon Sample Atte all Tube Sample A PP = Pocket Penne Shear Test A	SSA = Soli MSA = Holl RC = Rolle Attempt WOH = We enetrometer WOR/C = WOH/C = WO	d Stem Auge low Stem Aug	er . Hamme Is or Casi		S _{u(la} q _p = N-uno Hamr N ₆₀ =	b) = Lab Unconfir corrected ner Effic = SPT N	Vane Undrained Shear Strength (ned Compressive Strength (ksf) d = Raw Field SPT N-value iency Factor = Rig Specific Annua -uncorrected Corrected for Hammer er Efficiency Factor/60%)'N-unco	psf)	= Water Content, per Liquid Limit = Plastic Limit Plasticity Index Grain Size Analysis Consolidation Test	
			T	Sample Information	g								Laboratory
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	ne0 Casina	Blows	Elevation (ft.)	Graphic Log		escription and Remarks		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		54.0	
										Bottom of Exploration	n at 74.8 feet below grou	-74.8- ind surface.	
- 80 -													
- 85 -													
- 90 -													
- 95 -													
100													
Rem	l narks:	ı			I	1							
Stratit	fication line	es represent	approximate bou	undaries between soil types;	transitions m	ay be gra	dual.				Page 4 of 4		
			been made at tir	mes and under conditions sta	ated. Ground	water fluc	tuation	ns may o	ccur due	to conditions other	Boring No	.: BB-CHAI	R-202

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 <u>Laboratory Testing Summary Sheet</u>

Town(s): Charlotte Work Number: 21686.10

10011(0).	Onan								. 2100		
Boring & Sample	Station	Offset	Depth	Reference	G.S.D.C.		L.L.	P.I.		ssification	
Identification Number	(Feet)	(Feet)	(Feet)	Number	Sheet	%				AASHTO	
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		0
BB-CHAR-202, 3D	5+31.9	9.5 Rt.	10.0-12.0	380948		112.9			Ignition, (T 267) 1	8.4%
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
		-			-						
		-			-						
		-									
		-									
		 			ļ				ļ		

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 SIEVE ANALYSIS HYDROMETER ANALYSIS US Standard Sieve Numbers Grain Diameter, mm 2" 1 1/2" 1" 3/4" 1/2" 3/8" 1/4" #4 #8 #10 #16 #20 #60 #100 #200 0.05 0.03 0.010 0.001 100 0 10 90 80 20 Percent Retained by Weight 30 50 60 30 70 80 20 10 90 100 12.7 9.53 2.36 2.00 25.4 19.05 6.35 4.75 0.075 0.05 0.03 0.005 100 10 0.1 0.01 0.001 Grain Diameter, mm GRAVEL CLAY SAND SILT

UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
0	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
•									
X									

WI	IN		
021686.10			
Town			
Charlotte			
Reported by/Date			
WHITE, TERRYA	4/19/2024		



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

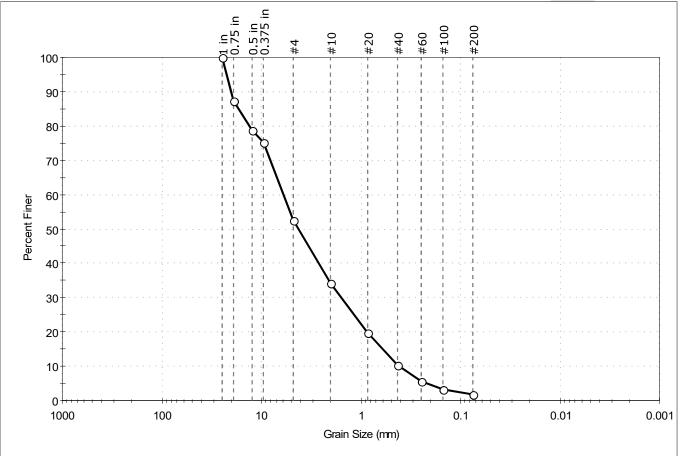
Depth: 14-16 ft Test Id: 414662

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		

<u>Coefficients</u>				
D ₈₅ =16.9377 mm	$D_{30} = 1.5593 \text{ mm}$			
D ₆₀ =5.9824 mm	$D_{15} = 0.5961 \text{ mm}$			
D ₅₀ =4.2337 mm	$D_{10} = 0.4095 \text{ mm}$			
C ₁₁ =14.609	$C_c = 0.992$			

ASTM Poorly graded sand with gravel (SP)

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

<u>Sample/Test Description</u> Sand/Gravel Particle Shape: ANGULAR

 ${\sf Sand/Gravel\; Hardness: HARD}$



Client: Golder Associates

Project: Station Road Culvert Replacement

Location: Charlotte, ME

Boring ID: BB-CHAR-101 Sample Type: jar Tested By: jbr Sample ID: 7D Test Date: 06/26/17 Checked By: emm

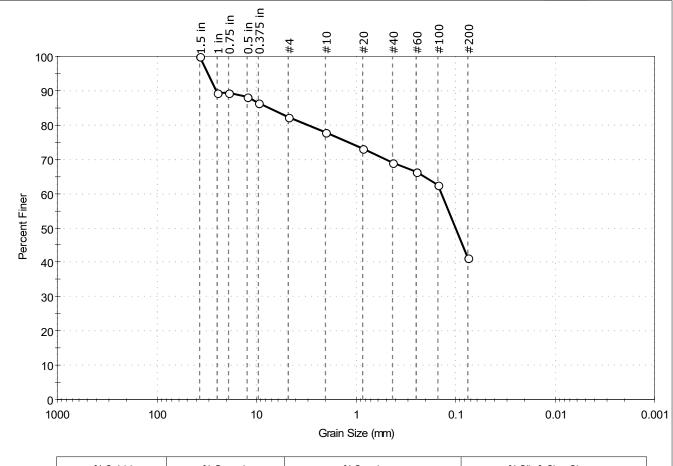
Depth: 25-27 ft Test Id: 414665

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		

	<u>Coefficients</u>
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

GTX-306601

Project No:

ASTM N/A Classification

AASHTO Silty Soils (A-4 (0))

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:

Boring ID: BB-CHAR-101 Sample Type: jar Tested By: jbr Sample ID: 10D Test Date: 06/27/17 Checked By: emm

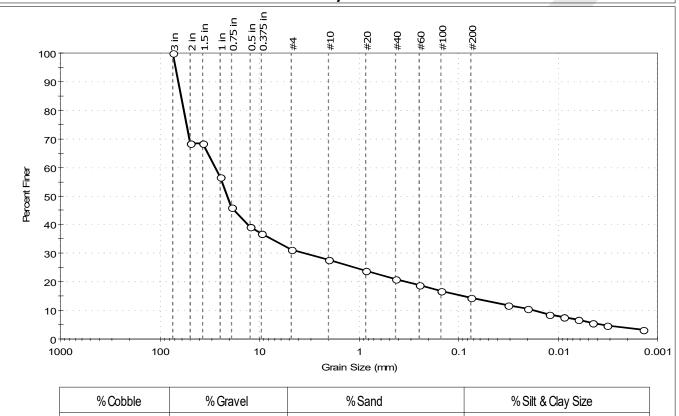
40-42 ft Test Id: Depth: 414666

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		

<u>Coefficients</u>				
D ₈₅ =61.8853 mm	$D_{30} = 3.5615 \text{ mm}$			
D ₆₀ = 28.1010 mm	D ₁₅ = 0.0874 mm			
D ₅₀ = 21.0464 mm	$D_{10} = 0.0179 \text{ mm}$			
C _u =1569.888	$C_c = 25.217$			

GTX-306601

Classification **ASTM** N/A

<u>AASHTO</u> Stone Fragments, Gravel and Sand (A-1-a(0))

<u>Sample/Test Description</u> 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



Client: Golder Associates

Project: Station Road Culvert Replacement

Location: Charlotte, ME Project No:

Boring ID: BB-CHAR-102 Sample Type: jar Tested By: jbr Sample ID: 2D Test Date: 06/26/17 Checked By: emm

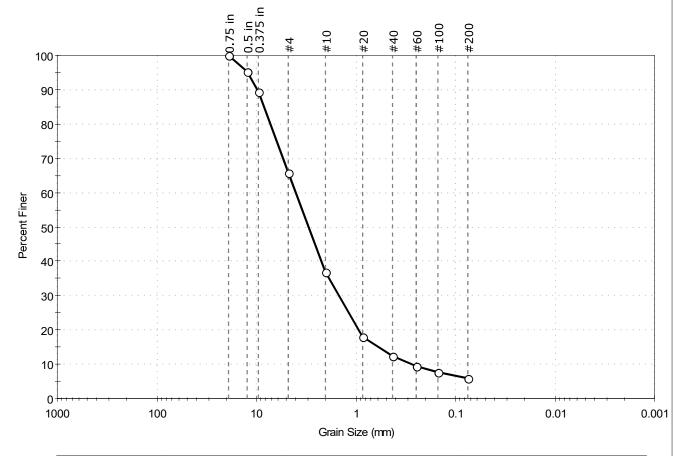
Depth: 3-5 ft Test Id: 414663

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		

<u>Coeffic</u>	<u>cients</u>
D ₈₅ =8.3398 mm	$D_{30} = 1.4650 \text{ mm}$
D ₆₀ = 3.9837 mm	$D_{15} = 0.5911 \text{ mm}$
D ₅₀ = 2.9581 mm	$D_{10} = 0.2719 \text{ mm}$
Cu =14.651	$C_c = 1.981$

GTX-306601

ASTM N/A Classification

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

<u>Sample/Test Description</u> Sand/Gravel Particle Shape: ANGULAR

 ${\sf Sand/Gravel\; Hardness: HARD}$



Client: Golder Associates

Project: Station Road Culvert Replacement

Location: Charlotte, ME

Boring ID: BB-CHAR-102 Sample Type: jar Tested By: jbr Test Date: Sample ID: 8D 06/26/17 Checked By: emm

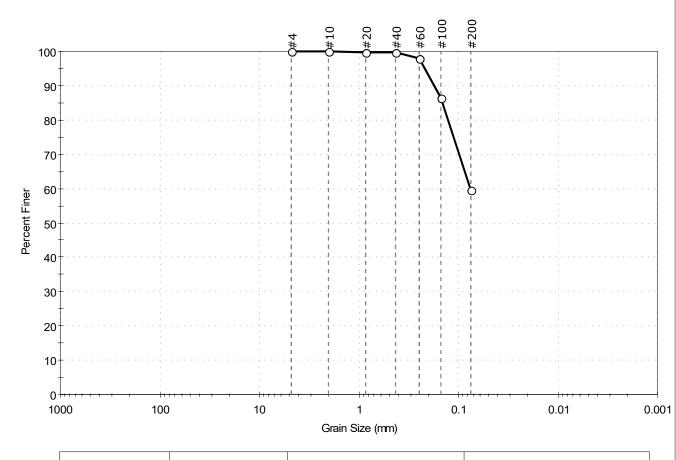
Depth: 21-23 ft Test Id: 414664

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		

<u>Coefficients</u>							
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						

Project No:

GTX-306601

Classification <u>ASTM</u> N/A AASHTO Silty Soils (A-4 (0))

<u>Sample/Test Description</u> Sand/Gravel Particle Shape : ---Sand/Gravel Hardness: ---



Client: Golder Associates

Project:

Station Road Culvert Replacement Location: Charlotte, ME

Boring ID: BB-CHAR-102 Sample Type: jar Tested By: jbr Sample ID: 14D Test Date: 06/27/17 Checked By: emm

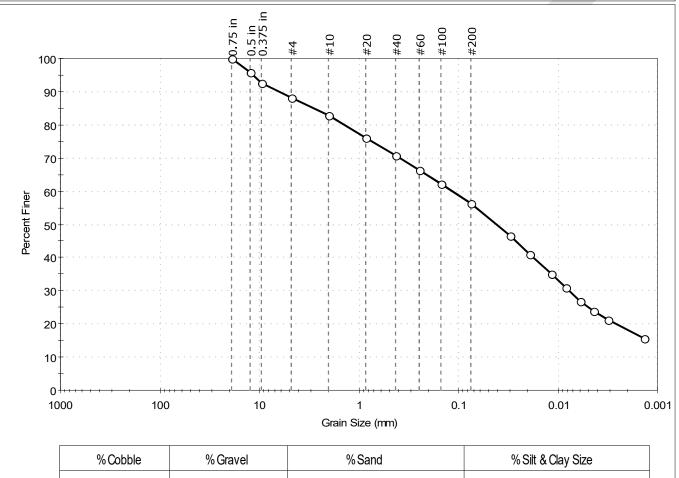
Depth: 45-47 ft Test Id: 414667

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		

<u>Coefficients</u>							
D ₈₅ = 2.8100 mm	$D_{30} = 0.0078 \text{ mm}$						
D ₆₀ = 0.1156 mm	$D_{15} = N/A$						
D ₅₀ = 0.0415 mm	$D_{10} = N/A$						
$C_{ij} = N/A$	$C_c = N/A$						

Project No:

GTX-306601

Classification <u>ASTM</u> N/A AASHTO Silty Soils (A-4 (0))

<u>Sample/Test Description</u> 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



Client: Golder Associates

Project: Station Road Culvert Replacement

Location:Charlotte, MEProject No:GTX-306601Boring ID:BB-CHAR-102Sample Type: jarTested By:cam

Boring ID: BB-CHAR-102 Sample Type: jar Tested By: cam
Sample ID: 6DA Test Date: 06/28/17 Checked By: emm

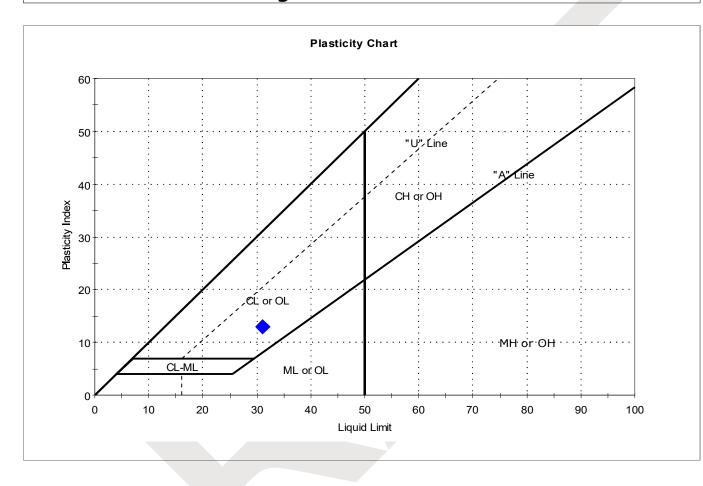
Depth: 17-19 ft Test Id: 414668

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	3-CHAR-1	(17-19 ft	25	31	18	13	0.5	

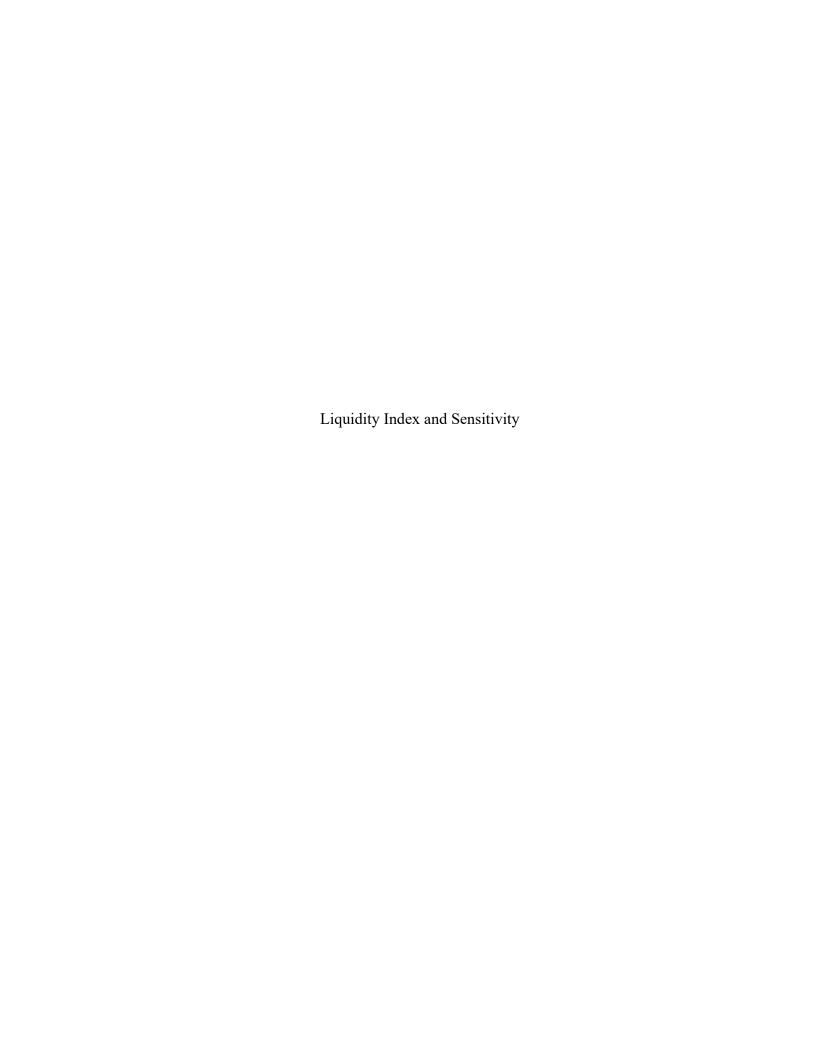
Sample Prepared using the WET method

Dry Strength: VERY HIGH

Dilatancy: SLOW Toughness: LOW

Appendix D

Calculations



Liquidity Index

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

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

BB-CHAR-102, 6D

$$WC := 25.0$$

$$LL := 31$$

$$PL := 18$$

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



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

$$A_g := 26.1 \cdot in^2$$

$$d := 13.8 \cdot in$$

$$b := 14.7 \cdot in$$

$$t_f := 0.615in$$

$$t_w := t_f$$

$$A_{box} := d \cdot b$$

$$A_{box} = 202.86 \cdot in^2$$

$$r_s := 3.53in$$

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

Pile yield strength

$$F_v := 50 \cdot ksi$$

$$E := 29000 \cdot ksi$$

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

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

From Table 6.9.4.2.1-1:

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

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

 $\frac{b_{\rm f}}{t_{\rm f}} = 11.951$ H-pile size is nonslender for flange members

For webs: $\lambda_{\rm rw} \coloneqq 1.09 \sqrt{\frac{E}{F_{\rm y}}} \qquad \text{where b}_{\rm w} \text{= Web height/distance between flanges}$

 $\lambda_{rw} = 26.251 \hspace{1cm} b_w \coloneqq d - 2 \cdot t_f \hspace{1cm} b_w = 12.57 \cdot in$

 $\frac{b_w}{t_w} = 20.439$ 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 $Pr = \varphi_c Pn$

Nominal Axial Structural Resistance

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

 $P_0 = 1305 \cdot kip$

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

I = "unbraced" length

 $l_{unbraced bot} := 0.1 \cdot ft$

Assume in pure compression

LRFD eq. 6.9.4.1.2-1

$$P_{e} := \frac{\pi^{2} \cdot E}{\left(\frac{K_{eff} \cdot l_{unbraced_bot}}{r_{s}}\right)^{2}} \cdot A_{g}$$

$$P_e = 1.53 \times 10^8 \cdot \text{kip}$$

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

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

$$\frac{P_o}{P_e} = 8.529 \times 10^{-6}$$
 If Po/Pe < or = 2.25, then: $P_n := \frac{P_o}{0.658} \cdot P_o$ LRFD Eq. 6.9.4.1.1-1

then:

this applies to all pile sizes

 $_{\rm p} = 1305 \cdot {\rm kip}$

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

 $= 652 \cdot \text{kip}$

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. Apile 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

$$\phi_c := 0.5$$

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

$$P_r = 652 \cdot \text{kip} \qquad 14x89$$

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

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

$$\frac{P_{r \text{ ee}} = 1305 \cdot \text{kip}}{14x89}$$

Drivability Analyses

Ref: LRFD Article 10.7.8

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

 $\varphi_{da} := 1.0$ 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.

 $\phi_{dvn} := 0.65$ 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).

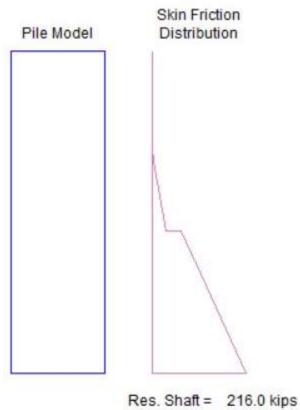
Use constant shaft resistances so that GRLWeap will assign approx. 191 kips as skin friction based on shaft resistance calculations and local experience in similar deposits.

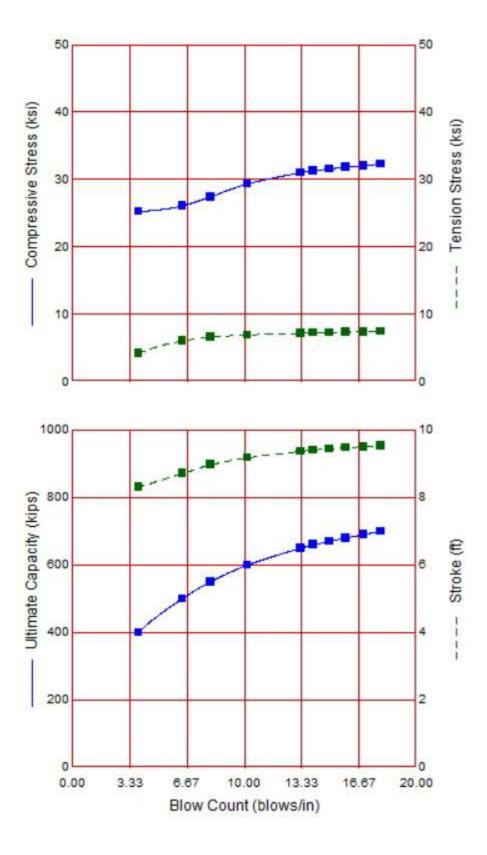
Use the GRLWEAP simple resistance distribution with varying soil layer consistencies added to the soil profile input. This assumes that the amount of skin friction developed between layers may vary and is influenced by soil consistency.

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





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

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

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

Extreme and Service Limit States

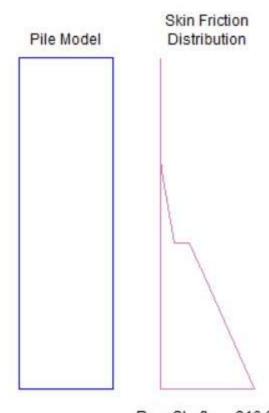
$$R_{dr} := R_{ndr} \cdot \varphi$$

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

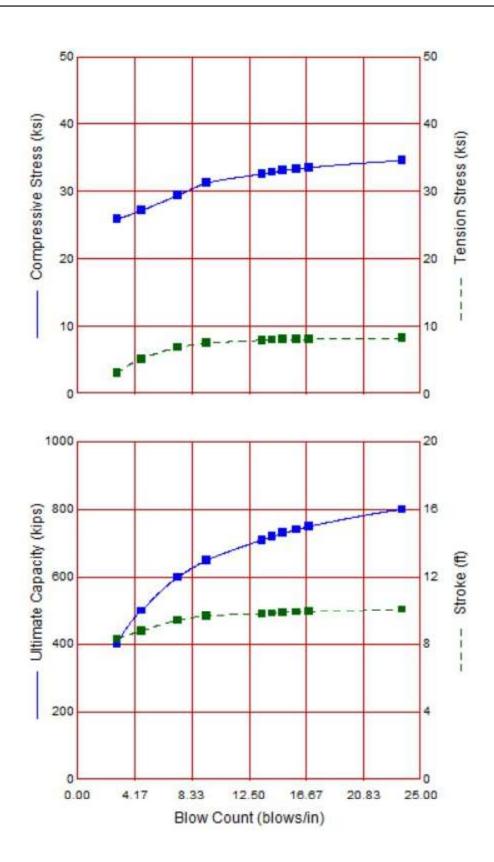
Abutment 1, Pile Size is 14 x 89, APE D25-42 Hammer

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 Efficiency		0.800	kips
Pressure		1425 (100%)	psi
Helmet W Hammer COR of H	Cushion		kips kips/in
Skin Qua Toe Quak Skin Dam Toe Dam	te nping		75.51
Pile Leng Pile Pene Pile Top	tration	70.00 49.00 26.10	ft



Res. Shaft = 216.0 kips (Constant Res. Shaft)



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

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

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

Extreme and Service Limit States

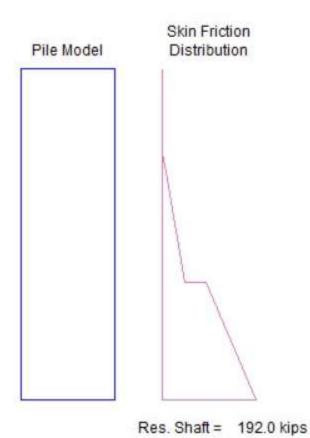
$$R_{dr}\!:=\,R_{ndr}\!\cdot\!\varphi$$

 $R_{dr} = 730 \cdot kip$

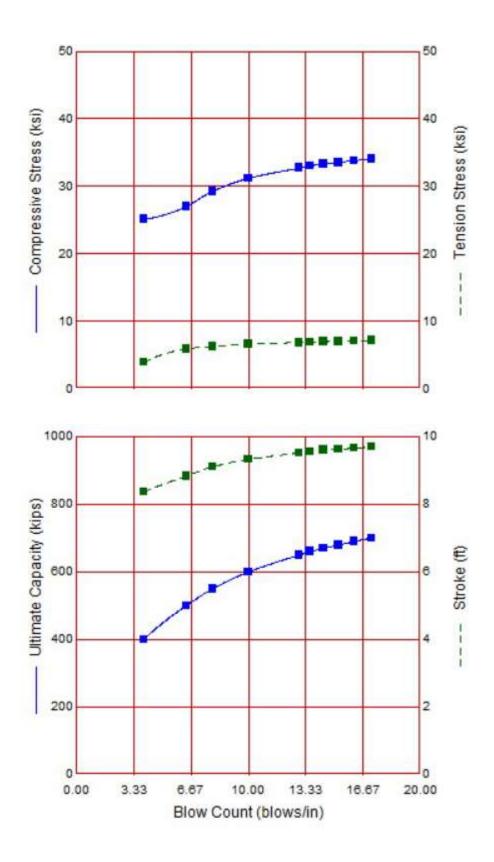
Abutment 2, Pile Size is 14 x 89, APE D19-42 Hammer

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



(Constant Res. Shaft)



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

 $R_{fdr} := \, R_{ndr} {\cdot} \varphi_{dyn}$

 $R_{fdr} = 436 \cdot kip$

Extreme and Service Limit States

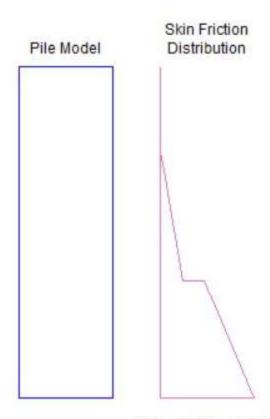
 $R_{dr}\!:=\,R_{ndr}\!\cdot\!\varphi$

 $R_{dr} = 670 \cdot kip$

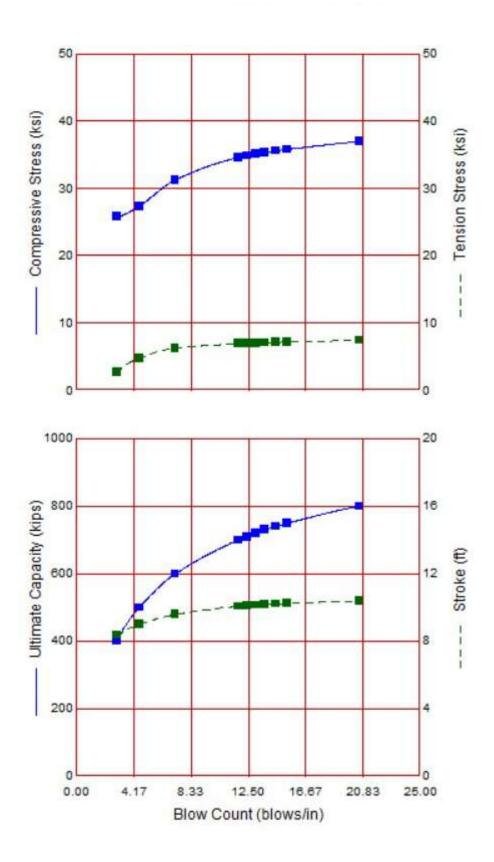
Abutment 2, Pile Size is 14 x 89, APE D25-42 Hammer

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



Res. Shaft = 192.0 kips (Constant Res. Shaft)



Maine DOT 21686 Charlotte 14x89 ABT #2 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	25.88	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

$$R_{fdr} := \, R_{ndr} {\cdot} \varphi_{dyn}$$

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

Extreme and Service Limit States

$$R_{dr}\!:=\,R_{ndr}\!\cdot\!\varphi$$

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

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	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
APE D19-42	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	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
APE D25-42	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	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
APE D19-42	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	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
APE D25-42	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	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
APE D19-42	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	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
APE D25-42	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	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
APE D19-42	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	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
APE D25-42	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	

HIGHWAY BRIDGES

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

Rock			C	(1)
Category	General Description	Rock Type	(ksf)	(psi)
Α	Carbonate rocks with well-	 Dolostone	700- 6,500	4,800-45,000
	developed crystal cleavage	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
В	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	Conglomerate	700- 4,600	4,800-32,000
•	crystals and poor cleavage	Sandstone	1,400- 3,600	9,700-25,000
		Quartzite	1,300- 8,000	9,000-55,000
D	Fine-grained igneous	Andesite	2,100- 3,800	14,000-26,000
	crystalline rock	Diabase	450-12,000	3,100-83,000
E	Coarse-grained igneous and	Amphibolite	2,500- 5,800	17,000-40,000
	metamorphic crystalline rock	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.

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

$$(4.4.8.2.2\text{-}2)$$

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):

$$E_m = \alpha_E E_0$$
 (4.4.8.2.2-3)

$$\alpha_{\rm E} = 0.0231({\rm RQD}) - 1.32 \ge 0.15 \quad (4.4.8.2.2-4)$$

For preliminary design or when site-specific test data cannot be obtained, guidelines for estimating values of $E_{\rm o}$ (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 $\alpha_{\rm E}=0.15$ should be used to estimate $E_{\rm m}.$

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:

21686.10

Backfill engineering strength parameters

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

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

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

Cohesion $c_1 := 0 \cdot psf$

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

 $Rise_{ABT1} := -0.3ft$ $Rise_{ABT2} := -0.3ft$ Run := 25ft

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

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

Integral Abutment - Passive Earth Pressure - Coulomb Theory

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

 ϕ_1 = Angle of internal friction $\phi' = 32 \cdot \text{deg}$

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

Use Coulomb for cases where interface friction is considered; typically gravity shaped structures, and integral abutments where the ratio of wall height to wall movement is .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)

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

$$\mathsf{K}_{\mathsf{p_coulomb}} \coloneqq \frac{\sin(\beta - \varphi')^2}{\sin(\beta)^2 \cdot \sin(\beta + \delta') \cdot \left(1 - \sqrt{\frac{\sin(\varphi' + \delta') \cdot \sin(\varphi' + \alpha_{\mathsf{ABT1}})}{\sin(\beta + \delta') \cdot \sin(\beta + \alpha_{\mathsf{ABT1}})}}\right)^2} \quad \begin{array}{l} \mathsf{Das}, \\ \mathsf{Principles} \ \mathsf{of} \\ \mathsf{Foundation} \\ \mathsf{Engineering} \\ \mathsf{7th} \ \mathsf{Ed}. \ \mathsf{p.} \ \mathsf{366} \\ \mathsf{Eg.} \ \mathsf{7.71} \end{array}$$

 $K_{p \text{ coulomb}} = 5.82$

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_{D} when $\alpha > 0$.

 α = Angle of fill slope to the horizontal

$$\alpha = -0.69 \cdot \text{deg}$$

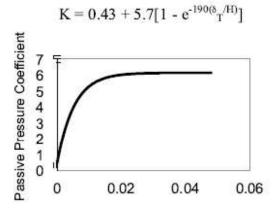
$$\mathsf{K}_{\mathsf{p_rank}} \coloneqq \cos(\alpha) \cdot \frac{\cos(\alpha) + \sqrt{\cos(\alpha)^2 - \cos(\varphi')^2}}{\cos(\alpha) - \sqrt{\cos(\alpha)^2 - \cos(\varphi')^2}}$$

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

 $K_{p_rank} = 3.25$

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

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



Relative Wall Displacement

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

Estimate Thermal Movement

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

where:

L = expansion length (in.)

 α = coefficient of thermal expansion (in./in./°F)

Bridge Span (Approximate Expansion Length)

$$L := 74ft$$
 $L = 888 \cdot in$

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

$$\alpha_{\text{steel}} \coloneqq 0.0000065$$

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

Bridge superstructure will consist of concrete beams, choose α_{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	

$$T_{\text{Max}} := 80$$

$$T_{Min} := 0$$

$$\Delta \coloneqq \alpha_{\text{concrete}} \!\cdot\! L \!\cdot\! \left(\mathsf{T}_{\text{Max}} - \mathsf{T}_{\text{Min}} \right)$$

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

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

$$\delta = 0.21 \cdot \text{in}$$

Compute Relative Wall Displacement

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

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

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

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)

	Friction	Coefficient of
	Angle, δ	Friction, tan δ
Interface Materials	• .	-
	(degrees)	(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:		
	22 to 26	0.40 to 0.49
 Clean gravel, gravel-sand mixture, well-graded rock fill with spalls 	17 to 22	0.31 to 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		
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_n

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.

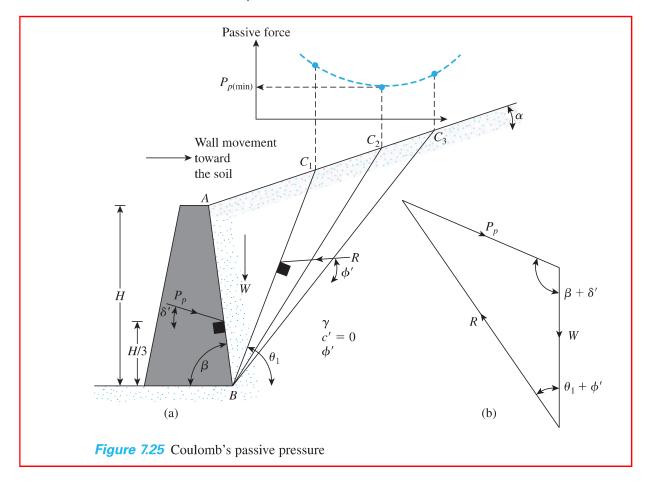
Table 7.9 (Continued)									
			c ' /γz						
$oldsymbol{\phi}'$ (deg)	lpha (deg)	0.025	0.050	0.100	0.500				
30	0	3.087	3.173	3.346	4.732				
	5	3.042	3.129	3.303	4.674				
	10	2.907	2.996	3.174	4.579				
	15	2.684	2.777	2.961	4.394				

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



			$oldsymbol{\delta}'$ (deg)		
$oldsymbol{\phi}'$ (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

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

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

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

where

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

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

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

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 kN/m²

Again, at z = 3 m,

$$\sigma'_o = (15.72)(2) + (\gamma_{\text{sat}} - \gamma_w)(1)$$

= 31.44 + (18.86 - 9.81)(1) = 40.49 kN/m²

Hence,

$$\sigma'_p = \sigma'_o K_{p(2)} + 2c'_2 \sqrt{K_{p(2)}} = 40.49(2.56) + (2)(10)(1.6)$$

= 135.65 kN/m²

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/m².

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
		$P_P \approx 223.3 \text{ kN/m}$

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

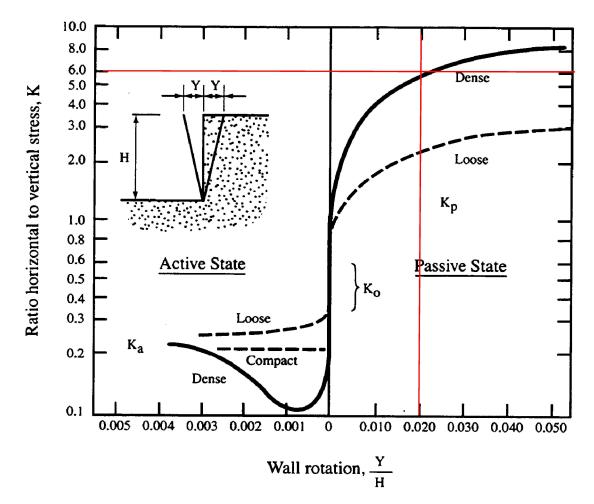
$$\sigma_p' = \gamma z K_p \tag{7.65}$$

and the passive force is

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

where

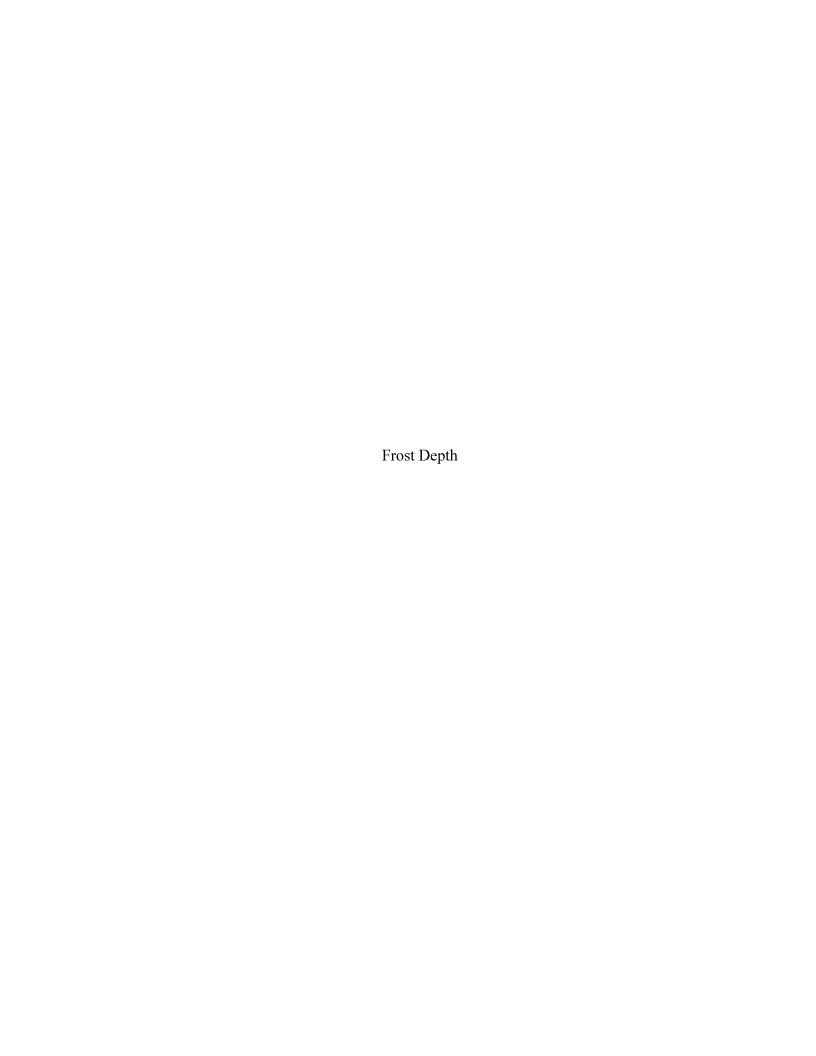
$$K_p = \cos \alpha \frac{\cos \alpha + \sqrt{\cos^2 \alpha - \cos^2 \phi'}}{\cos \alpha - \sqrt{\cos^2 \alpha - \cos^2 \phi'}}$$
 (7.67)



Magnitude of Wall Rotation to Reach Failure

Soil type and	Rotation, Y/H		
condition	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).



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%

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

d=Depth of Frost Penetration

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

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

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

$$DFI_3 := 1350$$

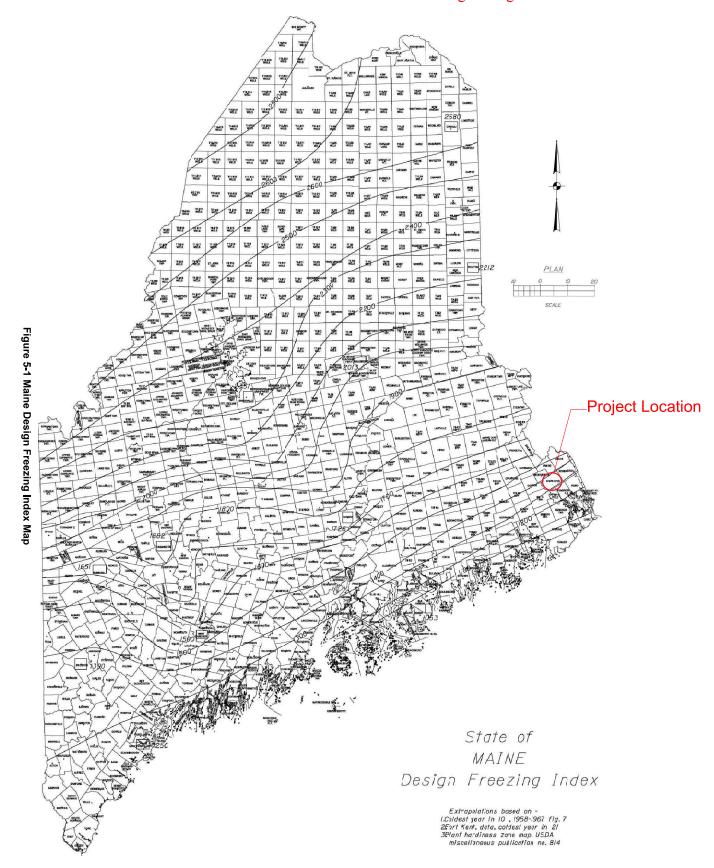
$$d_{coarse} \coloneqq d_1 + \left(DFI_3 - DFI_1\right) \cdot \frac{\left(d_2 - d_1\right)}{\left(DFI_2 - DFI_1\right)}$$

$$d_{coarse} = 77.8 \cdot in$$
 $d_{coarse} = 6.5 \cdot for$

$$d_{coarse} = 6.5 \cdot ft$$

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

MaineDOT Bridge Design Guide



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	Frost Penetration (in)					
Freezing	Coarse Grained		Fine Grained			
Index	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

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	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				
N ₆₀	di	di/N		
14	3	0.21		
5	2	0.40		
	2	0.40		
		1.33		
6	2	0.33		
6	2	0.33		
7	2	0.29		
11	2	0.18		
8	2	0.25		
5	5	1.00		
18		0.28		
29	5	0.17		
38	5	0.13		
42	5	0.12		
100		0.05		
100	5	0.05		
100	40	0.40		
	100	5.93		
	N ₆₀ 14 5 5 6 6 6 7 11 8 5 18 29 38 42 100 100	N ₆₀ di 14 3 5 2 5 2 6 8 6 2 7 2 11 2 8 2 5 5 18 5 18 5 29 5 38 5 42 5 100 5 100 5		

Мı	/di	/NI	28.00
u	u	/ 1 1	20.00

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

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 Sa (sec) (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

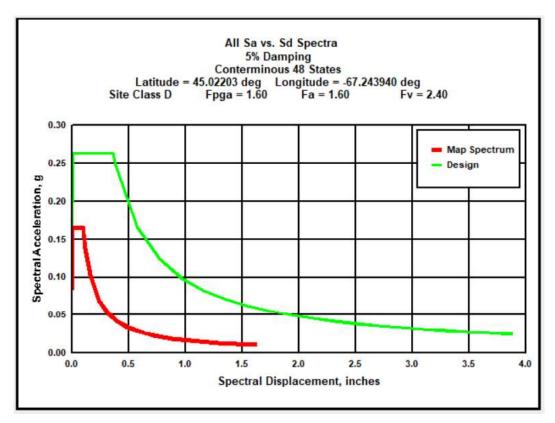
Latitude = 45.022028 Longitude = -067.243944

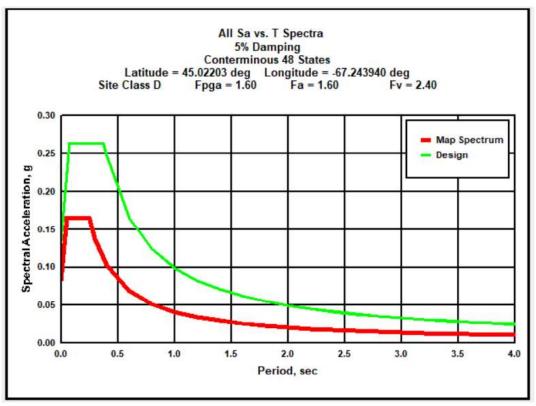
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1

Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40

Data are based on a 0.05 deg grid spacing.

Period Sa (sec) (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







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 years 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 ft²

 Load
 0.2 ksf

 Depth
 0 ft

 Installation Stage
 Stage 1 = 0 y

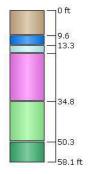
Coordinates

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

Soil Layers

Ground Surface Drained: Yes

Orouna C	dilace Brained: 165			
Layer #	Туре	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

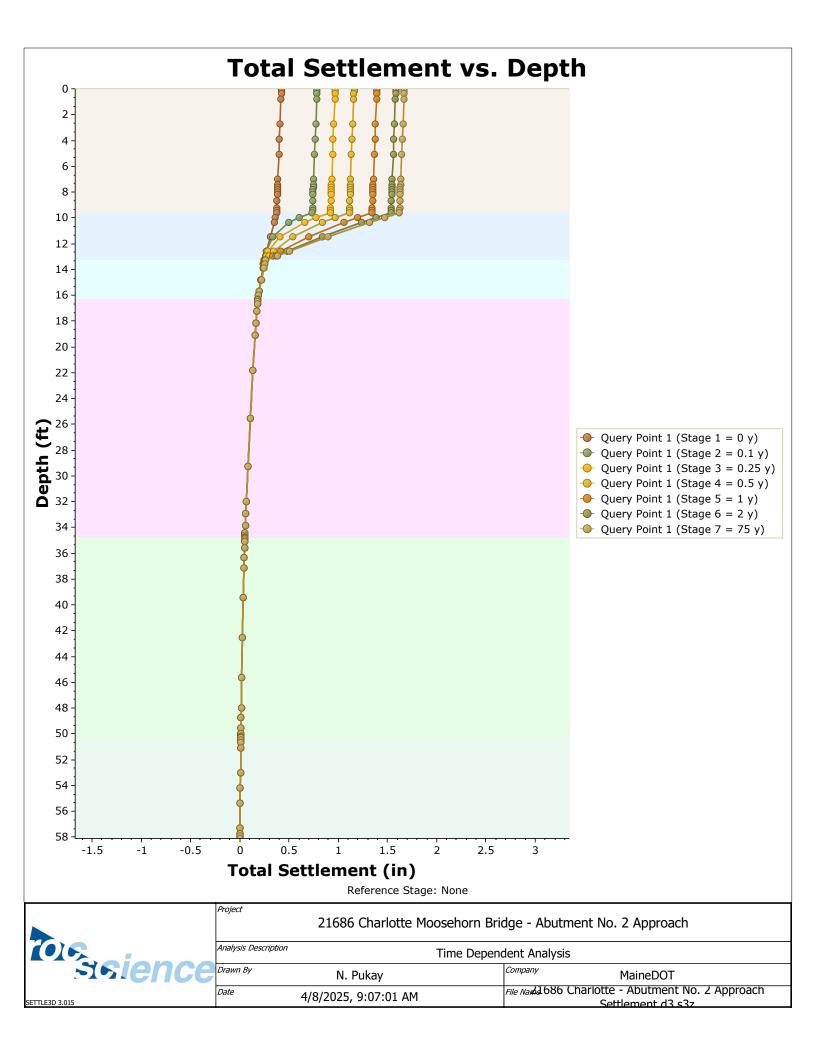
Groundwater

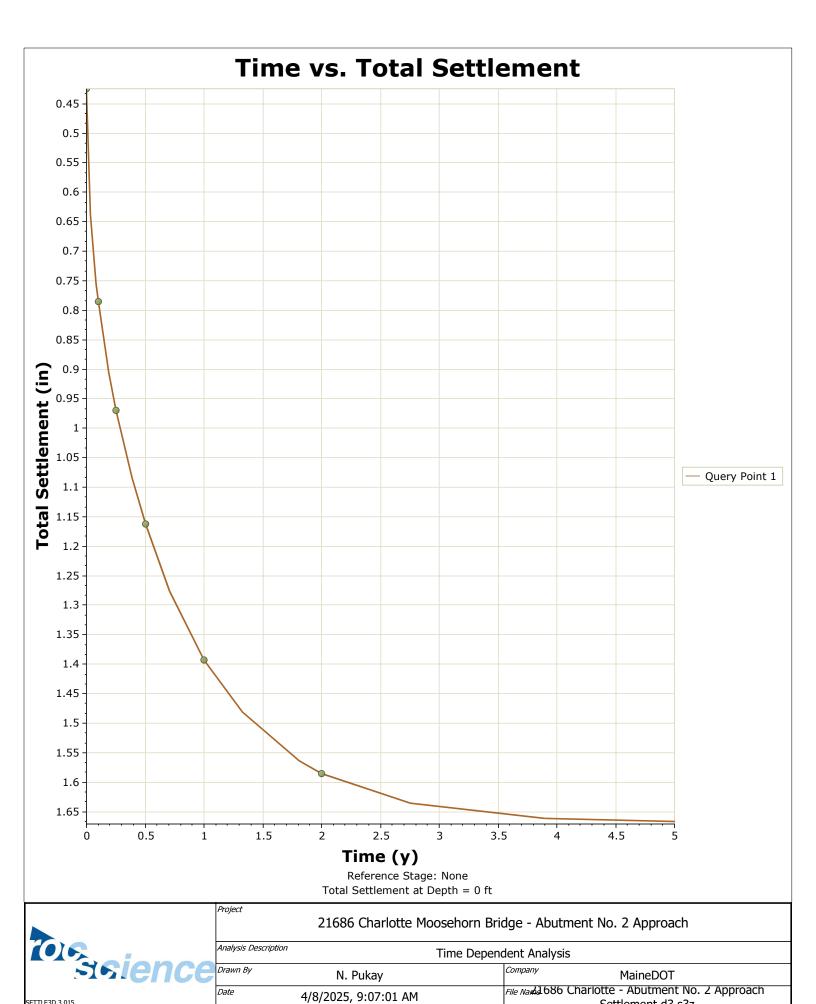
Piezometric Line Entities

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





Settlement d3 s3z

SETTLE3D 3.015

Calculated By: NPP 4-9-25

Checked By: LK 5-22-25

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

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

$$\gamma_{\text{fill}} := 125 \text{pcf}$$

BDG Table 3-3, Soil Type 4, Granular Borrow

$$\sigma_{\text{z_induced}} \coloneqq \gamma_{\text{fill}} \cdot H_{\text{fill}}$$

$$\sigma_{z_induced} = 0.2 \cdot ksf$$

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

$$\mathsf{E_{S1}} \coloneqq \frac{500 \Big(\mathsf{N_1} + 15 \Big)}{50} \mathsf{ksf}$$

Bowles Table 5-6, Equation for stress-strain modulus $\rm E_{\rm s}$ for Sand (normally consolidated)

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

$$\mathsf{E}_{ur1} \coloneqq 4\!\cdot\!\mathsf{E}_{s1}$$

Mayne and Cox, Eq. 5 Constitutive Model Input

Parameters,
$$E_s = E_{50}$$

 $\gamma_{ ext{dry1}} \coloneqq 125$ pcf

BDG Table 3-3, Soil Type 4, Granular Borrow

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

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

$$\gamma_{\mathsf{sat1}} = 131.3 \cdot \mathsf{pc1}$$

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

$$N_{60} = 3$$

$$\mathsf{E}_{\mathsf{S2}} \coloneqq \frac{300 \left(\mathsf{N}_{\mathsf{60}} - 2 + 6 \right)}{50} \mathsf{ksf}$$

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

E_s for Silt

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

$$\mathsf{E}_{ur2} \coloneqq 4 \cdot \mathsf{E}_{s2}$$

Mayne and Cox, Eq. 5 Constitutive Model Input

Parameters, $E_s = E_{50}$

$$\mathsf{E}_{\mathsf{ur}2} = 216 \cdot \mathsf{ksf}$$

$$\gamma_{\mathsf{dry2}} \coloneqq 51 \cdot \mathsf{pcf}$$

Das, Table 3.2: Dry Unit Weights, Soft Organic Clay

38-51 pcf

$$W_{sat2} := 112.9\%$$

BB-CHAR-202, 3D Natural Water Content

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

$$\gamma_{\mathsf{sat2}}$$
 = 109 · pcf

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

Das, Table 11.6: Correlations for Compression Index,

Organic soils, peats, organic silt, and clay

$$Cc_2 = 1.3$$

$$C_{r2} := 0.13$$

Assume 10% of Cc

OCR₂ := 1.0

Conservatively assume normally consolidated

 $e_2 := 1.5$ Assumed

$$C_{V2} := 6.458 \frac{ft^2}{yr}$$

Settle3D recommended values for organic silt - lower bound

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

 $N_{60} = 6$

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.

Calculated By: NPP 4-9-25

Checked By: LK 5-22-25

$$\mathsf{E}_{s3} := \frac{300 \left(\mathsf{N}_{60_3} + 6 \right)}{50} \mathsf{ksf}$$

Bowles Table 5-6, Equation for stress-strain modulus $\mathbf{E}_{\mathbf{s}}$ for Silt

 $E_{s3} = 72 \cdot ksf$

 $E_{ur3} := 4 \cdot E_{s3}$ Mayne and Cox, Eq. 5 Constitutive Model Input Parameters, $E_s = E_{50}$

 $E_{ur3} = 288 \cdot ksf$

 $\gamma_{\text{dry3}} := 93 \cdot \text{pcf}$ Das, Table 3.2: Dry Unit Weights, Soft Clay 73-93 pcf

w_{sat3} := 23.5% BB-CHAR-202, 4D Natural Water Content

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

 $\gamma_{sat3} = 115 \cdot pct$

 $C_{c3} := 0.13$ Edmunds Township, Soils Report 54-21

 $C_{-2} := 0.02$ Edmunds Township, Soils Report 54-21

 $d_3 := 82.8 \text{ft} - 69.5 \text{ft} = 13.3 \cdot \text{ft}$ Depth to top of soil layer 3

OCR₃ := 1.5 Andrews, Table IV, OCR at varying depths At depth=10, OCR=2.25; At depth=15, OCR=1.47

e₃ := 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.

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

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

$$N_{60} = 16$$

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

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

Calculated By: NPP 4-9-25

Checked By: LK 5-22-25

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

$$\mathsf{E}_{\mathsf{ur4}} \coloneqq 4 \cdot \mathsf{E}_{\mathsf{s4}}$$

Mayne and Cox, Eq. 5 Constitutive Model Input

Parameters,
$$E_s = E_{50}$$

$$\mathsf{E}_{\mathsf{ur4}} = 620 \cdot \mathsf{ksf}$$

$$\gamma_{\mathsf{dry4}} \coloneqq 105 \cdot \mathsf{pcf}$$

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

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

$$\gamma_{\mathsf{sat4}} = 128 \cdot \mathsf{pct}$$

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

$$N_{60_5} := 36$$

$$\mathsf{E_{85}} \coloneqq \frac{300 \cdot \left(\mathsf{N_{60_5}} + 6\right)}{50} \mathsf{ksf}$$

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

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

$$\mathsf{E}_{ur5} \coloneqq 4 \!\cdot\! \mathsf{E}_{s5}$$

Mayne and Cox, Eq. 5 Constitutive Model Input

Parameters,
$$E_s = E_{50}$$

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

$$\gamma_{\mathsf{dry5}} \coloneqq 120 \cdot \mathsf{pcf}$$

Holtz & Kovacs, Table 2-1: Dry Unit Weights, Glacial

Till 106-144 pcf.

$$W_{sat5} := 11.7\%$$

BB-CHAR-102, 14D Natural Water Content

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

$$\gamma_{sat5} = 134 \cdot pct$$

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

$$N_{60} = 50$$

$$\mathsf{E}_{s6} := \frac{250 \cdot \left(\mathsf{N}_{60_6} + 15 \right)}{50} \mathsf{ksf}$$

Bowles Table 5-6, Equation for stress-strain modulus $E_{\rm s}$, Sand (saturated)

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

$$\mathsf{E}_{ur6} \coloneqq 4 \cdot \mathsf{E}_{s6}$$

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

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

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

 $\hbox{Holtz \& Kovacs, Table 2-1: Dry Unit Weights, Glacial}\\$

Till 106-144 pcf.

$$W_{sat6} := 11.7\%$$

BB-CHAR-102, 14D Natural Water Content

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

$$\gamma_{\mathsf{sat6}} = 145 \cdot \mathsf{pc}$$

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

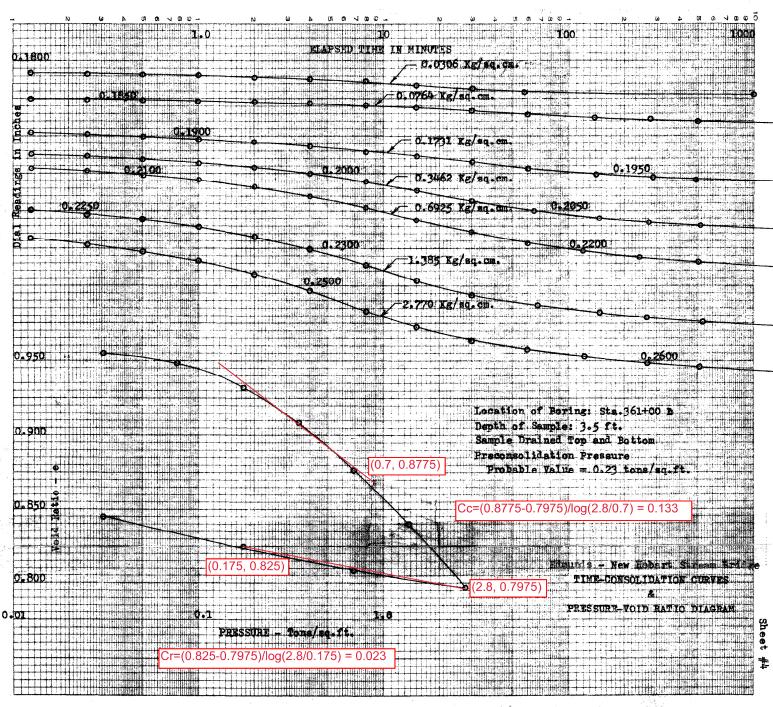


Table 11.6 Correlations for Compression Index, C_c^*

Equation	Reference	Region of applicability
$C_c = 0.007(LL - 7)$	Skempton (1944)	Remolded clays
$C_c = 0.01 w_N$		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
$C_c = 0.0115 w_N$		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

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

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

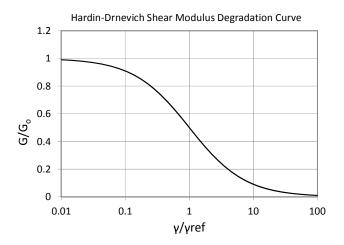


Figure 1. Shear modulus reduction curve (after Hardin and Drnevich 1972)

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

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

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),

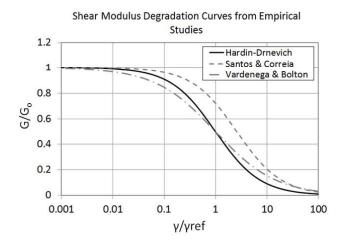


Figure 2. Reduction curves from fitted experimental data studies

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

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 $\gamma_{0.7}$.

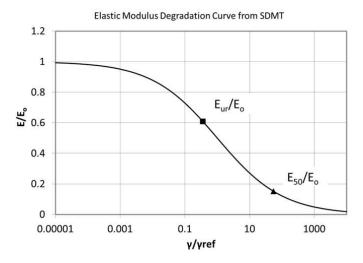


Figure 3. Elastic Modulus reduction curve using SDMT

Andrews, D. W. (1986). The engineering aspects of the Presumpscot formation.

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 = Apparent past vertical pressure, Pc 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 X 10-8 to 1 X 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.

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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	СРТ
Sand (normally consolidated)	$E_s = 500(N + 15)$ = 7000 \sqrt{N} = 6000N $\pm E_x = (15000 \text{ to } 22000) \cdot \ln N$	$E_s = (2 \text{ to } 4)q_u$ $= 8000 \sqrt{q_c}$ $$ $E_s = 1.2(3D_r^2 + 2)q_c$ $*E_s = (1 + D_r^2)q_c$
Sand (saturated)	$E_s = 250(N+15)$	$E_s = Fq_c$ e = 1.0 $F = 3.5e = 0.6$ $F = 7.0$
Sands, all (norm. consol.)	$\P E_s = (2600 \text{ to } 2900)N$	
Sand (overconsolidated)	$\dagger E_s = 40000 + 1050N$	$E_s = (6 \text{ to } 30)q_e$
Gravelly sand	$E_{s(OCR)} \approx E_{s,ne} \sqrt{OCR}$ $E_s = 1200(N+6)$ $= 600(N+6) N \le 15$ = 600(N+6) + 2000 N >	
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 ${}^{\$}E_s' = 2500 < q_c < 5000$ use $E_s' = 4000$ where $E_s' = 4000$ constrained modulus $= 4000$	$=4q_c+5000$
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}

Moi	ist unit weight (γ)	Dry ur	nit weight (γ_d)	Satura	ted unit weight (γ _{sat})
Given	Relationship	Given	Relationship	Given	Relationship
w, G_s, e	$\frac{(1+w)G_s\gamma_w}{1+e}$	γ, w	$\frac{\gamma}{1+w}$	G_s , e	$\frac{(G_s+e)\gamma_w}{1+e}$
S, G_s, e	$\frac{(G_s + Se)\gamma_w}{1 + e}$	G_s , e	$\frac{G_s\gamma_w}{1+e}$	<u> </u>	$[(1-n)G_s + n]\gamma_w$ $(1+w_{cat})$
w, G_s, S	$(1+w)G_s\gamma_w$	G_s , n		G_s , $w_{\rm sat}$	$\left(\frac{1+w_{sat}}{1+w_{sat}G_s}\right)G_s\gamma_w$
w, G_s, s	$\frac{(1+w)G_s\gamma_w}{1+\frac{wG_s}{S}}$	G_s, w, S	$\frac{G_s \gamma_w}{1 + \left(\frac{wG_s}{S}\right)}$	e, w_{sat}	$\left(rac{e}{w_{ m sat}} ight)\!\left(rac{1+w_{ m sat}}{1+e} ight)\!\gamma_w$
w, G_s, n S, G_s, n	$G_s \gamma_w (1-n)(1+w)$ $G_s \gamma_w (1-n) + nS \gamma_w$	e, w, S	$\frac{eS\gamma_w}{(1+e)w}$	$n, w_{\rm sat}$	$nigg(rac{1+w_{ m sat}}{w_{ m sat}}igg)\gamma_w$
		$\gamma_{ m sat},e$	$(1+e)w$ $\gamma_{\text{sat}} - \frac{e\gamma_w}{1+e}$	γ_d , e	$\gamma_d + \left(\frac{e}{1+e}\right) \gamma_w$
		$\gamma_{\rm sat}$, n	$\gamma_{\rm sat} - n\gamma_{m}$	γ_d , n	
			$\frac{(\gamma_{\text{sat}} - \gamma_w)G_s}{(G_s - 1)}$	γ_d , S	$\left(1-\frac{1}{G_s}\right)\gamma_d+\gamma_w$
		/ sat, os	(G_s-1)	$\gamma_d,w_{ m sat}$	$\gamma_d(1+w_{\rm sat})$

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

		Natural moisture content in a	Dry unit	weight, γ_d
Type of soil	Void ratio <i>, e</i>	saturated state (%)	lb/ft³	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

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

				Friction A	ngles				
		Clans		At mate		At Peak S	Strength		
	Αı	Slope agle of Repose		ngth	Mediu	m Dense	De	nse	
Classification	i(°)	Slope (vert. to hor.)	φ _{cv} (°)	tan ϕ_{cv} .	φ(°)	tan ϕ	φ(°)	tan ϕ	-
Silt (nonplastic)	26	1 on 2	₍ 26	0.488	28	. 0.532	30	0.577	
SE 2864530	to		to		to		to	4	
6 Frg. 11, 14 LAMBE/W	нт, 30	1 on 1.75	30	0.577	32	0.625	34	0.675	
Uniform fine to	- 26	1 on 2	26	0.488	30	0.577	32	0.675	-
medium sand	to	#	to		to		to		į.
mediam come	30	1 on 1.75	30	0.577	3,4	0.675	36	0.726	1
Well-graded sand	30	1 on 1.75	:30	0.577	34)	0.675	38	0.839	20
,0,,	to	າ ເ ໃ	/ to	35%	to		' to	need on expens	
	34	1 on 1.50 ³¹	34	0.675	40	0.839	46 GA	JAL 1.030	
Sand and gravel	32	1 on 1.60	32	0.625	36)	0.726	اسبي(40)	0.900	
	to		to		to		to		
	36	1 on 1.40	36	0.726	42	0.900	48	1.110	

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

			Void	Water -		Unit W	cight		
	Description med DENSESAND	Porosity (n)	Ratio (e)	Content (w)*	g/c	u cm	lb/d	u lt .	-
•	Y=118+130 =124			,,	7 d ^b	Yeat	γd	Yest	22125
	1. Uniform sand, loose	0.46	0.85	32	1,43	1.89	90	118 7	一次問題
	2. Uniform sand, dense	0.34	0.51	19	1.75	2.09	109 -	130	1 15.00
	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 -	3
	5. Windblown silt (loess)	0.50	0.99	21	1.36	1.86	85	116	1241135
ř	6. Glacial till, very mixed-grained	0.20	0.25	9	2.12	2.32	132	145	
l. ~	fi. Soft glacial clay	0.55	1.2	45	1.22	1.77	76	110	VI IX (-D
ual -	8. Stiff glacial clay	0.37	0.6	22	1.70	2.07	106	129	in lane
120	9. Soft slightly organic clay	0.66	1.9	70	0.93	1.58	58,	98	Wed, ban
124)	10. Soft very organic clay	0.75	3.0	110	0.68	1.43	43	89	C=130FC
	 Soft montinorillonitic clay (calcium bentonite) 	0.84	5.2	194	0.43	1.27	27_	80.	

^{&#}x27;w = water content when saturated, in per cent of dry weight.

 $V_{\gamma d} = \text{dry unit weight.}$

^{&#}x27; year = saturated unit weight.

of Soil Index Properties

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[2-1]

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BASIC SOILS ENGINEERING

Hough, 2nd Ed., 1969

Typical Values TABLE 2-3.

		Port.	Part. Size & Gradation	adation	دونات
	Approx	Size /	Approx.	Approx. Size Approx. Approx. Range	<u>.</u>
	Range (mm.)	mm.)	D_{10}	Unif. Coef.,	
	Отах	D_{\min}	(mm)	ບ ^ສ	-,24%
Granvlar Materials					
1. Uniform Materials					
a. Equal spheres (theoretical values)	ı	ŀ	1	1.0	
b. Standard Ottawa SAND	0.84	0.59	0.67	1.1	ومشدودي
c. Clean, uniform SAND (fine or medium)	1	1	1	1.2 to 2.0	
d. Uniform, inorganic SILT	0.05	0.005	0.012	1.2 to 2.0	
2. Well-graded Materials					·-
a. Silty SAND	2.0	0.005	0.02	5 to 10	
b. Clean, fine to coarse SAND	2.0	0.05	60.0	4 to 6	
c. Micaceous SAND	Ī	1	1	1	
d. Silty Sand & Gravel	100	0.005	0.02	15 to 300	
Mixed Soils					ناجير حي
1. Sandy or silty CLAY	2.0	0.001	0.003	10 to 30	*********
2. Skip-graded silty CLAY with stones or rk. frag.	250	0.001	1	1	
3. Well-graded GRAVEL, SAND, SILT & CLAY mixture	250	0.001	0.002	25 to 1000	
Clay Soils		-1			
1. CLAY (30 to 50% clay sizes)	0.05	0.05 0.5μ	0.001	ſ	•
2. Colloidal CLAY (-0.002 mm. 5 50%)	10.0	10Å	1	1	<u>`</u>
Organic Soils					
1. Organic SILT	1	ł	1	ı	ىدەنىد
2. Organic CLAY (30 to 50% clay sizes)	١	1	1	1	يتودمت

* Granular materials may reach enax when dry or only slightly moist. Clays can

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

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 To judge whether a soil at a given void ratio, e, is to be described as dense or loose, has an individual range of porosity or void ratio.

	ere ere ere ere ere ere ere ere ere ere	Voids*					Unit Wei	Unit Weight (1b./cv.ft)	/cu.ft)		
	Void Ratio	ıtio	Porosity (%)	ity (%)	Dry	y Wt., Ydry	dry	Wet Wt., Ywet	t., Ywet	Sub. W	W., \sub
e	e Ct.	emin	7 max	n _{min}	Min.	100%	Max.	Min.	Мах.	Mîn.	Max.
(loose)		(dense)	10	-	(loose)	Mod.	(dense)	(loose)	(dense)	(loose)	(dense)
											*
0.92	- 23	0.35	47.6	26.0	ı	1	ij	١	1	I	1
0.80	0 0.75	0.50	44	33	92	ı	110	93	131	57	69
1.0		0.40	50	29	83	115	118	84	136	52	73
1.1		0.40	52	29	80	1	118	81	136	21	73.
06.0		0.30	47	23	87	122	127	88	142	54	79
0.95	0.70	0.20	49	17	82	132	138	98	148	53	86
1.2		0.40	ວ	29	76	1	120	7.7	138	48	76
0.85	1	0.14	46	12	68	ı	146‡	06	155‡	56	92
H .00	1	0.25	64	20	. 09	130	135	100	147	88	88
1.0	1	0.20	20	1.7	84	1	140	115	151	53	89
0.70	0.	0.13	4	Π.	100	140	148§	125	156 §	62	94
4.2	1	0.50	12	. 88	20	105	112	96	133	31	17
12	1	09.0	92	27	13	06	106	77	128	α	99
										_	

† 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.

81

100

1 1

30

35

75

0.55

1 1

3.0

§ 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.

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, Da (sometimes, though not advisedly, referred to as degree of compaction), defined as,

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

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

$$dish = 462 g$$

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

ly 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_{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 the voids as well as the density of the buld range from slightly above 1000 to 2.4 Mg/m³). Typical values of ρ_s 800 kg/m³ (2.5 to 2.8 Mg/m³). Most 6 and 2.7 Mg/m³. For example, a

common mineral in sands is quartz; its $\rho_s = 2.65 \text{ Mg/m}^3$. Most clay soils have a value of ρ_s between 2.65 and 2.80 Mg/m³, depending on the predominant mineral in the soil, whereas organic soils may have a ρ_s as low as 2.5 Mg/m³. Consequently, it is usually close enough for geotechnical work to assume a ρ_s of 2.65 or 2.70 Mg/m³ for most phase problems, unless a specific value of ρ_s is given.

2.2 Basic Definitions and Phase Relations

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/m³ (1 g/cm³), 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/m³ = 1 Mg/m³.

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' .

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

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

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

Strictly speaking, total ρ should be used instead of $\rho_{\rm sat}$ 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 , $\rho_{\rm sat}$, 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 ρ_{sat} Sands and gravels $ \beta - 501.9 - 2.4$ Silts and clays $ \beta - \beta 1.4 - 2.1$		ρ' 1.0–1.3 &
		1.0–1.3
	}32 {2U.0−1.8	0.4-1.1 2
Slacial tills $ 3 = 502.1 - 2.4$		1 1_14
Crushed rock 119 -1371.9-2.2	98-1751.5-2.0	0.9-1.2
Peats $u^2 - v^2 = 1.0 - 1.1$	6 - 14 0.1−0.3	0.0-0.1
Organic silts and clays $ \beta - \beta \cdot 1.3 - 1.8$	0.5-1.5	0.3 - 0.8