MAINE DEPARTMENT OF TRANSPORTATION BRIDGE PROGRAM GEOTECHNICAL SECTION AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

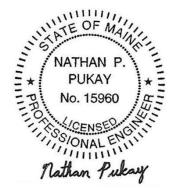
PERKINS BRIDGE AND LAKE BRIDGE LAKE ROAD OVER BLACK STREAM LEVANT, MAINE



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Penobscot County WIN 27098.00 Soils Report 2024-04 Bridge No. 6133 & 3359

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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 Perkins Bridge and Lake Bridge which carry Lake Road over Black Stream in Levant, Maine. This report presents the subsurface information obtained at the site during the subsurface investigations, geotechnical design recommendations, and construction recommendations for the new substructures.

The existing Perkins Bridge and Lake Bridge were constructed in 1986, and both structures consist of twin steel structural plate pipe arches with mitered ends. Perkins Bridge is aligned along the main stream channel with each pipe spanning 15-foot 10-inch and rising 10-foot 8-inch. Lake Bridge, installed for overflow during high water conditions, consists of smaller pipes each measuring 11-foot 10-inch span by 7-foot 7-inch rise. Perkins Bridge failed in April 2023 resulting in the closure of Lake Road. According to the 2022 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the FHWA Sufficiency Rating of Lake Bridge is 71.9. The condition rating of Lake Bridge pipe arches is a 4 (poor condition) due to rusting and pitting above the flow line.

Available as-built drawings indicate previous structures at both bridges consisted of a steel beam superstructure on stacked granite abutments.

The proposed replacement structure for Perkins Bridge consists of a 84-foot, single-span steel girder bridge with horizontal and vertical alignments that will closely match the existing. The bridge will be founded on pile-supported integral abutments with cantilevered, in-line wingwalls. Piles will be driven to bedrock. 1.75H:1V (horizontal:vertical) riprap slopes will be constructed in front of the new integral abutments. A wildlife shelf will be built into the riprap at Abutment No. 1. Due to the significant increase in hydraulic opening of the new Perkins Bridge in conjunction with consultations with the MaineDOT Environmental Office, Lake Bridge will be reduced to a single 8-foot diameter HDPE or aluminum culvert. To expedite the construction of this project, a separate contract allowing the Department to pre-buy the steel girders and bearing plates for Perkins Bridge was established and bid in November 2023.

Traffic is currently being detoured onto State Aid and Town roads. The existing detour will be maintained during construction.

2.0 GEOLOGIC SETTING

Perkins and Lake Bridges carries Lake Road over Black Stream as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of the Hermon Quadrangle, Maine, Open-File No. 13-13 (2013), indicates the surficial soils in the vicinity of the bridge project consist of stream alluvium, glaciomarine deposits (Presumpscot Formation), and glacial till.

Stream alluvium consists of sand, gravel, and silt deposited on flood plains and stream beds by postglacial streams and may include some wetland deposits. Glaciomarine deposits consist of silt, clay, and sand, deposited on the late-glacial sea floor. Glacial till is a heterogeneous mixture of sand, silt, clay, and stones deposited by glacial ice.

The MGS Reconnaissance Bedrock Geology of the Bangor Quadrangle, Maine, Open-File No. 76-23 (1976) maps the bedrock at the site as Feldspathic Wacke of the Vassalboro Formation, with thick interbeds of dark grey Phyllite. The bedrock cored in the test borings drilled at the site consisted of Graywacke with abundant layers of Phyllite.

3.0 Subsurface Investigation

Seven test borings were drilled to explore subsurface conditions at the project location. Borings BB-LBS-101 and BB-LBS-101A, were drilled at the proposed location of Perkins Bridge Abutment No. 1. BB-LBS-101A was drilled adjacent to BB-LBS-101 after the hole was abandoned due to broken drilling equipment. BB-LBS-102 and BB-LBS-201 were drilled at the proposed location of Perkins Bridge Abutment No. 2. BB-LBS-103, BB-LBS-202 and BB-LBS-203 were drilled at Lake Bridge. Four of the borings terminated in bedrock cores. The remaining borings explored the surficial soils and probed the apparent bedrock surface. The boring locations are shown on Sheet 2 – Boring Location Plan.

The 100-series borings were drilled in June 2023 by the MaineDOT drill crew. The crew returned to the project site in July 2023 and drilled the 200-series borings. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheets 4 and 5 – Boring Logs.

Borings were performed by using a combination of solid stem auger, cased wash boring and rock coring techniques. The borings were completed by backfilling and compacting the borehole with drill cuttings. 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 is 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 November 2022. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.906 to the raw field N-values. This hammer efficiency factor (0.906) and both the raw field N-value and corrected N-value (N60) are shown on the boring logs.

Bedrock was cored in the borings using NQ-2" core barrels and the Rock Quality Designation (RQD) of the cores calculated. The MaineDOT geotechnical engineer selected the boring locations and drilling methods, designated type and depth of sampling techniques, identified field-testing requirements, and logged the subsurface conditions encountered in the borings. The borings were located in the field using taped measurements at the completion of the drilling programs and then located by MaineDOT Survey.

4.0 LABORATORY TESTING

A laboratory testing program was conducted on selected soil samples recovered from the test borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the project site. Laboratory testing on soil samples consisted of five standard grain size analyses with natural water content, seven grain size analysis with hydrometer and natural water content, eight Atterberg limits tests, two loss on ignition (organic content) tests and one pH test.

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

5.0 Subsurface Conditions

Subsurface conditions encountered in the test borings generally consisted of Fill, Stream Alluvium with Wetland Deposits, Glacial Till, and Bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheets 4 and 5 – Boring Logs. A generalized subsurface profile is shown on Sheet 3 – Interpretive Subsurface Profile. The following paragraphs discuss the subsurface conditions encountered.

5.1 Fill

A layer of Fill was encountered in the test borings. The thickness of the fill unit encountered was approximately 9 to 14 feet. The fill materials encountered consisted of:

- Brown, SILT, little to some sand, little gravel; and
- Brown to grey-brown, Gravelly SAND, little silt.

Cobbles were encountered in the fill layer in boring BB-LBS-102.

Corrected SPT N-values in the fine-grained fill ranged from 27 to 45 blows per foot (bpf) indicating the fine-grained fill is very stiff to hard in consistency.

Corrected SPT N-values in the coarse-grained fill ranged from 23 to greater than 50 bpf, indicating the coarse-grained fill is medium dense to very dense in consistency.

Three grain size analyses performed on samples recovered from the granular fill unit indicated the material is classified as A-4 and A-1-a under the AASHTO Soil Classification System and CL-ML and SM under the Unified Soil Classification System (USCS). The natural water contents of the samples tested ranged from 4 to 16 percent.

5.2 Stream Alluvium with Wetland Deposits

Stream Alluvium with Wetland Deposits were encountered in BB-LBS-102, -103, -201, -202, and -203 beneath the fill unit. The encountered thickness was approximately 6 to 8 feet. The deposit was variable and consisted of:

- Grey, SAND, little silt, trace gravel;
- Grey, Gravelly SAND, little silt, trace clay;
- Grey, Sandy SILT, trace clay;
- Grey to dark brown, SILT, trace to some sand, trace clay;
- Grey-brown, Silty CLAY, trace gravel;
- Dark-brown to black PEAT; and
- Wood.

Corrected SPT N-values in the fine-grained Stream Alluvium and Wetland Deposits ranged from 5 to 20 bpf indicating those subunits are medium stiff to very stiff in consistency.

Corrected SPT N-values in the coarse-grained Stream Alluvium and Wetland Deposits ranged from 11 to 29 bpf indicating those subunits are medium dense in consistency.

A in-situ vane shear test was conducted with a Geonor rectangular vane in the Stream Alluvium deposit. A 16 x 32 mm vane was used. The maximum measurable vane torque was reached indicating an undrained shear strength exceeding 4181 psf. A disturbed sample of the material tested yielded a Gravelly SAND, therefore the undrained shear strength measured is not representative of the cohesive or organic soils found within this deposit.

Three grain size analyses conducted on samples of the deposit indicated the material is classified as A-2-4, A-1-b, and A-7-5 under the AASHTO Soil Classification System and SM, SC-SM, and OH under the USCS.

Atterberg limits tests were conducted on three samples of the Stream Alluvium with Wetland Deposits, and are summarized below:

Boring No. and Sample No.	Soil Description	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-LBS-102, 3D	Gravelly SAND	5	-	-	NP ¹	-
BB-LBS-201, 3D	SILT	-	-	-	NP ¹	-
BB-LBS-202, 4D	Silty CLAY	50	62	45	17	0.29

The plasticity indices of the samples indicate that the Stream Alluvium and Wetland Deposits vary from non-plastic to medium in plasticity (Burmister, 1949).

¹ Non-plastic (NP)

The natural water content of the silty clay sample was less than the liquid limit, with a liquidity index less than 1.0. Interpretation of these results is that the deposit is overconsolidated.

Loss on ignition tests performed on two samples containing peat indicated the samples had an organic content of 14 and 67 percent. One pH test conducted on a sample of peat measured a pH of 5.22. The natural water content of all test samples recovered from the deposit ranged from 5 to 431 percent.

5.3 Glacial Till

Glacial Till was encountered beneath either the Fill, Stream Alluvium or Wetland Deposits in the majority of the borings. The thickness of the Glacial Till encountered was approximately 4 to 57 feet. The Glacial Till subunits encountered generally consisted of:

- Brown, grey or olive-grey, SILT, little to some sand, trace to some gravel, trace to some clay;
- Olive-grey, Sandy SILT, trace to little gravel, trace clay;
- Olive-grey, SAND, some silt, some gravel, little clay; and
- Grey, Silty SAND, some gravel, trace clay;
- Grey, Clayey SILT, trace sand.
- Cobbles and boulders.

Corrected SPT N-values in the fine-grained Glacial Till ranged from 17 to greater than 50 bpf indicating those subunits are very stiff to hard in consistency.

Corrected SPT N-values in the coarse-grained Glacial Till ranged from 32 to greater than 50 bpf, indicating those subunits are dense to very dense in consistency.

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

Atterberg limits tests were conducted on five samples of the Glacial Till and are summarized below:

Boring No. and Sample No.	Soil Description	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-LBS-101, 3D	SILT	11	20	14	6	-0.50
BB-LBS-101, 4D	SAND	10	20	14	6	-0.67
BB-LBS-101, 6D	SILT	12	21	14	7	-0.29
BB-LBS-102, 4D	SILT	23	20	15	5	1.60
BB-LBS-202, 7D/A	Clayey SILT	31	28	20	8	1.38

The plasticity indices of the samples tested indicate the fine-grained Glacial Till soils have low plasticity (Burmister, 1949). The natural water content of the samples measured 10 to 34 percent and liquid limits ranged from 20 to 28. The resulting liquidity indices range from less than 0 to greater than 1.0. Generally the natural water contents were less than, or close to, the liquid limits, indicating the deposit is primarily normally consolidated to slightly overconsolidated. Those subunits with liquidity indices greater than 1.0 and intermediate water contents are somewhat unconsolidated soils but have a low potential to liquefy.

5.4 Bedrock

Bedrock was encountered and cored in borings BB-LBS-101A, -102, -103, and -201. 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, R3)
BB-LBS-101A	13+36	8.3 Rt	66.2	63.1	77, 77
BB-LBS-102	14+17.2	7.4 Rt	50.8	78.9	75, 78
BB-LBS-103	15+50.4	8.2 Rt	21.3	107.5	0, 69, 63
BB-LBS-201	14+21.2	7.3 Lt	47.2	82.7	83, 75

The bedrock of the site consisted of grey to dark grey, very fine to medium-grained, GRAYWACKE interbedded with layers of PHYLLITE, moderately soft to hard, fresh, with joint sets dipping at low to steep angles, spaced close to moderately close, with quartz or calcite annealed fractures. The RQD of the bedrock cores ranged from 0 to 83 percent, corresponding to a rock quality of very poor to good.

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

5.5 Groundwater

Groundwater was measured at depths ranging from 7 to 10 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

Based on the depth of bedrock and the span length requirement, integral abutments founded on driven H-piles was the preferred substructure type, allowing for a jointless bridge at Perkins Bridge. A new upstream alignment was considered to improve the roadway geometry, but the peat encountered in the preliminary borings indicated the potential need for costly settlement mitigation. A box culvert was initially considered for the replacement of Lake Bridge, but it was determined that an 8-foot culvert pipe would provide adequate overflow due to the increased size of Perkins Bridge.

7.0 GEOTECHNICAL DESIGN CONSIDERATIONS AND RECOMMENDATIONS

The following sections provide geotechnical design considerations and recommendations for Hpile supported integral abutments which is the proposed substructure type for the Perkins Bridge replacement project. Additional considerations are provided for the culvert pipe that will replace Lake Bridge.

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	119.0	63.1	58
Abutment No. 2	119.0	78.9	43

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.

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

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

7.1.1 Axial Pile Resistance – Strength Limit State

Structural Resistance. Preliminary 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, $\phi_c = 0.50$, 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 $\phi_c = 0.70$ 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.

<u>Drivability Analyses</u>. 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. 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.

The calculated factored axial compressive structural, geotechnical, and drivability resistances of driven H-piles at the strength limit states are summarized on the following page.

Strength Limit State Factored Axial Pile Resistance					
Pile Section	Structural Resistance ¹ ϕ_c =0.50 (kips)	Controlling Geotechnical Resistance ² ϕ_c =0.50 (kips)	Resis φ _{dyn} =	ability tance ³ = 0.65 ps)	Governing Axial Pile Resistance (kips)
HP 14 x 89	652	652	409^4	449 ⁵	409^4
HP 14 x 117	860	860	474 ⁴	514 ⁵	474 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.

7.1.1 Axial Pile Resistance – Service and Extreme Limit State

The design of H-piles at the service limit state shall consider tolerable transverse and longitudinal movement of the piles and pile group movements/stability. For the service limit state, resistance factors of $\phi = 1.0$ should be used in accordance with LRFD Article 10.5.5.1. The exception is the overall global stability of the foundation which should be investigated at the Service I load combination and a resistance factor, ϕ , of 0.65.

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

² Based on guidance in LRFD Article 10.7.3.2.3., *Piles Driven to Hard Rock*. The nominal axial geotechnical resistance in the strength limit state was calculated using the guidance in LRFD Article 10.7.3.2.3 which states the nominal bearing resistance of piles driven to point bearing on hard rock shall not exceed the nominal structural resistance values obtained from LRFD Article 6.9.4.1 with a resistance factor ϕ_c , of 0.50, for severe driving conditions applied when computing the factored resistance.

 $^{^3}$ Drivability analyses were performed to determine the pile resistance that might be achieved considering available diesel hammers. Nominal drivability resistances were determined based on a limiting driving criteria of 15 bpi and a maximum driving stress of 45 ksi. The drivability resistances were calculated using the resistance factor, ϕ_{dyn} , of 0.65, for a single pile in axial compression when a dynamic test is performed as specified in LRFD Table 10.5.5.2.3-1.

⁴ Drivability resistance based on a APE D19-42 Pile Hammer at Fuel Setting 4, Abutment 1 pile controls.

⁵ Drivability resistance based on a APE D25-42 Pile Hammer at Fuel Setting 4. Abutment 1 pile controls.

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)	Driva Resist $\phi =$ (ki	tance ³	Governing Axial Pile Resistance (kips)	
HP 14 x 89	1,305	1,305	630 ⁴	690 ⁵	630^{4}	
HP 14 x 117	1,720	1,720	730^{4}	790^{5}	730^{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 45 ksi.

⁴ Drivability resistance based on a APE D19-42 Pile Hammer at Fuel Setting 4, Abutment 1 pile controls.

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

7.1.2 Lateral Pile Resistance/Behavior

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

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

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

LPile Input Parameters Abutment No. 1						
Soil Layer	Soil/Rock Model	Top Elevation of Layer (ft)	Layer Thickness (ft)	γ _e ¹ (pcf)	φ' ² (deg)	k _s ³ (pci)
Granular Borrow	Reese Sand	130	11	125	32	90
Glacial Till	Reese Sand	119	56	83	38	125

LPile Input Parameters Abutment No. 2							
Top Layer						k _s ³ (pci)	
Granular Borrow	Reese Sand	130	11	125	32	90	
Stream Alluvium	Reese Sand	119	5	63	28	40	
Glacial Till	Reese Sand	114	35	73	36	90	

¹ Effective unit weight.

² Effective internal angle of friction.

³ Soil modulus constant.

7.1.3 Driven Pile Quality Control

The contract plans shall require the contractor to perform a wave equation analysis of the proposed pile-hammer system and conduct dynamic pile load tests with signal matching. The first pile driven at each abutment should be dynamically tested to confirm nominal pile resistance and verify the stopping criteria developed by the contractor in the wave equation analysis. Minimum 24-hour restrike tests will be required to verify time-dependent loss of pile resistance does not occur. If a loss in pile resistance does occur, the driving criteria shall be adjusted. Restrikes or additional dynamic tests may be required as part of the pile field quality control program should pile behavior vary radically between adjacent piles, should the pile tip be not firmly embedded in bedrock, or if piles "walk" out of position.

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

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

7.1.4 Corrosion Mitigation

Per LRFD Article 10.7.5, soils with a pH less than 5.5 should be considered as indicative of a potential corrosion situation. A pH test conducted on a representative sample of wetland deposits measured a pH of 5.22. Corrosion mitigation countermeasures for piles driven through the wetland deposits are therefore recommended. The borings conducted at Abutment No. 2 indicate the piles will be driven through the corrosive deposit.

The risk of corrosion will be substantially mitigated by predrilling oversized holes at the pile locations, installing an HDPE isolation casing to provide a barrier from the corrosive soils, and backfilling the casing with clean sand. The casing should extend two feet into the non-corrosive glacial till deposit. Other recommended corrosion mitigation countermeasures include upsizing the piles and designing for an assumed section loss or extending the concrete pile jacket into non-corrosive soils and requiring the concrete meet the permeability requirements of low permeability concrete as specified in MaineDOT Specification 502.05 – Composition and Proportioning.

Corrosive wetland deposits were also encountered at the plan installation location of the culvert pipe that will replace the existing Lake Bridge. An HDPE or aluminum pipe is recommended to increase the design life of the culvert.

7.2 Integral Abutment and Wingwall Design

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

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

- Internal Friction Angle (ϕ) = 32°
- Total Unit Weight $(\gamma) = 125 \text{ pcf}$
- Soil-Concrete Interface Friction Angle (δ) = 17° (ref: LRFD Table 3.11.5.3-1)

Integral abutments and in-line wingwalls shall be designed to withstand a lateral earth load equal to the passive pressure state. Estimation of passive earth pressure should consider LRFD C3.11.5.4, which states that the relative wall movement to induce full passive pressure is approximately 0.05 for dense backfill, and FHWA NHI-06-089 Figure 10-4 which supports a K_p of 6.0 and greater for dense backfills and wall rotations equal to or greater than 0.02. Using Rankine Theory, a lateral earth pressure coefficient of 3.25 is recommended, assuming a ratio of thermal expansion to abutment height (δ /H) of 0.003 and a level backfill. In general, when the calculated ratio of lateral movement to wall height exceeds 0.0036, a passive earth pressure coefficient can be estimated using MassDOT LRFD Bridge Design Manual 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	h _{eq}
(feet)	h _{eq} (feet)
5	4.0
10	3.0
≥20	2.0

In-line wingwalls shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to an equivalent height of soil of 2.0 feet. An at-rest earth pressure coefficient, K_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 vertical alignment of the new Perkins Bridge will closely match the existing. The bridge approach embankments will be constructed using granular borrow placed and compacted over generally very stiff fine-grained and dense coarse-grained fill. Any loose soils encountered at the subgrade elevation shall be thoroughly compacted prior to backfill operations. With these provisions, any settlement of the final roadway embankments is anticipated to be small and immediate.

The 8-foot culvert pipe replacing Lake Bridge will be installed on a 12-inch bed of granular borrow on a subgrade consisting of generally dense granular fill. The replacement pipe will be smaller and fabricated from a material lighter than the existing. Therefore, any settlement of the culvert pipe is anticipated to be minimal.

Conventional earth fill embankments constructed over the existing 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.

The project will require moderate widening and raising of the upstream sideslopes at the start of the project, then east of Perkins Bridge, there is moderate widening and raising of both sideslopes for the remainder of the project. Any peat or wetland deposits encountered at the subgrade of toes of reconstructed side slopes and abutment foreslopes should be excavated to a nominal depth of 1-foot and replaced with granular borrow.

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, Levant has a design freezing index (DFI) of approximately 1825 F-degree days. Fill soils are anticipated to be present at the abutments and embankments, either as reworked silty fill or granular fill. Based on the coarse-grained fill with a water content of 10 percent, the estimated depth of frost penetration is approximately 7.6 feet. It is recommended that any foundation bearing on soils be embedded 7.6 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)	0.07g
Acceleration Coefficient (A _S)	0.11g
S_{DS} (Period = 0.2 sec)	0.24g
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

Any peat, organics, soft or loose soils encountered at the subgrade elevation at either abutment shall be excavated in its entirety and replaced with Granular Borrow – Material for Underwater Backfill and the exposed subgrade then thoroughly compacted.

Any peat or wetland deposits encountered at the subgrade of toes of reconstructed side slopes and abutment foreslopes should be excavated to a nominal depth of 1-foot and replaced with granular borrow.

Excavation for the abutments is anticipated to be accomplished using sloped open cut methods in accordance with MaineDOT and OSHA requirements. Excavations will expose fine-grained soils that may become saturated and water seepage may occur during construction. There may be localized sloughing and instability in some excavations and cut slopes. The contractor should control groundwater, surface water infiltration, and soil erosion. Water should be controlled by pumping from sumps.

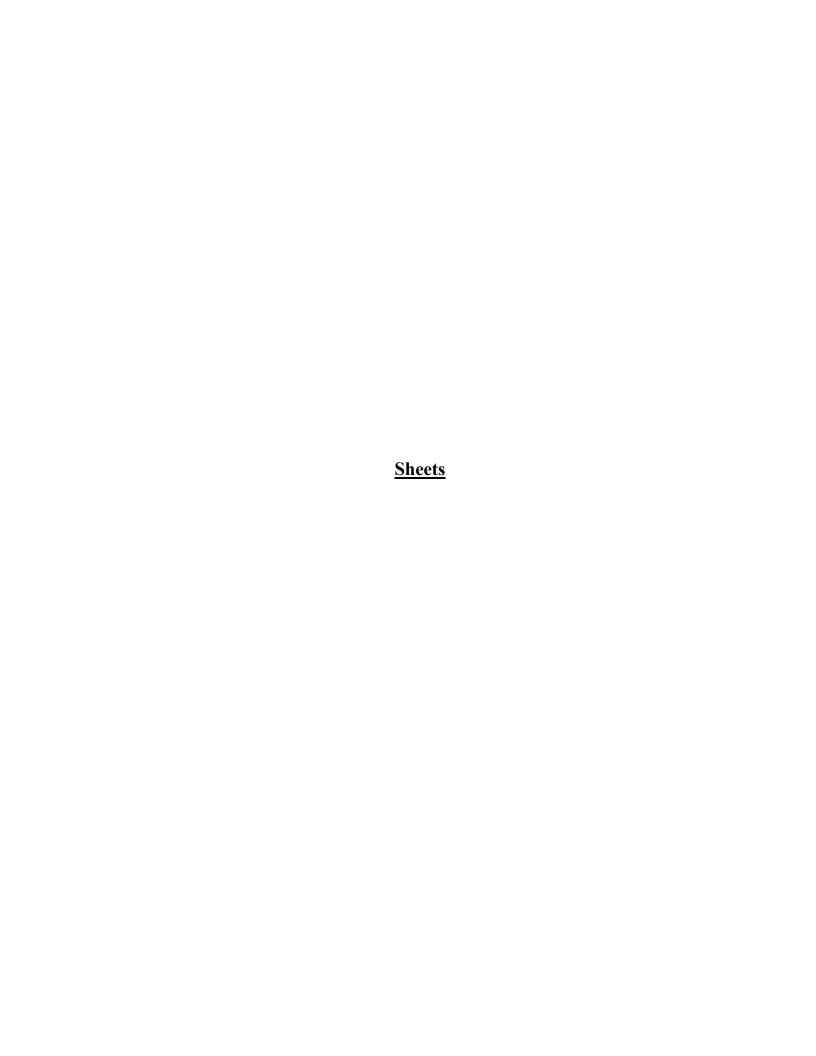
Cobbles and boulders were encountered in the glacial till deposit. There is potential for these obstructions to cause difficulties during pile driving operations. If obstructions are encountered prior to reaching the maximum required penetration resistance on bedrock, then they may be cleared by conventional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

9.0 CLOSURE

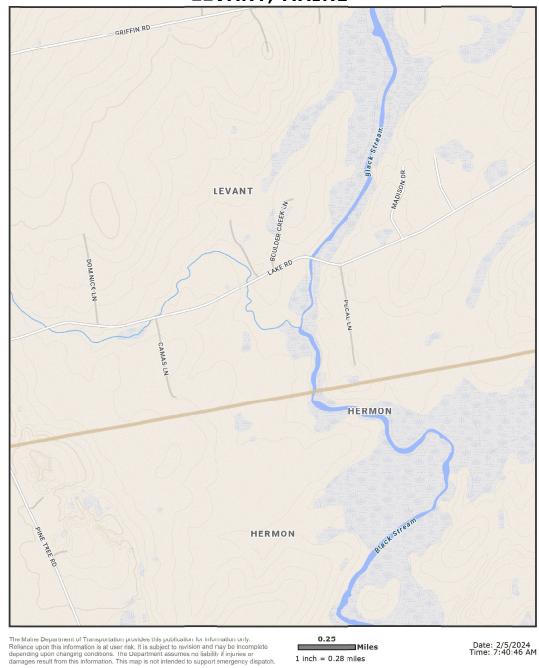
This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Perkins Bridge and Lake Bridge in Levant, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is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



LEVANT, MAINE



1 inch = 0.28 miles

SHEET NUMBER

5 OF

PERKINS AND LAKE BRIDGES BLACK STREAM

LEVANT PENOBSCOT COUNTY

LOCATION MAP

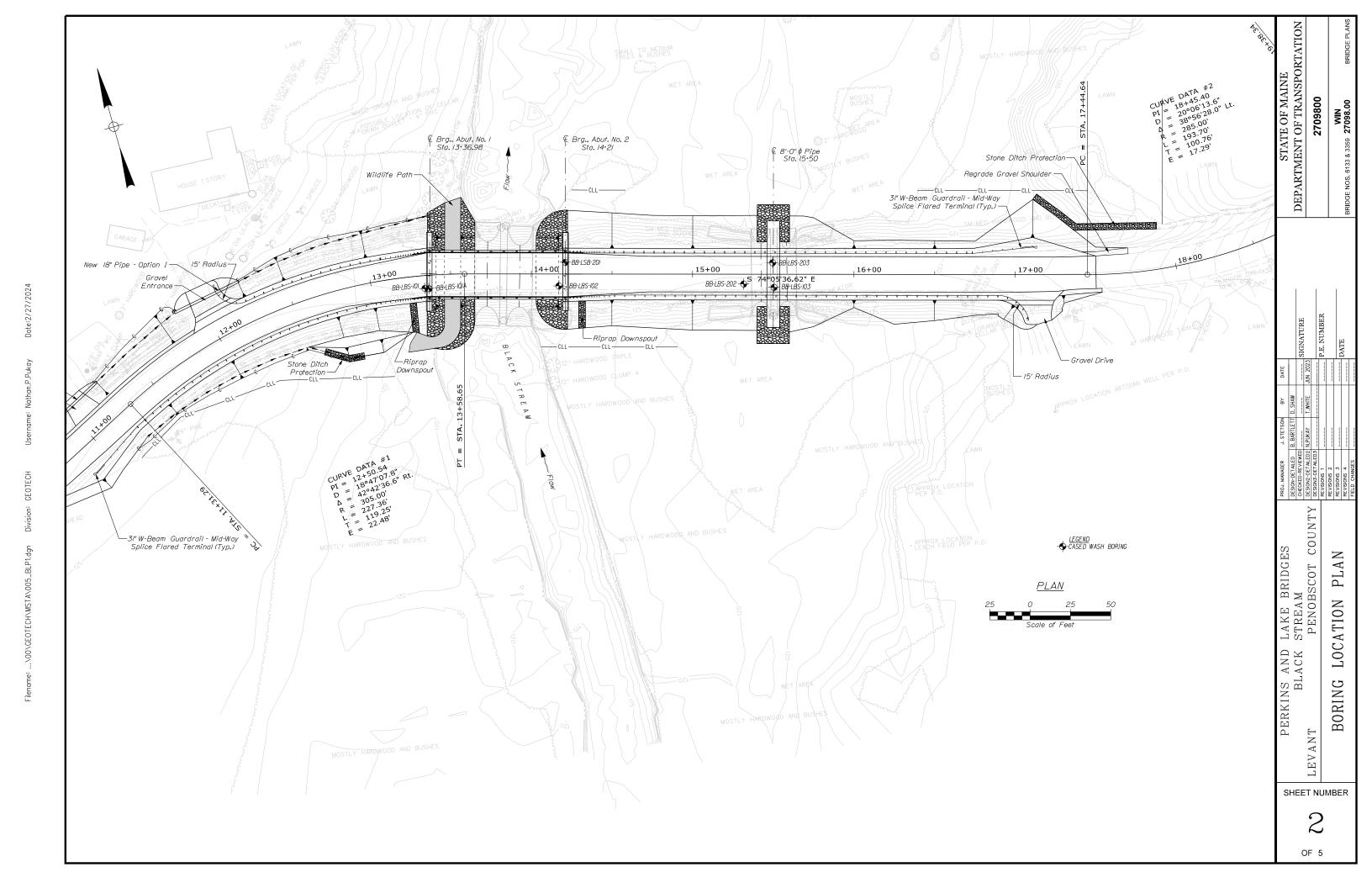
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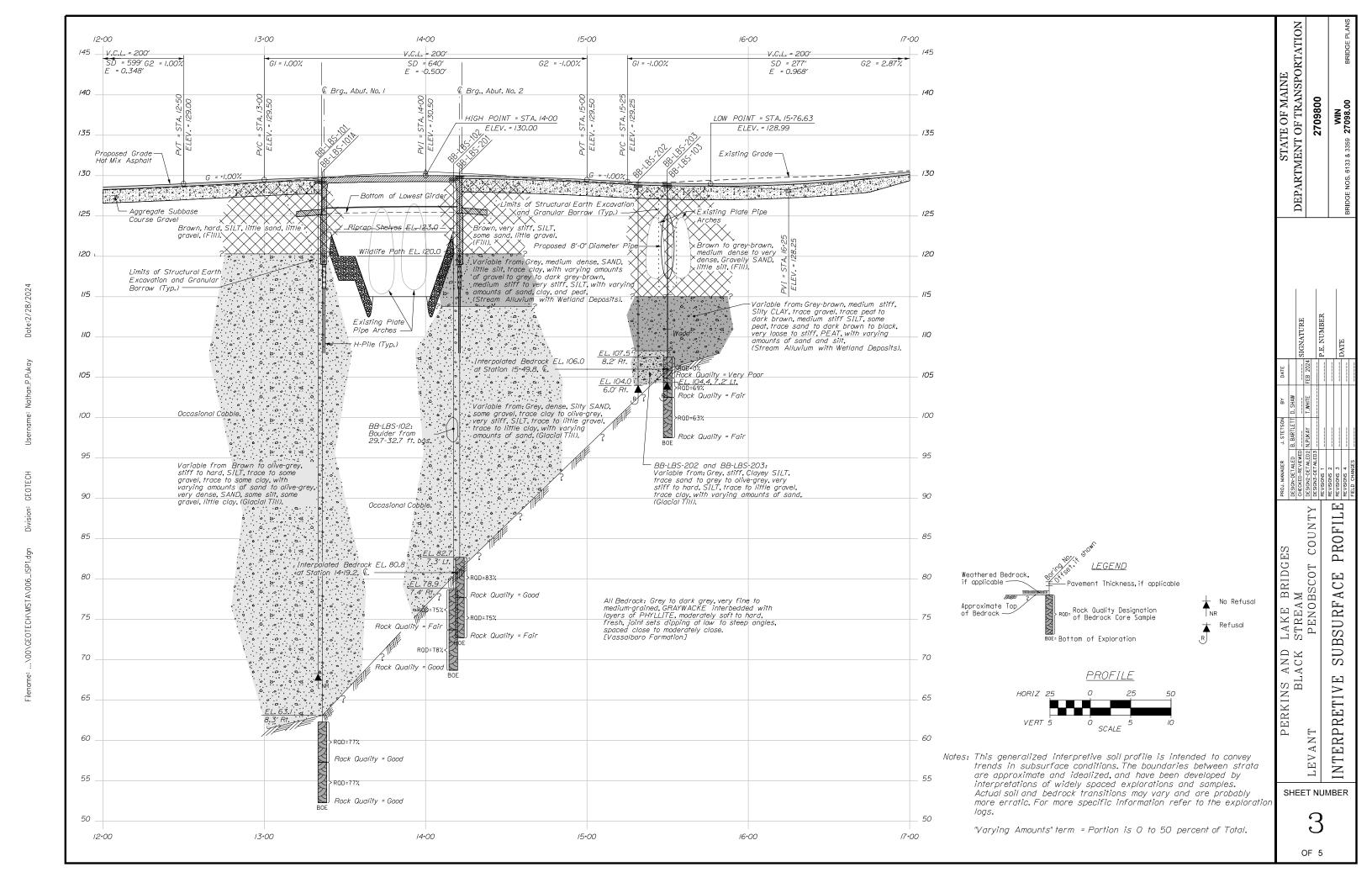
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WIN

BRIDGE NO. 6133 & 3359 **27098.00**

BRIDGE PLANS





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Date:2/28/2024

Division: GEOTECH

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BORING LOGS	
BLACK STREAM PENOBSCOT COUNTY PENOBSCOT COUNTY REVISIONS 1	5 Of 5
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Appendix A

Boring Logs

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f ma 0 sie		illies)			Cohesio	nless Soils		e (blows per foot)
(more than half of material is larger than No. 200 sieve size)	SANDS	CLEAN SANDS	sw	Well-graded sands, Gravelly sands, little or no fines	Lo Mediu	/ loose oose m Dense ense		0 - 4 5 - 10 11 - 30 31 - 50
(more	f coarse han No.	(little or no fines)	SP	Poorly-graded sands, Gravelly sand, little or no fines.		Dense <u>s</u> (more than half of m	naterial is smaller tha	> 50 n No. 200
	(more than half of coarse fraction is smaller than No. 4 sieve size)	SANDS WITH FINES	SM	Silty sands, sand-silt mixtures				Gravelly, Sandy ording to undrained shear
	(more fraction	(Appreciable amount of fines)	SC	Clayey sands, sand-clay mixtures.	Consistency of Cohesive soils	SPT N-Value (blows per foot)	Undrained Shear Strength (psf)	<u>Field</u> Guidelines
	AA 2T II2	ND CLAYS	ML	Inorganic silts and very fine sands, rock flour, Silty or Clayey fine sands, or Clayey silts with slight plasticity.	Very Soft Soft Medium Stiff	WOH, WOR, WOP, <2 2 - 4 5 - 8	0 - 250 250 - 500 500 - 1000	Fist easily penetrates Thumb easily penetrates Thumb penetrates with moderate effort
FINE- GRAINED	OIL TO AI	VD OLATO	CL	Inorganic clays of low to medium plasticity, Gravelly clays, Sandy	Stiff Very Stiff	9 - 15 16 - 30	1000 - 2000 2000 - 4000	Indented by thumb with great effort Indented by thumbnail
SOILS	(liquid limit	less than 50)	OL	clays, Silty clays, lean clays. Organic silts and organic Silty	Hard Rock Quality Des	>30	over 4000	Indented by thumbnail with difficulty
l is size)				clays of low plasticity.		sum of the lengths	of intact pieces of length of core a um NQ rock core (dvance
than half of material is than No. 200 sieve size)	SILTS AN	ND CLAYS	МН	Inorganic silts, micaceous or diatomaceous fine Sandy or Silty soils, elastic silts.		Rock Quality Ba Rock Quality Very Poor	,	,
ore than ha			СН	Inorganic clays of high plasticity, fat clays.		Poor Fair Good	26 - 50 51 - 75 76 - 90	
(mo smalle	(liquid limit gı	reater than 50)	ОН	Organic clays of medium to high plasticity, organic silts.	Color (Munsell o			able):
		ORGANIC DILS	Pt	Peat and other highly organic soils.	Rock Type (grad Hardness (very	itic, fine-grained, et nite, schist, sandsto hard, hard, mod. h sh. verv slight, sligh	one, etc.) ard, etc.)	severe, severe, etc.)
Desired So	il Observat	tions (in this	s order, if	applicable):	⊒	ntinuities/jointing:	, 25. a.5, 1115d.	, , ,
Color (Muns	sell color ch	art)				-dip (horiz - 0-5 de		deg., mod. dipping -
	ry, damp, m	oist, wet) om above ric	aht hand s	ide)				rtical - 85-90 deg.)
		coarse, etc.)		iue <i>j</i>		 -spacing (very clos close - 1-3 feet, 	e - <∠ incn, close - wide - 3-10 feet, v	
Name (San	d, Silty Sand	d, Clay, etc.,	including	portions - trace, little, etc.)		-tightness (tight, or	oen, or healed)	- /
		, poorly-grad		n, etc.) tely plastic, highly plastic)		-infilling (grain size		1
		stures, crack		iery piastic, mgmy piastic)		erville, Ellsworth, C ation to rock quality		
		ely, loosely,	,			032 and FHWA NE		
		oderate, or s		,	Site Characte	rization, Table 4-12	2	
Geologic Or Groundwate		rine clay, all	uvium, etc	·.) 	Rock Core Rate	inch and percentag e (X.X ft - Y.Y ft (mir	n:sec))	
	Maine L	Departme	nt of Tra	ansportation		ainer Labeling F		
		Geotechi		•	WIN Bridge Name	/ Town	Blow Counts Sample Recove	erv
Key	y to Soil a		Descrip	otions and Terms	Boring Number Sample Number Sample Depth	er oer	Date Personnel Initia	•

N	Aain	e Depa	artment	of Transporta	tio	n	Project:		e #6133, Lake Bridge #3359	Boring No.:	BB-LI	BS-101
			Soil/Rock Expl US CUSTOM				Locatio	carries Lake R n: Levant, Mai	toad over Black Stream ne	WIN:	2709	98.00
Drille	er:		MaineDOT		Ele	evation	l n (ft.)	129.3		Auger ID/OD:	5" Solid Stem	
Oper			Daggett/Andrl	le	+	tum:	(- /	NAVD88		Sampler:	Standard Split	Spoon
Logg	ed By:		N. Pukay		Rig	д Туре	:	CME 45C		Hammer Wt./Fall:	140#/30"	
Date	Start/Fi	inish:	6/7,12/2023		Dri	illing N	/lethod:	Cased Wash	Boring	Core Barrel:	N/A	
Boriı	ng Loca	tion:	13+33.4, 7.9 f	ì Rt.	Ca	sing II	D/OD:	NW(3.0"/3.5	5")	Water Level*:	8.0 ft bgs.	
		ciency F	actor: 0.906			mmer	Type:	Automatic 🗵	<u> </u>	Rope & Cathead		
MD = I U = Th MU = I V = Fie	lit Spoon S Jnsuccess in Wall Tu Jnsuccess eld Vane S	sful Split Spo be Sample sful Thin Wa Shear Test,	oon Sample Atten II Tube Sample A PP = Pocket Pei ne Shear Test Att	RC = Roller ttempt WOH = Wei netrometer WOR/C = W	Stem / w Sten Cone ght of 1 eight o	Auger n Auger 140lb. Ha f Rods o	r Casing	S _u (lab) = Lab q _p = Unconfin N-uncorrected Hammer Effici N ₆₀ = SPT N-	molded Field Vane Undrained She Vane Undrained Shear Strength (ig ed Compressive Strength (ksf) = Raw Field SPT N-value ency Factor = Rig Specific Annual uncorrected Corrected for Hamme er Efficiency Factor/60%)*N-uncor	osf) WC = LL = PL = Calibration Value PI = F r Efficiency G = 0	Pocket Torvane Shea Water Content, pero- Liquid Limit Plastic Limit Plasticity Index Brain Size Analysis Consolidation Test	
				Sample Information								Laboratory
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.) Graphic Log	Visual De	scription and Remarks		Testing Results/ AASHTO and Unified Class.
0							SSA	129.1	√3" HMA.		0.3-	
5 -	1D	24/17	5.00 - 7.00	15/15/15/20	30	45	34 48 25		Brown, moist, hard, SILT, I	ittle sand, little gravel, (F		G#380926 A-4, CL-ML WC=16.1
							27					
								120.3			9.0	
10 -) (D)	24/0	10.00 12.00	0/5/10/7	1.5	22	14		Medium dense.			
	MD	24/0	10.00 - 12.00	8/5/10/7	15	23	RC					
									Olive-grey, wet, SILT, som	e gravel, some sand, little	clay, (Glacial	
	2D	12/3	13.00 - 14.00	5/50(6")			7		Till).			
							35					
15 -	3D	24/10	15.00 - 17.00	8/13/12/20	25	38	41		Olive-grey, wet, hard, SILT (Glacial Till).	, some gravel, some sand	, little clay,	G#380927 A-4, CL-ML
							46					WC=10.8% LL=20
							187					PL=14 PI=6
							176					11 0
20 -							188		Olive-grey, wet, very dense	, SAND, some silt, some	gravel, little clay,	G#380928
	4D	24/12	20.00 - 22.00	12/19/19/17	38	57	RC		(Glacial Till).		,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,	A-4. CL-ML WC=10.2% LL=20 PL=14 PI=6
Rem	arks:	<u>I</u>										
				ing drilling on 6/12/2023 tole with the tip at 55.0 ft		El. 74.3). Casing	shoe measures	0.4 ft long.			

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 3

N	Tain	e Dep	artment	of Transport	atio	n	Proj	ect:			e #6133, Lake Bridge #3359	Boring No.:	BB-L	BS-101
			Soil/Rock Expl				Loc	atio	carries Leva		toad over Black Stream ne			
			US CUSTOMA	ARY UNITS						,		WIN:	270	98.00
Drille	r.		MaineDOT		TEL	evation	/ft \		129.	2		Auger ID/OD:	5" Solid Stem	
Oper			Daggett/Andrl	2	+	tum:	i (it.)			D88		Sampler:	Standard Split	Cnoon
	ed By:		N. Pukay	<u> </u>	+	g Type				E 45C		Hammer Wt./Fa		<u> Зроон</u>
	Start/F	inieh:	6/7,12/2023		_	illing N		٠d٠			Boring	Core Barrel:	N/A	
	g Loca		13+33.4, 7.9 f	t Rt	-	sing II				3.0"/3	-	Water Level*:	8.0 ft bgs.	
			actor: 0.906	i Kt.	+	mmer			Automa			Rope & Cathead	0.0 11 0gs.	
Definiti	ons:		uotor: 0.900	R = Rock C	ore Sar	nple	.,,,,		S _u =	Peak/Re	molded Field Vane Undrained She	ear Strength (psf)	T _V = Pocket Torvane She	
	lit Spoon Insucces:		oon Sample Attern	SSA = Solid npt HSA = Holl					S _{u(la}	_{o)} = Lab Jnconfin	Vane Undrained Shear Strength (p ed Compressive Strength (ksf)	psf)	WC = Water Content, per LL = Liquid Limit	cent
		ibe Sample	all Tube Sample A	RC = Roller ttempt WOH = We		140 lb. H	ammei				= Raw Field SPT N-value ency Factor = Rig Specific Annual	Calibration Value	PL = Plastic Limit PI = Plasticity Index	
V = Fie	ld Vane S	Shear Test,	PP = Pocket Per ane Shear Test Att	netrometer WOR/C = V	Veight o	f Rods o	r Casir		N ₆₀	SPT N-	uncorrected Corrected for Hamme er Efficiency Factor/60%)*N-uncor	er Efficiency	G = Grain Size Analysis C = Consolidation Test	
IVIV - C	madeces	sidi i leid va		Sample Information	eignt or	One r er	3011		1460	(Hallin	er Emiciency r actor/00/70/14-uncor	rected	C = Consolidation rest	1
Ī		<u></u>	£		pe									Laboratory Testing
<u> </u>	2	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected				_	Graphic Log	Visual De	scription and Rem	arks	Results/
Depth (ft.)	Sample No.	/Re	<u>a</u>	vs (/ ar ngth ngth	יסטר		ja j	\s	Elevation (ft.)	phic		•		AASHTO and
_ be	San	Pen	San (ft.)	Blov Stre (psf	Ž	N ₆₀	Cas	Blows	Elev (ft.)	Gra				Unified Class
25	5D	24/5	25.00 - 27.00	11/23/30/37	53	80				111111	Similar to 4D. Rock in tip of	of spoon.		
- }	30	24/3	25.00 - 27.00	11/23/30/37	- 55	- 00								
										业				
											Occasional cobble.			
ŀ														
30	6D	24/9	30.00 - 32.00	8/11/20/16	31	47					Olive-grey, wet, hard, SILT	, some sand, little g	ravel, little clay,	G#380929
- }	0.0	24/9	30.00 - 32.00	6/11/20/10	31	1					(Glacial Till).			A-4, CL-ML WC=11.5%
														LL=21
[PL=14 PI=7
ŀ											Occasional cobble.			
										4]]++				
35	7D	24/8	35.00 - 37.00	9/14/18/20	32	48					Similar to 6D.			
	,,,	20	33.00 37.00	3/11/10/20		10				I#. f.				
											Occasional cobble.			
ŀ														
40	8D	24/2	40.00 - 42.00	12/13/14/36	27	41					Olive-grey, wet, hard, SILT (Glacial Till). Rock in tip o		ravel, trace clay,	
ŀ											(Glaciai Tili). Rock ili tip o	r spoon.		
ļ														
ŀ											Occasional cobble.			
- }			+ +											
. , .]										HJ.K				
45	9D	24/17	45.00 - 47.00	23/38/35/38	73	110				\$ 1	Olive-grey, wet, hard, SILT (Glacial Till).	, some sand, little g	ravel, trace clay,	
ŀ							+				Occasional cobble.			
Į														
ŀ														
50														
Rema	arks:						•							•
1) W	ater lev	el measure	ed before resum	ing drilling on 6/12/202	3.									

2) NW sized steel casing shoe left in hole with the tip at 55.0 ft bgs (El. 74.3). Casing shoe measures 0.4 ft long.

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

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

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N	Main	e Depa	artment	of Transport	ation		Project:				e #6133, Lake Bridge #3359	Boring No.:	BB-L	BS-101
Soil/Rock Exploration Log US CUSTOMARY UNITS							Locatio				oad over Black Stream ne		250	20.00
		Ī	JS CUSTOM.	ARY UNITS								WIN:	2709	98.00
Drille	er:		MaineDOT		Elevat	tion	(ft.)	129	.3			Auger ID/OD:	5" Solid Stem	
Oper	ator:		Daggett/Andr	le	Datum	n:		NA	VD88	8		Sampler:	Standard Split	Spoon
Logg	jed By:		N. Pukay		Rig Ty	/pe:		CM	E 45	С		Hammer Wt./Fall:	140#/30"	•
	Start/Fi	inish:	6/7,12/2023		Drillin	g M	lethod:	Cas	ed W	/ash	Boring	Core Barrel:	N/A	
Borii	ng Loca	ition:	13+33.4, 7.9 1	ft Rt.	Casing	_		NW	(3.0"	"/3.5	(")	Water Level*:	8.0 ft bgs.	
Ham	mer Effi	iciency F	actor: 0.906		Hamm	ner '	Туре:	Autom	atic 🏻	×	Hydraulic □	Rope & Cathead □		
MD = I U = Th MU = I V = Fie	olit Spoon of Unsuccess in Wall Tu Unsuccess old Vane S	sful Split Spo ube Sample sful Thin Wa Shear Test,	oon Sample Atter II Tube Sample A PP = Pocket Pe ne Shear Test At	SSA = Soli MSA = Holl RC = Rolle WOH = We enetrometer WOR/C = N	core Sample d Stem Auge ow Stem Aug r Cone eight of 140 lb Veight of One	er ger b. Ha ds or	r Casing	S _{u(la} q _p = N-ur Ham N ₆₀	ab) = I Unco ncorre nmer E = SP1	Lab \ onfine ected Efficie T N-u	molded Field Vane Undrained She Vane Undrained Shear Strength (ksf) = Raw Field SPT N-value ency Factor = Rig Specific Annual uncorrected Corrected for Hamme er Efficiency Factor/60% 'N-uncor	osf)	= Pocket Torvane She = Water Content, pen = Liquid Limit = Plastic Limit : Plasticity Index Grain Size Analysis Consolidation Test	
				Sample Information					4					Laboratory
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)	ranhir		Visual De:	scription and Remarks	:	Testing Results/ AASHTO and Unified Class
50	10D	24/17	50.00 - 52.00	31/30/28/40	58	88				W	Similar to 9D, (Glacial Till)).		
- 55 -	11D	24/17	55.00 - 57.00 60.00 - 61.00	30/43/54/66 47/95(6")		46	OPEN	68.3		The state of the s	Cobble from 54.2-54.8 ft bg Olive grey, wet, hard, SILT (Glacial Till). Steel cuttings in wash at 55. Olive-grey, wet, Sandy SIL Bottom of Exploration HOLE ABANDONED, NW in hole with the tip at 55.0 ft long. Moved to BB-LBS-	T, little gravel, trace clay at 61.0 feet below group is ized steel casing shoe it bgs (El. 74.3). Casing	y, (Glacial Till). 61.0 und surface. broke off and left	
- 65 -														
- 70 -														
								1						
75 Rem	arks:	<u> </u>						<u> </u>	1					
		el messure	d before resum	ning drilling on 6/12/202	3									
				nole with the tip at 55.0		74.3). Casing	shoe m	easu	res (0.4 ft long.			

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

than those present at the time measurements were made.

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

Page 3 of 3

N	Maine	_		of Transport	atio	n	Pro	ject:			ge #6133, Lake Bridge #3359 Road over Black Stream	Boring No.:	BB-LE	BS-101A
			Soil/Rock Exp US CUSTOM.				Loc	atio	n: Lev			WIN:	270	98.00
Drille	er:		MaineDOT		Ele	vatior	ı (ft.))	129	.3		Auger ID/OD:	5" Solid Stem	
Oper	rator:		Daggett/Andr	le	Dat	tum:			NA'	VD88		Sampler:	Standard Split	Spoon
Logg	ged By:		N. Pukay		Rig	ј Туре	:		CM	E 45C		Hammer Wt./Fall:	140#/30"	
	Start/Fi	nish:	6/13-14/2023		Dri	lling N	/leth	od:	Cas	ed Was	h Boring	Core Barrel:	NQ-2"	
Bori	ng Loca	tion:	13+36, 8.3 ft	Rt.	Cas	sing II	D/OD):	NW	(3.0"/3	.5")	Water Level*:	8.0 ft bgs.	
			actor: 0.906			mmer			Autom		Hydraulic □	Rope & Cathead □		
Definit D = Sp MD = U = Th MU = V = Fie	tions: olit Spoon S Unsuccess nin Wall Tu Unsuccess eld Vane S	Sample sful Split Sp be Sample sful Thin Wa	oon Sample Atter	RC = Rolle Attempt WOH = We enetrometer WOR/C = 1	id Stem A low Stem er Cone eight of 14 Weight of	Auger n Auger 40lb. Ha f Rods o	r Casi		S _{u(la} q _p = N-un Ham N ₆₀	ab) = Lal Unconfi correcte mer Effi = SPT N	emolded Field Vane Undrained Sh Vane Undrained Shear Strength (hed Compressive Strength (ksf) d = Raw Field SPT N-value ciency Factor = Rig Specific Annua -uncorrected Corrected for Hamm- ner Efficiency Factor/60%) "N-uncor	ear Strength (psf) $T_V =$ (psf) WC LL = PL = Il Calibration Value PI = er Efficiency G =	Pocket Torvane She = Water Content, per Liquid Limit = Plastic Limit Plasticity Index Grain Size Analysis Consolidation Test	
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	asina	Blows	Elevation (ft.)	Graphic Log	Visual De	escription and Remarks		Laboratory Testing Results/ AASHTO and Unified Clas
0	ιχ	ď) ÿ €	ಹಹಹಲ್	Ż	ž			□ € 129.1	ڻ د د د د	¬ 3" HMA.			
- 5 -	S1 MD	12/4	9.00 - 10.00	7/7/8/7		23		SA	129.1		3" HMA. Reference BB-LBS-101 for samples, and casing blow of samples and casing blow of sample off flight sand, trace gravel, (Glacial 9.0-10.0 ft bgs.	ts): Brown, wet, SILT, so	——————————————————————————————————————	G#380930
- 15 -	MD	24/0	10.00 - 12.00	11110/1	15	23	N N	W			Stiff. Occasional cobble.			
								/						
- 20 -							+	¥	1		3			
								EN LE		淵紅	4			
											1			
							+		1		1			
							+		1		•			
25								\bigvee	<u> </u>					
	arks:											T		
		-	• •	ndaries between soil types; nes and under conditions sta		-	-		ns mav d	occur du	e to conditions other	Page 1 of 4		

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

N	Maine	e Dep	artment	of Transport	atio	n	Proj	ject:			e #6133, Lake Bridge #3359	Boring No.:	BB-LB	S-101A
Soil/Rock Exploration Log US CUSTOMARY UNITS							Loc	atio		s Lake I ant, Ma	Road over Black Stream ine	WIN:	2709	98.00
												*****		70.00
Drille			MaineDOT			vatior	າ (ft.)		129.			Auger ID/OD:	5" Solid Stem	
	ator:		Daggett/And	rle	_	tum:				/D88		Sampler:	Standard Split	Spoon
	ged By:		N. Pukay			Type				E 45C	- ·	Hammer Wt./Fall:	140#/30"	
	Start/Fi		6/13-14/2023		$\overline{}$	illing N					Boring	Core Barrel:	NQ-2"	
	ng Loca		13+36, 8.3 ft		_	sing II				(3.0"/3.		Water Level*:	8.0 ft bgs.	
Definiti		ciency i	actor: 0.906	R = Rock		mmer	туре) :	Autom:		Hydraulic □ molded Field Vane Undrained She	Rope & Cathead ear Strength (psf) T _v =	Pocket Torvane She	ar Strength (psf)
D = Sp MD = U U = Th MU = U V = Fie	olit Spoon S Unsuccess nin Wall Tu Unsuccess eld Vane S	ful Split Sp be Sample ful Thin Wa hear Test,	oon Sample Atte all Tube Sample a PP = Pocket Pa ane Shear Test A	SSA = Sol Mpt	lid Stem A llow Stem er Cone eight of 1 Weight of	Auger n Auger 40 lb. Ha f Rods o	r Casir		S _{u(la} q _p = N-un Ham N ₆₀	lb) = Lab Unconfir corrected mer Effic = SPT N-	Vane Undrained Shear Strength (ed Compressive Strength (ksf) = Raw Field SPT N-value ency Factor = Rig Specific Annua uncorrected Corrected for Hamme er Efficiency Factor/60%)*N-unco	psf)	= Water Content, per = Liquid Limit = Plastic Limit Plasticity Index Grain Size Analysis Consolidation Test	
-				Sample Information	T _	_	_			-				Laboratory
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing	Blows	Elevation (ft.)	Graphic Log	Visual De	scription and Remarks		Testing Results/ AASHTO and Unified Clas
25							SP	UN W			Pulled NW casing to install casing to 30.0 ft bgs and ro			
20											Occasional cobble.			
30 -														
35 -														
· 40 -											Spun casing to 45.0 ft bgs,	then roller coned ahead t	o 57.0 ft bgs.	
45 -														
ŀ							+							
50 Rema	arks:													<u> </u>
Stratific	cation line	s represent	approximate boo	undaries between soil types	; transitio	ns may b	oe grad	dual.				Page 2 of 4		
		-	been made at tir	mes and under conditions st nts were made.	ated. Gro	oundwat	er fluct	tuation	ns may o	ccur due	to conditions other	Boring No	.: BB-LBS-	101A

I	Main	e Dep	artment	of Transporta	ation		Projec				#6133, Lake Bridge #3359	Boring No.:	BB-LB	S-101A
			Soil/Rock Expl US CUSTOM/				Locati			Lake R nt, Mai	oad over Black Stream ne	WIN:	2709	98.00
Drill	er:		MaineDOT		Elevat	tion	(ft.)		129.3	3		Auger ID/OD:	5" Solid Stem	
-	rator:		Daggett/Andrl	e	Datum		()		NAV			Sampler:	Standard Split	Spoon
Log	ged By:		N. Pukay		Rig Ty	/pe:			CME	45C		Hammer Wt./Fall:	140#/30"	
Date	Start/Fi	inish:	6/13-14/2023		Drillin	g M	ethod:		Case	d Wash	Boring	Core Barrel:	NQ-2"	
Bori	ng Loca	tion:	13+36, 8.3 ft I	Rt.	Casin	g ID	/OD:		NW(3.0"/3.5	")	Water Level*:	8.0 ft bgs.	
		ciency F	actor: 0.906		Hamm		Гуре:	A	Automa			Rope & Cathead □		
MD = U = TI MU = V = Fi	olit Spoon of Unsuccess in Wall Tu Unsuccess eld Vane S	sful Split Sp be Sample sful Thin Wa Shear Test,	all Tube Sample A PP = Pocket Pe ne Shear Test Att	RC = Roller ttempt	Stem Auge ow Stem Aug Cone ght of 140 la /eight of Roo	er ger b. Ha ds or	Casing		S _{u(late} q _p = t N-und Hamn N ₆₀ =) = Lab Inconfine orrected ner Efficie SPT N-I	molded Field Vane Undrained She vane Undrained Shear Strength (de Compressive Strength (ksf) = Raw Field SPT N-value ency Factor = Rig Specific Annual uncorrected Corrected for Hamme er Efficiency Factor/60%)*N-uncor	psf) WC :	Pocket Torvane She = Water Content, per Liquid Limit Plastic Limit Plasticity Index Grain Size Analysis Consolidation Test	
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (pst) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows		Elevation (ft.)	Graphic Log	Visual De	scription and Remarks		Laboratory Testing Results/ AASHTO and Unified Class.
50				20000										
- 55 -														
- 65 -														
							ОН	+	63.1				66.2 ⁻	
	R1	60/58	67.00 - 72.00	RQD = 77%			NQ-2	2			Top of Bedrock at Elev. 63 Roller coned ahead to 67.0 R1: Bedrock: Grey to dark GRAYWACKE, interbedde moderately hard, competen spaced moderately close, w	ft bgs. grey, very fine to mediun ed with abundant layers o t, fresh, joint set dipping	f PHYLLITE, at low angles,	
- 70 -	R2	60/58	72.00 - 77.00	RQD = 77%							spaced moderatery close, w [Vassalboro Formation] Rock Quality = Good R1: Core Times (min:sec) 67.0-68.0 ft (2:10) 68.0-69.0 ft (2:00) 69.0-70.0 ft (1:53) 70.0-71.0 ft (2:01)	nn quanz or carche anne:	aicu nactures.	
											71.0-72.0 ft (2:33) 97% Recovery			
7.											R2: Bedrock: Similar to R1			
75 Rem	arks:	<u>I</u>								EII.Ne				1
		•		ndaries between soil types; t		-	-		s may oc	cur due	o conditions other	Page 3 of 4		

than those present at the time measurements were made.

N	Iaine			of Transpor	tation	1	Project:			e #6133, Lake Bridge #3359 toad over Black Stream	Boring No.:	BB-LB	8S-101A
			Soil/Rock Exp US CUSTOM				Locatio				WIN:	2709	98.00
Drille	r:		MaineDOT		Elev	/ation	(ft.)	129.	3		Auger ID/OD:	5" Solid Stem	
Opera	ator:		Daggett/Andr	le	Datu	um:	. ,	NA	VD88		Sampler:	Standard Split	Spoon
Logg	ed By:		N. Pukay		Rig	Type	:	CM	E 45C		Hammer Wt./Fall:	140#/30"	
Date :	Start/Fi	nish:	6/13-14/2023		Drill	ling N	lethod:	Case	ed Wasl	Boring	Core Barrel:	NQ-2"	
Borin	g Locat	ion:	13+36, 8.3 ft	Rt.	Casi	ing IC)/OD:	NW	(3.0"/3.	5")	Water Level*:	8.0 ft bgs.	
		ciency F	actor: 0.906		Ham	nmer	Туре:	Autom		Hydraulic □	Rope & Cathead		
MD = U U = Thi MU = U V = Fiel	lit Spoon S Insuccesst in Wall Tub Insuccesst Id Vane St	ful Split Spo be Sample ful Thin Wa hear Test,	oon Sample Atter Il Tube Sample A PP = Pocket Pe ne Shear Test At	SSA = Sc MPT	Veight of 140 Weight of F Weight of O	uger Auger 0 lb. Ha Rods or	r Casing	S _{u(la} q _p = N-un Ham N ₆₀	ab) = Lab Unconfir corrected mer Effic = SPT N	molded Field Vane Undrained Shk Vane Undrained Shear Strength (ed 6d Compressive Strength (ksf) = Raw Field SPT N-value ency Factor = Rig Specific Annua uncorrected Corrected for Hamme er Efficiency Factor/60%)*N-unco	psf) WC	Pocket Torvane She = Water Content, per Liquid Limit : Plastic Limit Plasticity Index Grain Size Analysis Consolidation Test	
ŀ				Sample Information				1	1				Laboratory
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)	Graphic Log	Visual De	scription and Remarks		Testing Results/ AASHTO and Unified Clas
75								52.3		[Vassalboro Formation] Rock Quality = Good R2: Core Times (min:sec) 72.0-73.0 ft (3:59) 73.0-74.0 ft (6:19) 74.0-75.0 ft (6:06) 75.0-76.0 ft (5:10)			
}										76.0-77.0 ft (4:39) 97% Recovery		77.0	_
80										Bottom of Exploration	n at 77.0 feet below grou	ind surface.	
-													
-													
85													
-													
-													
90													
-													
-													
95													
-													
-													
			1					I					Ī

N	Aain	-		of Transporta	tion		Project:		e #6133, Lake Bridge #3359	Boring No.:	BB-LI	BS-102
			Soil/Rock Expl US CUSTOMA	-			Locatio	: Levant, Ma	Road over Black Stream ine	WIN:	2709	08.00
Drille	r:		MaineDOT		Eleva	atior	n (ft.)	129.7		Auger ID/OD:	5" Solid Stem	
Oper	ator:		Daggett/Andrl	e	Datu	m:		NAVD88		Sampler:	Standard Split	Spoon
Logg	ed By:		N. Pukay		Rig 1	ype	:	CME 45C		Hammer Wt./Fall:	140#/30"	
Date	Start/Fi	nish:	6/5/2023-6/6/2	2023	Drilli	ng N	/lethod:	Cased Wash	Boring	Core Barrel:	NQ-2"	
Borir	ng Loca	tion:	14+17.2, 7.4 ft	t Rt.	Casi	ng II	D/OD:	HW(4.0"/4.	5"), NW(3.0"/3.5")	Water Level*:	10.0 ft bgs.	
		ciency F	actor: 0.906				Туре:	Automatic 🗵		Rope & Cathead		
MD = U U = Th MU = U V = Fie	lit Spoon S Jnsuccess in Wall Tu Jnsuccess eld Vane S	ful Split Spo be Sample ful Thin Wa hear Test,	oon Sample Attem all Tube Sample A PP = Pocket Per ine Shear Test Att	RC = Roller C ttempt WOH = Weig netrometer WOR/C = We	Stem Aug v Stem A Cone ht of 140 eight of R	jer uger lb. Ha ods o	r Casing	S _{u(lab)} = Lab q _p = Unconfin N-uncorrected Hammer Effic N ₆₀ = SPT N-	imolded Field Vane Undrained She Vane Undrained Shear Strength (pe ed Compressive Strength (ksf) = Raw Field SPT N-value iency Factor = Rig Specific Annual uncorrected Corrected for Hamme en Efficiency Factor/60%)*N-uncor	osf) William LL PL Calibration Value PI or Efficiency G	= Pocket Torvane She: C = Water Content, perce Liquid Limit = Plastic Limit = Plasticity Index = Grain Size Analysis = Consolidation Test	
ŀ				Sample Information			1					Laboratory
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.) Graphic Log	Visual De:	scription and Remark	s	Testing Results/ AASHTO and Unified Class.
0							SSA	129.4	4" HMA.		-0.3-	
									Occasional cobble.		0.5	
_												
5	1D	24/17	5.00 - 7.00	14/8/10/12	18	27	38		Brown, moist, very stiff, SI cobble in spoon. Set NW casing at 5.0 ft bgs.		avel, (Fill). Broken	
							47		Set IV W casing at 3.0 it ogs.	•		
-							48					
							55					
· 10							3	119.7			10.0	
	2D	24/12	11.00 - 13.00	4/2/5/3	7	11	10		Grey, wet, medium dense, S organics, (Stream Alluvium			G#380931 A-2-4, SM WC=36.3%
-							15		Clay in wash from 13.0-14.	0 ft bgs.		
ŀ			+ +		+				Wash went from grey to dar	rk brown from 14.0-14.	5 ft bgs.	
· 15 -	3D V1	24/10	15.00 - 17.00 15.00 - 16.00	1/5/14/30 Su >4181 psf	19	29	OPEN HOLE	113.7	Grey, wet, medium dense, C (Stream Alluvium with Wet 16x32 mm vane raw torque V1: >26.6 in-lbs.	tland Deposits).	ilt, trace clay, ———————————————————————————————————	G#380932 A-1-b, SC-SM WC=5.4% Non-Plastic
· 20 -												
20 7	4D	24/8	20.00 - 22.00	4/4/7/10	11	17	31		Olive-grey, wet, very stiff, state (Glacial Till).	SILT, some sand, little	clay, trace gravel,	G#380933 A-4, CL-ML WC=22.7%
							38					LL=20 PL=15
							54					PI=5
							81					
_25							136					
Rema	arks:											
1) W	ater leve	el measure	ed 6/6/2023 befo	ore resuming drilling.								

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

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

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Maine Department of Transportation Soil/Rock Exploration Log						1	Project:	Perkins Bridge		Boring No.:	BB-LI	BS-102
			Soil/Rock Expl US CUSTOM/				Location	carries Lake R 1: Levant, Mai	oad over Black Stream ne	WIN:	2709	98.00
D. 111.			W: DOT		1		(51.)	120.7		A	511.0 11.10	
Oper			MaineDOT		_	vation um:	ι (π.)	129.7 NAVD88		Auger ID/OD: Sampler:	5" Solid Stem	Cm a am
•	ed By:		Daggett/Andrl N. Pukay	е	+ -	Type		CME 45C		Hammer Wt./Fall:	Standard Split 140#/30"	Spoon
	Start/Fi	nish:	6/5/2023-6/6/2	2023	+		lethod:	Cased Wash	Boring	Core Barrel:	NQ-2"	
	ıg Loca		14+17.2, 7.4 f		_	sing IE			5"), NW(3.0"/3.5")	Water Level*:	10.0 ft bgs.	
Hamı	ner Effi	ciency F	actor: 0.906		Har	nmer	Туре:	Automatic ⊠	Hydraulic □	Rope & Cathead □		
MD = U U = Th MU = U V = Fie	lit Spoon S Jnsuccess in Wall Tu Jnsuccess eld Vane S	ful Split Spo be Sample ful Thin Wa hear Test,	oon Sample Atten ill Tube Sample A PP = Pocket Pe ne Shear Test Att	RC = Roller ttempt	Stem A w Stem Cone ght of 14 eight of	Auger Auger 40 lb. Ha Rods o	r Casing	S _{u(lab)} = Lab q _p = Unconfine N-uncorrected Hammer Effici N ₆₀ = SPT N-	molded Field Vane Undrained She Vane Undrained Shear Strength (p. 25 d Compressive Strength (ksf) = Raw Field SPT N-value ency Factor = Rig Specific Annual uncorrected Corrected for Hamme er Efficiency Factor/60%)*N-uncor	osf) WC LL PL Calibration Value PI: r Efficiency G =	= Pocket Torvane She. > Water Content, pere = Liquid Limit = Plastic Limit = Plasticity Index = Grain Size Analysis = Consolidation Test	
ł				Sample Information	-5							Laboratory
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.) Graphic Log	Visual De:	scription and Remark	S	Testing Results/ AASHTO and Unified Class.
25	5D	24/6	25.00 - 27.00	6/8/12/15	20	30	77		Similar to 4D.			
							90					
							86					
							97					
. 30							159		Boulder from 29.7-32.7 ft b		der. Pulled NW	
							OPEN HOLE		casing and installed HW cas	sing.		
. 35									Olive-grey, wet, very stiff,	SILT same sand little	rraval traca alay	
	6D	24/8	35.00 - 37.00	15/11/6/10	17	26			(Glacial Till).	one sand, nuic g	graver, trace cray,	
-												
ļ												
40	7D	24/9	40.00 - 42.00	8/10/9/11	19	29			Similar to 6D.			
-							\ /		Occasional Cobble.			
ŀ							$+ \parallel / \parallel$					
45	8D	24/15	45.00 - 47.00	11/6/9/24	15	23	41		Olive-grey, wet, very stiff, S	SILT, some sand, little g	gravel, (Glacial	G#380934
ŀ	30	2,713	77.00	(V) 21 A T			66		Till). Artisan water pressure at 45 Hole collapsed back to 40.0		NW casing to	A-4, CL-ML WC=13.7%
ŀ							131		telescope through boulder a		z caomg to	
ŀ							149					
. 50							164					
Rema	arks:											-
1) W	ater leve	el measure	ed 6/6/2023 befo	ore resuming drilling.								

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

•....

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

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N	Tain	e Depa	artment	of Tran	sporta	tion		Proje	ct:			ge #6133, Lake Bridge #3359	Boring No.:	BB-L	BS-102
			Soil/Rock Expl JS CUSTOMA		-			Loca	tior	carries		Road over Black Stream ine	WIN:	2709	98.00
Drille	· · ·		MaineDOT			Flov	ation	/ft \		129.	7		Auger ID/OD:	5" Solid Stem	
Oper			Daggett/Andrl	la.		Datu		(11.)			/ /D88		Sampler:	Standard Split	Cusan
<u> </u>				le		+					E 45C		Hammer Wt./Fall	•	Spoon
	ed By: Start/Fi	nich	N. Pukay 6/5/2023-6/6/2	2022		-	Type: ing M		۸.			n Boring	Core Barrel:	NQ-2"	
	ng Loca		14+17.2, 7.4 f			-	ing ID					5"), NW(3.0"/3.5")	Water Level*:	10.0 ft bgs.	
			actor: 0.906	t IXt.		_	mer			Automa			Rope & Cathead	10.0 it 0gs.	
Definiti D = Sp MD = U U = Th MU = U V = Fie	ons: lit Spoon S Jnsuccess in Wall Tu Jnsuccess ld Vane S	Sample sful Split Spo be Sample sful Thin Wa Shear Test,	oon Sample Atten II Tube Sample A PP = Pocket Pe ne Shear Test Att	ttempt netrometer	R = Rock Co SSA = Solid HSA = Hollo RC = Roller WOH = Wei WOR/C = W WO1P = We	ore Samp Stem Au w Stem A Cone ght of 140 eight of F	le ger Auger O Ib. Ha Rods or	mmer Casing		S _u = S _{u(la} q _p = N-un Hami N ₆₀ :	Peak/R b) = Lat Unconfi correcte mer Effi = SPT N	emolded Field Vane Undrained She Vane Undrained Shear Strength (ksf) and Compressive Strength (ksf) at Raw Field SPT N-value eiency Factor = Rig Specific Annual -uncorrected Corrected for Hamuel ere Efficiency Factor/60%)*N-uncor	ear Strength (psf) psf) Calibration Value er Efficiency	T _V = Pocket Torvane She WC = Water Content, per LL = Liquid Limit PL = Plastic Limit Pl = Plastict Jindex G = Grain Size Analysis C = Consolidation Test	cent
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength	(psf) or RQD (%)	N-uncorrected	N ₆₀	Casing	Blows	Elevation (ft.)	Graphic Log		scription and Rema		Laboratory Testing Results/ AASHTO and Unified Class.
50	9D	9.6/5	50.00 - 50.80	13/50(3.	.6")			130	0	78.9		Olive-grey, wet, SILT, som Till). Bedrock in tip of spoo		race clay, (Glacial	
	R1	60/60	51.00 - 56.00	RQD = 7	75%			NQ	-2	76.9		Top of Bedrock at Elev. 78 Roller Coned ahead to 51.0 R1: Bedrock: Grey to dark GRAYWACKE and thinly hard, fresh, joint sets dippin	.9 ft. ft bgs. grey, fine to medium- bedded PHYLLITE, ng at low to steep ang	moderately soft to	
- 55 -	R2	60/60	56.00 - 61.00	RQD = 7	78%							with quartz or calcite annea [Vassalboro Formation] Rock Quality = Fair R1: Core Times (min:sec) 51.0-52.0 ft (1:36) 52.0-53.0 ft (1:36)	led fractures.		
-												53.0-54.0 ft (1:35) 54.0-55.0 ft (1:34) 55.0-56.0 ft (1:57) 100% Recovery			
- 60 -									_	68.7		R2: Bedrock: Similar to R1 [Vassalboro Formation] Rock Quality = Good R2: Core Times (min:sec) 56.0-57.0 ft (2:04) 57.0-58.0 ft (2:28) 58.0-59.0 ft (2:23) 59.0-60.0 ft (2:28) 60.0-61.0 ft (3:53) 100% Recovery Bottom of Exploration		61.0	
- 65 -															
- 70 -															
75 Rema	arko:														<u> </u>
1) W	ater leve		d 6/6/2023 before			ansitions	may h	e gradı	ıal				Page 3 of 3		

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

N	Main	e Depa	artment	of Transport	atio	n	Project:			ge #6133, Lake Bridge #3359	Boring No.:	BB-LI	BS-103
			Soil/Rock Expl JS CUSTOMA				Location			Road over Black Stream ine	WIN:	2709	98.00
Drille	\r.		MaineDOT		T EI	evation	/# \	128.	0		Auger ID/OD:	5" Solid Stem	
Oper			Daggett/Andrl	2	-	tum:	i (it.)		VD88		Sampler:	Standard Split	Snoon
<u> </u>				<u>e</u>	+						-		Зроон
	ed By:		N. Pukay	0.12.00	-	g Type			E 45C	I.D. '	Hammer Wt./Fall:	140#/30"	
_	Start/Fi		6/7/2023; 08:0		-		lethod:			h Boring	Core Barrel:	NQ-2"	
_	ng Loca		15+50.4, 8.2 f	t Rt.	_	sing II		NW			Water Level*:	7.0 ft bgs.	
Definit D = Sp MD = I U = Th MU = I V = Fie	ions: olit Spoon Unsuccess in Wall Tu Unsuccess eld Vane S	Sample sful Split Spo lbe Sample sful Thin Wa Shear Test,	actor: 0.906 con Sample Atten Ill Tube Sample A PP = Pocket Per ne Shear Test Att	RC = Rolle ttempt WOH = We netrometer WOR/C = \	Core San d Stem / ow Sten r Cone eight of 1 Veight o	Auger n Auger 140lb. Ha of Rods o	mmer r Casing	S _{u(la} q _p = N-un Ham N ₆₀	Peak/Reab) = Lab Unconfine Correcte mer Effice = SPT N	Hydraulic □ modeld Field Vane Undrained She b Vane Undrained She b Vane Undrained Shear Strength (in ed Compressive Strength (ksf) d = Raw Field SPT N-value ciency Factor = Rig Specific Annual l-uncorrected Corrected for Hammen er Fficiency Factor/60%)*N-uncor 1	psf) W L P Calibration Value P er Efficiency G	V = Pocket Torvane Sheav VC = Water Content, pero L = Liquid Limit L = Plastic Limit I = Plasticity Index = Grain Size Analysis = 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 De	scription and Remar	ks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	128.5	***	√3½" HMA.		0.2	
- 5 -	1D 2D	24/14	5.00 - 7.00	13/13/8/9	21	32	44 35 31 30 33 25 49			Brown, moist, dense, Grave Brown, wet, medium dense		e silt, (Fill).	
	3D	24/1	14.00 - 16.00	1/1/2/2	3	5	25	115.2		Wood in wash at 13.6 ft bg. Dark brown, wet, medium s		13.6-	
- 15 -							35			Wetland Deposits). Roller Coned ahead to 16.0	ft bgs.		
	4D	24/1	16.00 - 18.00	2/2/3/3	5	8	19			Dark brown, wet, loose, Sil (Stream Alluvium with We		el, trace sand,	
							36			Wood chips and occasional wash.	dark brown coloring (presumed peat) in	
							30						
- 20 -	5D	14.4/1	20.00 - 21.20	4/4/50(2.4")			25	107.5	0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0	Dark brown, wet, WOOD, Alluvium with Wetland De		trace sand, (Stream	
	R1	31.2/18	21.30 - 23.90	RQD = 0%			a60 NQ-2	107.5		Top of Bedrock at Elev. 10	7.5 ft	21.3-	
25	R2	28.8/27	23.90 - 26.30	RQD = 69%						R1: Bedrock: Grey to dark GRAYWACKE interbedde moderately hard, fresh, thin spaced, with small quartz o [Vassalboro Formation]	grey, fine to medium-g d with PHYLLITE, mo ly bedded, steeply dip	oderately soft to ping joint set, closely	
Rem		el measure	d immediately	after drilling.						, accepted to manual			

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

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

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I	Maine	e Depa	artment	of Trans	portat	tion	- -	Project			lge #6133, Lake Bridge #3359	Boring No.:	BB-LI	BS-103	
		_	Soil/Rock Expl US CUSTOM/		_		ŀ	Locatio			Road over Black Stream aine	WIN:	2709	98.00	
Drill	O #1		MaineDOT			Flovet	L	/# \	128	. 0		Auger ID/OD:	5" Solid Stem		
-	rator:		Daggett/Andrl	do.		Elevat		(π.)		vd .vd .vd		Sampler:	Standard Split	Encon	
- 1	ged By:		N. Pukay	ic .		Rig Ty				IE 450		Hammer Wt./Fall:	140#/30"	эроон	
<u> </u>	Start/Fi	nieh:	6/7/2023; 08:0	00-12:00		Drillin	•	othod:			sh Boring	Core Barrel:	NQ-2"		
-	ng Loca		15+50.4, 8.2 f			Casing	<u> </u>			/-3"	an Doring	Water Level*:	7.0 ft bgs.		
\vdash			actor: 0.906			Hamm	_		Auton		Hydraulic □	Rope & Cathead □	7.0 10 085.		
Defini D = S MD = U = T MU = V = F	tions: plit Spoon S Unsuccess hin Wall Tu Unsuccess ield Vane S	Sample Iful Split Spo be Sample Iful Thin Wa Shear Test,	oon Sample Atten all Tube Sample A PP = Pocket Pe ane Shear Test Att	mpt HS RC Attempt Wenetrometer W	= Rock Cord SA = Solid S SA = Hollow C = Roller C OH = Weigh OR/C = Weigh O1P = Weigh	re Sample Stem Auge v Stem Aug Cone ht of 140 lb eight of Roo	er ger o. Har	mmer Casing	S _u = S _{u(l} q _p = N-u Han N ₆₀	= Peak/ ab) = L = Uncor ncorrec nmer Et = SPT	Remolded Field Vane Undrained She ib Vane Undrained Shear Strength (if fined Compressive Strength (isf) ed = Raw Field SPT N-value iciency Factor = Rig Specific Annual N-uncorrected Corrected for Hamme mer Efficiency Factor/60%)*N-uncor	$\begin{array}{ll} \text{par Strength (psf)} & T_V = 1 \\ \text{psf)} & WC = 1 \\ \text{LL} = 1 \\ \text{PL} = 1 \\ \text{Calibration Value} & PI = 1 \\ \text{pr Efficiency} & G = 0 \end{array}$	Pocket Torvane She Water Content, perc Liquid Limit Plastic Limit Plasticity Index Grain Size Analysis Consolidation Test	cent	
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf)		N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)	Graphic Log		scription and Remarks		Laboratory Testing Results/ AASHTO and Unified Class.	
25											Rock Quality = Very Poor R1: Core Times (min:sec)				
	R3	60/58	26.30 - 31.30	RQD = 63%	%				1		21.3-22.3 ft (2:27) 22.3-24.3 ft (4:48) Lost wat	ter at 22.8 ft has			
									┨		23.3-23.9 ft (3:15)	er at 22.0 ft 050.			
			-						-		58% Recovery				
											R2: Bedrock: Similar to R1 [Vassalboro Formation]				
											Rock Quality = Fair				
30 -								\	1		R2: Core Times (min:sec) 23.9-24.3 ft (1:11)				
			-					-	97.:		24.3-25.3 ft (2:00) 25.3-26.3 ft (1:48)				
] 97.:		93% Recovery				
- 35 - - 40 -											GRAYWACKE interbedde hard, fresh, joint sets moder quartz or calcite annealed fi weakly oxidized base metal [Vassalboro Formation] Rock Quality = Fair R3: Core Times (min:sec) 26.3-27.3 ft (2:05) 27.3-28.3 ft (2:24) 28.3-29.3 ft (3:16) 29.3-30.3 ft (3:03) 30.3-31.3 ft (3:19) 97% Recovery	5.3 ft (2:00) 5.3 ft (1:48) ecovery drock: Grey to dark grey, very fine to medium-grained, WACKE interbedded with zones of PHYLLITE, moderately resh, joint sets moderately dipping to steep, closely spaced, with or calcite annealed fractures, sulfide-rich zone highlighted by oxidized base metal minerals. lboro Formation] quality = Fair ore Times (min:sec) 7.3 ft (2:05) 8.3 ft (2:24) 9.3 ft (3:16) 0.3 ft (3:03) 1.3 ft (3:19)			
50															
	arks:				,						•				
1) \	Water leve	el measure	ed immediately	after drilling.											
Ctro+	ication line	e reprocest	annrovimete herr	ındaries between so	oil typos: tr-	neitions	13\1 h -	arodual				Page 2 of 2			
					•		•	_	ons may	occur d	ue to conditions other	1 aye 2 UI 2			

N	Main	- ;	artment Soil/Rock Exp US CUSTOM.		atioi	n			carri	ridge #6133, Lake Bridge # ake Road over Black Strean Maine			98.00
Drille	er:		MaineDOT		Ele	vation	(ft.)		129		Auger ID/OD:	5" Solid Stem	
	ator:		Daggett/Andr	le		tum:	- (,		NA	38		Standard Split	Spoon
	jed By:		N. Pukay		Rig	Туре	:		CM	5C	<u> </u>	140#/30"	1
Date	Start/Fi	nish:	7/26/2023; 08	:30-14:00	Dri	lling N	letho	od:	Ca	Vash Boring	Core Barrel:	NQ-2"	
Bori	ng Loca	tion:	14+21.2, 7.3 1	t Lt.	Cas	sing IE	D/OD	:	NV)"/3.5")	Water Level*:	None Observe	đ
Ham	mer Effi	ciency F	actor: 0.906		Hai	mmer	Туре	:	Auton		Rope & Cathead □		
MD = U = Th MU = V = Fi	olit Spoon S Unsuccess nin Wall Tu Unsuccess eld Vane S	ful Split Sp be Sample ful Thin Wa hear Test,	oon Sample Atter all Tube Sample A PP = Pocket Pe une Shear Test At	RC = Roller WOH = We Pretrometer	d Stem A ow Stem r Cone eight of 14 Veight of	Auger Auger 40lb. Ha f Rods ol	r Casir		S _{u(} q _p : N-u Har N ₆ (ak/Remolded Field Vane Undrai Lab Vane Undrained Shear St toonfined Compressive Strength ected = Raw Field SPT N-value Efficiency Factor = Rig Specific PT N-uncorrected Corrected for ammer Efficiency Factor/60%)*	rength (psf)	cket Torvane She ater Content, per uid Limit stic Limit sticity Index n Size Analysis solidation Test	
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/e in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing	Blows	Elevation (#)	Graphic Log	ual Description and Remarks		Laboratory Testing Results/ AASHTO and Unified Clas
0								SA	129.	3½" HMA.		-0.3	
5 -	1D	24/14	5.00 - 7.00	6/9/12/16	21	32	SP			Brown, moist, hard,	SILT, little sand, little gravel, (Fill)		
							N						
10 -	2D	24/12	10.00 - 12.00	2/2/3/3	5	8			120.		stiff, SILT, trace clay, trace sand, (S and Deposits).	9.0-	
	3D/A	24/16	12.00 - 14.00	3/4/5/5	9	14				Alluvium with Wetla 3D/A (Bottom 8") D	ret, stiff, SILT, some fine sand, trace and Deposits). ark grey-brown, wet, stiff, SILT, lit turn with Wetland Deposits).	• • • • • • • • • • • • • • • • • • • •	Non-Plasti #380877 WC=70.5%
15 -	4D	24/4	14.00 - 16.00	6/8/5/4	13	20				Wetland Deposits).	Sandy SILT, trace clay, (Stream A	lluvium with	Ignition Los 13.7%
	5D	24/6	16.00 - 18.00	3/8/13/18	21	32			113.	Grey, wet, dense, Sil	tty SAND, some gravel, trace clay,	———16.0 (Glacial Till).	
	6D	24/8	18.00 - 20.00	6/8/10/14	18	27				Olive-grey, wet, very (Glacial Till).	y stiff, Sandy SILT, trace clay, trace	e gravel,	
20 -													
25 _													
Kem	arks:												

N	Maine Department of Transpor				ation	Ī	Project			e #6133, Lake Bridge #3359	Boring No.:	BB-LI	3S-201
			•				Locatio		s Lake R ant, Mai	load over Black Stream ne			
		<u>.</u>	US CUSTOMA	ARY UNITS							WIN:	2709	98.00
Drille	er:		MaineDOT		Elevat	tion	(ft.)	129	.9		Auger ID/OD:	5" Solid Stem	
Ope	rator:		Daggett/Andrl	ie	Datum	ո։		NA	VD88		Sampler:	Standard Split	Spoon
Logg	ged By:		N. Pukay		Rig Ty	/pe:		CM	E 45C		Hammer Wt./Fall:	140#/30"	
_	Start/Fi		7/26/2023; 08:	:30-14:00	Drilling			Cas	ed Wash	Boring	Core Barrel:	NQ-2"	
	ng Loca		14+21.2, 7.3 f	t Lt.	Casing	_			(3.0"/3.5		Water Level*:	None Observed	1
Ham Definit		ciency F	actor: 0.906	R = Rock Co	Hamm ore Sample		ype:	Autom		Hydraulic □ molded Field Vane Undrained She	Rope & Cathead ear Strength (psf) Tue	Pocket Torvane She	ar Strength (psf)
D = S _I MD = U = TI MU = V = Fi	olit Spoon S Unsuccess nin Wall Tu Unsuccess eld Vane S	ful Split Spo be Sample ful Thin Wa hear Test,	oon Sample Atten	SSA = Solid HSA = Hollic RC = Roller WOH = Wei Inetrometer WOR/C = W WO1P = Wei WO1P = Wei	I Stem Auger ow Stem Aug Cone ight of 140 lb Veight of Roo	er ger b. Har ds or (Casing	S _{u(la} q _p = N-ur Ham N ₆₀	ab) = Lab Unconfin corrected mer Effici = SPT N-	Vane Undrained Shear Strength (ed Compressive Strength (ksf) = Raw Field SPT N-value ency Factor = Rig Specific Annua uncorrected Corrected for Hamme er Efficiency Factor/60%)'N-unco	psf)	= Water Content, percent liquid Limit = Plastic Limit = Plasticity Index Grain Size Analysis Consolidation Test	ent (par)
		<u> </u>		Sample Information	D D			1	1				Laboratory
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)	Graphic Log	Visual De	scription and Remarks	:	Testing Results/ AASHTO and Unified Class
25										Artesian water pressure at 2	25.0 ft bgs.		
- 30 - - 35 - - 40 -									A STATE OF THE STA	Artesian water pressure at 3			
		50/50						82.7				————47.2 ⁻	
	R1	60/60	47.20 - 52.20	RQD = 83%			NQ-2	1		Top of Bedrock at Elev. 82	.7 ft.		
										R1: Bedrock: Grey to dark GRAYWACKE interbedde thinly bedded, joint sets dip	d with PHYLLITE, mod	erately hard, fresh,	
50 Rem	arks:							<u> </u>	01/201		1 -0 5 to moderate	, •	
Stratif	ication line	s represent	approximate bour	ndaries between soil types; to	ransitions m	ay be	gradual.				Page 2 of 3		
		-	been made at time	es and under conditions state	ed. Ground	lwater	fluctuation	ons may o	ccur due	to conditions other	Boring No	o.: BB-LBS-2	201

N	Main	e Dep	artment	of Transport	ation	Ti	Project:			ge #6133, Lake Bridge #3359	Boring No.:	BB-L	BS-201
			Soil/Rock Exp US CUSTOM			ŀ	Locatio			Road over Black Stream ine	WIN:	270	98.00
			03 C03 TOW	ART UNITS							WIN.		98.00
Drille	er:		MaineDOT		Elevati	ion	(ft.)	129).9		Auger ID/OD:	5" Solid Stem	
Ope	rator:		Daggett/Andr	rle	Datum	:		NA	VD88		Sampler:	Standard Split	Spoon
Logg	ged By:		N. Pukay		Rig Ty	pe:		CM	1E 45C		Hammer Wt./Fall:	140#/30"	
Date	Start/F	inish:	7/26/2023; 08	3:30-14:00	Drilling	g Me	ethod:	Cas	sed Was	h Boring	Core Barrel:	NQ-2"	
Bori	ng Loca	ition:	14+21.2, 7.3	ft Lt.	Casing	j ID	OD:	NV	V(3.0"/3	5")	Water Level*:	None Observe	d
		iciency F	actor: 0.906		Hamm	er T	уре:		natic 🛭	Hydraulic □	Rope & Cathead □		
MD = U = TI MU = V = Fi	plit Spoon Unsuccess nin Wall Tu Unsuccess eld Vane S	sful Split Sp ube Sample sful Thin Wa Shear Test,	oon Sample Atter all Tube Sample A PP = Pocket Pe ane Shear Test At	SSA = Soli MSA = Hol RC = Rolle Attempt WOH = We enetrometer WOR/C = V	Core Sample d Stem Auger low Stem Aug r Cone eight of 140 lb Weight of One	jer . Har Is or	Casing	S _{u(} q _p : N-u Har N ₆₀	lab) = La = Unconf ncorrecte nmer Eff) = SPT I	emolded Field Vane Undrained Sh vane Undrained Shear Strength i ned Compressive Strength (ksf) d = Raw Field SPT N-value iency Factor = Rig Specific Annua -uncorrected Corrected for Hammner Efficiency Factor/60%)*N-unco	(psf) WC :	Pocket Torvane She Water Content, per Liquid Limit Plastic Limit Plasticity Index Grain Size Analysis Consolidation Test	
				Sample Information				_	4				Laboratory
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	09 _N	Casing Blows	Elevation (ff.)	Graphic Log		escription and Remarks		Testing Results/ AASHTO and Unified Class
50										spaced, with significant qu [Vassalboro Formation] Rock Quality = Good	artz or calcite annealed fra	actures.	
	R2	60/60	52.20 - 57.20	RQD = 75%						R1: Core Times (min:sec) 47.2-48.2 ft (4:02) 48.2-49.2 ft (3:12)			
				-				1	M. J.	49.2-50.2 ft (3:31) 50.2-51.2 ft (3:02)			
										51.2-52.2 ft (4:47)			
										100% Recovery			
- 55 -								1		R2: Bedrock: Grey to dark			
							1			GRAYWACKE interbedde joint sets steeply dipping, of			1
							$ \bigvee$ _			annealed fractures.			
							v	72.	7	[Vassalboro Formation] Rock Quality = Fair			
								ł		R2: Core Times (min:sec)			
										52.2-53.2 ft (3:16) 53.2-54.2 ft (2:51)			
										54.2-55.2 ft (1:58)			
- 60 -								1		55.2-56.2 ft (1:52) 56.2-57.2 ft (1:55)			
										100% Recovery		57.2	
										Bottom of Exploratio	n at 57.2 feet below grou	nd surface.	1
								-					
- 65 -								1					
								i					
								l					
70 -								1					
								ł					
								ł					
75													
	arks:				1			•					
Stratif	ication line	s represent	approximate bou	undaries between soil types;	transitions ma	ay be	gradual.				Page 3 of 3		
* Wate	er level rea	idings have	been made at tin	nes and under conditions sta	ated. Groundy	water	fluctuatio	ns may	occur du	to conditions other			

Boring No.: BB-LBS-202 Maine Department of Transportation Project: Perkins Bridge #6133, Lake Bridge #3359 carries Lake Road over Black Stream Soil/Rock Exploration Log Location: Levant, Maine US CUSTOMARY UNITS WIN: 27098.00 Driller: MaineDOT Elevation (ft.) 128.8 Auger ID/OD: 5" Solid Stem NAVD88 Operator: Daggett/Andrle Datum: Sampler: Standard Split Spoon Logged By: N. Pukay Rig Type: CME 45C Hammer Wt./Fall: 140#/30" Date Start/Finish: 7/25/2023; 13:15-15:15 **Drilling Method:** Cased Wash Boring Core Barrel: N/A **Boring Location:** 15+31.7, 6.0 ft Rt Casing ID/OD NW(3.0"/3.5") Water Level* 10.0 ft bgs Hammer Efficiency Factor: 0.906 Hammer Type: Rope & Cathead [Automatic ⊠ Hydraulic 🗆 S_{II} = Peak/Remolded Field Vane Undrained Shear Strength (psf) Definitions R = Rock Core Sample Ty = Pocket Torvane Shear Strength (psf) S_{u(lab)} = Lab Vane Undrained Shear Strength (psf) WC = Water Content, percent D = Split Spoon Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger qp = Unconfined Compressive Surging N-uncorrected = Raw Field SPT N-value = Unconfined Compressive Strength (ksf) LL = Liquid Limit MD = Unsuccessful Split Spoon Sample Attempt RC = Roller Cone U = Thin Wall Tube Sample PL = Plastic Limit MU = Unsuccessful Thin Wall Tube Sample Attempt WOH = Weight of 140lb. Hammer Hammer Efficiency Factor = Rig Specific Annual Calibration Value PI = Plasticity Index WOR/C = Weight of Rods or Casing N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency G = Grain Size Analysis V = Field Vane Shear Test. PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt WO1P = Weight of One Person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test Sample Information Laboratory Depth N-uncorrected Testing <u>=</u> Log 3lows (/6 in.) **RQD** (%) Results/ S Visual Description and Remarks Pen./Rec. Elevation (ft.) Strength (psf) Graphic AASHTO Sample | (ft.) Sample Casing Blows Depth Shear and ار ا Jnified Class 3" HMA. SSA 5 Brown, moist, medium dense, Gravelly SAND, little silt, (Fill). G#380878 1D 24/14 5.00 - 7.00 8/8/9/8 17 26 32 A-1-a, SM WC=4.3% 37 35 37 29 10 Brown, wet, medium dense, Gravelly SAND, little silt, (Fill). NW 2D10 00 - 12 00 5/5/10/11 23 24/8 15 3D (Top 8") Similar to 2D, except dense. 3D/A 24/14 12.00 - 14.00 10/7/15/13 22 33 3D/A (Bottom 6") Brown, wet, dense, Silty GRAVEL, little sand, little peat, trace wood, (Fill). 114.8 #380879 4D (Top 5") Grey-brown, wet, medium stiff, Silty CLAY, trace gravel, 4D/A 24/8 14.00 - 16.00 3/3/2/2 5 8 A-7-5, OH trace peat, (Stream Alluvium with Wetland Deposits) 15 WC=50.0% 4D/A (Bottom 3") Dark brown, wet, medium stiff, SILT, some peat, LL=62 trace sand, (Stream Alluvium with Wetland Deposits). PL=45 5D Black, wet, medium stiff, PEAT, some silt, trace sand, (Stream 5D 24/24 16.00 - 18.00 1/1/2/3 3 5 PI=17 Alluvium with Wetland Deposits). #380880 WC=431% Ignition Loss 6D Similar to 5D. 67.4% 18.00 - 20.00 1/2/1/2 3 5 6D 24/21 #317731 pH=5.22 20 7D (Top 8") Black, wet, stiff, Silty PEAT, trace sand, (Stream 7D/A 24/17 20.00 - 22.00 4/8/2/4 10 15 Alluvium with Wetland Deposits). 107.8 G#380881 7D/A (Bottom 9") Grey, wet, stiff, Clayey SILT, trace sand, (Glacial A-4, CL Till). WC=31.3% Grey, wet, hard, Sandy SILT, little gravel, trace clay, (Glacial Till). 8D 24/8 22.00 - 24.00 7/17/17/18 34 51 LL=28 PI = 20PI=8Grey, wet, SILT, some sand, trace clay, trace gravel, (Glacial Till). 24.00 - 24.80 9D 9.6/9 18/50(3.6") Remarks: 1) Water level measured immediately after drilling.

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

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

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I	Main	e Dep	artment	of Transport	ation		Project:			ge #6133, Lake Bridge #3359	Boring No.:	BB-L1	BS-202
			Soil/Rock Exp				Locatio			Road over Black Stream ine	l		
			JS CUSTOM	ARY UNITS							WIN:	2709	98.00
Drill	er:		MaineDOT		Eleva	ation	(ft.)	128.	8		Auger ID/OD:	5" Solid Stem	
Ope	rator:		Daggett/Andr	·le	Datu	ım:		NAV	/D88		Sampler:	Standard Split	Spoon
Log	ged By:		N. Pukay		Rig 1	Туре		CMI	E 45C		Hammer Wt./Fall:	140#/30"	
Date	Start/F	inish:	7/25/2023; 13	3:15-15:15	Drilli	ing N	lethod:	Case	d Was	h Boring	Core Barrel:	N/A	
Bori	ing Loca	ition:	15+31.7, 6.0	ft Rt.			O/OD:	NW	(3.0"/3	5")	Water Level*:	10.0 ft bgs.	
Ham Defini		iciency F	actor: 0.906	R = Rock (Туре:	Automa		Hydraulic □ emolded Field Vane Undrained She	Rope & Cathead acr Strongth (psf)	= Pocket Torvane She	or Strongth (not)
D = S MD = U = T MU = V = F	plit Spoon Unsuccess Thin Wall Tu Unsuccess ield Vane S	sful Split Sp ube Sample sful Thin Wa Shear Test,	oon Sample Atter Il Tube Sample A PP = Pocket Pe ne Shear Test Al	SSA = Soli	d Stem Aug ow Stem A r Cone eight of 140 Veight of R	ger Auger) lb. Ha Rods or	Casing	S _{u(la} q _p = N-un Hami N ₆₀ :	b) = Lab Unconfin correcte mer Effic = SPT N	Nane Undrained Shear Strength (ned Compressive Strength (ksf) d = Raw Field SPT N-value ciency Factor = Rig Specific Annua -uncorrected Corrected for Hammer Efficiency Factor/60%)*N-unco	(psf) W0 LL PL I Calibration Value PI er Efficiency G :	C = Water Content, per = Liquid Limit = Plastic Limit = Plasticity Index = Grain Size Analysis = Consolidation Test	
				Sample Information	ъ			<u> </u>					Laboratory
Depth (ff.)	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		escription and Remark		Testing Results/ AASHTO and Unified Class.
	narks:	el measure	d immediately	after drilling.						REFUSAL, presumed Top			
l 1),	water lev	ei measure	u mmediately	after drilling.									
Strati	fication line	es represent	approximate bou	indaries between soil types;	transitions	may b	e gradual.				Page 2 of 2		
			been made at tin me measuremer	nes and under conditions stants were made.	ted. Grour	ndwate	er fluctuatio	ns may o	ccur due	e to conditions other	Boring No	o.: BB-LBS-2	202

Boring No.: BB-LBS-203 Maine Department of Transportation Project: Perkins Bridge #6133, Lake Bridge #3359 carries Lake Road over Black Stream Soil/Rock Exploration Log Location: Levant, Maine US CUSTOMARY UNITS WIN: 27098.00 Driller: MaineDOT Elevation (ft.) 129.0 Auger ID/OD: 5" Solid Stem Datum: NAVD88 Operator: Daggett/Andrle Sampler: Standard Split Spoon CME 45C 140#/30" Logged By: N. Pukay Rig Type: Hammer Wt./Fall: Date Start/Finish: 7/25/2023; 09:30-13:00 **Drilling Method:** Cased Wash Boring Core Barrel: N/A **Boring Location:** 15+49.6, 7.2 ft Lt. Casing ID/OD: NW(3.0"/3.5") Water Level*: 9.0 ft bgs Hammer Efficiency Factor: 0.906 Hammer Type: Rope & Cathead [Automatic ⊠ Hydraulic 🗆 S_{II} = Peak/Remolded Field Vane Undrained Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf) Definitions R = Rock Core Sample WC = Water Content, percent D = Split Spoon Sample SSA = Solid Stem Auger Su(lab) = Lab Vane Undrained Shear Strength (psf) MD = Unsuccessful Split Spoon Sample Attempt HSA = Hollow Stem Auger qp = Unconfined Compressive Surging N-uncorrected = Raw Field SPT N-value = Unconfined Compressive Strength (ksf) LL = Liquid Limit RC = Roller Cone U = Thin Wall Tube Sample PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis MU = Unsuccessful Thin Wall Tube Sample Attempt WOH = Weight of 140lb. Hammer Hammer Efficiency Factor = Rig Specific Annual Calibration Value V = Field Vane Shear Test, PP = Pocket Penetrometer WOR/C = Weight of Rods or Casing N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency MV = Unsuccessful Field Vane Shear Test Attempt WO1P = Weight of One Person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test Sample Information Laboratory Testing Depth N-uncorrected <u>=</u> Log 3lows (/6 in.) **RQD** (%) Results/ S Visual Description and Remarks Depth (ft.) Pen./Rec. Elevation (ft.) Graphic AASHTO Sample | (ft.) Strength Sample Casing Blows Shear and **V**60 Jnified Class 3" HMA. SSA 5 Brown, moist, dense, Gravelly SAND, little silt, (Fill). 1D 24/5 5.00 - 7.00 9/11/13/13 24 36 10 Similar to 1D, except wet. NW 2D10.00 - 12.00 9/12/14/14 24/8 26 39 G#380882 Grey-brown, wet, very dense, Gravelly SAND, little silt, (Fill). 3D 24/12 12.00 - 14.00 22/18/26/22 44 66 A-1-a, SM WC=8.4% 115.0 Brown, wet, WOOD, trace sand, (Stream Alluvium with Wetland 4D 24/4 14.00 - 16.00 15/19/9/9 28 42 Deposits). 15 Black, wet, stiff, Silty PEAT, trace sand, (Stream Alluvium with 5D 24/15 16.00 - 18.00 6/6/6/4 12 18 Wetland Deposits). PEAT and WOOD in wash. MD 18.00 - 20.00 5/6/3/3 9 14 MD 19.00 - 21.00 20 108.7 -20.3OPEN HOLE Olive-grey, wet, very stiff, Sandy SILT, trace clay, trace gravel, 6D 24/11 21.00 - 23.00 2/3/8/13 11 17 (Glacial Till). Similar to 6D, except hard. Presumed bedrock in tip of spoon. 7D 19.2/8 23.00 - 24.60 9/12/10/50(1.2") 22 33 Remarks: 1) Water level measured immediately after drilling.

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

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

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I	Main	e Dep	artment	of Transport	ation		Project:			ge #6133, Lake Bridge #3359	Boring No.:	BB-L1	BS-203
		_ 5	Soil/Rock Exp	loration Log			Locatio			Road over Black Stream			
		<u> </u>	JS CUSTOM	ARY UNITS							WIN:	2709	98.00
Drill	er:		MaineDOT		Elev	ation	(ft.)	129.	0		Auger ID/OD:	5" Solid Stem	
Ope	rator:		Daggett/Andr	·le	Datu	ım:		NAV	/D88		Sampler:	Standard Split	Spoon
Log	ged By:		N. Pukay		Rig	Туре	:	CMI	E 45C		Hammer Wt./Fall:	140#/30"	
	Start/F		7/25/2023; 09	0:30-13:00	Drilli	ing N	lethod:	Case	d Was	h Boring	Core Barrel:	N/A	
Bori	ing Loca	ition:	15+49.6, 7.2	ft Lt.			D/OD:	NW	(3.0"/3	.5")	Water Level*:	9.0 ft bgs.	
Ham Defini		iciency F	actor: 0.906	R = Rock 0			Type:	Automa		Hydraulic □ emolded Field Vane Undrained Sh	Rope & Cathead	= Pocket Torvane She	or Strongth (not)
D = S MD = U = T MU = V = F	plit Spoon Unsuccess Thin Wall Tu Unsuccess ield Vane S	sful Split Sp ube Sample sful Thin Wa Shear Test,	oon Sample Atter Il Tube Sample A PP = Pocket Pe ne Shear Test Al	SSA = Soli	d Stem Aug low Stem A r Cone eight of 140 Weight of R	ger Auger) lb. Ha Rods or	r Casing	S _{u(la} q _p = N-un Hami N ₆₀ :	b) = Lab Unconfin correcte mer Effic = SPT N	Vane Undrained Shear Strength I ned Compressive Strength (ksf) d = Raw Field SPT N-value ciency Factor = Rig Specific Annua l-uncorrected Corrected for Hammer er Efficiency Factor/60%)*N-unco	(psf) W0 LL PL I Calibration Value P1: er Efficiency G =	C = Water Content, per = Liquid Limit = Plastic Limit = Plasticity Index = Grain Size Analysis = Consolidation Test	
				Sample Information	- O		Τ						Laboratory
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)	Graphic Log		escription and Remark		Testing Results/ AASHTO and Unified Class.
- 30 - 35 - 40 - 45 - 45 - 45 - 45 - 45 - 45 - 4	narks:									REFUSAL, presumed Top	n at 24.6 feet below gro	und surface.	
		el measure	d immediately	after drilling									
1)	water lev	ei measure	a miniediately	anci unillig.									
Strati	fication line	es represent	approximate bou	indaries between soil types;	transitions	may b	e gradual.				Page 2 of 2		
			been made at tin me measuremer	nes and under conditions stants were made.	ited. Groun	ndwate	er fluctuatio	ns may o	ccur due	e to conditions other	Boring No	o.: BB-LBS-2	203

Appendix B

Rock Core Photographs



MaineDOT Perkins Bridge #6133, Lake Bridge #3359 Carries Lake Road Over Black Stream Levant, ME

Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-LBS-101A	R1	67.0-72.0	60	58	46	77	GRAYWACKE/PHYLLITE	1
BB-LBS-101A	R2	72.0-77.0	60	58	46	77	GRAYWACKE/PHYLLITE	2



Notes: 1. "Box row" indicates the section of the box where the core run is contained: 1 = top, 4 = bottom.

2. Top of each core run is on the left and increases with depth to the right.



MaineDOT Perkins Bridge #6133, Lake Bridge #3359 Carries Lake Road Over Black Stream Levant, ME

Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-LBS-102	R1	51.0-56.0	60	60	45	75	GRAYWACKE/PHYLLITE	1
BB-LBS-102	R2	56.0-61.0	60	60	47	78	GRAYWACKE/PHYLLITE	2
BB-LBS-103	R1	21.3-23.9	31.2	18	0	0	GRAYWACKE/PHYLLITE	3
BB-LBS-103	R2	23.9-26.3	28.8	27	20	69	GRAYWACKE/PHYLLITE	3
BB-LBS-103	R3	26.3-31.3	60	58	38	63	GRAYWACKE/PHYLLITE	4



Notes: 1. "Box row" indicates the section of the box where the core run is contained: 1 = top, 4 = bottom.

2. Top of each core run is on the left and increases with depth to the right.



MaineDOT Perkins Bridge #6133, Lake Bridge #3359 Carries Lake Road Over Black Stream Levant, ME

Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-LBS-201	R1	47.2-52.2	60	60	50	83	GRAYWACKE/PHYLLITE	1
BB-LBS-201	R2	52.2-57.2	60	60	45	75	GRAYWACKE/PHYLLITE	2



Notes: 1. "Box row" indicates the section of the box where the core run is contained: 1 = top, 4 = bottom.

2. Top of each core run is on the left and increases with depth to the right.

Appendix C

Laboratory Test Results

State of Maine - Department of Transportation **Laboratory Testing Summary Sheet**

Town(s):	Levar	nt			Work	·Νι	ımk	er	2709	00.86	
Boring & Sample	Station	Offset	Depth	Reference	G.S.D.C.	W.C.	L.L.	P.I.	Cla	ssification	1
Identification Number	(Feet)	(Feet)	(Feet)	Number	Sheet	%			Unified	AASHTO	Frost
BB-LBS-101, 1D	13+33.4	7.9 Rt.	5.0-7.0	380926	1	16.1			CL-ML	A-4	IV
BB-LBS-101, 3D	13+33.4	7.9 Rt.	15.0-17.0	380927	1	10.8	20	6	CL-ML	A-4	IV
BB-LBS-101, 4D	13+33.4	7.9 Rt.	20.0-22.0	380928	1	10.2	20	6	CL-ML	A-4	IV
BB-LBS-101, 6D	13+33.4	7.9 Rt.	30.0-32.0	380929	1	11.5	21	7	CL-ML	A-4	IV
BB-LBS-101A, S1	13+36	8.3 Rt.	9.0-10.0	380930	1	34.0			CL	A-4	IV
BB-LBS-102, 2D	14+17.2	7.4 Rt.	11.0-13.0	380931	2	36.3			SM	A-2-4	- II
BB-LBS-102, 3D	14+17.2	7.4 Rt.	15.0-17.0	380932	2	5.4	-N	P-	SC-SM	A-1-b	- II
BB-LBS-102, 4D	14+17.2	7.4 Rt.	20.0-22.0	380933	2	22.7	20	5	CL-ML	A-4	IV
BB-LBS-102, 8D	14+17.2	7.4 Rt.	45.0-47.0	380934	2	13.7			CL-ML	A-4	IV
BB-LBS-201, 3D	14+21.2	7.3 Lt.	12.0-12.67	380876			-N	P-			
BB-LBS-201, 3D/A	14+21.2	7.3 Lt.	13.3-14.0	380877		70.5	Los	s on	Ignition (T 267) 13	3.7%
BB-LBS-202, 1D	15+31.7	6.0 Rt.	5.0-7.0	380878	3	4.3			SM	A-1-a	П
BB-LBS-202, 4D	15+31.7	6.0 Rt.	14.0-14.42	380879		50.0	62	17	OH	A-7-5	III
BB-LBS-202, 5D	15+31.7	6.0 Rt.	16.0-18.0	380880		431	Los	s on	Ignition (T 267) 6	7.4%
BB-LBS-202, 6D	15+31.7	6.0 Rt.	18.0-20.0	317731		рŀ	1 5.22	2 by (GeoTesti	ng Expre	ss
BB-LBS-202, 7D/A	15+31.7	6.0 Rt.	21.25-22.0	380881	3	31.3	28	8	CL	A-4	IV
BB-LBS-203, 3D	15+49.6	7.2 Lt.	12.0-14.0	380882	3	8.4			SM	A-1-a	П
		ļ									
								•			

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

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

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

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

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

NP = Non Plastic

Maine Department of Transportation Grain Size Distribution Curve SIEVE ANALYSIS HYDROMETER ANALYSIS US Standard Sieve Numbers Grain Diameter, mm #200 0.010 0.001 1/2" 3/8" 1/4" #4 #8 #10 #40 #60 #100 0.05 0.03 100 0 10 90 80 20 Percent Retained by Weight 30 Percent Finer by Weight 50 60 30 70 80 20 10 90 100 2.36 2.00 76.2 12.7 9.53 6.35 0.075 0.005 100 10 0.1 0.01 0.001 Grain Diameter, mm

UNIFIED CLASSIFICATION

SAND

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
0	BB-LBS-101/1D	13+33.4	7.9 RT	5.0-7.0	SILT, little sand, little gravel.	16.1			
♦	BB-LBS-101/3D	13+33.4	7.9 RT	15.0-17.0	SILT, some gravel, some sand, little clay.	10.8	20	14	6
	BB-LBS-101/4D	13+33.4	7.9 RT	20.0-22.0	SAND, some silt, some gravel, little clay.	10.2	20	14	6
•	BB-LBS-101/6D	13+33.4	7.9 RT	30.0-32.0	SILT, some sand, little gravel, little clay.	11.5	21	14	7
\blacktriangle	BB-LBS-101A/S1	13+36	8.3 RT	9.0-10.0	SILT, some clay, little sand, trace gravel.	34			
×									

GRAVEL

WI	N
027098.00	
Tov	wn
Levant	
Reported	by/Date
WHITE, TERRY A	7/18/2023

CLAY

SILT

Maine Department of Transportation Grain Size Distribution Curve SIEVE ANALYSIS HYDROMETER ANALYSIS Grain Diameter, mm US Standard Sieve Numbers 3" 2" 1 1/2" 1" 3/4" #8 #10 #100 #200 0.05 0.03 0.001 100 90 10 20 30 40 50 60 Percent Retained by Weight Percent Finer by Weight 50 40 20 80 10 90 0 50.8 38.1 12.7 9.53 6.35 4.75 1.18 0.85 0.15 0.075 0.03 100 10 0.1 0.01 0.001 Grain Diameter, mm GRAVEL SAND CLAY SILT

UNIFIED CLASS	IFICATION
---------------	-----------

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
0	BB-LBS-102/2D	14+17.2	7.4 RT	11.0-13.0	11.0-13.0 SAND, little silt, trace gravel.				
•	BB-LBS-102/3D	14+17.2	7.4 RT	15.0-17.0	15.0-17.0 Gravelly SAND, little silt, trace clay.				NP
	BB-LBS-102/4D	14+17.2	7.4 RT	20.0-22.0	20.0-22.0 SILT, some sand, little clay, trace gravel.		20	15	5
•	BB-LBS-102/8D	14+17.2	7.4 RT	45.0-47.0	SILT, some sand, little gravel.	13.7			
×									

WI	N			
027098.00				
Tov	wn			
Levant				
Reported by/Date				
WHITE, TERRY A	7/18/2023			

SHEET 2

Maine Department of Transportation Grain Size Distribution Curve SIEVE ANALYSIS HYDROMETER ANALYSIS US Standard Sieve Numbers Grain Diameter, mm #200 0.010 0.001 2" 1 1/2" 1" 3/4= 1/2" 3/8" 1/4" #4 #8 #10 #16 #20 #40 #60 #100 0.05 0.03 100 0 10 90 80 20 Percent Retained by Weight 30 Percent Finer by Weight 50 60 30 70 80 20 10 90 100 76.2 12.7 9.53 6.35 4.75 2.36 2.00 0.075 0.005 100 10 0.1 0.01 0.001 Grain Diameter, mm GRAVEL SAND CLAY SILT

UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
0	BB-LBS-202/1D	15+31.7	6.0 RT	5.0-7.0	Gravelly SAND, little silt.	4.3			
♦	BB-LBS-202/7D(A)	15+31.7	6.0 RT	21.25-22.0	Clayey SILT, trace sand.	31.3	28	20	8
	BB-LBS-203/3D	15+49.6	7.2 LT	12.0-14.0	Gravelly SAND, little silt.	8.4			
•									
\blacktriangle									
X									

V	VIN
027098.00	
To	own
Levant	
Reporte	d by/Date
WHITE, TERRY A	9/14/2023



SAMPLE INFORMATION

Reference No. Boring No./Sample No. Sample Description Sampled Received

380927 BB-LBS-101/3D GEOTECHNICAL (DISTURBED)

6/7/2023 6/26/2023

Sample Type: GEOTECHNICAL Location: Station: 13+33.4 Offset, ft: 7.9 RT Dbfg, ft: 15.0-17.0

WIN/Town 027098.00 - LEVANT Sampler: NATHAN PUKAY

TEST RESULTS

Sieve Analysis (T 88)						
Wash Method	d					
SIEVE SIZE	%					
U.S. [SI]	Passing					
3 in. [75.0 mm]	100.0					
1 in. [25.0 mm]	96.5					
¾ in. [19.0 mm]	88.9					
½ in. [12.5 mm]	80.9					
3/8 in. [9.5 mm]	79.2					
1/4 in. [6.3 mm]	74.4					
No. 4 [4.75 mm]	71.9					
No. 10 [2.00 mm]	62.5					
No. 20 [0.850 mm]						
No. 40 [0.425 mm]	53.3					
No. 60 [0.250 mm]						
No. 100 [0.150 mm]						
No. 200 [0.075 mm]	44.9					
[0.0310 mm]	39.3					
[0.0201 mm]	35.4					
[0.0119 mm]	31.4					
[0.0086 mm]	27.6					
[0.0062 mm]	23.6					
[0.0032 mm]	15.8					
[0.0013 mm]	9.8					

Miscellaneous Tests						
Liquid Limit @ 25 blows (T 89), %	20					
Plastic Limit (T 90), %	14					
Plasticity Index (T 90), %	6					
Specific Gravity, Corrected to 20°C (T 100)	2.60					
Loss on Ignition, % (T 267)						
Water Content (T 265), %	10.8					

Consolidation (T 216)							
Trimmings							
	Initial	Final		Void Ratio	% Strain		
Water Content, %			Pmin				
Dry Density, lbs/ft³			Рр				
Void Ratio			Pmax				
Saturation, %			Cc/C'c				

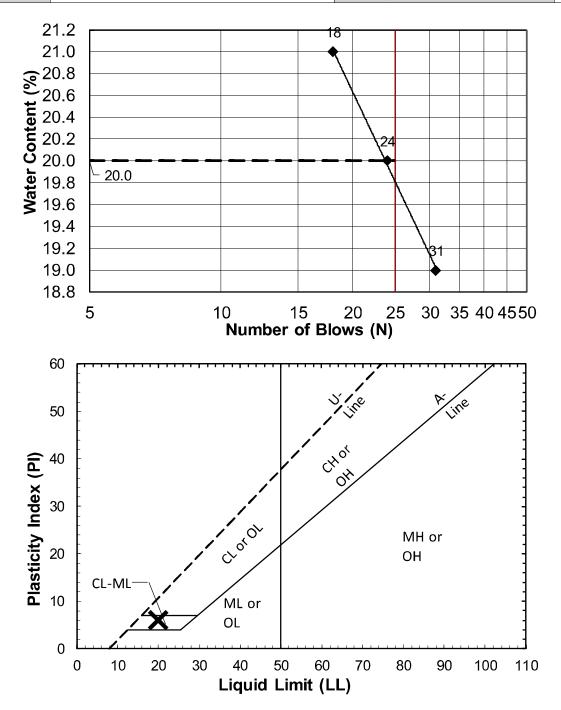
Vane Shear Test on Shelby Tubes (Maine DOT)								
Depth taken	3 ln.		6 In.		Water			
in tube, ft	U. Shear	Remold	U. Shear	Remold	Content,	Description of Material Sampled at the Various Tube Depths		
	tons/ft² tons/ft²		tons/ft ² tons/ft ²		%	various Tube Depths		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**Date Reported: 7/14/2023

TOWN	Levant	Reference No.	380927
WIN	027098.00	Water Content, %	10.8
Sampled	6/7/2023	Liquid Limit @ 25 blows (T 89), %	20
Boring No./Sample No.	BB-LBS-101/3D	Plastic Limit (T 90), %	14
Station	13+33.4	Plasticity Index (T 90), %	6
Depth	15.0-17.0	Tested By	BBURR





SAMPLE INFORMATION

Reference No. Boring No./Sample No. Sample Description Sampled Received

380928 BB-LBS-101/4D GEOTECHNICAL (DISTURBED)

6/7/2023 6/26/2023

Sample Type: GEOTECHNICAL Location: Station: 13+33.4 Offset, ft: 7.9 RT Dbfg, ft: 20.0-22.0

WIN/Town 027098.00 - LEVANT Sampler: NATHAN PUKAY

TEST RESULTS

Sieve Analysis (T 88)				
Wash Method	d			
SIEVE SIZE U.S. [SI]	% Passing			
	1 assing			
3 in. [75.0 mm]				
1 in. [25.0 mm]	100.0			
¾ in. [19.0 mm]	96.8			
½ in. [12.5 mm]	89.7			
% in. [9.5 mm]	84.7			
1/4 in. [6.3 mm]	80.6			
No. 4 [4.75 mm]	77.5			
No. 10 [2.00 mm]	69.6			
No. 20 [0.850 mm]				
No. 40 [0.425 mm]	55.6			
No. 60 [0.250 mm]				
No. 100 [0.150 mm]				
No. 200 [0.075 mm]	43.0			
[0.0310 mm]	42.0			
[0.0198 mm]	40.0			
[0.0117 mm]	35.7			
[0.0086 mm]	29.4			
[0.0062 mm]	25.2			
[0.0032 mm]	16.8			
[0.0014 mm]	8.4			

Miscellaneous Tests				
Liquid Limit @ 25 blows (T 89), %	20			
Plastic Limit (T 90), %	14			
Plasticity Index (T 90), %	6			
Specific Gravity, Corrected to 20°C (T 100)	2.58			
Loss on Ignition, % (T 267)				
Water Content (T 265), %	10.2			

Consolidation (T 216)							
Trimmings							
	Initial	Final		Void Ratio	% Strain		
Water Content, %			Pmin				
Dry Density, lbs/ft³			Рр				
Void Ratio			Pmax				
Saturation, %			Cc/C'c				

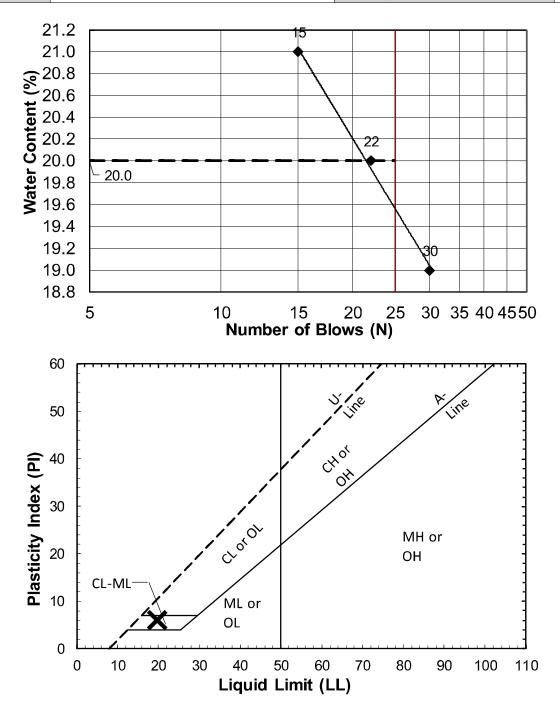
Vane Shear Test on Shelby Tubes (Maine DOT)							
Depth taken	3	n.	6 1	n.	Water		
in tube, ft	U. Shear	Remold	U. Shear	Remold	Content,	Description of Material Sampled at the Various Tube Depths	
	tons/ft²	tons/ft²	tons/ft²	tons/ft²	%	various rube Deptils	

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**Date Reported: 7/14/2023

TOWN	Levant	Reference No.	380928
WIN	027098.00	Water Content, %	10.2
Sampled	6/7/2023	Liquid Limit @ 25 blows (T 89), %	20
Boring No./Sample No.	BB-LBS-101/4D	Plastic Limit (T 90), %	14
Station	13+33.4	Plasticity Index (T 90), %	6
Depth	20.0-22.0	Tested By	BBURR





SAMPLE INFORMATION

Reference No. Boring No./Sample No. Sample Description Sampled Received

380929 BB-LBS-101/6D GEOTECHNICAL (DISTURBED) 6/7/2023 6/26/2023

Sample Type: GEOTECHNICAL Location: Station: 13+33.4 Offset, ft: 7.9 RT Dbfg, ft: 30.0-32.0

WIN/Town 027098.00 - LEVANT Sampler: NATHAN PUKAY

TEST RESULTS

Sieve Analysis (T 88)				
Wash Method	d			
SIEVE SIZE U.S. [SI]	% Passing			
3 in. [75.0 mm]				
1 in. [25.0 mm]				
3/4 in. [19.0 mm]	100.0			
½ in. [12.5 mm]	94.7			
% in. [9.5 mm]	91.5			
1/4 in. [6.3 mm]	85.7			
No. 4 [4.75 mm]	83.1			
No. 10 [2.00 mm]	73.4			
No. 20 [0.850 mm]				
No. 40 [0.425 mm]	66.7			
No. 60 [0.250 mm]				
No. 100 [0.150 mm]				
No. 200 [0.075 mm]	60.8			
[0.0300 mm]	51.2			
[0.0196 mm]	46.5			
[0.0117 mm]	39.5			
[0.0085 mm]	34.9			
[0.0062 mm]	27.9			
[0.0032 mm]	18.6			
[0.0013 mm]	11.6			

Miscellaneous Tests				
Liquid Limit @ 25 blows (T 89), %	21			
Plastic Limit (T 90), %	14			
Plasticity Index (T 90), %	7			
Specific Gravity, Corrected to 20°C (T 100)	2.61			
Loss on Ignition, % (T 267)				
Water Content (T 265), %	11.5			

Consolidation (T 216)						
Trimmings						
	Initial	Final		Void Ratio		
Water Content, %			Pmin			
Dry Density, lbs/ft³			Рр			
Void Ratio			Pmax			
Saturation, %			Cc/C'c			

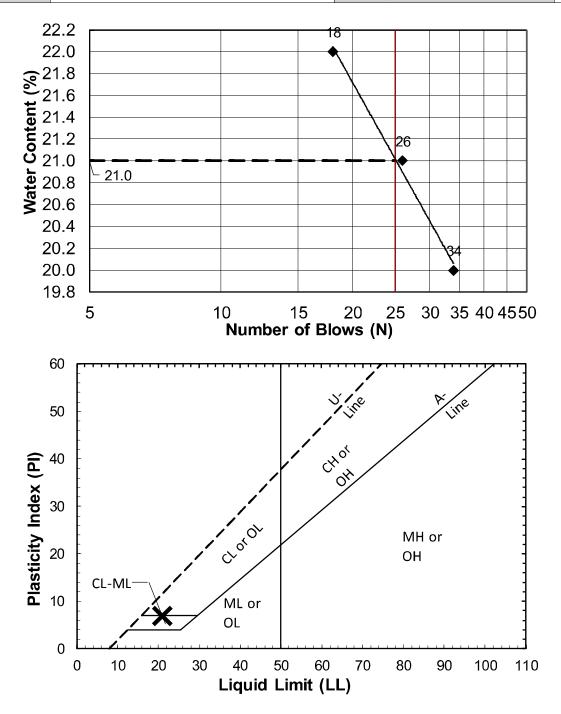
Vane Shear Test on Shelby Tubes (Maine DOT)							
Depth taken	3 I	n.	6 1	n.	Water	D 1.71 CM 1.10 1.14	
in tube, ft	U. Shear	Remold	U. Shear	Remold	Content,	Description of Material Sampled at the Various Tube Depths	
	tons/ft²	tons/ft²	tons/ft²	tons/ft²	%	various Tube Deptils	

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**Date Reported: 7/15/2023

TOWN	Levant	Reference No.	380929
WIN	027098.00	Water Content, %	11.5
Sampled	6/7/2023	Liquid Limit @ 25 blows (T 89), %	21
Boring No./Sample No.	BB-LBS-101/6D	Plastic Limit (T 90), %	14
Station	13+33.4	Plasticity Index (T 90), %	7
Depth	30.0-32.0	Tested By	BBURR





SAMPLE INFORMATION

Reference No. Boring No./Sample No. Sample Description Sampled Received

380932 BB-LBS-102/3D GEOTECHNICAL (DISTURBED)

6/5/2023 6/26/2023

Sample Type: GEOTECHNICAL Location: Station: 14+17.2 Offset, ft: 7.4 RT Dbfg, ft: 15.0-17.0

WIN/Town 027098.00 - LEVANT Sampler: NATHAN PUKAY

TEST RESULTS

Sieve Analysis (T 88)				
Wash Method	d			
SIEVE SIZE U.S. [SI]	% Passing			
3 in. [75.0 mm]				
1 in. [25.0 mm]	100.0			
3/4 in. [19.0 mm]	93.4			
½ in. [12.5 mm]	82.2			
3/8 in. [9.5 mm]	77.3			
1/4 in. [6.3 mm]	67.7			
No. 4 [4.75 mm]	62.7			
No. 10 [2.00 mm]	47.8			
No. 20 [0.850 mm]				
No. 40 [0.425 mm]	29.2			
No. 60 [0.250 mm]				
No. 100 [0.150 mm]				
No. 200 [0.075 mm]	18.1			
[0.0348 mm]	14.8			
[0.0224 mm]	11.9			
[0.0132 mm]	8.9			
[0.0094 mm]	8.9			
[0.0067 mm]	5.9			
[0.0033 mm]	4.4			
[0.0014 mm]	1.5			

Miscellaneous Tests				
Liquid Limit @ 25 blows (T 89), %				
Plastic Limit (T 90), %				
Plasticity Index (T 90), %	NP			
Specific Gravity, Corrected to 20°C (T 100)	2.59			
Loss on Ignition, % (T 267)				
Water Content (T 265), %	5.4			

Consolidation (T 216)								
Trimmings, Water Content, %								
	Void Ratio	% Strain						
Water Content, %			Pmin					
Dry Density, lbs/ft³			Рр					
Void Ratio			Pmax					
Saturation, %			Cc/C'c					

Vane Shear Test on Shelby Tubes (Maine DOT)										
Depth taken	3	n.	6 1	n.	Water					
in tube, ft	U. Shear	Remold	U. Shear	Remold	Content,	Description of Material Sampled at the Various Tube Depths				
	tons/ft²	tons/ft²	tons/ft²	tons/ft²	ft ² % various rube Depths					

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**Date Reported: 7/14/2023



SAMPLE INFORMATION

Reference No. Boring No./Sample No. Sample Description Sampled Received

380933 BB-LBS-102/4D GEOTECHNICAL (DISTURBED)

6/5/2023 6/26/2023

Sample Type: GEOTECHNICAL Location: Station: 14+17.2 Offset, ft: 7.4 RT Dbfg, ft: 20.0-22.0

WIN/Town 027098.00 - LEVANT Sampler: NATHAN PUKAY

TEST RESULTS

Sieve Analysis (T 88)					
Wash Method	d				
SIEVE SIZE U.S. [SI]	% Passing				
3 in. [75.0 mm]					
1 in. [25.0 mm]					
3/4 in. [19.0 mm]	100.0				
½ in. [12.5 mm]	99.1				
3/4 in. [9.5 mm]	96.6				
1/4 in. [6.3 mm]	93.6				
No. 4 [4.75 mm]	91.3				
No. 10 [2.00 mm]	83.1				
No. 20 [0.850 mm]					
No. 40 [0.425 mm]	75.0				
No. 60 [0.250 mm]					
No. 100 [0.150 mm]					
No. 200 [0.075 mm]	63.5				
[0.0280 mm]	60.2				
[0.0182 mm]	55.7				
[0.0111 mm]	46.4				
[0.0081 mm]	41.7				
[0.0059 mm]	34.8				
[0.0031 mm]	23.2				
[0.0013 mm]	13.9				

Miscellaneous Tests					
Liquid Limit @ 25 blows (T 89), %	20				
Plastic Limit (T 90), %	15				
Plasticity Index (T 90), %	5				
Specific Gravity, Corrected to 20°C (T 100)	2.63				
Loss on Ignition, % (T 267)					
Water Content (T 265), %	22.7				

Consolidation (T 216)								
Trimmings, Water Content, %								
	Initial	Final		Void Ratio	% Strain			
Water Content, %			Pmin					
Dry Density, lbs/ft³			Рр					
Void Ratio			Pmax					
Saturation, %			Cc/C'c					

Vane Shear Test on Shelby Tubes (Maine DOT)										
Depth taken	3 I	n.	6 1	n.	Water	D 1.71 CM 1.10 1.14				
in tube, ft	U. Shear	Remold	U. Shear	Remold	Content,	Description of Material Sampled at the Various Tube Depths				
	tons/ft²	tons/ft²	tons/ft²	tons/ft²	ft ² % various Tube Depths					

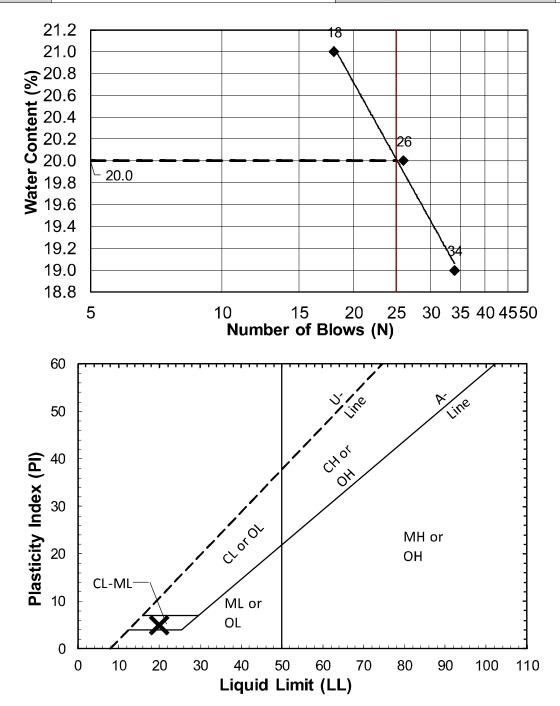
Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: GREGORY LIDSTONE

Date Reported: 7/15/2023

TOWN	Levant	Reference No.	380933
WIN	027098.00	Water Content, %	22.7
Sampled	6/5/2023	Liquid Limit @ 25 blows (T 89), %	20
Boring No./Sample No.	BB-LBS-102/4D	Plastic Limit (T 90), %	15
Station	14+17.2	Plasticity Index (T 90), %	5
Depth	20.0-22.0	Tested By	BBURR





SAMPLE INFORMATION

Reference No. Boring No./Sample No. Sample Description Sampled Received

380876 BB-LBS-201/3D GEOTECHNICAL (DISTURBED) 7/26/2023 8/11/2023

Sample Type: GEOTECHNICAL Location: Station: 14+21.2 Offset, ft: 7.3 LT Dbfg, ft: 12.0-12.67

WIN/Town 027098.00 - LEVANT Sampler: NATHAN PUKAY

TEST RESULTS

Sieve Analysis (T 27)						
Wash Method	d					
SIEVE SIZE U.S. [SI]	% Passing					
3 in. [75.0 mm]						
1 in. [25.0 mm]						
3/4 in. [19.0 mm]						
½ in. [12.5 mm]						
3/4 in. [9.5 mm]						
½ in. [6.3 mm]						
No. 4 [4.75 mm]						
No. 10 [2.00 mm]						
No. 20 [0.850 mm]						
No. 40 [0.425 mm]						
No. 60 [0.250 mm]						
No. 100 [0.150 mm]						
No. 200 [0.075 mm]						

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

Consolidation (T 216)								
Trimmings, Water Content, %								
	Initial	Final		Void Ratio	% Strain			
Water Content, %			Pmin					
Dry Density, lbs/ft³			Рр					
Void Ratio			Pmax					
Saturation, %			Cc/C'c					

Vane Shear Test on Shelby Tubes (Maine DOT)							
Depth taken	U. Shear	ln. Remold	6 In. Water mold U. Shear Remold Content. Description of Material Sampled at				
in tube, ft	tons/ft²	tons/ft²	tons/ft²	tons/ft²	Content, %	Various Tube Depths	
	-						

Comments:

Sampler request to prioritize Limits. Insufficient material to run T 88 and T 100

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**

Paper Copy: Lab File; Project File; Geotech File

Date Reported: 9/13/2023



SAMPLE INFORMATION

Reference No. Boring No./Sample No. Sample Description Sampled Received

380877 BB-LBS-201/3D(A) GEOTECHNICAL (DISTURBED) 7/26/2023 8/11/2023

Sample Type: GEOTECHNICAL Location: Station: 14+21.2 Offset, ft: 7.3 LT Dbfg, ft: 13.3-14.0

WIN/Town 027098.00 - LEVANT Sampler: NATHAN PUKAY

TEST RESULTS

Sieve Analysis (T 27, T 11)						
Wash Method	d					
Procedure A	\					
SIEVE SIZE U.S. [SI]	% Passing					
3 in. [75.0 mm]						
1 in. [25.0 mm]						
3/4 in. [19.0 mm]						
½ in. [12.5 mm]						
3/4 in. [9.5 mm]						
½ in. [6.3 mm]						
No. 4 [4.75 mm]						
No. 10 [2.00 mm]						
No. 20 [0.850 mm]						
No. 40 [0.425 mm]						
No. 60 [0.250 mm]						
No. 100 [0.150 mm]						
No. 200 [0.075 mm]						

Miscellaneous Tests					
Liquid Limit @ 25 blows (T 89), %					
Plastic Limit (T 90), %					
Plasticity Index (T 90), %					
Specific Gravity, Corrected to 20°C (T 100)					
Loss on Ignition, % (T 267)	13.7				
Water Content (T 265), %	70.5				

Consolidation (T 216)								
Trimmings, Water Content, %								
	Initial	Final		Void Ratio	% Strain			
Water Content, %			Pmin					
Dry Density, lbs/ft³			Рр					
Void Ratio			Pmax					
Saturation, %			Cc/C'c					

Vane Shear Test on Shelby Tubes (Maine DOT)															
Depth taken	th taken 3 ln. 6 ln. Water 5										3 In.		6 In.		December of Material Committee of the
in tube, ft	U. Shear	Remold	U. Shear	Remold	Content	Description of Material Sampled at the Various Tube Depths									
	tons/ft²	tons/ft²	various Tube Deptils												
					*										

Comments:			

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**Date Reported: 8/28/2023



SAMPLE INFORMATION

Reference No. Boring No./Sample No. Sample Description Sampled Received

380879 BB-LBS-202/4D GEOTECHNICAL (DISTURBED) 7/25/2023 8/11/2023

Sample Type: GEOTECHNICAL Location: Station: 15+31.7 Offset, ft: 6.0 RT Dbfg, ft: 14.0-14.42

WIN/Town 027098.00 - LEVANT Sampler: NATHAN PUKAY

TEST RESULTS

Sieve Analysis (T 27)						
Wash Method	d					
SIEVE SIZE U.S. [SI]	% Passing					
3 in. [75.0 mm]						
1 in. [25.0 mm]						
3/4 in. [19.0 mm]						
½ in. [12.5 mm]						
3/4 in. [9.5 mm]						
½ in. [6.3 mm]						
No. 4 [4.75 mm]						
No. 10 [2.00 mm]						
No. 20 [0.850 mm]						
No. 40 [0.425 mm]						
No. 60 [0.250 mm]						
No. 100 [0.150 mm]						
No. 200 [0.075 mm]						

Miscellaneous Tests					
Liquid Limit @ 25 blows (T 89), %	62				
Plastic Limit (T 90), %	45				
Plasticity Index (T 90), %	17				
Specific Gravity, Corrected to 20°C (T 100)					
Loss on Ignition, % (T 267)					
Water Content (T 265), %	50.0				

Consolidation (T 216)								
Trimmings, Water Content, %								
	Initial	Final		Void Ratio	% Strain			
Water Content, %			Pmin					
Dry Density, lbs/ft³			Рр					
Void Ratio			Pmax					
Saturation, %			Cc/C'c					

Vane Shear Test on Shelby Tubes (Maine DOT)									
Depth taken 3 In. 6 In. Water									
in tube, ft	U. Shear	Remold	U. Shear	Remold	Content	Description of Material Sampled at the Various Tube Depths			
	tons/ft²	tons/ft²	tons/ft²	tons/ft²	%	various Tube Deptils			
					•				

Comments:

Sampler requested to prioritize Limits. Insufficient amount of material to run T 88 and T 100.

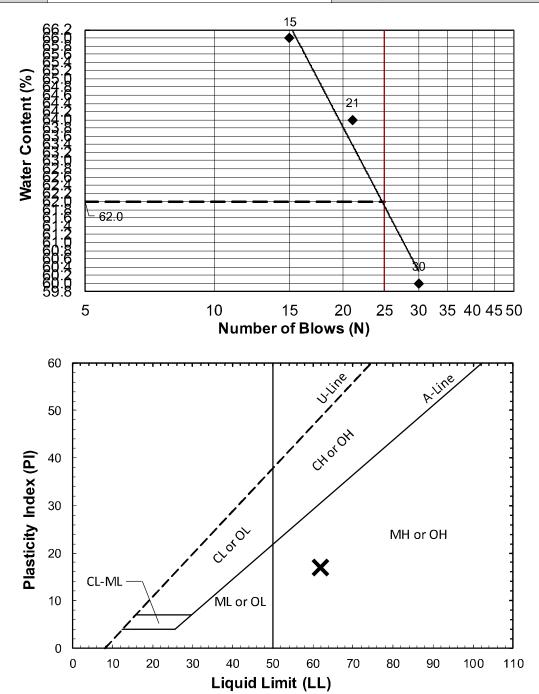
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**

Paper Copy: Lab File; Project File; Geotech File

Date Reported: 9/13/2023

TOWN	Levant	Reference No.	380879
WIN	027098.00	Water Content, %	50
Sampled	7/25/2023	Liquid Limit @ 25 blows (T 89), %	62
Boring No./Sample No.	BB-LBS-202/4D	Plastic Limit (T 90), %	45
Station	15+31.7	Plasticity Index (T 90), %	17
Depth	14.0-14.42	Tested By	BBURR





SAMPLE INFORMATION

Reference No. Boring No./Sample No. Sample Description Sampled Received

380880 BB-LBS-202/5D GEOTECHNICAL (DISTURBED) 7/25/2023 8/11/2023

Sample Type: GEOTECHNICAL Location: Station: 15+31.7 Offset, ft: 6.0 RT Dbfg, ft: 16.0-18.0

WIN/Town 027098.00 - LEVANT Sampler: NATHAN PUKAY

TEST RESULTS

Sieve Analysis (T 27)						
Wash Method	d					
SIEVE SIZE U.S. [SI]	% Passing					
3 in. [75.0 mm]						
1 in. [25.0 mm]						
3/4 in. [19.0 mm]						
½ in. [12.5 mm]						
3/4 in. [9.5 mm]						
1/4 in. [6.3 mm]						
No. 4 [4.75 mm]						
No. 10 [2.00 mm]						
No. 20 [0.850 mm]						
No. 40 [0.425 mm]						
No. 60 [0.250 mm]						
No. 100 [0.150 mm]	·					
No. 200 [0.075 mm]						

Miscellaneous Tests					
Liquid Limit @ 25 blows (T 89), %					
Plastic Limit (T 90), %					
Plasticity Index (T 90), %					
Specific Gravity, Corrected to 20°C (T 100)					
Loss on Ignition, % (T 267)	67.4				
Water Content (T 265), %	431.0				

Consolidation (T 216)								
Trimmings, Water Content, %								
	Void Ratio	% Strain						
Water Content, %			Pmin					
Dry Density, lbs/ft³			Рр					
Void Ratio			Pmax					
Saturation, %			Cc/C'c					

Vane Shear Test on Shelby Tubes (Maine DOT)								
Depth taken	3 ln.		6 In.		Water	5		
in tube, ft	U. Shear	Remold	U. Shear	Remold	Content,	Description of Material Sampled at the Various Tube Depths		
	tons/ft²	tons/ft²	tons/ft²	tons/ft²	%	various Tube Deptils		

Comments:

There are 2 sample cups. Please combine for tests.

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**

Paper Copy: Lab File; Project File; Geotech File

Date Reported: 8/28/2023



SAMPLE INFORMATION

Reference No. Boring No./Sample No. Sample Description Sampled Received

380881 BB-LBS-202/7D(A) GEOTECHNICAL (DISTURBED) 7/25/2023 8/11/2023

Sample Type: GEOTECHNICAL Location: Station: 15+31.7 Offset, ft: 6.0 RT Dbfg, ft: 21.25-22.0

WIN/Town 027098.00 - LEVANT Sampler: NATHAN PUKAY

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]					
% in. [9.5 mm]					
½ 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				
[0.0254 mm]	95.8				
[0.0170 mm]	86.5				
[0.0104 mm]	74.2				
[0.0077 mm]	64.9				
[0.0057 mm]	52.5				
[0.0030 mm]	37.1				
[0.0013 mm]	21.6				

Miscellaneous Tests					
Liquid Limit @ 25 blows (T 89), %	28				
Plastic Limit (T 90), %	20				
Plasticity Index (T 90), %	8				
Specific Gravity, Corrected to 20°C (T 100)	2.64				
Loss on Ignition, % (T 267)					
Water Content (T 265), %	31.3				

Consolidation (T 216)							
Trimmings							
	Initial	Final		Void Ratio	% Strain		
Water Content, %			Pmin				
Dry Density, lbs/ft³			Рр				
Void Ratio			Pmax				
Saturation, %			Cc/C'c				

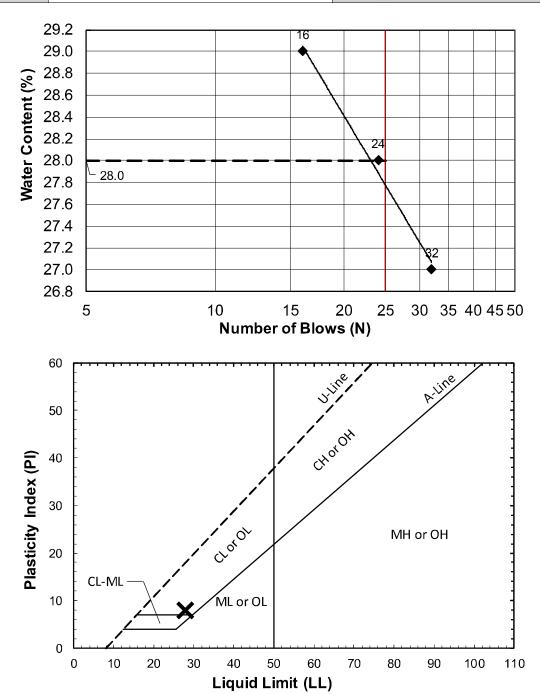
Vane Shear Test on Shelby Tubes (Maine DOT)									
Depth taken	3 I	n.	6 In.		Water				
in tube, ft	U. Shear	Remold	U. Shear	Remold	Content,	Description of Material Sampled at the Various Tube Depths			
	tons/ft²	tons/ft²	tons/ft²	tons/ft²	%	various rube Deptils			

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: 9/13/2023

TOWN	Levant	Reference No.	380881
WIN	027098.00	Water Content, %	31.3
Sampled	7/25/2023	Liquid Limit @ 25 blows (T 89), %	28
Boring No./Sample No.	BB-LBS-202/7D(A)	Plastic Limit (T 90), %	20
Station	15+31.7	Plasticity Index (T 90), %	8
Depth	21.25-22.0	Tested By	BBURR





Client: Maine Department of Transportation

317731

Project Name: Perkins Bridge 6133 & Lake Bridge 3359

Project Location: Levant. ME

Test Date: 08/28/23

Tested By: nlb
Checked By: ank

Laboratory pH of Soil by ASTM G51

Boring ID	Sample ID	Depth, ft	Description	Soil Temperature, ° C	Average pH Reading
BB-LBS-202	6D	18-20	Moist, black silt with sand	23	5.22

Notes:

Appendix D

Calculations



Liquidity Index

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

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

Stream Alluvium - Wetland Deposit

BB-LBS-202, 4D

$$WC := 50$$

$$PL := 45$$

$$LI := \frac{WC - PL}{LL - PL} = 0.29$$

Glacial Till

BB-LBS-101, 3D

$$WC := 11$$

$$LI := \frac{WC - PL}{LL - PL} = -0.5$$

BB-LBS-101, 4D

$$WC := 10$$

$$LL := 20$$

$$PL := 14$$

$$LI := \frac{WC - PL}{LL - PL} = -0.67$$

BB-LBS-101, 6D

$$WC := 12$$

$$LL := 21$$

$$LI := \frac{WC - PL}{LL - PL} = -0.29$$

BB-LBS-102, 4D

$$WC := 23$$

$$LI := \frac{WC - PL}{LL - PL} = 1.6$$

BB-LBS-202, 7D/A

$$WC := 31$$

$$PL := 20$$

$$LI := \frac{WC - PL}{LL - PL} = 1.38$$



Design of H-piles

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

Bedrock Properties

BB-LBS-101A, R1 RQD = 77%, R2 RQD = 77%

Rock Type: GRAYWACKE (hard) and PHYLLITE (moderately hard), fresh

BB-LBS-102, R1 RQD = 75%, R2 RQD = 78%

Rock Type: GRAYWACKE (hard) and PHYLLITE (moderately soft), fresh

BB-LBS-201, R1 RQD = 83%, R2 RQD = 75%

Rock Type: GRAYWACKE (moderately hard) and PHYLLITE (moderately hard), fresh

Sandstone Co = 9,700-25,000 psi Phyllite Co = 3,500-35,000 psi

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

For Design Purposes: RQD = 75%, Co = 12000 psi

Pile Properties

Use the following piles: 14x89, 14x117

$$A_g := {26.1 \choose 34.4} \cdot in^2$$

$$d := \begin{pmatrix} 13.8 \\ 14.2 \end{pmatrix} \cdot in$$

$$b := \begin{pmatrix} 14.7 \\ 14.9 \end{pmatrix} \cdot in$$

$$A_g := \binom{26.1}{34.4} \cdot in^2 \qquad \qquad d := \binom{13.8}{14.2} \cdot in \qquad \qquad b := \binom{14.7}{14.9} \cdot in \qquad \qquad t_f := \binom{0.615}{0.805} in \qquad t_w := t_f$$

Note: All matrices set up in this order

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

$$A_{\text{box}} = \begin{pmatrix} 202.86 \\ 211.58 \end{pmatrix} \cdot \text{in}^2$$

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

$$F_v := 50 \cdot ksi$$

E = Elastic Modulus

Check For Slender Members

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

LRFD eq. 6.9.4.2.1-1

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

From Table 6.9.4.2.1-1:

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

 $\lambda_{\rm rf} = 13.487$ $b_{\rm f} := 0.5 \cdot b$ $b_{\rm f} = \begin{pmatrix} 7.35 \\ 7.45 \end{pmatrix} \cdot in$

 $\frac{b_f}{t_f} = \begin{pmatrix} 11.951 \\ 9.255 \end{pmatrix}$ Both H-pile sizes are nonslender for flange members

For webs: $\lambda_{rw} := 1.09 \sqrt{\frac{E}{F_v}} \qquad \text{where b}_w \text{ = Web height/distance between flanges}$

 $\lambda_{rw} = 26.251$ $b_w := d - 2 \cdot t_f$ $b_w = \begin{pmatrix} 12.57 \\ 12.59 \end{pmatrix} \cdot in$

 $\frac{b_{w}}{t_{w}} = \begin{pmatrix} 20.439\\15.64 \end{pmatrix}$ Both H-Pile sizes are nonslender for web members

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

Use LRFD Equation 6.9.2.1-1 $Pr = \varphi_c Pn$

Nominal Axial Structural Resistance

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

$$P_o = {1305 \choose 1720} \cdot kip$$

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

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

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

K = effective length factor

 $K_{eff} := 0.65$

LRFD Table C4.6.2.5-1. Use K=0.65 for assumed segment in pure compression. Fixed top and

bottom

I = "unbraced" length

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

Assume in pure compression

LRFD eq. 6.9.4.1.2-1

$$P_e := \boxed{ \frac{\pi^2 \cdot E}{\left(\frac{K_{eff} \cdot l_{unbraced_bot}}{r_s}\right)^2 \cdot A_g}}$$

$$P_{e} = \begin{pmatrix} 2 \times 10^{8} \\ 2 \times 10^{8} \end{pmatrix} \cdot \text{kip}$$

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

$$\frac{P_{o}}{P_{e}} = \begin{pmatrix} 8.529 \times 10^{-6} \\ 8.247 \times 10^{-6} \end{pmatrix} \text{ If Po/Pe < or = 2.25, then:} \qquad P_{n} := \frac{P_{o}}{0.658 \cdot P_{o}}$$

$$P_{n} := \overbrace{\begin{pmatrix} \frac{P_{o}}{P_{e}} \\ 0.658 \end{pmatrix}^{P_{e}} \cdot P_{o}}$$

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

$$P_{\rm r} = \binom{652}{860} \cdot {\rm kip}$$

LRFD 10.7.3.2.3 - Piles Driven to Hard Rock -

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

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

Nominal Structural Resistance Previously Calculated:

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

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

$$\phi_c := 0.5$$

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

$$P_{r} = \binom{652}{860} \cdot \text{kip}$$
 14x89 14x117

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

$$\phi_c := 1.0$$
 LRFD 6.5.5

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

$$P_{r_ee} = \begin{pmatrix} 1305 \\ 1720 \end{pmatrix} \cdot \text{kip}$$
 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,

 $\varphi_{da} \coloneqq 1.0 \\ \text{Resistance factor from LRFD Table 10.5.5.2.3-1, Drivablity Analysis, steel} \\ \text{piles}$

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

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

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

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

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

 $\varphi_{dvn} \coloneqq 0.65 \qquad \qquad \text{Reference LRFD Table 10.5.5.2.3-1 - for Strength Limit State}$

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

GRLWeap Soil and Pile Model Assumptions

Abutment #1:

Based on proposed bottom of footing of elevation 119 at abutment #1, the estimated pile length will be approx. 56 feet. Assume contractor drives pile lengths of 65 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. 200 kips as skin friction based on local experience in similar deposits.

Abutment #2:

Based on proposed bottom of footing of elevation 119 at abutment #2, the estimated pile length will be approx. 40 feet. Assume contractor drives pile lengths of 45 ft (extra length accommodates for attachment of dynamic testing equipment, embedment into abutment, variation in bedrock surface).

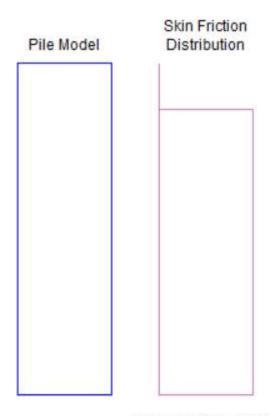
Use constant shaft resistances so that GRLWeap will assign approx. 120 kips as skin friction based on local experience in similar deposits.

22250.00 Levant Perkins Bridge #6133 November 30, 2023 by: N.Pukay Checked by: LK 1-16-24

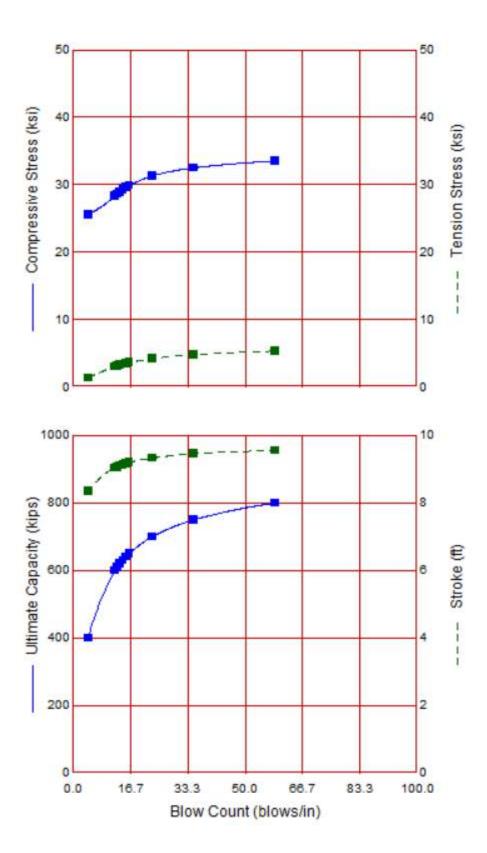
Abutment 1, Pile Size is 14 x 89, APE D19-42 Hammer

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

APE D 19-42		
Ram Weight Efficiency Pressure	4.19 0.800 1710 (100%)	kips
Helmet Weight Hammer Cushior COR of H.C.	3.00	kips kips/in
Skin Quake Toe Quake Skin Damping Toe Damping		
Pile Length Pile Penetration Pile Top Area	65.00 56.00 26.10	ft



Res. Shaft = 200.0 kips (Constant Res. Shaft) 6 of 29



Maine DOT 27098 Levant 14x89 ABT #1 D19-42

30-Nov-2023 GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	25.61	1.37	4.5	8.34	20.49
600.0	28.30	3.07	12.2	9.04	22.36
610.0	28.61	3.18	12.9	9.06	22.46
620.0	28.94	3.30	13.6	9.11	22.56
630.0	29.27	3.41	14.4	9.13	22.67
640.0	29.60	3.53	15.4	9.16	22.71
650.0	29.89	3.65	16.3	9.20	22.80
700.0	31.32	4.25	23.0	9.33	23.17
750.0	32.51	4.85	35.0	9.46	23.52
800.0	33.54	5.38	58.8	9.56	23.77

Limit to 15 bpi

$$R_{ndr} := 630 \cdot kip$$

Strength Limit State

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

$$R_{fdr} = 409 \cdot kip$$

Extreme and Service Limit States

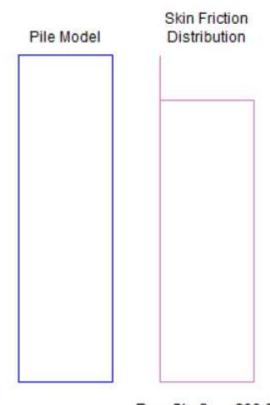
$$R_{dr} := R_{ndr} \cdot \varphi$$

$$R_{dr} = 630 \cdot kip$$

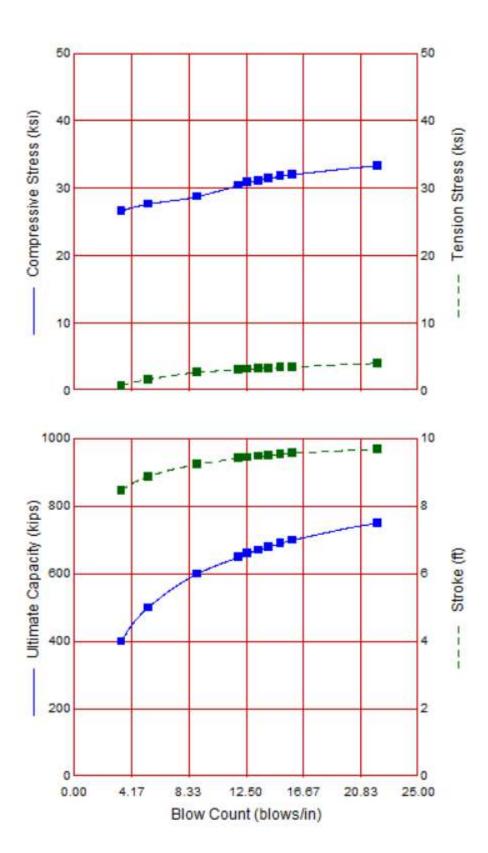
Abutment 1, Pile Size is 14 x 89, APE D25-42 Hammer

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

APE	D 25-42		
Ram (Efficie Press	C	5.51 0.800 1425 (100%)	
Hamn	et Weight ner Cushion of H.C.		kips kips/in
		0.100 0.040 0.100 0.150	in sec/ft
	ength enetration op Area	65.00 56.00 26.10	ft



Res. Shaft = 200.0 kips (Constant Res. Shaft)



Maine DOT 27098 Levant 14x89 ABT #1 D25-42 01-Dec-2023 GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	26.65	0.79	3.4	8.46	23.92
500.0	27.68	1.71	5.4	8.88	25.33
600.0	28.80	2.75	8.9	9.24	26.61
650.0	30.47	3.15	11.9	9.42	27.19
660.0	30.89	3.23	12.6	9.45	27.37
670.0	31.16	3.30	13.4	9.48	27.40
680.0	31.45	3.37	14.2	9.51	27.52
690.0	31.79	3.46	15.0	9.54	27.63
700.0	32.04	3.54	15.9	9.57	27.74
750.0	33.34	4.10	22.1	9.69	28.16

Limit to 15 bpi

 $R_{ndr} := 690 \cdot kip$

Strength Limit State

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

 $R_{fdr} = 449 \cdot kip$

Extreme and Service Limit States

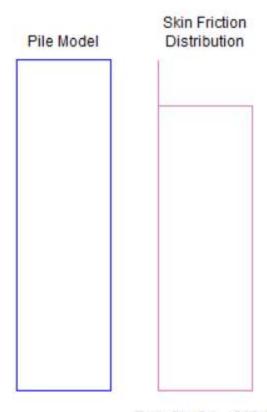
 $R_{dr} := \, R_{ndr} \cdot \varphi$

 $R_{dr} = 690 \cdot kip$

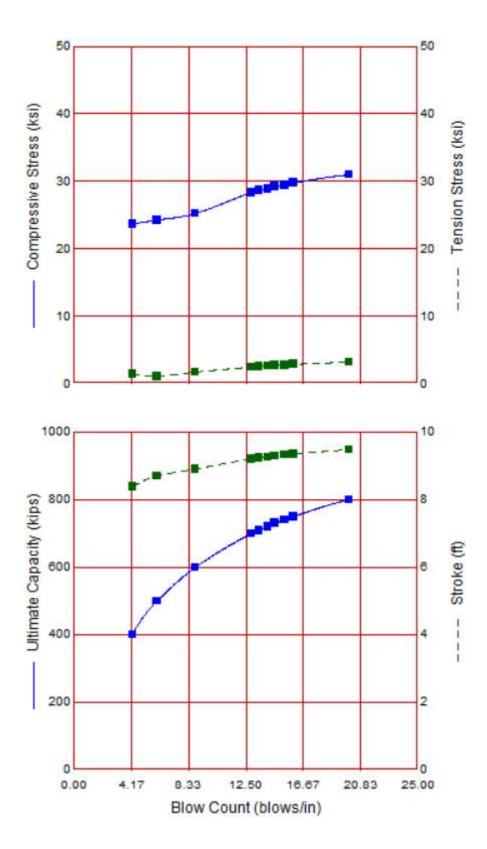
Abutment 1, Pile Size is 14 x 117, APE D19-42 Hammer

The 14x117 pile can be driven to the resistances below with a APE D19-42 hammer at fuel setting 4 (100% of max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE D 19-42		
Ram Weight Efficiency Pressure	4.19 0.800 1710 (100%)	
VALUE AND ACCOUNTS OF	1 1 2 2 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	psi
Helmet Weight	3.00	kips
Hammer Cushion	n 34825	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.100	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	65.00	ft
Pile Penetration	56.00	ft
Pile Top Area	34.40	in2



Res. Shaft = 200.0 kips (Constant Res. Shaft) 12 of 29



Maine DOT 27098 Levant 14x117 ABT #1 D19-42

01-Dec-2023 GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	23.71	1.44	4.2	8.39	19.58
500.0	24.30	1.12	6.0	8.70	20.38
600.0	25.25	1.75	8.8	8.91	20.91
700.0	28.35	2.52	12.9	9.21	21.79
710.0	28.68	2.60	13.4	9.24	21.88
720.0	28.91	2.68	14.1	9.26	21.90
730.0	29.26	2.75	14.6	9.30	22.05
740.0	29.44	2.83	15.3	9.33	22.06
750.0	29.76	2.89	15.9	9.35	22.16
800.0	30.99	3.23	20.0	9.48	22.50

Limit to 15 bpi

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

Strength Limit State

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

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

Extreme and Service Limit States

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

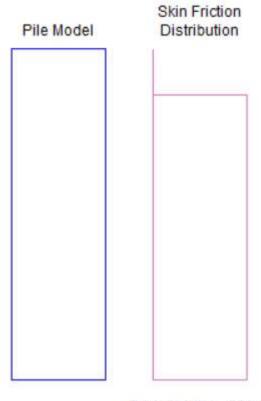
$$R_{dr} = 730 \cdot kip$$

Levant Perkins Bridge #6133 November 30, 2023 by: N.Pukay Checked by: LK 1-16-24

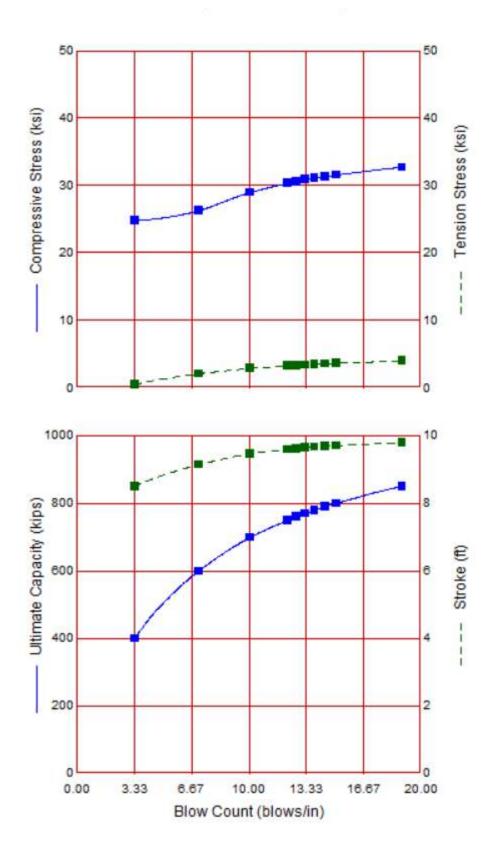
Abutment 1, Pile Size is 14 x 117, APE D25-42 Hammer

The 14x117 pile can be driven to the resistances below with a APE D25-42 hammer at fuel setting 4 (100% of max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE	D 25-42		
Ram (Efficie Press		5.51 0.800 1425 (100%)	kips psi
Hamn	et Weight ner Cushion of H.C.		kips kips/in
	Table 1		
	ength enetration op Area	65.00 56.00 34.40	ft



Res. Shaft = 200.0 kips (Constant Res. Shaft) 15 of 29



Maine DOT 27098 Levant 14x117 ABT #1 D25-42

01-Dec-2023 GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	24.77	0.53	3.4	8.51	22.67
600.0	26.28	2.12	7.1	9.14	24.58
700.0	28.98	2.90	10.1	9.47	25.68
750.0	30.32	3.29	12.3	9.59	26.05
760.0	30.58	3.37	12.8	9.61	26.10
770.0	30.87	3.44	13.3	9.64	26.22
780.0	31.06	3.52	13.8	9.66	26.30
790.0	31.34	3.59	14.4	9.68	26.35
800.0	31.55	3.67	15.1	9.70	26.42
850.0	32.70	4.02	18.9	9.80	26.76

Limit to 15 bpi

$$R_{ndr} := 790 \cdot kip$$

Strength Limit State

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

$$R_{fdr} = 514 \cdot kip$$

Extreme and Service Limit States

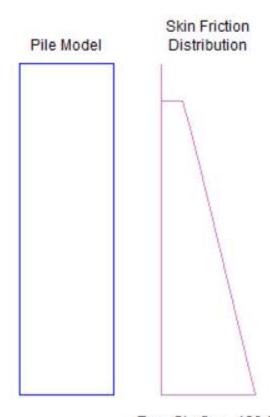
$$R_{dr} := R_{ndr} \cdot \varphi$$

$$R_{dr} = 790 \cdot kip$$

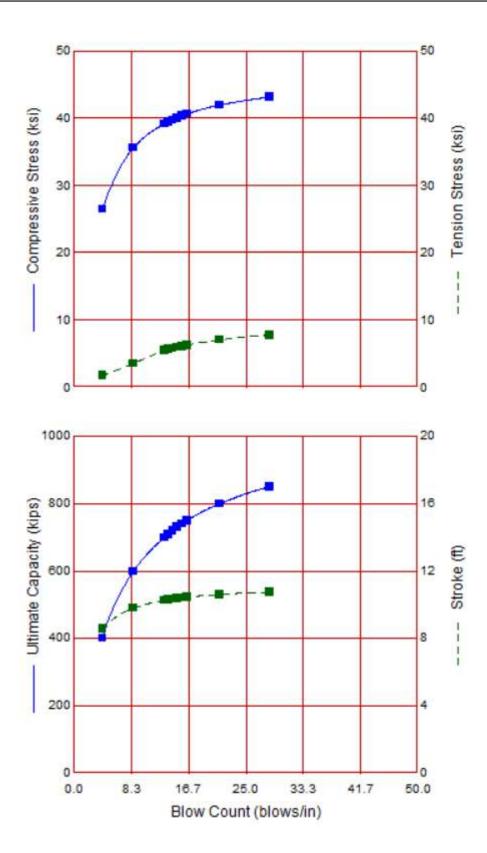
Abutment 2, Pile Size is 14 x 89, APE D19-42 Hammer

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

APE D 19-42		
Ram Weight Efficiency Pressure	4.19 0.800 1710 (100%)	kips nsi
Helmet Weight Hammer Cushior COR of H.C.	3.00	kips kips/in
Skin Quake Toe Quake Skin Damping Toe Damping	0.100 0.040 0.100 0.150	in
Pile Length Pile Penetration Pile Top Area	45.00 40.00 26.10	ft



Res. Shaft = 120.0 kips (Constant Res. Shaft)



Maine DOT 27098 Levant 14x89 ABT #2 D19-42

01-Dec-2023 GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	26.56	1.78	4.1	8.56	20.72
600.0	35.63	3.62	8.6	9.80	23.47
700.0	39.17	5.49	13.1	10.30	24.67
710.0	39.47	5.67	13.7	10.33	24.76
720.0	39.74	5.83	14.3	10.37	24.84
730.0	40.04	6.00	15.0	10.41	24.94
740.0	40.34	6.16	15.7	10.44	25.03
750.0	40.64	6.32	16.4	10.48	25.17
800.0	41.95	7.10	21.1	10.64	25.57
850.0	43.13	7.83	28.5	10.78	25.91

Limit to 15 bpi

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

Strength Limit State

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

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

Extreme and Service Limit States

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

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

22250.00

Perkins Bridge #6133

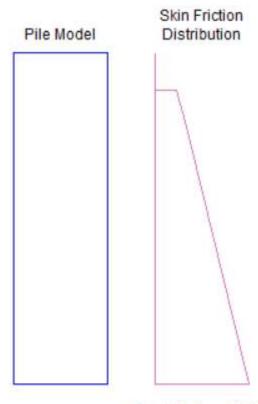
Levant

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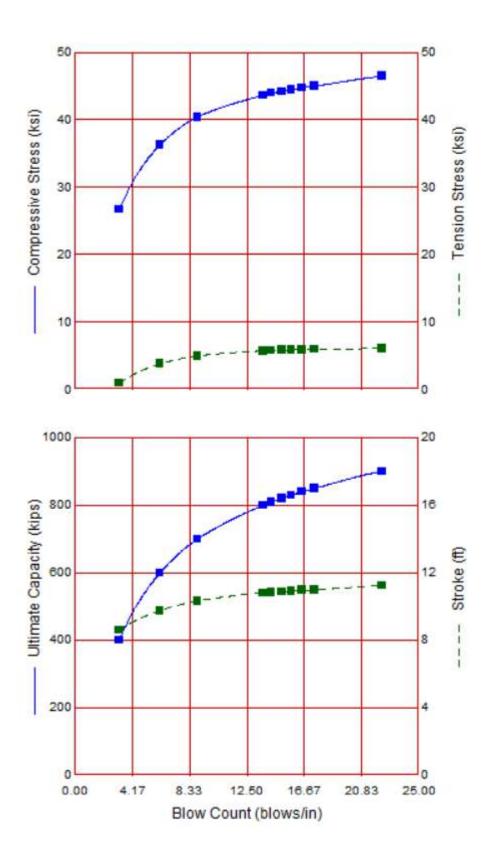
Abutment 2, Pile Size is 14 x 89, APE D25-42 Hammer

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

APE	D 25-42		
Ram W Efficien	icy	0.800	kips
Pressu	ire	1425 (100%)	psi
	t Weight er Cushion f H.C.	3.00 34825 0.800	(-17 M) (-7 -)
Skin Qu Toe Qu Skin Da Toe Da	ake amping		717-470
Pile Le Pile Pe Pile To	netration	45.00 40.00 26.10	ft



Res. Shaft = 120.0 kips (Constant Res. Shaft) 21 of 29



Maine DOT 27098 Levant 14x89 ABT #2 D25-42

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Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	26.75	1.00	3.2	8.60	23.89
600.0	36.30	3.84	6.2	9.75	26.56
700.0	40.41	5.03	8.9	10.35	28.26
800.0	43.63	5.78	13.6	10.82	29.70
810.0	43.97	5.82	14.3	10.86	29.84
820.0	44.20	5.86	15.0	10.89	29.92
830.0	44.47	5.90	15./	10.93	30.07
840.0	44.75	5.95	16.5	10.97	30.21
850.0	44.98	5.97	17.4	11.00	30.29
900.0	46.43	6.14	22.3	11.25	31.06

Limit to 15 bpi

 $R_{ndr} := 820 \cdot kip$

Strength Limit State

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

 $R_{fdr} = 533 \cdot kip$

Extreme and Service Limit States

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

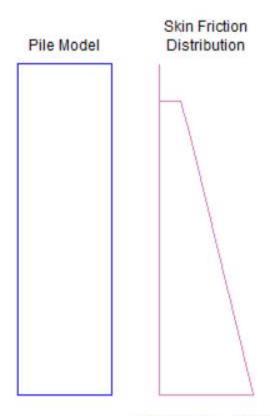
 $R_{dr} = 820 \cdot kip$

22250.00 Levant Perkins Bridge #6133 November 30, 2023 by: N.Pukay Checked by: LK 1-16-24

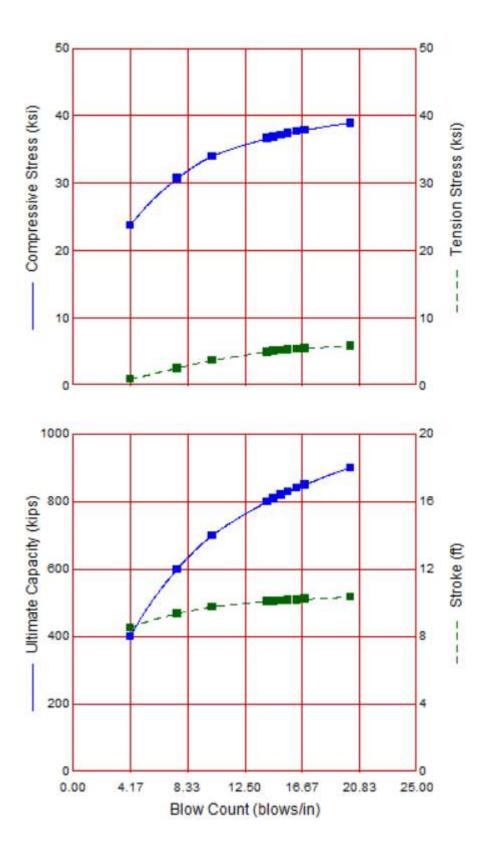
Abutment 2, Pile Size is 14 x 117, APE D19-42 Hammer

The 14x117 pile can be driven to the resistances below with a APE D19-42 hammer at fuel setting 4 (100% of max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE D 19-42		
Ram Weight Efficiency	4.19 0.800	kips
Pressure	1710 (100%)	psi
Helmet Weight Hammer Cushior COR of H.C.		kips kips/in
Skin Quake Toe Quake Skin Damping Toe Damping		
Pile Length Pile Penetration Pile Top Area	45.00 40.00 34.40	ft



Res. Shaft = 120.0 kips (Constant Res. Shaft) 24 of 29



Maine DOT 27098 Levant 14x117 ABT #2 D19-42

01-Dec-2023 GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	23.79	0.99	4.2	8.52	19.98
600.0	30.70	2.62	7.6	9.36	21.73
700.0	34.00	3.77	10.1	9.78	22.86
800.0	36.63	5.01	14.1	10.09	23.60
810.0	36.89	5.14	14.6	10.12	23.70
820.0	37.15	5.26	15.1	10.15	23.78
830.0	37.39	5.38	15.6	10.18	23.87
840.0	37.66	5.47	16.3	10.20	23.91
850.0	37.88	5.55	16.8	10.23	23.99
900.0	38.92	5.94	20.2	10.36	24.32

Limit to 15 bpi

$$R_{ndr} := 810 \cdot kip$$

Strength Limit State

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

$$R_{fdr} = 527 \cdot kip$$

Extreme and Service Limit States

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

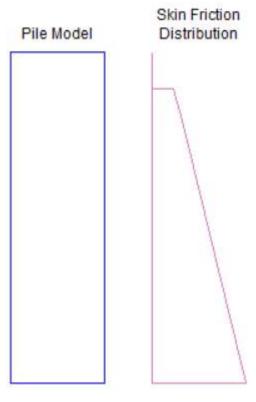
 $R_{dr} = 810 \cdot kip$

November 30, 2023 by: N.Pukay Checked by: LK 1-16-24

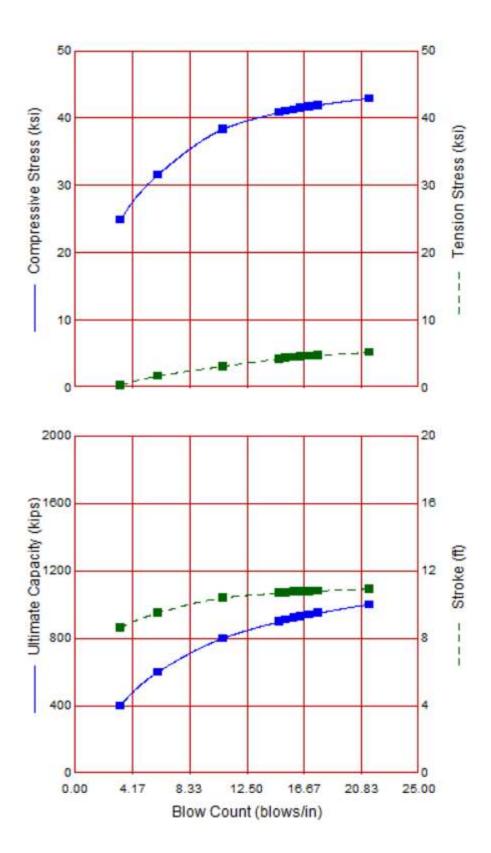
Abutment 2, Pile Size is 14 x 117, APE D25-42 Hammer

The 14x117 pile can be driven to the resistances below with a APE D25-42 hammer at fuel setting 4 (100% of max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE	D 25-42		
Ram (Efficie Press		5.51 0.800 1425 (100%)	kips psi
Hamn	et Weight ner Cushion of H.C.	3.00 34825 0.800	kips kips/in
150000	ength enetration op Area	45.00 40.00 34.40	ft



Res. Shaft = 120.0 kips (Constant Res. Shaft) 27 of 29



Maine DOT 27098 Levant 14x117 ABT #2 D25-42

01-Dec-2023 GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	24.89	0.41	3.3	8.63	22.79
600.0	31.56	1.75	6.0	9.51	24.78
800.0	38.31	3.19	10.8	10.41	27.58
900.0	40.82	4.30	14.8	10.69	28.44
910.0	41.04	4.41	15.3	10.72	28.56
920.0	41.23	4.50	15.9	10.75	28.62
930.0	41.50	4.61	16.4	10.77	28.74
940.0	41.68	4.70	17.0	10.79	28.79
950.0	41.91	4.79	17.7	10.82	28.85
1000.0	42.89	5.26	21.4	10.93	29.21

Limit to 15 bpi

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

Strength Limit State

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

 $R_{fdr} = 585 \cdot kip$

Extreme and Service Limit States

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

 $R_{dr} = 900 \cdot kip$

2 HP14x117 45 APE D19-42 4 0.10 0.04 0.10 0.15 120 810 36.89 5.14 14.6 10.12 2 HP14x117 45 APE D25-42 3 0.10 0.04 0.05 0.15 120 830 37.23 3.77 14.9 9.35		Abutment	Pile Size	Pile Length	Hammer	Fuel Setting	Shaft Quake	Toe Quake	Shaft Damping	Toe Damping	Skin Friction	Ultimate Capacity	Max Comp Stress	Max Tension Stress	Blows/In	Stroke	Energy
Abutment #1 14x88		1	HP 14x89	65	APE D19-42	3	0.10	0.04	0.05	0.15	200	620	29.12	4.28	14.4	8.19	19.70
APE D19-42 APE D1		1	HP 14x89	65	APE D19-42	3	0.10	0.04	0.10	0.15	200	580	26.06	2.76	14.5	8.12	19.23
APE D19-42 1		1	HP 14x89	65	APE D19-42	3	0.10	0.07	0.05	0.15	200	610	27.88	3.06	15.0	8.09	19.46
1	Abutment #1 14x89	1	HP 14x89	65	APE D19-42	3	0.10	0.07	0.10	0.15	200	570	24.99	1.75	15.0	8.03	19.03
1	APE D19-42	1	HP 14x89	65	APE D19-42	4	0.10	0.04	0.05	0.15	200	680	32.92	5.29	15.0	9.23	23.20
1		1	HP 14x89	65	APE D19-42	4	0.10	0.04	0.10	0.15	200	630	29.27	3.41	14.4	9.13	22.67
Abutment #1 14x17 APE D19-42 Abutment #1 14x117 APE D19-42 ABUTMENT Abutment #1 Abutm		1	HP 14x89	65	APE D19-42	4		0.07	0.05	0.15	200		31.47			9.10	22.88
Abutment #1 14x17 APE D19-42 Abutment #1 14x17 APE D19-42 Abutment #1 14x17 APE D25-42 APE L1 H=P14889 65 APE D25-42 3 0.10 0.07 0.05 0.15 200 650 28.99 3.04 14.9 8.42 APE D25-42 APE D25-		1	HP 14x89	65	APE D19-42	4	0.10	0.07	0.10	0.15	200	620	27.89	2.39	15.0	9.04	22.43
Abutment #1 14x89 1		1	HP 14x89	65	APE D25-42	3	0.10	0.04	0.05	0.15	200	660	30.45	4.18	14.9	8.43	22.89
Apt the H 1 1 4x89		1	HP 14x89	65	APE D25-42	3	0.10	0.04	0.10	0.15	200	620	27.40	2.94	14.6	8.43	22.56
APE D25-42 APE D2		1	HP 14x89	65	APE D25-42	3	0.10	0.07	0.05	0.15	200	650	28.99	3.04	14.9	8.42	23.03
1	Abutment #1 14x89	1	HP 14x89	65	APE D25-42	3	0.10	0.07	0.10	0.15	200	610	26.38	1.86	15.0	8.35	22.38
1	APE D25-42	1	HP 14x89	65	APE D25-42	4	0.10	0.04	0.05	0.15	200	730	35.40	4.83	14.4	9.64	28.37
He He He He He He He He		1	HP 14x89	65	APE D25-42	4	0.10	0.04	0.10	0.15	200	690	31.79	3.46	15.0	9.54	27.63
Abutment #1 14x117 APE D19-42 Abutment #1 14x117 APE D29-42 Abutment #1 14x117 APE D29-42 Abutment #1 14x117 APE D39-42 Abutment #1 14x117 APE D49-42 Abutment #1 4x147 APE D49-42 ABUTMANA ABUTM		1	HP 14x89	65	APE D25-42	4	0.10	0.07	0.05	0.15	200	710	33.39	3.65	14.3	9.51	28.02
Abutment #1 14x117 APE D19-42 APE D19-43 APE D19-42 APE D19-42		1	HP 14x89	65	APE D25-42	4	0.10	0.07	0.10	0.15	200	670	29.63	2.25	14.8	9.43	27.30
Abutment #1 14x117		1	HP14x117	65	APE D19-42	3	0.10	0.04	0.05	0.15	200	710	28.34	3.04	14.6	8.32	19.06
Abutment #1 14x117 APE D19-42 APE D19-42		1	HP14x117	65	APE D19-42	3	0.10	0.04	0.10	0.15	200	670	26.02	2.29	14.7	8.25	18.70
APE D19-42 APE D19-42 I PIP14x117 65 APE D19-42 4 0.10 0.04 0.05 0.15 200 780 32.05 3.65 14.9 9.39 I HP14x117 65 APE D19-42 4 0.10 0.07 0.05 0.15 200 750 30.45 2.31 14.7 9.25 I HP14x117 65 APE D19-42 4 0.10 0.07 0.05 0.15 200 750 30.45 2.31 14.7 9.25 I HP14x117 65 APE D19-42 4 0.10 0.07 0.05 0.15 200 750 30.45 2.31 14.7 9.25 I HP14x117 65 APE D19-42 4 0.10 0.07 0.10 0.15 200 750 30.45 2.31 14.7 9.25 I HP14x117 65 APE D25-42 3 0.10 0.04 0.10 0.15 200 760 29.89 3.66 14.8 8.63 I HP14x117 65 APE D25-42 3 0.10 0.04 0.10 0.15 200 760 29.89 3.66 14.8 8.63 I HP14x117 65 APE D25-42 3 0.10 0.07 0.05 0.15 200 770 27.48 2.90 14.8 8.58 I HP14x117 65 APE D25-42 3 0.10 0.07 0.05 0.15 200 730 27.78 2.58 14.4 8.50 APE D25-42 1 HP14x17 65 APE D25-42 3 0.10 0.07 0.05 0.15 200 730 27.78 2.58 14.4 8.50 APE D25-42 1 HP14x117 65 APE D25-42 4 0.10 0.07 0.05 0.15 200 730 27.78 2.58 14.4 8.50 APE D25-42 1 HP14x117 65 APE D25-42 4 0.10 0.07 0.05 0.15 200 730 27.78 2.58 14.4 9.68 APE D19-42 1 HP14x117 65 APE D25-42 4 0.10 0.04 0.05 0.15 200 790 31.34 3.59 14.4 9.68 APE D19-42 2 HP14x89 45 APE D19-42 3 0.10 0.04 0.05 0.15 200 770 29.44 2.58 11.8 9.58 APE D19-42 2 HP14x89 45 APE D19-42 3 0.10 0.04 0.05 0.15 120 660 35.58 5.11 14.7 9.08 APE D25-42 4 0.10 0.04 0.05 0.15 120 760 42.73 6.56 11.8 10.5 9.66 APE D25-42 4 0.10 0.04 0.05 0.15 120 760 42.73 6.56 11.8 10.5 9.66 APE D25-42 4 0.10 0.04 0.05 0.15 120 760 42.73 6.56 11.8 10.5 9.00 ADAUTHENT #1 14x89 45 APE D19-42 3 0.10 0.04 0.05 0.15 120 760 42.73 6.56 11.4 14.7 9.08 APE D19-42 4 HP14x89 45 APE D25-42 3 0.10 0.04 0.05 0.15 120 760 42.73 6.56 11.4 14.7 9.08 APE D19-42 4 HP14x89 45 APE D19-42 3 0.10 0.04 0.05 0.15 120 770 34.93 4.96 14.7 10.30 APE D19-42 4 HP14x89 45 APE D19-42 3 0.10 0.04 0.05 0.15 120 770 34.93 4.96 14.7 10.50 APE D19-42 4 HP14x89 45 APE D19-42 3 0.10 0.04 0.05 0.15 120 770 34.93 4.96 14.7 10.50 APE D19-42 4 HP14x89 45 APE D19-42 3 0.10 0.04 0.05 0.15 120 880 44.09 5.85 1.1 14.7 10.24 APE D19-42 2 HP14x817 45 APE D19-42 3 0.10 0.04 0.05 0.15 120		1	HP14x117	65	APE D19-42	3	0.10	0.07	0.05	0.15	200	680	26.63	1.99	14.3	8.18	18.70
HP14x117 65 APE D19-42 4 0.10 0.04 0.10 0.15 200 730 29.26 2.75 14.6 9.30	Abutment #1 14x117	1	HP14x117	65	APE D19-42	3	0.10	0.07	0.10	0.15	200	640	24.02	1.50	14.4	8.13	18.34
1	APE D19-42	1	HP14x117	65	APE D19-42	4	0.10	0.04	0.05	0.15	200	780	32.05	3.65	14.9	9.39	22.48
HPI4x117 65 APE D19-42 4 0.10 0.07 0.10 0.15 200 700 27.40 1.85 14.4 9.16		1	HP14x117	65	APE D19-42	4	0.10	0.04	0.10	0.15	200	730	29.26	2.75	14.6	9.30	22.05
Abutment #1 14x117 APE D25-42 AButment #1 14x117 APE D19-42 APE D25-42 APE D19-42 APE		1	HP14x117	65	APE D19-42	4	0.10	0.07	0.05	0.15	200	750	30.45	2.31	14.7	9.25	22.15
Abutment #1 14x17		1	HP14x117	65	APE D19-42	4	0.10	0.07	0.10	0.15	200	700	27.40	1.85	14.4	9.16	21.66
Abutment #1 14x117		1	HP14x117	65	APE D25-42	3	0.10	0.04	0.05	0.15	200	760	29.89	3.66	14.8	8.63	22.13
Abutment #1 14x117 APE D25-42 APE		1	HP14x117	65	APE D25-42	3	0.10	0.04	0.10	0.15	200	720	27.48	2.90	14.8	8.58	21.71
APE D25-42 APE D25-42 APE D25-42 APE D25-42 APE D25-42 A 0.10 O.04 O.05 O.05 O.05 O.05 O.05 O.05 O.05 O.05 O.05 O.07 O.09 O.07 O.09 O.09		1	HP14x117	65	APE D25-42	3	0.10	0.07	0.05	0.15	200	730	27.78	2.58	14.4	8.50	21.79
1	Abutment #1 14x117	1	HP14x117	65	APE D25-42	3	0.10	0.07	0.10	0.15	200	690	25.17	2.07	14.5	8.46	21.30
HP14x117 G5	APF D25-42	1	HP14x117	65	APE D25-42	4	0.10	0.04	0.05	0.15	200	840	34.30	4.19	14.7	9.75	26.95
HP14x117 G5	7.1.2.3.23 1.2	1	HP14x117	65	APE D25-42	4	0.10	0.04	0.10	0.15	200	790	31.34	3.59	14.4	9.68	26.35
Abutment #2 14x89 APE D19-42 Abutment #2 14x89 APE D19-42 APE D19-		1	HP14x117	65	APE D25-42	4	0.10	0.07	0.05	0.15	200	820	32.62	3.16	15.0	9.66	26.81
Abutment #2 14x89		1	HP14x117	65	APE D25-42	4	0.10	0.07	0.10	0.15	200	770	29.44	2.58	14.8	9.58	26.13
APE D19-42 2 HP 14x89 45 APE D19-42 4 0.10 0.04 0.05 0.15 120 760 42.73 6.56 14.8 10.57 2 HP 14x89 45 APE D19-42 4 0.10 0.04 0.10 0.15 120 730 40.04 6.00 15.0 10.41 Abuttment #2 14x89 2 HP 14x89 45 APE D25-42 3 0.10 0.04 0.05 0.15 120 760 41.03 4.99 14.9 9.55 APE D25-42 2 HP 14x89 45 APE D25-42 3 0.10 0.04 0.05 0.15 120 730 38.50 4.91 15.0 9.42 APE D25-42 2 HP 14x89 45 APE D25-42 4 0.10 0.04 0.05 0.15 120 730 38.50 4.91 15.0 9.42 APE D25-42 2 HP 14x89 45 APE D25-42 4 0.10 0.04 0.05 0.15 120 850 46.99 5.93 14.7 11.03 APE D19-42 2 HP 14x117 45 APE D19-42 3 0.10 0.04 0.05 0.15 120 820 44.20 5.86 15.0 10.89 APE D19-42 2 HP 14x117 45 APE D19-42 3 0.10 0.04 0.05 0.15 120 770 34.93 4.96 14.7 9.08 APE D19-42 2 HP 14x117 45 APE D19-42 4 0.10 0.04 0.05 0.15 120 750 33.44 4.50 15.0 9.00 APE D19-42 2 HP 14x117 45 APE D19-42 4 0.10 0.04 0.05 0.15 120 840 38.86 5.72 14.7 10.24 2 HP 14x117 45 APE D19-42 4 0.10 0.04 0.05 0.15 120 840 38.86 5.72 14.7 10.24 2 HP 14x117 45 APE D19-42 3 0.10 0.04 0.05 0.15 120 840 38.86 5.72 14.7 10.24 2 HP 14x117 45 APE D19-42 4 0.10 0.04 0.05 0.15 120 840 36.89 5.14 14.6 10.12		2	HP 14x89	45	APE D19-42	3	0.10	0.04	0.05	0.15	120	690	37.99	5.67	15.0	9.22	21.37
Abutment #2 14x117 APE D19-42	Abutment #2 14x89	2	HP 14x89	45	APE D19-42	3	0.10	0.04	0.10	0.15	120	660	35.58	5.11	14.7	9.08	20.97
Abutment #2 14x19	APF D19-42	2	HP 14x89	45	APE D19-42	4	0.10	0.04	0.05	0.15	120	760	42.73	6.56	14.8	10.57	25.43
Abutment #2 14x89	711 2 3 12	2	HP 14x89	45	APE D19-42	4	0.10	0.04	0.10	0.15	120	730	40.04	6.00	15.0	10.41	24.94
Abutment #2 14x89		2	HP 14x89	45	APE D25-42	3	0.10	0.04	0.05	0.15	120	760	41.03	4.99	14.9	9.55	24.74
APE D25-42 2 HP 14x89 45 APE D25-42 4 0.10 0.04 0.05 0.15 120 850 46.99 5.93 14.7 11.03 2 HP 14x89 45 APE D25-42 4 0.10 0.04 0.10 0.15 120 820 44.20 5.86 15.0 10.89 Abutment #2 14x117 2 HP14x117 45 APE D19-42 3 0.10 0.04 0.05 0.15 120 770 34.93 4.96 14.7 9.08 APE D19-42 1 HP14x117 45 APE D19-42 3 0.10 0.04 0.10 0.15 120 750 33.44 4.50 15.0 9.00 APE D19-42 2 HP14x117 45 APE D19-42 4 0.10 0.04 0.05 0.15 120 840 38.86 5.72 14.7 10.24 2 HP14x117 45 APE D19-42 4 0.10 0.04 0.10 0.15 120 840 36.89 5.14 14.6 10.12 3 HP14x117 45 APE D19-42 3 0.10 0.04 0.05 0.15 120 810 36.89 5.14 14.6 10.12	Abutment #2 14x89	2	HP 14x89	45	APE D25-42	3		0.04	0.10	0.15	120	730	38.50	4.91	15.0		24.29
Abutment #2 14x117		2	HP 14x89	45	APE D25-42	4	-	0.04	0.05	0.15	120	850					30.39
Abutment #2 14x117	/ \(\(\bullet \bulle																29.92
Abutment #2 14x117 2 HP14x117 45 APE D19-42 3 0.10 0.04 0.10 0.15 120 750 33.44 4.50 15.0 9.00 APE D19-42 2 HP14x117 45 APE D19-42 4 0.10 0.04 0.05 0.15 120 840 38.86 5.72 14.7 10.24 2 HP14x117 45 APE D19-42 4 0.10 0.04 0.10 0.15 120 810 36.89 5.14 14.6 10.12 3 HP14x117 45 APE D25-42 3 0.10 0.04 0.05 0.15 120 830 37.23 3.77 14.9 9.35																	20.58
APE D19-42 2 HP14x117 45 APE D19-42 4 0.10 0.04 0.05 0.15 120 840 38.86 5.72 14.7 10.24 2 HP14x117 45 APE D19-42 4 0.10 0.04 0.10 0.15 120 810 36.89 5.14 14.6 10.12 3 HP14x117 45 APE D25-42 3 0.10 0.04 0.05 0.15 120 830 37.23 3.77 14.9 9.35					_												20.34
2 HP14x117 45 APE D19-42 4 0.10 0.04 0.10 0.15 120 810 36.89 5.14 14.6 10.12 2 HP14x117 45 APE D25-42 3 0.10 0.04 0.05 0.15 120 830 37.23 3.77 14.9 9.35																10.24	24.10
2 HP14x117 45 APE D25-42 3 0.10 0.04 0.05 0.15 120 830 37.23 3.77 14.9 9.35																	23.70
							-										23.41
	Abutment #2 14x117	2	HP14x117	45	APE D25-42	3	0.10	0.04	0.10	0.15	120	800	35.35	3.46	14.7	9.25	23.00
	APE D25-42						 						<u> </u>				28.97
APE D25-42 2 HP14x117 45 APE D25-42 4 0.10 0.04 0.05 0.15 120 900 40.82 4.30 14.8 10.69																	28.44

Hammer Information:

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

D19-42 #1 1247 psi #2 1385 psi #3 1539 psi #4 1710 psi

D25-42 #1 1040 psi #2 1155 psi #3 1280 psi #4 1425 psi

HIGHWAY BRIDGES

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

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

⁽¹⁾Range of Uniaxial Compressive Strength values reported by various investigations.

(2)Not including oil shale.

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

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

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

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

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

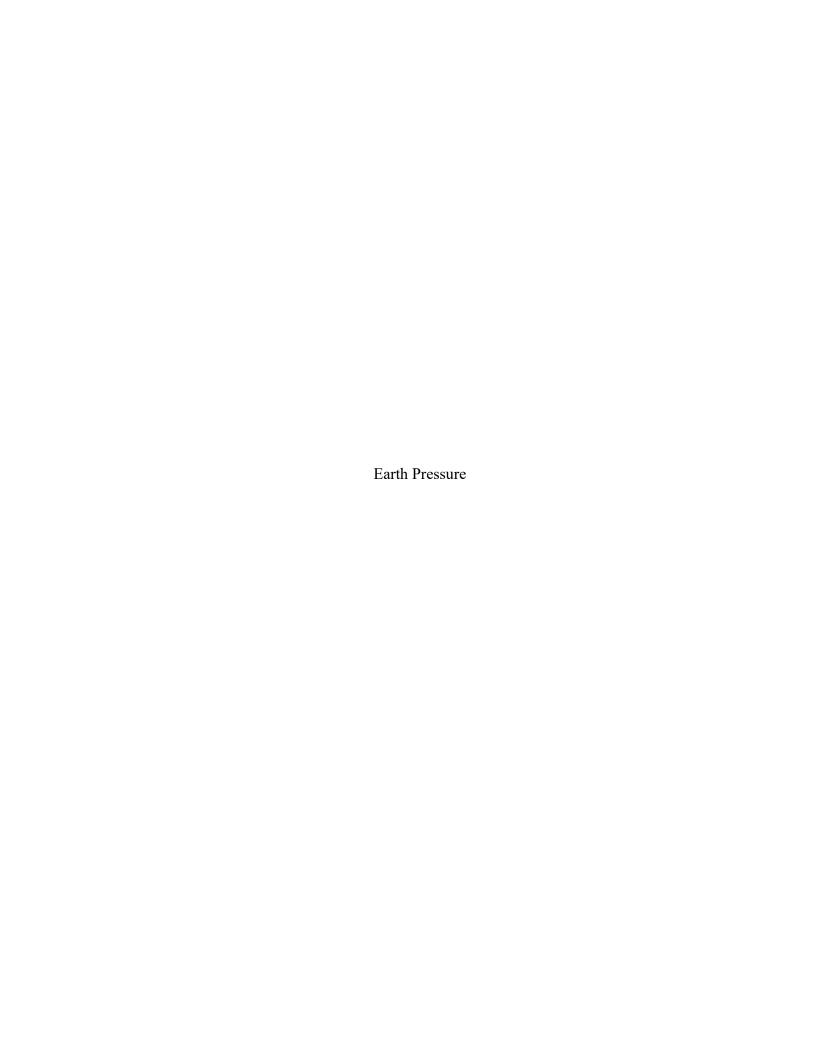
For preliminary design or when site-specific test data cannot be obtained, guidelines for estimating values of E_{o} (such as presented in Table 4.4.8.2.2B or Figure 4.4.8.2.2A) may be used. For preliminary analyses or for final design when in-situ test results are not available, a value of $\alpha_{\text{E}}=0.15$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

4.4.8.2.3 Tolerable Movement

Refer to Article 4.4.7.2.3.

4.4.9 Overall Stability

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



Earth Pressure:

Backfill engineering strength parameters

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

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

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

Cohesion $c_1 := 0 \cdot psf$

Integral Abutment - Passive Earth Pressure - Coulomb Theory

 α = Angle of fill slope to the horizontal $\alpha := -0.57 \cdot \deg$

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

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

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

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

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

 $\delta' := 17 \cdot \deg$

 $K_{p_coulomb} = 5.85$

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

Integral Abutment and Wingwall - Passive Earth Pressure - Rankine Theory

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

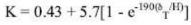
 α = Angle of fill slope to the horizontal $\alpha := -0.57 \cdot \text{deg}$

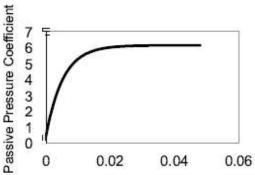
$$\mathsf{K}_{\mathsf{p_rank}} \coloneqq \cos(\alpha) \cdot \frac{\cos(\alpha) + \sqrt{\cos(\alpha)^2 - \cos(\varphi')^2}}{\cos(\alpha) - \sqrt{\cos(\alpha)^2 - \cos(\varphi')^2}} \\ \mathsf{Das}, \text{ Principles of Foundation Engineering 7th Ed. p. 363 Eq. 7.67}$$

 $K_{p_rank} = 3.25$

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

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





Relative Wall Displacement

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

Thermal displacement at each abutment: $\delta := 0.43$ in

Abutment height: h := 11ft $h = 132 \cdot in$

Relative wall displacement: $\frac{\delta}{h} = 0.0033$

 $K := 0.43 + 5.7 \cdot [1 - \exp[-190(0.0033)]]$

K = 3.09

< K_{p rank} of 3.25, therefore recommend K=3.25

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

	Friction	Coefficient of
	Angle, δ	Friction, tan δ
Interface Materials	(degrees)	(dim.)
Mass concrete on the following foundation materials:		
Clean sound rock	35	0.70
Clean gravel, gravel-sand mixtures, coarse sand	29 to 31	0.55 to 0.60
Clean fine to medium sand, silty medium to coarse sand, silty or clayey	2, 10 0 1	0,00 000
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
and the state of t		
Masonry on foundation materials has same friction factors.		
Steel sheet piles against the following soils:		
Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	22	0.40
Clean sand, silty sand-gravel mixture, single-size hard rock fill	17	0.31
Silty sand, gravel or sand mixed with silt or clay	14	0.25
• Fine sandy silt, nonplastic silt	11	0.19
Formed or precast concrete or concrete sheet piling against the following		
soils:		
	22 to 26	0.40 to 0.49
 Clean gravel, gravel-sand mixture, well-graded rock fill with spalls 	17 to 22	0.31 to 0.40
Clean sand, silty sand-gravel mixture, single-size hard rock fill	17	0.31
Silty sand, gravel or sand mixed with silt or clay	· 14	0.25
• Fine sandy silt, nonplastic silt		
Various structural materials:		
Masonry on masonry, igneous and metamorphic rocks:		
o dressed soft rock on dressed soft rock	35	0.70
o dressed hard rock on dressed soft rock	33	0.65
o dressed hard rock on dressed hard rock	29	0.55
Masonry on wood in direction of cross grain	26	0.49
Steel on steel at sheet pile interlocks	17	0.31

3.11.5.4—Passive Lateral Earth Pressure Coefficient, k_n

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

For cohesive soils, passive pressures may be estimated by:

C3.11.5.4

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

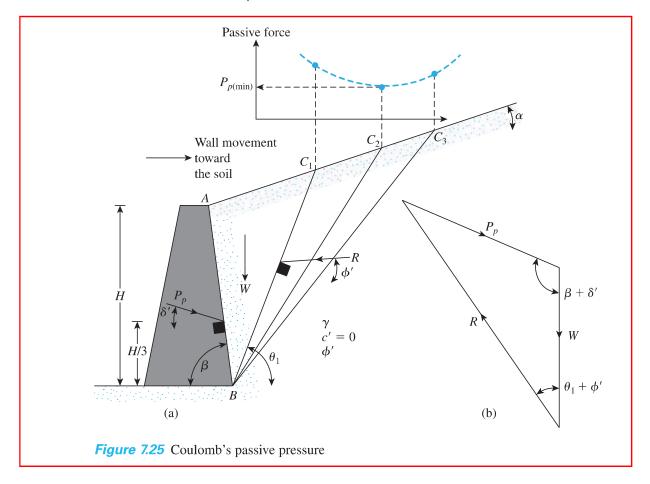
lable 7.9 (Continued)						
		c ' /γz				
$oldsymbol{\phi}'$ (deg)	lpha (deg)	0.025	0.050	0.100	0.500	
30	0	3.087	3.173	3.346	4.732	
	5	3.042	3.129	3.303	4.674	
	10	2.907	2.996	3.174	4.579	
	15	2.684	2.777	2.961	4.394	

7.12 Coulomb's Passive Earth Pressure

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

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

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



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

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

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

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

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

where

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

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

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

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

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

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

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

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

$$\sigma'_p = \sigma'_o K_{p(2)} + 2c'_2 \sqrt{K_{p(2)}} = 31.44(2.56) + 2(10)\sqrt{2.56}$$

= 80.49 + 32 = 112.49 kN/m²

Again, at z = 3 m,

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

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

Hence,

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

= 135.65 kN/m²

Note that, because a water table is present, the hydrostatic stress, u, also has to be taken into consideration. For z = 0 to 2 m, u = 0; z = 3 m, $u = (1)(\gamma_w) = 9.81$ kN/m².

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

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

7.11 Rankine Passive Earth Pressure: Vertical Backface and Inclined Backfill

Granular Soil

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

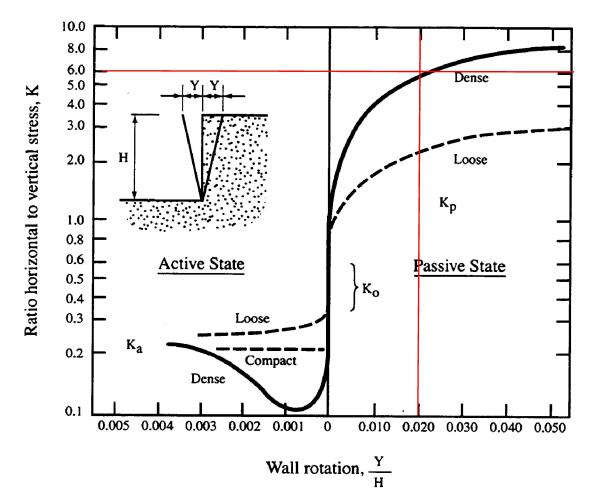
$$\sigma_p' = \gamma z K_p \tag{7.65}$$

and the passive force is

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

where

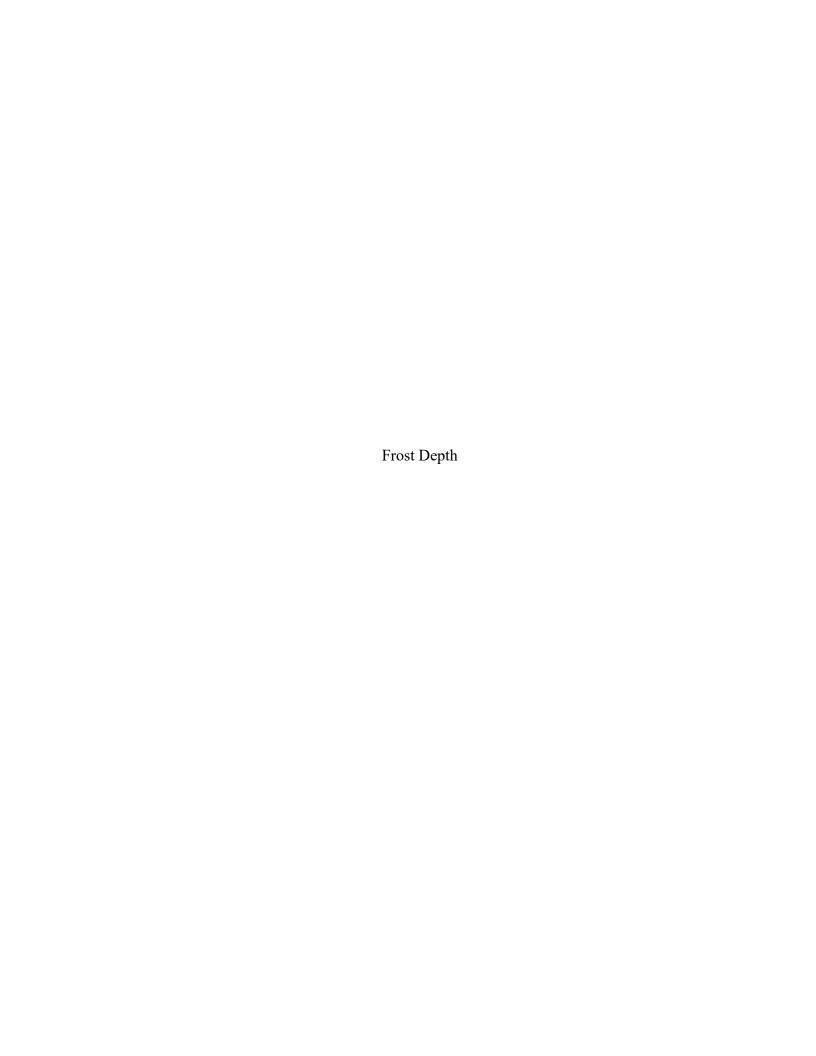
$$K_p = \cos \alpha \frac{\cos \alpha + \sqrt{\cos^2 \alpha - \cos^2 \phi'}}{\cos \alpha - \sqrt{\cos^2 \alpha - \cos^2 \phi'}}$$
 (7.67)



Magnitude of Wall Rotation to Reach Failure

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

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



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

From Design Freezing Index Map: Levant, Maine

DFI = 1825 degree-days.

Fine-Grained Fill w=16% (BB-LBS-101 1D)

Coarse-Grained Fill w=10% (BB-LBS-203 3D)

Fine-Grained Fill

For DFI = 1800, Fine-Grained Soil, w=10%

d=Depth of Frost Penetration

$$d_1 := 64.0in$$
 DFI₁ := 1800

For DFI = 1900, Fine-Grained Soil, w=10%

$$d_2 := 65.8in$$
 DFI₂ := 1900

Interpolate for DFI = 1825, Fine-Grained Soil, w=10%

$$DFI_3 := 1825$$

$$d_{f10} := d_1 + (DFI_3 - DFI_1) \cdot \frac{(d_2 - d_1)}{(DFI_2 - DFI_1)}$$

$$d_{f10} = 64.5 \cdot in$$
 $d_{f10} = 5.4 \cdot ft$

For DFI = 1800, Fine-Grained Soil, w=20%

$$d_1 := 55.1 \text{in}$$
 DFI₁ := 1800

For DFI = 1900, Fine-Grained Soil, w=20%

$$d_2 := 56.7$$
in DFI₂ := 1900

Interpolate for DFI = 1825, Fine-Grained Soil, w=20%

$$DFI_3 := 1825$$

$$d_{f20} := d_1 + (DFI_3 - DFI_1) \cdot \frac{(d_2 - d_1)}{(DFI_2 - DFI_1)}$$

$$d_{f20} = 55.5 \cdot in$$
 $d_{f20} = 4.6 \cdot ft$

Interpolate for DFI = 1825, Fine-Grained Soil, w=16%

$$d_{f16} := d_{f10} + (0.16 - 0.10) \cdot \frac{\left(d_{f20} - d_{f10}\right)}{(0.20 - 0.10)}$$

$$d_{f16} = 59.1 \cdot in$$
 $d_{f16} = 4.9 \cdot ft$ for Fine-Grained Fill

Coarse-Grained Fill

For DFI = 1800, Coarse-Grained Soil, w=10%

$$d_1 := 90.1 in$$
 DFI₁ := 1800

For DFI = 1900, Coarse-Grained Soil, w=10%

$$d_2 := 92.6 in$$
 DFI₂ := 1900

Interpolate for DFI = 1825, Coarse-Grained Soil, w=10%

$$DFI_3 := 1825$$

$$d_{c10} \coloneqq d_1 + \left(\mathrm{DFI}_3 - \mathrm{DFI}_1 \right) \cdot \frac{\left(d_2 - d_1 \right)}{\left(\mathrm{DFI}_2 - \mathrm{DFI}_1 \right)}$$

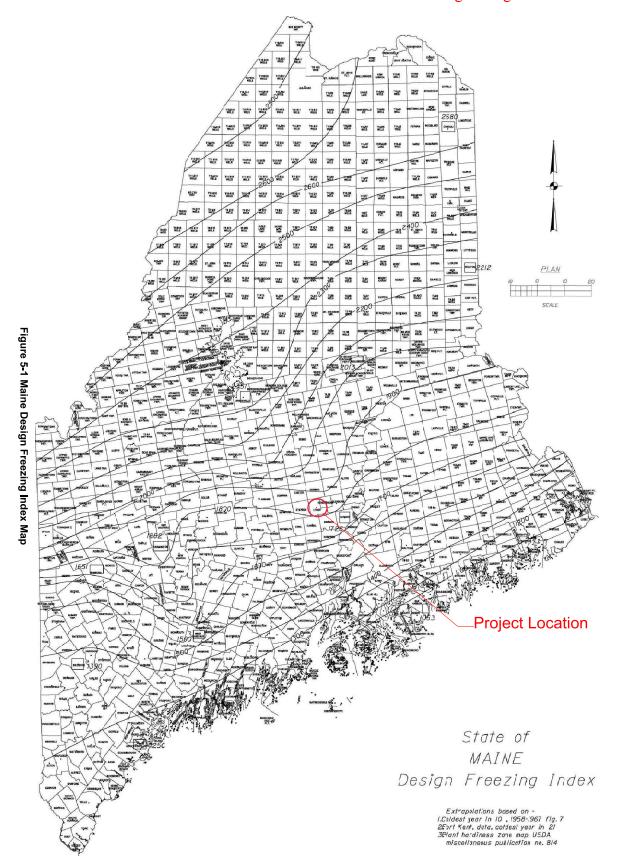
$$d_{c10} = 90.7 \cdot in$$

$$d_{c10} = 7.6 \cdot ft$$

for Coarse-Grained Fill

Recommend any foundation bearing on soils be embedded 7.6 feet for frost protection.

MaineDOT Bridge Design Guide



5.2 General

MaineDOT Bridge Design Guide

5.2.1 Frost

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

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

Table 5-1 Depth of Frost Penetration

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

March 2014 5-3



BB-LBS-101/101A				
Depth	N ₆₀	di	di/N	
5	45	9	0.20	
10	23	4	0.17	
15	38	7	0.18	
20	57	5	0.09	
25	80	5	0.06	
30	47	5	0.11	
35	48	5	0.10	
40	41	5	0.12	
45	100	5	0.05	
50	88	5	0.06	
55	100	5	0.05	
60	100	40	0.40	
SUM		100	1.60	

BB-LBS-102					
Depth	N ₆₀	di	di/N		
5	27	10	0.37		
11	11	5	0.45		
15	29	1	0.03		
20	17	8	0.47		
25	30	8	0.27		
35	26	8	0.31		
40	29	5	0.17		
45	23	5	0.22		
50	100	50	0.50		
SUM		100	2.79		

di/di/N 35.79

di/di/N 62.59

SUM Nav. 49.19

15 < Nav. < 50 bpf

Conclusion: Site Class D

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

Abutment No. 1 and 2 Seismic Parameters

Conterminous 48 States 2007 AASHTO Bridge Design Guidelines AASHTO Spectrum for 7% PE in 75 years

Latitude = 44.857917 Longitude = -068.954583

Site Class B

Data are based on a 0.05 deg grid spacing.

Period Sa (sec) (g) 0.0 0.070 PGA - Site Class B 0.2 0.150 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 = 44.857917 Longitude = -068.954583

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

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

Data are based on a 0.05 deg grid spacing.

Period Sa (sec) (g) 0.0 0.111 As - Site Class D 0.2 0.239 SDs - Site Class D 1.0 0.108 SD1 - Site Class D

