MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

FISH BRIDGE
GARLAND ROAD OVER PATTEE POND BROOK
WINSLOW, MAINE

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1.0 INTRODUCTION

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

The existing Fish Bridge was constructed in 1921 and is a 20-foot span concrete frame bridge/culvert with concrete foundation pads with heels for the wingwalls. The foundation footings may bear directly on soil (there are no historical bridge plans) and portions of the footings are exposed due to scour and erosion. According to the 2021 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the structure has exposed rebar, large spalls, and heavy scaling and is rated a 4. The frame culvert has a FHWA Sufficiency Rating of 70.0 and is classified as Structurally Deficient.

The proposed replacement structure consists of a 65-foot, single-span concrete NEXT beam bridge founded on integral abutments on steel H-piles driven to bedrock. 2.0H:1V (horizontal:vertical) riprap slopes will be constructed in front of the new integral abutments.

The new bridge will be located on a similar horizontal alignment as the existing bridge, with an increased span length. The new bridge will have a raise in grade of less than 0.5 foot.

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

2.0 GEOLOGIC SETTING

Fish Bridge carries Garland Road over Pattee Pond Brook as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of the Fairfield Quadrangle, Open-File No. 15-12 (2015), indicates the surficial soils in the vicinity of the bridge project consist of the Presumpscot Formation with glacial till mapped nearby. The Presumpscot Formation consists of glaciomarine silts, clays, and sands, deposited on the late-glacial sea floor and commonly overlies glacial till. Glacial till typically consists of very compact sand, silt, and gravel.

The MGS Geological Map and Structural Sections of the Waterville-Vassalboro Area (1961) maps the bedrock at the project site as thinly bedded Pelite and Quartzite of the Waterville Formation. Greywacke and thin beds of Phyllite of the Vassalboro Formation are mapped nearby.

3.0 SUBSURFACE INVESTIGATION

Five test borings were drilled to explore subsurface conditions at the site in September 2020. Boring BB-WPPB-101 was drilled behind the southwest corner of the existing structure. Borings BB-WPPB-102, -102A, B-102B and BB-WPPB-103 were drilled behind the northeast corner of the existing structure. Borings BB-WPPB-102, -102A, and -102B refused on cobbles or boulders at approximately 6 to 8 feet below the ground surface (bgs). The remaining two borings were advanced to bedrock and terminated with 10-foot bedrock cores.

The boring locations are shown on Sheet 2 – Boring Location Plan. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 4– Boring Logs.

Borings were performed by using solid stem auger, cased wash boring and rock coring techniques. Soil samples were typically obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The drill rig used in the subsurface investigation was equipped with an automatic hammer to drive the split spoon. The hammer was calibrated per ASTM D 4633 "Standard Test Method for Energy Measurement for Dynamic Penetrometers" in June 2020. All N-values discussed in this report are corrected N-values computed by applying an average energy transfer of 0.89. The hammer efficiency factor (0.89) and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring logs.

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

4.0 LABORATORY TESTING

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

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

5.0 Subsurface Conditions

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

5.1 Fill

Fill materials were encountered below approximately 8-inches of asphalt in the borings. The thickness of the fill unit encountered was approximately 10 to 14 feet at the boring locations and generally consisted of granular soils and reworked, native clays and silts, described as:

- Olive-brown CLAY, some silt, trace fine sand;
- Olive, Clayey SILT, trace fine sand;
- Brown SAND, little gravel;
- Brown, Gravelly SAND, little silt, trace organics.

Cobbles were encountered in the fill layer in borings BB-WPPB-101 and BB-WPPB-103. Three borings (BB-WPPB-102, -102A and -102B) refused on boulders or cobbles at depths of 6 to 8 ft bgs. Wood was encountered in BB-WPPB-103 at approximately 13 feet bgs.

Corrected SPT N-values in the fill ranged from 13 to 18 blows per foot (bpf) indicating the fill is medium dense in consistency. Two grain size analyses performed on samples resulted in A-1-a and A-6 material classifications according to the AASHTO Soil Classification System and SM and CL classifications according to the Unified Soil Classification System (USCS). The natural water content of the samples tested ranged from approximately 20 to 25 percent.

5.2 Glaciomarine and Marine Sand Deposits

Glaciomarine and Marine Sand Deposits were encountered beneath the Fill layer. The encountered thickness was approximately 15 to 25 feet at the boring locations. The Deposits encountered consisted of:

- Grey, Silty CLAY;
- Grey, very soft CLAY, little silt, trace fine sand;
- Grey, very soft, Clayey SILT, trace fine SAND, trace gravel;
- Grey, Silty SAND, trace clay, trace gravel; and
- Grey and dark grey, Clayey SILT, trace fine to medium sand.

In-situ vane shear tests were conducted with Geonor rectangular vanes in the Glaciomarine Deposit. A 55 x 110 mm vane was used. Twelve (12) successful vane shear tests conducted within the glaciomarine deposit showed measured undisturbed undrained shear strengths ranging from approximately 491 psf to 1295 psf, indicating that the deposit is soft to stiff in consistency. The remolded shear strengths at the test intervals ranged from approximately 134 to 277 psf. Based on the ratio of peak to remolded shear strength at all test intervals, the deposit has a sensitivity ranging from 2.4 to 5.3 and is classified as moderately sensitive to sensitive.

Undisturbed vane shear test results within the Glaciomarine Deposit indicate the deposit is soft to medium stiff, with one stiff reading in the clay “crust”. Five grain size analyses conducted on samples of the deposit indicated the material is classified as A-4, A-6, or A-7-6 under the AASHTO Soil Classification System and CL under the USCS.

Atterberg limits tests were conducted on six samples of the Glaciomarine Deposit and are summarized below:

Boring No. and Sample No.	Soil Description	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-WPPB-101, 1D	CLAY	25	38	22	16	0.19
BB-WPPB-101, 3D	CLAY	42	50	24	26	0.71
BB-WPPB-101, 4D	Clayey SILT	36	33	23	10	1.27
BB-WPPB-101, 3U	Clayey SILT	36	40	24	16	0.72
BB-WPPB-103, 3D	Clayey SILT	38	36	23	13	1.13
BB-WPPB-103, 1U	Clayey SILT	40	38	24	14	1.14

The plasticity indices of the samples indicate that the clay and silty clays have medium to high plasticity (Burmister, 1949). The natural water contents of the tested samples ranged from approximately 25 to 42 percent and liquid limits ranged from 33 to 50. The resulting liquidity indices are generally close to, or in excess of, 1.0, and the natural water contents generally exceed the liquid limits. Interpretation of these results indicates that most of the deposit has the potential to convert into a viscous fluid with the slightest disturbance. Soils with liquidity indices in excess of 1 have a high liquefaction or “quick” potential.

5.3 Bedrock

Bedrock was encountered and cored in borings BB-WPPB-101 and BB-WPPB-103. The table below summarizes the depth to bedrock, corresponding top of bedrock elevations and RQD's.

Boring	Station	Offset (feet)	Approximate Depth to Bedrock (feet)	Approximate Elevation of Bedrock Surface (feet)	RQD (%) (R1, R2)
BB-WPPB-101	51+21.3	8.5 Lt	39.8	24.1	16, 22
BB-WPPB-103	51+82.5	8.8 Rt	33.0	29.0	23, 52

The bedrock of the site consisted of black and grey banded, very fine-grained, hard, fresh to slightly weathered, PHYLLITE to METASILTSTONE with calcite veins, closely spaced breaks along steep foliation/bedding, with a second, close, subhorizontal joint set. The RQD of the bedrock cores ranged from 16 to 52 percent, corresponding to a rock quality of very poor to fair.

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

5.4 Groundwater

Groundwater was measured at 25 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

Single-span precast voided slab or concrete NEXT F bridges, both supported on pile-supported integral abutments, were considered as bridge replacement alternatives, as well as a do nothing alternative. A full replacement option was selected due to the age and condition of the bridge. The NEXT F beams on pile-supported integral abutments option was chosen due to cost, availability, and ease of construction.

7.0 GEOTECHNICAL DESIGN CONSIDERATIONS AND RECOMMENDATIONS

The following sections provide geotechnical design considerations and recommendations for H-pile supported integral abutments which is the proposed substructure type for the Fish Bridge replacement project.

7.1 Integral Abutment H-Piles

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

Piles may be HP 14x89 or 14x117 depending on the factored design axial loads and behavior under lateral loading. 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	54.4	24.1	33
Abutment No. 2	53.0	29.0	26

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

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

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

The pile groups should be designed to resist all lateral earth loads, vehicular loads, dead and live loads, and lateral forces transferred through the pile caps.

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

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

7.1.1 Axial Pile Resistance – Strength Limit State

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

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

Drivability Analyses. Drivability analyses were performed to determine the pile resistance that might be achieved considering available diesel hammers. LRFD 10.7.8 limits driving stresses to $0.90f_y$, which for 50 ksi steel piles is 45 ksi. The drivability resistances were calculated using the resistance factor, φ_{dyn}, of 0.65, for a single pile in axial compression when a dynamic test is performed as specified in LRFD Table 10.5.5.2.3-1.

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

<table>
<tr><th colspan="7">Strength Limit State
Factored Axial Pile Resistance</th></tr>
<tr><th>Pile Section</th><th>Structural Resistance[1]
ϕ_c=0.50
(kips)</th><th>Controlling Geotechnical Resistance[2]
ϕ_c=0.50
(kips)</th><th colspan="2">Drivability Resistance[3]
φ_{dyn} = 0.65
(kips)</th><th>Governing Axial Pile Resistance
(kips)</th></tr>
<tr><td>HP 14 x 89</td><td>652</td><td>652</td><td>403[4]</td><td>390[5]</td><td>403[4]</td></tr>
<tr><td>HP 14 x 117</td><td>860</td><td>860</td><td>468[4]</td><td>520[5]</td><td>468[4]</td></tr>
</table>

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.

[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 limiting driving criteria of 15 bpi and a maximum driving stress of 45 ksi. These theoretical pile resistances may not be achievable if piles walk out of position before reaching the specified driving criteria. The drivability resistances were calculated using the resistance factor, φ_{dyn}, of 0.65, for a single pile in axial compression when a dynamic test is performed as specified in LRFD Table 10.5.5.2.3-1.

[4] Drivability resistance based on a Delmag D19-42 at Full Fuel Setting.
[5] Drivability resistance based on a Delmag D25-52 at Fuel Setting 3.

7.1.2 Axial Pile Resistance – Service and Extreme Limit State

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

Extreme limit state design checks for the driven H-piles shall include pile axial compressive resistance, overall global stability of the pile group, pile failure by uplift in tension, and structural failure. The extreme event load combinations are those related to seismic forces and vehicle collision. Resistance factors for extreme limit states, per LRFD Article 10.5.5.3, shall be taken as $\phi = 1.0$ with the exception of uplift of piles, for which the resistance factor, φ_{up}, shall be 0.80 or less per LRFD Article 10.5.5.3.2.

The calculated factored axial structural, geotechnical and drivability resistances of two (2) H-pile sections for the service and extreme limit states are summarized below.

Service and Extreme Limit State Factored Axial Pile Resistance					
Pile Section	Structural Resistance[1] $\phi = 1.0$ (kips)	Controlling Geotechnical Resistance[2] $\phi = 1.0$ (kips)	Drivability Resistance[3] $\phi = 1.0$ (kips)		Governing Axial Pile Resistance (kips)
HP 14 x 89	1,305	1,305	620[4]	600[5]	620[4]
HP 14 x 117	1,720	1,720	720[4]	800[5]	720[4]

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

[2] 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 limiting driving criteria of 15 bpi and a maximum driving stress of 45 ksi. These theoretical pile resistances may not be achievable if piles walk out of position before reaching the specified driving criteria.

[13] Drivability resistance based on a Delmag D19-42 at Full Fuel Setting.
[14] Drivability resistance based on a Delmag D25-52 at Fuel Setting 3.

LRFD Article 10.7.3.2.3 states that the nominal axial compressive resistance of piles driven to hard rock is typically controlled by the structural resistance. However, the estimated factored axial pile resistances from the drivability analyses for the H-pile sections are less than the controlling factored axial geotechnical resistance and the structural resistance calculated for a braced pile segment. Therefore, drivability controls and the recommended governing resistances for pile design are the resistances provided in the rightmost column "Governing Axial Pile Resistance (kips)" in the table above.

The maximum applied factored axial pile load for the service and extreme limit states shall not exceed the governing factored axial pile resistance shown in the table above.

7.1.3 Lateral Pile Resistance/Behavior

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

A series of lateral pile resistance analyses should be performed to evaluate pile behavior at the abutments using LPile, or similar, software. Lateral pile analyses should be utilized to evaluate the associated pile stresses, bending moments, and fixity due to factored pile head loads and displacements. The models developed should emulate appropriate structural parameters and pile-head boundary conditions for the pile section(s) being analyzed.

7.1.4 Driven Pile Quality Control

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

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

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

7.2 Integral Abutment and Wingwall Design

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

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

- Internal Friction Angle (ϕ) = 32°
- Total Unit Weight (γ) = 125 pcf
- Soil-Concrete Interface Friction Angle (δ) = 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 MassDOT LRFD Bridge Design Manual Figure 3.10.8-1, a lateral earth pressure coefficient of 3.93 is recommended, assuming a ratio of thermal expansion to abutment height (δ/H) of 0.005 and a level backfill. In general, when the calculated ratio of lateral movement to wall height is less than or greater than 0.01, a passive earth pressure coefficient can be estimated using MassDOT Figure 3.10.8-1. This figure is reproduced in Appendix D – Calculations. A load factor for passive earth pressure is not specified in LRFD. For purposes of the integral abutment backwall reinforcing steel design, use a maximum load factor (γ_{EH}) of 1.50 to calculate factored passive earth pressures.

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

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

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

7.3 Abutment Sections

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

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

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

7.4 Settlement and Embankment Construction

The existing bridge approach embankments overlay soft to medium stiff silty clay. Raises in grade of 0.1 and 2.0 inches (not including 4.0 inches of asphalt) are proposed at Abutments No. 1 and No. 2, respectively. Estimated post-constructions settlements will be provided in a subsequent geotechnical memorandum. It is anticipated that these settlements will be minimal and will be mitigated by the use of an approach slab.

The proposed bridge span will be 45 feet greater than the existing frame culvert. The implication of the proposed widened structure is a net unloading of vertical overburden pressures in front of the proposed abutments. Earth fill approach embankments reconstructed 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. Disturbance of the sensitive Marine Clay subgrade at the toe of the embankment slopes should be avoided during the reconstruction process.

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, Winslow has a design freezing index (DFI) of approximately 1600 F-degree days. The anticipated coarse-grained fill soil was assigned a water content of 20%, and the anticipated fine-grained fill soil was assigned a water content of 25%. These components correlate to a frost depth of 4.1 to 5.9 feet. It is recommended that any foundation bearing on soils be embedded 5.9 feet for frost protection.

Pile-supported integral abutments shall be embedded a minimum of 5.9 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)	0.075g
Acceleration Coefficient (A_S)	0.187g
S_{DS} (Period = 0.2 sec)	0.39g
S_{D1} (Period = 1.0 sec)	0.16g
Site Class	E
Seismic Zone	2

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

The new abutments will be constructed approximately 20 feet behind the existing structure and will require pile driving. The contractor shall be responsible for excavating the existing substructures in their entirety.

The subgrade at the proposed abutments is anticipated to consist of soft, sensitive clay and sandy fill soils. Use of a crushed stone mat or a flowable fill (mud) mat to provide a stable subgrade will be required for pile driving and construction of the pile cap. The subgrade soils are expected to be sensitive, so care should be taken to limit disturbance to the subgrade surface. The subgrade should be protected from unnecessary construction traffic and disturbance from heavy equipment.

The underlying sensitive Marine Clay deposit is subject stain-softening (liquefaction) when disturbed. Therefore, ground vibration by heavy equipment should be avoided.

Any loose or soft soil, and organics, encountered at the abutment subgrades shall be removed and replaced with Granular Borrow – Material for Underwater Backfill and the exposed subgrade then compacted.

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

9.0 CLOSURE

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

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

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

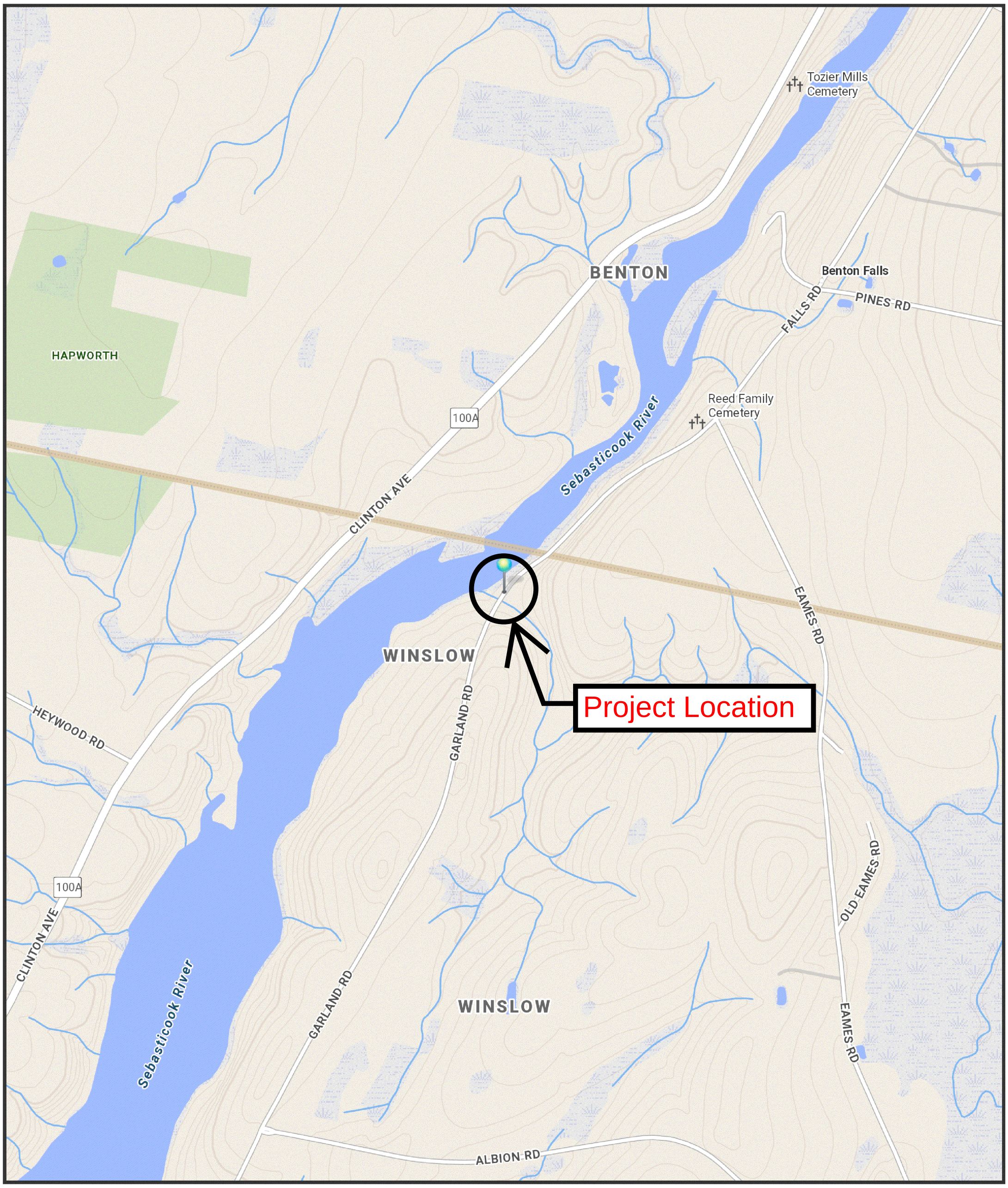
Sheets

Date:8/15/2023

Division: GEOTECH

Username: terry.white

WINSLOW, MAINE

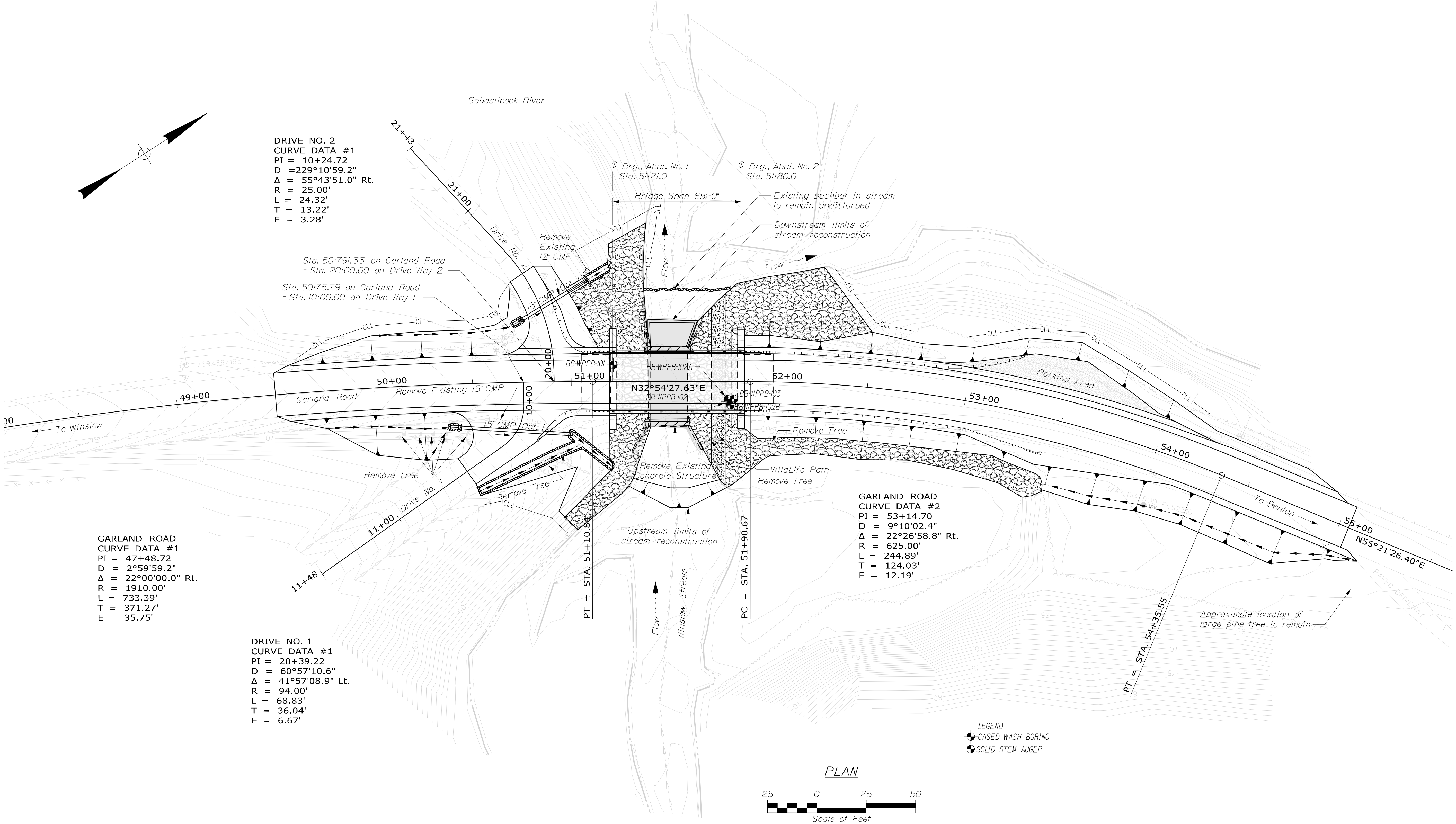


0.25 Miles

1 inch = 0.28 miles

Date: 7/14/2023
Time: 9:28:48 AM

SHEET NUMBER	FISH BRIDGE	STATE OF MAINE DEPARTMENT OF TRANSPORTATION
1	PATTEE POND BROOK	2226800
OF 4	WINSLOW KENNEBEC COUNTY	WIN
	LOCATION MAP	BRIDGE NO. 0509 22268.00 BRIDGE PLANS



DRIVE NO. 2
CURVE DATA #1
PI = 10+24.72
D = 229°10'59.2"
Δ = 55°43'51.0" Rt.
R = 25.00'
L = 24.32'
T = 13.22'
E = 3.28'

GARLAND ROAD
CURVE DATA #1
PI = 47+48.72
D = 2°59'59.2"
Δ = 22°00'00.0" Rt.
R = 1910.00'
L = 733.39'
T = 371.27'
E = 35.75'

DRIVE NO. 1
CURVE DATA #1
PI = 20+39.22
D = 60°57'10.6"
Δ = 41°57'08.9" Lt.
R = 94.00'
L = 68.83'
T = 36.04'
E = 6.67'

GARLAND ROAD
CURVE DATA #2
PI = 53+14.70
D = 9°10'02.4"
Δ = 22°26'58.8" Rt.
R = 625.00'
L = 244.89'
T = 124.03'
E = 12.19'

LEGEND
CASED WASH BORING
SOLID STEM AUGER

PLAN

25 0 25 50
Scale of Feet

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2226800

BRIDGE NO. 0509

WIN 22268.00

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	M. KERSBERGEN	
DESIGN-DETAILED	R. NADUS	MRP	FEB 2020
CHECKED-REVIEWED		
DESIGN2-DETAILED2	L. KRUSINSKI	T. WHITE	OCT 2023
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

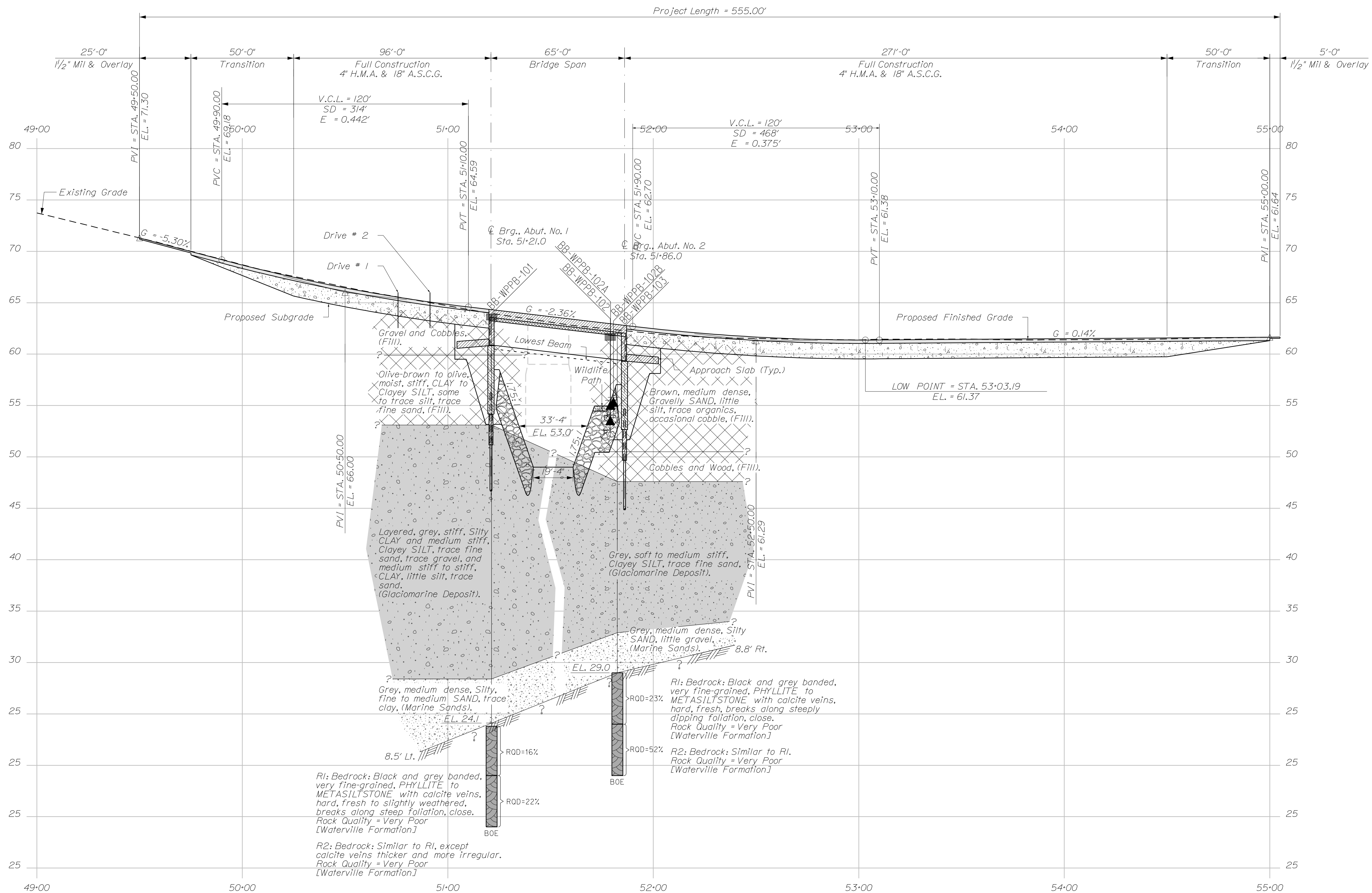
DATE

FISH BRIDGE
PATTEE POND BROOK
WINSLOW KENNEBEC COUNTY

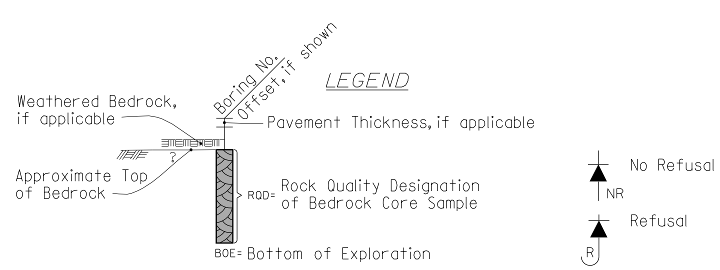
BORING LOCATION PLAN

SHEET NUMBER
2
OF 4

Filename: ...\00\GEOTECH\MSTA\006_BLP1.dgn Division: GEOTECH Username: Laura.Krusinski Date:10/26/2023



LEGEND



PROFILE

HORIZ 25 0 25 50
VERT 5 0 5 10
SCALE

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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2226800

BRIDGE PLANS
WIN 22268.00
BRIDGE NO. 0509

PROJ. MANAGER	M. KERSBERGEN	BY	DATE
DESIGN-DETAILED	R. NADUS	MRP	FEB 2020
CHECKED-REVIEWED	J. MANAHAN	T. WHITE	DEC 2020
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE
P.E. NUMBER
DATE

FISH BRIDGE
PATTEE POND BROOK
WINSLOW KENNEBEC COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
3
OF 4

Filename: ...\00\GEOTECH\MSTA\007_ISP1.dgn Division: GEOTECH Username: Laura.Krusinski Date:10/23/2023

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

Project: Fish Bridge #0509 carries Garland Road over Pattee Pond Brook
Location: Winslow, Maine

Boring No.: BB-WPPB-101
WIN: 22268.00

Driller:	MaineDOT	Elevation (ft.)	63.9	Auger ID/OD:	5" Solid Stem
Operator:	Daggett/Wilder	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	J. Manahan	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	9/10/2020 & 9/14/2020	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	51+21.3, 8.5 ft Lt.	Casing ID/OD:	HW-4" & NW-3"	Water Level*:	25.0 ft bgs.
Hammer Efficiency Factor: 0.89		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	63.3	7" HMA. —0.6	
									Brown, dry to moist, Gravel cuttings, (Fill). Cobble from 1.0-1.3 ft bgs. Cobbles from 1.5-2.2 ft bgs.	
								59.9	—4.0	
5	1D	24/24	5.00 - 7.00	6/3/6/8	9	13			Olive brown, moist, stiff, CLAY, some silt , trace fine sand. (Fill: Reworked Native Soils)	G#336991 A-6, CL WC=25.0% LL=38 PL=22 PI=16
10	2D MV	24/24	10.00 - 12.00 10.63 - 10.63	3/4/5/4 Would Not Push	9	13	HP	53.1	(2D/A 10.0-10.8 ft bgs) Olive, moist, stiff, Clayey SILT, trace fine sand. HP = Hydraulic Push Failed 55x110 mm vane attempt. —10.8 (2D/B 10.8-12.0 ft bgs) Grey, moist, stiff, Silty CLAY. (Glaciomarine Deposit) 800-900# down pressure	
	1U	24/24	12.00 - 14.00	Hydraulic Push						
	V1		14.63 - 15.00	Su=1295/246 psf			38		55x110 mm vane raw torque readings: V1: 29.0/5.5 ft-lbs	
15	MV		15.63 - 15.63	Would Not Push			OPEN HOLE		Failed 55x110 mm vane attempt.	
	3D V2	24/24	16.00 - 18.00 16.63 - 17.00	WOH/WOH/WOH/2 Su=1094/223 psf	---				(3D 16.00-18.00 ft bgs) Grey, moist, medium stiff to stiff, CLAY, little silt, trace fine sand. (Glaciomarine Deposit) 55x110 mm vane raw torque readings: V2: 24.5/5.0 ft-lbs V3: 21.5/6.2 ft-lbs	G#336992 A-7-6, CL WC=42.4% LL=50 PL=24 PI=26
	V3		17.63 - 18.00	Su=692/277 psf						
20	2U	24/24	20.00 - 22.00	Hydraulic Push					Similar, expect medium stiff. (Glaciomarine Deposit)	
	V4		22.63 - 23.00	Su=603/156 psf					55x110 mm vane raw torque readings: V4: 13.5/3.5 ft-lbs	
	V5		23.63 - 24.00	Su=625/134 psf					V5: 14.0/3.0 ft-lbs	
25	4D V6	24/24	25.00 - 27.00 25.63 - 26.00	WOR/WOR/WOR/WOH Su=580/179 psf	---				(4D) Grey, moist, medium stiff, Clayey SILT trace fine sand, trace gravel. (Glaciomarine Deposit) 55x110 mm vane raw torque readings: V6: 13.0/4.0 ft-lbs V7: 13.5/4.0 ft-lbs	G#336993 A-4, CL WC=35.7% LL=33 PL=23 PI=10
	V7		26.63 - 27.00	Su=603/179 psf						
30	3U	24/24	30.00 - 32.00	WOR/WOR/WOR/WOR	---				Dark grey, moist, soft, Clayey SILT, trace fine sand. (Glaciomarine Deposit).	G,C#336994 A-7-6, CL WC=35.5% LL=40 PL=24 PI=16
	V8		32.63 - 33.00	Su=714/134 psf					Similar, exept medium stiff. 55x110 mm vane raw torque readings: V8: 16.0/3.0 ft-lbs V9: 16.0/4.0 ft-lbs	
	V9		33.63 - 34.00	Su=714/179 psf						
35	5D MV	24/20	35.00 - 37.00 35.63 - 35.63	2/7/12/20 Would Not Push	19	28		28.4	—35.5 (5D) Grey, moist, medium dense, Silty, fine to medium SAND, trace clay, (Marine Sand). Weathered Rock 35.5-39.8 ft bgs Failed 55x110 mm vane attempt.	
							55			
							66			
							76			
							a284		a284 blows for 0.8 ft.	
40	6D R1	1/0 57.6/ 57.6	40.00 - 40.08 40.10 - 44.90	20(1") RQD = 16%	---		NQ-2	24.1 23.8	—39.8 Top of Bedrock at Elev. 24.1 ft. —40.1 R1: Bedrock: Black and grey banded, very fine-grained, PHYLLITE to METASILTSTONE, with calcite veins, hard, slightly weathered to fresh, breaks along steep foliation/bedding, very closedly spaced, planar and tight. Rock Quality = Very Poor [Waterville Formation] R1: Core Times (min:sec) 40.1-41.1 ft (2:24) 41.1-42.1 ft (3:02) 42.1-43.1 ft (1:45) 43.1-44.1 ft (2:32) 44.1-44.9 ft (2:49) Core Blocked 100% Recovery	
45	R2	60/51	44.90 - 49.90	RQD = 22%				19.0	44.9 R2: Bedrock: Similar to R1 except more calcite veins and additional low angle breaks at close spacing. Rock Quality = Very Poor R2: Core Times (min:sec) 44.9-45.9 ft (1:16) 45.9-46.9 ft (2:38) 46.9-47.9 ft (2:48) 47.9-48.9 ft (2:35) 48.9-49.9 ft (4:28) 85% Recovery	
50								14.0	—49.9 Bottom of Exploration at 49.9 feet below ground surface.	

Remarks:

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

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

Page 1 of 1 | Boring No.: BB-WPPB-101

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

Project: Fish Bridge #0509 carries Garland Road over Pattee Pond Brook
Location: Winslow, Maine

Boring No.: BB-WPPB-102
WIN: 22268.00

Driller:	MaineDOT	Elevation (ft.)	62.0	Auger ID/OD:	5" Dia.
Operator:	Daggett/Wilder	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	J. Manahan	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	9/15/2020; 07:30-08:30	Drilling Method:	Solid Stem Auger	Core Barrel:	N/A
Boring Location:	51+79.1, 10.0 ft Rt.	Casing ID/OD:	N/A	Water Level*:	None Observed
Hammer Efficiency Factor: 0.89		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	61.3	8" HMA. —0.7	
5	1D	24/1	5.00 - 7.00	5/6/4/21	10	15			Brown, damp, medium dense, SAND, little gravel. (Fill)	
								54.0	—8.0 Bottom of Exploration at 8.0 feet below ground surface. REFUSAL	

Remarks:

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

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

Page 1 of 1 | Boring No.: BB-WPPB-102

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

Project: Fish Bridge #0509 carries Garland Road over Pattee Pond Brook
Location: Winslow, Maine

Boring No.: BB-WPPB-102A
WIN: 22268.00

Drilling Contractor:	MaineDOT	Elevation (ft.)	62.0	Auger ID/OD:	5" Dia.
Operator:	Daggett/Wilder	Datum:	NAVD88	Sampler:	N/A
Logged By:	J. Manahan	Rig Type:	CME 45C	Hammer Wt./Fall:	N/A
Date Start/Finish:	9/15/2020; 08:30-08:50	Drilling Method:	Solid Stem Auger	Core Barrel:	N/A
Boring Location:	51+79.2, 8.8 ft Rt.	Casing ID/OD:	N/A	Water Level*:	None Observed

Definitions: D = Split Spoon Sample; S = Sample off Auger Flights; B = Bucket Sample off Auger Flights; MD = Unsuccessful Split Spoon Sample Attempt; U = Thin Wall Tube Sample; MV = Unsuccessful Field Vane Shear Test Attempt; V = Field Vane Shear Test, PP = Pocket Penetrometer; MU = Unsuccessful Thin Wall Tube Sample Attempt; R = Rock Core Sample; SSA = Solid Stem Auger; HSA = Hollow Stem Auger; RC = Roller Cone; WOH = Weight of 140lb. Hammer; WOR/C = Weight of Rods or Casing; WO1P = Weight of 1 Person; S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf); $S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf); q_p = Unconfined Compressive Strength (ksf); N-value = Raw Field SPT N-value; T_v = Pocket Torvane Shear Strength (psf); WC = Water Content, percent; # = Similar or Equal too; LL = Liquid Limit; PL = Plastic Limit; PI = Plasticity Index; G = Grain Size Analysis; C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0						SSA		Brown, Sandy cuttings with gravel. (Fill)	
							55.50	—6.50 Bottom of Exploration at 6.5 feet below ground surface. REFUSAL	

Remarks:

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

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

Page 1 of 1 | Boring No.: BB-WPPB-102A

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

Project: Fish Bridge #0509 carries Garland Road over Pattee Pond Brook
Location: Winslow, Maine

Boring No.: BB-WPPB-102B
WIN: 22268.00

Drilling Contractor:	MaineDOT	Elevation (ft.)	61.8	Auger ID/OD:	5" Dia.
Operator:	Daggett/Wilder	Datum:	NAVD88	Sampler:	N/A
Logged By:	J. Manahan	Rig Type:	CME 45C	Hammer Wt./Fall:	N/A
Date Start/Finish:	9/15/2020; 08:50-09:00	Drilling Method:	Solid Stem Auger	Core Barrel:	N/A
Boring Location:	51+80.6, 12.0 ft Rt.	Casing ID/OD:	N/A	Water Level*:	None Observed

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0						SSA		Brown SAND and GRAVEL cuttings. (Fill)	
							55.80	—6.00 Bottom of Exploration at 6.0 feet below ground surface. REFUSAL	

Remarks:

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

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

Page 1 of 1 | Boring No.: BB-WPPB-102B

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

Project: Fish Bridge #0509 carries Garland Road over Pattee Pond Brook
Location: Winslow, Maine

Boring No.: BB-WPPB-103
WIN: 22268.00

Driller:	MaineDOT	Elevation (ft.)	62.0	Auger ID/OD:	5" Solid Stem
Operator:	Daggett/Wilder	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	J. Manahan	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	9/15/2020 & 9/16/2020	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	51+82.5, 8.8 ft RT.	Casing ID/OD:	HW-4" & NW-3"	Water Level*:	None Observed
Hammer Efficiency Factor: 0.89		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA		Brown Sand and Gravel cuttings, (Fill).	
10	1D	18/12	10.00 - 11.50	6/6/6	12	18	80		Brown, damp, medium dense, Gravelly SAND, little silt, trace organics. (Fill)	G#336995 A-1-a, SM WC=19.9%
							32 OPEN HOLE	50.5	—11.5 Cobbles at 11.5-13.2 ft bgs.	
									Wood in cuttings at 13.2 ft bgs. Cobble from 13.7-14.4 ft bgs.	
								47.6	—14.4	
15	2D	24/24	15.00 - 17.00	3/2/4/4	6	9			Olive grey, moist, medium stiff, Clayey SILT, trace fine sand, (Glaciomarine Deposit).	
									Cuttings become grey at 19.0 ft bgs.	
20	3D V1	24/24	20.00 - 22.00 20.63 - 21.00	WOR/WOR/WOH/WOH Su=491/201 psf	---				Grey, moist, soft to medium stiff, Clayey SILT, trace fine to medium sand, (Glaciomarine Deposit). 55x110 mm vane raw torque readings: V1: 11.0/4.5 ft-lbs V2: 12.0/3.0 ft-lbs	G#336996 A-6, CL WC=37.7% LL=36 PL=23 PI=13
	V2		21.63 - 22.00	Su=536/134 psf						
25	1U	24/24	25.00 - 27.00	WOR					Dark grey, moist, soft to medium stiff, Clayey SILT, trace fine sand, (Glaciomarine Deposit).	G,C#336997 A-6, CL WC=39.9% LL=38 PL=24 PI=14
	V3		27.63 - 28.00	Su=759/223 psf					55x110 mm vane raw torque readings: V3: 17.0/5.0 ft-lbs	
	MV		28.63 - 28.63	Would Not Push				32.9	Failed 55x110 mm vane attempt. —29.1	
30	4D	24/3	30.00 - 32.00	9/9/7/10	16	24	14		Grey, wet, medium dense, Silty SAND, little gravel, (Marine Sand).	
							43			
							100	29.0	—33.0	
	R1	60/60	33.00 - 38.00	RQD = 23%			NQ-2		Top of Bedrock at Elev. 29.0 ft. R1: Bedrock: Black and grey banded, very-grained, PHYLLITE to METASILTSTONE with calcite veins, hard, fresh, breaks along steep, very close, tight foliation/ bedding, low angle breaks are planar and close. Rock Quality = Very Poor [Waterville Formation] R1: Core Times (min:sec) 33.0-34.0 ft (1:53) 34.0-35.0 ft (1:54) 35.0-36.0 ft (2:13) 36.0-37.0 ft (2:46) 37.0-28.0 ft (3:50) 100% Recovery	
	R2	60/60	38.00 - 43.00	RQD = 52%				19.0	R2: Bedrock: Similar to R1 except upper core is more massive, and lower core is more fractured along finer bedding. Rock Quality = Fair R2: Core Times (min:sec) 38.0-39.0 ft (3:12) 39.0-40.0 ft (3:02) 40.0-41.0 ft (3:08) 41.0-42.0 ft (3:30) 42.0-43.0 ft (5:19) 100% Recovery —43.0 Bottom of Exploration at 43.0 feet below ground surface.	

Remarks:

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

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

Page 1 of 1 | Boring No.: BB-WPPB-103

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
2226800
WIN 22268.00
BRIDGE PLANS
BRIDGE NO. 0509

	BY	DATE
PROJ. MANAGER	M. KERSBERGEN	
DESIGN-DETAILED	R. NADUS	
CHECKED-REVIEWED	L. KRUSINSKI	
DESIGN2-DETAILED2	MRP	FEB 2020
DESIGN3-DETAILED3	T. WHITE	OCT 2023
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

FISH BRIDGE
PATTEE POND BROOK
WINSLOW KENNEBEC COUNTY

BORING LOGS

SHEET NUMBER 4 OF 4

Filename: ...\MSTA\008_BORING LOGS1.dgn Division: GEOTECH Username: Laura.Krusinski Date:10/24/2023

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

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

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

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

MODIFIED BURMISTER SYSTEM

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

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

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

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

Rock Quality Designation (RQD):

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

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

Rock Quality Based on RQD

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

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

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Fish Bridge #0509 carries Garland Road over Pattee Pond Brook
Location: Winslow, Maine

Boring No.: BB-WPPB-101

WIN: 22268.00

Driller:	MaineDOT	**Elevation (ft.)**	63.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Wilder	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	J. Manahan	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/10/2020 & 9/14/2020	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	51+21.3, 8.5 ft Lt.	**Casing ID/OD:**	HW-4" & NW-3"	**Water Level*:**	25.0 ft bgs.
Hammer Efficiency Factor:	0.89	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	63.3		7" HMA. ——0.6	
										Brown, dry to moist, Gravel cuttings, (Fill). Cobble from 1.0-1.3 ft bgs. Cobbles from 1.5-2.2 ft bgs.	
								59.9		— — — 4.0	
5	1D	24/24	5.00 - 7.00	6/3/6/8	9	13				Olive brown, moist, stiff, CLAY, some silt , trace fine sand. (Fill; Reworked Native Soils)	G#336991 A-6, CL WC=25.0% LL=38 PL=22 PI=16
10	2D	24/24	10.00 - 12.00	3/4/5/4	9	13	HP	53.1		(2D/A 10.0-10.8 ft bgs) Olive, moist, stiff, Clayey SILT, trace fine sand. HP = Hydraulic Push Failed 55x110 mm vane attempt.	
	~~MV~~		~~10.63 - 10.63~~	~~Would Not Push~~						——10.8	
	1U	24/24	12.00 - 14.00	Hydraulic Push						(2D/B 10.8-12.0 ft bgs) Grey, moist, stiff, Silty CLAY. (Glaciomarine Deposit) 800-900# down pressure	
	V1		14.63 - 15.00	Su=1295/246 psf			38			55x110 mm vane raw torque readings: V1: 29.0/5.5 ft-lbs	
15	MV		15.63 - 15.63	Would Not Push			~~OPEN HOLE~~			Failed 55x110 mm vane attempt.	
	3D	24/24	16.00 - 18.00	WOH/WOH/WOH/2	---					(3D 16.00-18.00 ft bgs) Grey, moist, medium stiff to stiff, CLAY, little silt, trace fine sand. (Glaciomarine Deposit) 55x110 mm vane raw torque readings: V2: 24.5/5.0 ft-lbs V3: 21.5/6.2 ft-lbs	G#336992 A-7-6, CL WC=42.4% LL=50 PL=24 PI=26
	~~V2~~		~~16.63 - 17.00~~	~~Su=1094/223 psf~~							
	V3		17.63 - 18.00	Su=692/277 psf							
20	2U	24/24	20.00 - 22.00	Hydraulic Push						Similar, expect medium stiff. (Glaciomarine Deposit)	
	V4		22.63 - 23.00	Su=603/156 psf						55x110 mm vane raw torque readings: V4: 13.5/3.5 ft-lbs	
	V5		23.63 - 24.00	Su=625/134 psf						V5: 14.0/3.0 ft-lbs	
25											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Fish Bridge #0509 carries Garland Road over Pattee Pond Brook
Location: Winslow, Maine

Boring No.: BB-WPPB-101
WIN: 22268.00

Driller:	MaineDOT	**Elevation (ft.)**	63.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Wilder	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	J. Manahan	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/10/2020 & 9/14/2020	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	51+21.3, 8.5 ft Lt.	**Casing ID/OD:**	HW-4" & NW-3"	**Water Level*:**	25.0 ft bgs.
Hammer Efficiency Factor:	0.89	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	4D	24/24	25.00 - 27.00	WOR/WOR/WOR/ WOH	---					(4D) Grey, moist, medium stiff, Clayey SILT trace fine sand, trace gravel. (Glaciomarine Deposit) 55x110 mm vane raw torque readings: V6: 13.0/4.0 ft-lbs V7: 13.5/4.0 ft-lbs	G#336993 A-4, CL WC=35.7% LL=33 PL=23 PI=10
	V6		25.63 - 26.00	Su=580/179 psf							
	V7		26.63 - 27.00	Su=603/179 psf							
30	3U	24/24	30.00 - 32.00	WOR/WOR/WOR/ WOR	---					Dark grey, moist, soft, Clayey SILT, trace fine sand. (Glaciomarine Deposit).	G,C#336994 A-7-6, CL WC=35.5% LL=40 PL=24 PI=16
	V8		32.63 - 33.00	Su=714/134 psf						Similar, exept medium stiff. 55x110 mm vane raw torque readings: V8: 16.0/3.0 ft-lbs V9: 16.0/4.0 ft-lbs	
	V9		33.63 - 34.00	Su=714/179 psf							
35	5D	24/20	35.00 - 37.00	2/7/12/20	19	28		28.4		35.5 (5D) Grey, moist, medium dense, Silty, fine to medium SAND, trace clay, (Marine Sand). Weathered Rock 35.5-39.8 ft bgs Failed 55x110 mm vane attempt.	
	MV		35.63 - 35.63	Would Not Push			55				
							66				
							76				
							a284			a284 blows for 0.8 ft.	
40	6D	1/0	40.00 - 40.08	20(1")	---		NQ-2	24.1		39.8 Top of Bedrock at Elev. 24.1 ft.	
	R1	57.6/57.6	40.10 - 44.90	RQD = 16%						R1: Bedrock: Black and grey banded, very fine-grained, PHYLLITE to METASILTSTONE, with calcite veins, hard, slightly weathered to fresh, breaks along steep foliation/bedding, very closedly spaced, planar and tight. Rock Quality = Very Poor [Waterville Formation] R1: Core Times (min:sec) 40.1-41.1 ft (2:24) 41.1-42.1 ft (3:02) 42.1-43.1 ft (1:45) 43.1-44.1 ft (2:32) 44.1-44.9 ft (2:49) Core Blocked 100% Recovery	
45	R2	60/51	44.90 - 49.90	RQD = 22%						R2: Bedrock: Similar to R1 except more calcite veins and additional low angle breaks at close spacing. Rock Quality = Very Poor R2: Core Times (min:sec) 44.9-45.9 ft (1:16) 45.9-46.9 ft (2:38) 46.9-47.9 ft (2:48)	
50											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Fish Bridge #0509 carries Garland Road over Pattee Pond Brook
Location: Winslow, Maine

Boring No.: BB-WPPB-101
WIN: 22268.00

Driller:	MaineDOT	**Elevation (ft.)**	63.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Wilder	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	J. Manahan	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/10/2020 & 9/14/2020	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	51+21.3, 8.5 ft Lt.	**Casing ID/OD:**	HW-4" & NW-3"	**Water Level*:**	25.0 ft bgs.
Hammer Efficiency Factor:	0.89	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50								14.0		47.9-48.9 ft (2:35) 48.9-49.9 ft (4:28) 85% Recovery	
										49.9	
										Bottom of Exploration at 49.9 feet below ground surface.	
55											
60											
65											
70											
75											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Fish Bridge #0509 carries Garland Road over Pattee Pond Brook
Location: Winslow, Maine

Boring No.: BB-WPPB-102
WIN: 22268.00

Driller:	MaineDOT	**Elevation (ft.)**	62.0	**Auger ID/OD:**	5" Dia.
Operator:	Daggett/Wilder	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	J. Manahan	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/15/2020; 07:30-08:30	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	N/A
Boring Location:	51+79.1, 10.0 ft Rt.	**Casing ID/OD:**	N/A	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.89	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
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C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks		Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	61.3		8" HMA.	0.7	
5	1D	24/1	5.00 - 7.00	5/6/4/21	10	15				Brown, damp, medium dense, SAND, little gravel. (Fill)		
								54.0		**Bottom of Exploration at 8.0 feet below ground surface.** REFUSAL	8.0	
10												
15												
20												
25												

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Boring No.: BB-WPPB-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Fish Bridge #0509 carries Garland Road over Pattee Pond Brook
Location: Winslow, Maine

Boring No.: BB-WPPB-102A
WIN: 22268.00

Driller:	MaineDOT	**Elevation (ft.)**	62.0	**Auger ID/OD:**	5" Dia.
Operator:	Daggett/Wilder	**Datum:**	NAVD88	**Sampler:**	N/A
Logged By:	J. Manahan	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	9/15/2020; 08:30-08:50	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	N/A
Boring Location:	51+79.2, 8.8 ft Rt.	**Casing ID/OD:**	N/A	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.89	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			Brown, Sandy cuttings with gravel. (Fill)	
5											
								55.5		6.5	
										Bottom of Exploration at 6.5 feet below ground surface. REFUSAL	
10											
15											
20											
25											

Remarks:

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Boring No.: BB-WPPB-102A

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Fish Bridge #0509 carries Garland Road over Pattee Pond Brook	**Boring No.:**	BB-WPPB-102B		
Location:	Winslow, Maine	**WIN:**	22268.00		

Driller:	MaineDOT	**Elevation (ft.)**	61.8	**Auger ID/OD:**	5" Dia.
Operator:	Daggett/Wilder	**Datum:**	NAVD88	**Sampler:**	N/A
Logged By:	J. Manahan	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	9/15/2020; 08:50-09:00	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	N/A
Boring Location:	51+80.6, 12.0 ft Rt.	**Casing ID/OD:**	N/A	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.89	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			Brown SAND and GRAVEL cuttings. (Fill)	
5											
								55.8		6.0 **Bottom of Exploration at 6.0 feet below ground surface.** REFUSAL	
10											
15											
20											
25											

Remarks:

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

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

Boring No.: BB-WPPB-102B

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Fish Bridge #0509 carries Garland Road over Pattee Pond Brook
Location: Winslow, Maine

Boring No.: BB-WPPB-103
WIN: 22268.00

Driller:	MaineDOT	**Elevation (ft.)**	62.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Wilder	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	J. Manahan	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/15/2020 & 9/16/2020	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	51+82.5, 8.8 ft RT.	**Casing ID/OD:**	HW-4" & NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.89	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			Brown Sand and Gravel cuttings, (Fill).	
5											
10	1D	18/12	10.00 - 11.50	6/6/6	12	18	80			Brown, damp, medium dense, Gravelly SAND, little silt, trace organics. (Fill)	G#336995 A-1-a, SM WC=19.9%
							32 OPEN HOLE	50.5		11.5 Cobbles at 11.5-13.2 ft bgs.	
										Wood in cuttings at 13.2 ft bgs. Cobble from 13.7-14.4 ft bgs.	
								47.6		14.4	
15	2D	24/24	15.00 - 17.00	3/2/4/4	6	9				Olive grey, moist, medium stiff, Clayey SILT, trace fine sand, (Glaciomarine Deposit).	
										Cuttings become grey at 19.0 ft bgs.	
20	3D	24/24	20.00 - 22.00	WOR/WOR/WOH/ WOH	---					Grey, moist, soft to medium stiff, Clayey SILT, trace fine to medium sand, (Glaciomarine Deposit). 55x110 mm vane raw torque readings: V1: 11.0/4.5 ft-lbs V2: 12.0/3.0 ft-lbs	G#336996 A-6, CL WC=37.7% LL=36 PL=23 PI=13
	V1		20.63 - 21.00	Su=491/201 psf							
	V2		21.63 - 22.00	Su=536/134 psf							
25											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Fish Bridge #0509 carries Garland Road over Pattee Pond Brook
Location: Winslow, Maine

Boring No.: BB-WPPB-103
WIN: 22268.00

Driller:	MaineDOT	**Elevation (ft.)**	62.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Wilder	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	J. Manahan	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/15/2020 & 9/16/2020	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	51+82.5, 8.8 ft RT.	**Casing ID/OD:**	HW-4" & NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.89	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	1U	24/24	25.00 - 27.00	WOR						Dark grey, moist, soft to medium stiff, Clayey SILT, trace fine sand, (Glaciomarine Deposit).	G,C#336997 A-6, CL WC=39.9% LL=38 PL=24 PI=14
	V3		27.63 - 28.00	Su=759/223 psf						55x110 mm vane raw torque readings: V3: 17.0/5.0 ft-lbs	
	MV		28.63 - 28.63	Would Not Push						Failed 55x110 mm vane attempt.	
								32.9		29.1	
30	4D	24/3	30.00 - 32.00	9/9/7/10	16	24	14			Grey, wet, medium dense, Silty SAND, little gravel, (Marine Sand).	
							43				
							100				
								29.0		33.0	
	R1	60/60	33.00 - 38.00	RQD = 23%			NQ-2			Top of Bedrock at Elev. 29.0 ft. R1: Bedrock: Black and grey banded, very-grained, PHYLLITE to METASILTSTONE with calcite veins, hard, fresh, breaks along steep, very close, tight foliation/bedding, low angle breaks are planar and close. Rock Quality = Very Poor [Waterville Formation] R1: Core Times (min:sec) 33.0-34.0 ft (1:53) 34.0-35.0 ft (1:54) 35.0-36.0 ft (2:13) 36.0-37.0 ft (2:46) 37.0-28.0 ft (3:50) 100% Recovery	
35											
	R2	60/60	38.00 - 43.00	RQD = 52%						R2: Bedrock: Similar to R1 except upper core is more massive, and lower core is more fractured along finer bedding. Rock Quality = Fair R2: Core Times (min:sec) 38.0-39.0 ft (3:12) 39.0-40.0 ft (3:02) 40.0-41.0 ft (3:08) 41.0-42.0 ft (3:30) 42.0-43.0 ft (5:19) 100% Recovery	
40											
								19.0		43.0	
										Bottom of Exploration at 43.0 feet below ground surface.	
45											
50											

Remarks:

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

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

Boring No.: BB-WPPB-103

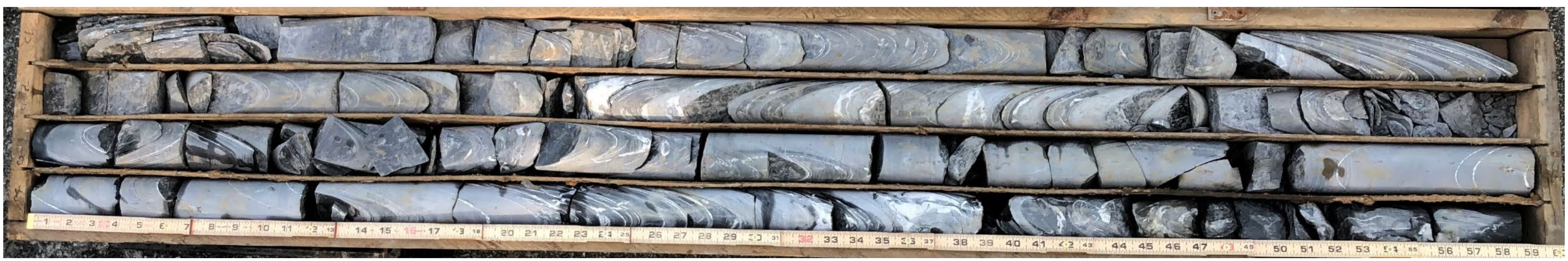
Appendix B

Rock Core Photographs

MaineDOT

Fish Bridge #0509 Carries Garland Road Over Pattee Pond Brook
Winslow, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Pentration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-WPPB-101	R1	40.1-44.9	58	58	9	16	PHYLLITE to METASILTSONE	1
BB-WPPB-101	R2	44.9-49.9	60	51	13	22	PHYLLITE TO METASILTSTONE	2
BB-WPPB-103	R1	33.0-38.0	60	60	14	23	PHYLLITE to METASILTSTONE	3
BB-WPPB-103	R2	38.0-43.0	60	60	31	52	PHYLLITE to METASILTSTONE	4



Notes: 1. “Box row” indicates the section of the box where the core run is contained: 1 = top, 4 = bottom.
2. Top of rock core is placed on left side of core box.

Appendix C

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Winslow   Work Number: 22268.00

Boring & Sample	Station	Offset	Depth	Reference	G.S.D.C.	W.C.	L.L.	P.I.	Classification		
Identification Number	(Feet)	(Feet)	(Feet)	Number	Sheet	%			Unified	AASHTO	Frost
BB-WPPB-101, 1D	51+21.3	8.5 Lt.	5.0-7.0	336991	1	25.0	38	16	CL	A-6	III
BB-WPPB-101, 3D	51+21.3	8.5 Lt.	16.0-18.0	336992	1	42.4	50	26	CL	A-7-6	III
BB-WPPB-101, 4D	51+21.3	8.5 Lt.	25.0-27.0	336993	1	35.7	33	10	CL	A-4	IV
BB-WPPB-101, 3U	51+21.3	8.5 Lt.	30.0-32.0	336994	1	35.5	40	16	CL	A-7-6	III
BB-WPPB-103, 1D	51+82.5	8.8 Rt.	10.0-11.5	336995	2	19.9			SM	A-1-a	II
BB-WPPB-103, 3D	51+82.5	8.8 Rt.	20.0-22.0	336996	2	37.7	36	13	CL	A-6	III
BB-WPPB-103, 1U	51+82.5	8.8 Rt.	25.0-27.0	336997	2	39.9	38	14	CL	A-6	III

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

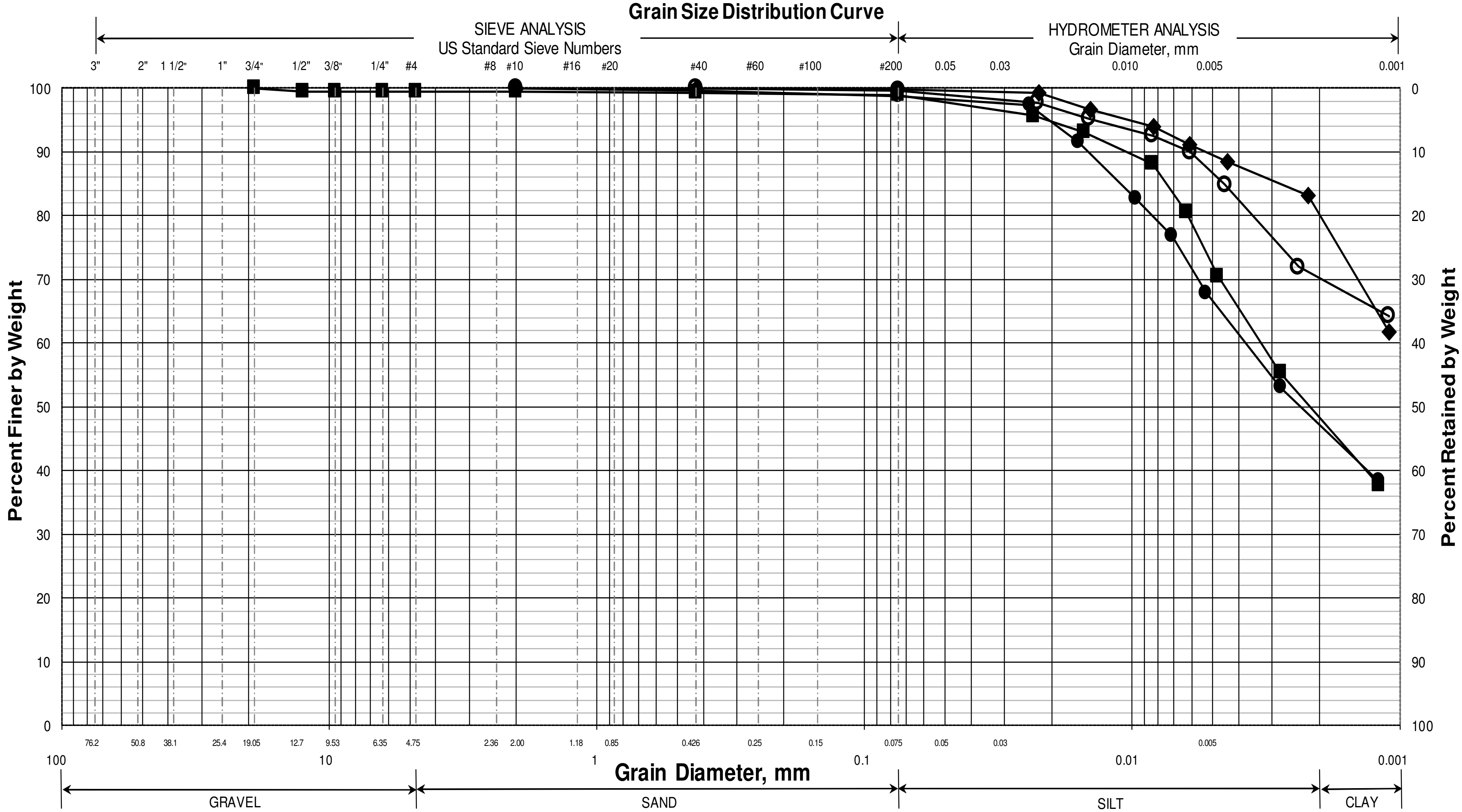
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

Maine Department of Transportation
Grain Size Distribution Curve



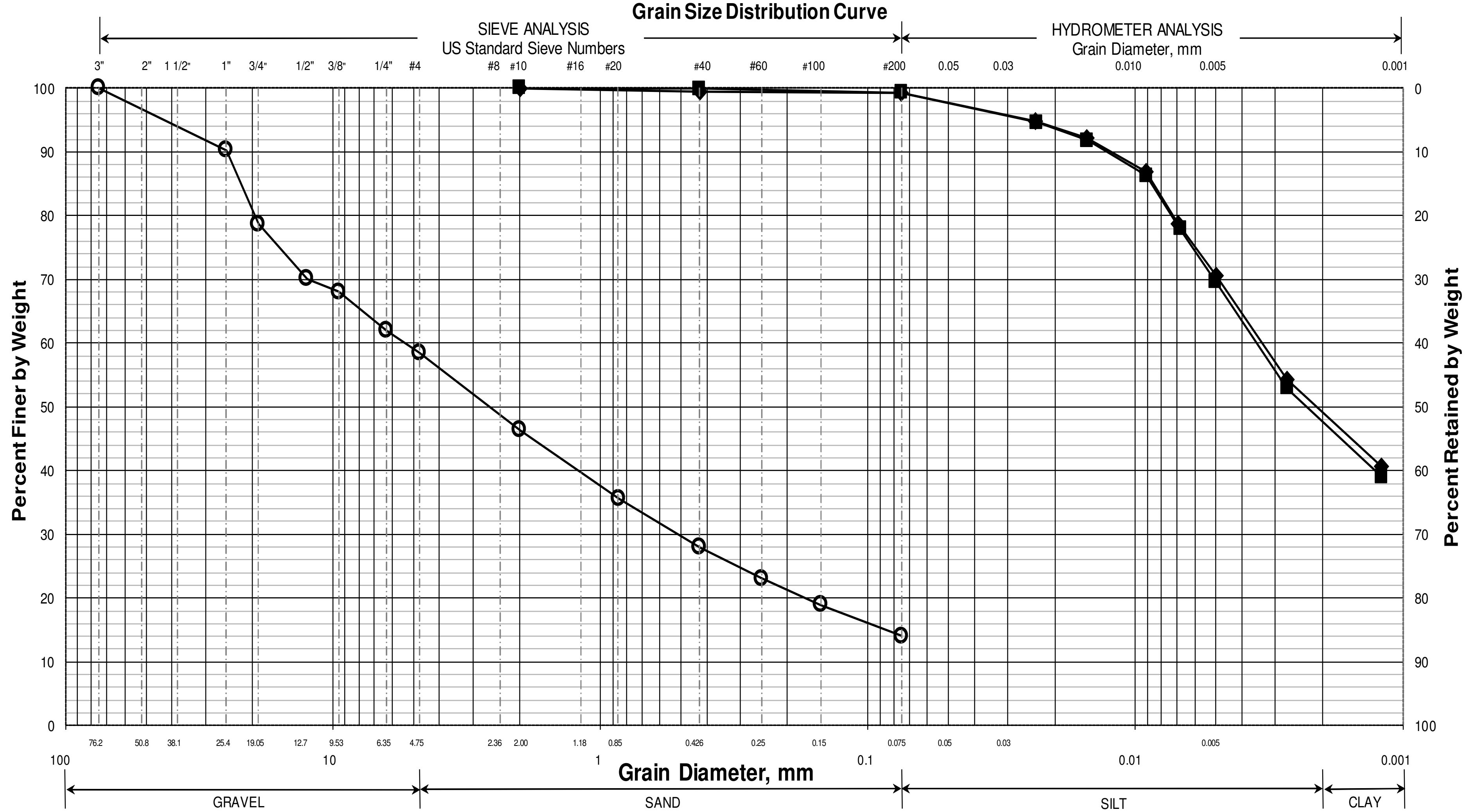
UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-WPPB-101/1D	51+21.3	8.5 LT	5.0-7.0	CLAY, some silt, trace sand.	25	38	22	16
◆	BB-WPPB-101/3D	51+21.3	8.5 LT	16.0-18.0	CLAY, little silt, trace sand..	42.4	50	24	26
■	BB-WPPB-101/4D	51+21.3	8.5 LT	25.0-27.0	Clayey SILT, trace sand, trace gravel.	35.7	33	23	10
●	BB-WPPB-101/3U	51+21.3	8.5 LT	30.0-32.0	Clayey SILT, trace sand.	35.5	40	24	16
▲									
×									

WIN	
022268.00	
Town	
Winslow	
Reported by/Date	
WHITE, TERRY A	10/30/2020

SHEET 1

Maine Department of Transportation
Grain Size Distribution Curve



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-WPPB-103/1D	51+82.5	8.8 RT	10.0-11.5	Gravelly SAND, little silt.	19.9			
◆	BB-WPPB-103/3D	51+82.5	8.8 RT	20.0-22.0	Clayey SILT, trace sand.	37.7	36	23	13
■	BB-WPPB-103/1U	51+82.5	8.8 RT	25.0-27.0	Clayey SILT, trace sand.	39.9	38	24	14
●									
▲									
×									

WIN	
022268.00	
Town	
Winslow	
Reported by/Date	
WHITE, TERRY A	10/30/2020

SHEET 2

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
336991	BB-WPPB-101/1D	GEOTECHNICAL (DISTURBED)	9/10/2020	9/30/2020

Sample Type: **GEOTECHNICAL** Location: Station: **51+21.3** Offset, ft: **8.5** **LT** Dbfg, ft: **5.0-7.0**

WIN/Town **022268.00 - WINSLOW** Sampler: **JAMES MANAHAN**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	100.0
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	100.0
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	99.7
[0.0226 mm]	97.7
[0.0145 mm]	95.2
[0.0084 mm]	92.6
[0.0061 mm]	90.0
[0.0045 mm]	84.9
[0.0024 mm]	72.0
[0.0011 mm]	64.3

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	38
Plastic Limit (T 90), %	22
Plasticity Index (T 90), %	16
Specific Gravity, Corrected to 20°C (T 100)	2.67
Loss on Ignition, % (T 267)	
Water Content (T 265), %	25.0

Consolidation (T 216)

Trimmings, Water Content, %

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

Vane Shear Test on Shelby Tubes (Maine DOT)

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

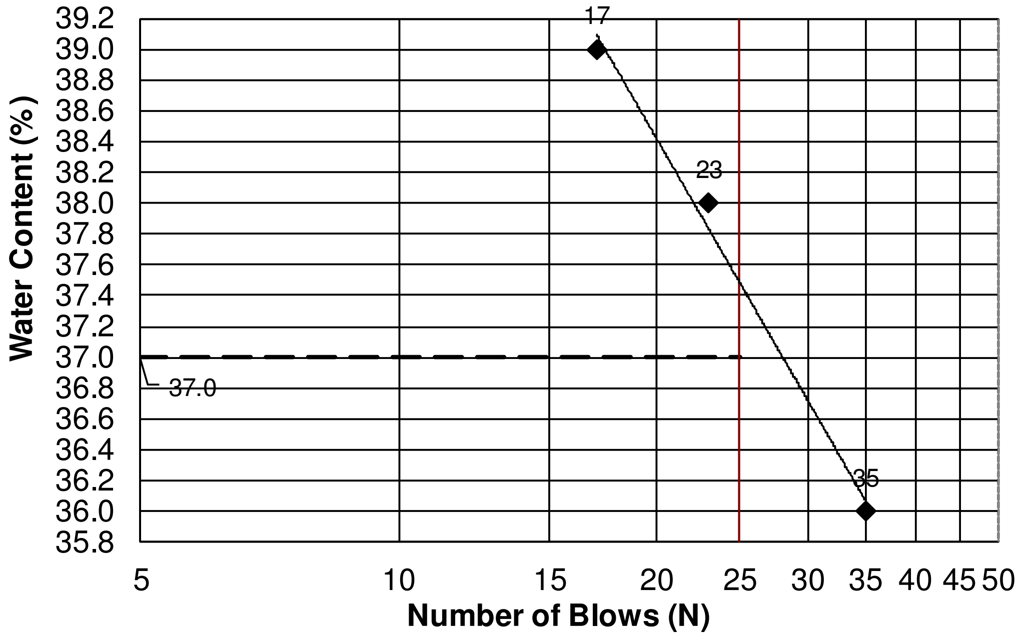
Comments:

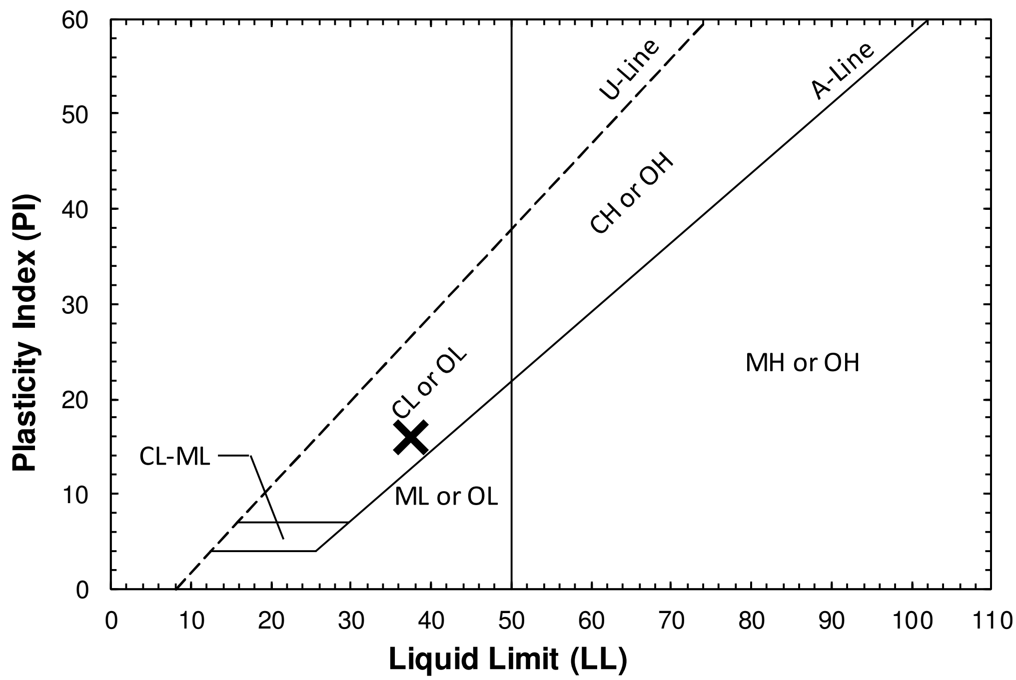
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **10/14/2020**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Winslow	Reference No.	336991
WIN	022268.00	Water Content, %	25
Sampled	9/10/2020	Liquid Limit @ 25 blows (T 89), %	38
Boring No./Sample No.	BB-WPPB-101/1D	Plastic Limit (T 90), %	22
Station	51+21.3	Plasticity Index (T 90), %	16
Depth	5.0-7.0	Tested By	BBURR





GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
336992	BB-WPPB-101/3D	GEOTECHNICAL (DISTURBED)	9/10/2020	9/30/2020

Sample Type: **GEOTECHNICAL** Location: Station: **51+21.3** Offset, ft: **8.5 LT** Dbfg, ft: **16.0-18.0**

WIN/Town **022268.00 - WINSLOW** Sampler: **JAMES MANAHAN**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	100.0
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	99.9
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	99.8
[0.0223 mm]	99.2
[0.0143 mm]	96.6
[0.0083 mm]	93.9
[0.0061 mm]	91.2
[0.0044 mm]	88.5
[0.0022 mm]	83.1
[0.0011 mm]	61.7

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	50
Plastic Limit (T 90), %	24
Plasticity Index (T 90), %	26
Specific Gravity, Corrected to 20°C (T 100)	2.75
Loss on Ignition, % (T 267)	
Water Content (T 265), %	42.4

Consolidation (T 216)

Trimmings, Water Content, %

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

Vane Shear Test on Shelby Tubes (Maine DOT)

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

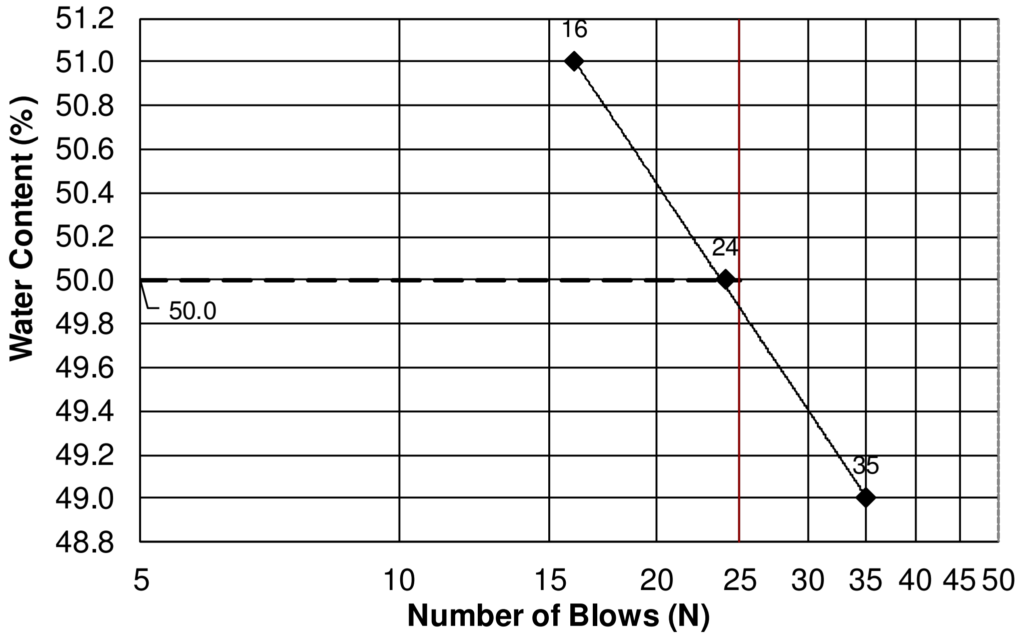
Comments:

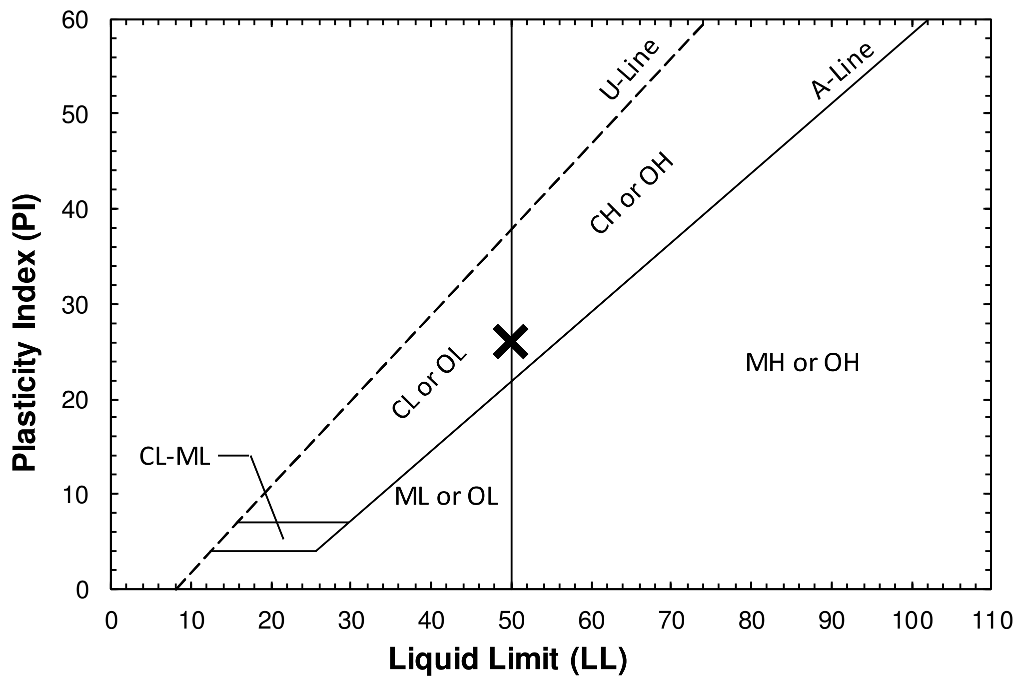
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **10/14/2020**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Winslow	Reference No.	336992
WIN	022268.00	Water Content, %	42.4
Sampled	9/10/2020	Liquid Limit @ 25 blows (T 89), %	50
Boring No./Sample No.	BB-WPPB-101/3D	Plastic Limit (T 90), %	24
Station	51+21.3	Plasticity Index (T 90), %	26
Depth	16.0-18.0	Tested By	BBURR





GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
336993	BB-WPPB-101/4D	GEOTECHNICAL (DISTURBED)	9/10/2020	9/30/2020

Sample Type: **GEOTECHNICAL** Location: Station: **51+21.3** Offset, ft: **8.5 LT** Dbfg, ft: **25.0-27.0**

WIN/Town **022268.00 - WINSLOW** Sampler: **JAMES MANAHAN**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	100.0
½ in. [12.5 mm]	99.5
⅜ in. [9.5 mm]	99.5
¼ in. [6.3 mm]	99.5
No. 4 [4.75 mm]	99.5
No. 10 [2.00 mm]	99.5
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	99.3
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	99.0
[0.0234 mm]	95.7
[0.0151 mm]	93.1
[0.0084 mm]	88.2
[0.0063 mm]	80.6
[0.0048 mm]	70.5
[0.0028 mm]	55.4
[0.0012 mm]	37.8

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	33
Plastic Limit (T 90), %	23
Plasticity Index (T 90), %	10
Specific Gravity, Corrected to 20°C (T 100)	2.71
Loss on Ignition, % (T 267)	
Water Content (T 265), %	35.7

Consolidation (T 216)

Trimmings, Water Content, %

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

Vane Shear Test on Shelby Tubes (Maine DOT)

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

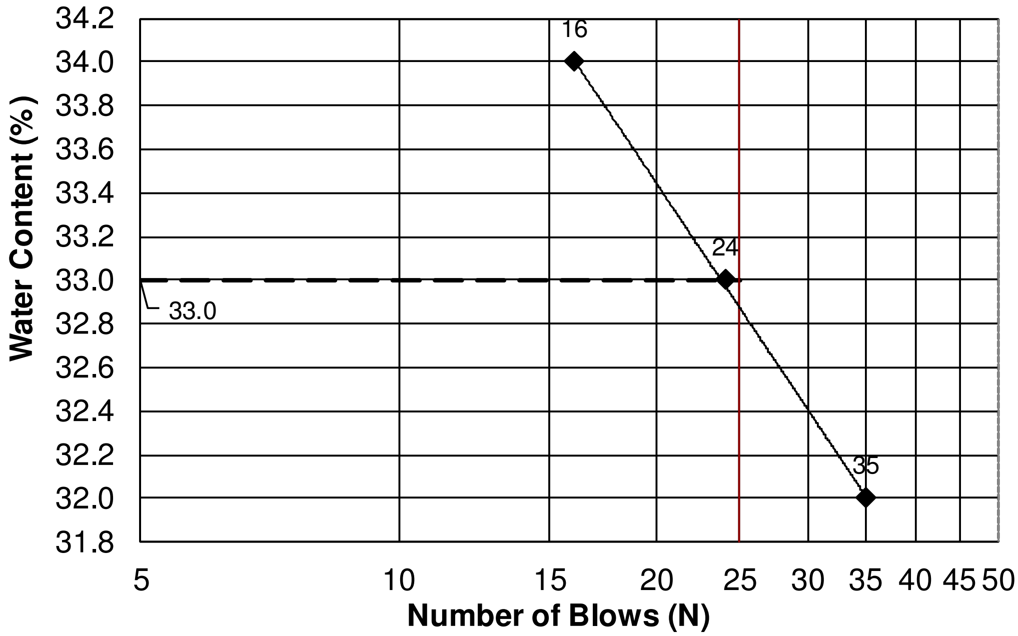
Comments:

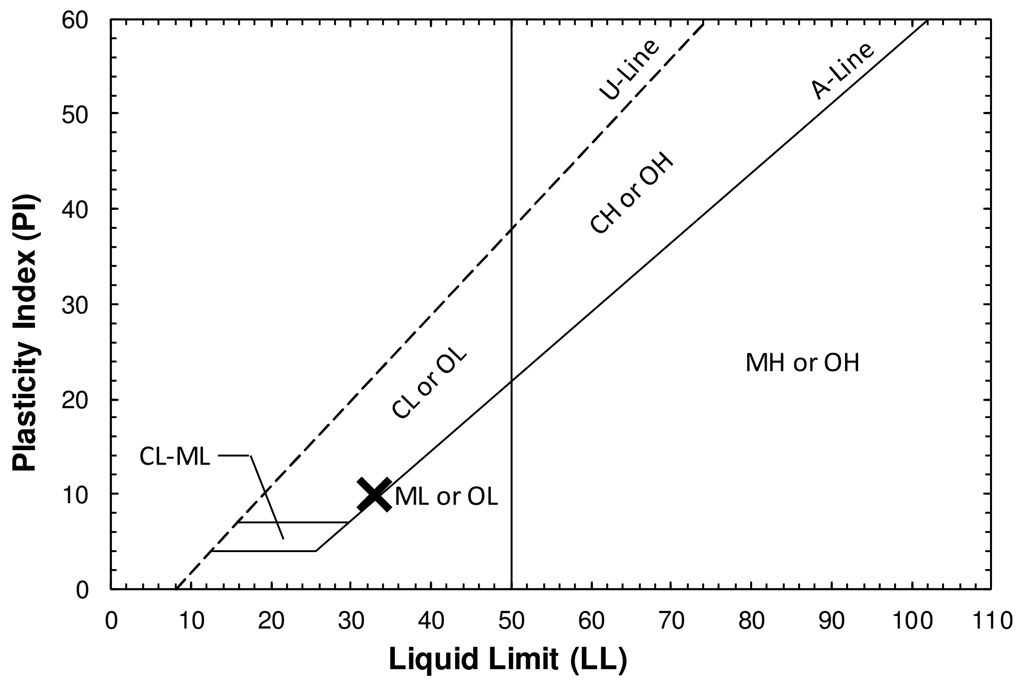
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **10/14/2020**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Winslow	Reference No.	336993
WIN	022268.00	Water Content, %	35.7
Sampled	9/10/2020	Liquid Limit @ 25 blows (T 89), %	33
Boring No./Sample No.	BB-WPPB-101/4D	Plastic Limit (T 90), %	23
Station	51+21.3	Plasticity Index (T 90), %	10
Depth	25.0-27.0	Tested By	BBURR





GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
336994	BB-WPPB-101/3U	GEOTECHNICAL (UNDISTURBED)	9/15/2020	9/30/2020

Sample Type: **GEOTECHNICAL** Location: Station: **51+21.3** Offset, ft: **8.5 LT** Dbfg, ft: **30.0-32.0**

WIN/Town **022268.00 - WINSLOW** Sampler: **JAMES MANAHAN**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	100.0
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	99.6
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	98.8
[0.0242 mm]	97.4
[0.0159 mm]	91.5
[0.0097 mm]	82.7
[0.0071 mm]	76.8
[0.0053 mm]	67.9
[0.0028 mm]	53.1
[0.0012 mm]	38.4

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	40
Plastic Limit (T 90), %	24
Plasticity Index (T 90), %	16
Specific Gravity, Corrected to 20°C (T 100)	2.70
Loss on Ignition, % (T 267)	
Water Content (T 265), %	35.5

Consolidation (T 216)

Trimmings, Water Content, %: **43.4**

	Initial	Final		Void Ratio	% Strain
Water Content, %	44.5	30.61	Pmin		
Dry Density, lbs/ft³	77.916	92.288	Pp		
Void Ratio	1.16	0.826	Pmax		
Saturation, %	103.28	100	Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths
0-6	0.115	0	0.167	0	45.1	Medium dark grey clay, black streaks and spots, trace shell fragments
7.5-12	0.23	0.01	0.24	0.021	42.6	As above
12-18	0.251	0.01	0.219	0.01	43.9	As above
18-24	0.24	0.01	0.199	0.021	44.7	As above

Comments:

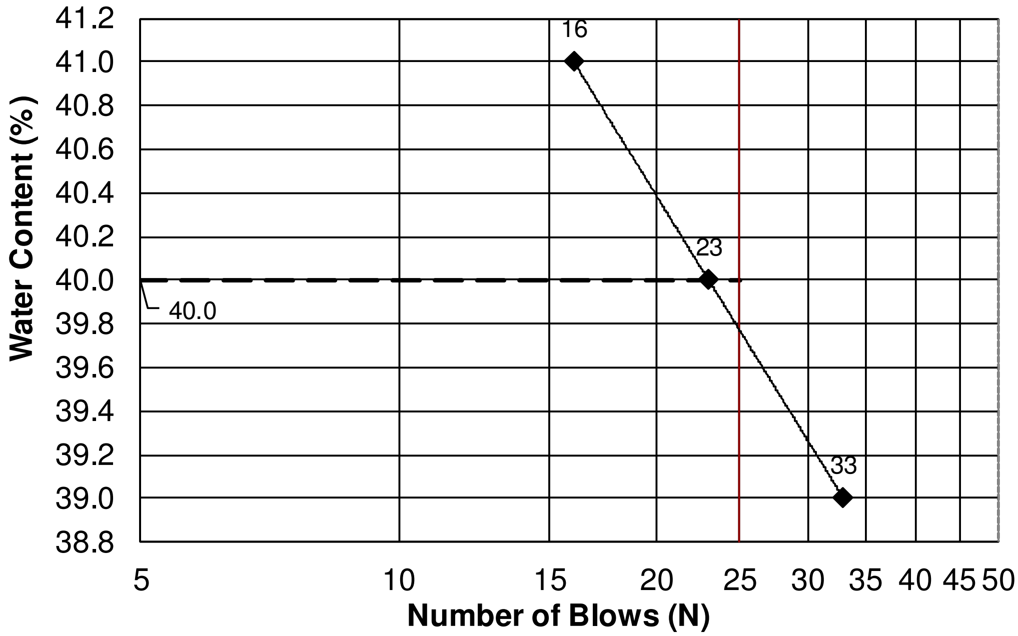
Maine Sensitive Loading Sequence

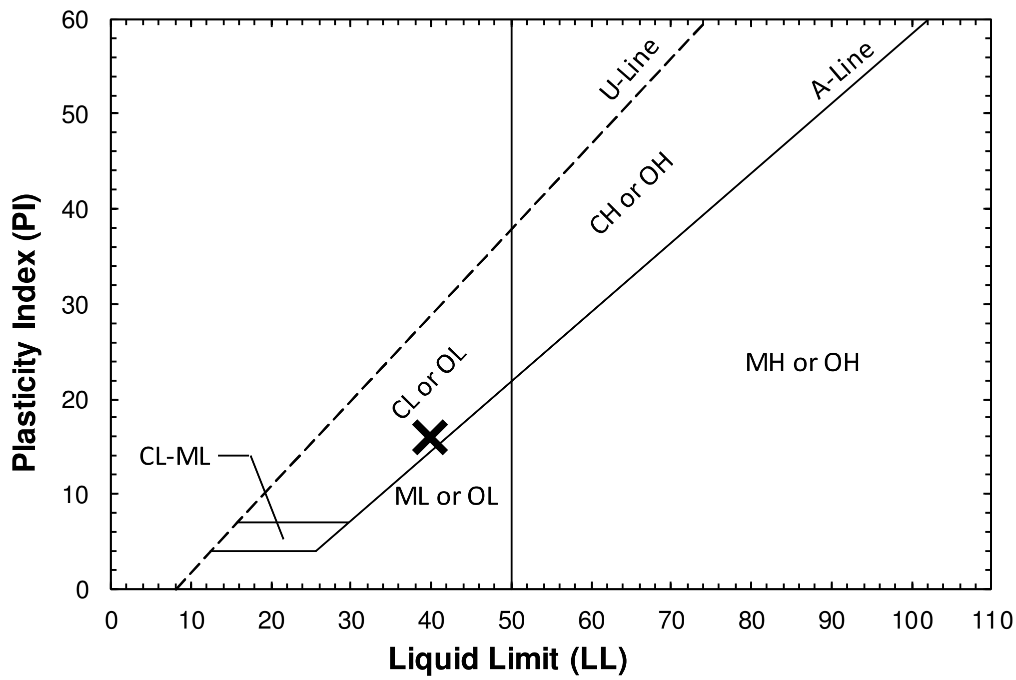
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **10/28/2020**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Winslow	Reference No.	336994
WIN	022268.00	Water Content, %	35.5
Sampled	9/15/2020	Liquid Limit @ 25 blows (T 89), %	40
Boring No./Sample No.	BB-WPPB-101/3U	Plastic Limit (T 90), %	24
Station	51+21.3	Plasticity Index (T 90), %	16
Depth	30.0-32.0	Tested By	BBURR





MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
336996	BB-WPPB-103/3D	GEOTECHNICAL (DISTURBED)	9/15/2020	9/30/2020

Sample Type: **GEOTECHNICAL** Location: Station: **51+82.5** Offset, ft: **8.8** **RT** Dbfg, ft: **20.0-22.0**

WIN/Town **022268.00 - WINSLOW** Sampler: **JAMES MANAHAN**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	100.0
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	99.5
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	99.3
[0.0237 mm]	94.9
[0.0152 mm]	92.2
[0.0091 mm]	86.8
[0.0069 mm]	78.7
[0.0050 mm]	70.5
[0.0027 mm]	54.3
[0.0012 mm]	40.7

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	36
Plastic Limit (T 90), %	23
Plasticity Index (T 90), %	13
Specific Gravity, Corrected to 20°C (T 100)	2.67
Loss on Ignition, % (T 267)	
Water Content (T 265), %	37.7

Consolidation (T 216)

Trimmings, Water Content, %

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

Vane Shear Test on Shelby Tubes (Maine DOT)

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

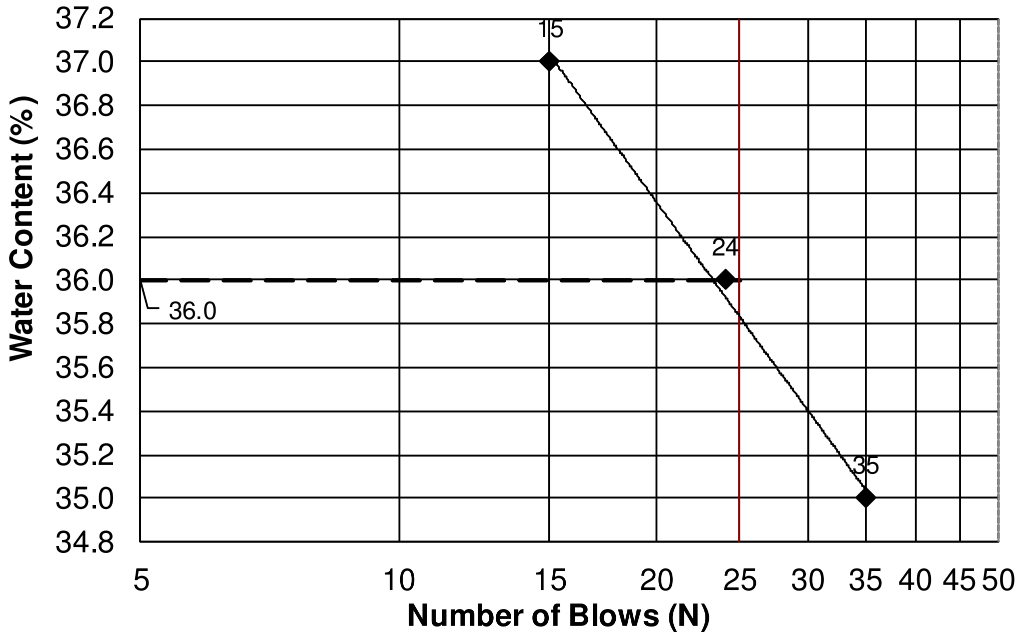
Comments:

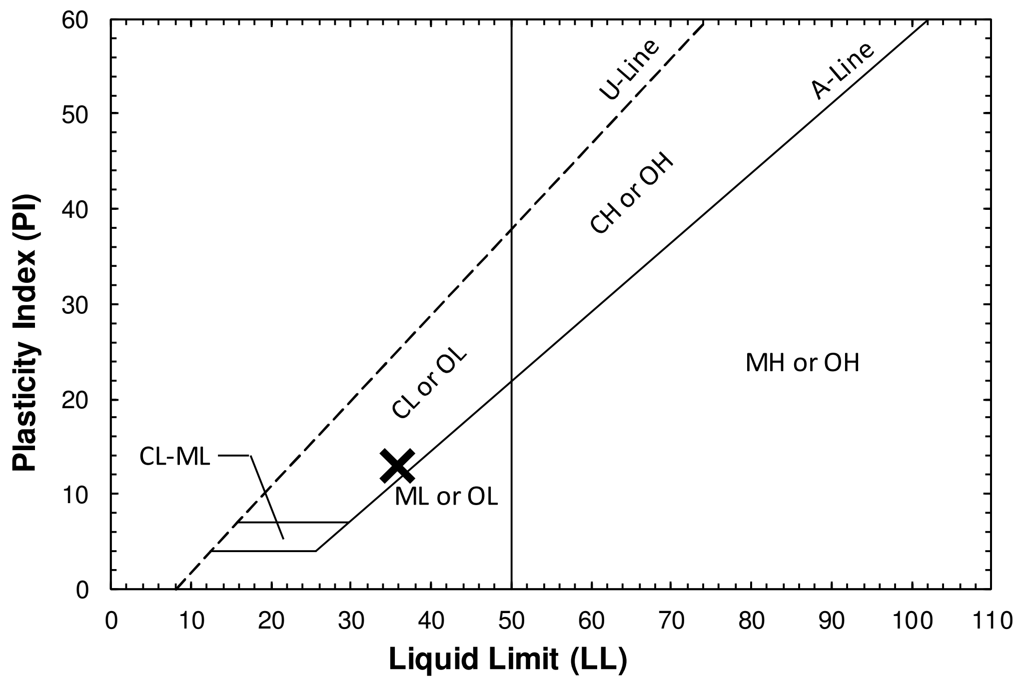
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **10/14/2020**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Winslow	Reference No.	336996
WIN	022268.00	Water Content, %	37.7
Sampled	9/15/2020	Liquid Limit @ 25 blows (T 89), %	36
Boring No./Sample No.	BB-WPPB-103/3D	Plastic Limit (T 90), %	23
Station	51+82.5	Plasticity Index (T 90), %	13
Depth	20.0-22.0	Tested By	GLIDS





GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
336997	BB-WPPB-103/1U	GEOTECHNICAL (UNDISTURBED)	9/15/2020	9/30/2020

Sample Type: **GEOTECHNICAL** Location: Station: **51+82.5** Offset, ft: **8.8** **RT** Dbfg, ft: **25.0-27.0**

WIN/Town **022268.00 - WINSLOW** Sampler: **JAMES MANAHAN**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	100.0
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	99.9
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	99.3
[0.0233 mm]	94.6
[0.0151 mm]	91.8
[0.0091 mm]	86.3
[0.0068 mm]	77.9
[0.0050 mm]	69.6
[0.0027 mm]	52.9
[0.0012 mm]	39.0

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	38
Plastic Limit (T 90), %	24
Plasticity Index (T 90), %	14
Specific Gravity, Corrected to 20°C (T 100)	2.75
Loss on Ignition, % (T 267)	
Water Content (T 265), %	39.9

Consolidation (T 216)

Trimmings, Water Content, % **42.8**

	Initial	Final		Void Ratio	% Strain
Water Content, %	43.96	29.97	Pmin		
Dry Density, lbs/ft³	78.155	94.106	Pp		
Void Ratio	1.2	0.824	Pmax		
Saturation, %	101.02	100	Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths
0-6	0.251	0.021	0.24	0.031	42.8	Medium dark grey clay, black streaks and spots, trace shell fragments
7.5-12	0.199	0.01	0.188	0.021	44.0	As above
12-18	0.199	0.01	0.219	0.01	44.4	As above, silt line at 16.5"
18-24	0.188	0.01	0.178	0.01	46.2	Medium dark grey clay, black streaks and spots, trace shell fragments

Comments:

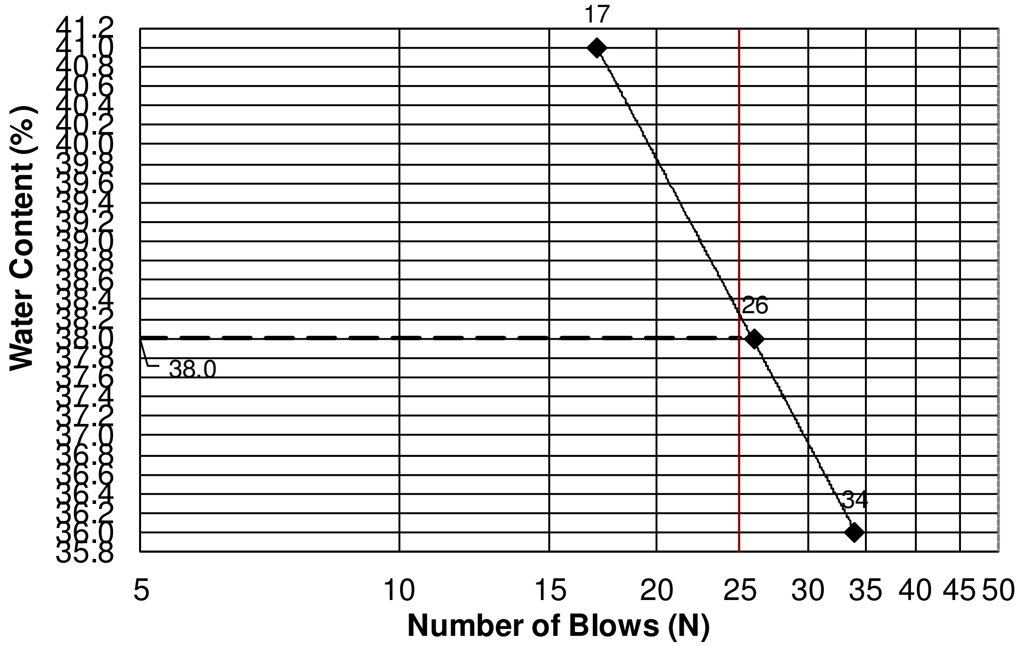
Maine Sensitive Loading Sequence

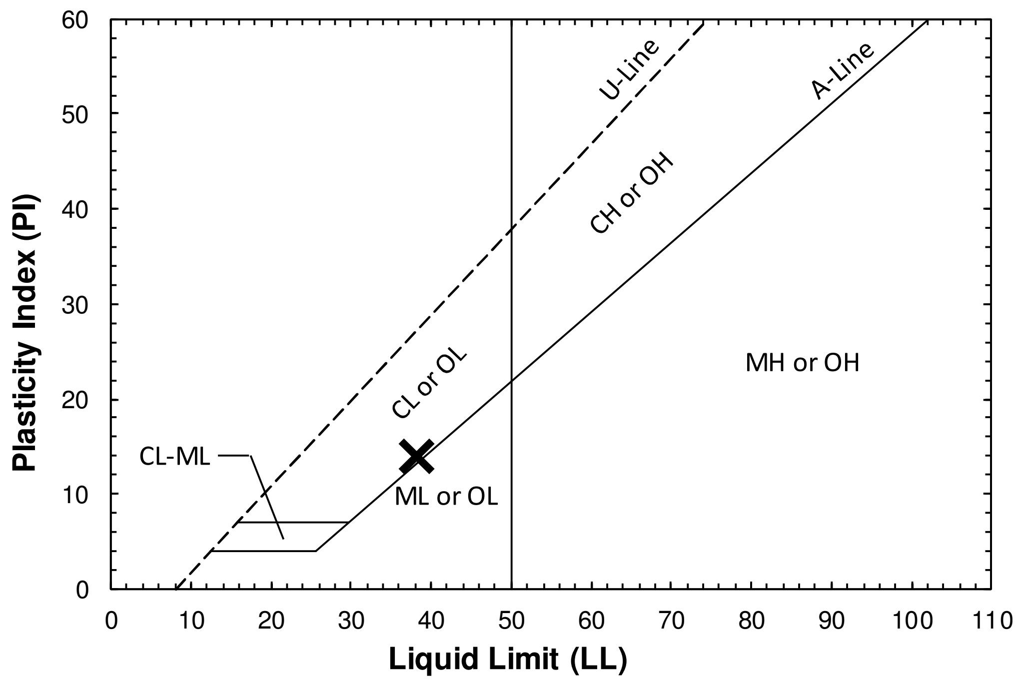
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **10/27/2020**

Paper Copy: Lab File; Project File; Geotech File

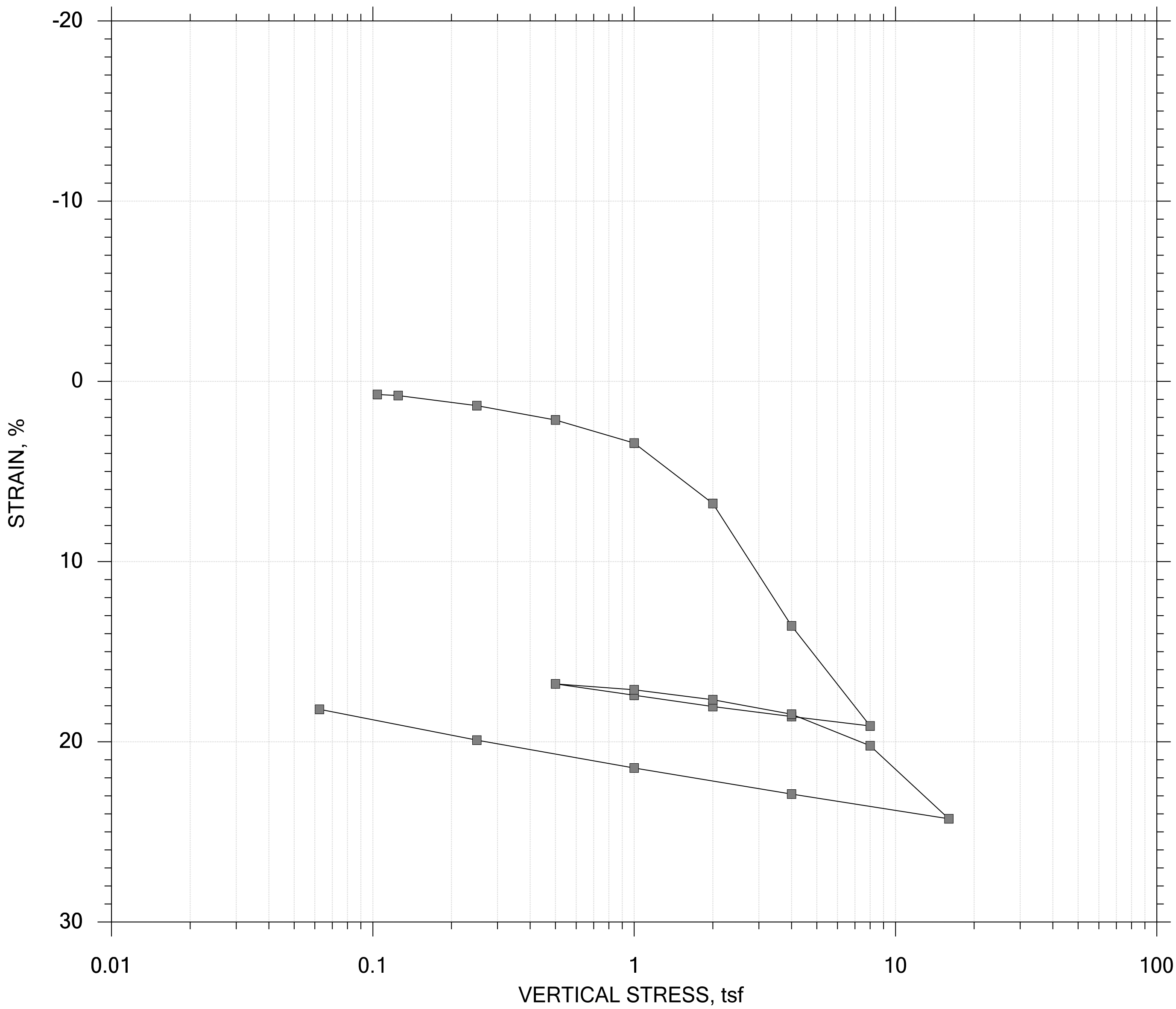
TOWN	Winslow	Reference No.	336997
WIN	022268.00	Water Content, %	39.9
Sampled	9/15/2020	Liquid Limit @ 25 blows (T 89), %	38
Boring No./Sample No.	BB-WPPB-103/1U	Plastic Limit (T 90), %	24
Station	51+82.5	Plasticity Index (T 90), %	14
Depth	25.0-27.0	Tested By	BBURR





One-Dimensional Consolidation by ASTM D2435 - Method B

SUMMARY REPORT



					Before Test	After Test
Current Vertical Effective Stress: ---				Water Content, %	44.50	30.61
Preconsolidation Stress: ---				Dry Unit Weight, pcf	77.916	92.288
Compression Ratio: ---				Saturation, %	103.28	100.00
Diameter: 2.495 in		Height: 0.9921 in		Void Ratio	1.16	0.83
LL: 40	PL: 24	PI: 16	GS: 2.70			

	Project: Winslow	Location: --	Project No.: 022268.00
	Boring No.: BB-WPPB-101	Tested By: GSL	Checked By: --
	Sample No.: 3U	Test Date: 10/9/2020	Test No.: 336994
	Depth: 30.0-32.0 FT	Sample Type: Undisturbed	Elevation: --
	Description: Grey Clay		
	Remarks: Maine Sensitive Load/Unload/Reload/Unload Consolidation Test		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

Project: Winslow	Location: --	Project No.: 022268.00
Boring No.: BB-WPPB-101	Tested By: GSL	Checked By: --
Sample No.: 3U	Test Date: 10/9/2020	Depth: 30.0-32.0 FT
Test No.: 336994	Sample Type: Undisturbed	Elevation: --

Soil Description: Grey Clay
Remarks: Maine Sensitive Load/Unload/Reload/Unload Consolidation Test

Measured Specific Gravity: 2.70	Liquid Limit: 40	Specimen Diameter: 2.50 in
Initial Void Ratio: 1.16	Plastic Limit: 24	Initial Height: 0.99 in
Final Void Ratio: 0.826	Plasticity Index: 16	Final Height: 0.84 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	205	RING	RING	218
Wt. Container + Wet Soil, gm	90.320	405.56	391.78	194.98
Wt. Container + Dry Soil, gm	82.620	361.42	361.42	164.66
Wt. Container, gm	64.890	262.21	262.21	65.600
Wt. Dry Soil, gm	17.730	99.205	99.205	99.060
Water Content, %	43.43	44.50	30.61	30.61
Void Ratio	---	1.16	0.826	---
Degree of Saturation, %	---	103.28	100.00	---
Dry Unit Weight, pcf	---	77.916	92.288	---

One-Dimensional Consolidation by ASTM D2435 - Method B

Project: Winslow
Boring No.: BB-WPPB-101
Sample No.: 3U
Test No.: 336994

Location: --
Tested By: GSL
Test Date: 10/9/2020
Sample Type: Undisturbed

Project No.: 022268.00
Checked By: --
Depth: 30.0-32.0 FT
Elevation: --

Soil Description: Grey Clay
Remarks: Maine Sensitive Load/Unload/Reload/Unload Consolidation Test

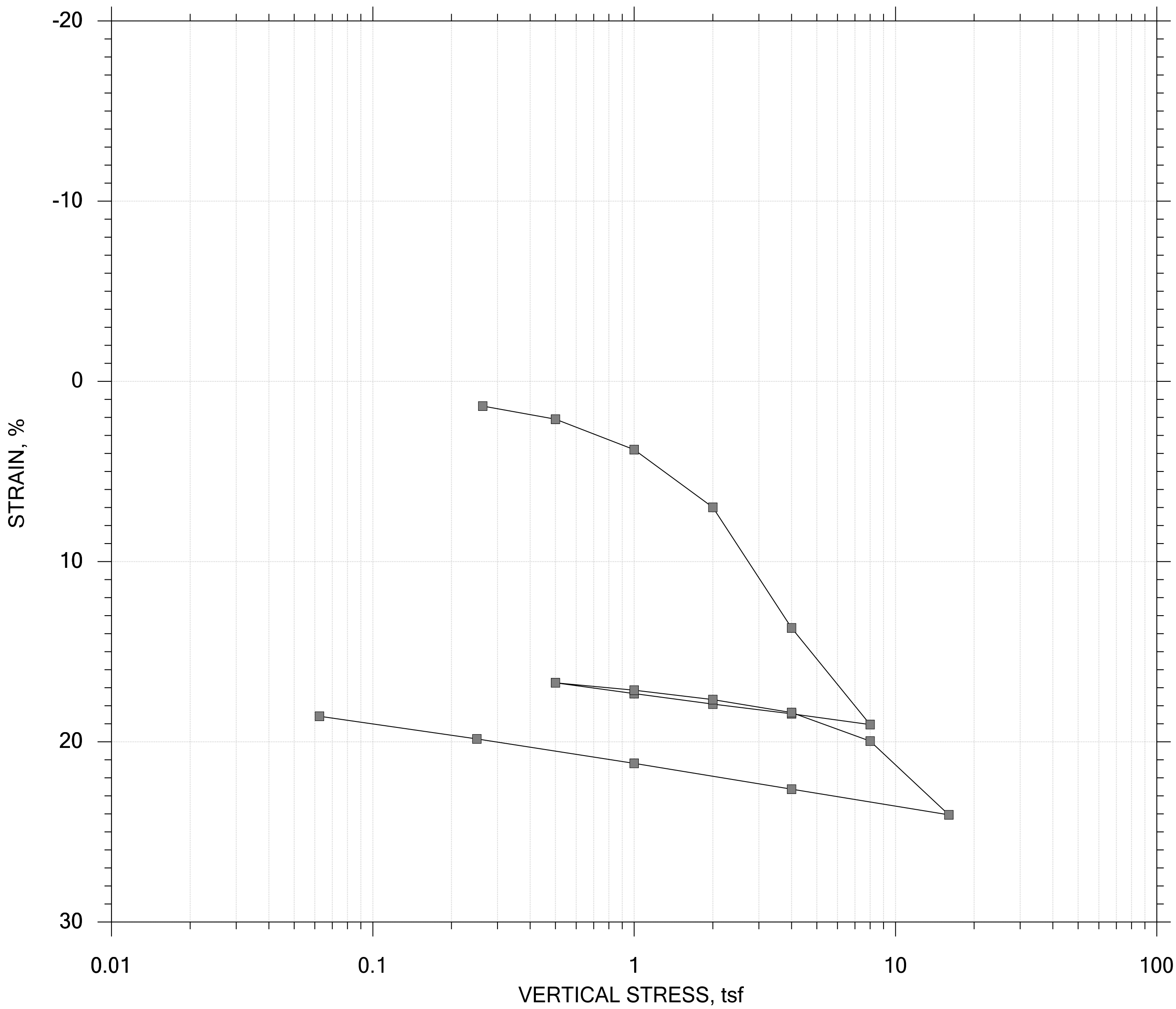
Displacement at End of Increment

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Sq.Rt T90 min	Cv ft²/sec	Mv 1/tsf	k ft/day
1	0.104	0.007215	1.15	0.727	34.333	6.98e-007	6.99e-002	1.32e-004
2	0.125	0.007807	1.15	0.787	0.000	0.00e+000	2.85e-002	0.00e+000
3	0.250	0.01335	1.13	1.35	9.711	2.43e-006	4.47e-002	2.94e-004
4	0.500	0.02127	1.12	2.14	5.008	4.66e-006	3.19e-002	4.01e-004
5	1.00	0.03397	1.09	3.42	7.045	3.24e-006	2.56e-002	2.24e-004
6	2.00	0.06725	1.02	6.78	18.495	1.18e-006	3.35e-002	1.06e-004
7	4.00	0.1345	0.870	13.6	22.807	8.54e-007	3.39e-002	7.82e-005
8	8.00	0.1897	0.750	19.1	8.914	1.90e-006	1.39e-002	7.11e-005
9	4.00	0.1845	0.761	18.6	0.838	1.90e-005	1.30e-003	6.66e-005
10	2.00	0.1790	0.773	18.0	1.351	1.19e-005	2.77e-003	8.90e-005
11	1.00	0.1728	0.787	17.4	3.307	4.94e-006	6.30e-003	8.39e-005
12	0.500	0.1665	0.800	16.8	10.192	1.63e-006	1.26e-002	5.53e-005
13	1.00	0.1698	0.793	17.1	5.764	2.89e-006	6.51e-003	5.08e-005
14	2.00	0.1752	0.781	17.7	2.117	7.78e-006	5.54e-003	1.16e-004
15	4.00	0.1832	0.764	18.5	2.218	7.31e-006	4.02e-003	7.92e-005
16	8.00	0.2006	0.726	20.2	6.012	2.61e-006	4.37e-003	3.08e-005
17	16.0	0.2408	0.638	24.3	4.776	3.06e-006	5.07e-003	4.18e-005
18	4.00	0.2272	0.668	22.9	0.780	1.81e-005	1.14e-003	5.58e-005
19	1.00	0.2128	0.699	21.5	5.142	2.84e-006	4.81e-003	3.69e-005
20	0.250	0.1975	0.733	19.9	28.601	5.31e-007	2.06e-002	2.95e-005
21	0.0625	0.1805	0.770	18.2	111.586	1.42e-007	9.12e-002	3.49e-005

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Log T50 min	Cv ft²/sec	Mv 1/tsf	k ft/day	Ca %
1	0.104	0.007215	1.15	0.727	0.000	0.00e+000	6.99e-002	0.00e+000	0.00e+000
2	0.125	0.007807	1.15	0.787	0.000	0.00e+000	2.85e-002	0.00e+000	0.00e+000
3	0.250	0.01335	1.13	1.35	1.971	2.79e-006	4.47e-002	3.36e-004	0.00e+000
4	0.500	0.02127	1.12	2.14	1.177	4.60e-006	3.19e-002	3.96e-004	0.00e+000
5	1.00	0.03397	1.09	3.42	1.560	3.40e-006	2.56e-002	2.35e-004	0.00e+000
6	2.00	0.06725	1.02	6.78	0.000	0.00e+000	3.35e-002	0.00e+000	0.00e+000
7	4.00	0.1345	0.870	13.6	4.763	9.51e-007	3.39e-002	8.69e-005	0.00e+000
8	8.00	0.1897	0.750	19.1	2.123	1.85e-006	1.39e-002	6.93e-005	0.00e+000
9	4.00	0.1845	0.761	18.6	0.000	0.00e+000	1.30e-003	0.00e+000	0.00e+000
10	2.00	0.1790	0.773	18.0	0.362	1.03e-005	2.77e-003	7.72e-005	0.00e+000
11	1.00	0.1728	0.787	17.4	1.030	3.69e-006	6.30e-003	6.26e-005	0.00e+000
12	0.500	0.1665	0.800	16.8	0.000	0.00e+000	1.26e-002	0.00e+000	0.00e+000
13	1.00	0.1698	0.793	17.1	0.000	0.00e+000	6.51e-003	0.00e+000	0.00e+000
14	2.00	0.1752	0.781	17.7	0.649	5.90e-006	5.54e-003	8.81e-005	0.00e+000
15	4.00	0.1832	0.764	18.5	0.456	8.26e-006	4.02e-003	8.95e-005	0.00e+000
16	8.00	0.2006	0.726	20.2	0.962	3.79e-006	4.37e-003	4.47e-005	0.00e+000
17	16.0	0.2408	0.638	24.3	1.145	2.96e-006	5.07e-003	4.05e-005	0.00e+000
18	4.00	0.2272	0.668	22.9	0.187	1.75e-005	1.14e-003	5.41e-005	0.00e+000
19	1.00	0.2128	0.699	21.5	1.063	3.20e-006	4.81e-003	4.15e-005	0.00e+000
20	0.250	0.1975	0.733	19.9	0.000	0.00e+000	2.06e-002	0.00e+000	0.00e+000
21	0.0625	0.1805	0.770	18.2	0.000	0.00e+000	9.12e-002	0.00e+000	0.00e+000

One-Dimensional Consolidation by ASTM D2435 - Method B

SUMMARY REPORT



					Before Test	After Test
Current Vertical Effective Stress: ---				Water Content, %	43.96	29.97
Preconsolidation Stress: ---				Dry Unit Weight, pcf	78.155	94.106
Compression Ratio: ---				Saturation, %	101.02	100.00
Diameter: 2.495 in		Height: 0.9988 in		Void Ratio	1.20	0.82
LL: 38	PL: 24	PI: 14	GS: 2.75			

	Project: Winslow	Location: --	Project No.: 22268.00
	Boring No.: BB-WPPB-103	Tested By: GSL	Checked By: --
	Sample No.: 1U	Test Date: 10/3/2019	Test No.: 336997
	Depth: 25.0-27.0 FT	Sample Type: Undisturbed	Elevation: --
	Description: Grey Clay		
	Remarks: Maine Sensitive Load/Unload/Reload/Unload Consolidation Test		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

Project: Winslow
Location: --
Project No.: 22268.00
Boring No.: BB-WPPB-103
Tested By: GSL
Checked By: --
Sample No.: 1U
Test Date: 10/3/2019
Depth: 25.0-27.0 FT
Test No.: 336997
Sample Type: Undisturbed
Elevation: --

Soil Description: Grey Clay
Remarks: Maine Sensitive Load/Unload/Reload/Unload Consolidation Test

Measured Specific Gravity: 2.75
Initial Void Ratio: 1.20
Final Void Ratio: 0.824
Liquid Limit: 38
Plastic Limit: 24
Plasticity Index: 14
Specimen Diameter: 2.50 in
Initial Height: 1.00 in
Final Height: 0.83 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	41	RING	RING	129
Wt. Container + Wet Soil, gm	101.17	406.40	392.39	191.68
Wt. Container + Dry Soil, gm	89.820	362.36	362.36	161.70
Wt. Container, gm	63.270	262.18	262.18	61.680
Wt. Dry Soil, gm	26.550	100.18	100.18	100.02
Water Content, %	42.75	43.96	29.97	29.97
Void Ratio	---	1.20	0.824	---
Degree of Saturation, %	---	101.02	100.00	---
Dry Unit Weight, pcf	---	78.155	94.106	---

One-Dimensional Consolidation by ASTM D2435 - Method B

Project: Winslow
Boring No.: BB-WPPB-103
Sample No.: 1U
Test No.: 336997

Location: --
Tested By: GSL
Test Date: 10/3/2019
Sample Type: Undisturbed

Project No.: 22268.00
Checked By: --
Depth: 25.0-27.0 FT
Elevation: --

Soil Description: Grey Clay
Remarks: Maine Sensitive Load/Unload/Reload/Unload Consolidation Test

Displacement at End of Increment

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Sq.Rt T90 min	Cv ft²/sec	Mv 1/tsf	k ft/day
1	0.263	0.01368	1.17	1.37	14.663	1.65e-006	5.20e-002	2.31e-004
2	0.500	0.02089	1.15	2.09	2.709	8.73e-006	3.05e-002	7.19e-004
3	1.00	0.03780	1.11	3.78	13.412	1.72e-006	3.39e-002	1.57e-004
4	2.00	0.06983	1.04	6.99	15.367	1.43e-006	3.21e-002	1.23e-004
5	4.00	0.1366	0.896	13.7	16.209	1.21e-006	3.34e-002	1.10e-004
6	8.00	0.1902	0.778	19.0	7.554	2.27e-006	1.34e-002	8.19e-005
7	4.00	0.1842	0.791	18.4	55.823	2.90e-007	1.48e-003	1.16e-006
8	2.00	0.1788	0.803	17.9	1.654	9.91e-006	2.72e-003	7.28e-005
9	1.00	0.1730	0.816	17.3	2.660	6.25e-006	5.82e-003	9.81e-005
10	0.500	0.1670	0.829	16.7	14.889	1.13e-006	1.20e-002	3.65e-005
11	1.00	0.1711	0.820	17.1	27.728	6.09e-007	8.19e-003	1.35e-005
12	2.00	0.1763	0.809	17.7	2.549	6.55e-006	5.23e-003	9.24e-005
13	4.00	0.1835	0.793	18.4	1.403	1.17e-005	3.61e-003	1.14e-004
14	8.00	0.1993	0.758	20.0	3.599	4.44e-006	3.95e-003	4.73e-005
15	16.0	0.2402	0.668	24.1	4.302	3.46e-006	5.12e-003	4.78e-005
16	4.00	0.2260	0.700	22.6	0.760	1.89e-005	1.19e-003	6.07e-005
17	1.00	0.2116	0.731	21.2	3.369	4.43e-006	4.78e-003	5.71e-005
18	0.250	0.1981	0.761	19.8	24.565	6.30e-007	1.80e-002	3.06e-005
19	0.0625	0.1856	0.788	18.6	78.392	2.04e-007	6.70e-002	3.68e-005

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Log T50 min	Cv ft²/sec	Mv 1/tsf	k ft/day	Ca %
1	0.263	0.01368	1.17	1.37	0.000	0.00e+000	5.20e-002	0.00e+000	0.00e+000
2	0.500	0.02089	1.15	2.09	0.000	0.00e+000	3.05e-002	0.00e+000	0.00e+000
3	1.00	0.03780	1.11	3.78	0.000	0.00e+000	3.39e-002	0.00e+000	0.00e+000
4	2.00	0.06983	1.04	6.99	0.000	0.00e+000	3.21e-002	0.00e+000	0.00e+000
5	4.00	0.1366	0.896	13.7	4.406	1.04e-006	3.34e-002	9.36e-005	0.00e+000
6	8.00	0.1902	0.778	19.0	1.964	2.03e-006	1.34e-002	7.32e-005	0.00e+000
7	4.00	0.1842	0.791	18.4	0.000	0.00e+000	1.48e-003	0.00e+000	0.00e+000
8	2.00	0.1788	0.803	17.9	0.362	1.05e-005	2.72e-003	7.73e-005	0.00e+000
9	1.00	0.1730	0.816	17.3	0.938	4.12e-006	5.82e-003	6.46e-005	0.00e+000
10	0.500	0.1670	0.829	16.7	0.000	0.00e+000	1.20e-002	0.00e+000	0.00e+000
11	1.00	0.1711	0.820	17.1	0.000	0.00e+000	8.19e-003	0.00e+000	0.00e+000
12	2.00	0.1763	0.809	17.7	0.481	8.07e-006	5.23e-003	1.14e-004	0.00e+000
13	4.00	0.1835	0.793	18.4	0.330	1.16e-005	3.61e-003	1.13e-004	0.00e+000
14	8.00	0.1993	0.758	20.0	0.600	6.19e-006	3.95e-003	6.60e-005	0.00e+000
15	16.0	0.2402	0.668	24.1	1.118	3.09e-006	5.12e-003	4.27e-005	0.00e+000
16	4.00	0.2260	0.700	22.6	0.000	0.00e+000	1.19e-003	0.00e+000	0.00e+000
17	1.00	0.2116	0.731	21.2	0.980	3.54e-006	4.78e-003	4.56e-005	0.00e+000
18	0.250	0.1981	0.761	19.8	0.000	0.00e+000	1.80e-002	0.00e+000	0.00e+000
19	0.0625	0.1856	0.788	18.6	15.250	2.43e-007	6.70e-002	4.40e-005	0.00e+000

Appendix D

Calculations

Liquidity Index

Liquidity Index

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

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

BB-WPPB-101, 1D

$WC := 25$

$LL := 38$

$PL := 22$

$$LI := \frac{WC - PL}{LL - PL} = 0.19$$

BB-WPPB-101, 3D

$WC := 42.4$

$LL := 50$

$PL := 24$

$$LI := \frac{WC - PL}{LL - PL} = 0.71$$

BB-WPPB-101, 4D

$WC := 35.7$

$LL := 33$

$PL := 23$

$$LI := \frac{WC - PL}{LL - PL} = 1.27$$

BB-WPPB-101, 3U

$WC := 35.5$

$LL := 40$

$PL := 24$

$$LI := \frac{WC - PL}{LL - PL} = 0.72$$

BB-WPPB-103, 3D

$WC := 37.7$

$LL := 36$

$PL := 23$

$$LI := \frac{WC - PL}{LL - PL} = 1.13$$

BB-WPPB-103, 1U

$WC := 39.9$

$LL := 38$

$PL := 24$

$$LI := \frac{WC - PL}{LL - PL} = 1.14$$

Sensitivity

BB-WPPB-101/V1

$Su := 1295psf$

$Su_{re} := 246psf$

$\frac{Su}{Su_{re}} = 5.26$

BB-WPPB-101/V2

$Su := 1094psf$

$Su_{re} := 223psf$

$\frac{Su}{Su_{re}} = 4.91$

BB-WPPB-101/V3

$Su := 692psf$

$Su_{re} := 277psf$

$\frac{Su}{Su_{re}} = 2.5$

BB-WPPB-101/V4

$Su := 603psf$

$Su_{re} := 156psf$

$\frac{Su}{Su_{re}} = 3.87$

BB-WPPB-101/V5

$Su := 625psf$

$Su_{re} := 134psf$

$\frac{Su}{Su_{re}} = 4.66$

BB-WPPB-101/V6

$Su := 580psf$

$Su_{re} := 179psf$

$$\frac{Su}{Su_{re}} = 3.24$$

BB-WPPB-101/V7

$Su := 603psf$

$Su_{re} := 179psf$

$$\frac{Su}{Su_{re}} = 3.37$$

BB-WPPB-101/V8

$Su := 714psf$

$Su_{re} := 134psf$

$$\frac{Su}{Su_{re}} = 5.33$$

BB-WPPB-101/V9

$Su := 714psf$

$Su_{re} := 179psf$

$$\frac{Su}{Su_{re}} = 3.99$$

Sensitivity

BB-WPPB-103/V1

$Su := 491psf$

$Su_{re} := 201psf$

$$\frac{Su}{Su_{re}} = 2.44$$

BB-WPPB-103/V2

$Su := 536psf$

$Su_{re} := 134psf$

$$\frac{Su}{Su_{re}} = 4$$

BB-WPPB-103/V3

$Su := 759psf$

$Su_{re} := 233psf$

$$\frac{Su}{Su_{re}} = 3.26$$

Driven H-Pile Resistance

Design of H-piles

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

Bedrock Properties

Rock Type: Phyllite

ϕ = 20-34 (AASHTO LRFD Table C.10.4.6.4-1, between slate, 27-34, and schist, 20-27);
Phyllite Co = 3,500 - 35,000 psi (AASHTO Standard Specifications for Bridges 17th Edition, Table 4.4.8.1.2B)

For Design Purposes, use bedrock data from BB-WPPB-101 R1: RQD = 16% and an Unconfined Compressive Strength of 3,500 psi based on the lower bound of Phyllite Co.

Pile Properties

Use the following piles: 14x89, 14x117

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

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

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

r_s= radius of gyration $\qquad r_s := \begin{pmatrix} 3.53 \\ 3.59 \end{pmatrix} \cdot in$

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

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

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

Use LRFD Equation 6.9.2.1-1 $\quad Pr = \phi\, Pn$

Nominal Axial Structural Resistance

Determine equivalent yield resistance $P_o = F_y A_s$ (LRFD 6.9.4.1.1)

$$P_o := F_y \cdot A_s \qquad \boxed{P_o = \begin{pmatrix} 1305 \\ 1720 \end{pmatrix} \cdot kip}$$

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

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

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

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

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

LRFD eq. 6.9.4.1.2-1

$$P_e := \overrightarrow{\left[\frac{\pi^2 \cdot E}{\left(\frac{K_{eff} \cdot l_{unbraced\_bot}}{r_s} \right)^2} \cdot A_s \right]} \qquad P_e = \begin{pmatrix} 2 \times 10^8 \\ 2 \times 10^8 \end{pmatrix} \cdot kip$$

LRFD Article 6.9.4.1.1

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

If Pe/Po > or = 0.44, then:

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

LRFD Eq. 6.9.4.1.1-1

then:

this applies to all pile sizes $\quad \boxed{P_n = \begin{pmatrix} 1305 \\ 1720 \end{pmatrix} \cdot kip}$

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

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

LRFD 10.7.3.2.3 - Piles Driven to Hard Rock -

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

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

Nominal Structural Resistance Previously Calculated:

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

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

$$\phi_c := 0.5$$

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

$$\boxed{P_r = \begin{pmatrix} 653 \\ 860 \end{pmatrix} \cdot kip}$$

14x89
14x117

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

$$\phi_c := 1.0$$

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

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

14x89
14x117

Drivability Analyses

Ref: LRFD Article 10.7.8

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

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

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

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

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

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

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

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

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

GRLWeap Soil and Pile Model Assumptions

Based on proposed bottom of footing of elevations of approximately 54.9 at abutment 1 and 53.6 at abutment 2, estimated pile lengths will be approx. 31 ft at abutment 1 and 25 feet at abutment 2. Sensitivity analysis shows abutment 1 governs. Therefore, assume 31 ft of pile embedment. Assume contractor drives pile lengths of 40 ft (extra length accommodates for attachment of dynamic testing equipment, embedment into abutment, variation in bedrock surface).

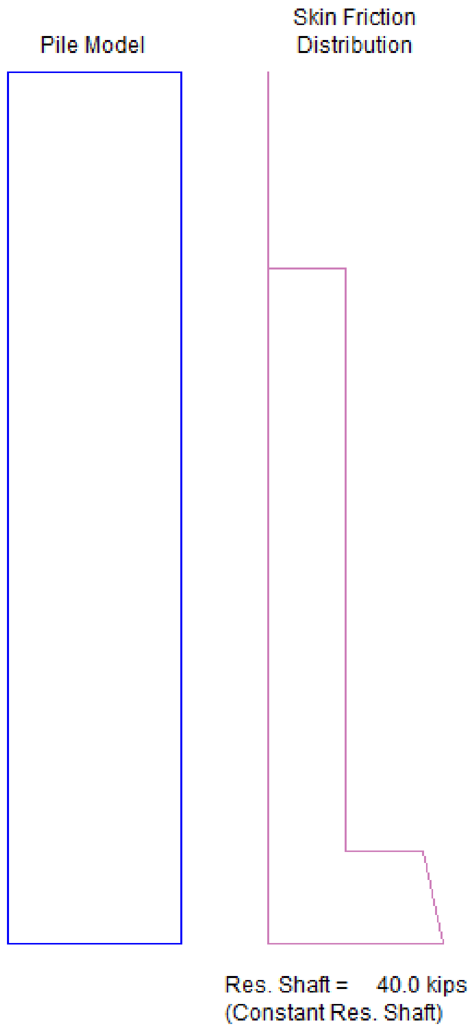
Use constant shaft resistances so that GRLWeap will assign approx. 40 kips as skin friction.

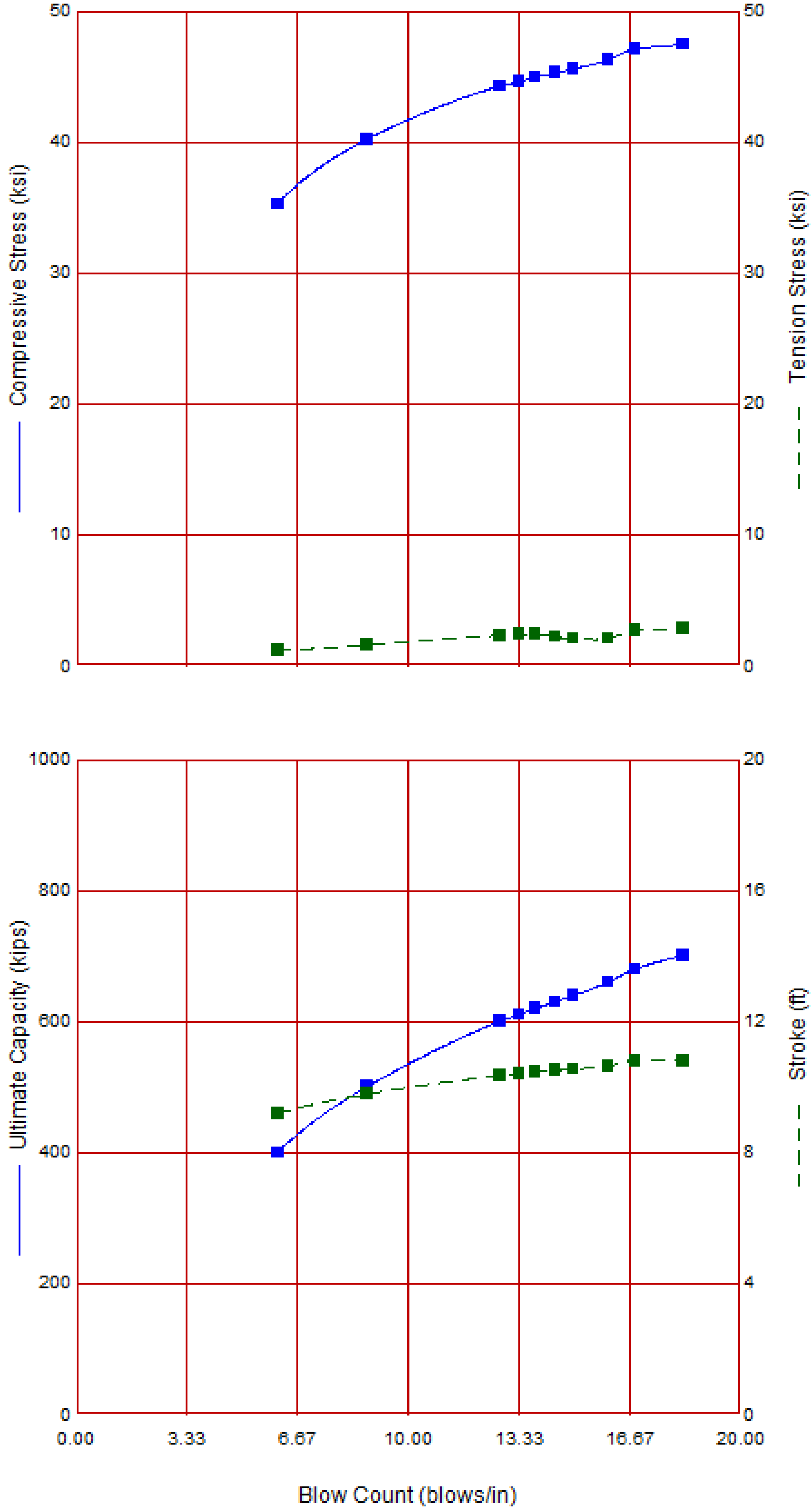
Pile Size is 14 x 89

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

DELMAG D 19-42

Ram Weight	4.00	kips
Efficiency	0.800	
Pressure	1600 (100%)	psi
Helmet Weight	1.90	kips
Hammer Cushion	60155	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.070	in
Skin Damping	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	40.00	ft
Pile Penetration	31.00	ft
Pile Top Area	26.10	in2




Compressive Stress (ksi)
Tension Stress (ksi)
50
40
30
20
10
0
Ultimate Capacity (kips)
Stroke (ft)
1000
800
600
400
200
0
20
16
12
8
4
0.00
3.33
6.67
10.00
13.33
16.67
20.00
Blow Count (blows/in)

Maine DOT
Winslow Fish Bridge Abutment 1 14x89

09-May-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	35.26	1.20	6.1	9.19	18.35
500.0	40.19	1.59	8.8	9.78	19.77
600.0	44.26	2.34	12.8	10.35	21.11
610.0	44.58	2.40	13.3	10.40	21.20
620.0	44.96	2.43	13.8	10.45	21.36
630.0	45.27	2.23	14.4	10.50	21.47
640.0	45.59	2.07	15.0	10.55	21.55
660.0	46.26	2.11	16.0	10.64	21.75
680.0	47.10	2.74	16.8	10.81	22.17
700.0	47.42	2.88	18.3	10.81	22.22

Limit to 45 ksi

$R_{ndr} := 620 \cdot kip$

Strength Limit State

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

$\boxed{R_{fdr} = 403 \cdot kip}$

Extreme and
Service Limit States

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

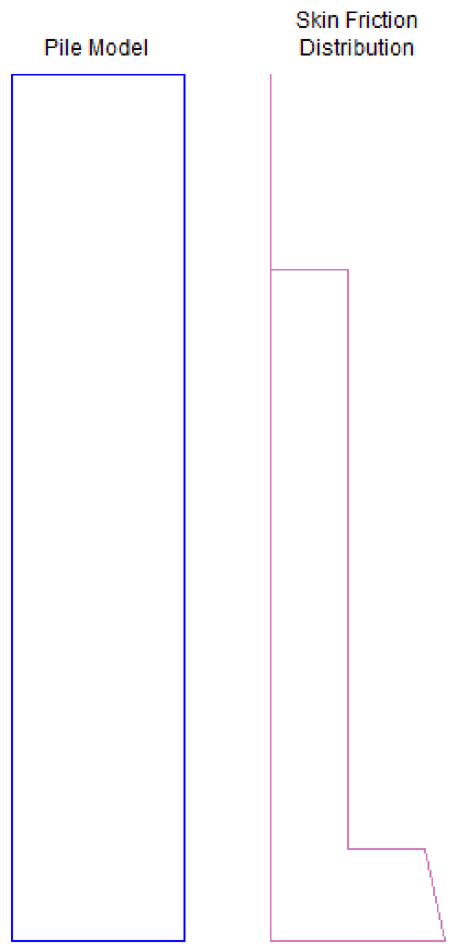
$\boxed{R_{dr} = 620 \cdot kip}$

Pile Size is 14 x 89

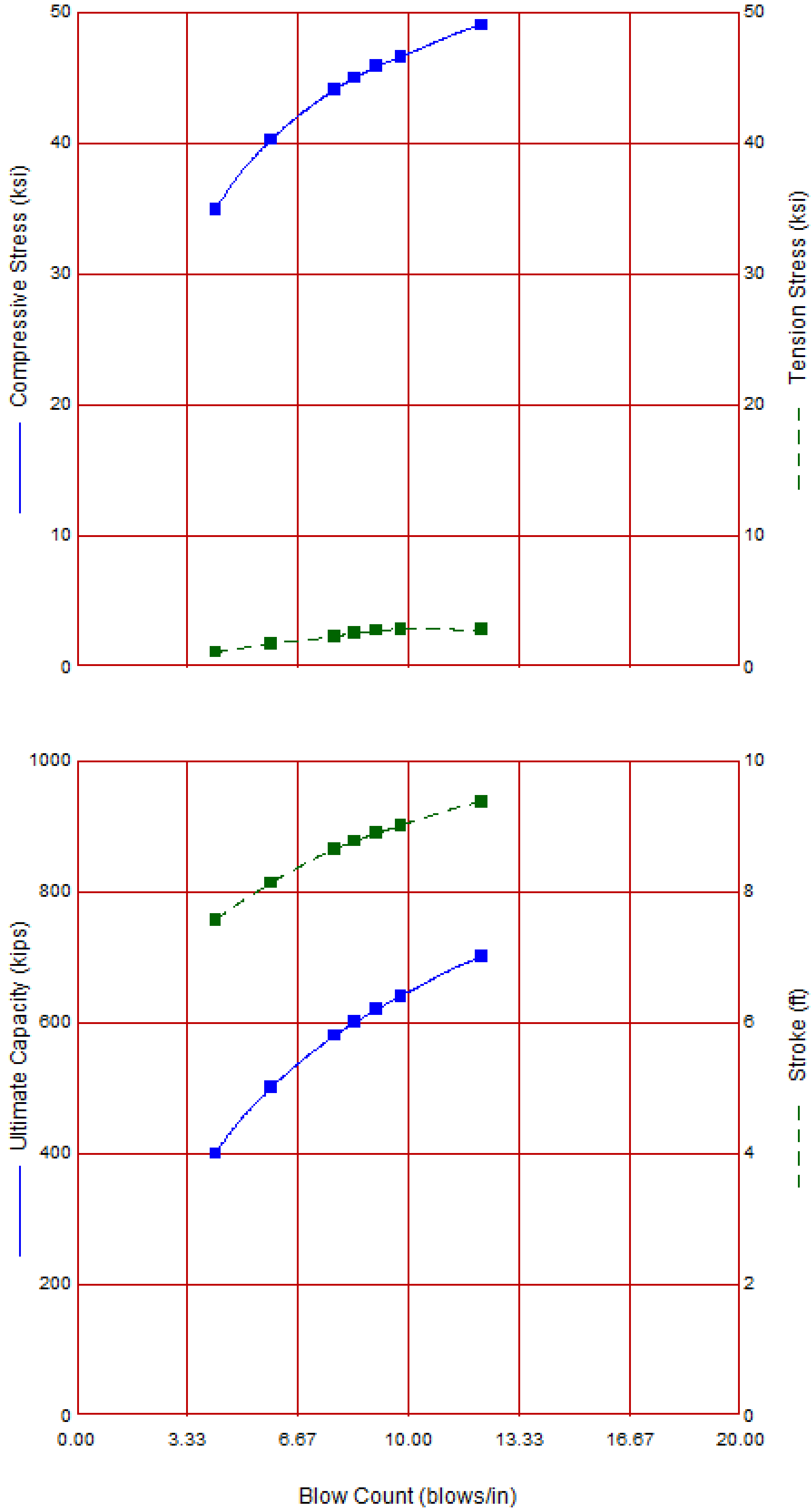
The 14x89 pile can be driven to the resistances below with a D 25-52 hammer at fuel setting 3 (81% of Max) and 1.9 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

DELMAG D 25-52

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1215 (81%)	psi
Helmet Weight	1.90	kips
Hammer Cushion	60155	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.070	in
Skin Damping	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	40.00	ft
Pile Penetration	31.00	ft
Pile Top Area	26.10	in2



Res. Shaft = 40.0 kips
(Constant Res. Shaft)


Compressive Stress (ksi)
Tension Stress (ksi)
50
40
30
20
10
0
Ultimate Capacity (kips)
Stroke (ft)
1000
800
600
400
200
0
10
8
6
4
2
0.00
3.33
6.67
10.00
13.33
16.67
20.00
Blow Count (blows/in)

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	34.90	1.11	4.2	7.56	21.59
500.0	40.22	1.75	5.9	8.13	23.09
580.0	44.07	2.32	7.8	8.65	24.76
600.0	44.96	2.58	8.4	8.77	25.18
620.0	45.82	2.76	9.0	8.89	25.60
640.0	46.55	2.89	9.8	9.01	26.01
700.0	48.98	2.83	12.2	9.37	27.23

Limit to 45 ksi

$R_{ndr} := 600 \cdot kip$

Strength Limit State

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

$\boxed{R_{fdr} = 390 \cdot kip}$

Extreme and
Service Limit States

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

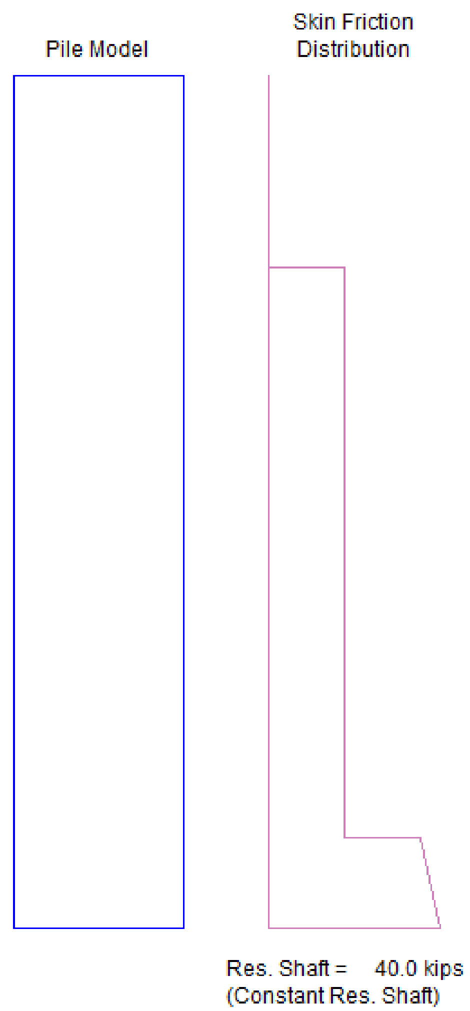
$\boxed{R_{dr} = 600 \cdot kip}$

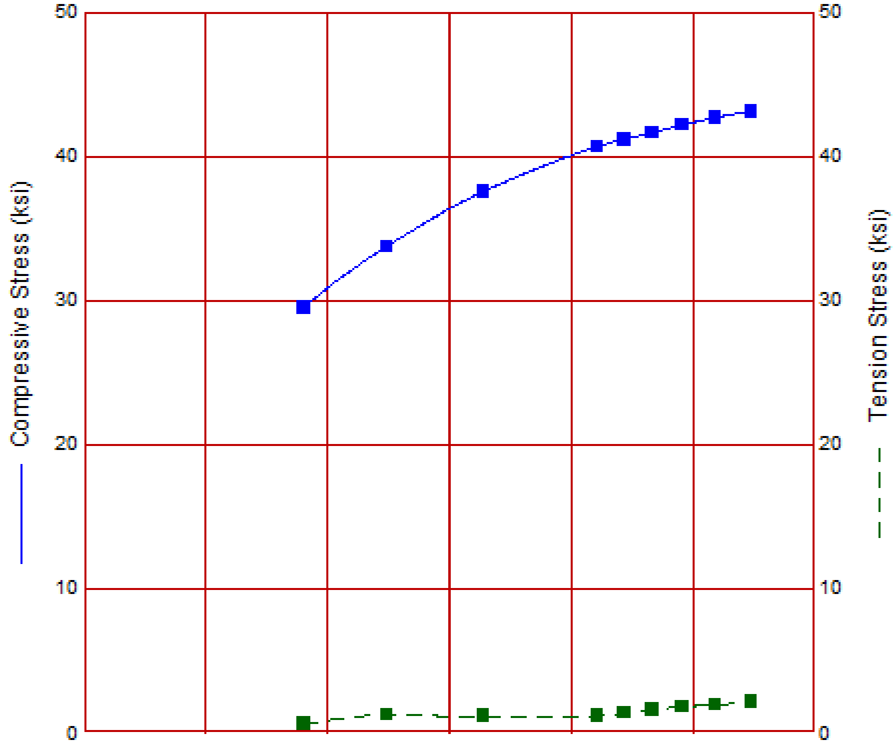
Pile Size is 14 x 117

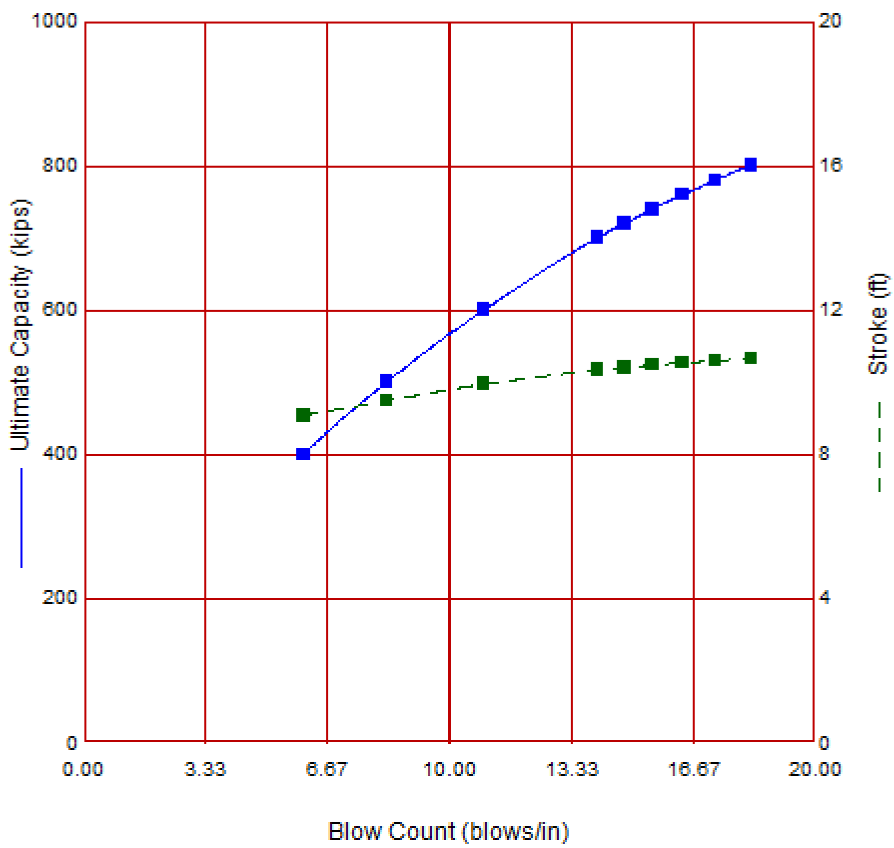
The 14x117 pile can be driven to the resistances below with a D 19-42 hammer at fuel setting 1 (100% of Max) and 1.9 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

DELMAG D 19-42

Ram Weight	4.00	kips
Efficiency	0.800	
Pressure	1600 (100%)	psi
Helmet Weight	1.90	kips
Hammer Cushion	60155	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.070	in
Skin Damping	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	40.00	ft
Pile Penetration	31.00	ft
Pile Top Area	34.40	in2




50
40
30
20
10
0
Compressive Stress (ksi)
50
40
30
20
10
0
Tension Stress (ksi)


1000
800
600
400
200
0
Ultimate Capacity (kips)
20
16
12
8
4
0
Stroke (ft)
0.00
3.33
6.67
10.00
13.33
16.67
20.00
Blow Count (blows/in)

Maine DOT 09-May-2023
Winslow Fish Bridge 14x117 D19-42 GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	29.46	0.62	6.0	9.09	17.63
500.0	33.73	1.24	8.3	9.49	18.49
600.0	37.55	1.17	10.9	9.95	19.58
700.0	40.66	1.18	14.0	10.34	20.45
720.0	41.18	1.38	14.8	10.41	20.62
740.0	41.63	1.59	15.5	10.48	20.80
760.0	42.18	1.84	16.4	10.54	20.94
780.0	42.69	1.95	17.3	10.60	21.07
800.0	43.10	2.13	18.2	10.66	21.20

Limit to 15 bpi

$$R_{ndr} := 720 \cdot kip$$

Strength Limit State

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

$$\boxed{R_{fdr} = 468 \cdot kip}$$

Extreme and
Service Limit States

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

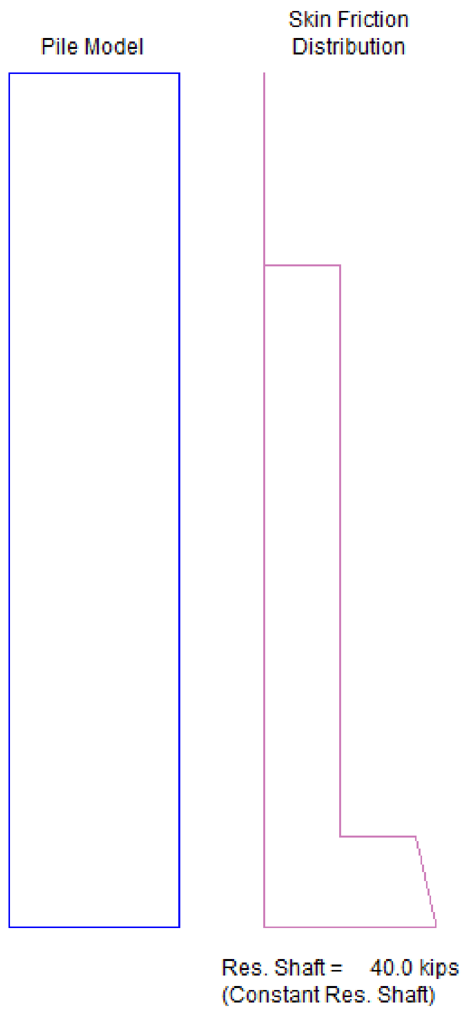
$$\boxed{R_{dr} = 720 \cdot kip}$$

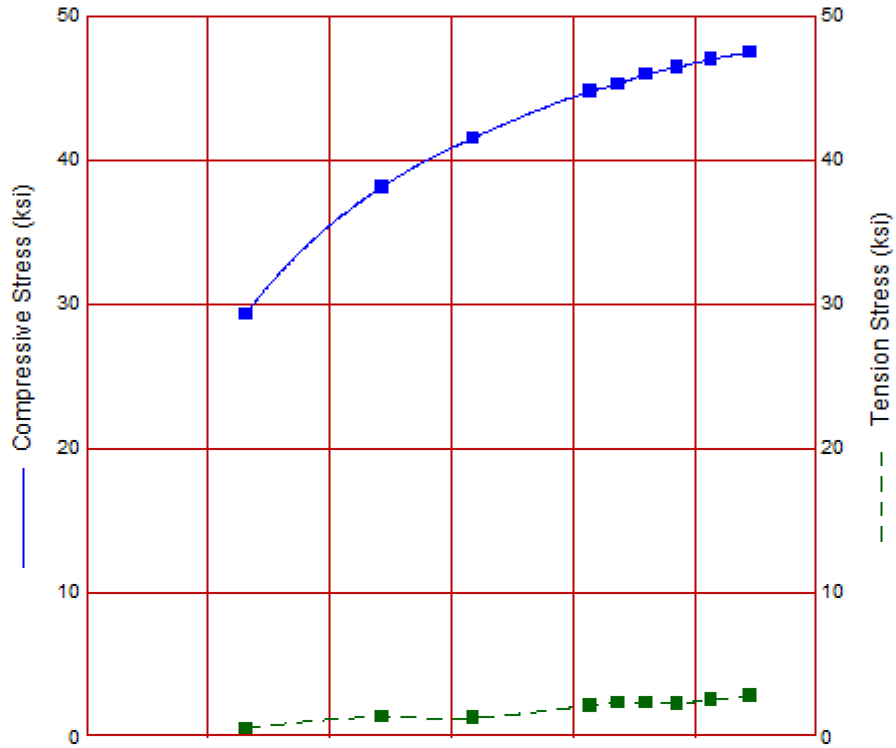
Pile Size is 14 x 117

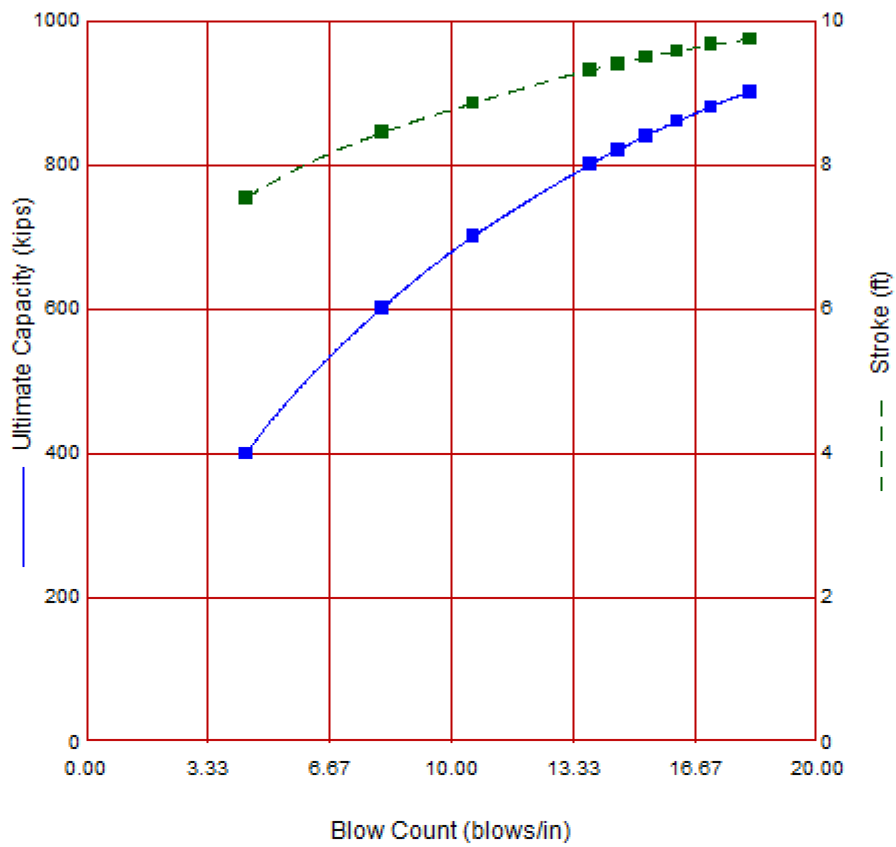
The 14x117 pile can be driven to the resistances below with a D 25-52 hammer at fuel setting 3 (81% of Max) and 1.9 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

DELMAG D 25-52

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1215 (81%)	psi
Helmet Weight	1.90	kips
Hammer Cushion	60155	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.070	in
Skin Damping	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	40.00	ft
Pile Penetration	31.00	ft
Pile Top Area	34.40	in2




50
40
30
20
10
0
Compressive Stress (ksi)
Tension Stress (ksi)


1000
800
600
400
200
0
10
8
6
4
2
Ultimate Capacity (kips)
Stroke (ft)
0.00
3.33
6.67
10.00
13.33
16.67
20.00
Blow Count (blows/in)

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	29.30	0.59	4.4	7.54	20.54
600.0	38.09	1.40	8.1	8.44	23.23
700.0	41.51	1.29	10.6	8.85	24.58
800.0	44.74	2.18	13.8	9.31	26.04
820.0	45.24	2.37	14.5	9.39	26.33
840.0	45.91	2.35	15.3	9.49	26.67
860.0	46.42	2.34	16.2	9.57	26.96
880.0	46.96	2.56	17.1	9.66	27.24
900.0	47.44	2.84	18.2	9.74	27.46

Limit to 45 ksi

$$R_{ndr} := 800 \cdot kip$$

Strength Limit State

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

$$\boxed{R_{fdr} = 520 \cdot kip}$$

Extreme and
Service Limit States

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

$$\boxed{R_{dr} = 800 \cdot kip}$$

Earth Pressure

Earth Pressure:

Backfill engineering strength parameters

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

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

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

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

Integral Abutment - Passive Earth Pressure - Coulomb Theory

α = Angle of fill slope to the horizontal $\alpha := 1 \cdot \text{deg}$

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

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

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

For formed concrete IAB abutment against clean sand, silty sand-gravel mixture use δ = 17, 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

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

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

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

$\boxed{K_{p\_coulomb} = 6.34}$

Integral Abutment and Wingwall - Passive Earth Pressure - Rankine Theory

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

α = Angle of fill slope to the horizontal $\alpha := 1 \cdot \text{deg}$

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

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

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

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

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

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

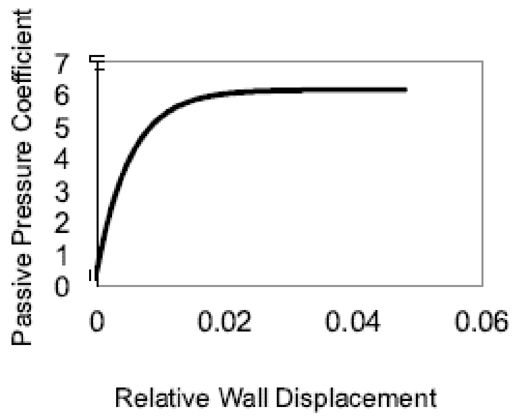


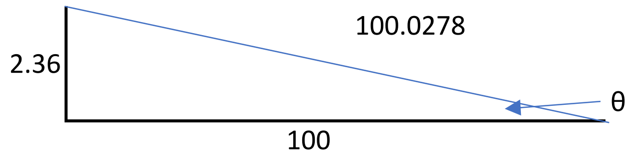
Figure 3.10.8-1: Plot of Passive Pressure Coefficient, K, vs. Relative Wall Displacement, δ_T/H.

Based on an estimated Relative Wall Displacement of 0.5"/102"=0.005:

$K := 0.43 + 5.7\cdot[1 - \exp[-190(0.005)]]$

$\boxed{K = 3.93}$

2.36 % slope



tan (θ)=2.36/100

θ = arctan (2.36/100)

θ =	0.023596	rad
	1.351929	degrees

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

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

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

For cohesive soils, passive pressures may be estimated by:

C3.11.5.4

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

Table 7.9 (*Continued*)

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

7.12 Coulomb's Passive Earth Pressure

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

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

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

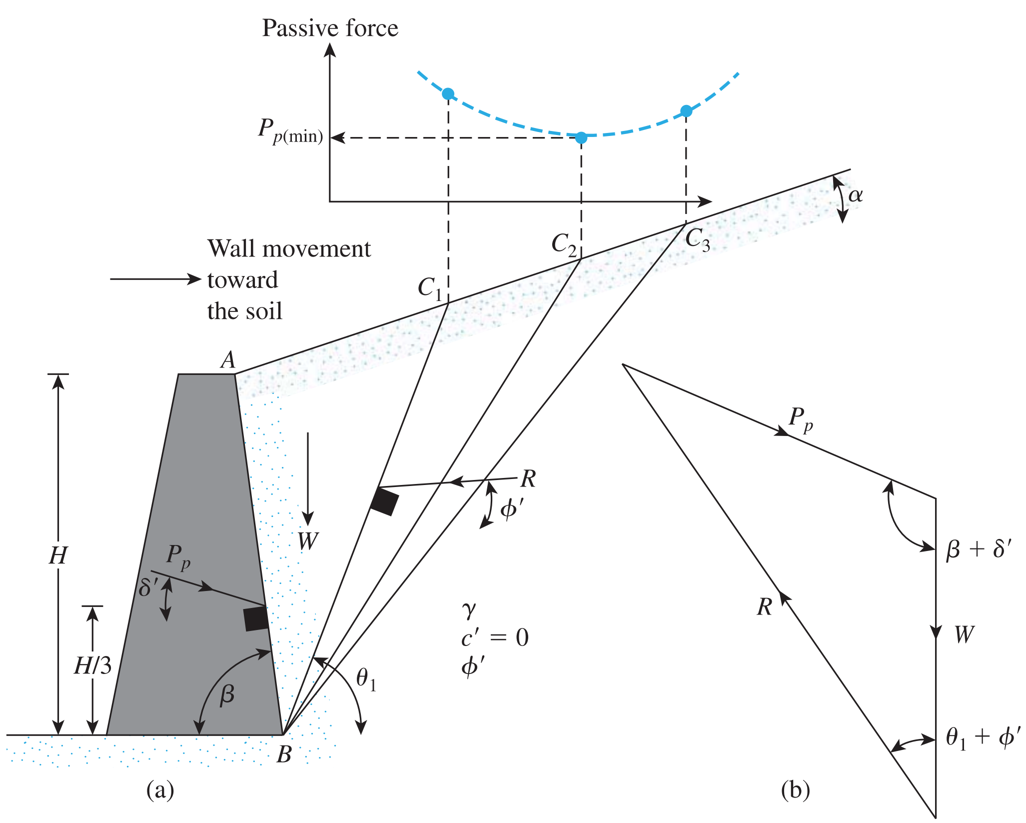


Figure 7.25 Coulomb's passive pressure

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

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

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

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

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

where

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

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

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

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

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

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

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

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

Again, at $z = 3$ m,

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

Hence,

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

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

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

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

■

7.11 Rankine Passive Earth Pressure: Vertical Backface and Inclined Backfill

Granular Soil

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

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

and the passive force is

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

where

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

using compacted gravel borrow backfill shall be estimated using the equation:

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

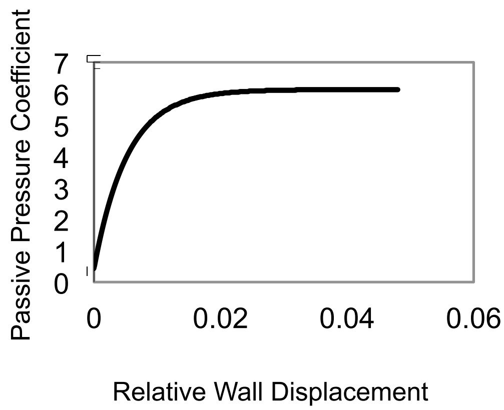


Figure 3.10.8-1: Plot of Passive Pressure Coefficient, K, vs. Relative Wall Displacement, δ_T/H.

The simplified approach may be used to calculate moments and shears in the abutment walls, assuming the abutment wall acting as a simple span between piles and then taking 80% of simple span moments to account for continuity. Shears may be taken equal to simple span shears. Due to the relatively large dimensions of the abutment walls, minimum reinforcement is usually sufficient to satisfy the strength requirements.

The longitudinal reinforcement of the pile cap shown in Chapter 12 of Part II of this Bridge Manual represents an upper–bound for the required reinforcement assuming the girders are located at the positions that produce maximum effects on the pile cap and assuming a conservative value of other dead loads on the abutment wall.

Stirrups intended to resist horizontal shear forces acting on the pile cap due to soil passive pressure shall be provided as shown in Part II of this Bridge Manual.

L-shaped connection reinforcing bars indicated in the standard drawings of Chapter 12 of Part II and Chapter 2 of Part III of this Bridge Manual shall be provided to transfer the maximum expected connection moment between the abutment and the superstructure. These bars shall be #6 @ 9” for girders up to 8 feet deep. For deeper girders they shall be designed. The vertical leg of the connection bars shall be placed as close as practical to the back face of the abutment. The horizontal leg shall be extended into the deck beyond the inside face of the abutment diaphragm at the elevation of the deck top longitudinal reinforcement for a length equal to 10% of the span plus the development length, for simple span bridges. For continuous span bridges the bars shall be extended to 10% of the end span plus the development length.

Refer to Chapter 12 of Part II and Chapter 2 of Part III of this Manual for more information on the integral abutment reinforcement.

Frost Depth

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

From Design Freezing Index Map: **Winslow, Maine**
DFI = 1600 degree-days.
Case 1 - fine grained fill soils W=25% (BB-WPPB-101, 1D).
Case 2 - coarse grained granular fill soils W=20% (BB-WPPB-103, 1D)

Depth of Frost Penetration - Case 1

For DFI = 1600

at w=20% $d_1 := 51.9in$

at w=30% $d_2 := 46.9in$

Depth of Frost Penetration - Case 2

$$d := \frac{d_2 + d_1}{2} \qquad d = 49.4 \cdot in \qquad d = 4.1 \cdot ft$$

For DFI = 1600

at w=20% $d_3 := 70.2in$ $d_3 = 5.85\ ft$

Recommend Depth of Frost Penetration - Case 2

$d_3 = 70.2 \cdot in$ $d_3 = 5.9 \cdot ft$

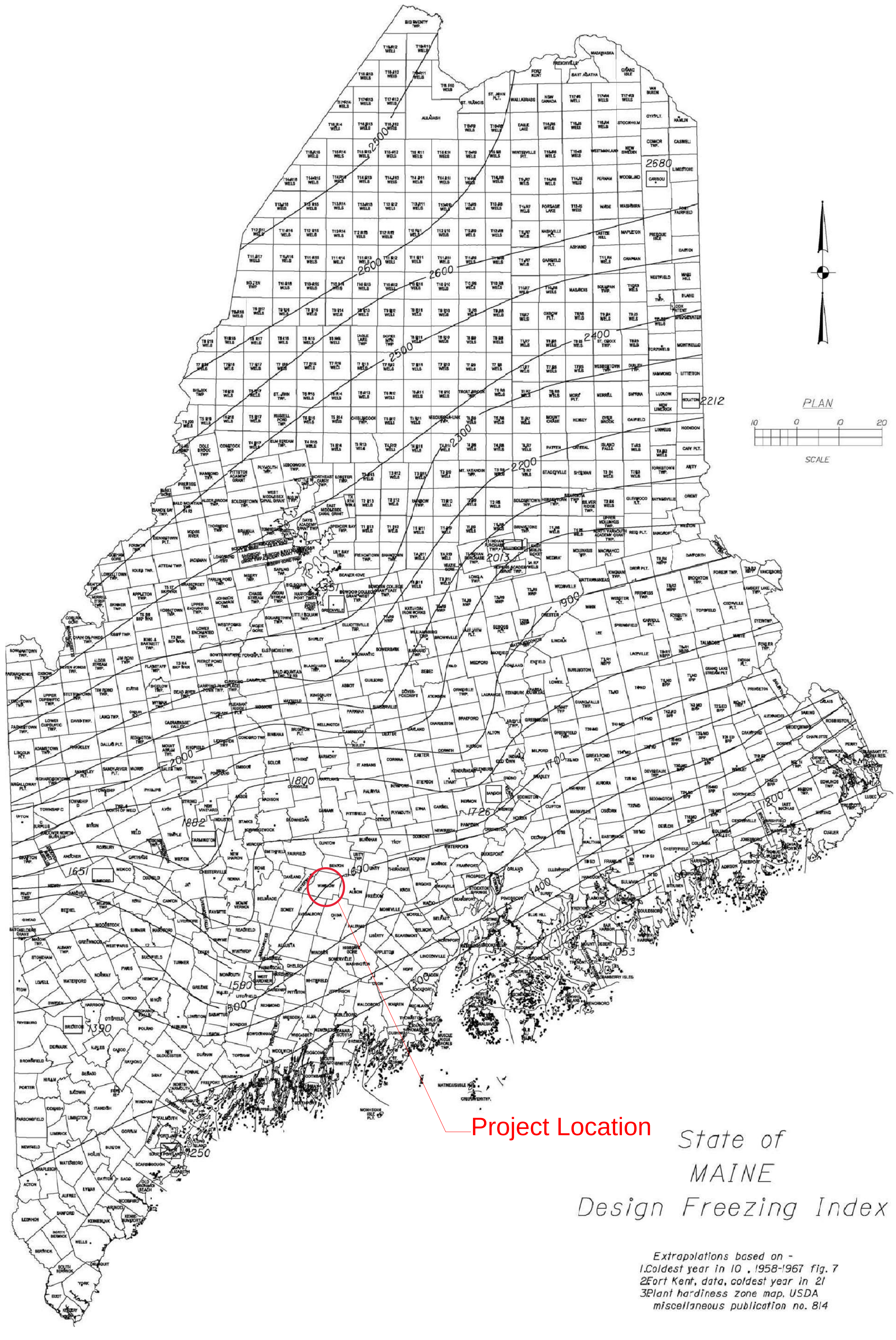


Figure 5-1 Maine Design Freezing Index Map

5.2 General

MaineDOT Bridge Design Guide

5.2.1 Frost

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

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

Table 5-1 Depth of Frost Penetration

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

Seismic Parameters

BB-WPPB-101			
Depth	**N_{60}**	**di**	**di/N**
5	13	11	0.85
25	1	25	25.00
35	28	4	0.14
39.8	100	60.0	0.60
SUM		**100**	**26.59**
		di/di/N	**3.76**

BB-WPPB-102, 103			
Depth	**N_{60}**	**di**	**di/N**
5	15	14	0.93
10	9	4	0.44
15	1	11	11.00
30	24	4	0.17
33	100	67	0.67
SUM		**100**	**13.21**
		di/di/N	**7.57**

SUM	**Nav.**	**5.66**

N_{av}. < 15 bpf

Conclusion: Site Class E

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

Winslow Fish Bridge #0509
WIN 22268.00
June 6, 2023

Seismic Parameters

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 44.569927
Longitude = -069.563583
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.075	PGA - Site Class B
0.2	0.157	Ss - Site Class B
1.0	0.046	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
Latitude = 44.569927
Longitude = -069.563583
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class E - Fpga = 2.50, Fa = 2.50, Fv = 3.50
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.187	As - Site Class E
0.2	0.394	SDs - Site Class E
1.0	0.160	SD1 - Site Class E