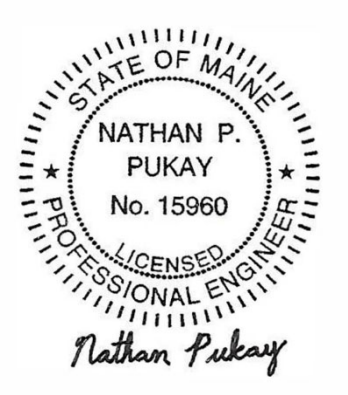
MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

KINGMAN ROAD BRIDGE
STATE ROUTE 170 OVER MACWAHOC STREAM
MACWAHOC PLANTATION, 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 Kingman Road Bridge which carries State Route 170 over Macwahoc Stream in Macwahoc Plantation, 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 Kingman Road Bridge was constructed in 1919 and is a 35-foot, single-span, concrete tee beam bridge. The substructure consists of mass concrete abutments and wingwalls founded on spread footings bearing on soil. According to the 2021 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the FHWA Sufficiency Rating of the bridge is 39.7. The bridge is in poor condition due to widespread spauling and efflorescence across the concrete substructure and superstructure. The southeast wingwall has failed, and the top has broken off and fallen into the river. The bridge is also scour critical.

The proposed replacement structure consists of a 90-foot, single-span steel girder bridge founded on pile-supported integral abutments with cantilevered, in-line wingwalls. Piles will be driven to bedrock. 1.75H:1V (horizontal:vertical) riprap slopes will be constructed in front of the new integral abutments. A wildlife shelf will be built into the riprap at the north abutment.

The new bridge will be located on a horizontal alignment that will approximately match the existing. The vertical alignment will be raised approximately 8 inches at the north abutment and match the existing at the south abutment to improve the roadway geometry. The abutments will be skewed to align with the stream to improve hydraulics at the project site.

Traffic will be maintained on a temporary detour built on the downstream side of the existing bridge.

2.0 GEOLOGIC SETTING

Kingman Road Bridge carries State Route 170 (Kingman Road) over Macwahoc Stream as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of the Mattawamkeag Quadrangle, Maine, Open-File No. 81-43 (1981), indicates the surficial soils in the vicinity of the bridge project consist of stream alluvium, glacial stream deposits, and glacial till. Stream alluvium consists of sand, gravel, and silt deposited on flood plains and stream beds by postglacial streams. Glacial-stream deposits consist of sand and gravel deposited by meltwater streams and currents during melting of the Late Wisconsinan glaciar. Glacial till is a heterogeneous mixture of sand, silt, clay, and stones deposited by glacial ice.

The MGS Bedrock Geology of the Mattawamkeag Quadrangle, Maine, Open-File No. 81-46 (1981) maps the bedrock at the site as quartz-rich Graywacke of the Lawler Ridge Formation, with thin beds of grey to black Slate or Phyllite. The bedrock cored in the test borings drilled at the site consisted of Graywacke and Phyllite.

3.0 SUBSURFACE INVESTIGATION

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

Boring BB-MPMS-102 was drilled in December 2021 by the MaineDOT drill crew. The remaining 100-series borings were drilled in April 2022 by S.W. Cole Explorations. S.W. Cole Explorations returned to the project site in March 2023 and drilled the 200-series borings. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheets 4 and 5 – Boring Logs.

Borings were performed by using a combination of solid stem auger, cased wash boring and rock coring techniques. 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 rigs used in the subsurface investigation were equipped with automatic hammers to drive the split spoon. The hammers were calibrated per ASTM D 4633 “Standard Test Method for Energy Measurement for Dynamic Penetrometers” to establish hammer efficiency factors. All N-values discussed in this report are corrected N-values computed by applying the hammer efficiency factors. The hammer efficiency factors and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring logs.

Bedrock was cored in the borings using NQ-2” core barrels and the Rock Quality Designation (RQD) of the cores calculated. The MaineDOT geotechnical engineer selected the boring locations and drilling methods, designated type and depth of sampling techniques, identified field-testing requirements, and logged the subsurface conditions encountered in the borings. The borings were located in the field using taped measurements at the completion of the drilling 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 nine standard grain size analyses with natural water content and one grain size analysis with hydrometer and natural water content.

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 Sheets 4 and 5 – Boring Logs.

5.0 SUBSURFACE CONDITIONS

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

5.1 Fill

A layer of Fill was encountered in the test borings. The thickness of the fill unit encountered was approximately 11 to 15 feet. The fill materials encountered consisted of:

- Brown, SAND, some gravel, little silt;
- Brown, Sandy GRAVEL, little silt; and
- Brown, Gravelly SAND, little silt, trace organics.

Corrected SPT N-values in the fill layer ranged from 6 to greater than 50 blows per foot (bpf) indicating the fill is loose to very dense in consistency. Two grain size analyses performed on samples recovered from the fill unit indicated the material is classified as A-1-a and A-1-b under the AASHTO Soil Classification System and SM and GM under the Unified Soil Classification System (USCS). The natural water contents of the samples tested measured approximately 13 percent.

5.2 Glacial Stream Deposit

A Glacial Stream Deposit was encountered in the test borings beneath the fill soils. The encountered thickness was approximately 28 to 35 feet. The deposit was variable and consisted of:

- Brown to grey, SAND, trace to some gravel, trace to little silt;
- Grey, Gravelly SAND, trace silt;
- Brown to grey, Sandy GRAVEL, trace silt;
- Grey, GRAVEL, trace to little sand, trace silt;
- Cobbles;
- Wood.

Corrected SPT N-value within the Glacial Stream Deposit ranged from 14 to greater than 50 bpf, indicating the deposit is medium dense to very dense. Seven grain size analyses conducted on samples of the deposit indicated the material is classified as A-3, A-2-4, A-1-a, and A-1-b under the AASHTO Soil Classification System and SP-SM, SW-SM, SP, and GW-GM under the USCS. The natural water contents of the samples tested ranged from 11 to 22 percent.

5.3 Glacial Till

Glacial Till was encountered in the borings underlying the Glacial Stream Deposit. The thickness of the Glacial Till deposit encountered was approximately 6 to 42 feet. The Glacial Till varied from:

- Brown-grey, SAND, some gravel, little silt, trace clay;
- Brown-grey, SAND, some silt, little to some gravel;
- Grey, Gravelly SAND, trace silt;
- Grey, Sandy GRAVEL, little silt;
- Grey, GRAVEL, little sand, trace silt ;
- Grey, Silty GRAVEL (bedrock chips), little sand;
- Grey, SILT, some gravel (rock chips), trace sand;
- Grey, Sandy SILT, little to some gravel; and
- Cobbles and boulders.

Corrected SPT N-values within the Glacial Till all exceeded 50 bpf indicating the deposit is very dense in consistency. One grain size analysis performed on a sample recovered from the deposit resulted in the material being classified as A-1-b under the AASHTO Soil Classification System and SC-SM under the USCS. The natural water content of the sample tested measured approximately 10 percent.

5.4 Bedrock

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

Boring	Station	Offset (feet)	Approximate Depth to Bedrock (feet)	Approximate Elevation of Bedrock Surface (feet)	RQD (%) (R1, R2, R3, R4)
BB-MPMS-101A	9+45.1	16.5 Lt	57.3	257.0	100, 58, 42
BB-MPMS-102	10+41	7.0 Rt	89.0	226.3	20, 7
BB-MPMS-201	9+52.1	6.7 Rt	52.6	263.1	0, 0, 0, 44
BB-MPMS-202	10+38.1	5.5 Lt	59.3	256.1	67, 70

The bedrock of the site consisted of grey to dark grey, fine to medium-grained, GRAYWACKE, moderately hard, fresh to slightly weathered, joints dipping at moderate to vertical angles, highly fractured zones, and Grey, fine-grained PHYLLITE, moderately soft, fresh to moderately weathered, joints dipping at moderate to vertical angles, with highly fractured zones. The RQD of the bedrock cores ranged from 0 to 100 percent, corresponding to a rock quality of very poor to excellent.

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

5.5 Groundwater

Groundwater was measured at depths ranging from 9 to 11 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

Due to the depth of bedrock and the span length requirement, integral abutments founded on driven pile was the preferred substructure design, allowing for a jointless bridge. A new bridge alignment was considered, but was not pursued due to the extensive roadway work and property rights it would require. Preliminary design also investigated rehabilitation of the existing bridge, but the cost was estimated to be comparable to that of a full replacement.

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 Kingman Road Bridge replacement project.

7.1 Integral Abutment H-Piles

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

Piles may be HP 14x89 or 14x117 depending on the factored design axial loads and ability to resist lateral loads. H-piles shall be 50 ksi, Grade A572 steel. The piles shall be fitted with driving pile points conforming to MaineDOT Standard Specification 711.10 to protect pile tips and improve penetration into bedrock.

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

Abutment	Approximate Bottom Elevation of Proposed Abutment (feet)	Approximate Top of Bedrock Elevation (feet)	Estimated Pile Lengths[1] (feet)
Abutment No. 1 (BB-MPMS-101A, -201)	306.5	257.0	53
Abutment No. 2 - Right of Centerline (BB-MPMS-102)	305.5	226.3	83
Abutment No. 2 - Left of Centerline (BB-MPMS-202)	305.5	256.1	53

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.

[1] Estimated pile lengths include 3-foot embedment into the pile cap, (rounded up to foot increments).

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

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

7.1.1 Axial Pile Resistance – Strength Limit State

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

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

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 on the following page.

Strength Limit State Factored Axial Pile Resistance					
Pile Section	Structural Resistance[1] ϕ_c=0.50 (kips)	Controlling Geotechnical Resistance[2] ϕ_c=0.50 (kips)	Drivability Resistance[3] φ_{dyn} = 0.65 (kips)		Governing Axial Pile Resistance (kips)
HP 14 x 89	652	652	423[4]	455[5]	423[4]
HP 14 x 117	860	860	504[6]		504[6]

LRFD Article 10.7.3.2.3 states that the nominal axial compressive resistance of piles driven to hard rock is typically controlled by the structural resistance with a resistance factor for severe driving conditions applied. However, for the site conditions, the estimated factored axial pile resistances from the drivability analyses for the H-pile sections are less than the controlling factored axial compressive resistances. Local experience also supports the estimated factored resistances from the drivability analyses. Therefore, drivability controls and the recommended governing resistances for pile design are the resistances provided in the rightmost column "Governing Axial Pile Resistance (kips)" in the table.

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

7.1.2 Axial Pile Resistance – Service and Extreme Limit State

The design of H-piles at the service limit state shall consider tolerable transverse and longitudinal movement of the piles and pile group movements/stability. For the service limit state, resistance factors of ϕ = 1.0 should be used in accordance with LRFD Article 10.5.5.1. 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.

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

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

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

[4] Drivability resistance based on a APE D19-42 Pile Hammer at Fuel Setting 4, Abutment 2 85-ft pile controls.
[5] Drivability resistance based on a APE D25-42 Pile Hammer at Fuel Setting 4. Abutment 2 85-ft pile controls.
[6] Drivability resistance based on a APE D25-42 Pile Hammer at Fuel Setting 4, Abutment 2 85-ft pile controls, (Will require a note on the plans that a minimum 62,000 ft-lb hammer will be provided by the contractor).

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	650[4]	700[5]	650[4]
HP 14 x 117	1,720	1,720	775[6]		775[6]

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

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

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

[2] Based on guidance in LRFD Article 10.7.3.2.3., *Piles Driven to Hard Rock.* The nominal axial geotechnical resistance in the strength limit state was calculated using the guidance in LRFD Article 10.7.3.2.3 which states the nominal bearing resistance of piles driven to point bearing on hard rock shall not exceed the nominal structural resistance values obtained from LRFD Article 6.9.4.1

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

[4] Drivability resistance based on a APE D19-42 Pile Hammer at Fuel Setting 4, Abutment 2 85-ft pile controls.
[5] Drivability resistance based on a APE D25-42 Pile Hammer at Fuel Setting 4. Abutment 2 85-ft pile controls.
[6] Drivability resistance based on a APE D25-42 Pile Hammer at Fuel Setting 4, Abutment 2 85-ft pile controls, (Will require a note on the plans that a minimum 62,000 ft-lb hammer will be provided by the contractor).

7.1.3 Lateral Pile Resistance/Behavior

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

A series of lateral pile resistance analyses should be performed to evaluate pile behavior at the abutments using LPile, or similar, software. The designer should utilize the lateral pile analyses to evaluate the associated pile stresses, bending moments, and fixity due to factored pile head loads and displacements.

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

LPile Input Parameters Abutment No. 1						
Soil Layer	Soil/Rock Model	Top Elevation of Layer (ft)	Layer Thickness (ft)	γ_e[1] (pcf)	ϕ'[2] (deg)	k_s[3] (pci)
Granular Borrow	Reese Sand	316	10	125	32	90
Fill	Reese Sand	306	2	63	32	60
Glacial Stream Deposits	Reese Sand	304	35	63	35	60
Glacial Till	Reese Sand	269	12	83	40	125

LPile Input Parameters Abutment No. 2						
Soil Layer	Soil/Rock Model	Top Elevation of Layer (ft)	Layer Thickness (ft)	γ_e[1] (pcf)	ϕ'[2] (deg)	k_s[3] (pci)
Granular Borrow	Reese Sand	315	10	125	32	90
Fill	Reese Sand	305	5	68	32	100
Glacial Stream Deposits	Reese Sand	300	32	68	38	90
Glacial Till	Reese Sand	268	12 to 42	83	40	125

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

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 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 3.46 is recommended, assuming a ratio of thermal expansion to abutment height (δ/H) of 0.004 and a level backfill. In general, when the calculated ratio of lateral movement to wall height is less than or greater than 0.004, a passive earth pressure coefficient can be estimated using MassDOT Figure 3.10.8-1. This figure is reproduced in Appendix D – Calculations. A load factor for passive earth pressure is not specified in LRFD. For purposes of the integral abutment backwall reinforcing steel design, use a maximum load factor (γ_{EH}) of 1.50 to calculate factored passive earth pressures.

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

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

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

7.3 Abutment Sections

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

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

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

7.4 Settlement and Embankment Stability

The project calls for the vertical alignment of the new structure to be approximately 8-inches higher at Abutment No. 1 and match the existing at Abutment No. 2. The bridge approach embankments will be constructed using granular borrow placed over loose to dense granular fill, and medium dense to very dense, coarse-grained native soil deposits. Any loose soils encountered at the subgrade elevation shall be thoroughly compacted prior to backfill operations. With these provisions, any settlement at the proposed bridge approaches is anticipated to be small and immediate.

Conventional earth fill embankments constructed over the existing soils using MaineDOT Standard Specifications, with side slopes of 2H:1V or flatter, are anticipated to satisfy stability requirements. Slopes steeper than 2H:1V should be treated with riprap using MaineDOT standard details.

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, Macwahoc Plantation has a design freezing index (DFI) of approximately 2000 F-degree days. The anticipated coarse-grained fill soil was assigned a water content of 10%. These components correlate to a frost depth of 7.9 feet. It is recommended that any foundation bearing on soils be embedded 7.9 feet for frost protection.

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

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

7.6 Seismic Design Considerations

The United States Geological Survey Seismic Design CD (Version 2.1) provided with the 2014 LRFD Code (7th Edition), and LRFD Articles 3.10.3.1 and 3.10.6 were used to develop parameters for seismic design. Based on site coordinates, the software provided the recommended AASHTO Response Spectra for a 7 percent probability of exceedance in 75 years. These results are summarized in the table on the following page:

Parameter	Design Value
Peak Ground Acceleration (PGA)	0.067g
Acceleration Coefficient (A_S)	0.11g
S_{DS} (Period = 0.2 sec)	0.24g
S_{D1} (Period = 1.0 sec)	0.11g
Site Class	D
Seismic Zone	1

In conformance with LRFD Table 4.7.4.3-1 seismic analysis is not required for single-span bridges regardless of seismic zone. However, superstructure connections and minimum support length requirements shall be designed per LRFD Articles 3.10.9.2 and 4.7.4.4, respectively.

8.0 CONSTRUCTION RECOMMENDATIONS AND CONSIDERATIONS

8.1 Excavation and Dewatering

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

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

Based on a Q1.1 water level of El. 307.06, a cofferdam will likely be necessary to successfully dewater and construct the abutments. Wood chips were noted in BB-MPMS-201 at the top of the glacial stream deposit. Wood chips indicate the presence of wood debris or timber and may obstruct the installation of a cofferdam. These obstructions may need to be removed by conventional excavation methods.

The new integral abutments will be constructed behind the existing abutments. Interaction of the new construction with the existing substructure is not anticipated, but it is the responsibility of the contractor to remove any resulting obstructions.

8.2 Pile Installation

Cobbles and boulders were encountered in the glacial till deposit. Wood chips were observed at the top of the glacial stream deposit in one boring, indicating the presence of wood debris or timber at the project location. There is potential for these obstructions to cause difficulties during pile driving operations. If obstructions are encountered prior to reaching the maximum required penetration resistance on bedrock they may be cleared by conventional excavation methods, pre-augering, predrilling, spudding, use of rock chisels, or down-hole hammers.

A casing shoe was abandoned in BB-MPMS-101 at approximately El. 264.6. Piles should be located to avoid conflict with the abandoned casing shoe.

9.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Kingman Road Bridge in Macwahoc Plantation, 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:10/18/2023

Division: GEOTECH

Username: Nathan.P.Pukay

MACWAHOC PLT., MAINE



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

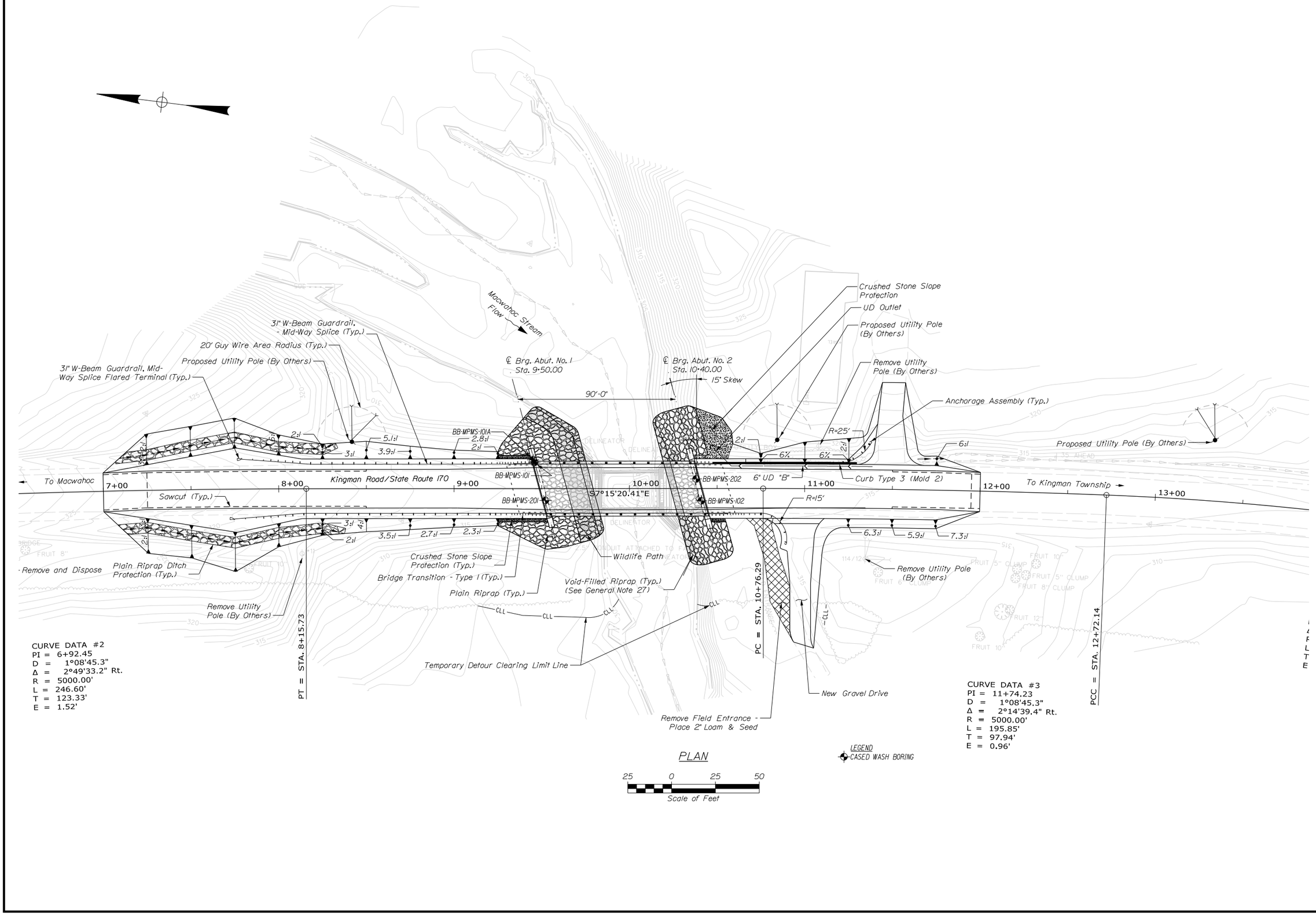
0.5 Miles

1 inch = 0.57 miles

Date: 9/13/2023
Time: 7:46:15 AM

SHEET NUMBER 1 OF 5	KINGMAN ROAD BRIDGE MACWAHOC STREAM MACWAHOC PLT. AROOSTOOK C.	STATE OF MAINE DEPARTMENT OF TRANSPORTATION
		2225000
		WIN
	LOCATION MAP	BRIDGE NO. 5021 22250.00 BRIDGE PLANS

Filename: ...\00\GEOTECH\MST#\005_BLP1.dgn Division: GEOTECH Username: Nathan.P.Pukay Date:12/13/2023



CURVE DATA #2
PI = 6+92.45
D = 1°08'45.3"
Δ = 2°49'33.2" Rt.
R = 5000.00'
L = 246.60'
T = 123.33'
E = 1.52'

CURVE DATA #3
PI = 11+74.23
D = 1°08'45.3"
Δ = 2°14'39.4" Rt.
R = 5000.00'
L = 195.85'
T = 97.94'
E = 0.96'

PLAN

LEGEND
CASED WASH BORING

25 0 25 50
Scale of Feet

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2225000

WIN 22250.00

BRIDGE NO. 5021

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	----------	------
DESIGN-DETAILED	N.PUKAY	DEC 2021
CHECKED-REVIEWED	-----	-----
DESIGN2-DETAILED2	----------	------
DESIGN3-DETAILED3	----------	------
REVISIONS 1	------	------
REVISIONS 2	------	------
REVISIONS 3	------	------
REVISIONS 4	------	------
FIELD CHANGES	------	------

SIGNATURE

P.E. NUMBER

DATE

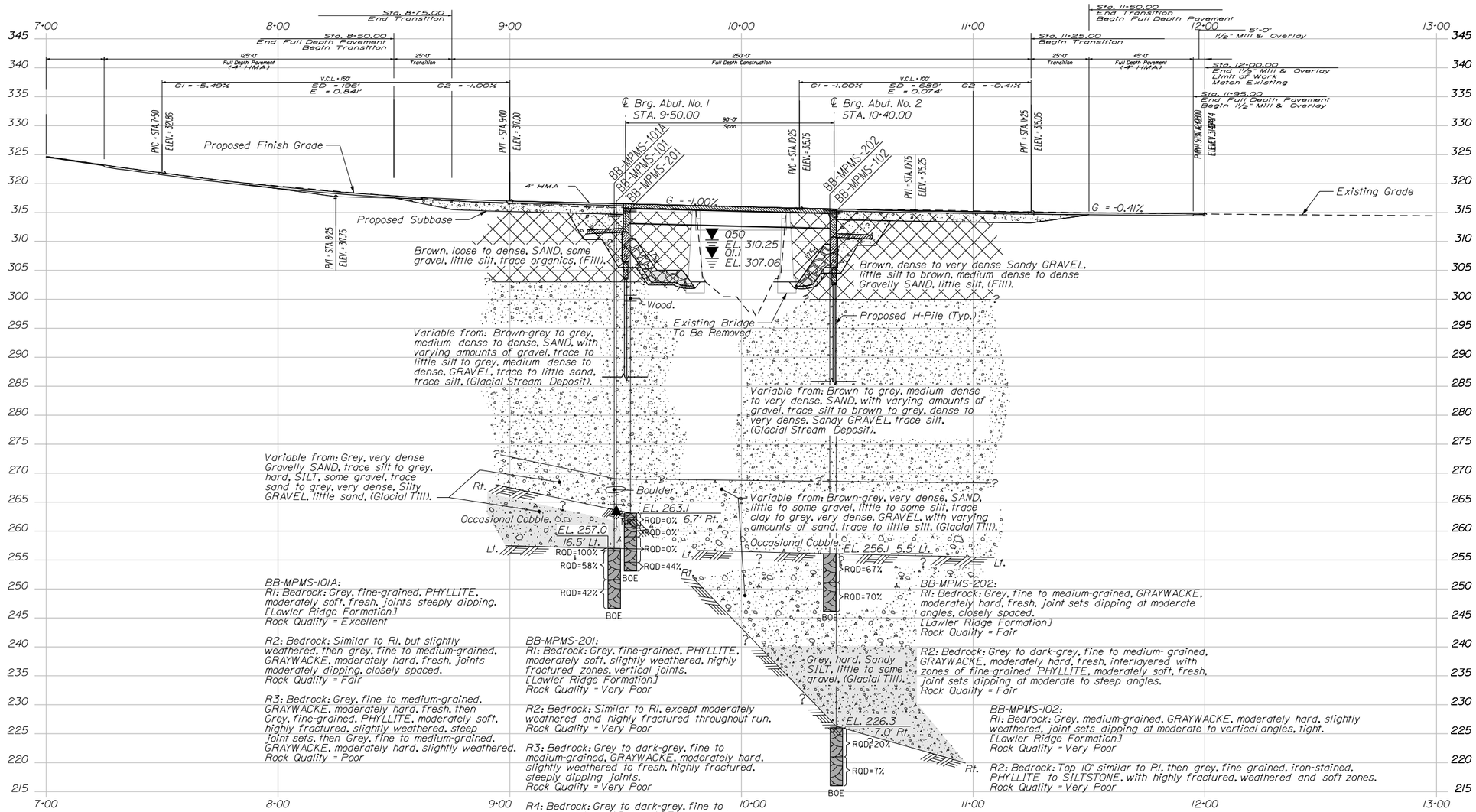
KINGMAN ROAD BRIDGE
MACWAHOC STREAM
MACWAHOC PLT. AROOSTOOK COUNTY

BORING LOCATION PLAN

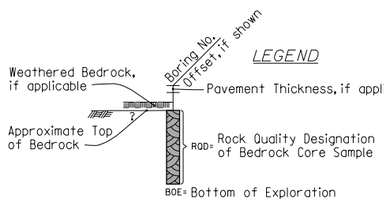
SHEET NUMBER

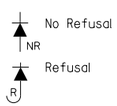
2

OF 5



LEGEND





PROFILE

HORIZ 25 0 25 50
VERT 10 0 10 20
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

2225000

WIN 22250.00

BRIDGE PLANS

BRIDGE NO. 5021

	BY	DATE
PROJ. MANAGER	----------	
DESIGN-DETAILED	T. WHITE	JAN 2022
CHECKED-REVIEWED	N.PUKAY	-----
DESIGN2-DETAILED2	----------	-----
DESIGN3-DETAILED3	----------	-----
REVISIONS 1	----------	-----
REVISIONS 2	----------	-----
REVISIONS 3	----------	-----
REVISIONS 4	----------	-----
FIELD CHANGES	----------	-----

SIGNATURE

P.E. NUMBER

DATE

KINGMAN ROAD BRIDGE
MACWAHOC STREAM
MACWAHOC PLT. AROOSTOOK COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

3

OF 5

Filename: ...\00\GEOTECH\MSTA\006_ISP1.dgn Division: GEOTECH Username: Nathan.P.Pukay Date:12/13/2023

Username: Nathan.P.Pukay Date:12/14/2023
Division: GEOTECH
Filename: ...\MSTA\007_BORING LOGS1.dgn

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

Project: Kingman Road Bridge #5021 carries Route 170 over Macwahoc Stream
Location: Macwahoc Plantation, Maine
Boring No.: BB-MPMS-101
WIN: 22250.00

Driller: S.W. Cole | Elevation (ft.): 314.6 | Auger ID/OD: 5" Solid Stem
Operator: Kevin Brton | Datum: NAVD88 | Sampler: Standard Split Spoon
Logged By: Nathan Pukay | Rig Type: Diedrich D-50 | Hammer Wt./Fall: 140#/30.0"
Date Start/Finish: 4/4/20221 09:45-14:15 | Drilling Method: Cased Wash Boring | Core Barrel: N/A
Boring Location: 9+46. 15.0 ft Lt. | Casing ID/OD: HW(4.0"/4.5"), NW(3.0"/3.5") | Water Level(ft): None Observed
Hammer Efficiency Factor: 0.91 | Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐

Soil/Rock Legend:
D = Split Spoon Sample
SSA = Solid Stem Auger
WO = Unsuccessful Split Spoon Sample Attempt
HSA = Hollow Stem Auger
U = Thin Wall Tube Sample
RC = Roller Cone
WA = Unsuccessful Thin Wall Tube Sample Attempt
WOH = Weight of 140lb. Hammer
V = Field Vane Shear Test.
PP = Pocket Penetrometer
WOR/C = Weight of Rods or Casing
WH = Unsuccessful Field Vane Shear Test Attempt
WOH = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(UC)}$ = Lab Vane Undrained Shear Strength (psf)
q_u = Unconfined Compressive Strength (tsf)
N-uncorrected = Raw Field SPT N-value
Hammer Eff/Safety Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Eff/Safety
N_{60} = Hammer Eff/Safety Factor-60/Hammer-uncorrected
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. Penetration Resistance or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							554				
5	10	24/6	5.00 - 7.00	3/3/3/3	6	9				Brown, damp, loose, SAND, some gravel, little silt, trace organics. (Fill).	GM241510 A-1-b, SM WC=12.9%
10	20	24/1	10.00 - 12.00	3/3/7/13	10	15	20			Similar to above, except medium dense.	
							40	303.6		11.0	
15	30	24/12	15.00 - 17.00	15/7/6/6	13	20	71			Brown-grey, wet, medium dense, SAND, little silt, trace gravel. (Glacial Stream Deposit).	GM241511 A-2-4, SM WC=19.0%
							107				
							124+				
20	40	24/17	20.00 - 22.00	4/7/9/9	16	24	30			Grey, wet, medium dense, SAND, little silt, trace gravel. (Glacial Stream Deposit).	GM241512 A-2-4, SP-SM WC=19.7%
							54				
							64				
							70				
25	MD	24/0	25.00 - 27.00	6/6/6/6	12	18	50			Roller Coned ahead to 32.0 ft bgs.	
							84				
30	MD	24/0	30.00 - 32.00	14/9/8/5	17	26	56			Grey, wet, medium dense, Gravelly SAND, trace silt, (Glacial Stream Deposit).	GM241513 A-1-b, SM WC=11.0%
							63				
	50	24/10	32.00 - 34.00	10/5/4/4	9	14	64				
							85				
35	MD	24/0	34.50 - 36.50	12/9/11/9	20	30	71				
							60				
							97				
							101				
							95				
40	MD	24/0	39.50 - 41.50	30/39/18/9	57	86	94			Very dense.	
							96				
	60	24/8	41.50 - 43.50	8/5/5/7	10	15	85			Grey, wet, medium dense, Gravelly SAND, trace silt, (Glacial Stream Deposit).	
							75				
							73				
45							81				
	70	18/8	45.00 - 46.50	8/28/50	78	118	81	269.1		45.5	
							90			Grey, wet, very dense, Gravelly SAND, trace silt, (Glacial Till).	
										Casing advanced to 50.0 ft bgs. Hole abandoned, casing shoe broke off and left in hole.	
50								264.6		50.0	

Bottom of Exploration at 50.0 feet below ground surface.

Remarks:
Hammer #367
Note: This hole was abandoned due to a casing shoe (metal) breaking off at El. 264.6.

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-MPMS-101

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

Project: Kingman Road Bridge #5021 carries Route 170 over Macwahoc Stream
Location: Macwahoc Plantation, Maine
Boring No.: BB-MPMS-101A
WIN: 22250.00

Driller: S.W. Cole | Elevation (ft.): 314.3 | Auger ID/OD: 5" Solid Stem
Operator: Kevin Brton | Datum: NAVD88 | Sampler: Standard Split Spoon
Logged By: Nathan Pukay | Rig Type: Diedrich D-50 | Hammer Wt./Fall: 140#/30.0"
Date Start/Finish: 4/4/2022-4/5/2022 | Drilling Method: Cased Wash Boring | Core Barrel: NQ-2"
Boring Location: 9+45.1, 16.5 ft Lt. | Casing ID/OD: HW(4.0"/4.5"), NW(3.0"/3.5") | Water Level(ft): None Observed
Hammer Efficiency Factor: 0.91 | Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐

Soil/Rock Legend: (same as BB-MPMS-101)

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows/6 in. Penetration Resistance or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							50			Drove HW Casing to 45.0 ft bgs., see BB-MPMS-101 for material descriptions and casing blow counts up to 25.0 ft bgs.	
5											
10											
15											
20											
25	10	24/9	25.00 - 27.00	6/5/5/5	10	15		289.3		25.0	
										Grey, wet, medium dense, GRAVEL, trace sand, trace silt, (Glacial Stream Deposit).	
30											
35	20	24/5	34.50 - 36.50	26/14/10/9	24	36					GM241514 A-1-b, SM-SW WC=10.6%
40	30	24/6	40.00 - 42.00	15/7/5/4	12	18				Grey, wet, medium dense, SAND, trace gravel, trace silt, (Glacial Stream Deposit).	GM241515 A-1-b, SP WC=22.4%
45							85				
							150	267.6		46.7	
							141			Boulder from 46.7-47.7 ft bgs. Roller Coned ahead to 50.0 ft bgs.	
							200				
50	40	6/6	50.00 - 50.50	100/6"	---		OPEN HOLE			Grey, wet, hard, SILT, some gravel (Rock Chips), trace sand, (Glacial Till), Occasional cobble.	
55	MD	2/0	55.00 - 55.17	75/2"	---						
	R1	4.8/4.8	57.30 - 57.53	RQD = 100% (95-1.5-0)	---		NQ-2	257.0		57.3	
	R2	60/53	57.53 - 57.70	MOD = 58%						Top of Bedrock at Elev. 257.0 ft. R1: Bedrock: Grey, fine-grained, PHYLLITE, moderately soft, fresh, joints steeply dipping, annealed with calcite. [Lower Ridge Formation] Rock Quality = Excellent R1: Core Times (min:sec): 57.3-57.7 ft (5:11) Core Blocked, little wash return when coring, set in NW Casing. 100% Recovery (50) Grey, wet, very dense, Rock Chips, trace silt, R2: Bedrock: Similar to R1, but slightly weathered and with iron staining on joint faces, then grey, fine to medium-grained, GRAYWACKE, moderately hard, fresh, joints moderately dipping, closely spaced. [Lower Ridge Formation] Rock Quality = Fair R2: Core Times (min:sec): 57.7-58.7 ft (3:16) 58.7-59.7 ft (3:01) 59.7-60.7 ft (4:06) 60.7-61.7 ft (2:43) 61.7-62.7 ft (2:22) some dirty water 100% Recovery R3: Bedrock: Grey, fine to medium-grained, GRAYWACKE, moderately hard, fresh, then Grey, fine-grained, PHYLLITE, moderately soft, highly fractured, slightly weathered, iron staining on joint faces, steep joint sets, undulating, with significant quartz-calcite annealed joint, then Grey, fine to medium-grained, GRAYWACKE, moderately hard, slightly weathered. [Lower Ridge Formation] Rock Quality = Poor R3: Core Times (min:sec): 62.7-63.7 ft (2:46) 63.7-64.7 ft (2:05) 64.7-65.7 ft (2:14) 65.7-66.7 ft (2:38) 66.7-67.7 ft (3:02) 100% Recovery	
60	R3	60/60	62.70 - 67.70	ROD = 42%				246.6		67.7	
										Bottom of Exploration at 67.7 feet below ground surface.	
70											

Remarks:
Hammer #367

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-MPMS-101A

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

Project: Kingman Road Bridge #5021 carries Route 170 over Macwahoc Stream
Location: Macwahoc Plantation, Maine
Boring No.: BB-MPMS-102
WIN: 22250.00

Driller: MaineDOT | Elevation (ft.): 315.3 | Auger ID/OD: 5" Solid Stem
Operator: Doggett/Brooks | Datum: NAVD88 | Sampler: Standard Split Spoon
Logged By: N.Pukay/B.Miller | Rig Type: CME 45C | Hammer Wt./Fall: 140#/30.0"
Date Start/Finish: 12/7.9-10.13/2021 | Drilling Method: Cased Wash Boring | Core Barrel: NQ-2"
Boring Location: 10+41. 7.0 ft Rt. | Casing ID/OD: NW(3.0"/3.5") | Water Level(ft): 10.5 ft bgs.
Hammer Efficiency Factor: 0.974 | Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐

Soil/Rock Legend: (same as BB-MPMS-101)

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows/6 in. Penetration Resistance or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							554	315.0		4" HMA. 0.3	
5	10	24/12	5.00 - 7.00	8/20/25/17	45	73				Brown, damp, very dense, Sandy GRAVEL, little silt, (Fill).	
10	20	24/14	10.00 - 12.00	13/10/16/22	26	42	22			Brown, wet, dense, Sandy GRAVEL, little silt, (Fill).	GM337502 A-1-b, GM WC=12.9%
							57				
							71				
15							46				
	30	24/13	15.00 - 17.00	8/10/12/9	22	36	51	300.3		15.0	
							24			Brown, wet, dense, Sandy GRAVEL, trace silt, (Glacial Stream Deposit).	GM337503 A-1-b, GW-GM WC=15.4%
20							41				
	40	24/14	20.00 - 22.00	10/6/10/13	16	26	51				
							45			Similar to above, except medium dense.	
							53				
							70				
25							44				
	50	24/9	25.00 - 27.00	15/20/12/26	32	52	39				
							42			Brown, wet, very dense, Sandy GRAVEL, trace silt (Glacial Stream Deposit).	
							74				
30							86				
	60	24/15	30.00 - 32.00	7/7/6/7	13	21	77				
							66			Brown-grey, wet, medium dense, SAND, little gravel, trace silt, (Glacial Stream Deposit).	GM337504 A-1-b, SP-SM WC=16.1%
							22				
35							38				
	70	24/12	35.00 - 37.00	8/7/8/8	15	24	65				
							82			Similar to above, except grey.	
							61				
40							48				
	80	24/14	40.00 - 42.00	13/14/20/12	34	55	69			Grey, wet, very dense, Gravelly, SAND, trace silt, (Glacial Stream Deposit).	
							48				
							73				
45							71				
	90	24/14	45.00 - 47.00	4/7/5/5	12	19	54			Grey, wet, medium dense, SAND, trace gravel, trace silt, (Glacial Stream Deposit).	
							55				
							67			47.0	
							104	268.3			
50	100	24/17	50.00 - 52.00	7/9/24/31	33	54	OPEN HOLE			Brown-grey, wet, very dense, SAND, some gravel, little silt, trace clay, (Glacial Till), Roller Coned ahead to 55.0 ft bgs.	GM337505 A-1-b, SC-SM WC=9.6%
							106				
							126			Cobble from 52.5-52.8 ft bgs.	
55	110	24/15	55.00 - 57.00	34/28/40/50	68	110				Brown-grey, wet, very dense, SAND, some gravel, some silt, (Glacial Till), Roller Coned ahead to 60.0 ft bgs.	
60	120	8.4/8.4	60.00 - 60.70	41/60 (2.4")	---					Cobble from 57.0-57.5 ft bgs.	
										Similar to above. Roller Coned ahead to 65.0 ft bgs. Occasional cobble.	
65	130	1.2/1.2	65.00 - 65.10	55(1.2")	---					Brown-grey, wet, very dense, SAND, some silt, little gravel, (Glacial Till), Roller Coned ahead to 70.0 ft bgs.	
70	MD	1.2/0	70.00 - 70.10	70(1.2")	---					Roller Coned ahead to 75.0 ft bgs.	

Remarks:

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

Page 1 of 2
Boring No.: BB-MPMS-102

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

Project: Kingman Road Bridge #5021 carries Route 170 over Macwahoc Stream
Location: Macwahoc Plantation, Maine
Boring No.: BB-MPMS-102
WIN: 22250.00

Driller: MaineDOT | Elevation (ft.): 315.3 | Auger ID/OD: 5" Solid Stem
Operator: Doggett/Brooks | Datum: NAVD88 | Sampler: Standard Split Spoon
Logged By: N.Pukay/B.Miller | Rig Type: CME 45C | Hammer Wt./Fall: 140#/30.0"
Date Start/Finish: 12/7.9-10.13/2021 | Drilling Method: Cased Wash Boring | Core Barrel: NQ-2"
Boring Location: 10+41. 7.0 ft Rt. | Casing ID/OD: NW(3.0"/3.5") | Water Level(ft): 10.5 ft bgs.
Hammer Efficiency Factor: 0.974 | Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐

Soil/Rock Legend: (same as BB-MPMS-101)

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows/6 in. Penetration Resistance or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
75	140	1.2/1.2	75.00 - 75.10	100(1.2")	---					Grey, moist, hard, Sandy SILT, little gravel, (Glacial Till), Roller Coned ahead to 80.0 ft bgs.	
80	150	1.2/1.2	80.00 - 80.10	70(1.2")	---					Grey, wet, hard, Sandy SILT, some gravel, (Glacial Till), Roller Coned ahead to 89.3 ft bgs.	
85											
90	R1	60/60	89.30 - 94.30	ROD = 20%			NQ-2	226.3		89.0	
										Top of Bedrock at Elev. 226.3 ft. R1: Bedrock: Grey, medium-grained, GRAYWACKE, with calcite veins, moderately hard, slightly weathered, joint sets dipping at moderate to vertical angles, tight. [Lower Ridge Formation] Rock Quality = Very Poor R1: Core Times (min:sec): 89.3-90.3 ft (2:43) 90.3-91.3 ft (2:49) 91.3-92.3 ft (3:05) 92.3-93.3 ft (2:22) 93.3-94.3 ft (2:45) 100% Recovery R2: Bedrock: Top 10" similar to R1, then grey, fine-grained, iron-stained, PHYLLITE to SILTSTONE, with highly fractured, weathered and soft zones. [Lower Ridge Formation] Rock Quality = Very Poor R2: Core Times (min:sec): 94.3-95.3 ft (2:52) 95.3-96.3 ft (2:47) 96.3-97.3 ft (3:20) dirty brown water 97.3-98.3 ft (5:10) 98.3-99.3 ft (9:30) had to work the bit, added tfm. 80% Recovery	
95	R2	60/48	94.30 - 99.30	ROD = 7%				216.0		99.3	
										Bottom of Exploration at 99.3 feet below ground surface.	
100											
105											
110											
115											
120											
125											

Remarks:

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

Page 2 of 2
Boring No.: BB-MPMS-102

	BY	DATE
PROJ. MANAGER	----------	------
DESIGN-DETAILED	N.PUKAY	DEC 2023
CHECKED-REVIEWED	----------	------
DESIGN2-DETAILED2	----------	------
DESIGN3-DETAILED3	----------	------
REVISIONS 1	----------	------
REVISIONS 2	----------	------
REVISIONS 3	----------	------
REVISIONS 4	----------	------
FIELD CHANGES	----------	------

SIGNATURE

P.E. NUMBER

DATE

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2225000

WIN 22250.00

BRIDGE NO. 5021

BRIDGE PLANS

KINGMAN ROAD BRIDGE
MACWAHOC STREAM
MACWAHOC PLT. AROOSTOOK COUNTY

BORING LOGS

SHEET NUMBER
4
OF 5

Filename: ...\MSTA\008_BORING LOGS2.dgn Username: Nathan.P.Pukay Date:12/14/2023

Division: GEOTECH

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Kingman Road Bridge #5021 carries Route 170 over Macwahoc Stream
Location: Macwahoc Plantation, Maine

Boring No.: BB-MPMS-201
WIN: 22250.00

Driller:	S.W. Cole	Elevation (ft.) 315.7	Auger ID/OD: 5" Solid Stem
Operator:	Kevin/Brian	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By:	N.Pukay	Rig Type: Diedrich D-50	Hammer Wt./Falls: 140#/30.0"
Date Start/Finish:	3/24/2023: 08:15-14:00	Drilling Methods: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location:	9+52.1', 6.7' ft Rt.	Casing ID/OD: HW:4.0"/4.5")	Water Level*: 9.0 ft bgs.
Hammer Efficiency Factor: 0.92		Hammer Type: Automatic ☒ Hydraulic ☐	Rope & Cathead ☐

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
WU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test; PP = Pocket Penetrometer
WR = 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
WOP = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf); $S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_u = Unconfined Compressive Strength (tsf)
Unconfined = Raw Field SPT Number
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-value Corrected for Hammer Efficiency
N_{60} = Hammer Efficiency Factor/60%(Hammer-Unconfined
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
S = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Rec./Rec. (in)	Sample Depth (ft.)	Blows / 6 in. Undrained Shear Strength (psf) or RQD (%)	Uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	315.7		7" HMA. (0.6)	
5	10	24/4	5.00 - 7.00	2/6/18/8	24	37	15			Brown, damp, dense, SAND, some gravel, little silt. (Fill).	
							35				
							27				
10							17				
	MD	24/0	10.00 - 12.00	3/2/2/2	4	6	14			Loose.	
							12				
							15				
							72				
15							80				
	20	24/1	15.00 - 17.00	18/12/8/5	20	31	37	302.7		Dark brown organics and wood chips in wash water from 13.0-14.5 ft bgs. (13.0)	
							42			Gray, wet, dense, SAND, little silt, little gravel, wood chips, (Glacial Stream Deposit). Wood chips in wash water from 15.0-16.0 ft bgs.	
							34				
							39				
20							42				
	30	24/5	20.00 - 22.00	4/5/6/8	11	17	27			Gray, wet, medium dense, SAND, little silt, trace gravel. (Glacial Stream Deposit).	
							35				
							58				
							77				
25							100				
	40	24/10	25.00 - 27.00	10/11/15/16	26	40	66			Gray, wet, dense, SAND, little silt, trace gravel. (Glacial Stream Deposit).	
							102			Occasional Cobble	
							94				
							71				
30							70				
	50	24/4	30.00 - 32.00	12/7/7/16	14	21	60			Gray, wet, medium dense, SAND, some gravel, trace silt. (Glacial Stream Deposit).	
							69				
							88				
							81				
35							76				
	60	24/2	35.00 - 37.00	11/13/7/8	20	31	66			Gray, wet, dense, GRAVEL, little sand, trace silt. (Glacial Stream Deposit).	
							88				
							71				
40							88				
							72				
	70	24/1	40.00 - 42.00	7/6/7/10	13	20	69			Similar to 60, except medium dense.	
							80				
							86				
45							81				
							84				
	80	24/2	45.00 - 47.00	8/6/5/4	11	17	90			Gray, wet, medium dense, SAND, trace gravel, trace silt. (Glacial Stream Deposit).	
							100				
							103	268.7		(47.0)	
							97				
50							181				
	90	24/12	50.00 - 52.00	16/14/29/42	43	66	69			Gray, wet, very dense, Silty GRAVEL (bedrock chips), little sand, (Glacial Till).	
							90				
	R1	30/30	52.60 - 55.10	RQD = 0%			0101 NQ-2	263.1		0101 blows for 0.6 ft. (52.6) Top of Bedrock at Elev. 263.1 ft. R1: Bedrock: Grey, fine-grained, PHYLLITE, moderately soft, slightly weathered, highly fractured zones. [Joints are vertical with iron staining and rock flour. [Lower Ridge Formation] Rock Quality = Very Poor R1: Core Times (min:sec) 52.6-53.6 ft (2:10) 53.6-54.6 ft (2:43) 54.6-55.1 ft (1:51) Core Blocked 100% Recovery	
55											
	R2	19.2/15	55.10 - 56.70	RQD = 0%						R2: Similar to R1, except moderately weathered and highly fractured throughout run. [Lower Ridge Formation] Rock Quality = Very Poor R2: Core Times (min:sec) 55.1-55.6 ft (0:59) 55.6-56.6 ft (3:03) 56.6-56.7 ft (0:35) Core Blocked 78% Recovery	
	R3	52.8/25	56.70 - 61.10	RQD = 0%							
60											
	R4	18/18	61.10 - 62.60	RQD = 44%				253.1		R3: Bedrock: Grey, fine-grained, GRAYBACKE, moderately hard, slightly weathered, highly fractured, joints are steeply dipping with iron staining, then dark-grey, fine to medium-grained, GRAYBACKE, moderately hard, fresh, highly fractured, joints steeply dipping, with several mthy, low angle joints annealed with quartz or calcite. [Lower Ridge Formation] Rock Quality = Very Poor R3: Core Times (min:sec) 56.-57.6 ft (2:35) 57.6-58.6 ft (2:34) 58.6-59.6 ft (5:20) 59.6-60.6 ft (6:22) 60.6-61.1 ft (3:40) Core Blocked 67% Recovery	
65											
70										R4: Bedrock: Grey to dark-grey, fine to medium-grained, GRAYBACKE, moderately hard, fresh, joint sets dipping at steep angles, closely spaced, undulating. [Lower Ridge Formation] Rock Quality = Poor. R4: Core Times (min:sec) 61.1-61.6 ft (2:14) 61.6-62.6 ft (4:04) 100% Recovery	
										(62.6) Bottom of Exploration at 62.6 feet below ground surface.	
75											

Remarks:
Auto Hammer #367

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-MPMS-201

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Kingman Road Bridge #5021 carries Route 170 over Macwahoc Stream
Location: Macwahoc Plantation, Maine

Boring No.: BB-MPMS-202
WIN: 22250.00

Driller:	S.W. Cole	Elevation (ft.) 314.8	Auger ID/OD: 5" Solid Stem
Operator:	Kevin/Brian	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By:	N.Pukay	Rig Type: Diedrich D-50	Hammer Wt./Falls: 140#/30.0"
Date Start/Finish:	3/27/2023-3/28/2023	Drilling Methods: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location:	10+38.1', 5.5 Ft Lt.	Casing ID/OD: HW:4.0"/4.5", NW:3.0"/3.5")	Water Level*: 9.0 ft bgs.
Hammer Efficiency Factor: 0.92		Hammer Type: Automatic ☒ Hydraulic ☐	Rope & Cathead ☐

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
WU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test; PP = Pocket Penetrometer
WR = 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
WOP = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf); $S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_u = Unconfined Compressive Strength (tsf)
Unconfined = Raw Field SPT Number
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-value Corrected for Hammer Efficiency
N_{60} = Hammer Efficiency Factor/60%(Hammer-Unconfined
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
S = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Rec./Rec. (in)	Sample Depth (ft.)	Blows / 6 in. Undrained Shear Strength (psf) or RQD (%)	Uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	314.8		7" HMA. (0.6)	
5	10	24/3	5.00 - 7.00	6/10/12/17	22	34				Brown, damp, dense, Gravelly SAND, little silt. (Fill).	
							30				
10							40				
	20	24/16	10.00 - 12.00	11/11/8/9	19	29	47			Brown, damp, medium dense, Gravelly SAND, little silt, trace organics. (Fill).	
							60				
							59				
							73				
15							93				
	30	18/4	15.00 - 16.50	12/22/83	105	161	38	300.4		Brown, wet, very dense, Sandy GRAVEL, trace silt. (Glacial Stream Deposit). Cobble from 15.3-16.5 ft bgs. Roller Coned ahead to 17.0 ft bgs. (15.0)	
							102				
							169				
							111				
20							84				
	MD	24/0	20.00 - 22.00	14/9/11/14	20	31	37			Dense.	
							53				
							69				
							51				
25							50				
	40	24/2	25.00 - 27.00	9/10/14/15	24	37	49			Grey-brown, wet, dense, Sandy GRAVEL, trace silt. (Glacial Stream Deposit).	
							51				
							89				
30							88				
	50	24/5	30.00 - 32.00	18/9/8/7	17	26	91			Similar to 40, except medium dense.	
							55				
							32				
35							59				
	60	24/1	35.00 - 37.00	6/11/28/13	39	60	74			Grey, wet, very dense, Sandy GRAVEL, trace silt. (Glacial Stream Deposit).	
							53				
							20				
40							77				
							134				
	70	24/4	40.00 - 42.00	43/17/8/7	25	38	148			Grey-brown, wet, dense, Sandy GRAVEL, trace silt. (Glacial Stream Deposit).	
							160				
							79				
							61				
45							97	272.4		(43.0)	
							104				
	80	24/1	45.00 - 47.00	20/30/36/36	66	101	84			Grey, wet, very dense, Sandy GRAVEL, little silt, (Glacial Till). Roller Coned ahead to 48.0 ft bgs.	
							102				
							168				
							190				
50							266				
	90	24/2	50.00 - 52.00	26/36/43/57	79	121	95			Grey, wet, very dense, Sandy GRAVEL, little sand, trace silt. (Glacial Till). Roller Coned ahead to 55.0 ft bgs.	
							139				
							103				
							166				
55							176				
							102				
	100	16.8/10	55.00 - 56.40	46/47/52(4.8")	---		0/0 Split Tube			Grey, wet, very dense, Sandy GRAVEL, little silt. (Glacial Till). 0/0 blows for 0.5 ft. Roller Coned ahead to 59.3 ft bgs. Rock chips in wash water. Set 7in NR Casing.	
60	R1	60/50	59.30 - 64.30	RQD = 67%			NQ-2	256.1		(59.3) Top of Bedrock at Elev. 256.1 ft. R1: Bedrock: Grey to dark-grey, fine to medium-grained, GRAYBACKE, moderately hard, fresh, joint sets dipping at moderate angles, closely spaced, iron stained. [Lower Ridge Formation] Rock Quality = Fair R1: Core Times (min:sec) 59.3-60.3 ft (3:32) 60.3-61.3 ft (2:56) 61.3-62.3 ft (3:40) 62.3-63.3 ft (3:18) 63.3-64.3 ft (4:47) 83% Recovery	
65	R2	60/60	64.30 - 69.30	RQD = 70%						R2: Bedrock: Grey to dark-grey, fine to medium-grained, GRAYBACKE, moderately hard, fresh, interlayered with zones of fine-grained PHYLLITE, moderately soft, fresh, joint sets dipping at moderate to steep angles, slightly quartz-calcite along joint faces. [Lower Ridge Formation] Rock Quality = Fair R2: Core Times (min:sec) 64.3-65.3 ft (3:05) 65.3-66.3 ft (2:58) 66.3-67.3 ft (2:17) 67.3-68.3 ft (2:31) 69.3-69.3 ft (2:25) 100% Recovery	
70											
										(69.3) Bottom of Exploration at 69.3 feet below ground surface.	
75											

Remarks:
Auto Hammer #367

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-MPMS-202

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2225000

WIN 22250.00

BRIDGE NO. 5021

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	_____	_____
DESIGN-DETAILED	N.PUKAY	DEC 2023
CHECKED-REVIEWED	_____	_____
DESIGN2-DETAILED2	_____	_____
DESIGN3-DETAILED3	_____	_____
REVISIONS 1	_____	_____
REVISIONS 2	_____	_____
REVISIONS 3	_____	_____
REVISIONS 4	_____	_____
FIELD CHANGES	_____	_____

SIGNATURE

P.E. NUMBER

DATE

KINGMAN ROAD BRIDGE
MACWAHOC STREAM
MACWAHOC PLT. AROOSTOOK COUNTY

BORING LOGS

SHEET NUMBER

5

OF 5

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

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):

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))

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Kingman Road Bridge #5021 carries Route 170 over Macwahoc Stream
Location: Macwahoc Plantation, Maine

Boring No.: BB-MPMS-101
WIN: 22250.00

Driller:	S.W. Cole	**Elevation (ft.)**	314.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Nathan Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30.0"
Date Start/Finish:	4/4/2022; 09:45-14:15	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	9+46, 15.0 ft Lt.	**Casing ID/OD:**	HW(4.0"/4.5"), NW(3.0"/3.5")	**Water Level*:**	None Observed
Hammer Efficiency Factor: 0.91		**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				
5	1D	24/6	5.00 - 7.00	3/3/3/3	6	9				Brown, damp, loose, SAND, some gravel, little silt, trace organics, (Fill).	G#241510 A-1-b, SM WC=12.9%
10	2D	24/1	10.00 - 12.00	3/3/7/13	10	15	20			Similar to above, except medium dense.	
							40	303.6		11.0	
							71				
							107				
							124+				
15	3D	24/12	15.00 - 17.00	15/7/6/6	13	20	85			Brown-grey, wet, medium dense, SAND, little gravel, trace silt, (Glacial Stream Deposit).	G#241511 A-3, SP-SM WC=19.0%
							40				
							39				
							40				
							35				
20	4D	24/17	20.00 - 22.00	4/7/9/9	16	24	30			Grey, wet, medium dense, SAND, little silt, trace gravel, (Glacial Stream Deposit).	G#241512 A-2-4, SP-SM WC=19.7%
							54				
							64				
							70				
25							50				

Remarks:

Hammer #367
Note: This hole was abandoned due to a casing shoe (metal) breaking off at El. 264.6.

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Kingman Road Bridge #5021 carries Route 170 over Macwahoc Stream
Location: Macwahoc Plantation, Maine

Boring No.: BB-MPMS-101
WIN: 22250.00

Driller:	S.W. Cole	**Elevation (ft.)**	314.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Nathan Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30.0"
Date Start/Finish:	4/4/2022; 09:45-14:15	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	9+46, 15.0 ft Lt.	**Casing ID/OD:**	HW(4.0"/4.5"), NW(3.0"/3.5")	**Water Level*:**	None Observed
Hammer Efficiency Factor: 0.91		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
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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	MD	24/0	25.00 - 27.00	6/6/6/6	12	18	32				
							54				
							56				
							88				
							84				
30	MD	24/0	30.00 - 32.00	14/9/8/5	17	26	56			Roller Coned ahead to 32.0 ft bgs.	
							63				
	5D	24/10	32.00 - 34.00	10/5/4/4	9	14	64			Grey, wet, medium dense, Gravelly SAND, trace silt, (Glacial Stream Deposit).	G#241513 A-1-a, SW-SM WC=11.0%
							85				
	MD	24/0	34.50 - 36.50	12/9/11/9	20	30	71				
35							60				
							97				
							101				
							95				
	MD	24/0	39.50 - 41.50	30/39/18/9	57	86	94			Very dense.	
40							96				
	6D	24/8	41.50 - 43.50	8/5/5/7	10	15	85			Grey, wet, medium dense, Gravelly SAND trace silt, (Glacial Stream Deposit).	
							75				
							73				
							81				
45	7D	18/8	45.00 - 46.50	8/28/50	78	118	81	269.1		45.5	
							90			Grey, wet, very dense, Gravelly SAND, trace silt, (Glacial Till).	
										Casing advanced to 50.0 ft bgs. Hole abandoned, casing shoe broke off and left in hole.	
50											

Remarks:

Hammer #367
Note: This hole was abandoned due to a casing shoe (metal) breaking off at El. 264.6.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Kingman Road Bridge #5021 carries Route 170 over Macwahoc Stream
Location: Macwahoc Plantation, Maine

Boring No.: BB-MPMS-101
WIN: 22250.00

Driller:	S.W. Cole	**Elevation (ft.)**	314.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Nathan Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30.0"
Date Start/Finish:	4/4/2022; 09:45-14:15	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	9+46, 15.0 ft Lt.	**Casing ID/OD:**	HW(4.0"/4.5"), NW(3.0"/3.5")	**Water Level*:**	None Observed
Hammer Efficiency Factor: 0.91		**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 Information: 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								264.6		**Bottom of Exploration at 50.0 feet below ground surface.** 50.0	
										Bottom of Exploration at 50.0 feet below ground surface.	
55											
60											
65											
70											
75											

Remarks:

Hammer #367
Note: This hole was abandoned due to a casing shoe (metal) breaking off at El. 264.6.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Kingman Road Bridge #5021 carries Route 170 over Macwahoc Stream
Location: Macwahoc Plantation, Maine

Boring No.: BB-MPMS-101A

WIN: 22250.00

Driller:	S.W. Cole	**Elevation (ft.)**	314.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Nathan Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30.0"
Date Start/Finish:	4/4/2022-4/5/2022	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	9+45.1, 16.5 ft Lt.	**Casing ID/OD:**	HW(4.0"/4.5"), NW(3.0"/3.5")	**Water Level*:**	None Observed
Hammer Efficiency Factor: 0.91		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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 Information: 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							HW			Drove HW Casing to 45.0 ft bgs., see BB-MPMS-101 for material descriptions and casing blow counts up to 25.0 ft bgs.	
5											
10											
15											
20											
25											

Remarks:

Hammer #367

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Kingman Road Bridge #5021 carries Route 170 over Macwahoc Stream
Location: Macwahoc Plantation, Maine

Boring No.: BB-MPMS-101A
WIN: 22250.00

Driller:	S.W. Cole	**Elevation (ft.)**	314.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Nathan Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30.0"
Date Start/Finish:	4/4/2022-4/5/2022	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	9+45.1, 16.5 ft Lt.	**Casing ID/OD:**	HW(4.0"/4.5"), NW(3.0"/3.5")	**Water Level*:**	None Observed
Hammer Efficiency Factor: 0.91		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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WOH = Weight of 140 lb. Hammer
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q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
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WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: 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	1D	24/9	25.00 - 27.00	6/5/5/5	10	15		289.3		Grey, wet, medium dense, GRAVEL, trace sand, trace silt, (Glacial Stream Deposit). 25.0	
30											
35	2D	24/5	34.50 - 36.50	26/14/10/9	24	36				Grey-brown, wet, dense, SAND, some gravel, trace silt, (Glacial Stream Deposit).	G#241514 A-1-b, SW-SM WC=10.6%
40	3D	24/6	40.00 - 42.00	15/7/5/4	12	18				Grey, wet, medium dense, SAND, trace gravel, trace silt, (Glacial Stream Deposit).	G#241515 A-1-b, SP WC=22.4%
45							85				
							150	267.6		46.7	
							141			Boulder from 46.7-47.7 ft bgs. Roller Coned ahead to 50.0 ft bgs.	
							200				
50							OPEN				

Remarks:

Hammer #367

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Kingman Road Bridge #5021 carries Route 170 over Macwahoc Stream
Location: Macwahoc Plantation, Maine

Boring No.: BB-MPMS-101A
WIN: 22250.00

Driller:	S.W. Cole	**Elevation (ft.)**	314.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Nathan Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30.0"
Date Start/Finish:	4/4/2022-4/5/2022	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	9+45.1, 16.5 ft Lt.	**Casing ID/OD:**	HW(4.0"/4.5"), NW(3.0"/3.5")	**Water Level*:**	None Observed
Hammer Efficiency Factor: 0.91		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
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WOH = Weight of 140 lb. Hammer
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$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
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N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_V = Pocket Torvane Shear Strength (psf)
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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	4D	6/6	50.00 - 50.50	100(6")	---		HOLE			Grey, wet, hard, SILT, some gravel (Rock Chips), trace sand, (Glacial Till). Occasional cobble.	
55	MD	2/0	55.00 - 55.17	75(2")	---						
	R1	4.8/4.8	57.30 - 57.70	RQD = 100%	---		NQ-2	257.0		57.3 Top of Bedrock at Elev. 257.0 ft. R1: Bedrock: Grey, fine-grained, PHYLLITE, moderately soft, fresh, joints steeply dipping, annealed with calcite. [Lawler Ridge Formation] Rock Quality = Excellent R1: Core Times (min:sec) 57.3-57.7 ft (5:11) Core Blocked, little wash return when coring, set in NW Casing. 100% Recovery	
	~~5D~~	~~1.5/1.5~~	~~57.40 - 57.53~~	~~105(1.5")~~						(5D) Grey, wet, very dense, Rock Chips, trace silt.	
	R2	60/53	57.70 - 62.70	RQD = 58%						R2: Bedrock: Similar to R1, but slightly weathered and with iron staining on joint faces, then grey, fine to medium-grained, GRAYWACKE, moderately hard, fresh, joints moderately dipping, closely spaced. [Lawler Ridge Formation] Rock Quality = Fair R2: Core Times (min:sec) 57.7-58.7 ft (3:16) 58.7-59.7 ft (3:01) 59.7-60.7 ft (4:06) 60.7-61.7 ft (2:43) 61.7-62.7 ft (2:22) some dirty water 88% Recovery	
60											
	R3	60/60	62.70 - 67.70	RQD = 42%				246.6		R3: Bedrock: Grey, fine to medium-grained, GRAYWACKE, moderately hard, fresh, then Grey, fine-grained, PHYLLITE, moderately soft, highly fractured, slightly weathered, iron staining on joint faces, steep joint sets, undulating, with significant quartz/calcite annealed joint, then Grey, fine to medium-grained, GRAYWACKE, moderately hard, slightly weathered. [Lawler Ridge Formation] Rock Quality = Poor R3: Core Times (min:sec) 62.7-63.7 ft (2:46) 63.7-64.7 ft (2:05) 64.7-65.7 ft (2:14) 65.7-66.7 ft (2:38) 66.7-67.7 ft (3:02)	
65											
70											
75											

Remarks:

Hammer #367

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Kingman Road Bridge #5021 carries Route 170 over Macwahoc Stream
Location: Macwahoc Plantation, Maine

Boring No.: BB-MPMS-101A
WIN: 22250.00

Driller:	S.W. Cole	**Elevation (ft.)**	314.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Nathan Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30.0"
Date Start/Finish:	4/4/2022-4/5/2022	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	9+45.1, 16.5 ft Lt.	**Casing ID/OD:**	HW(4.0"/4.5"), NW(3.0"/3.5")	**Water Level*:**	None Observed
Hammer Efficiency Factor: 0.91		**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 Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75										100% Recovery	
										67.7	
										Bottom of Exploration at 67.7 feet below ground surface.	
80											
85											
90											
95											
100											

Remarks:

Hammer #367

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Kingman Road Bridge #5021 carries Route 170 over Macwahoc Stream
Location: Macwahoc Plantation, Maine

Boring No.: BB-MPMS-102
WIN: 22250.00

Driller:	MaineDOT	**Elevation (ft.)**	315.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Brooks	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N.Pukay/B.Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30.0"
Date Start/Finish:	12/7,9-10,13/2021	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	10+41, 7.0 ft Rt.	**Casing ID/OD:**	NW(3.0"/3.5")	**Water Level*:**	10.5 ft bgs.
Hammer Efficiency Factor:	0.974	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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	315.0		4" HMA. 0.3	
5	1D	24/12	5.00 - 7.00	8/20/25/17	45	73				Brown, damp, very dense, Sandy GRAVEL, little silt, (Fill).	
10	2D	24/14	10.00 - 12.00	13/10/16/22	26	42	22			Brown, wet, dense, Sandy GRAVEL, little silt, (Fill).	G#337502 A-1-a, GM WC=12.9%
							57				
							71				
							46				
							51				
15	3D	24/13	15.00 - 17.00	8/10/12/9	22	36	24	300.3		15.0 Brown, wet, dense, Sandy GRAVEL, trace silt, (Glacial Stream Deposit).	G#337503 A-1-a, GW-GM WC=15.4%
							41				
							51				
							40				
							41				
20	4D	24/14	20.00 - 22.00	10/6/10/13	16	26	45			Similar to above, except medium dense.	
							53				
							70				
							44				
25							39				

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Kingman Road Bridge #5021 carries Route 170 over Macwahoc Stream
Location: Macwahoc Plantation, Maine

Boring No.: BB-MPMS-102
WIN: 22250.00

Driller:	MaineDOT	**Elevation (ft.)**	315.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Brooks	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N.Pukay/B.Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30.0"
Date Start/Finish:	12/7,9-10,13/2021	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	10+41, 7.0 ft Rt.	**Casing ID/OD:**	NW(3.0"/3.5")	**Water Level*:**	10.5 ft bgs.
Hammer Efficiency Factor:	0.974	**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	5D	24/9	25.00 - 27.00	15/20/12/26	32	52	42			Brown, wet, very dense, Sandy GRAVEL, trace silt (Glacial Stream Deposit).	
							74				
							86				
							77				
							66				
30	6D	24/15	30.00 - 32.00	7/7/6/7	13	21	22			Brown-grey, wet, medium dense, SAND, little gravel, trace silt, (Glacial Stream Deposit).	G#337504 A-1-b, SP-SM WC=16.1%
							38				
							65				
							82				
							61				
35	7D	24/12	35.00 - 37.00	8/7/8/8	15	24	48			Similar to above, except grey.	
							69				
							61				
							70				
							98				
40	8D	24/14	40.00 - 42.00	13/14/20/12	34	55	48			Grey, wet, very dense, Gravelly, SAND, trace silt, (Glacial Stream Deposit).	
							73				
							71				
							54				
							55				
45	9D	24/14	45.00 - 47.00	4/7/5/5	12	19	55			Grey, wet, medium dense, SAND, trace gravel, trace silt, (Glacial Stream Deposit).	
							67				
							104	268.3		47.0	
							106				
50							126				

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Kingman Road Bridge #5021 carries Route 170 over Macwahoc Stream
Location: Macwahoc Plantation, Maine

Boring No.: BB-MPMS-102
WIN: 22250.00

Driller:	MaineDOT	**Elevation (ft.)**	315.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Brooks	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N.Pukay/B.Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30.0"
Date Start/Finish:	12/7,9-10,13/2021	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	10+41, 7.0 ft Rt.	**Casing ID/OD:**	NW(3.0"/3.5")	**Water Level*:**	10.5 ft bgs.
Hammer Efficiency Factor:	0.974	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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WOH = Weight of 140 lb. Hammer
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WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
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Hammer Efficiency Factor = Rig Specific Annual Calibration Value
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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_V = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
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G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: 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	10D	24/17	50.00 - 52.00	7/9/24/31	33	54	OPEN HOLE			Brown-grey, wet, very dense, SAND, some gravel, little silt, trace clay, (Glacial Till). Roller Coned ahead to 55.0 ft bgs.	G#337505 A-1-b, SC-SM WC=9.6%
										Cobble from 52.5-52.8 ft bgs.	
55	11D	24/15	55.00 - 57.00	34/28/40/50	68	110				Brown-grey, wet, very dense, SAND, some gravel, some silt, (Glacial Till). Roller Coned ahead to 60.0 ft bgs.	
										Cobble from 57.0-57.5 ft bgs.	
60	12D	8.4/8.4	60.00 - 60.70	41/60 (2.4")	---					Similar to above. Roller Coned ahead to 65.0 ft bgs. Occasional cobble.	
65	13D	1.2/1.2	65.00 - 65.10	55(1.2")	---					Brown-grey, wet, very dense, SAND, some silt, little gravel, (Glacial Till). Roller Coned ahead to 70.0 ft bgs.	
70	MD	1.2/0	70.00 - 70.10	70(1.2")	---					Roller Coned ahead to 75.0 ft bgs.	
75											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Kingman Road Bridge #5021 carries Route 170 over Macwahoc Stream
Location: Macwahoc Plantation, Maine

Boring No.: BB-MPMS-102
WIN: 22250.00

Driller:	MaineDOT	**Elevation (ft.)**	315.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Brooks	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N.Pukay/B.Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30.0"
Date Start/Finish:	12/7,9-10,13/2021	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	10+41, 7.0 ft Rt.	**Casing ID/OD:**	NW(3.0"/3.5")	**Water Level*:**	10.5 ft bgs.
Hammer Efficiency Factor:	0.974	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75	14D	1.2/1.2	75.00 - 75.10	100(1.2")	---					Grey, moist, hard, Sandy SILT, little gravel, (Glacial Till). Roller coned ahead to 80.0 ft bgs.	
80	15D	1.2/1.2	80.00 - 80.10	70(1.2")	---					Grey, wet, hard, Sandy SILT, some gravel, (Glacial Till). Roller Coned ahead to 89.3 ft bgs.	
85											
								226.3		89.0	
	R1	60/60	89.30 - 94.30	RQD = 20%			NQ-2			Top of Bedrock at Elev. 226.3 ft. R1: Bedrock: Grey, medium-grained, GRAYWACKE, with calcite veins, moderately hard, slightly weathered, joint sets dipping at moderate to vertical angles, tight. [Lawler Ridge Formation] Rock Quality = Very Poor R1: Core Times (min:sec) 89.3-90.3 ft (2:43) 90.3-91.3 ft (2:49) 91.3-92.3 ft (3:05) 92.3-93.3 ft (2:22) 93.3-94.3 ft (2:45) 100% Recovery	
90											
95	R2	60/48	94.30 - 99.30	RQD = 7%						R2: Bedrock: Top 10" similar to R1, then grey, fine-grained, iron-stained, PHYLLITE to SILTSTONE, with highly fractured, weathered and soft zones. [Lawler Ridge Formation] Rock Quality = Very Poor R2: Core Times (min:sec) 94.3-95.3 ft (2:52) 95.3-96.3 ft (2:47) 96.3-97.3 ft (3:20) dirty brown water 97.3-98.3 ft (5:10)	
								216.0			
100											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Kingman Road Bridge #5021 carries Route 170 over Macwahoc Stream
Location: Macwahoc Plantation, Maine

Boring No.: BB-MPMS-102
WIN: 22250.00

Driller:	MaineDOT	**Elevation (ft.)**	315.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Brooks	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N.Pukay/B.Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30.0"
Date Start/Finish:	12/7,9-10,13/2021	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	10+41, 7.0 ft Rt.	**Casing ID/OD:**	NW(3.0"/3.5")	**Water Level*:**	10.5 ft bgs.
Hammer Efficiency Factor:	0.974	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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N-uncorrected = Raw Field SPT N-value
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Depth (ft.)	Sample Information: 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.
100										98.3-99.3 ft (9:30) had to work the bit, added time. 80% Recovery 99.3 **Bottom of Exploration at 99.3 feet below ground surface.**	
105											
110											
115											
120											
125											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Kingman Road Bridge #5021 carries Route 170 over Macwahoc Stream
Location: Macwahoc Plantation, Maine

Boring No.: BB-MPMS-201
WIN: 22250.00

Driller:	S.W. Cole	**Elevation (ft.)**	315.7	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N.Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30.0"
Date Start/Finish:	3/24/2023; 08:15-14:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	9+52.1, 6.7 ft Rt.	**Casing ID/OD:**	HW(4.0"/4.5")	**Water Level*:**	9.0 ft bgs.
Hammer Efficiency Factor: 0.92		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	315.1		7" HMA. 0.6	
5	1D	24/4	5.00 - 7.00	2/6/18/8	24	37				Brown, damp, dense, SAND, some gravel, little silt, (Fill).	
							15				
							35				
							27				
							17				
10	MD	24/0	10.00 - 12.00	3/2/2/2	4	6	14			Loose.	
							12				
							15				
							72	302.7		13.0	
							80			Dark brown organics and wood chips in wash water from 13.0-14.5 ft bgs.	
15	2D	24/1	15.00 - 17.00	18/12/8/5	20	31	37			Grey, wet, dense, SAND, little silt, little gravel, wood chips, (Glacial Stream Deposit). Wood chips in wash water from 15.0-16.0 ft bgs.	
							42				
							34				
							39				
							42				
20	3D	24/5	20.00 - 22.00	4/5/6/8	11	17	27			Grey, wet, medium dense, SAND, little silt, trace gravel, (Glacial Stream Deposit).	
							35				
							58				
							77				
25							100				

Remarks:

Auto Hammer #367

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Kingman Road Bridge #5021 carries Route 170 over Macwahoc Stream
Location: Macwahoc Plantation, Maine

Boring No.: BB-MPMS-201
WIN: 22250.00

Driller:	S.W. Cole	**Elevation (ft.)**	315.7	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N.Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30.0"
Date Start/Finish:	3/24/2023; 08:15-14:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	9+52.1, 6.7 ft Rt.	**Casing ID/OD:**	HW(4.0"/4.5")	**Water Level*:**	9.0 ft bgs.
Hammer Efficiency Factor:	0.92	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
				Sample Information							
25	4D	24/10	25.00 - 27.00	10/11/15/16	26	40	66			Grey, wet, dense, SAND, little silt, trace gravel, (Glacial Stream Deposit).	
							102				
							94			Occasional Cobble	
							71				
							70				
30	5D	24/4	30.00 - 32.00	12/7/7/16	14	21	60			Grey, wet, medium dense, SAND, some gravel, trace silt, (Glacial Stream Deposit).	
							69				
							88				
							81				
							76				
35	6D	24/2	35.00 - 37.00	11/13/7/8	20	31	66			Grey, wet, dense, GRAVEL, little sand, trace silt, (Glacial Stream Deposit).	
							88				
							71				
							88				
							72				
40	7D	24/1	40.00 - 42.00	7/6/7/10	13	20	69			Similar to 6D, except medium dense.	
							80				
							86				
							81				
							84				
45	8D	24/2	45.00 - 47.00	8/6/5/4	11	17	90			Grey, wet, medium dense, SAND, trace gravel, trace silt, (Glacial Stream Deposit).	
							100				
							103	268.7		47.0	
							97				
50							181				

Remarks:

Auto Hammer #367

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Soil/Rock Exploration Log
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Location: Macwahoc Plantation, Maine

Boring No.: BB-MPMS-201
WIN: 22250.00

Driller:	S.W. Cole	**Elevation (ft.)**	315.7	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N.Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30.0"
Date Start/Finish:	3/24/2023; 08:15-14:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	9+52.1, 6.7 ft Rt.	**Casing ID/OD:**	HW(4.0"/4.5")	**Water Level*:**	9.0 ft bgs.
Hammer Efficiency Factor: 0.92		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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	9D	24/12	50.00 - 52.00	16/14/29/42	43	66	69			Grey, wet, very dense, Silty GRAVEL (bedrock chips), little sand, (Glacial Till) .	
							90				
	R1	30/30	52.60 - 55.10	RQD = 0%			[a]101 NQ-2	263.1		[a]101 blows for 0.6 ft. 52.6 Top of Bedrock at Elev. 263.1 ft. R1: Bedrock: Grey, fine-grained, PHYLLITE, moderately soft, slightly weathered, highly fractured zones, joints are vertical with iron staining and rock flour. [Lawler Ridge Formation] Rock Quality = Very Poor R1: Core Times (min:sec) 52.6-53.6 ft (2:10) 53.6-54.6 ft (2:43) 54.6-55.1 ft (1:51) Core Blocked 100% Recovery	
55	R2	19.2/15	55.10 - 56.70	RQD = 0%						R2: Bedrock: Similar to R1, except moderately weathered and highly fractured throughout run. [Lawler Ridge Formation] Rock Quality = Very Poor R2: Core Times (min:sec) 55.1-55.6 ft (0:59) 55.6-56.6 ft (3:03) 56.6-56.7 ft (0:35) Core Blocked 79% Recovery	
	R3	52.8/25	56.70 - 61.10	RQD = 0%							
60	R4	18/18	61.10 - 62.60	RQD = 44%				253.1		R3: Bedrock: Grey, fine-grained, GRAYWACKE, moderately hard, slightly weathered, highly fractured, joints are steeply dipping with iron staining, then dark-grey, fine to medium-grained, GRAYWACKE, moderately hard, fresh, highly fractured, joints steeply dipping, with several thin, low angle joints annealed with quartz or calcite. [Lawler Ridge Formation] Rock Quality = Very Poor R3: Core Times (min:sec) 56.7-57.6 ft (2:35) 57.6-58.6 ft (2:34) 58.6-59.6 ft (5:20) 59.6-60.6 ft (6:22) 60.6-61.1 ft (3:40) Core Blocked 47% Recovery	
65											
70										R4: Bedrock: Grey to dark-grey, fine to medium-grained, GRAYWACKE, moderately hard, fresh, joint sets dipping at steep angles, closely spaced, undulating. [Lawler Ridge Formation] Rock Quality = Poor. R4: Core Times (min:sec) 61.1-61.6 ft (2:14) 61.6-62.6 ft (4:04) 100% Recovery	
										62.6 **Bottom of Exploration at 62.6 feet below ground surface.**	
75											

Remarks:

Auto Hammer #367

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: Kingman Road Bridge #5021 carries Route 170 over Macwahoc Stream
Location: Macwahoc Plantation, Maine

Boring No.: BB-MPMS-202
WIN: 22250.00

Driller:	S.W. Cole	**Elevation (ft.)**	315.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N.Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30.0"
Date Start/Finish:	3/27/2023-3/28/2023	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	10+38.1, 5.5 ft Lt.	**Casing ID/OD:**	HW(4.0"/4.5"), NW(3.0"/3.5")	**Water Level*:**	9.0 ft bgs.
Hammer Efficiency Factor: 0.92		**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	314.8		7" HMA. 0.6	
5	1D	24/3	5.00 - 7.00	6/10/12/17	22	34				Brown, damp, dense, Gravelly SAND, little silt, (Fill).	
							30				
							40				
10	2D	24/16	10.00 - 12.00	11/11/8/9	19	29	47			Brown, damp, medium dense, Gravelly SAND, little silt, trace organics, (Fill).	
							60				
							59				
							73				
							93				
15	3D	18/4	15.00 - 16.50	12/22/83	105	161	38	300.4		15.0 Brown, wet, very dense, Sandy GRAVEL, trace silt, (Glacial Stream Deposit). Cobble from 15.9-16.5 ft bgs. Roller Coned ahead to 17.0 ft bgs.	
							102				
							169				
							111				
							84				
20	MD	24/0	20.00 - 22.00	14/9/11/14	20	31	37			Dense.	
							53				
							69				
							51				
25							50				

Remarks:

Auto Hammer #367

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Kingman Road Bridge #5021 carries Route 170 over Macwahoc Stream
Location: Macwahoc Plantation, Maine

Boring No.: BB-MPMS-202
WIN: 22250.00

Driller:	S.W. Cole	**Elevation (ft.)**	315.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N.Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30.0"
Date Start/Finish:	3/27/2023-3/28/2023	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	10+38.1, 5.5 ft Lt.	**Casing ID/OD:**	HW(4.0"/4.5"), NW(3.0"/3.5")	**Water Level*:**	9.0 ft bgs.
Hammer Efficiency Factor: 0.92		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	4D	24/2	25.00 - 27.00	9/10/14/15	24	37	49			Grey-brown, wet, dense, Sandy GRAVEL, trace silt, (Glacial Stream Deposit).	
							51				
							89				
							88				
							91				
30	5D	24/5	30.00 - 32.00	18/9/8/7	17	26	55			Similar to 4D, except medium dense.	
							32				
							59				
							74				
							53				
35	6D	24/1	35.00 - 37.00	6/11/28/13	39	60	20			Grey, wet, very dense, Sandy GRAVEL, trace silt, (Glacial Stream Deposit).	
							77				
							134				
							148				
							160				
40	7D	24/4	40.00 - 42.00	43/17/8/7	25	38	79			Grey-brown, wet, dense, Sandy GRAVEL, trace silt, (Glacial Stream Deposit).	
							61				
							97				
							104	272.4		43.0	
							102				
45	8D	24/7	45.00 - 47.00	20/30/36/36	66	101	84			Grey, wet, very dense, Sandy GRAVEL, little silt, (Glacial Till). Roller Coned ahead to 48.0 ft bgs.	
							168				
							190				
							266				
50							95				

Remarks:

Auto Hammer #367

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: Kingman Road Bridge #5021 carries Route 170 over Macwahoc Stream
Location: Macwahoc Plantation, Maine

Boring No.: BB-MPMS-202
WIN: 22250.00

Driller:	S.W. Cole	**Elevation (ft.)**	315.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N.Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30.0"
Date Start/Finish:	3/27/2023-3/28/2023	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	10+38.1, 5.5 ft Lt.	**Casing ID/OD:**	HW(4.0"/4.5"), NW(3.0"/3.5")	**Water Level*:**	9.0 ft bgs.
Hammer Efficiency Factor: 0.92		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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SSA = Solid Stem Auger
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S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_V = Pocket Torvane Shear Strength (psf)
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	9D	24/2	50.00 - 52.00	26/36/43/57	79	121	139			Grey, wet, very dense, GRAVEL, little sand, trace silt, (Glacial Till). Roller Coned ahead to 55.0 ft bgs.	
							103				
							166				
							176				
							102				
55	10D	16.8/10	55.00 - 56.40	46/47/52(4.8")	---		[a]79 Spin NW			Grey, wet, very dense, Sandy GRAVEL, little silt, (Glacial Till). [a]79 blows for 0.5 ft. Roller Coned ahead to 59.3 ft bgs. Rock chips in wash water. Set in NW Casing.	
	R1	60/50	59.30 - 64.30	RQD = 67%			NQ-2	256.1		59.3 Top of Bedrock at Elev. 256.1 ft. R1: Bedrock: Grey to dark-grey, fine to medium-grained, GRAYWACKE, moderately hard, fresh, joint sets dipping at moderate angles, closely spaced, iron stained. [Lawler Ridge Formation] Rock Quality = Fair R1: Core Times (min:sec) 59.3-60.3 ft (3:32) 60.3-61.3 ft (2:56) 61.3-62.3 ft (3:40) 62.3-63.3 ft (3:18) 63.3-64.3 ft (4:47) 83% Recovery	
60											
	R2	60/60	64.30 - 69.30	RQD = 70%						R2: Bedrock: Grey to dark-grey, fine to medium-grained, GRAYWACKE, moderately hard, fresh, interlayered with zones of fine-grained PHYLLITE, moderately soft, fresh, joint sets dipping at moderate to steep angles, significant quartz/calcite along joint faces. [Lawler Ridge Formation] Rock Quality = Fair R2: Core Times (min:sec) 64.3-65.3 ft (3:05) 65.3-66.3 ft (2:58) 66.3-67.3 ft (2:17) 67.3-68.3 ft (2:31) 69.3-69.3 ft (2:25) 100% Recovery	
65											
								246.1		69.3 **Bottom of Exploration at 69.3 feet below ground surface.**	
70											
75											

Remarks:

Auto Hammer #367

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

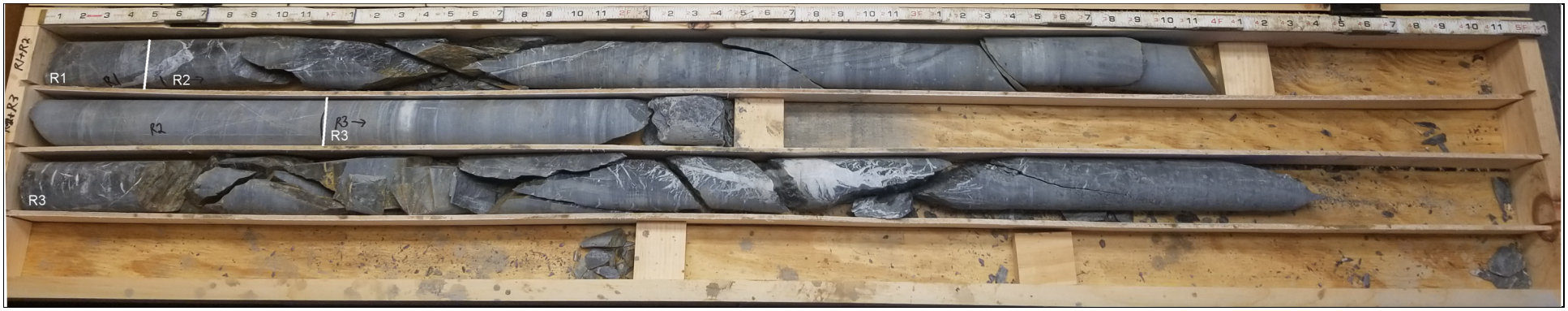
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Appendix B

Rock Core Photographs

MaineDOT
Kingman Road Bridge #5021 Carries Route 170 Over Macwahoc Stream
Macwahoc Plantation, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-MPMS-101A	R1	57.3-57.7	4.8	4.8	4.8	100	PHYLLITE	1
BB-MPMS-101A	R2	57.7-62.7	60	53	35	58	PHYLLITE/GRAYWACKE	1+2
BB-MPMS-101A	R3	62.7-67.7	60	60	25	42	PHYLLITE/GRAYWACKE	2+3



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

MaineDOT
Kingman Road Bridge #5021 Carries Route 170 Over Macwahoc Stream
Macwahoc Plantation, ME
Rock Core Photographs

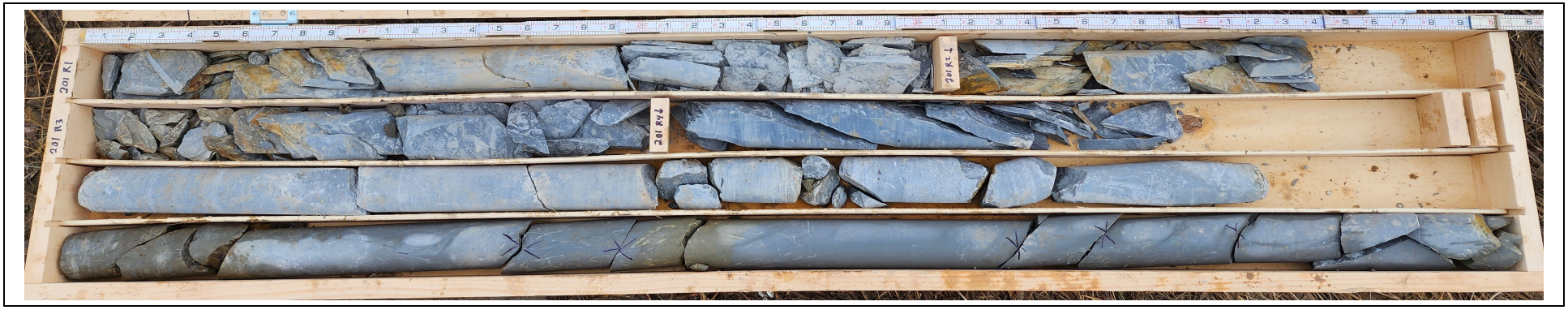
Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-MPMS-102	R1	89.3-94.3	60	60	12	20	GRAYWACKE	1
BB-MPMS-102	R2	94.3-99.3	60	48	4	7	GRAYWACKE/PHYLLITE/SILTSTONE	2



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

MaineDOT
Kingman Road Bridge #5021 Carries Route 170 Over Macwahoc Stream
Macwahoc Plantation, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-MPMS-201	R1	52.6-55.1	30	30	0	0	PHYLLITE	1
BB-MPMS-201	R2	55.1-56.7	19.2	15	0	0	PHYLLITE	1
BB-MPMS-201	R3	56.7-61.1	52.8	25	0	0	GRAYWACKE	2
BB-MPMS-201	R4	61.1-62.6	18	18	8	44	GRAYWACKE	2
BB-MPMS-202	R1	59.3-64.3	60	50	40	67	GRAYWACKE	3
BB-MPMS-202	R2	64.3-69.3	60	60	42	70	GRAYWACKE/PHYLLITE	4



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

Appendix C

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Macwahoc Plt. Work Number: 22250.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-MPMS-101, 1D	9+46	15.0 Lt.	5.0-7.0	241510	2	12.9			SM	A-1-b	II
BB-MPMS-101, 3D	9+46	15.0 Lt.	15.0-17.0	241511	2	19.0			SP-SM	A-3	0
BB-MPMS-101, 4D	9+46	15.0 Lt.	20.0-22.0	241512	2	19.7			SP-SM	A-2-4	0
BB-MPMS-101, 5D	9+46	15.0 Lt.	32.0-34.0	241513	2	11.0			SW-SM	A-1-a	0
BB-MPMS-101A, 2D	9+45.1	16.5 Lt.	34.5-36.5	241514	2	10.6			SW-SM	A-1-b	0
BB-MPMS-101A, 3D	9+45.1	16.5 Lt.	40.0-42.0	241515	2	22.4			SP	A-1-b	0
BB-MPMS-102, 2D	10+41	7.0 Rt.	10.0-12.0	337502	1	12.9			GM	A-1-a	I
BB-MPMS-102, 3D	10+41	7.0 Rt.	15.0-17.0	337503	1	15.4			GW-GM	A-1-a	0
BB-MPMS-102, 6D	10+41	7.0 Rt.	30.0-32.0	337504	1	16.1			SP-SM	A-1-b	0
BB-MPMS-102, 10D	10+41	7.0 Rt.	50.0-52.0	337505	1	9.6			SC-SM	A-1-b	II

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.

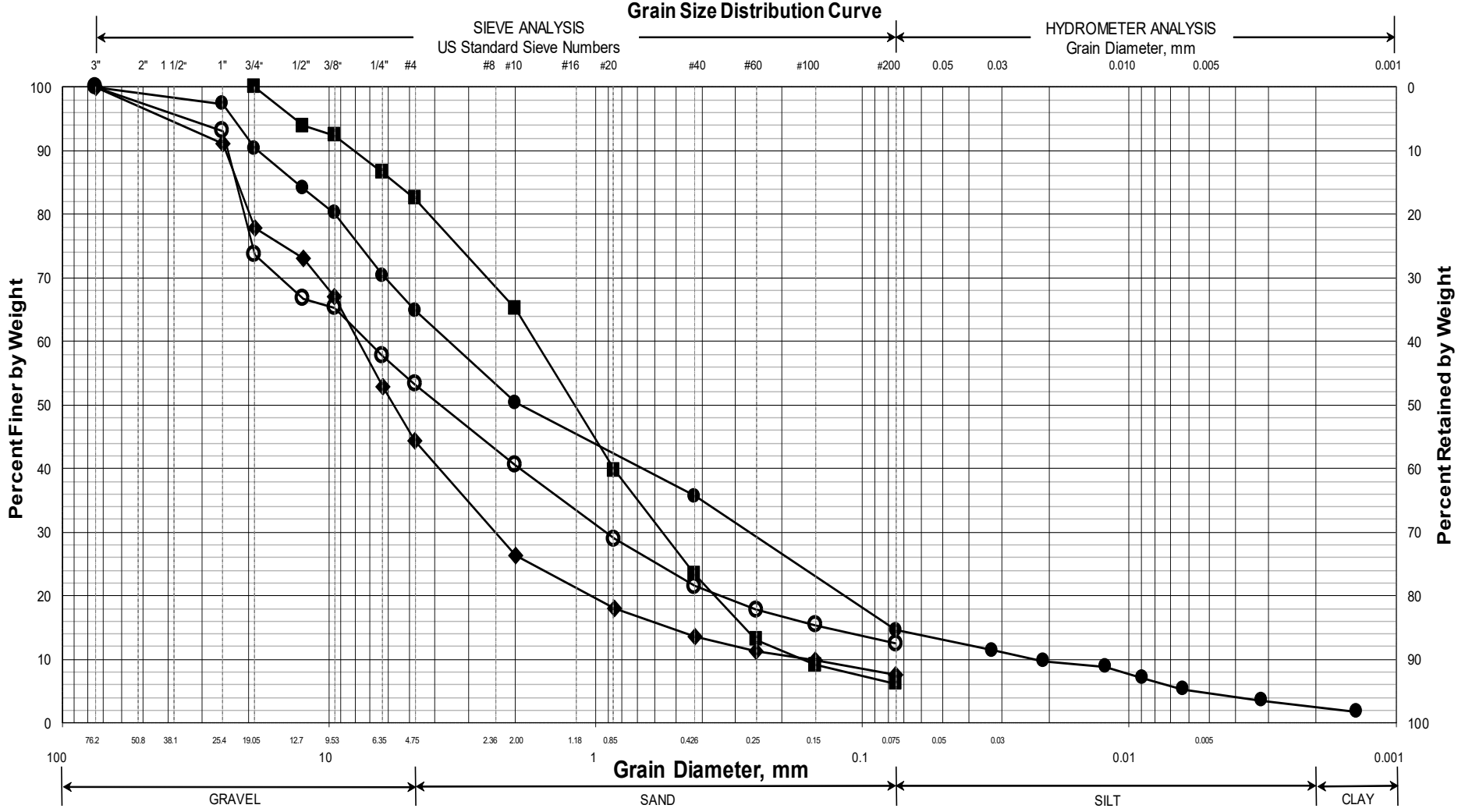
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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



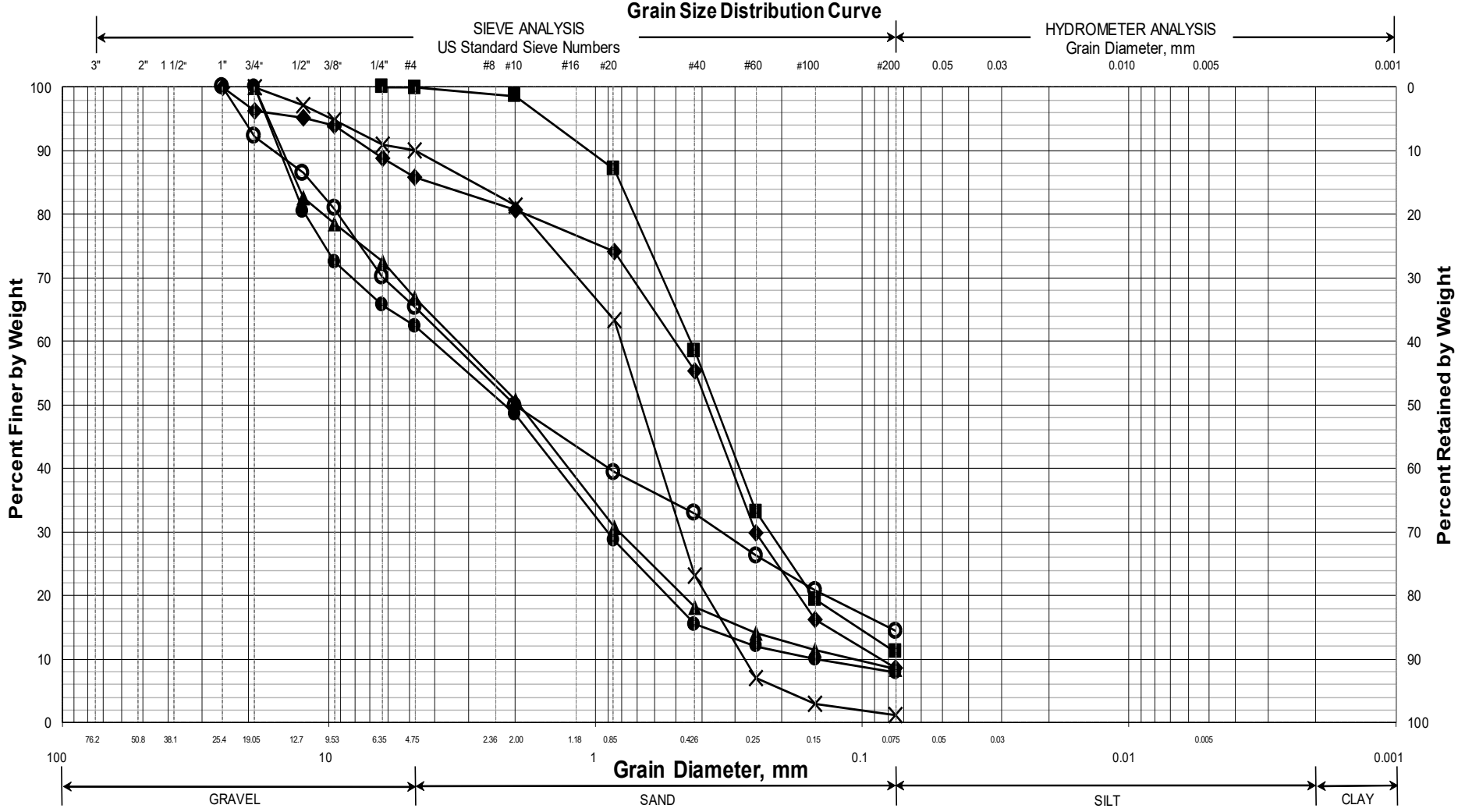
UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-MPMS-102/2D	10+41	7.0 RT	10.0-12.0	Sandy GRAVEL, little silt.	12.9			
◆	BB-MPMS-102/3D	10+41	7.0 RT	15.0-17.0	Sandy GRAVEL, trace silt.	15.4			
■	BB-MPMS-102/6D	10+41	7.0 RT	30.0-32.0	SAND, little gravel, trace silt.	16.1			
●	BB-MPMS-102/10D	10+41	7.0 RT	50.0-52.0	SAND, some gravel, little silt, trace clay.	9.6			
▲									
×									

WIN
022250.00
Town
Macwahoc Plt
Reported by/Date
WHITE, TERRY A 1/24/2022

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-MPMS-101/1D	9+46	15.0 LT	5.0-7.0	SAND, some gravel, little silt.	12.9			
◆	BB-MPMS-101/3D	9+46	15.0 LT	15.0-17.0	SAND, little gravel, trace silt.	19			
■	BB-MPMS-101/4D	9+46	15.0 LT	20.0-22.0	SAND, litte silt, trace gravel.	19.7			
●	BB-MPMS-101/5D	9+46	15.0 LT	32.0-34.0	Gravelly SAND, trace silt.	11			
▲	BB-MPMS-101A/2D	9+45.1	16.5 LT	34.5-36.5	SAND, some gravel, trace silt.	10.6			
×	BB-MPMS-101A/3D	9+45.1	16.5 LT	40.0-42.0	SAND, trace gravel, trace silt.	22.4			

WIN
022250.00
Town
Macwahoc Plt
Reported by/Date
WHITE, TERRY A 5/16/2022

Appendix D

Calculations

Driven H-Pile Resistance

Design of H-piles

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

Bedrock Properties

BB-MPMS-101A, R1 RQD = 100%, R2 RQD = 58%, R3 RQD = 42%
Rock Type: SANDSTONE, hard, fresh

BB-MPMS-102, R1 RQD = 20%, R2 RQD = 7%
Rock Type: SANDSTONE (R1 and top 10" of R2), hard, slightly weathered
PHYLLITE TO SILTSTONE (R2 bottom 38"), soft, highly weathered

BB-MPMS-201, R1 RQD = 0%, R2 RQD = 0%, R3 = RQD = 0%
Rock Type: SANDSTONE, moderately hard, moderately weathered

BB-MPMS-202, R1 RQD = 67%, R2 RQD = 70%
Rock Type: SANDSTONE, moderately hard, fresh

ϕ = 27-34 (AASHTO LRFD Table C.10.4.6.4-1);

Sandstone Co = 9,700-25,000 psi
Siltstone Co = 1,400-17,000 psi
Phyllite Co = 3,500-35,000 psi

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

For Design Purposes: RQD = 20%, Co = 8000 psi

Pile Properties

Use the following piles: 14x89, 14x117

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

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

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

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

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

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

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

Check For Slender Members

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

LRFD eq. 6.9.4.2.1-1

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

From Table 6.9.4.2.1-1:

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

$\lambda_{rf} = 13.487$ $\quad b_f := 0.5 \cdot b$ $\quad b_f = \begin{pmatrix} 7.35 \\ 7.45 \end{pmatrix} \cdot in$

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

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

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

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

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

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

Nominal Axial Structural Resistance

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

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

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

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

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

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

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

LRFD eq. 6.9.4.1.2-1

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

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

$$\frac{P_o}{P_e} = \begin{pmatrix} 8.529 \times 10^{-6} \\ 8.247 \times 10^{-6} \end{pmatrix}$$ If Po/Pe < or = 2.25, then: $$P_n := \left(0.658^{\frac{P_o}{P_e}} \cdot P_o \right) \rightarrow$$ LRFD Eq. 6.9.4.1.1-1

then:

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

Factored Axial Structural Resistance for the Strength Limit State

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

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

Factored structural compressive resistance, P_r $$P_r = \begin{pmatrix} 652 \\ 860 \end{pmatrix} \cdot kip$$

LRFD 10.7.3.2.3 - Piles Driven to Hard Rock -

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

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

Nominal Structural Resistance Previously Calculated:

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

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

$$\phi_c := 0.5$$

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

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

14x89
14x117

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

$$\phi_c := 1.0$$ LRFD 6.5.5

$$P_{r_ee} := \phi_c \cdot P_n$$

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

14x89
14x117

Drivability Analyses

Ref: LRFD Article 10.7.8

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

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

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

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

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

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

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

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

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

GRLWeap Soil and Pile Model Assumptions

Abutment #1:
Based on proposed bottom of footing of elevation 305.5 at abutment #1, the estimated pile length will be approx. 48.5 feet. Assume contractor drives pile lengths of 55 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. 150 kips as skin friction based on local experience in similar deposits.

Abutment #2:
Based on proposed bottom of footing of elevation 304.5 at abutment #2, the estimated pile length will vary from 48.4 feet (BB-MPMS-202, left of centerline) to 78.2 feet (BB-MPMS-102, right of centerline). Assume contractor drives pile lengths of 55 ft to 85 ft (extra length accommodates for attachment of dynamic testing equipment, embedment into abutment, variation in bedrock surface).

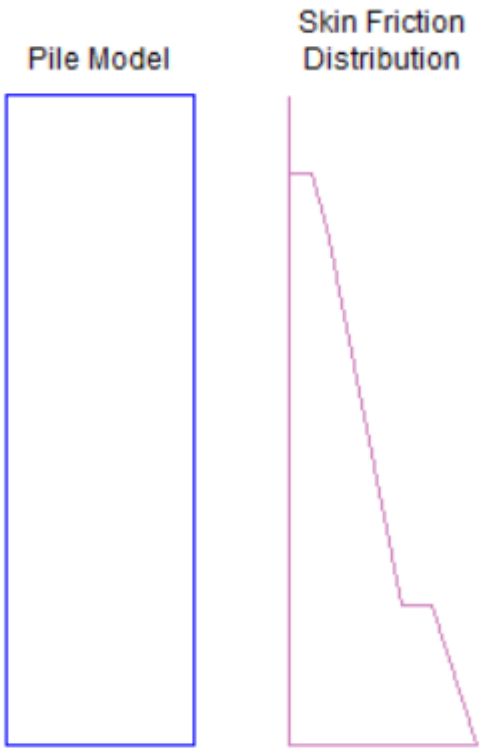
Use constant shaft resistances so that GRLWeap will assign approx. 250 kips as skin friction based on local experience in similar deposits.

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

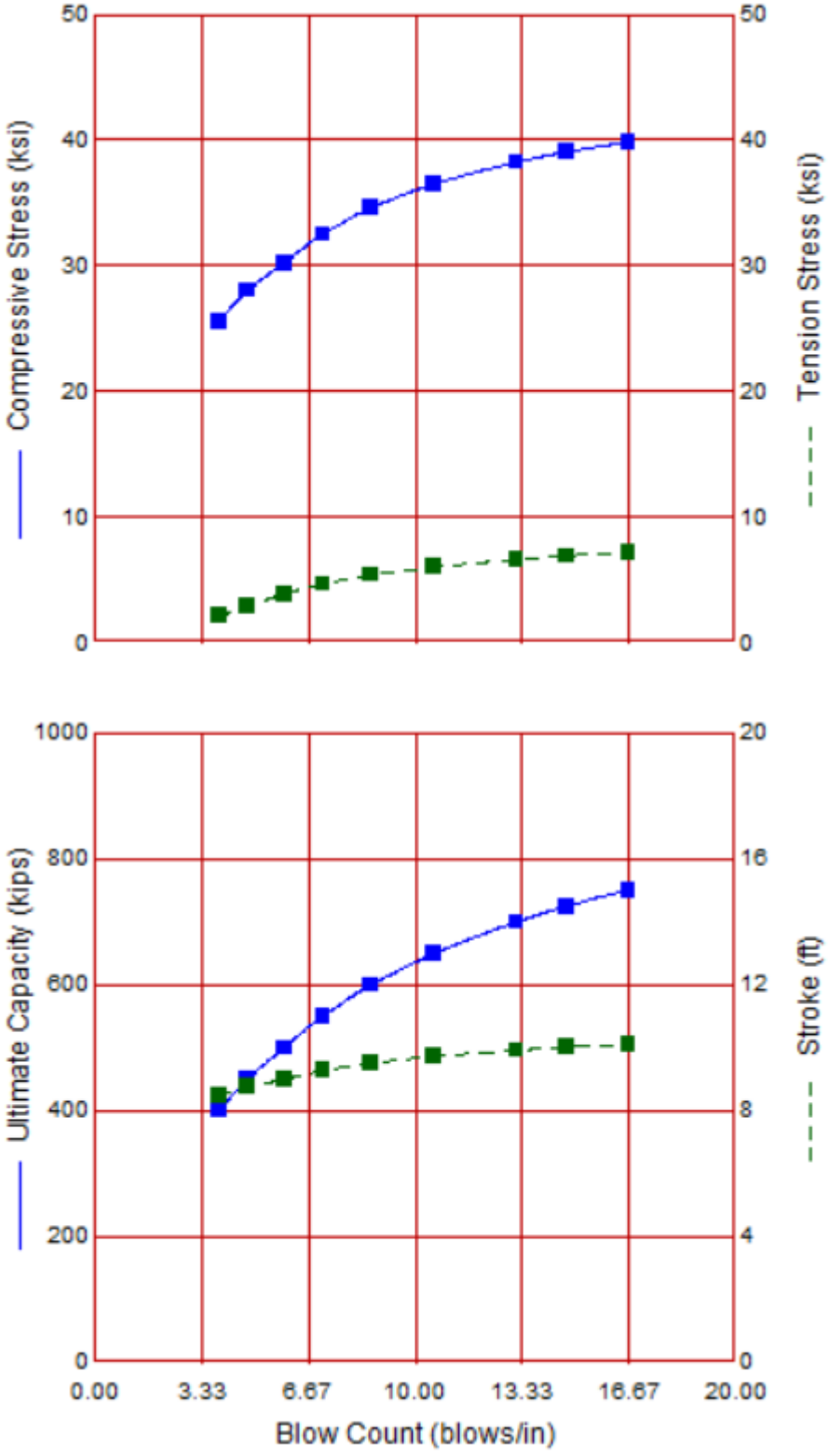
The 14x89 pile can be driven to the resistances below with an APE D19-42 hammer at fuel setting 4 (100% of Max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE D 19-42

Ram Weight	4.19	kips
Efficiency	0.800	
Pressure	1710 (100%)	psi
Helmet Weight	3.00	kips
Hammer Cushion	34825	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	55.00	ft
Pile Penetration	48.50	ft
Pile Top Area	26.10	in2



Res. Shaft = 152.0 kips
(Constant Res. Shaft)


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

Maine DOT
22250 Macwahoc 14x89 ABT #1 D19-42

20-Sep-2023
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	25.52	2.16	3.9	8.45	21.16
450.0	28.02	2.94	4.8	8.77	21.89
500.0	30.21	3.94	5.9	8.99	22.47
550.0	32.51	4.73	7.1	9.28	23.22
600.0	34.61	5.46	8.6	9.52	23.88
650.0	36.52	6.05	10.6	9.73	24.39
700.0	38.24	6.67	13.2	9.93	24.92
725.0	39.06	6.97	14.7	10.02	25.19
750.0	39.81	7.26	16.7	10.11	25.40

Limit to 15 bpi

$$R_{ndr} := 725 \cdot kip$$

Strength Limit State

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

$$R_{fdr} = 471 \cdot kip$$

Extreme and
Service Limit States

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

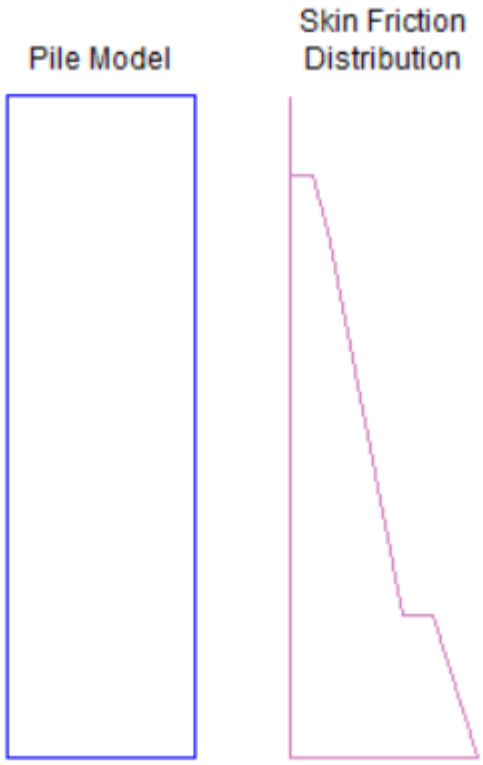
$$R_{dr} = 725 \cdot kip$$

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

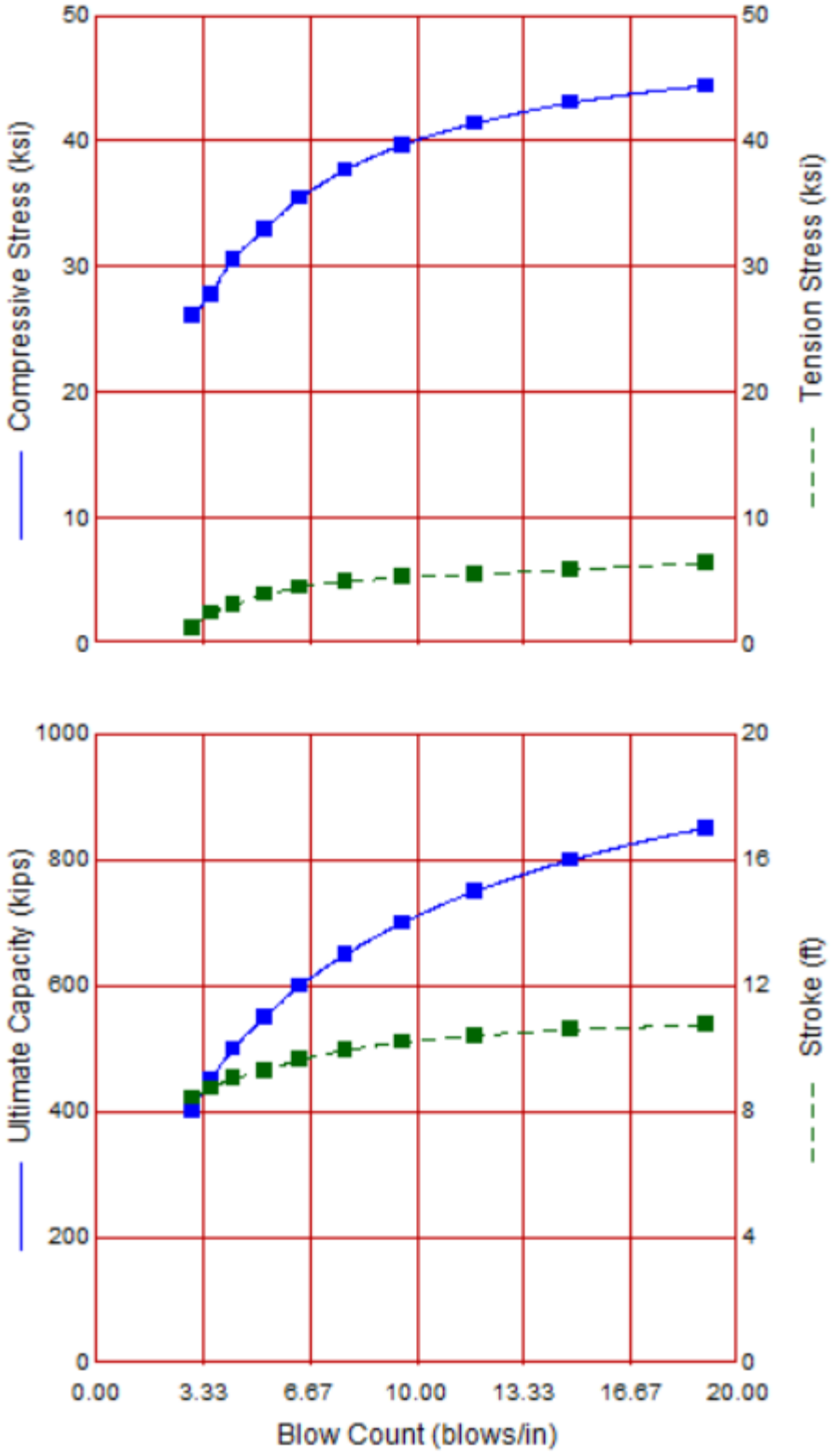
The 14x89 pile can be driven to the resistances below with a APE D25-42 hammer at fuel setting 4 (100% of max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1425 (100%)	psi
Helmet Weight	3.00	kips
Hammer Cushion	34825	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	55.00	ft
Pile Penetration	48.50	ft
Pile Top Area	26.10	in2



Res. Shaft = 152.0 kips
(Constant Res. Shaft)


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

Maine DOT
22250 Macwahoc 14x89 ABT #1 D25-42

20-Sep-2023
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	26.08	1.25	3.0	8.43	24.48
450.0	27.84	2.46	3.6	8.74	25.21
500.0	30.57	3.09	4.3	9.05	25.95
550.0	32.97	3.96	5.2	9.30	26.76
600.0	35.44	4.52	6.3	9.65	27.88
650.0	37.68	4.99	7.8	9.95	28.86
700.0	39.67	5.30	9.6	10.21	29.73
750.0	41.40	5.59	11.8	10.42	30.42
800.0	43.03	5.97	14.8	10.60	31.07
850.0	44.38	6.42	19.0	10.76	31.58

Limit to 15 bpi

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

Strength Limit State

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

$R_{fdr} = 520 \cdot kip$

Extreme and
Service Limit States

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

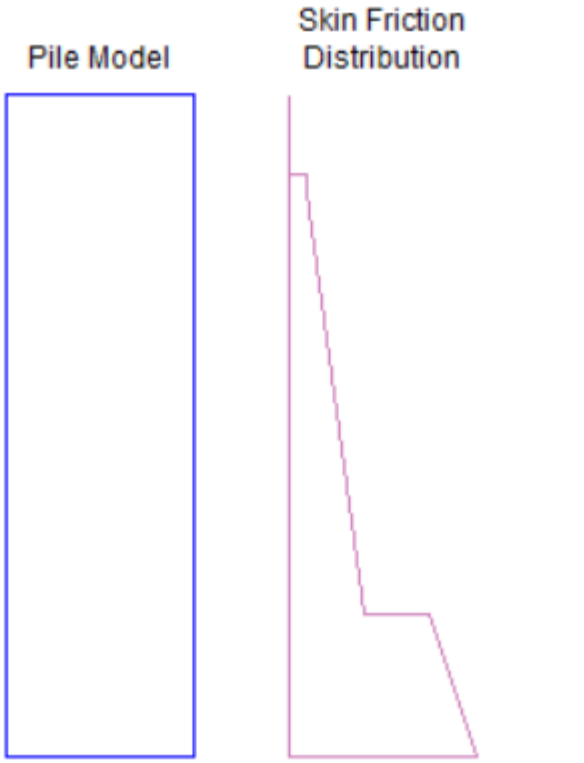
$R_{dr} = 800 \cdot kip$

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

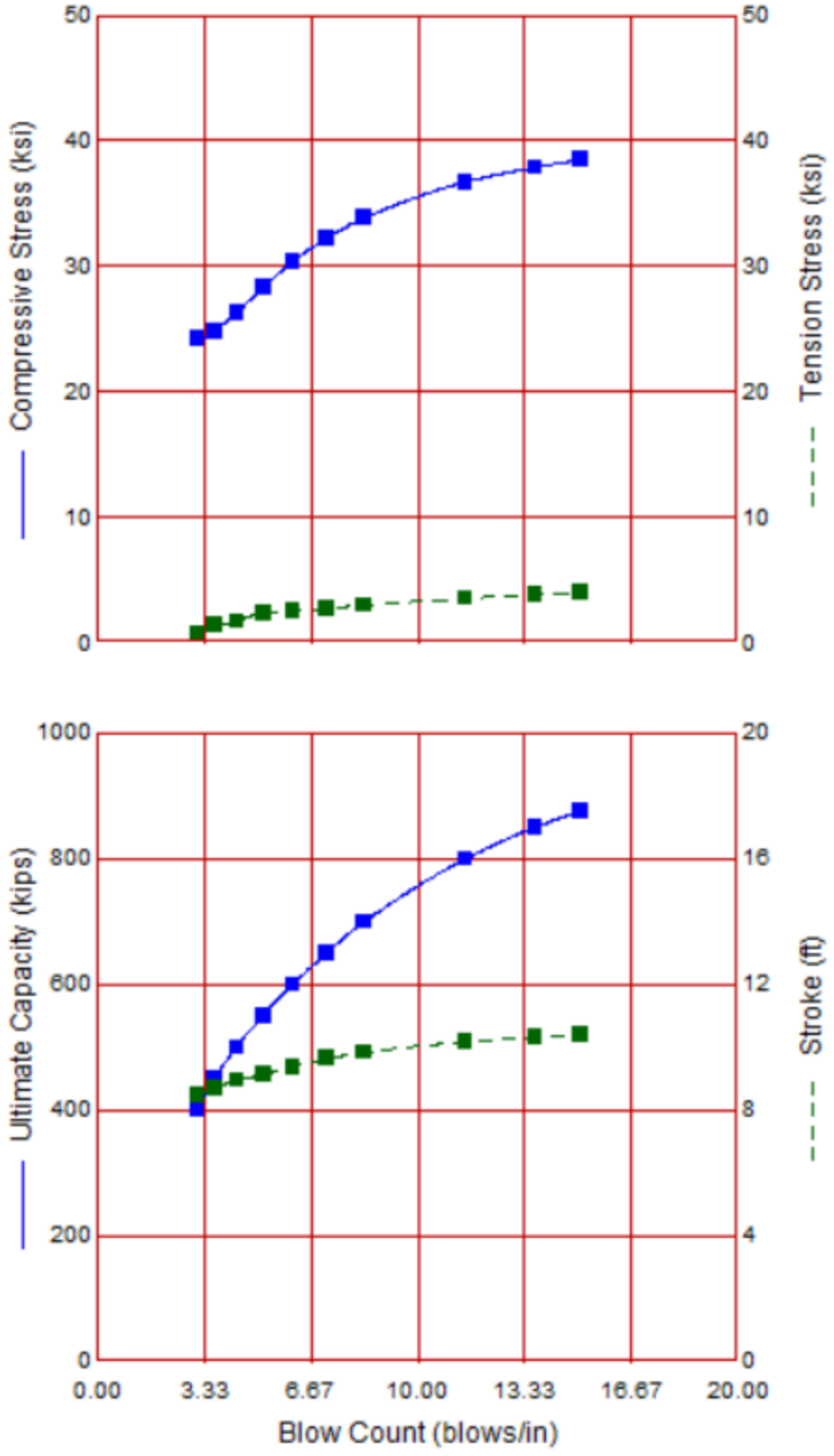
The 14x117 pile can be driven to the resistances below with a APE D25-42 hammer at fuel setting 4 (100% of max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1425 (100%)	psi
Helmet Weight	3.00	kips
Hammer Cushion	34825	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.070	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	55.00	ft
Pile Penetration	48.50	ft
Pile Top Area	34.40	in2



Res. Shaft = 152.0 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
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
22250 Macwahoc 14x117 ABT#1 D25-42

21-Sep-2023
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	24.25	0.81	3.1	8.47	23.59
450.0	24.85	1.43	3.7	8.71	24.01
500.0	26.28	1.78	4.3	8.96	24.50
550.0	28.28	2.44	5.2	9.13	25.05
600.0	30.38	2.62	6.1	9.39	25.91
650.0	32.22	2.82	7.2	9.64	26.69
700.0	33.87	3.06	8.3	9.85	27.40
800.0	36.73	3.60	11.5	10.18	28.47
850.0	37.88	3.94	13.7	10.33	28.97
875.0	38.46	4.10	15.1	10.39	29.09

Limit to 15 bpi

$R_{ndr} := 850 \cdot kip$

Strength Limit State

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

$R_{fdr} = 553 \cdot kip$

Extreme and
Service Limit States

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

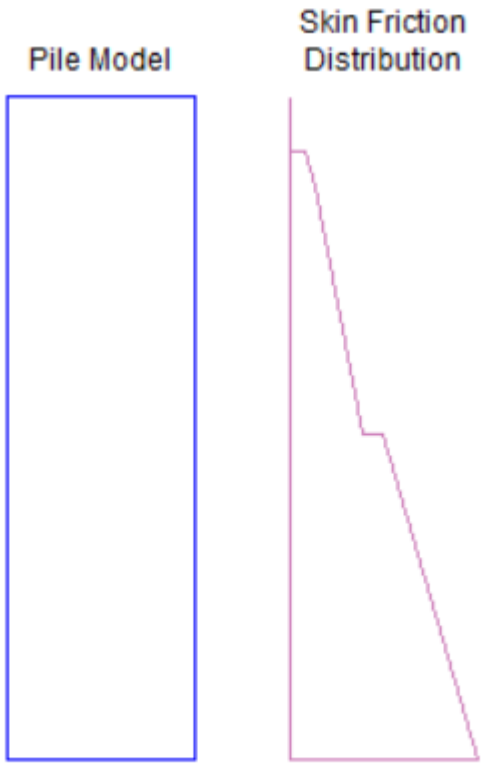
$R_{dr} = 850 \cdot kip$

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

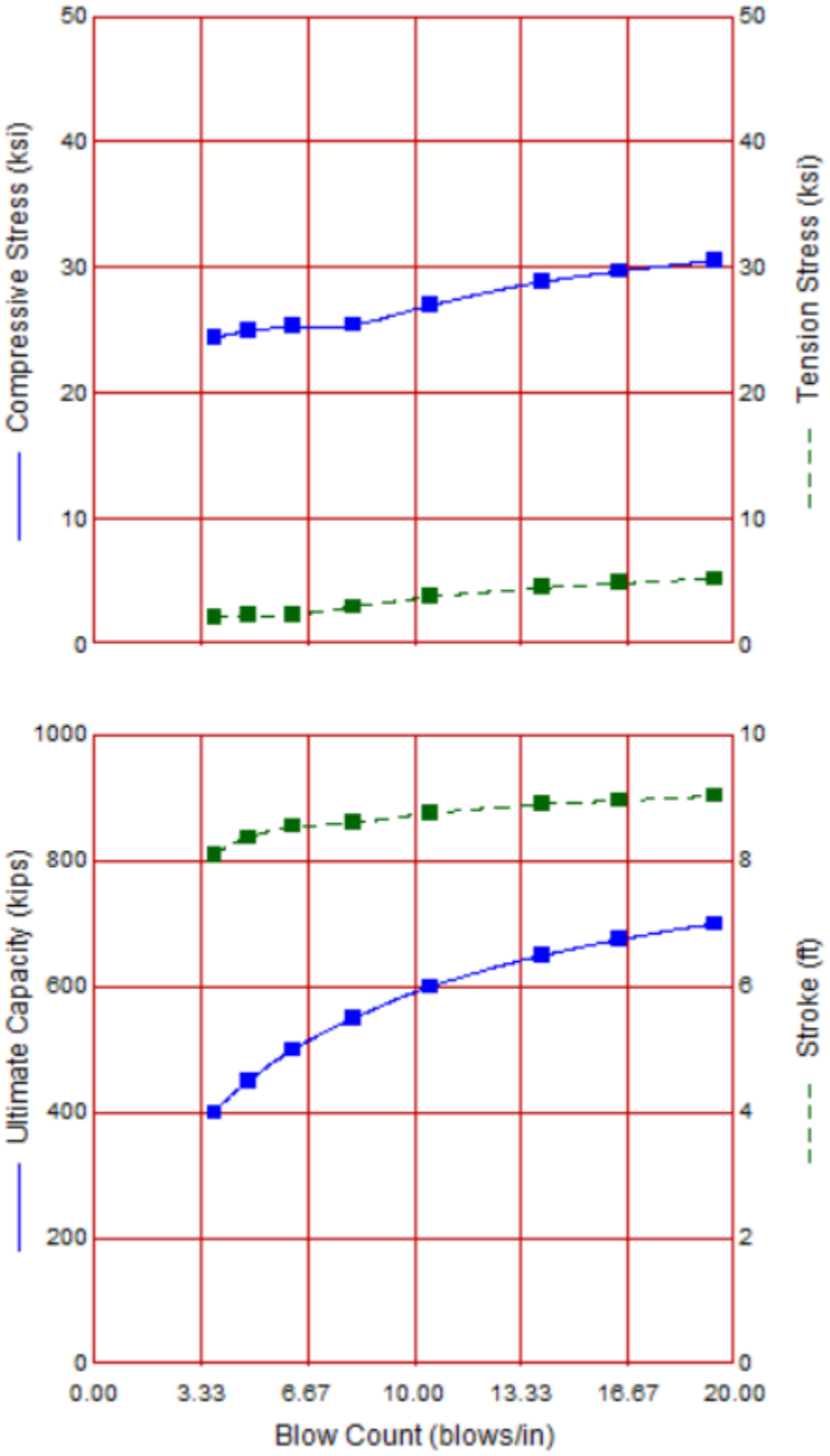
The 14x89 pile can be driven to the resistances below with a APE D19-42 hammer at fuel setting 4 (100% of max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE D 19-42

Ram Weight	4.19	kips
Efficiency	0.800	
Pressure	1710 (100%)	psi
Helmet Weight	3.00	kips
Hammer Cushion	34825	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	85.00	ft
Pile Penetration	78.20	ft
Pile Top Area	26.10	in2



Res. Shaft = 248.0 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
0.00
3.33
6.67
10.00
13.33
16.67
20.00
Blow Count (blows/in)

Maine DOT
22250 Macwahoc 14x89 ABT #2 Long D19-42

20-Sep-2023
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	24.41	2.24	3.8	8.10	21.10
450.0	24.95	2.41	4.8	8.37	21.81
500.0	25.34	2.42	6.2	8.55	22.29
550.0	25.43	3.05	8.1	8.60	22.44
600.0	27.01	3.83	10.5	8.76	22.88
650.0	28.89	4.57	14.0	8.90	23.26
675.0	29.72	4.93	16.4	8.96	23.43
700.0	30.53	5.30	19.4	9.03	23.62

Limit to 15 bpi

$R_{ndr} := 650 \cdot kip$

Strength Limit State

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

$R_{fdr} = 423 \cdot kip$

Extreme and
Service Limit States

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

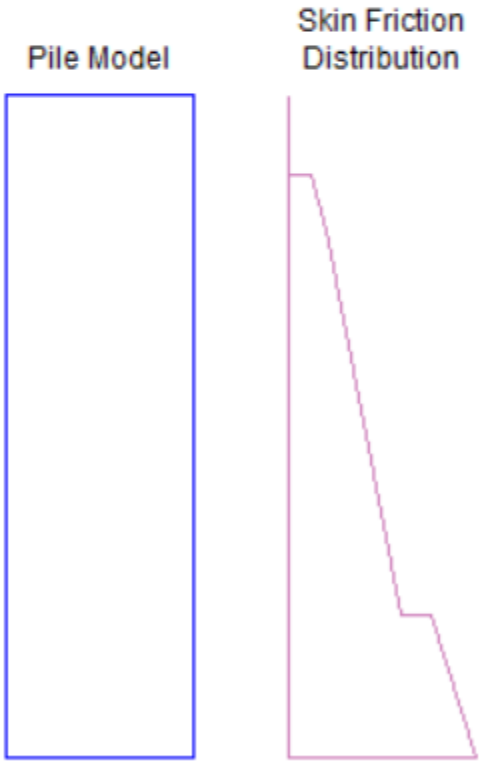
$R_{dr} = 650 \cdot kip$

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

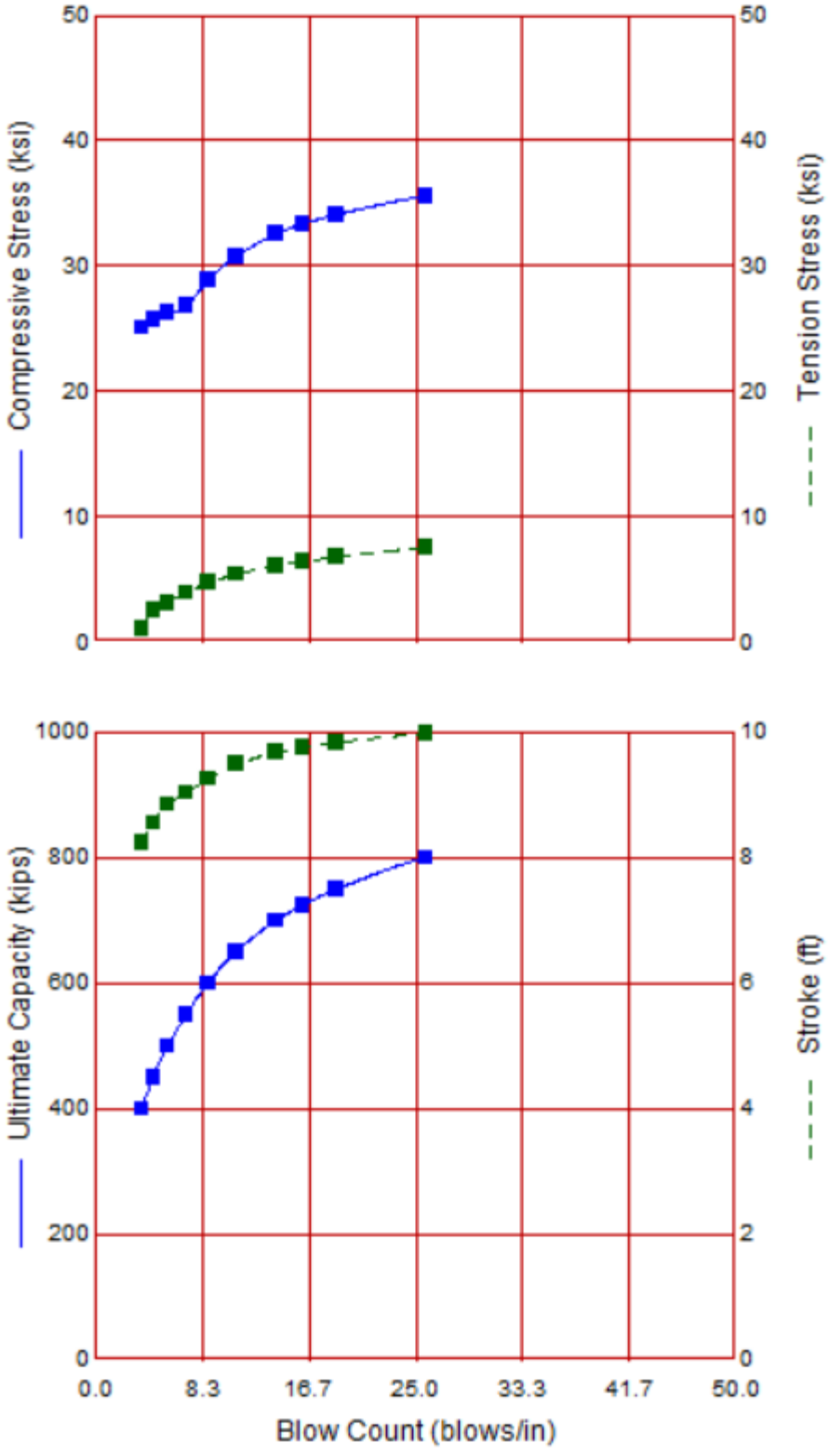
The 14x89 pile can be driven to the resistances below with a APE D19-42 hammer at fuel setting 4 (100% of max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE D 19-42

Ram Weight	4.19	kips
Efficiency	0.800	
Pressure	1710 (100%)	psi
Helmet Weight	3.00	kips
Hammer Cushion	34825	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	55.00	ft
Pile Penetration	48.40	ft
Pile Top Area	26.10	in2



Res. Shaft = 248.0 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
0.0
8.3
16.7
25.0
33.3
41.7
50.0
Blow Count (blows/in)

Maine DOT
22250 Macwahoc 14x89 ABT #2 Short D19-42

20-Sep-2023
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	25.10	1.13	3.6	8.25	20.60
450.0	25.73	2.53	4.5	8.56	21.19
500.0	26.29	3.19	5.6	8.85	21.89
550.0	26.79	3.98	7.0	9.03	22.37
600.0	28.84	4.75	8.7	9.26	22.96
650.0	30.76	5.45	10.9	9.49	23.57
700.0	32.54	6.15	14.0	9.68	24.11
725.0	33.38	6.46	16.1	9.77	24.30
750.0	34.12	6.81	18.8	9.84	24.45
800.0	35.56	7.60	25.8	9.98	24.89

Limit to 15 bpi

$R_{ndr} := 700 \cdot kip$

Strength Limit State

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

$R_{fdr} = 455 \cdot kip$

Extreme and
Service Limit States

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

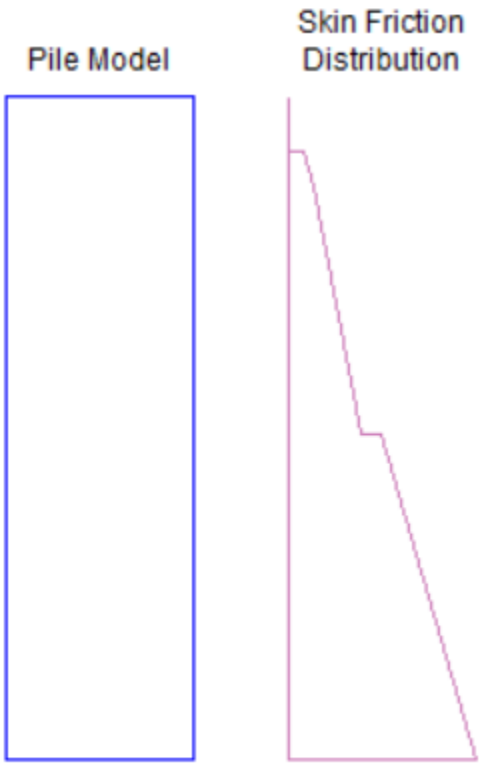
$R_{dr} = 700 \cdot kip$

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

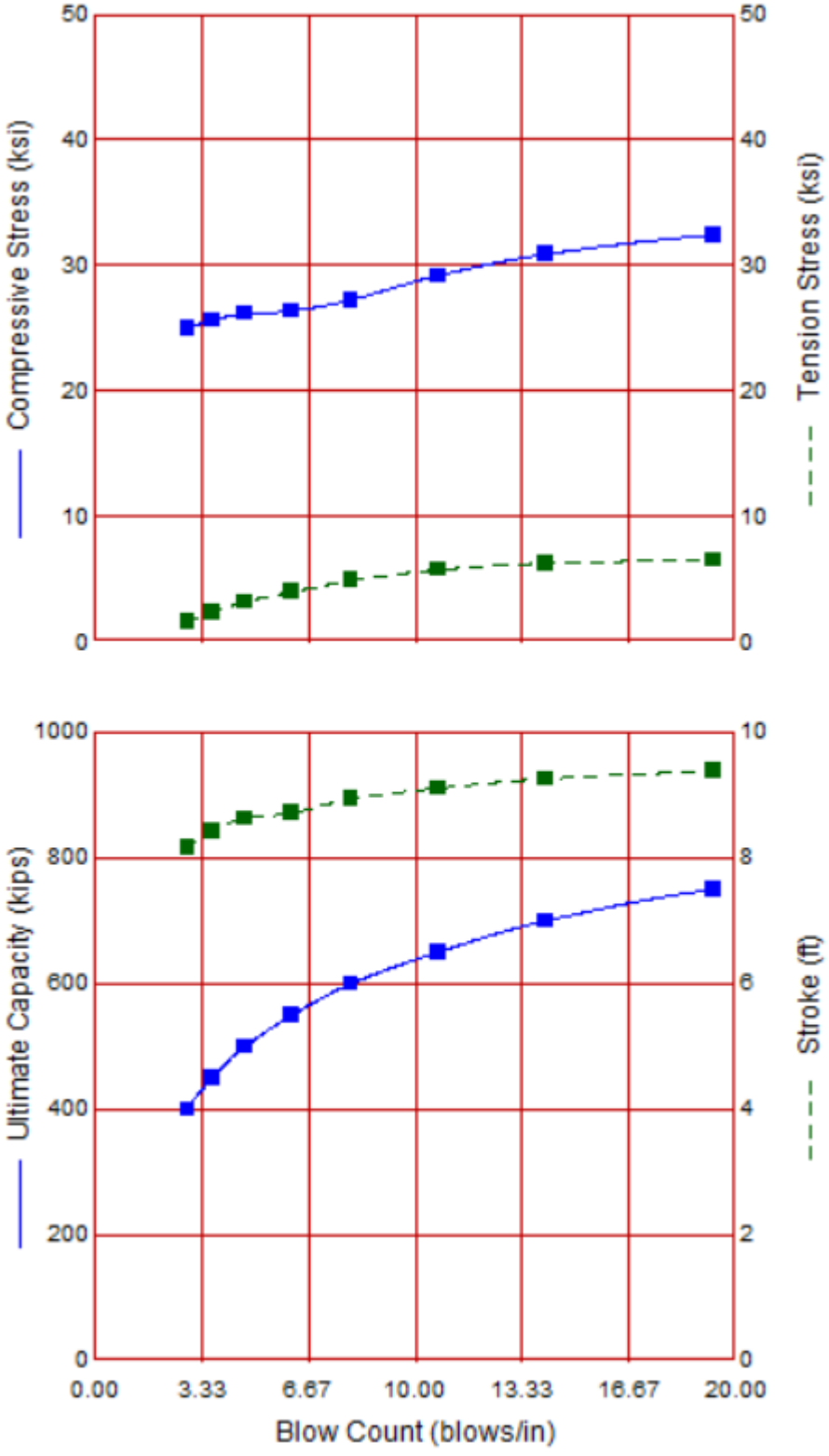
The 14x89 pile can be driven to the resistances below with a APE D25-42 hammer at fuel setting 4 (100% of max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1425 (100%)	psi
Helmet Weight	3.00	kips
Hammer Cushion	34825	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	85.00	ft
Pile Penetration	78.20	ft
Pile Top Area	26.10	in2



Res. Shaft = 248.0 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)

Maine DOT
22250 Macwahoc 14x89 ABT #2 Long D25-42

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Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	24.98	1.64	2.9	8.16	24.85
450.0	25.67	2.34	3.7	8.42	25.81
500.0	26.18	3.25	4.7	8.63	26.51
550.0	26.40	4.04	6.1	8.72	26.84
600.0	27.27	4.93	8.0	8.94	27.68
650.0	29.19	5.79	10.7	9.11	28.24
700.0	30.90	6.32	14.1	9.26	28.82
750.0	32.44	6.58	19.3	9.38	29.24

Limit to 15 bpi

$R_{ndr} := 700 \cdot kip$

Strength Limit State

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

$R_{fdr} = 455 \cdot kip$

Extreme and
Service Limit States

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

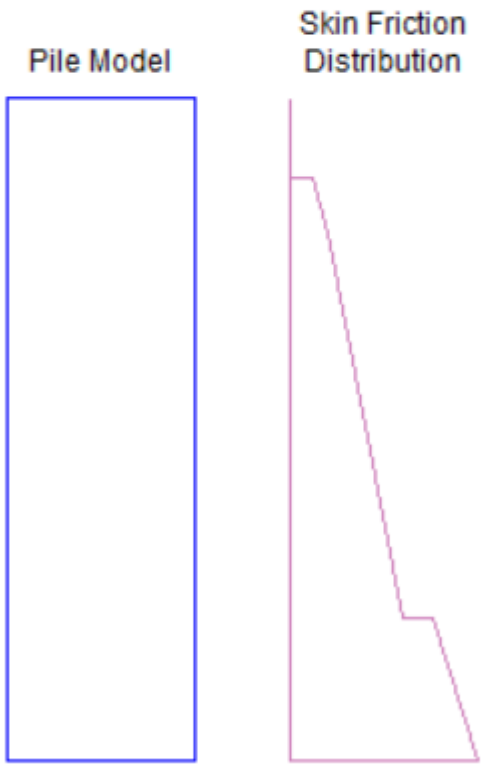
$R_{dr} = 700 \cdot kip$

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

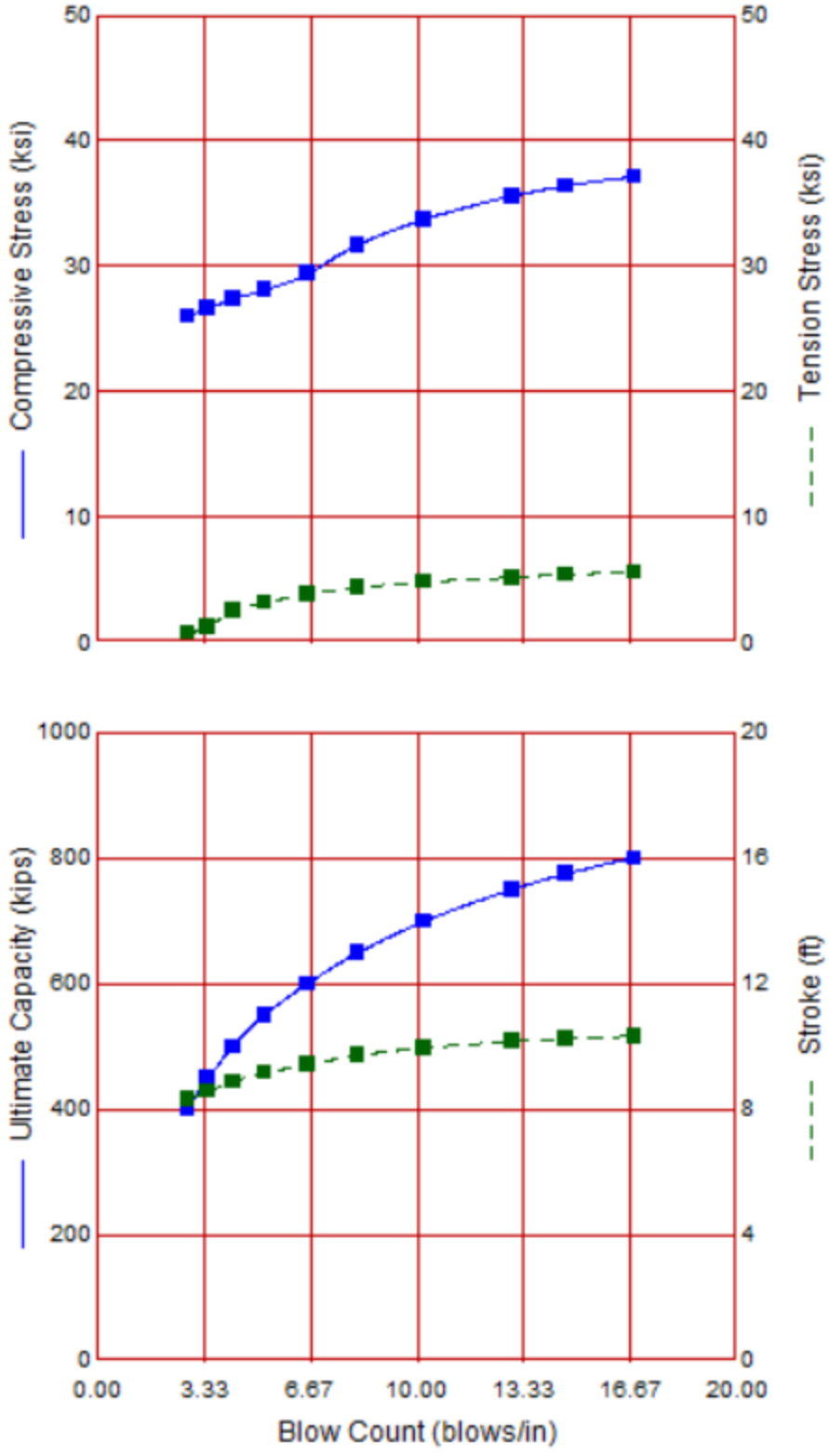
The 14x89 pile can be driven to the resistances below with a APE D25-42 hammer at fuel setting 4 (100% of max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1425 (100%)	psi
Helmet Weight	3.00	kips
Hammer Cushion	34825	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	55.00	ft
Pile Penetration	48.40	ft
Pile Top Area	26.10	in2



Res. Shaft = 248.0 kips
(Constant Res. Shaft)


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

Maine DOT
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Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	25.99	0.80	2.8	8.32	23.96
450.0	26.65	1.29	3.4	8.58	24.53
500.0	27.41	2.62	4.2	8.89	25.23
550.0	28.10	3.24	5.2	9.18	26.14
600.0	29.38	3.87	6.5	9.42	26.87
650.0	31.70	4.43	8.1	9.73	27.91
700.0	33.75	4.88	10.2	9.97	28.78
750.0	35.58	5.22	12.9	10.17	29.38
775.0	36.40	5.46	14.6	10.26	29.70
800.0	37.13	5.63	16.8	10.34	29.90

Limit to 15 bpi

$R_{ndr} := 775 \cdot kip$

Strength Limit State

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

$R_{fdr} = 504 \cdot kip$

Extreme and
Service Limit States

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

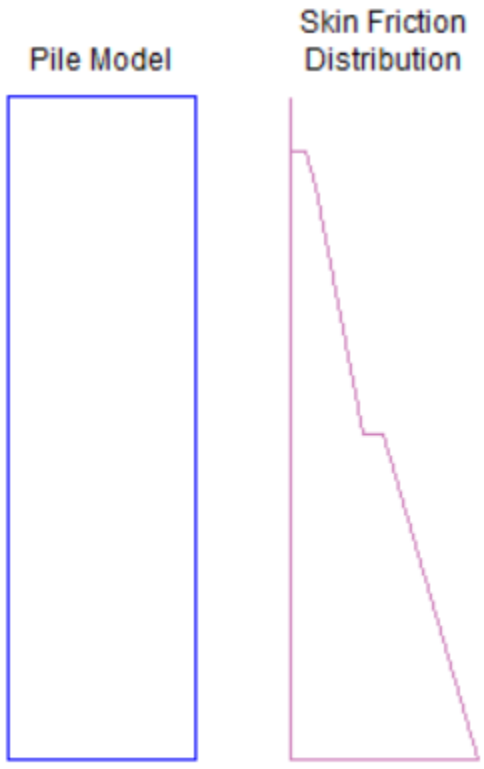
$R_{dr} = 775 \cdot kip$

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

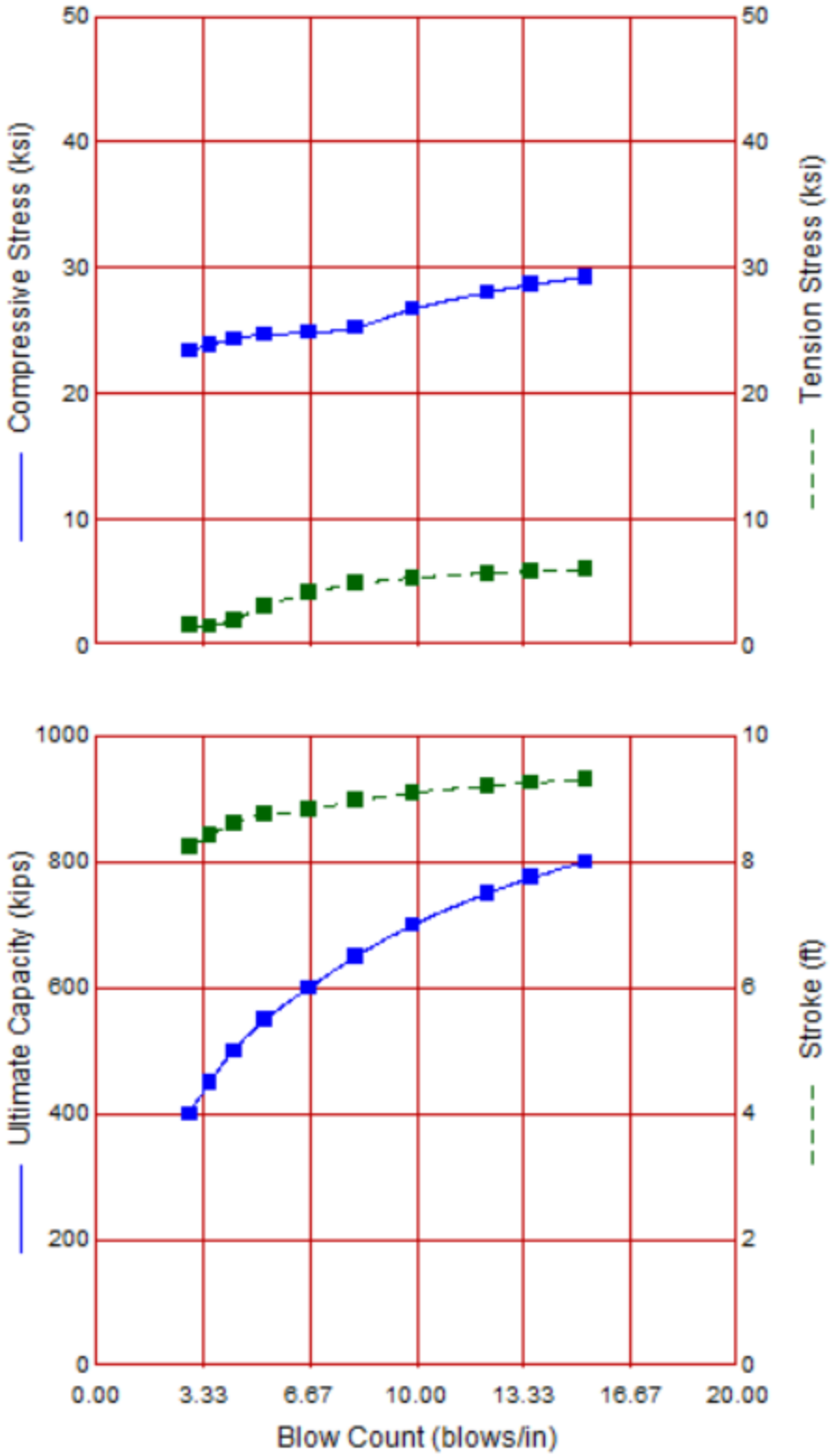
The 14x117 pile can be driven to the resistances below with a APE D25-42 hammer at fuel setting 4 (100% of max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1425 (100%)	psi
Helmet Weight	3.00	kips
Hammer Cushion	34825	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.070	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	85.00	ft
Pile Penetration	78.20	ft
Pile Top Area	34.40	in2



Res. Shaft = 248.0 kips
(Constant Res. Shaft)


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

Maine DOT
22250 Macwahoc 14x117 ABT #2 Long D25-42

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Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	23.45	1.68	2.9	8.24	23.41
450.0	23.92	1.57	3.5	8.43	23.87
500.0	24.38	2.09	4.3	8.61	24.49
550.0	24.75	3.16	5.3	8.76	25.01
600.0	24.89	4.22	6.6	8.82	25.17
650.0	25.27	4.95	8.1	8.97	25.73
700.0	26.73	5.42	9.9	9.10	26.12
750.0	28.07	5.79	12.2	9.21	26.48
775.0	28.68	5.94	13.6	9.26	26.69
800.0	29.25	6.08	15.3	9.31	26.85

Limit to 15 bpi

$R_{ndr} := 775 \cdot kip$

Strength Limit State

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

$R_{fdr} = 504 \cdot kip$

Extreme and
Service Limit States

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

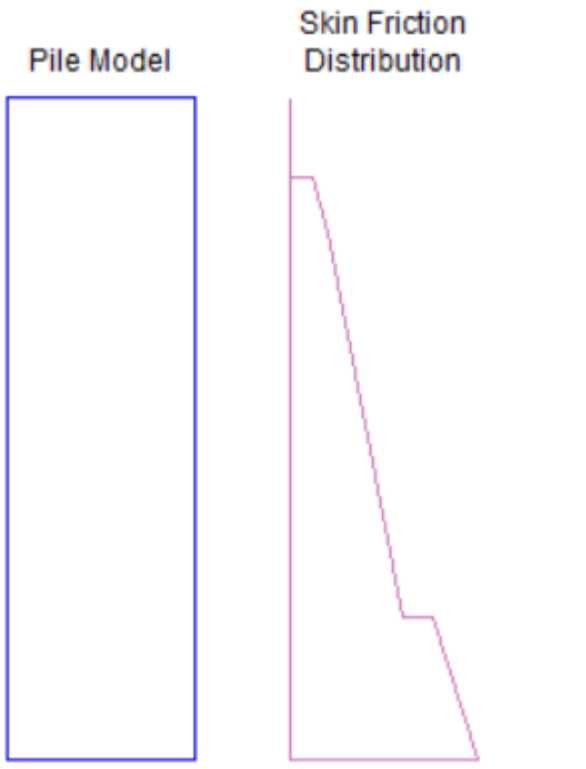
$R_{dr} = 775 \cdot kip$

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

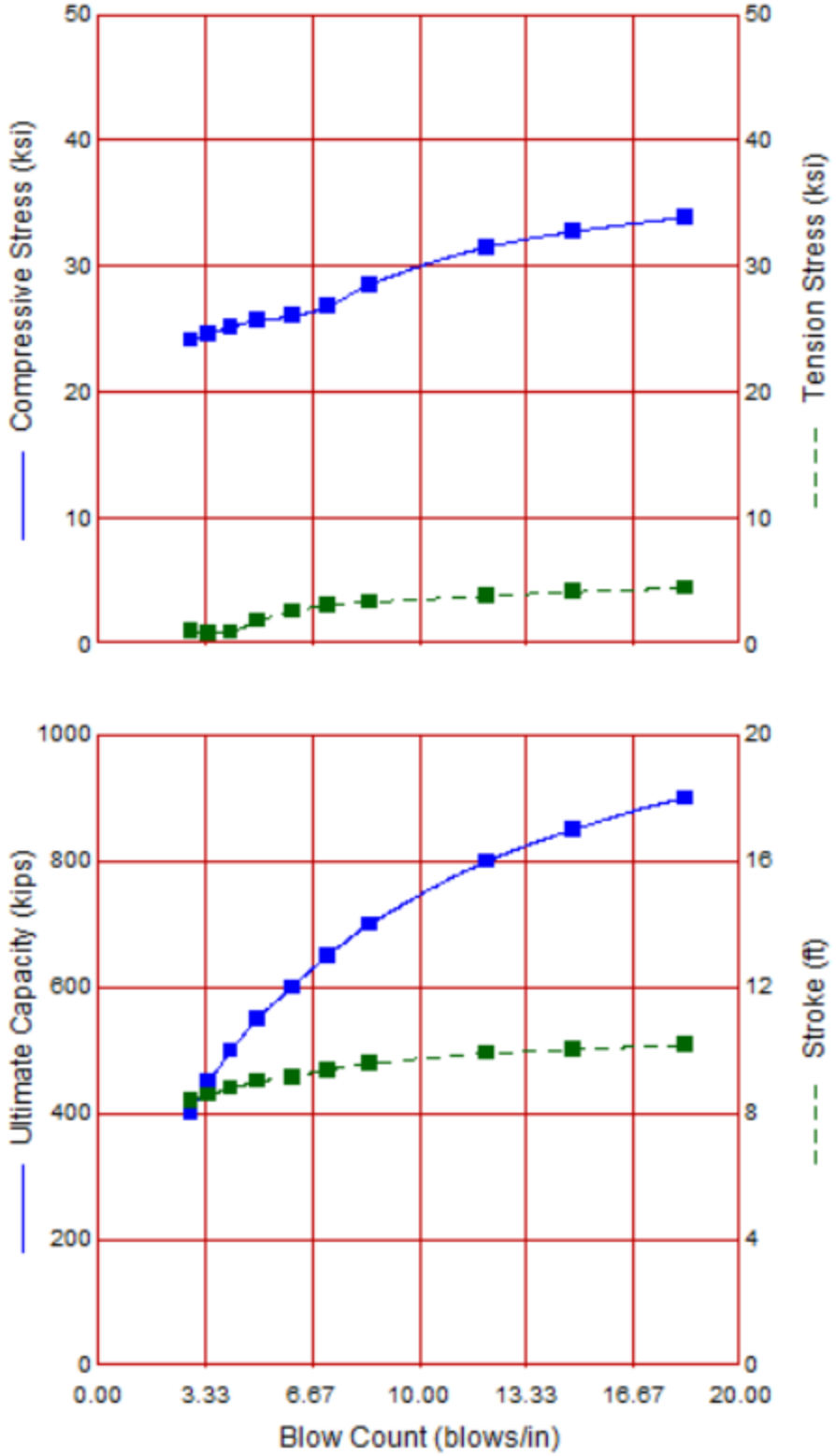
The 14x117 pile can be driven to the resistances below with a APE D25-42 hammer at fuel setting 4 (100% of max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1425 (100%)	psi
Helmet Weight	3.00	kips
Hammer Cushion	34825	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.070	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	55.00	ft
Pile Penetration	48.40	ft
Pile Top Area	34.40	in2



Res. Shaft = 248.0 kips
(Constant Res. Shaft)


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

Maine DOT
22250 Macwahoc 14x117 ABT 2 Short D25-42

24-Aug-2023
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Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	24.15	1.11	2.9	8.38	23.00
450.0	24.67	0.92	3.5	8.59	23.29
500.0	25.21	1.03	4.1	8.80	23.70
550.0	25.73	1.91	4.9	9.02	24.39
600.0	26.07	2.66	6.0	9.16	24.78
650.0	26.81	3.16	7.2	9.39	25.52
700.0	28.57	3.41	8.5	9.59	26.13
800.0	31.53	3.89	12.1	9.91	27.16
850.0	32.77	4.22	14.8	10.04	27.60
900.0	33.86	4.52	18.3	10.17	28.04

Limit to 15 bpi

$R_{ndr} := 850 \cdot kip$

Strength Limit State

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

$R_{fdr} = 553 \cdot kip$

Extreme and
Service Limit States

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

$R_{dr} = 850 \cdot kip$

22250.00 Macwahoc Plantation

GRL WEAP INPUT + RESULT SUMMARY

	Abutment	Pile Size	Pile Length	Hammer	Fuel Setting	Skin Quake	Toe Quake	Skin Damping	Toe Damping	Skin Friction	Ultimate Capacity	Max Comp Stress	Max Tension Stress	Blows/In	Stroke	Energy
14x89, APE D19-42, TOE QUAKE = 0.04	1	HP 14x89	55	APE D19-42	3	0.1	0.04	0.05	0.15	152	650	34.49	5.87	13.6	8.83	21.35
	1	HP 14x89	55	APE D19-42	4	0.1	0.04	0.05	0.15	152	725	39.06	6.97	14.7	10.02	25.19
	2	HP 14x89	85	APE D19-42	3	0.1	0.04	0.05	0.15	248	600	25.51	3.64	14.0	7.93	19.82
	2	HP 14x89	85	APE D19-42	4	0.1	0.04	0.05	0.15	248	650	28.89	4.57	14.0	8.90	23.26
	2	HP 14x89	55	APE D19-42	3	0.1	0.04	0.05	0.15	248	650	28.96	5.10	14.7	8.59	20.49
	2	HP 14x89	55	APE D19-42	4	0.1	0.04	0.05	0.15	248	700	32.54	6.15	14.0	9.68	24.11
14x89, APE D19-42, TOE QUAKE = 0.08	1	HP 14x89	55	APE D19-42	3	0.1	0.08	0.05	0.15	152	625	32.57	5.13	14.1	8.64	20.95
	1	HP 14x89	55	APE D19-42	4	0.1	0.08	0.05	0.15	152	675	36.48	5.74	13.6	9.74	24.6
	2	HP 14x89	85	APE D19-42	3	0.1	0.08	0.05	0.15	248	575	23.82	3.38	12.9	7.82	19.53
	2	HP 14x89	85	APE D19-42	4	0.1	0.08	0.05	0.15	248	625	27.01	4.04	13.1	8.78	22.97
	2	HP 14x89	55	APE D19-42	3	0.1	0.08	0.05	0.15	248	625	26.77	4.78	14.2	8.41	20.18
	2	HP 14x89	55	APE D19-42	4	0.1	0.08	0.05	0.15	248	675	30.52	5.56	13.8	9.49	23.71
14x89, APE D25-42, TOE QUAKE = 0.04	1	HP 14x89	55	APE D25-42	3	0.1	0.04	0.05	0.15	152	700	36.63	4.90	13.9	9.11	24.89
	1	HP 14x89	55	APE D25-42	4	0.1	0.04	0.05	0.15	152	800	43.03	5.97	14.8	10.60	31.07
	2	HP 14x89	85	APE D25-42	3	0.1	0.04	0.05	0.15	248	625	26.23	5.03	13.4	8.17	23.41
	2	HP 14x89	85	APE D25-42	4	0.1	0.04	0.05	0.15	248	700	30.90	6.32	14.1	9.26	28.82
	2	HP 14x89	55	APE D25-42	3	0.1	0.04	0.05	0.15	248	675	30.27	4.35	13.2	8.81	23.63
	2	HP 14x89	55	APE D25-42	4	0.1	0.04	0.05	0.15	248	775	36.40	5.46	14.6	10.26	29.70
14x89, APE D25-42, TOE QUAKE = 0.08	1	HP 14x89	55	APE D25-42	3	0.1	0.08	0.05	0.15	152	675	33.89	4.74	14.4	8.94	24.53
	1	HP 14x89	55	APE D25-42	4	0.1	0.08	0.05	0.15	152	750	39.46	5.55	14.0	10.34	30.39
	2	HP 14x89	85	APE D25-42	3	0.1	0.08	0.05	0.15	248	625	24.87	4.79	14.7	8.11	23.26
	2	HP 14x89	85	APE D25-42	4	0.1	0.08	0.05	0.15	248	675	28.46	5.84	13.5	9.11	28.31
	2	HP 14x89	55	APE D25-42	3	0.1	0.08	0.05	0.15	248	675	28.07	4.01	14.9	8.73	23.53
	2	HP 14x89	55	APE D25-42	4	0.1	0.08	0.05	0.15	248	750	33.44	4.62	14.7	10.07	29.23
14x117, APE D25-42, TOE QUAKE = 0.04	1	HP14x117	55	APE D25-42	3	0.1	0.04	0.05	0.15	152	800	34.72	4.03	14.2	9.10	23.71
	1	HP14x117	55	APE D25-42	4	0.1	0.04	0.05	0.15	152	900	40.26	4.82	14.4	10.51	29.34
	2	HP14x117	85	APE D25-42	3	0.1	0.04	0.05	0.15	248	650	22.07	3.32	14.5	7.35	17.68
	2	HP14x117	85	APE D25-42	4	0.1	0.04	0.05	0.15	248	800	30.30	6.3	13.9	9.36	27.03
	2	HP14x117	55	APE D25-42	3	0.1	0.04	0.05	0.15	248	775	29.49	3.65	14.2	8.82	22.44
	2	HP14x117	55	APE D25-42	4	0.1	0.04	0.05	0.15	248	875	34.77	4.57	14.6	10.17	27.90
14x117, APE D25-42, TOE QUAKE = 0.07	1	HP14x117	55	APE D25-42	3	0.1	0.07	0.05	0.15	152	775	32.86	3.39	14.7	9.00	23.50
	1	HP14x117	55	APE D25-42	4	0.1	0.07	0.05	0.15	152	850	37.88	3.94	13.7	10.33	28.97
	2	HP14x117	85	APE D25-42	3	0.1	0.07	0.05	0.15	248	700	24.41	5.00	13.7	8.20	21.74
	2	HP14x117	85	APE D25-42	4	0.1	0.07	0.05	0.15	248	775	28.68	5.94	13.6	9.26	26.69
	2	HP14x117	55	APE D25-42	3	0.1	0.07	0.05	0.15	248	750	27.42	3.43	14.1	8.72	22.18
	2	HP14x117	55	APE D25-42	4	0.1	0.07	0.05	0.15	248	850	32.77	4.22	14.8	10.04	27.60
14x117, APE D25-42, GRL WEAP AUTO ASSIGNED VALUES BASED ON SOIL LAYERS (QUAKE, DAMPING, SKIN FRICTION)	1	HP14x117	55	APE D25-42	3	0.1	0.121	0.05	0.15	104	700	29.87	2.89	13.7	8.85	23.29
	1	HP14x117	55	APE D25-42	4	0.1	0.121	0.05	0.15	104	800	35.43	3.32	14.5	10.25	28.96
	2	HP14x117	85	APE D25-42	3	0.1	0.121	0.05	0.15	224	675	23.08	3.21	14.4	8.10	21.60
	2	HP14x117	85	APE D25-42	4	0.1	0.121	0.05	0.15	224	750	25.90	3.57	14.4	9.17	26.61
	2	HP14x117	55	APE D25-42	3	0.1	0.121	0.05	0.15	132	700	28.10	2.92	14.0	8.72	22.8
	2	HP14x117	55	APE D25-42	4	0.1	0.121	0.05	0.15	132	800	33.56	3.39	14.8	10.10	28.42

Hammer Information:

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

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

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

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

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

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

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

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

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

4.4.8.2.3 Tolerable Movement

Refer to Article 4.4.7.2.3.

4.4.9 Overall Stability

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

Earth Pressure

Earth Pressure:

Backfill engineering strength parameters

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

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

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

Cohesion $c_1 := 0 \cdot psf$

Integral Abutment - Passive Earth Pressure - Coulomb Theory

s_{ABT1} = Angle of fill slope to the horizontal at Abutment 1 (%) $s_{ABT1} := 1\%$

s_{ABT2} = Angle of fill slope to the horizontal at Abutment 2 (%) $s_{ABT2} := -1\%$

α_{ABT1} = Angle of fill slope to the horizontal at Abutment 1 (degrees) - **Governs**

$\alpha_{ABT1} := \operatorname{atan}(s_{ABT1})$ $\alpha_{ABT1} = 0.57 \cdot deg$

α_{ABT2} = Angle of fill slope to the horizontal at Abutment 2 (degrees)

$\alpha_{ABT2} := \operatorname{atan}(s_{ABT2})$ $\alpha_{ABT2} = -0.57 \cdot deg$

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

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

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

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

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

$\delta' := 17 \cdot deg$

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

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

$K_{p_coulomb}(\alpha_{ABT1}) = 6.2$ Abutment 1 (**Governs**)

$K_{p_coulomb}(\alpha_{ABT2}) = 5.85$ Abutment 2

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 $\alpha_{ABT2} = -0.57 \cdot deg$

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

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

$K_{p_rank}(\alpha_{ABT2}) = 3.25$ Abutment 2

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

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

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

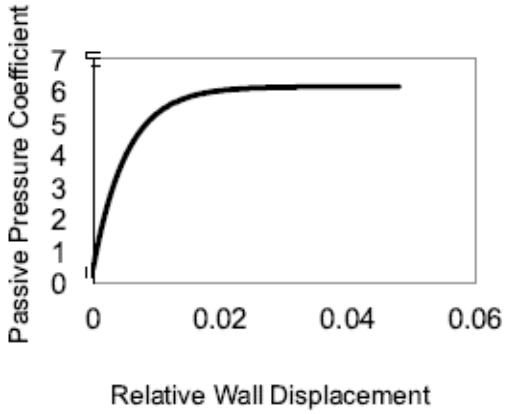


Figure 3.10.8-1: Plot of Passive Pressure Coefficient, K, vs. Relative Wall Displacement, δ_T/H.

Thermal displacement at each abutment: $\delta := 0.42in$ (thermal displacement provided by HNTB for LPile analysis)

Abutment height: $h := 10ft$ $h = 120 \cdot in$

Relative wall displacement: $\frac{\delta}{h} = 0.004$

$K := 0.43 + 5.7 \cdot [1 - \exp[-190(0.004)]]$

$K = 3.46$

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

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

3.11.5.4—Passive Lateral Earth Pressure Coefficient, k_p

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

For cohesive soils, passive pressures may be estimated by:

C3.11.5.4

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

Das, Priniciples of Foundation Engineering, 7th Ed.

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

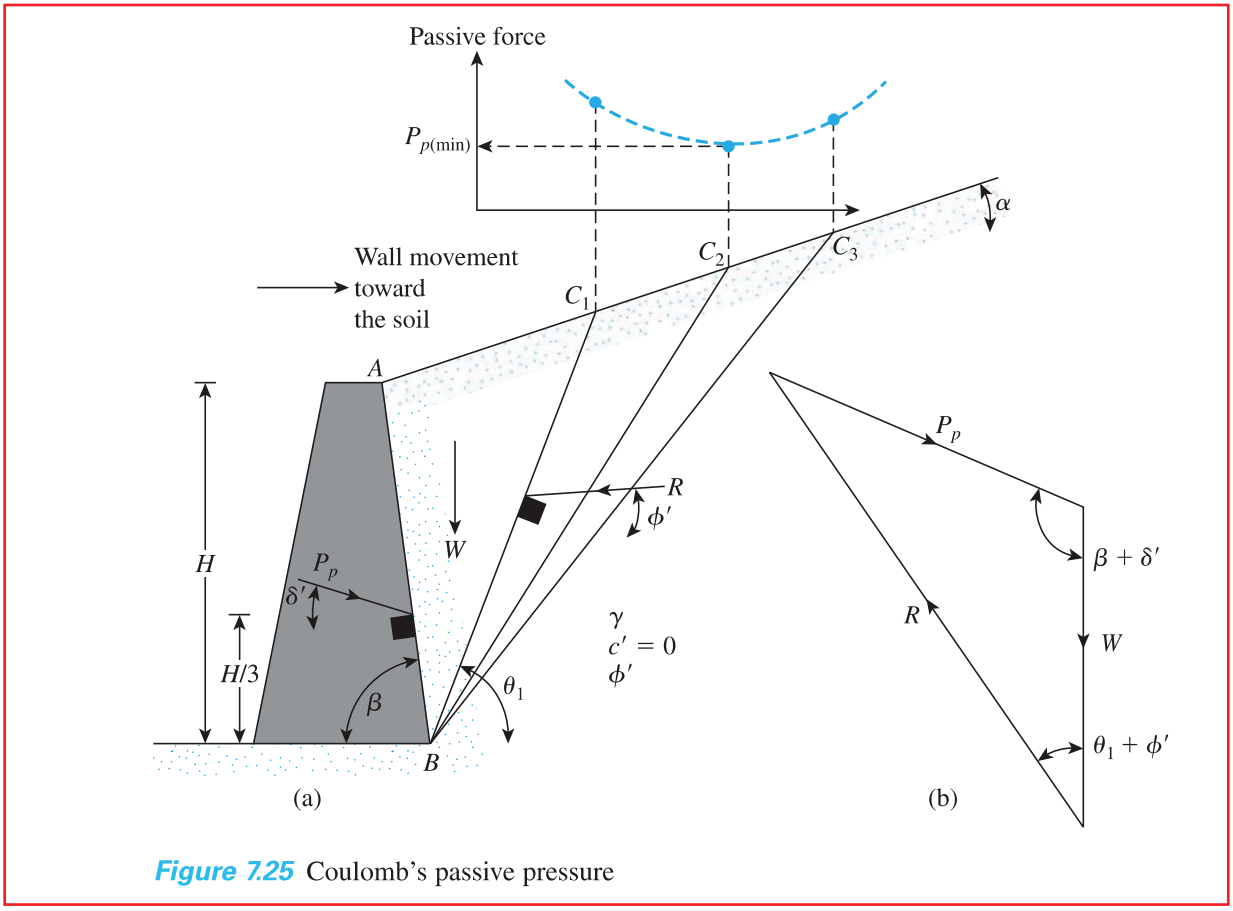


Figure 7.25 Coulomb's passive pressure

Das, Priniciples of Foundation Engineering, 7th Ed.

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

Das, Priniciples of Foundation Engineering, 7th Ed.

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

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

Again, at $z = 3$ m,

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

Hence,

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

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

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

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

7.11 Rankine Passive Earth Pressure: Vertical Backface and Inclined Backfill

Granular Soil

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

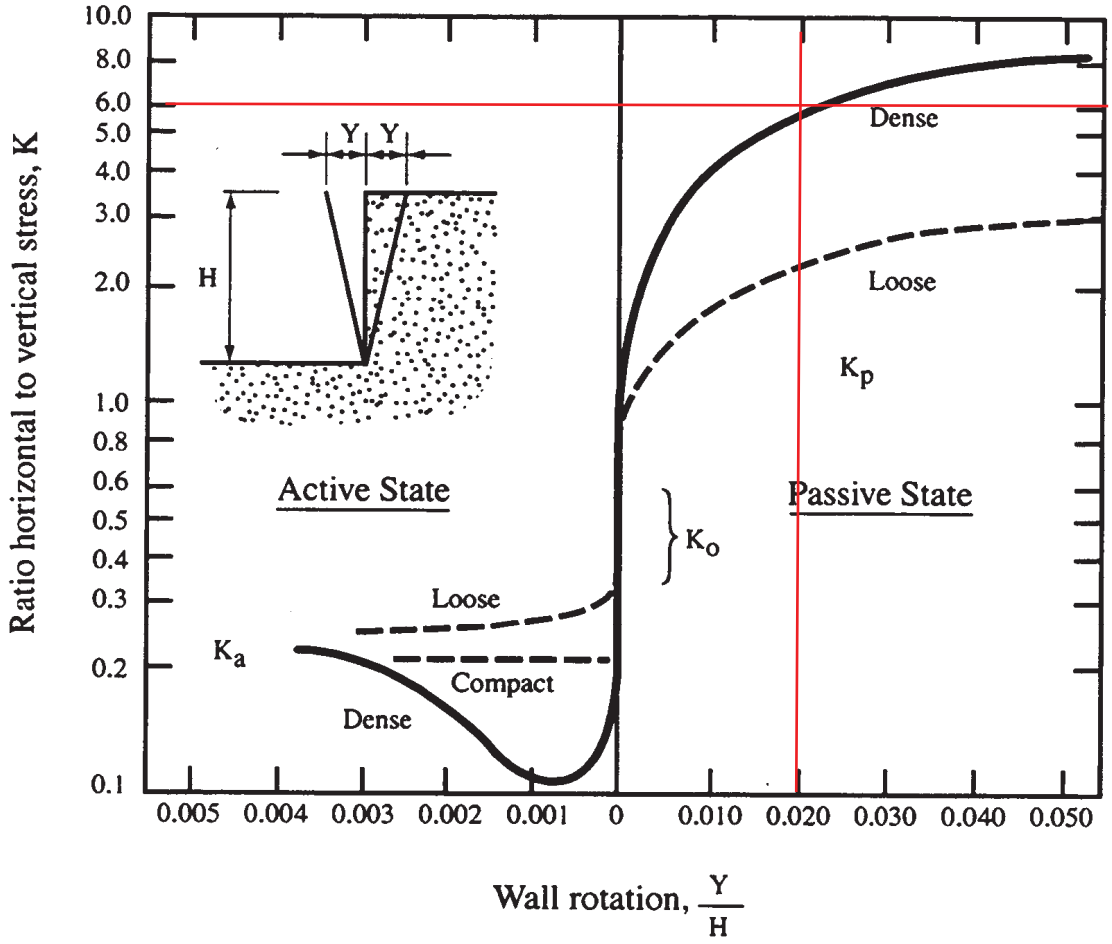
$$\sigma'_p = \gamma z K_p \tag{7.65}$$

and the passive force is

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

where

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



Magnitude of Wall Rotation to Reach Failure

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

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

Frost Depth

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

From Design Freezing Index Map: Macwahoc Plantation, Maine
DFI = 2000 degree-days.
Case 1 - Coarse Grained Soils W=10% (BB-MPMS-101 1D).

For DFI = 2000

at w=10% $d_1 := 95.1\,in$ $d_1 = 7.9 \cdot ft$ Depth of Frost Penetration

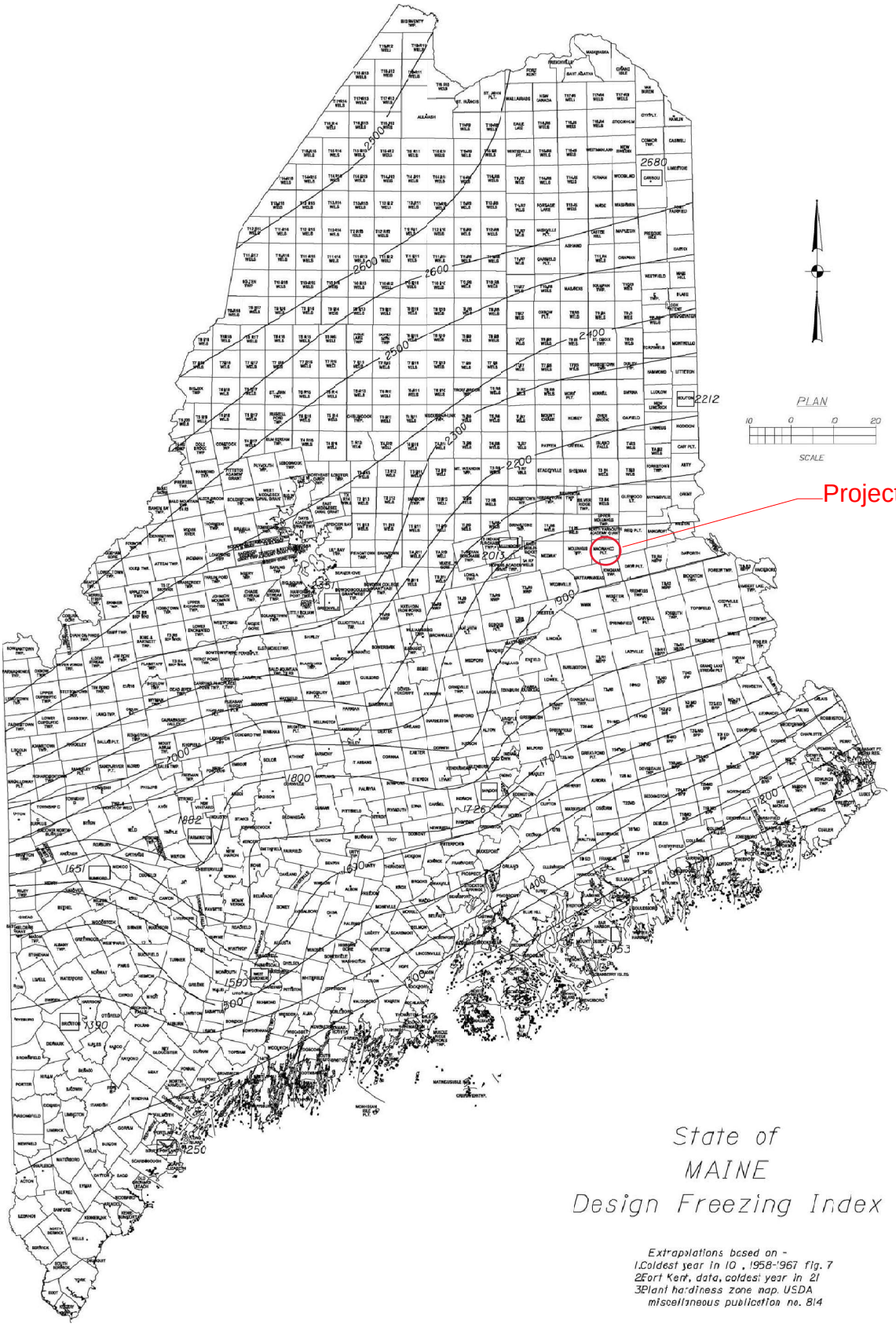


Figure 5-1 Maine Design Freezing Index Map

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-MPMS-101/101A			
Depth	**N_{60}**	**di**	**di/N**
5	9	7	0.78
10	15	4	0.27
15	20	6	0.30
20	24	5	0.21
25	18	5	0.28
30	26	5	0.19
32	14	2	0.14
34.5	30	2.5	0.08
39.5	86	5	0.06
41.5	15	4	0.27
45	100	1	0.01
50	100	4	0.04
55	100	7	0.07
57.3	100	42.5	0.43
SUM		**100**	**3.12**
		di/di/N	**32.06**

BB-MPMS-102			
Depth	**N_{60}**	**di**	**di/N**
5	73	7	0.10
10	42	8	0.19
15	36	2	0.06
20	26	5	0.19
25	52	5	0.10
30	21	5	0.24
35	24	5	0.21
40	55	5	0.09
45	19	5	0.26
50	54	5	0.09
55	100	5	0.05
60	100	4	0.04
65	100	4	0.04
70	100	5	0.05
75	100	5	0.05
80	100	14	0.14
89	100	11	0.11
SUM		**100**	**2.00**
		di/di/N	**49.91**

BB-MPMS-201			
Depth	**N_{60}**	**di**	**di/N**
5	37	7	0.19
10	6	6	1.00
15	31	4	0.13
20	17	5	0.29
25	40	5	0.13
30	21	5	0.24
35	31	5	0.16
40	20	5	0.25
45	17	5	0.29
50	66	6	0.09
52.6	100	47	0.47
SUM		**100**	**3.24**
		di/di/N	**30.85**

BB-MPMS-202			
Depth	**N_{60}**	**di**	**di/N**
5	34	7	0.21
10	29	8	0.28
15	100	1.5	0.02
20	31	5.5	0.18
25	37	5	0.14
30	26	5	0.19
35	60	5	0.08
40	38	6	0.16
45	100	4	0.04
50	100	5	0.05
55	100	7	0.07
59.3	100	41	0.41
SUM		**100**	**1.81**
		di/di/N	**55.16**

SUM	**Nav.**	**42.00**

15 < Nav. < 50 bpf

Conclusion: Site Class D

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

Macwahoc Plantation, Kingman Road Bridge #5021
WIN 22250.00
August 14, 2023

N. Pukay
August 2023
Check by: LK 12-19-23

Abutment No. 1 and 2 Seismic Parameters

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 45.624278
Longitude = -068.258778
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.067	PGA - Site Class B
0.2	0.150	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 = 45.624278
Longitude = -068.258778
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.106	As - Site Class D
0.2	0.239	SDs - Site Class D
1.0	0.111	SD1 - Site Class D

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