MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

BABCOCK BRIDGE
STATE ROUTES 9 AND 126 OVER COBBOSSEECONTEE STREAM
LITCHFIELD – WEST GARDINER, MAINE

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Kennebec County | Soils Report 2022-03
WIN 23094.01 | Bridge No. 2029

Federal Project No. 02309401
March 1, 2022

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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 Babcock Bridge which carries State Routes 9 and 126 (Lewiston Road) over Cobbosseecontee Stream between Litchfield and West Gardiner, Maine. This report presents the subsurface information obtained at the site during the subsurface investigation, geotechnical design recommendations, and construction recommendations for the new substructures.

The existing Babcock Bridge was constructed in 1931 and is a 55-foot span, single-span, concrete tee beam bridge. The mass concrete abutments and wingwalls are founded on concrete footings. The bridge is built on a granular and rock fill causeway due to the width of Cobbosseecontee Stream.

According to the 2020 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the FHWA Sufficiency Rating of the bridge is 51.6. The channel is in fair condition with major bank erosion. The bridge has a low load rating and is structurally deficient.

The proposed replacement structure consists of a 88-foot, single-span steel girder bridge founded on pile-supported integral abutments. Piles will be driven to bedrock. 1.75H:1V (horizontal:vertical) riprap slopes will be placed in front of the new integral abutments.

The bridge and the approaches will be constructed on a new horizontal alignment to improve site distance, and the bridge will be widened to match the existing approach shoulders, resulting in widened approach sections, mostly to the west. The vertical elevation at the bridge and approaches will be increased by up to 1.5 feet.

The existing bridge will be closed during construction and traffic detoured onto local roads.

2.0 GEOLOGIC SETTING

The existing structure carries State Routes 9 and 126 (Lewiston Road) over Cobbosseecontee Stream as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of the Purgatory Quadrangle, Maine, Open-File No. 05-46 (2005), indicates the surficial soils in the vicinity of the bridge project consist the Presumpscot Formation and artificial fill. The Presumpscot Formation is generally comprised of silt, clay, sand, and minor amounts of gravel. This soil deposit typically overlies an irregular surface of glacial till and may include areas of till exposed at the ground surface.

The MGS Bedrock Geology of the Purgatory Quadrangle, Maine, Open-File No. 10-21 (2010), indicates the project site is mapped within the Litchfield Pluton, which is an igneous formation consisting of nepheline SYENITE, named “Litchfieldite” by Bayley (1892). Bedrock cores retrieved at the site are identified as Syenite.

3.0 SUBSURFACE INVESTIGATION

Three test borings were drilled to explore subsurface conditions at the site. Boring BB-LCS-101 was drilled at the proposed location of Abutment No. 1. Borings BB-LCS-102 and BB-LCS-102A were drilled at the proposed location of Abutment No. 2. The boring locations are shown on Sheet 2 – Boring Location Plan.

The test borings were drilled in October 2019 by the MaineDOT Drill Crew. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 4 – 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 in the subsurface investigation was equipped with an automatic hammer to drive the split spoon. The hammer was calibrated per ASTM D 4633 “Standard Test Method for Energy Measurement for Dynamic Penetrometers” in June 2019. All N-values discussed in this report are corrected N-values computed by applying an average energy transfer of 0.886. The hammer efficiency factor (0.886) and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring logs.

Bedrock was cored using an NQ-2” core barrel and the Rock Quality Designation (RQD) of the core calculated. A MaineDOT geotechnical engineer selected the boring locations and drilling methods, designated type and depth of sampling techniques, and reviewed the field logs for accuracy. A MaineDOT NETTCP Certified Subsurface Inspector 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.

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 five standard grain size analyses with natural water content, one grain size analysis with hydrometer and natural water content, and one Atterberg limits test.

Soil laboratory testing was performed at 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 Sheet 4 -Boring Logs.

5.0 Subsurface Conditions

Subsurface conditions encountered in the test borings generally consisted of Fill, Glaciomarine Deposits, Glacial Till, and Igneous Bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs. A generalized subsurface profile is shown on Sheet 3 – Interpretive Subsurface Profile. The following paragraphs discuss the subsurface conditions encountered.

5.1 Fill

Fill materials were encountered in the borings. The thickness of the Fill unit encountered was approximately 18.5 to 25.5 feet and it consisted of Granular Fill and Rock Fill. The Granular Fill encountered was approximately 8.5 to 12.5 feet thick and it generally consisted of:

- Brown, SAND, trace to some silt, little gravel;
- Brown, Gravelly SAND, little silt.

The Rock Fill was approximately 6 to 17 feet thick and it generally consisted of:

- Grey, SAND, some rock fragments, trace silt;
- Grey COBBLES and GRAVEL.

Corrected SPT N-values in the Granular Fill ranged from 13 to 27 blows per foot (bpf) indicating the granular fill is medium dense in consistency. Corrected SPT N-values in the Rock Fill ranged from 21 to 47 bpf indicating the rock fill is medium dense to dense in consistency.

Two grain size analyses performed on samples recovered from the granular fill unit indicated the material is classified as A-2-4 and A-1-b under the AASHTO Soil Classification System and SM under the Unified Soil Classification System (USCS). The natural water contents of the samples tested ranged from approximately 8 to 9 percent.

5.2 Glaciomarine Deposit

A Glaciomarine Deposit was encountered beneath the fill layer in BB-LCS-102. The encountered thickness was approximately 2.5 feet. The Glaciomarine Deposit consisted of grey, Silty CLAY, trace fine sand.

Atterberg limits tests were conducted on one sample of the Glaciomarine deposit and are summarized below:

Boring No. and Sample No.	Soil Description	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-LCS-102, 4D	Silty CLAY	36.5	35	21	14	1.1

The plasticity index of the sample indicate that the soils have slight plasticity (Burmister, 1949). The natural water content of the tested sample was approximately 37 percent and liquid limit was 29. The liquidity index was 1.1. Interpretation of these results indicates that the soils are on the verge of being a viscous liquid, with a liquidity index in excess of 1, as the natural water content exceeds the liquid limit. Soils with liquidity indices in excess of 1 have a high liquefaction potential.

One corrected SPT N-value within the Glaciomarine Deposit was 6 bpf, indicating the deposit is medium stiff. One grain size analysis conducted on a sample of the deposit indicated the material is classified as A-6 under the AASHTO Soil Classification System and CL under the USCS. The natural water content of the sample tested was approximately 37 percent.

5.3 Glacial Till

Glacial till was encountered in the borings underlying the Fill layer and Glaciomarine Deposit. The thickness of the Glacial Till encountered was approximately 20 to 21 feet. The Glacial Till varied from:

- Grey to dark grey, Gravelly SAND, trace to some silt;
- Grey SAND, little silt, little gravel;
- Grey, Silty, fine SAND, little rock fragments;
- Grey, Silty SAND, little gravel, occasional cobble; to
- Grey GRAVEL and COBBLES.

Corrected SPT N-values from successful SPT tests within the Glacial Till ranged from 28 to greater than 100 bpf indicating the deposit is medium dense to very dense in consistency. Three grain size analyses performed on samples recovered from the deposit resulted in the soils being classified as A-4 and A-1-b under the AASHTO Soil Classification System and SM under the USCS. The natural water contents of the samples tested ranged from approximately 9 to 11 percent.

5.4 Bedrock

Bedrock was encountered and cored in borings BB-LCS-101 and BB-LCS-102A. The table below summarizes 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)
BB-LCS-101	13+62.9	5.2 Lt	38.9	102.1	100, 100
BB-LCS-102A	14+56.8	7.4 Rt	48.0	93.6	62, 100

The bedrock of the site consisted of black and white coarse-grained, very hard, fresh, massive riebeckite SYENITE of the Litchfield Pluton. The RQD of the bedrock cores ranged from 62 to 100 percent, corresponding to a rock quality of fair to excellent.

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

5.5 Groundwater

Groundwater was measured at depths ranging from 8 to 12 feet below the roadway surface 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

A steel girder bridge and a concrete NEBT bridge, both supported on pile-supported integral abutments, were considered as bridge replacement alternatives. Based on the greater weight and beam depth of the concrete beam alternative, the preferred alternative is a steel welded plate girder bridge supported on integral abutments on driven H-piles.

7.0 GEOTECHNICAL DESIGN CONSIDERATIONS AND RECOMMENDATIONS

The following sections provide geotechnical design considerations and recommendations for H-pile supported integral abutments which is the proposed substructure type for the Babcock 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 may be HP 14x89 or 14x117 depending on the factored design axial loads and ability to resist lateral loads. H-piles 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 Bedrock Elevation (feet)	Estimated Pile Lengths[1] (feet)
Abutment No. 1	132.5	102.1	33
Abutment No. 2	132.5	93.6	41

The estimated pile lengths in the table above 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.

Per AASHTO LRFD Bridge Design Specifications 9th Edition (LRFD) Article 6.5.4.2, at the strength limit state, the axial resistance factor ϕ_c = 0.50 (severe driving conditions) shall be applied to the structural compressive resistance of the pile. Since the H-piles will be subjected to lateral loading, the piles shall also be checked for combined axial compression and flexure as prescribed in LRFD Articles 6.9.2.2 and 6.15.2. This design axial load may govern the design. Per LRFD Article 6.5.4.2, at the strength limit state, the axial resistance factor ϕ_c = 0.70 and the flexural resistance factor ϕ_f = 1.0 shall be applied to the combined axial and flexural resistance of the pile in the interaction equation (LRFD Eq. 6.9.2.2-1 or -2). H-piles shall also be analyzed for fixity using LPile® v2016 (LPile) software, or similar.

[1] Estimated pile lengths include 2-foot embedment into the pile cap.

7.1.1 Axial Pile Resistance – Strength Limit State

Structural Resistance. Preliminary estimates of the factored structural axial resistance of two H-pile sections were 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, ϕ_c = 0.50, for severe driving conditions. It is the responsibility of the structural engineer to calculate the factored axial structural compressive resistances based on the lengths of the upper and lower unbraced pile segments, as determined from LPile, using a resistance factor of ϕ_c = 0.70 for combined axial and bending and appropriate effective length factors (K). These resistances may be the controlling values.

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

Drivability Analyses. Drivability analyses were performed 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. To mitigate the risk of piles “walking” due to potentially sloping bedrock, drivability analyses were performed using a limiting driving stress of 40 ksi. The drivability resistances were calculated using the resistance factor, $\varphi_{dyn,}$ of 0.65, for a single pile in axial compression when a dynamic test is performed as specified in LRFD Table 10.5.5.2.3-1.

A summary of the calculated factored axial compressive structural, geotechnical, and drivability resistances of driven H-piles at the strength limit states are summarized below.

Strength Limit State Factored Axial Pile Resistance				
Pile Section	Structural Resistance[1] ϕ_c=0.50 (kips)	Controlling Geotechnical Resistance[2] ϕ_c=0.50 (kips)	Drivability Resistance[3] φ_{dyn} = 0.65 (kips)	Governing Axial Pile Resistance (kips)
HP 14 x 89	652	652	312	312
HP 14 x 117	860	860	429[4]	429[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 table above.

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 ϕ = 1.0 should be used in accordance with LRFD Article 10.5.5.1.

[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 ϕ_c = 0.70 for combined axial loading and bending. These resistances may be the controlling values.

[2] 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 40 ksi. These theoretical pile resistances may not be achievable if piles walk out of position before reaching the specified driving criteria. 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 Delmag D19-42.

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 two (2) H-pile sections for the service and extreme limit states are summarized below.

Service and Extreme Limit State Factored Axial Pile Resistance				
Pile Section	Structural Resistance[1] $\phi = 1.0$ (kips)	Controlling Geotechnical Resistance[2] $\phi = 1.0$ (kips)	Drivability Resistance[3] $\phi = 1.0$ (kips)	Governing Axial Pile Resistance (kips)
HP 14 x 89	1,305	1,305	480	480
HP 14 x 117	1,720	1,720	660[4]	660[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. 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.

[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] 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 40 ksi. These theoretical pile resistances may not be achievable if piles walk out of position before reaching the specified driving criteria.

[4] Drivability resistance based on a Delmag D19-42.

7.1.3 Lateral Pile Resistance/Behavior

In accordance with LRFD Article 6.15.1, the structural analysis of pile groups subjected to lateral loads shall include explicit consideration of soil-structure interaction effects as specified in LRFD Article 10.7.3.12. Assumptions regarding a fixed or pinned condition at the pile tip should be also confirmed with soil-structure interaction analyses.

A series of lateral pile resistance analyses should be performed to evaluate pile behavior at the abutments using LPile, or similar, 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.

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

LPile Input Parameters Abutment No. 1						
Soil Layer	Soil/Rock Model	Top Elevation of Layer (ft)	Layer Thickness (ft)	γ_e[1] (pcf)	ϕ'[2] (deg) / S_u[3] (psf)	k_s[4] (pci) / ε_{50}[5]
Pre-auger (Predrill) Alternative						
Granular Borrow	Reese Sand	142	10	125	32°	80
Predrill Backfill	Reese Sand	132	10	58	30°	40
Glacial Till	Reese Sand	122	8	68	36°	100
Glacial Till	Reese Sand	114	7	68	38°	120
Glacial Till	Reese Sand	107	5	73	40°	150
Driven Pile without Pre-auger Alternative						
Granular Borrow	Reese Sand	142	10	125	32°	80
Fill	Reese Sand	132	4	63	32°	50
Rock Fill	Reese Sand	128	6	68	38°	110
Glacial Till	Reese Sand	122	8	68	36°	100
Glacial Till	Reese Sand	114	7	68	38°	120
Glacial Till	Reese Sand	107	5	73	40°	150

[1] Effective Unit Weight.
[2] Effective Internal angle of friction.
[3] Undrained shear strength.
[4] Soil modulus constant.
[5] Strain at 50 percent of the ultimate stress.

LPile Input Parameters Abutment No. 2						
Soil Layer	Soil/Rock Model	Top Elevation of Layer (ft)	Layer Thickness (ft)	γ_e[1] (pcf)	ϕ'[2] (deg) / S_u[3] (psf)	k_s[4] (pci) / ε_{50}[5]
Pre-auger (Predrill) Alternative						
Granular Borrow	Reese Sand	142	10	125	32°	80
Predrill Backfill	Reese Sand	132	17	58	30°	40
Glaciomarine Deposit	Matlock Clay	116	3	48	600	0.010
Glacial Till	Reese Sand	113	19	68	38°	120
Driven Pile without Pre-auger Alternative						
Granular Borrow	Reese Sand	142	10	125	32°	80
Rock Fill	Reese Sand	132	16	68	38°	110
Glaciomarine Deposit	Matlock Clay	116	3	48	600	0.010
Glacial Till	Reese Sand	113	19	68	38°	120

7.1.4 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.

[1] Effective Unit Weight.
[2] Effective Internal angle of friction.
[3] Undrained shear strength.
[4] Soil modulus constant.
[5] Strain at 50 percent of the ultimate stress.

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

7.2 Integral Abutment and Wingwall Design

Integral abutment sections shall be designed for all relevant strength, service, and extreme limit states and load combinations specified in LRFD Articles 3.4.1 and 11.5.5. A resistance factor (ϕ) of 1.0 shall be used to assess abutment design at the service limit state, including: settlement and excessive horizontal movement. The overall stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, ϕ, of 0.65. Resistance factors for extreme limit state shall be taken as 1.0.

The designer may assume Soil Type 4 (MaineDOT Bridge Design Guide (BDG) Section 3.6.1) for abutment backfill material soil properties. The backfill properties are as follows:

- Internal Friction Angle (ϕ) = 32°
- Total Unit Weight (γ) = 125 pcf
- Soil-Concrete Interface Friction Angle (δ) = 20°

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. Using MassDOT LRFD Bridge Design Manual Figure 3.10.8-1, a lateral earth pressure coefficient of 5.28 is recommended, assuming a ratio of thermal expansion to abutment height (δ/H) of 0.01 and a level backfill. If the calculated ratio of lateral movement to wall height is less than or greater than 0.01, a passive earth pressure coefficient can be estimated using MassDOT Figure 3.10.8-1. This figure is reproduced in Appendix D – Calculations. A load factor for passive earth pressure is not specified in LRFD. For purposes of the integral abutment backwall reinforcing steel design, use a maximum load factor (γ_{EH}) of 1.50 to calculate factored passive earth pressures.

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

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

In-line wingwalls shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to an equivalent height of soil of 2.0 feet. An at-rest earth pressure coefficient, K_o, of 0.47 should be used for live load surcharge loads placed upon wingwalls cantilevered off of abutments with the top of the wall restrained from movement.

7.3 Abutment Sections

The abutment design shall include a drainage system behind the abutment to intercept any groundwater. Drainage behind the structure shall be in accordance with MaineDOT BDG Section 5.4.2.13.

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 and Embankment Stability

The bridge approach embankments will be constructed using granular borrow placed over medium dense fills and dense native soil deposits. Conventional earth fill embankments constructed over dense, competent soils, using MaineDOT Standard Specifications, with side slopes of 2H:1V or flatter, are anticipated to satisfy stability requirements. Slopes steeper than 2H:1V should be treated with riprap using MaineDOT standard details.

Settlement of the steel H-piles bearing on bedrock will be limited to elastic compression of the piles and is anticipated to be minimal.

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, Litchfield and West Gardiner have a design freezing index (DFI) of approximately 1600 F-degree days. The anticipated coarse-grained fill soil was assigned a water content of 10%. These components correlate to a frost depth of 7.1 feet. It is recommended that any foundation bearing on soils be embedded 7.1 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 table below:

Parameter	Design Value
Peak Ground Acceleration (PGA)	0.080g
Acceleration Coefficient (A_S)	0.13g
S_{DS} (Period = 0.2 sec)	0.26g
S_{D1} (Period = 1.0 sec)	0.11g
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

8.1 Excavation and Dewatering

The new abutments will be constructed approximately 10 feet behind the heels of the existing abutment footings and will require pile driving. The contractor shall be responsible for excavating the existing substructures and concrete approach slabs in their entirety.

Any loose, soft or unsuitable soil encountered at the subgrade level for both abutments shall be excavated in its entirety and replaced with Granular Borrow – Material for Underwater Backfill and the exposed subgrade then thoroughly 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.

8.2 Pile Installation

Rock fill was encountered in the borings behind both of the existing abutments. There is potential for difficult driving conditions. A plan note is recommended to require pre-augering (pre-drilling) at each pile location to approximately El. 122 at Abutment No. 1 and El. 115 at Abutment No. 2. Pre-augering can be accomplished with continuous flight-augers or drilling tools operated by a Kelly bar.

We recommend the following procedure: (1) excavate to the approximate subgrade for the integral abutment, (2) drill a minimum 20-inch diameter hole to approximately El. 122 and 115 at Abutments No. 1 and 2, respectively, (3) place temporary casing to prevent hole cave-in, (4) place sand or pea stone in the excavation, and (5) drive the piles.

Alternative methods to remove boulders, cobbles and rocks that conflict with installing the piles include conventional excavation methods, spudding, use of rock chisels or down-hole hammers.

9.0 CLOSURE

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

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

It is recommended that a geotechnical engineer be provided the opportunity for a review of the final design and specifications in order that the earthwork and foundation recommendations and construction considerations presented in this report are properly interpreted and implemented in the design and specifications.

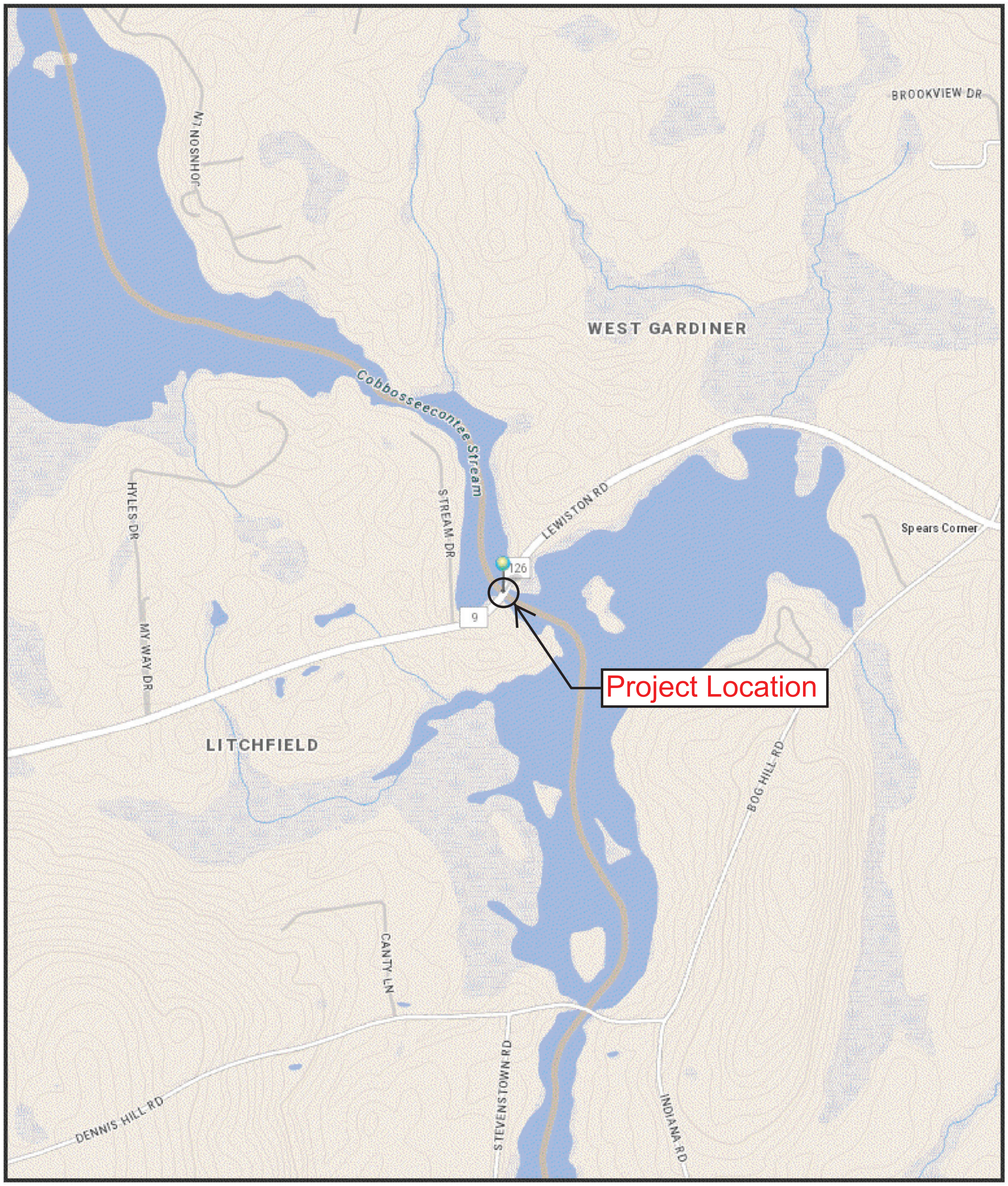
Sheets

Date:8/30/2021

Division: GEOTECH

Username: terry.white

LITCHFIELD-WEST GARDINER, MAINE



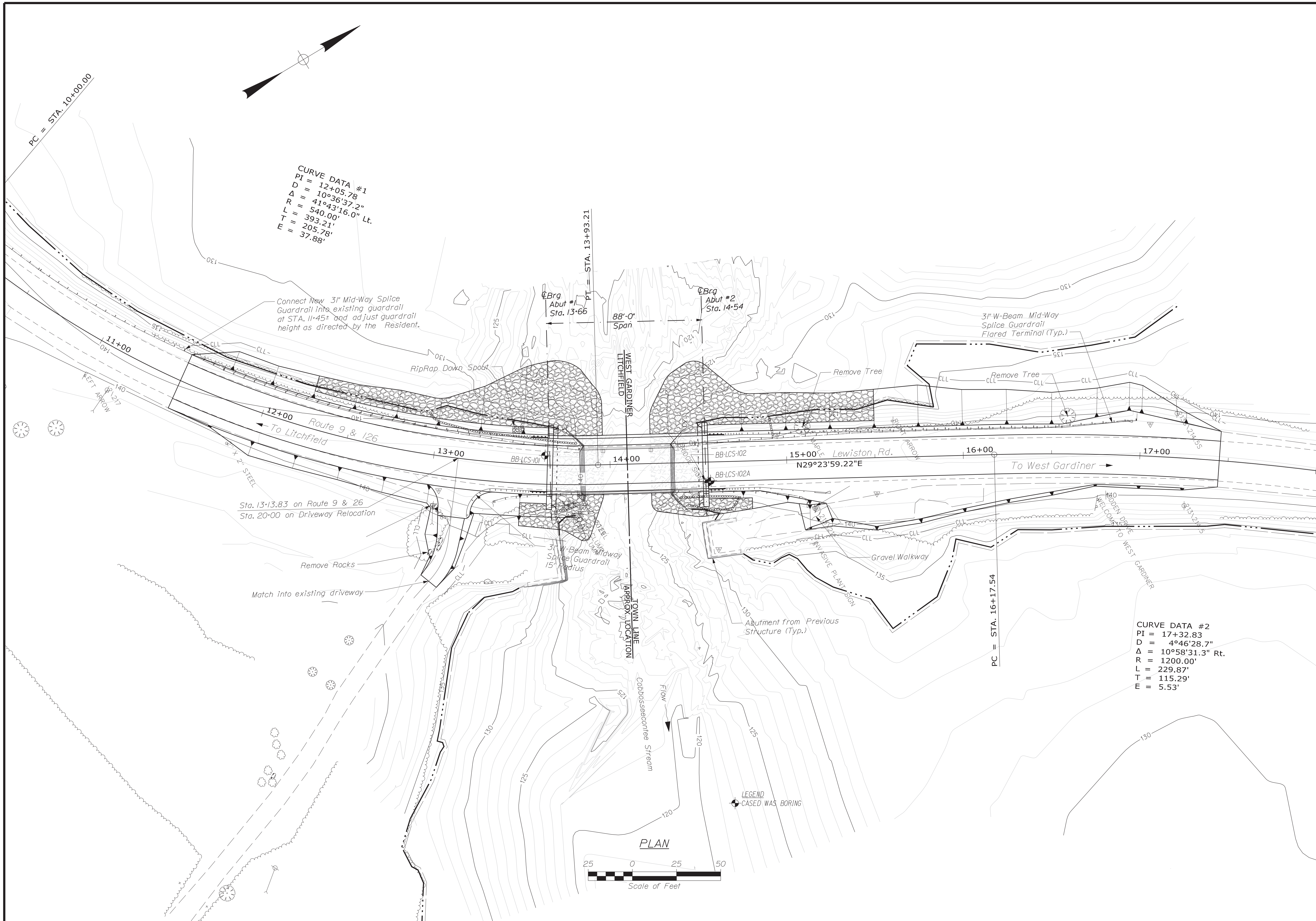
The Maine Department of Transportation provides this publication for information only. Reliance upon this information is at user risk. It is subject to revision and may be incomplete depending upon changing conditions. The Department assumes no liability if injuries or damages result from this information. This map is not intended to support emergency dispatch.

0.25 Miles
1 inch = 0.28 miles

Date: 8/30/2021
Time: 10:49:12 AM

SHEET NUMBER 1 OF 4	BABCOCK BRIDGE COBBOSSECONTEE STREAM LITCHFIELD-WEST GARDINER KENNEBEC COUNTY	STATE OF MAINE DEPARTMENT OF TRANSPORTATION
		2309400
	LOCATION MAP	WIN 23094.00 BRIDGE PLANS

Filename: ...\00\GEOTECH\MSTA\005_BLP1.dgn Division: GEOTECH Username: terry.white Date:1/18/2022



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

02309401

WIN 023094.01

BRIDGE PLANS

BRIDGE NO. 2029

		BY	DATE
PROJ. MANAGER	DEVAN EATON		
DESIGN-DETAILED	K. NASH	J.BRUNELLE	12/21
CHECKED-REVIEWED	M.MANAHAN	T. WHITE	JAN 2022
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

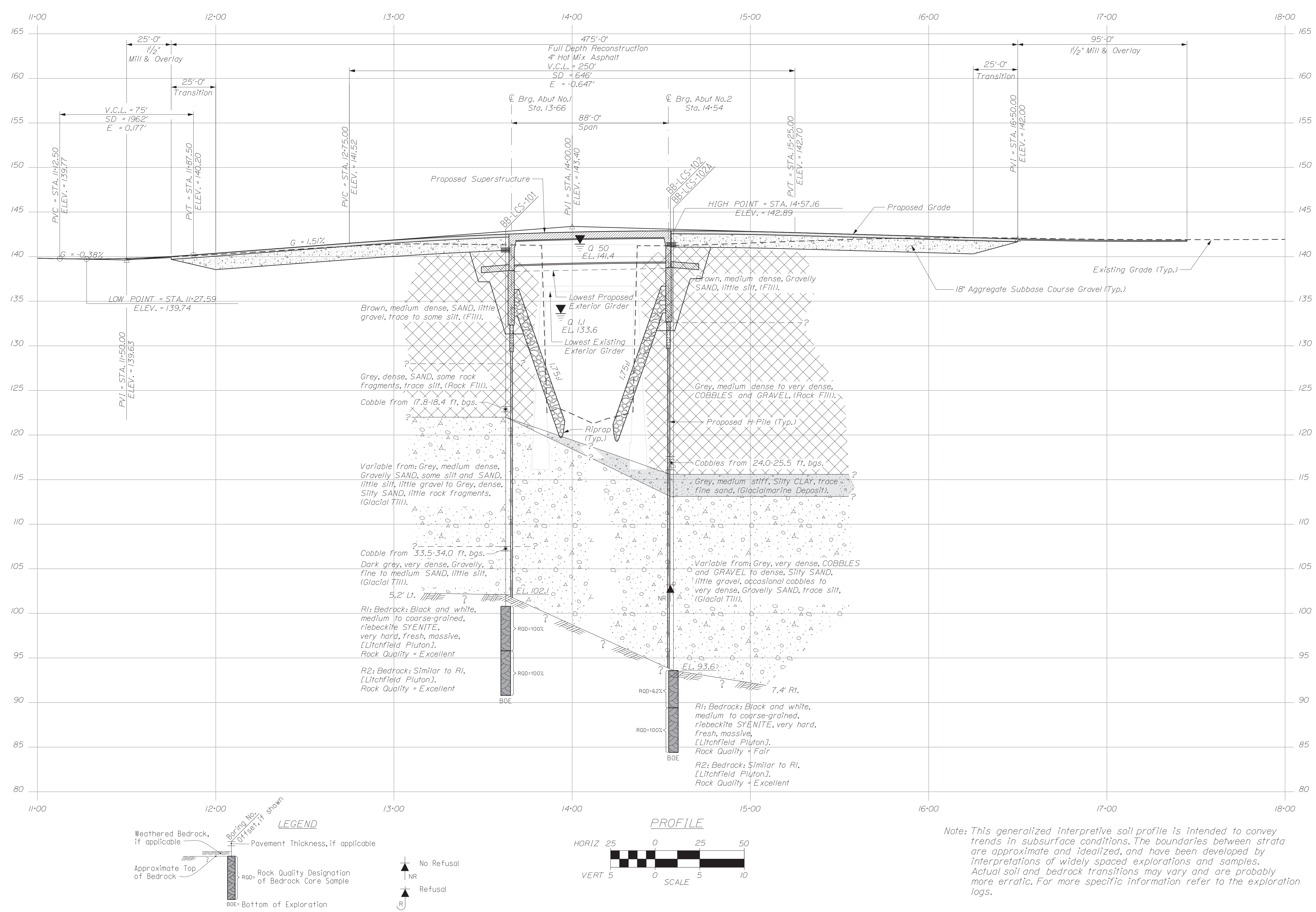
P.E. NUMBER

DATE

BABCOCK BRIDGE
COBBOSSEECONTEE STREAM
LITCHFIELD-WEST GARDINER KENNEBEC COUNTY

BORING LOCATION PLAN

SHEET NUMBER
2
OF 4



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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

02309401

BRIDGE PLANS

WIN 023094.01

BRIDGE NO. 2029

	BY	DATE
PROJ. MANAGER	DEVAN EATON	
DESIGN-DETAILED	K. NASH	J.BRUNELLE	12\21
CHECKED-REVIEWED	-----	-----
DESIGN2-DETAILED2	MANAHAN	T. WHITE	JAN 2022
DESIGN3-DETAILED3	-----	-----
REVISIONS 1	-----	-----
REVISIONS 2	-----	-----
REVISIONS 3	-----	-----
REVISIONS 4	-----	-----
FIELD CHANGES	-----	-----

SIGNATURE

P.E. NUMBER

DATE

BABCOCK BRIDGE
COBBOSSEECONTEE STREAM
LITCHFIELD-WEST GARDINER KENNEBEC COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
3
OF 4

Date:2/11/2022 Username: terry.white Division: GEOTECH Filename: ...\MSTA\007_BORING LOGS1.dgn

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

Project: Babcock Bridge #2029 carries Route 126 over Cobbosseecontee
Location: Litchfield-West Gardiner, Maine

Boring No.: BB-LCS-101
WIN: 23094.00

Driller:	MaineDOT	Elevation (ft.)	141.0	Auger ID/OD:	5" Solid Stem
Operator:	Daggett/Niles	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/24-25/2019	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	13+62.9, 5.2 ft Lt.	Casing ID/OD:	NW-3"	Water Level*:	9.5 ft bgs.
Hammer Efficiency Factor: 0.886		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)
0							SSA	140.5
5	1D	24/16	5.00 - 7.00	3/5/13/11	18	27		
10	2D	24/9	10.00 - 12.00	4/5/10/9	15	22	34	
							45	
							52	
							53	128.0
							51	
15	3D	24/12	15.00 - 17.00	15/18/13/6	31	46	31	
							138	
							182	
							170	
							32	122.0
20	4D	24/14	20.00 - 22.00	13/13/6/7	19	28	28	
							34	
							40	
							76	
							62	
25	5D	24/14	25.00 - 27.00	12/9/10/12	19	28	23	
							36	
							53	
							50	
							56	
30	6D	24/4	30.00 - 32.00	16/13/16/22	29	43	32	
							43	
							89	
							177	107.5
							75	
35	7D	24/15	35.00 - 37.00	15/20/20/26	40	59	37	
							38	
							61	
							a126	
							RC	102.1
40	R1	60/60	40.20 - 45.20	RQD = 100%			NQ-2	
45	R2	60/60	45.20 - 50.20	RQD = 100%				
50								90.8

Visual Description and Remarks / Laboratory Testing Results/AASHTO and Unified Class:

- 6" HMA. —0.5
- Brown, damp, medium dense, fine to coarse SAND, little gravel, some silt, (Fill). — G#337366, A-2-4, SM, WC=9.3%
- Brown, wet, medium dense, fine to coarse SAND, little gravel, trace silt, (Fill).
- —13.0
- Grey, wet, dense, fine to coarse SAND, some rock fragments, trace silt, (Rock Fill).
- Cobble from 17.8-18.4 ft bgs.
- —19.0
- Grey, wet, medium dense, Gravelly, fine to coarse SAND, some silt, (Glacial Till). — G#337367, A-1-b, SM, WC=9.7%
- Grey, wet, medium dense, fine to coarse SAND, little silt, little gravel, (Glacial Till).
- Grey, wet, dense, Silty fine SAND, little rock fragments, (Glacial Till).
- —33.5
- Cobble from 33.5-34.0 ft bgs.
- Roller Coned ahead from 33.5-35.0 ft bgs.
- Dark grey, wet, very dense, Gravelly fine to medium SAND, little silt, (Glacial Till), (rock fragments from cobble). — G#337368, A-1-b, SM, WC=9.2%
- a126 blows for 0.9 ft.
- —38.9
- Top of Bedrock at Elev. 102.1 ft.
- Roller Coned ahead to 40.2 ft bggs.
- R1: Bedrock: Black and white, medium to coarse-grained, riebeckite SYENITE, very hard, fresh, massive. [Litchfield Pluton]
  Rock Quality: Excellent.
  R1: Core Times (min:sec)
  40.2-41.2 ft (3:22)
  41.2-42.2 ft (3:13)
  42.2-43.2 ft (3:44)
  43.2-44.2 ft (3:32)
  44.2-45.2 ft (4:00)
  100% Recovery
- R2: Bedrock: Similar to R1.
  [Litchfield Pluton]
  Rock Quality = Excellent
  R2: Core Times (min:sec)
  45.2-46.2 ft (2:11)
  46.2-47.2 ft (2:10)
  47.2-48.2 ft (2:19)
  48.2-49.2 ft (2:33)
  49.2-50.2 ft (2:25)
  100% Recovery
- —50.2
- Bottom of Exploration at 50.2 feet below ground surface.

Remarks:

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

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

Page 1 of 1

Boring No.: BB-LCS-101

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

Project: Babcock Bridge #2029 carries Route 126 over Cobbosseecontee
Location: Litchfield-West Gardiner, Maine

Boring No.: BB-LCS-102
WIN: 23094.00

Driller:	MaineDOT	Elevation (ft.)	141.6	Auger ID/OD:	5" Solid Stem
Operator:	Daggett/Aaron/Niles	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/3/2019, 10/7/2019	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	14+55.1, 11.7 ft Rt.	Casing ID/OD:	HW-4"/NW-3"	Water Level*:	8.0 ft bgs.
Hammer Efficiency Factor: 0.886		Hammer Type:	Automatic ☐ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)
0							SSA	141.1
5	1D	24/17	5.00 - 7.00	2/2/7/9	9	13		
								132.6
10	2D	24/5	10.00 - 12.00	7/18/14/12	32	47	36	
							70	
							62	
							101	
							53	
15	3D	24/4	15.00 - 17.00	4/11/3/3	14	21	59	
							74	
							69	
							71	
	MD	24/0	19.00 - 21.00	9/19/10/9	29	43	48	
20							93	
							166	
							159	
							141	
							38	
25							74	115.6
	4D	24/20	26.00 - 28.00	5/2/2/3	4	6	48	
							29	
							133	113.1
							12	
30	MD	18/0	30.00 - 31.50	5/14/55	69	102	31	
							52	
							250	
							48	
	5D	24/16	34.00 - 36.00	13/12/18/18	30	44	110	
35							36	
							70	
							45	
							a40	103.2

Visual Description and Remarks / Laboratory Testing Results/AASHTO and Unified Class:

- 5.5" HMA. —0.5
- Brown, damp, medium dense, Gravelly, fine to coarse SAND, little silt, (Fill). — G#337370, A-1-b, SM, WC=7.5%
- —9.0
- Grey, wet, very dense, COBBLES and GRAVEL, (Rock Fill).
- Similar to above, except medium dense, (Rock Fill).
- Similar to above in wash water, dense, (Rock Fill).
- Cobbles from 24.0-25.5 ft bgs.
- —26.0
- Grey, wet, medium stiff, Silty CLAY, trace fine sand, (Glaciomarine Deposit). — G#337369, A-6, CL, WC=36.5%, LL=35, PL=21, PI=14
- —28.5
- Cobbles and Gravel.
- Set in NW Casing at 30.0 ft bgs.
- Grey, wet, dense, Silty fine to coarse SAND, little gravel, occasional cobble, (Glacial Till). — G#337371, A-4, SM, WC=10.7%
- Roller Coned ahead to 38.4 ft bgs.
- Cobble from 36.7-37.5 ft bgs.
- a40 blows for 0.4 ft.
- —38.4
- Bottom of Exploration at 38.4 feet below ground surface. Broke NW Casing at 38.4 ft bgs.

Remarks:

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

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

Page 1 of 1

Boring No.: BB-LCS-102

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

Project: Babcock Bridge #2029 carries Route 126 over Cobbosseecontee
Location: Litchfield-West Gardiner, Maine

Boring No.: BB-LCS-102A
WIN: 23094.00

Driller:	MaineDOT	Elevation (ft.)	141.6	Auger ID/OD:	5" Solid Stem
Operator:	Daggett/Niles	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/22,24/2019	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	14+56.8, 10.7 ft Rt.	Casing ID/OD:	HW-4"/NW-3"	Water Level*:	12.0 ft bgs.
Hammer Efficiency Factor: 0.886		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)
0							SSA	
10							DROVE	
	1D	12/6	39.00 - 40.00	75/60	---			102.6
45	2D	24/16	45.00 - 47.00	35/32/22/27	54	80		
	R1	50.4/50.4	48.00 - 52.20	RQD = 62%			NQ-2	93.6
	R2	60/60	52.20 - 57.20	RQD = 100%				
								84.4

Visual Description and Remarks:

- No soil samples retrieved. See BB-LCS-102 for material descriptions.
- Drove HW Casing to 34.0 ft bgs.
- —39.0
- Grey, wet, very dense, GRAVEL and COBBLES.
- Grey, wet, very dense, Gravelly fine to coarse SAND, trace silt, (Glacial Till).
- Roller Coned ahead to 48.0 ft bgs.
- —48.0
- Bedrock at Elev. 93.6 ft.
- R1: Bedrock: Black and white, medium to coarse-grained, riebeckite SYENITE, hard, fresh, massive. [Litchfield Pluton]
  Rock Quality = Fair
  R1: Core Times (min:sec)
  48.0-49.0 ft (1:17)
  49.0-50.0 ft (1:32)
  50.0-51.0 ft (1:55)
  51.0-52.0 ft (1:54)
  52.0-52.2 ft (1:00) Core Blocked
  100% Recovery
- R2: Bedrock: Similar to R1.
  [Litchfield Pluton]
  Rock Quality = Excellent
  R2: Core Times (min:sec)
  52.2-53.2 ft (1:37)
  53.2-54.2 ft (1:55)
  54.2-55.2 ft (1:52)
  55.2-56.2 ft (1:59)
  56.2-57.2 ft (2:05)
  100% Recovery
- —57.2
- Bottom of Exploration at 57.2 feet below ground surface.

Remarks:

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

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

Page 1 of 1

Boring No.: BB-LCS-102A

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
02309401
WIN 023094.01
BRIDGE PLANS
BRIDGE NO. 2029

PROJ. MANAGER	DEVAN EATON	BY	DATE
DESIGN-DETAILED	K. NASH	J.BRUNELLE	12\21
CHECKED-REVIEWED	-----	-----	-----
DESIGN2-DETAILED2	J.MANAHAN	T.WHITE	FEB 2022
DESIGN3-DETAILED3	-----	-----	-----
REVISIONS 1	-----	-----	-----
REVISIONS 2	-----	-----	-----
REVISIONS 3	-----	-----	-----
REVISIONS 4	-----	-----	-----
FIELD CHANGES	-----	-----	-----

SIGNATURE
P.E. NUMBER
DATE

BABCOCK BRIDGE
COBBOSSEECONTEE STREAM
LITCHFIELD-WEST GARDINER KENNEBEC COUNTY

BORING LOGS

SHEET NUMBER 4 OF 4

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations (in this order, if applicable):

Color (Munsell color chart)
Moisture (dry, damp, moist, wet)
Density/Consistency (from above right hand side)
Texture (fine, medium, coarse, etc.)
Name (Sand, Silty Sand, Clay, etc., including portions - trace, little, etc.)
Gradation (well-graded, poorly-graded, uniform, etc.)
Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)
Structure (layering, fractures, cracks, etc.)
Bonding (well, moderately, loosely, etc.,)
Cementation (weak, moderate, or strong)
Geologic Origin (till, marine clay, alluvium, etc.)
Groundwater level

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

MODIFIED BURMISTER SYSTEM

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

TERMS DESCRIBING DENSITY/CONSISTENCY

Coarse-grained soils (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) Silty or Clayey gravels; and (3) Silty, Clayey or Gravelly sands. Density is rated according to standard penetration resistance (N-value).

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

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

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

Rock Quality Designation (RQD):

$$\text{RQD (\%)} = \frac{\text{sum of the lengths of intact pieces of core* > 4 inches}}{\text{length of core advance}}$$

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

Rock Quality Based on RQD

Rock Quality	RQD (%)
Very Poor	≤25
Poor	26 - 50
Fair	51 - 75
Good	76 - 90
Excellent	91 - 100

Desired Rock Observations (in this order, if applicable):

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Rock Type (granite, schist, sandstone, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
- -dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.)
- -spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet)
- -tightness (tight, open, or healed)
- -infilling (grain size, color, etc.)

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock quality (very poor, poor, etc.)
ref: ASTM D6032 and FHWA NHI-16-072 GEC 5 - Geotechnical Site Characterization, Table 4-12
Recovery (inch/inch and percentage)
Rock Core Rate (X.X ft - Y.Y ft (min:sec))

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Babcock Bridge #2029 carries Route 126 over Cobbosseecontee Stream
Location: Litchfield-West Gardiner, Maine

Boring No.: BB-LCS-101
WIN: 23094.00

Driller:	MaineDOT	**Elevation (ft.)**	141.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/24-25/2019	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	13+62.9, 5.2 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	9.5 ft bgs.
Hammer Efficiency Factor:	0.886	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	140.5		6" HMA. 0.5	
5	1D	24/16	5.00 - 7.00	3/5/13/11	18	27				Brown, damp, medium dense, fine to coarse SAND, little gravel, some silt, (Fill).	G#337366 A-2-4, SM WC=9.3%
10	2D	24/9	10.00 - 12.00	4/5/10/9	15	22	34			Brown, wet, medium dense, fine to coarse SAND, little gravel, trace silt, (Fill).	
							45				
							52				
							53	128.0		13.0	
							51				
15	3D	24/12	15.00 - 17.00	15/18/13/6	31	46	31			Grey, wet, dense, fine to coarse SAND, some rock fragments, trace silt, (Rock Fill).	
							138				
							182			Cobble from 17.8-18.4 ft bgs.	
							170				
							32	122.0		19.0	
20	4D	24/14	20.00 - 22.00	13/13/6/7	19	28	28			Grey, wet, medium dense, Gravelly, fine to coarse SAND, some silt, (Glacial Till).	G#337367 A-1-b, SM WC=9.7%
							34				
							40				
							76				
25							62				

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Babcock Bridge #2029 carries Route 126 over Cobbossecontee Stream
Location: Litchfield-West Gardiner, Maine

Boring No.: BB-LCS-101
WIN: 23094.00

Driller:	MaineDOT	**Elevation (ft.)**	141.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/24-25/2019	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	13+62.9, 5.2 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	9.5 ft bgs.
Hammer Efficiency Factor:	0.886	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/14	25.00 - 27.00	12/9/10/12	19	28	23			Grey, wet, medium dense, fine to coarse SAND, little silt, little gravel, (Glacial Till).	
							36				
							53				
							50				
							56				
30	6D	24/4	30.00 - 32.00	16/13/16/22	29	43	32			Grey, wet, dense, Silty fine SAND, little rock fragments, (Glacial Till).	
							43				
							89				
							177	107.5		33.5	
							75			Cobble from 33.5-34.0 ft bgs. Roller Coned ahead from 33.5-35.0 ft bgs.	
35	7D	24/15	35.00 - 37.00	15/20/20/26	40	59	37			Dark grey, wet, very dense, Gravelly fine to medium SAND, little silt, (Glacial Till), (rock fragments from cobble).	G#337368 A-1-b, SM WC=9.2%
							38				
							61				
							a126			a126 blows for 0.9 ft.	
							RC	102.1		38.9 Top of Bedrock at Elev. 102.1 ft. Roller Coned ahead to 40.2 ft bggs.	
40	R1	60/60	40.20 - 45.20	RQD = 100%			NQ-2			R1: Bedrock: Black and white, medium to coarse-grained, riebeckite SYENITE, very hard, fresh, massive. [Litchfield Pluton] Rock Quality: Excellent. R1: Core Times (min:sec) 40.2-41.2 ft (3:22) 41.2-42.2 ft (3:13) 42.2-43.2 ft (3:44) 43.2-44.2 ft (3:32) 44.2-45.2 ft (4:00) 100% Recovery	
45	R2	60/60	45.20 - 50.20	RQD = 100%						R2: Bedrock: Similar to R1. [Litchfield Pluton] Rock Quality = Excellent R2: Core Times (min:sec) 45.2-46.2 ft (2:11) 46.2-47.2 ft (2:10) 47.2-48.2 ft (2:19) 48.2-49.2 ft (2:33) 49.2-50.2 ft (2:25)	
50											

Remarks:

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Babcock Bridge #2029 carries Route 126 over Cobbossecontee Stream Location: Litchfield-West Gardiner, Maine	Boring No.: BB-LCS-101 WIN: 23094.00

Driller:	MaineDOT	Elevation (ft.)	141.0	Auger ID/OD:	5" Solid Stem
Operator:	Daggett/Niles	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/24-25/2019	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	13+62.9, 5.2 ft Lt.	Casing ID/OD:	NW-3"	Water Level*:	9.5 ft bgs.
Hammer Efficiency Factor:	0.886	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140 lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
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WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50							V	90.8		100% Recovery 50.2 **Bottom of Exploration at 50.2 feet below ground surface.**	
55											
60											
65											
70											
75											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Babcock Bridge #2029 carries Route 126 over Cobbossecontee Stream
Location: Litchfield-West Gardiner, Maine

Boring No.: BB-LCS-102
WIN: 23094.00

Driller:	MaineDOT	**Elevation (ft.)**	141.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Aaron/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/3/2019, 10/7/2019	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	14+55.1, 11.7 ft Rt.	**Casing ID/OD:**	HW-4"/NW-3"	**Water Level*:**	8.0 ft bgs.

Hammer Efficiency Factor: 0.886 **Hammer Type:** Automatic ☐ Hydraulic ☐ Rope & Cathead ☐

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	141.1		5½" HMA. 0.5	
5	1D	24/17	5.00 - 7.00	2/2/7/9	9	13				Brown, damp, medium dense, Gravelly, fine to coarse SAND, little silt, (Fill).	G#337370 A-1-b, SM WC=7.5%
								132.6		9.0	
10	2D	24/5	10.00 - 12.00	7/18/14/12	32	47	36			Grey, wet, very dense, COBBLES and GRAVEL, (Rock Fill).	
							70				
							62				
							101				
							53				
15	3D	24/4	15.00 - 17.00	4/11/3/3	14	21	59			Similar to above, except medium dense, (Rock Fill).	
							74				
							69				
							71				
	MD	24/0	19.00 - 21.00	9/19/10/9	29	43	48			Similar to above in wash water, dense, (Rock Fill).	
20							93				
							166				
							159				
							141				
							38			Cobbles from 24.0-25.5 ft bgs.	
25											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Babcock Bridge #2029 carries Route 126 over Cobbossecontee Stream
Location: Litchfield-West Gardiner, Maine

Boring No.: BB-LCS-102
WIN: 23094.00

Driller:	MaineDOT	**Elevation (ft.)**	141.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Aaron/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/3/2019, 10/7/2019	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	14+55.1, 11.7 ft Rt.	**Casing ID/OD:**	HW-4"/NW-3"	**Water Level*:**	8.0 ft bgs.
Hammer Efficiency Factor:	0.886	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☐		

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S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							74			
	4D	24/20	26.00 - 28.00	5/2/2/3	4	6	48	115.6	26.0 Grey, wet, medium stiff, Silty CLAY, trace fine sand, (Glaciomarine Deposit).	G#337369 A-6, CL WC=36.5% LL=35 PL=21 PI=14
							29			
							133	113.1	28.5 Cobbles and Gravel.	
							12			
30	MD	18/0	30.00 - 31.50	5/14/55	69	102	31		Set in NW Casing at 30.0 ft bgs.	
							52			
							250			
							48			
	5D	24/16	34.00 - 36.00	13/12/18/18	30	44	110		Grey, wet, dense, Silty fine to coarse SAND, little gravel, occasional cobble, (Glacial Till).	G#337371 A-4, SM WC=10.7%
35							36			
							70		Roller Coned ahead to 38.4 ft bgs.	
									Cobble from 36.7-37.5 ft bgs.	
							45			
							a40	103.2	a40 blows for 0.4 ft. 38.4	
									Bottom of Exploration at 38.4 feet below ground surface. Broke NW Casing at 38.4 ft bgs.	
40										
45										
50										

Remarks:

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Babcock Bridge #2029 carries Route 126 over Cobbossecontee Stream Location: Litchfield-West Gardiner, Maine	Boring No.: BB-LCS-102A WIN: 23094.00

Driller:	MaineDOT	**Elevation (ft.)**	141.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/22,24/2019	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+56.8, 10.7 ft Rt.	**Casing ID/OD:**	HW-4"/NW-3"	**Water Level*:**	12.0 ft bgs.
Hammer Efficiency Factor:	0.886	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			No soil samples retrieved. See BB-LCS-102 for material descriptions.	
5											
10							DROVE			Drove HW Casing to 34.0 ft bgs.	
15											
20											
25											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Babcock Bridge #2029 carries Route 126 over Cobbossecontee Stream
Location: Litchfield-West Gardiner, Maine

Boring No.: BB-LCS-102A
WIN: 23094.00

Driller:	MaineDOT	**Elevation (ft.)**	141.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/22,24/2019	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+56.8, 10.7 ft Rt.	**Casing ID/OD:**	HW-4"/NW-3"	**Water Level*:**	12.0 ft bgs.
Hammer Efficiency Factor:	0.886	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25											
30											
35											
								102.6		39.0	
	1D	12/6	39.00 - 40.00	75/60	---					Grey, wet, very dense, GRAVEL and COBBLES.	
40											
45	2D	24/16	45.00 - 47.00	35/32/22/27	54	80				Grey, wet, very dense, Gravelly fine to coarse SAND, trace silt, (Glacial Till).	
								93.6		Roller Coned ahead to 48.0 ft bgs. 48.0	
	R1	50.4/50.4	48.00 - 52.20	RQD = 62%			NQ-2			Bedrock at Elev. 93.6 ft. R1: Bedrock: Black and white, medium to coarse-grained, riebeckite SYENITE, hard, fresh, massive.	
50											

Remarks:

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

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

Boring No.: BB-LCS-102A

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Babcock Bridge #2029 carries Route 126 over Cobbossecontee Stream
Location: Litchfield-West Gardiner, Maine

Boring No.: BB-LCS-102A

WIN: 23094.00

Driller:	MaineDOT	**Elevation (ft.)**	141.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/22,24/2019	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+56.8, 10.7 ft Rt.	**Casing ID/OD:**	HW-4"/NW-3"	**Water Level*:**	12.0 ft bgs.
Hammer Efficiency Factor:	0.886	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										[Litchfield Pluton] Rock Quality = Fair R1: Core Times (min:sec) 48.0-49.0 ft (1:17) 49.0-50.0 ft (1:32) 50.0-51.0 ft (1:55) 51.0-52.0 ft (1:54) 52.0-52.2 ft (1:00) Core Blocked 100% Recovery	
	R2	60/60	52.20 - 57.20	RQD = 100%							
55										R2: Bedrock: Similar to R1. [Litchfield Pluton] Rock Quality = Excellent R2: Core Times (min:sec) 52.2-53.2 ft (1:37) 53.2-54.2 ft (1:55) 54.2-55.2 ft (1:52) 55.2-56.2 ft (1:59) 56.2-57.2 ft (2:05) 100% Recovery	
								84.4		57.2	
										Bottom of Exploration at 57.2 feet below ground surface.	
60											
65											
70											
75											

Remarks:

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

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

Appendix B

Rock Core Photographs

Babcock Bridge #2029 Carries Route 126 Over Cobbossecontee Stream
Litchfield, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Penetration (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-LCS-102A	R1	48.00 – 52.20	50.4	50.4	31	62%	SYENITE (LITCHFIELDITE)	1
BB-LCS-102A	R2	52.20 – 57.20	60	60	60	100%	SYENITE (LITCHFIELDITE)	2
BB-LCS-101	R1	40.20 – 45.20	60	60	60	100%	SYENITE (LITCHFIELDITE)	3
BB-LCS-101	R2	45.20 – 50.20	60	60	60	100%	SYENITE (LITCHFIELDITE)	4

See Note 3

Notes: 1. “Box row” indicates the section of the box where the core run is contained: 1 = top, 4 = bottom.
2. Top of core run at left. Increasing depth left to right.
3. 9” segment of rock core removed.

Appendix C

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Litchfield-West Gardiner Work Number: 23094.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification Unified	Classification AASHTO	Classification Frost
BB-LCS-101, 1D	13+62.9	5.2 Lt.	5.0-7.0	337366	1	9.3			SM	A-2-4	II
BB-LCS-101, 4D	13+62.9	5.2 Lt.	20.0-22.0	337367	1	9.7			SM	A-1-b	II
BB-LCS-101, 7D	13+62.9	5.2 Lt.	35.0-37.0	337368	1	9.2			SM	A-1-b	II
BB-LCS-102, 1D	14+55.1	11.7 Rt.	5.0-7.0	337370	1	7.5			SM	A-1-b	II
BB-LCS-102, 4D	14+55.1	11.7 Rt.	26.0-28.0	337369	1	36.5	35	14	CL	A-6	III
BB-LCS-102, 5D	14+55.1	11.7 Rt.	34.0-36.0	337371	1	10.7			SM	A-4	III

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

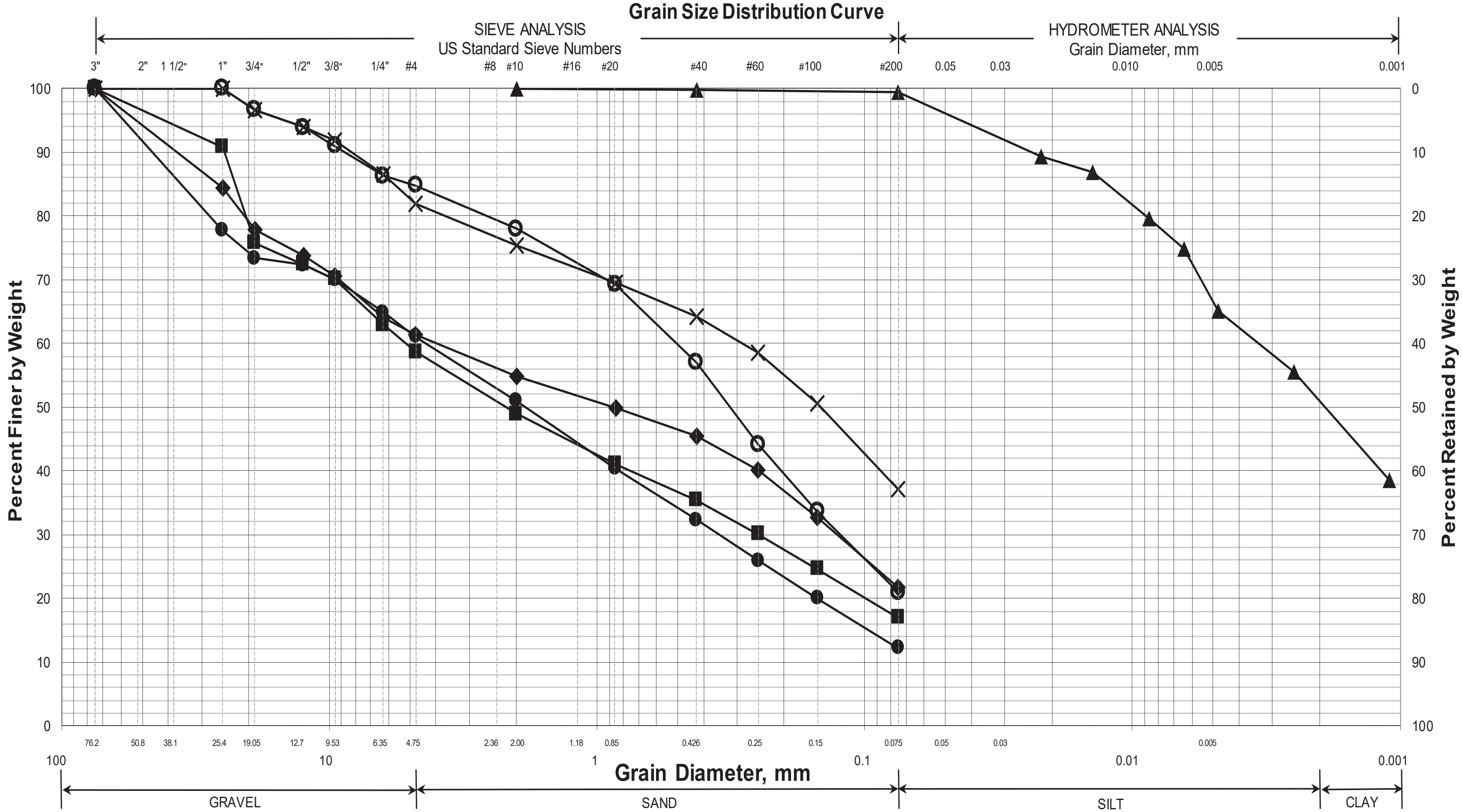
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

Maine Department of Transportation
Grain Size Distribution Curve



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-LCS-101/1D	13+62.9	5.2 LT	5.0-7.0	SAND, some silt, little gravel.	9.3			
◆	BB-LCS-101/4D	13+62.9	5.2 LT	20.0-22.0	Gravelly SAND, some silt.	9.7			
■	BB-LCS-101/7D	13+62.9	5.2 LT	35.0-37.0	Gravelly SAND, little silt.	9.2			
●	BB-LCS-102/1D	14+55.1	11.7 RT	5.0-7.0	Gravelly SAND, little silt.	7.5			
▲	BB-LCS-102/4D	14+55.1	11.7 RT	26.0-28.0	Silty CLAY, trace sand.	36.5	35	21	14
×	BB-LCS-102/5D	14+55.1	11.7 RT	34.0-36.0	Silty SAND, little gravel.	10.7			

WIN	
023094.00	
Town	
Litchfield, West Gardiner	
Reported by/Date	
WHITE, TERRY A	1/18/2022

SHEET 1

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
337369	**BB-LCS-102/4D**	**GEOTECHNICAL (DISTURBED)**	**10/3/2019**	**11/4/2019**

Sample Type: **GEOTECHNICAL** Location: Station: **14+55.1** Offset, ft: **11.7 RT** Dbfg, ft: **26.0-28.0**

WIN/Town **023094.00 - LITCHFIELD, WEST GAR** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	100.0
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	99.8
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	99.4
[0.0220 mm]	89.3
[0.0141 mm]	86.8
[0.0087 mm]	79.6
[0.0064 mm]	74.8
[0.0048 mm]	65.1
[0.0025 mm]	55.5
[0.0011 mm]	38.6

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	35
Plastic Limit (T 90), %	21
Plasticity Index (T 90), %	14
Specific Gravity, Corrected to 20°C (T 100)	2.80
Loss on Ignition, % (T 267)	
Water Content (T 265), %	36.5

Consolidation (T 216)

Trimmings, Water Content, %

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

Vane Shear Test on Shelby Tubes (Maine DOT)

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

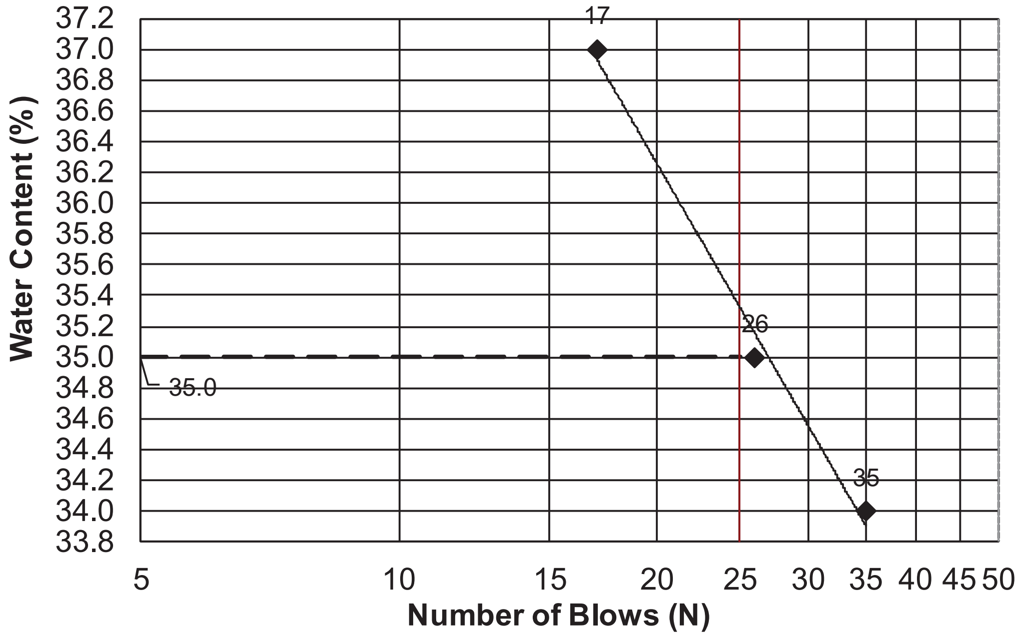
Comments:

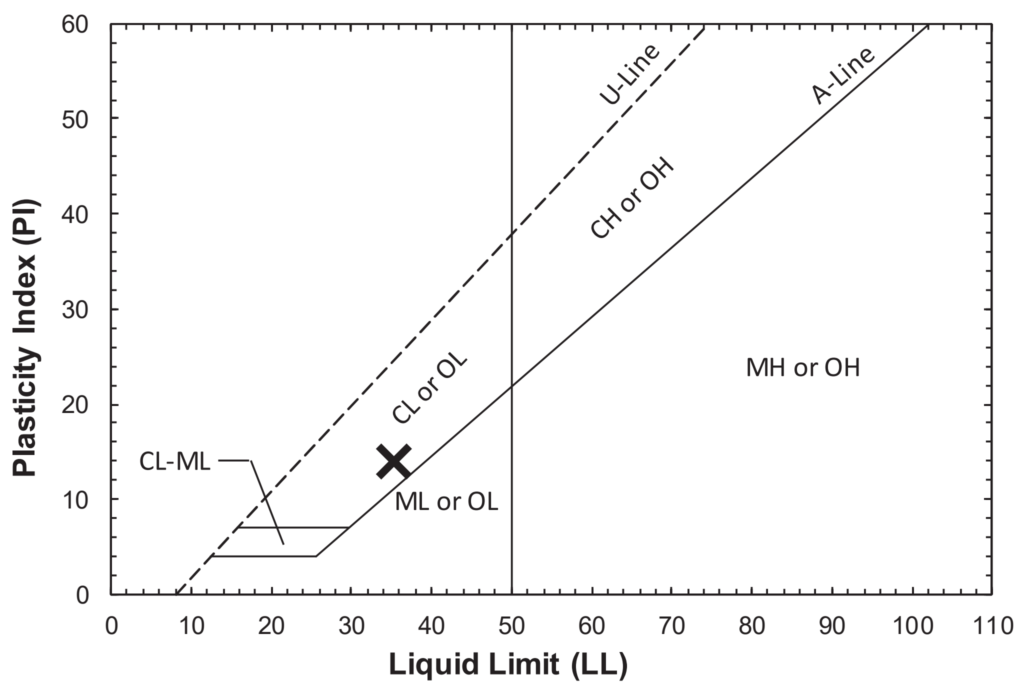
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **11/12/2019**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Litchfield, West Gardiner	Reference No.	337369
WIN	023094.00	Water Content, %	36.5
Sampled	10/3/2019	Liquid Limit @ 25 blows (T 89), %	35
Boring No./Sample No.	BB-LCS-102/4D	Plastic Limit (T 90), %	21
Station	14+55.1	Plasticity Index (T 90), %	14
Depth	26.0-28.0	Tested By	BBURR





Appendix D

Calculations

Liquidity Index

Liquidity Index

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

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

BB-WGSC-102, 4D

$WC := 36.5$

$LL := 35$

$PL := 21$

$$LI := \frac{WC - PL}{LL - PL} = 1.11$$

Driven H-Pile Resistance

Design of H-piles

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

Bedrock Properties

Rock Type: Syenite

ϕ = 34-40 (AASHTO LRFD Table C.10.4.6.4-1);
Syenite Co = 26,000 - 62,000 psi (AASHTO Standard Specifications for Bridges 17th Edition, Table 4.4.8.1.2B)

For Design Purposes, use bedrock data from BB-LCS-102A: RQD = 62% and an Unconfined Compressive Strength of 26,000 psi based on the lower bound of Syenite Co. BB-LCS-102A R1.

Pile Properties

Use the following piles: 14x89, 14x117

$A_s := \begin{pmatrix} 26.1 \\ 34.4 \end{pmatrix} \cdot in^2$ $d := \begin{pmatrix} 13.8 \\ 14.2 \end{pmatrix} \cdot in$ $b := \begin{pmatrix} 14.7 \\ 14.9 \end{pmatrix} \cdot in$

$A_{box} := \overrightarrow{(d \cdot b)}$ $A_{box} = \begin{pmatrix} 202.86 \\ 211.58 \end{pmatrix} \cdot in^2$

Note: All matrices set up in this order
14x89
14x117

r_s= radius of gyration $r_s := \begin{pmatrix} 3.53 \\ 3.59 \end{pmatrix} \cdot in$ radius of gyration about the Y-Y or weak axis per LRFD Article C6.9.4.1.2.

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

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

Use LRFD Equation 6.9.2.1-1 Pr = ϕ Pn

Nominal Axial Structural Resistance

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

$Q := 1.0$ LRFD Article 6.9.4.2

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

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

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

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

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

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

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

LRFD eq. 6.9.4.1.2-1

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

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

LRFD Article 6.9.4.1.1

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

If Pe/Po > or = 0.44, then:

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

LRFD Eq. 6.9.4.1.1-1

then:

this applies to all pile sizes

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

Factored Axial Structural Resistance for the Strength Limit State

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

$\phi_c := 0.5$

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

$P_r := \phi_c \cdot P_n$

Factored structural compressive resistance, P_r

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

LRFD 10.7.3.2.3 - Piles Driven to Hard Rock -

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

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

Nominal Structural Resistance Previously Calculated:

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

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

$\phi_c := 0.5$

$P_r := \phi_c \cdot P_n$

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

14x89
14x117

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

$$\phi_c := 1.0$$

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

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

14x89
14x117

Drivability Analyses

Ref: LRFD Article 10.7.8

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

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

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

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

Limit driving stress to 40 ksi and blow count to 15 blows per inch (bpi).

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

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

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

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

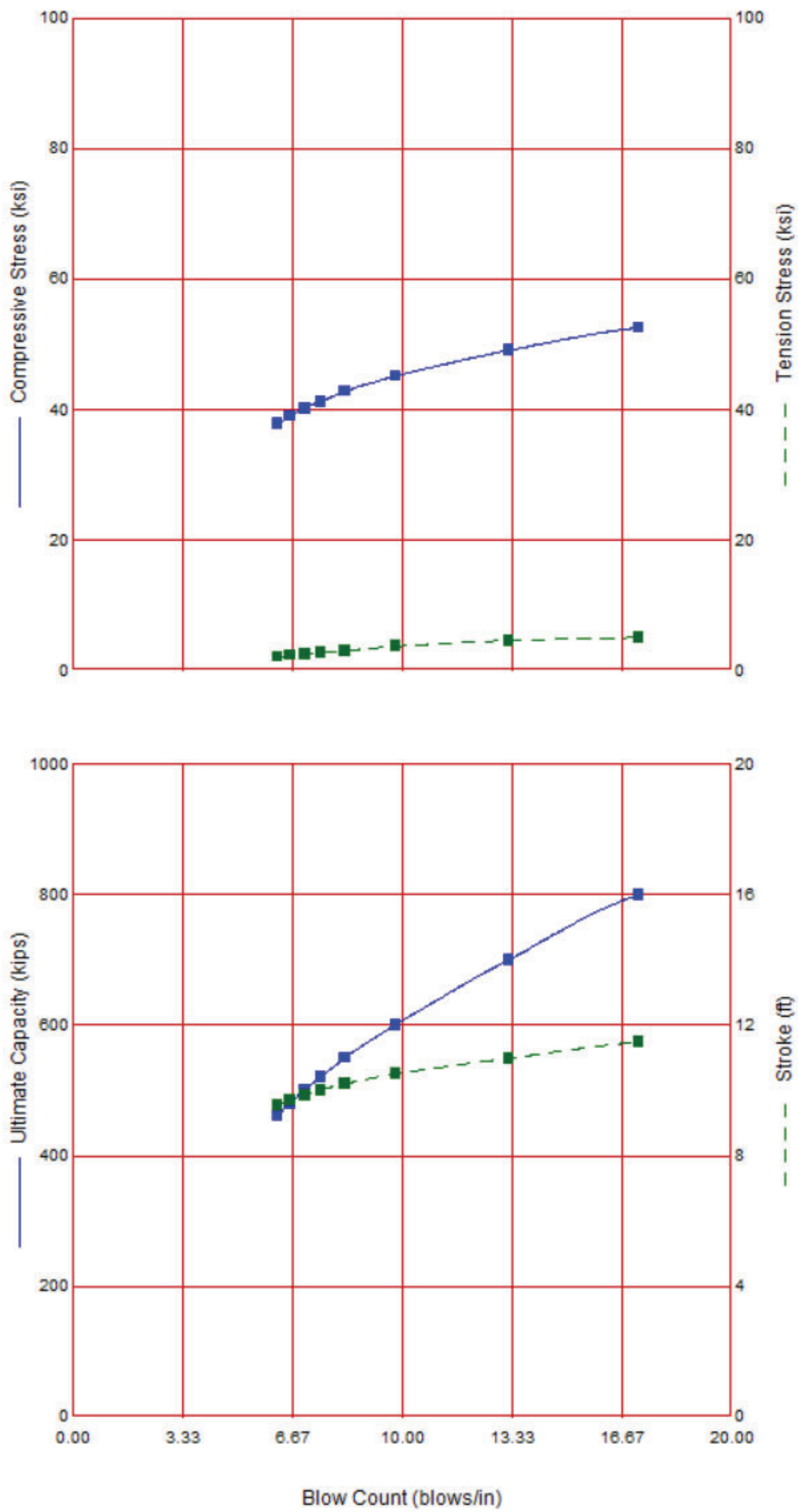
GRLWeap Soil and Pile Model Assumptions

Based on proposed bottom of footing of elevation 132.5 at the abutments, estimated pile lengths will be approx. 30 ft. Assume contractor drives pile lengths of 40 ft (extra length accommodates for attachment of dynamic testing equipment, embedment into abutment, variation in bedrock surface).

Use constant shaft resistances so that GRLWeap will assign approx. 80 kips as skin friction.

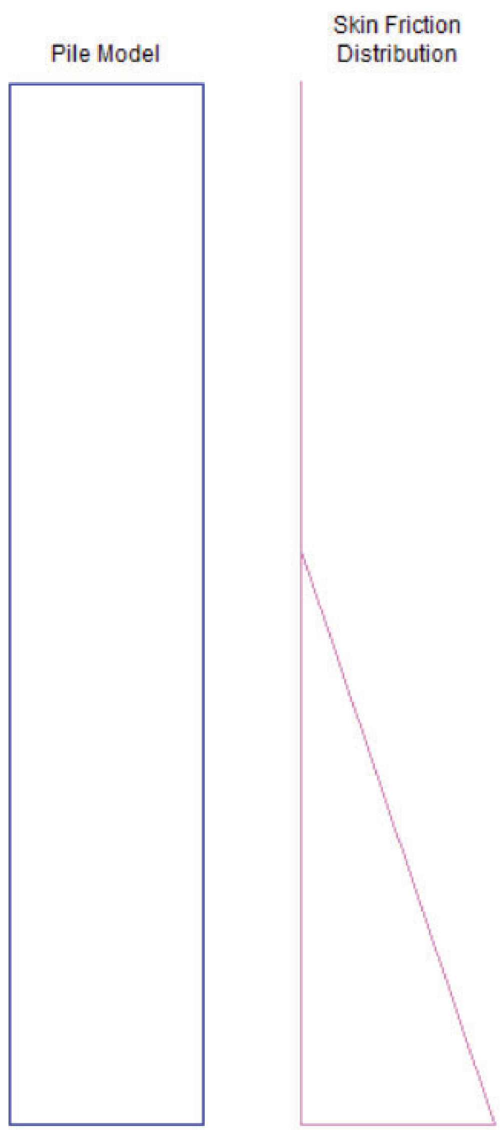
Pile Size is 14 x 89

The 14x89 pile can be driven to the resistances below with a D 19-42 hammer at fuel setting 1 (100% of Max) and 1.9 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:



DELMAG D 19-42

Ram Weight	4.00	kips
Efficiency	0.800	
Pressure	1600 (100%)	psi
Helmet Weight	1.90	kips
Hammer Cushion	60155	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	40.00	ft
Pile Penetration	22.00	ft
Pile Top Area	26.10	in2



Res. Shaft = 78.2 kips
(Constant Res. Shaft)

Maine DOT
23094 Litchfield Babcock 14x89 D19-42

09-Jun-2021
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
460.0	37.79	2.09	6.2	9.53	19.05
480.0	38.97	2.33	6.6	9.69	19.45
500.0	40.21	2.54	7.0	9.85	19.83
520.0	41.22	2.70	7.5	10.01	20.15
550.0	42.78	3.07	8.3	10.20	20.66
600.0	45.10	3.72	9.8	10.50	21.37
700.0	49.06	4.63	13.2	10.98	22.51
800.0	52.56	5.10	17.2	11.48	23.72

Limit to 40 ksi

$R_{ndr} := 480 \cdot kip$

Strength Limit State

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

$R_{fdr} = 312 \cdot kip$

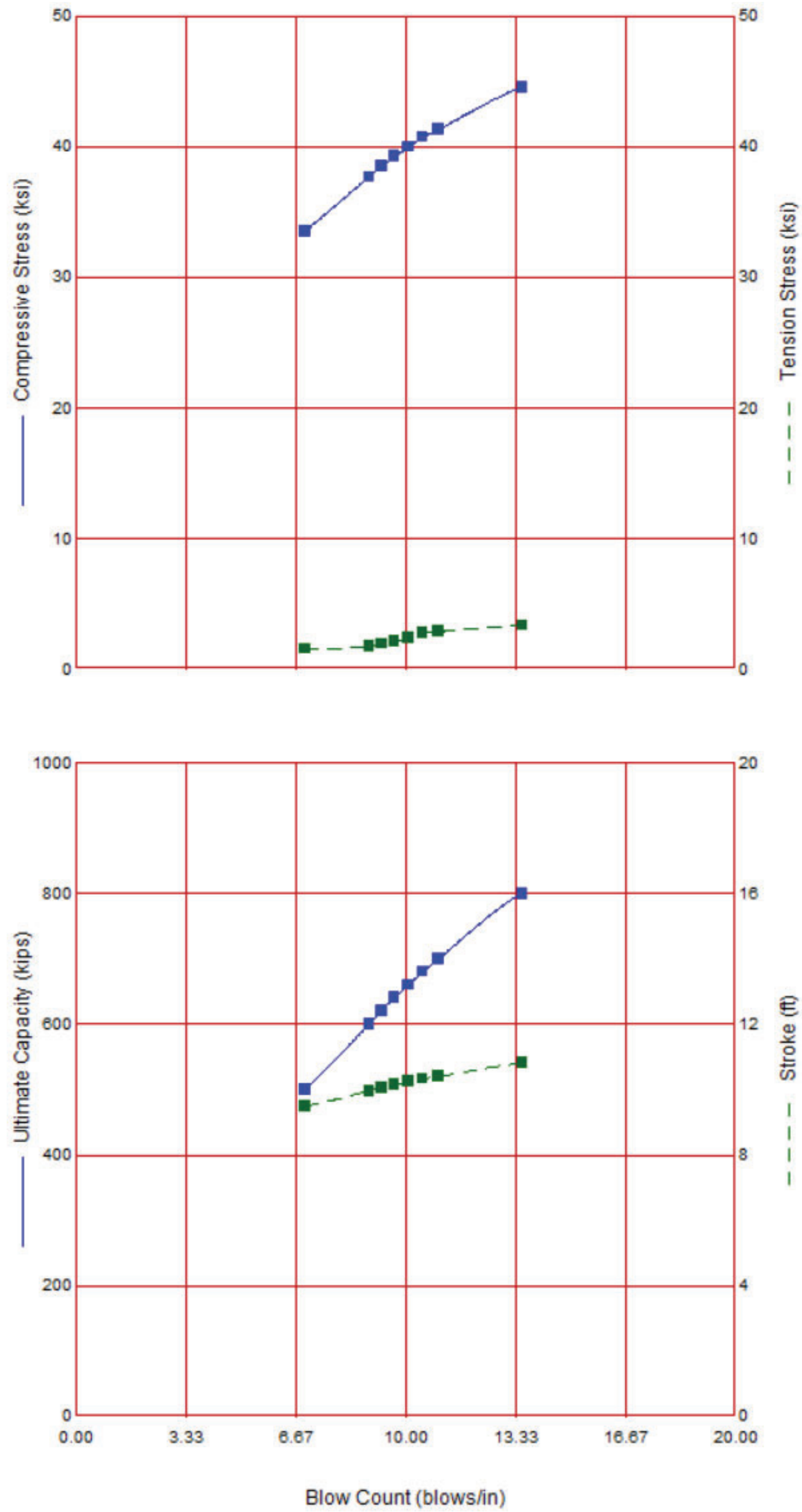
Extreme and
Service Limit States

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

$R_{dr} = 480 \cdot kip$

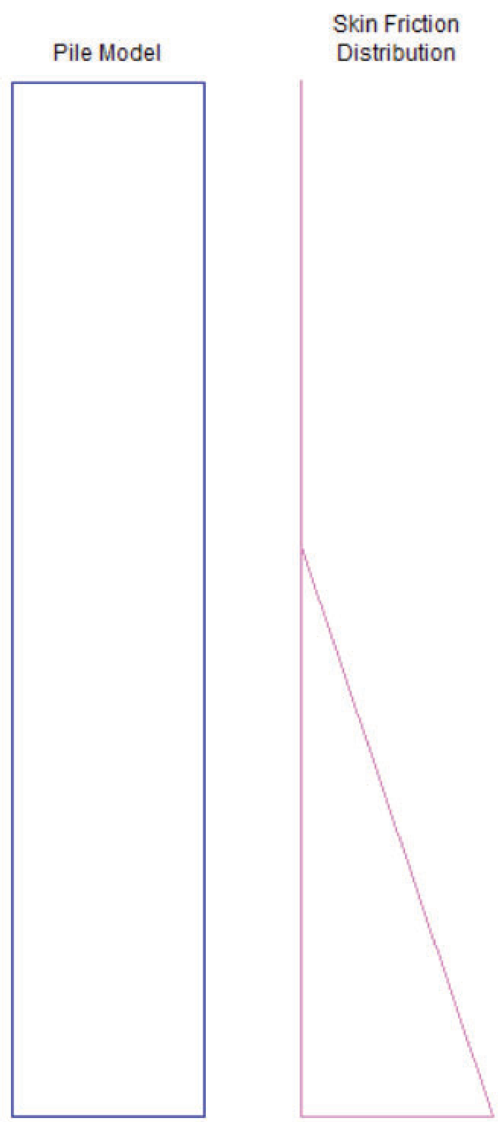
Pile Size is 14 x 117

The 14x117 pile can be driven to the resistances below with a D 19-42 hammer at fuel setting 1 (100% of max) and 1.9 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:



DELMAG D 19-42

Ram Weight	4.00	kips
Efficiency	0.800	
Pressure	1600 (100%)	psi
Helmet Weight	1.90	kips
Hammer Cushion	60155	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	40.00	ft
Pile Penetration	22.00	ft
Pile Top Area	34.40	in2



Res. Shaft = 80.0 kips
(Constant Res. Shaft)

Maine DOT
23094 Litchfield Babcock 14x117 D19-42

03-Jun-2021
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
500.0	33.51	1.58	6.9	9.49	18.42
600.0	37.68	1.76	8.9	9.96	19.48
620.0	38.46	1.94	9.3	10.06	19.77
640.0	39.24	2.16	9.6	10.15	19.99
660.0	39.96	2.37	10.1	10.24	20.16
680.0	40.71	2.79	10.5	10.32	20.41
700.0	41.30	2.88	11.0	10.40	20.52
800.0	44.53	3.35	13.5	10.81	21.52

Limit to 40 ksi

$R_{ndr} := 660 \cdot kip$

Strength Limit State

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

$R_{fdr} = 429 \cdot kip$

Extreme and
Service Limit States

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

$R_{dr} = 660 \cdot kip$

LPile Soil Layers and References for Soil Model Parameters

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project:	Babcock Bridge #2029 carries Route 126 over Cobbossecontee Stream	**Boring No.:**	BB-LCS-101
Location:	Litchfield-West Gardiner, Maine	**WIN:**	23094.00

Driller:	MaineDOT	**Elevation (ft.)**	141.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/24-25/2019	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	13+62.9, 5.2 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	9.5 ft bgs.
Hammer Efficiency Factor:	0.886	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	140.5		6" HMA. 0.5	
5	1D	24/16	5.00 - 7.00	3/5/13/11	18	27				Brown, damp, medium dense, fine to coarse SAND, little gravel, some silt, (Fill). Layer 1 (142-132)	G#337366 A-2-4, SM WC=9.3%
10	2D	24/9	10.00 - 12.00	4/5/10/9	15	22	34			Brown, wet, medium dense, fine to coarse SAND, little gravel, trace silt, (Fill). Layer 2 (132-128)	
							45				
							52				
							53	128.0		13.0	
							51				
15	3D	24/12	15.00 - 17.00	15/18/13/6	31	46	31			Grey, wet, dense, fine to coarse SAND, some rock fragments, trace silt, (Rock Fill). Layer 3 (128-122)	
							138				
							182			Cobble from 17.8-18.4 ft bgs.	
							170				
							32	122.0		19.0	
20	4D	24/14	20.00 - 22.00	13/13/6/7	19	28	28			Grey, wet, medium dense, Gravelly, fine to coarse SAND, some silt, (Glacial Till). Layer 4 (122-114)	G#337367 A-1-b, SM WC=9.7%
							34				
							40				
							76				
25							62				

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Babcock Bridge #2029 carries Route 126 over Cobbossecontee Stream
Location: Litchfield-West Gardiner, Maine

Boring No.: BB-LCS-101
WIN: 23094.00

Driller:	MaineDOT	**Elevation (ft.)**	141.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/24-25/2019	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	13+62.9, 5.2 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	9.5 ft bgs.
Hammer Efficiency Factor:	0.886	**Hammer Type:**	Automatic ☒	Hydraulic ☐	Rope & Cathead ☐

Definitions:
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N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
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G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/14	25.00 - 27.00	12/9/10/12	19	28	23			Grey, wet, medium dense, fine to coarse SAND, little silt, little gravel, (Glacial Till).	
							36				
							53				
							50			Layer 5 (114-107)	
							56				
30	6D	24/4	30.00 - 32.00	16/13/16/22	29	43	32			Grey, wet, dense, Silty fine SAND, little rock fragments, (Glacial Till).	
							43				
							89				
							177	107.5		33.5	
							75			Cobble from 33.5-34.0 ft bgs. Roller Coned ahead from 33.5-35.0 ft bgs.	
35	7D	24/15	35.00 - 37.00	15/20/20/26	40	59	37			Dark grey, wet, very dense, Gravelly fine to medium SAND, little silt, (Glacial Till), (rock fragments from cobble).	G#337368 A-1-b, SM WC=9.2%
							38			Layer 6 (107-102)	
							61				
							a126			a126 blows for 0.9 ft.	
							RC	102.1		38.9 Top of Bedrock at Elev. 102.1 ft. Roller Coned ahead to 40.2 ft bggs.	
40	R1	60/60	40.20 - 45.20	RQD = 100%			NQ-2			R1: Bedrock: Black and white, medium to coarse-grained, riebeckite SYENITE, very hard, fresh, massive. [Litchfield Pluton] Rock Quality: Excellent. R1: Core Times (min:sec) 40.2-41.2 ft (3:22) 41.2-42.2 ft (3:13) 42.2-43.2 ft (3:44) 43.2-44.2 ft (3:32) 44.2-45.2 ft (4:00) 100% Recovery	
45	R2	60/60	45.20 - 50.20	RQD = 100%						R2: Bedrock: Similar to R1. [Litchfield Pluton] Rock Quality = Excellent R2: Core Times (min:sec) 45.2-46.2 ft (2:11) 46.2-47.2 ft (2:10) 47.2-48.2 ft (2:19) 48.2-49.2 ft (2:33) 49.2-50.2 ft (2:25)	
50											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Babcock Bridge #2029 carries Route 126 over Cobbossecontee Stream
Location: Litchfield-West Gardiner, Maine

Boring No.: BB-LCS-101
WIN: 23094.00

Driller:	MaineDOT	**Elevation (ft.)**	141.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/24-25/2019	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	13+62.9, 5.2 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	9.5 ft bgs.
Hammer Efficiency Factor:	0.886	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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WOH = Weight of 140 lb. Hammer
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WO1P = Weight of One Person
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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50							V	90.8		100% Recovery 50.2 **Bottom of Exploration at 50.2 feet below ground surface.**	
55											
60											
65											
70											
75											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Babcock Bridge #2029 carries Route 126 over Cobbossecontee Stream
Location: Litchfield-West Gardiner, Maine

Boring No.: BB-LCS-102
WIN: 23094.00

Driller:	MaineDOT	**Elevation (ft.)**	141.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Aaron/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/3/2019, 10/7/2019	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	14+55.1, 11.7 ft Rt.	**Casing ID/OD:**	HW-4"/NW-3"	**Water Level*:**	8.0 ft bgs.
Hammer Efficiency Factor:	0.886	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
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$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
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N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	141.1		5½" HMA. 0.5	
5	1D	24/17	5.00 - 7.00	2/2/7/9	9	13				Brown, damp, medium dense, Gravelly, fine to coarse SAND, little silt, (Fill). Layer 1 (142-132)	G#337370 A-1-b, SM WC=7.5%
								132.6		9.0	
10	2D	24/5	10.00 - 12.00	7/18/14/12	32	47	36			Grey, wet, very dense, COBBLES and GRAVEL, (Rock Fill). Layer 2 (132-116)	
							70				
							62				
							101				
							53				
15	3D	24/4	15.00 - 17.00	4/11/3/3	14	21	59			Similar to above, except medium dense, (Rock Fill).	
							74				
							69				
							71				
	MD	24/0	19.00 - 21.00	9/19/10/9	29	43	48			Similar to above in wash water, dense, (Rock Fill).	
20							93				
							166				
							159				
							141				
							38			Cobbles from 24.0-25.5 ft bgs.	
25											

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Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Babcock Bridge #2029 carries Route 126 over Cobbossecontee Stream

Location: Litchfield-West Gardiner, Maine

Boring No.: BB-LCS-102

WIN: 23094.00

Driller:	MaineDOT	**Elevation (ft.)**	141.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Aaron/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/3/2019, 10/7/2019	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	14+55.1, 11.7 ft Rt.	**Casing ID/OD:**	HW-4"/NW-3"	**Water Level*:**	8.0 ft bgs.
Hammer Efficiency Factor:	0.886	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							74	115.6	26.0	
	4D	24/20	26.00 - 28.00	5/2/2/3	4	6	48		Grey, wet, medium stiff, Silty CLAY, trace fine sand, (Glaciomarine Deposit).	G#337369 A-6, CL WC=36.5% LL=35 PL=21 PI=14
							29			
							133	113.1	28.5	
							12		Cobbles and Gravel.	
30	MD	18/0	30.00 - 31.50	5/14/55	69	102	31		Set in NW Casing	
							52			
							250			
							48			
	5D	24/16	34.00 - 36.00	13/12/18/18	30	44	110		Grey, wet, dense, Silty fine to coarse SAND, little gravel, occasional cobble, (Glacial Till).	G#337371 A-4, SM WC=10.7%
35							36			
							70		Roller Coned ahead to 38.4 ft bgs.	
							45		Cobble from 36.7-37.5 ft bgs.	
							a40	103.2	a40 blows for 0.4 ft. 38.4	
									Bottom of Exploration at 38.4 feet below ground surface. Broke NW Casing at 38.4 ft bgs.	
40										
45										
50										

Layer 3 (116-113)

Layer 4 (113-94)

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Babcock Bridge #2029 carries Route 126 over Cobbossecontee Stream Location: Litchfield-West Gardiner, Maine	Boring No.: BB-LCS-102A WIN: 23094.00

Driller:	MaineDOT	**Elevation (ft.)**	141.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/22,24/2019	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+56.8, 10.7 ft Rt.	**Casing ID/OD:**	HW-4"/NW-3"	**Water Level*:**	12.0 ft bgs.
Hammer Efficiency Factor:	0.886	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			No soil samples retrieved. See BB-LCS-102 for material descriptions.	
5											
10							DROVE			Drove HW Casing to 34.0 ft bgs.	
15											
20											
25											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Babcock Bridge #2029 carries Route 126 over Cobbossecontee Stream
Location: Litchfield-West Gardiner, Maine

Boring No.: BB-LCS-102A
WIN: 23094.00

Driller:	MaineDOT	**Elevation (ft.)**	141.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/22,24/2019	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+56.8, 10.7 ft Rt.	**Casing ID/OD:**	HW-4"/NW-3"	**Water Level*:**	12.0 ft bgs.
Hammer Efficiency Factor:	0.886	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25											
30											
35											
								102.6		39.0	
	1D	12/6	39.00 - 40.00	75/60	---					Grey, wet, very dense, GRAVEL and COBBLES.	
40											
45	2D	24/16	45.00 - 47.00	35/32/22/27	54	80				Grey, wet, very dense, Gravelly fine to coarse SAND, trace silt, (Glacial Till).	
								93.6		Roller Coned ahead to 48.0 ft bgs. 48.0	
	R1	50.4/50.4	48.00 - 52.20	RQD = 62%			NQ-2			Bedrock at Elev. 93.6 ft. R1: Bedrock: Black and white, medium to coarse-grained, riebeckite SYENITE, hard, fresh, massive.	
50											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Babcock Bridge #2029 carries Route 126 over Cobbossecontee Stream
Location: Litchfield-West Gardiner, Maine

Boring No.: BB-LCS-102A
WIN: 23094.00

Driller:	MaineDOT	**Elevation (ft.)**	141.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/22,24/2019	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+56.8, 10.7 ft Rt.	**Casing ID/OD:**	HW-4"/NW-3"	**Water Level*:**	12.0 ft bgs.
Hammer Efficiency Factor:	0.886	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
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LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										[Litchfield Pluton] Rock Quality = Fair R1: Core Times (min:sec) 48.0-49.0 ft (1:17) 49.0-50.0 ft (1:32) 50.0-51.0 ft (1:55) 51.0-52.0 ft (1:54) 52.0-52.2 ft (1:00) Core Blocked 100% Recovery	
	R2	60/60	52.20 - 57.20	RQD = 100%						R2: Bedrock: Similar to R1. [Litchfield Pluton] Rock Quality = Excellent R2: Core Times (min:sec) 52.2-53.2 ft (1:37) 53.2-54.2 ft (1:55) 54.2-55.2 ft (1:52) 55.2-56.2 ft (1:59) 56.2-57.2 ft (2:05) 100% Recovery	
55								84.4			
										57.2	
60										**Bottom of Exploration at 57.2 feet below ground surface.**	
65											
70											
75											

Remarks:

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

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

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

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

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

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

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

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

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

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

b. Compute the power of the parabolic section using

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

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

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

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

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

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

LPile will assign a default value for *k* if the user enters a value of zero. The value of *k* is determined from the angle of friction and it is assumed that the sand is fine. The equations used by LPile to determine *k* as a function of friction angle for fine sand are shown in Figure 3-34. Whether the sand is above or below the water table will be determined from the input value of effective unit weight. If the effective unit weight is less than 77.76 pcf (12.225 kN/m^3) the sand is considered to be below the water table. If the input value of ϕ is greater than 45 degrees, a *k* value corresponding to 45 degrees is used by LPile. The two correlation lines intersect at a friction angle value of 27.6423 degrees and a *k* value of 10.2068 pci. If the input value of ϕ is less than 27.6423 degrees, the value of *k* linearly varies from a value of zero at zero degrees to a value of 10.2068 pci at 27.6423 degrees

If the sand profile is coarse or well-graded sand, the user may consider using a higher value of *k* that those suggested in the Figure 3-34. While experimental data for *k* in well-graded sands is sparse, use of *k* values 10 to 50 percent higher may be appropriate in dense and very dense well-graded sands that do not contain any compressible minerals such as mica.

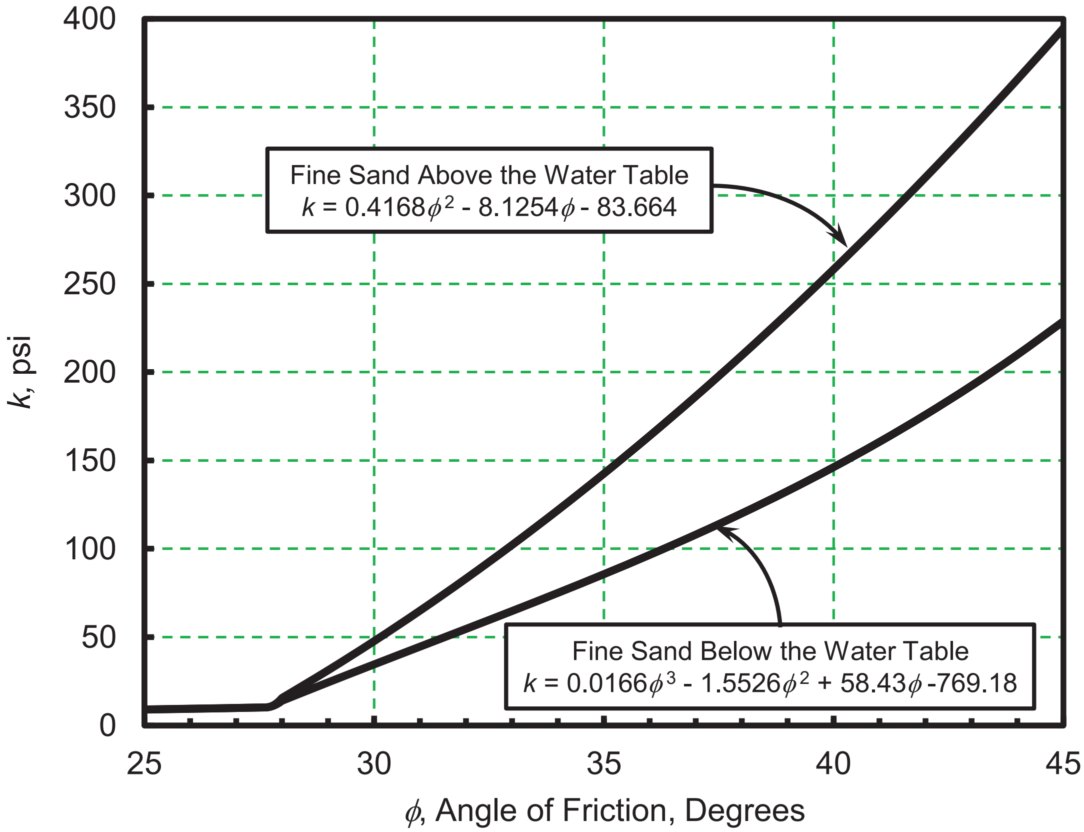


Figure 3-34 Value of *k* versus Friction Angle for Fine Sand Used in LPile

3-4-3-3 Example Curves

An example set of *p-y* curves was computed for sand above the water table, using the API criteria. The soil properties are unit weight $\gamma' = 0.07$ pci, and internal-friction angle $\phi' = 35$ degrees. The sand layer exists from the ground surface to a depth of 40 feet. The pile is of loading. The load was applied in two directions, with the load in the forward direction being more than twice as large as the load in the backward direction. After a significant number of cycles, the deflection at the top of the pile was either stable or creeping slowly, so an equilibrium condition was assumed. The *p-y* curves for cyclic loading are intended to represent a lower-bound condition. Thus, a designer might possibly be computing an overly conservative response of a pile, if the cyclic *p-y* curves are used and if there are only a small number of applications of the design load (the factored load).

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

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

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

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

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

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

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

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

where

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

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

c = shear strength at depth x, and

b = width of pile.

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

Earth Pressure

Earth Pressure:

Backfill engineering strength parameters

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

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

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

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

Integral Abutment - Passive Earth Pressure - Coulomb Theory

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

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

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

Use Coulomb for cases where interface friction is considered; typically gravity shaped structures, and integral abutments where the ratio of wall height to wall movement is .020 or greater. Coulomb should also be used when the fill slope is greater than horizontal.

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

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

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

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

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

$K_{p\_coulomb} = 6.02$

Integral Abutment and Wingwall - Passive Earth Pressure - Rankine Theory

Use Rankine only if the ratio of wall height to wall movement is 0.005 or less and the fill slope is horizontal to the top of the wall. Bowles does not recommend use of Rankine method for K_p when $\alpha > 0$.

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

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

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

$K_{p\_rank} = 3.25$

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

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

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

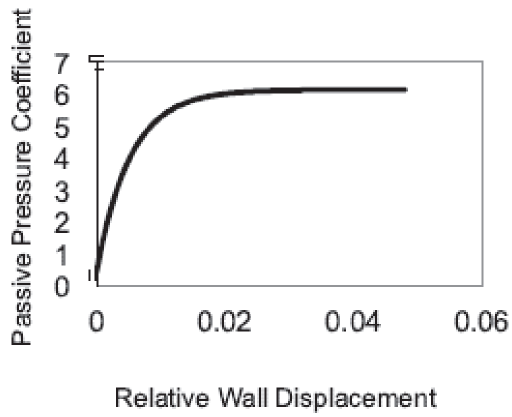


Figure 3.10.8-1: Plot of Passive Pressure Coefficient, K, vs. Relative Wall Displacement, δ_T/H.

Based on an estimated Relative Wall Displacement of 0.01:

$K := 0.43 + 5.7 \cdot [1 - \exp[-190(0.01)]]$

$K = 5.28$

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

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

3.11.5.4—Passive Lateral Earth Pressure Coefficient, k_p

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

For cohesive soils, passive pressures may be estimated by:

C3.11.5.4

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

Table 7.9 (*Continued*)

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

7.12 Coulomb's Passive Earth Pressure

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

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

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

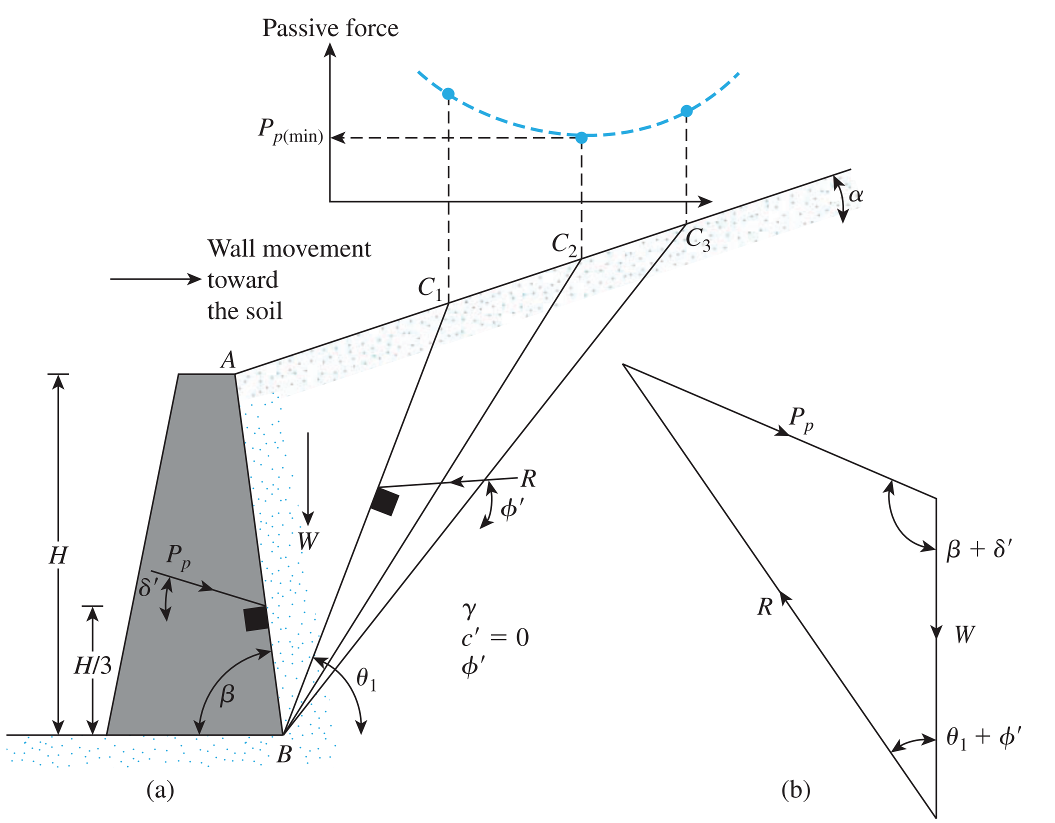


Figure 7.25 Coulomb's passive pressure

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

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

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

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

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

where

$$K_p = \text{Coulomb's passive pressure coefficient} = \frac{\sin^2(\beta-\phi')}{\sin^2\beta \sin(\beta+\delta')\left[1-\sqrt{\dfrac{\sin(\phi'+\delta')\sin(\phi'+\alpha)}{\sin(\beta+\delta')\sin(\beta+\alpha)}}\right]^2} \tag{7.71}$$

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

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

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

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

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

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

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

Again, at $z = 3$ m,

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

Hence,

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

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

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

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

■

7.11 Rankine Passive Earth Pressure: Vertical Backface and Inclined Backfill

Granular Soil

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

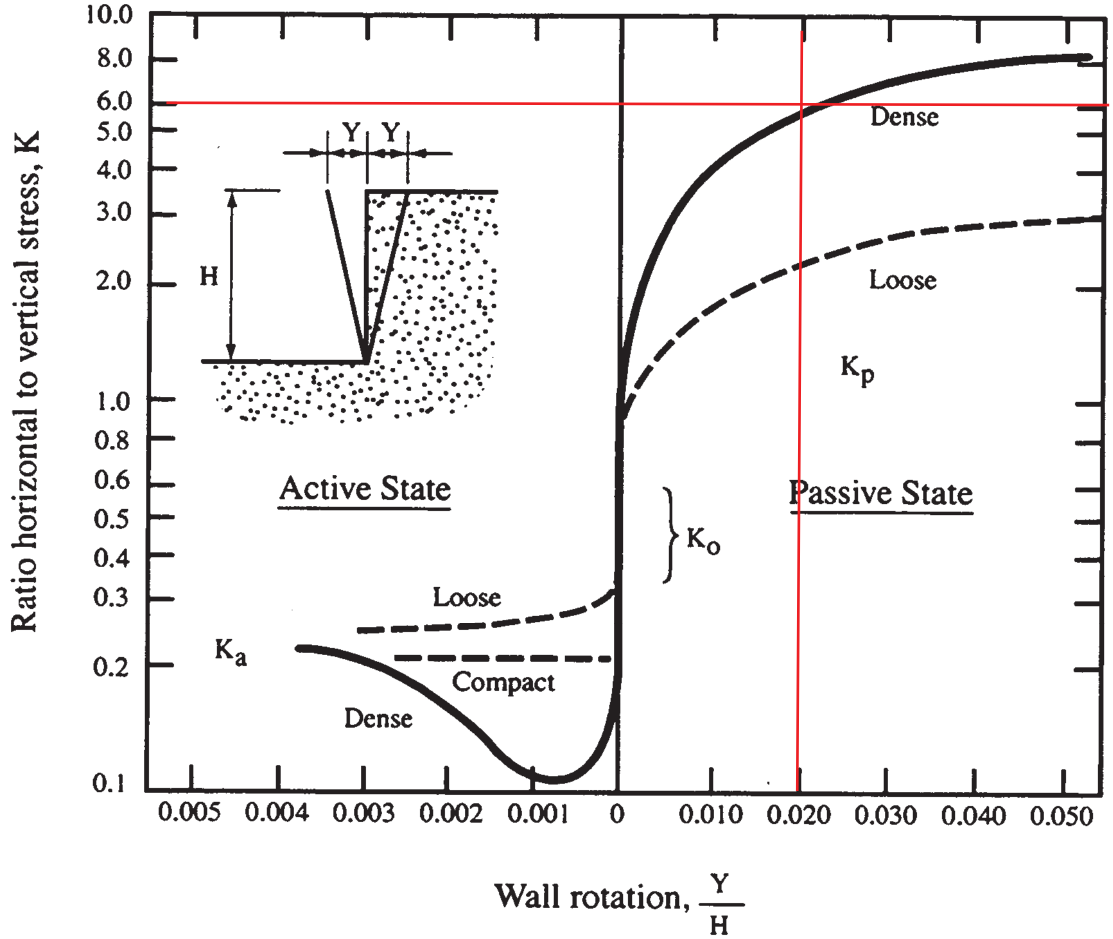
$$\sigma'_p = \gamma z K_p \tag{7.65}$$

and the passive force is

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

where

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



Magnitude of Wall Rotation to Reach Failure

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

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

Frost Depth

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

From Design Freezing Index Map: **West Gardiner, Maine**
DFI = 1600 degree-days.
Case 1 - coarse grained granular fill soils W=10% (BB-LCS-101 1D).

For DFI = 1600

at w=10% $d_1 := 84.8in$

Depth of Frost Penetration

$d := d_1$ $d = 84.8 \cdot in$ $d = 7.1 \cdot ft$

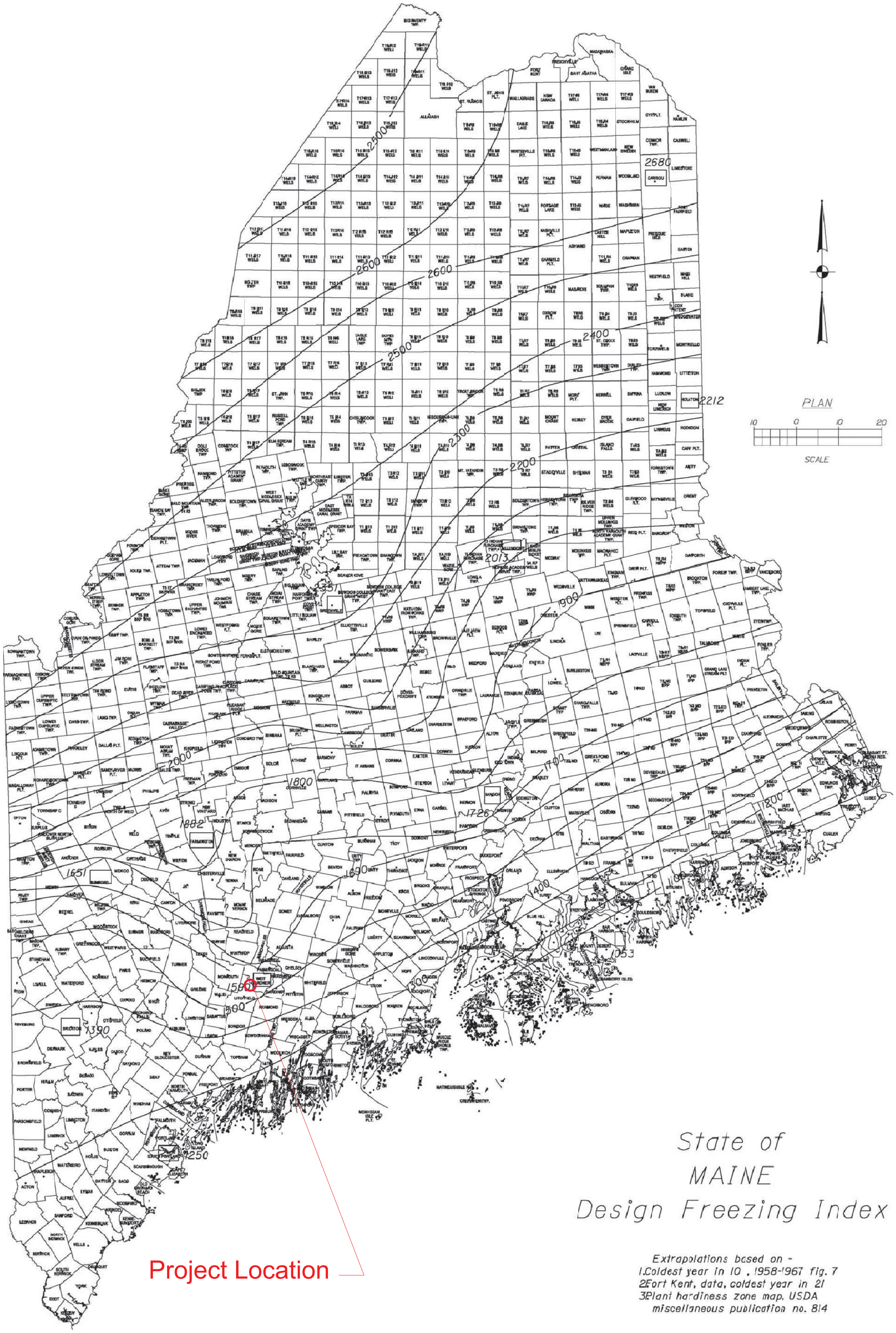


Figure 5-1 Maine Design Freezing Index Map

5.2 General

MaineDOT Bridge Design Guide

5.2.1 Frost

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

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

Table 5-1 Depth of Frost Penetration

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

Seismic Parameters

BB-LCS-101			
Depth	**N_{60}**	**di**	**di/N**
5	27	5	0.19
10	22	8	0.36
15	46	6	0.13
20	28	6	0.21
25	28	4	0.14
30	43	3	0.07
35	59	8.9	0.15
38.9	100	59.1	0.59
SUM		**100**	**1.85**
		di/di/N	**54.11**

BB-LCS-102, -102A			
Depth	**N_{60}**	**di**	**di/N**
5	13	9	0.69
10	47	6	0.13
15	21	4	0.19
20	43	7	0.16
25	6	2.5	0.42
30	102	5.5	0.05
35	44	4.4	0.10
40	50	6.6	0.13
45	80	3	0.04
48	100	52	0.52
SUM		**100**	**2.43**
		di/di/N	**41.10**

SUM	**Nav.**	**47.60**

15 < N_{av}. < 50 bpf

Conclusion: Site Class D

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

Litchfield Babcock Bridge #2029
WIN 23094.00
July 27, 2021

Abutment No. 1 and 2 Seismic Parameters

2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 44.203006
Longitude = -069.899278
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.080	PGA - Site Class B
0.2	0.165	Ss - Site Class B
1.0	0.045	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
Latitude = 44.203006
Longitude = -069.899278
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.129	As - Site Class D
0.2	0.264	SDs - Site Class D
1.0	0.109	SD1 - Site Class D

Litchfield Babcock Bridge #2029
WIN 23094.00
July 27, 2021