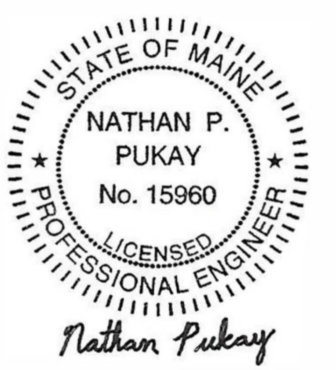
Maine Department of Transportation
Bridge Program
Geotechnical Section
Augusta, Maine

Geotechnical Design Report

For the Replacement of:

Puddle Dock Bridge
State Route 161 over Pattee Brook
Fort Fairfield, Maine



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Soils Report 2024-17
Bridge No. 2691

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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 Puddle Dock Bridge which carries State Route 161 over Pattee Brook in Fort Fairfield, 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 Puddle Dock Bridge was constructed in 1930 and is a 30-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 17.3. The bridge is in poor condition with full height cracks in the abutments, spalled concrete and efflorescent staining at the abutments, and cracking, delamination and efflorescent staining of the concrete superstructure.

Available as-built drawings indicate a previous structure at the bridge consisted of wood planked I-beams on log crib abutments.

The proposed replacement structure consists of a 76-foot, single-span, precast concrete Northeast Extreme Tee (NEXT) beam bridge founded on pile-supported integral abutments with cantilevered, in-line wingwalls. Piles will be driven to bedrock. 1.75H:1V (horizontal:vertical) riprap slopes will be constructed in front of the new integral abutments. The new bridge will be located on a horizontal alignment that will approximately match the existing. The vertical alignment will be raised up to 12 inches to improve the roadway geometry.

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

2.0 GEOLOGIC SETTING

Puddle Dock Bridge carries State Route 161 over Pattee Brook as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of the Fort Fairfield Quadrangle, Maine, Open-File No. 86-54 (1986), indicates the surficial soils in the vicinity of the bridge project consist of stream alluvium and glacial till. Stream alluvium consists of sand, gravel, and silt deposited on flood plains and stream beds by postglacial streams. Glacial till is a heterogeneous mixture of sand, silt, clay, and stones deposited by glacial ice.

The MGS Bedrock Geology of Maine (1985) maps the bedrock at the site as interbedded pelite, limestone, and/or dolostone of the Spragueville Formation.

3.0 SUBSURFACE INVESTIGATION

Four test borings were drilled to explore subsurface conditions at the site. Borings BB-FFPB-101, and BB-FFPB-102 were drilled at or near the location of proposed Abutment No. 1. Borings BB-FFPB-103 and BB-FFPB-103A were drilled at the location of proposed Abutment No. 2. The boring locations are shown on Sheet 2 – Boring Location Plan.

The borings were drilled in May 2022 and August 2022 by S.W. Cole Explorations. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheets 4 and 5 – Boring Logs.

Borings were performed by using a combination of solid stem auger, cased wash boring and rock coring techniques. Soil samples were typically obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The drill rig used in the subsurface investigation is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated per ASTM D 4633 "Standard Test Method for Energy Measurement for Dynamic Penetrometers" in September 2021. All N-values discussed in this report are corrected N-values computed by applying an average energy transfer of 0.91 to the raw field N-values. This hammer efficiency factor (0.91) 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, and identified field-testing requirements. The MaineDOT geotechnical engineer and a MaineDOT NETTCP Certified Subsurface Inspector logged the subsurface conditions encountered in the borings. The borings were located in the field using taped measurements at the completion of the drilling program 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 twelve standard grain size analyses with 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, Stream Alluvium, and Glacial Till overlying Bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheets 4 and 5 – Boring Logs. A generalized subsurface profile is shown on Sheet 3 – Interpretive Subsurface Profile. The following paragraphs discuss the subsurface conditions encountered.

5.1 Fill

A layer of Fill was encountered in the test borings. The thickness of the Fill unit encountered was approximately 12 to 15 feet. The fill materials encountered consisted of:

- Brown, Gravelly SAND, little silt;
- Brown, SAND, some silt, trace to some gravel;
- Brown, Silty SAND, trace gravel;
- Brown, Sandy SILT, little gravel; and
- Wood.

One corrected SPT N-value in the fine-grained Fill unit was 8 blows per foot (bpf) indicating the fine-grained fill is medium stiff in consistency.

Corrected SPT N-values in the coarse-grained Fill unit ranged from 6 to 21 bpf indicating the coarse-grained fill is loose to medium dense in consistency.

Four grain size analyses performed on samples recovered from the Fill unit indicated the material is classified as A-2-4 and A-4 under the AASHTO Soil Classification System and SM and CL under the Unified Soil Classification System (USCS). The natural water contents of the samples tested ranged from 12 to 37 percent.

5.2 Stream Alluvium

A deposit of Stream Alluvium was encountered in the test borings beneath the Fill layer. The encountered thickness was approximately 18 to 22 feet. The deposit was variable and consisted of:

- Grey, fine SAND, little silt;
- Brown, SAND, some gravel, some silt;
- Grey to grey-brown, Sandy GRAVEL, little to some silt; and
- Brown to grey, GRAVEL, trace to some sand, trace to little silt.

Corrected SPT N-value within the Stream Alluvium deposit ranged from 8 to greater than 50 bpf, indicating the deposit is loose to very dense. Three grain size analyses conducted on samples of the deposit indicated the material is classified as A-2-4 and A-1-b under the AASHTO Soil Classification System and SM and GM under the USCS. The natural water contents of the samples tested ranged from 17 to 26 percent.

5.3 Glacial Till

Glacial Till was encountered in the borings underlying the Stream Alluvium deposit. The thickness of the Glacial Till deposit encountered was approximately 62 to 72 feet. The Glacial Till varied from:

- Brown to grey, SAND, little to some silt, trace to some gravel;
- Brown to grey-brown, Gravelly SAND, trace to some silt;
- Grey to brown, Silty SAND, little gravel;
- Grey, SILT, some sand, little gravel;
- Grey to grey-brown, GRAVEL, some sand, little to some silt; and
- Cobbles.

One corrected SPT N-value within the fine-grained Glacial Till was greater than 50 bpf indicating the fine-grained Glacial Till is hard in consistency.

Corrected SPT N-values within the coarse-grained Glacial Till ranged from 26 to greater than 50 bpf indicating the deposit is medium dense to very dense in consistency.

Five grain size analysis performed on samples recovered from the deposit resulted in the material being classified as A-1-b and A-2-4 under the AASHTO Soil Classification System and SM and SW-SM under the USCS. The natural water content of the samples tested ranged from 8 to 14 percent.

5.4 Bedrock

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

Boring	Station	Offset (feet)	Approximate Depth to Bedrock (feet)	Approximate Elevation of Bedrock Surface (feet)	RQD (%) (R1, R2, R3, R4)
BB-FFPB-102	3+00.6	6.6 Lt	105.1	263.2	76, 0, 48, 92
BB-FFPB-103A	3+75.4	9.2 Rt	95.0	269.8	82, 28, 88

Bedrock at the site consisted of grey to dark greenish-grey, fine-grained, thin-bedded, SILTSTONE, moderately hard, fresh, with rock flour and iron oxide staining on some fracture planes, joints dipping at low to vertical angles, spaced close to moderately close, with some quartz or calcite infilling. The RQD of the bedrock cores ranged from 0 to 92 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 8 to 16 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 chosen span length, integral abutments founded on driven piles was the preferred substructure design due to cost, ease of construction, and reduced maintenance costs.

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 Puddle Dock 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	358.1	263.2	97
Abutment No. 2	355.2	269.8	88

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

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

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

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

Per AASHTO LRFD Bridge Design Specifications 9th Edition (LRFD) Article 6.5.4.2, at the strength limit state, the axial resistance factor $\phi_c = 0.50$ (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.

[1] Estimated pile lengths include 2-foot embedment into the pile cap, (rounded up to foot increments).

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 in the table below.

Strength Limit State Factored Axial Pile Resistance					
Pile Section	Structural Resistance[1] ϕ_c=0.50 (kips)	Controlling Geotechnical Resistance[2] ϕ_c=0.50 (kips)	Drivability Resistance[3] φ_{dyn} = 0.65 (kips)		Governing Axial Pile Resistance[6] (kips)
HP 14 x 89	652	652	409[4]	436[5]	409[4]
HP 14 x 117	860	860	474[4]	501[5]	474[4]

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

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

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

[4] Drivability resistance based on a APE D19-42 Pile Hammer at Fuel Setting 4.

[5] Drivability resistance based on a APE D25-42 Pile Hammer at Fuel Setting 4.

[6] Drivability evaluations performed for both Abutments No.1 and 2 piles. Governing resistances for the 14x89 and 14x117 pile sections were the same at both Abutments.

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

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

7.1.2 Axial Pile Resistance – Service and Extreme Limit State

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

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

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

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[6] (kips)
HP 14 x 89	1,305	1,305	630[4]	670[5]	630[4]
HP 14 x 117	1,720	1,720	730[4]	770[5]	730[4]

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

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

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.

[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.

[5] Drivability resistance based on a APE D25-42 Pile Hammer at Fuel Setting 4.

[6] Drivability evaluations performed for both Abutments No.1 and 2 piles. Governing resistances for the 14x89 and 14x117 pile sections were the same at both Abutments.

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

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

LPile Input Parameters Abutment No. 1						
Soil Layer	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	369	11	125	32	90
Fill	Reese Sand	358	3	120	28	25
Fill	Reese Sand	355	2	58	28	20
Stream Alluvium	Reese Sand	353	18	68	36	60
Glacial Till	Reese Sand	335	72	83	38	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	366	11	125	32	90
Fill	Reese Sand	355	4	58	27	20
Stream Alluvium	Reese Sand	351	10	63	32	60
Stream Alluvium	Reese Sand	341	9	68	36	90
Glacial Till	Reese Sand	332	42	83	38	125
Glacial Till	Reese Sand	290	20	78	38	125

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

7.1.4 Scour and Pile Buckling Evaluation and Pile Lateral Resistance

In consideration of LRFD Article 3.7.5, it is recommended that the bridge designer evaluate the potential for buckling of the piles due to scour effects. The design shall consider the maximum anticipated depth of scour as per the site-specific scour analysis. The assessment should account for the reduction in lateral support to the pile provided by the surrounding soil as a result of scour.

The design should ensure that the piles remain stable under the combined effects of axial and lateral loads and the loss of lateral support caused by scour. The bridge designer should refer to LRFD Article 10.7.3.13.1 for guidance on pile buckling analysis.

The effect of scour should also be considered in the determination of minimum pile embedment to ensure fixity is satisfied after the design scour event; Refer to LRFD 10.7.3.6.

7.1.5 Driven Pile Quality Control

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

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

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

7.2 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. Considering a backfill slope exceeding 0 degrees, Coulomb Theory was used to calculate the passive earth pressure coefficient at Abutment No. 1. Assuming a ratio of thermal expansion to abutment height (δ/H) of 0.002 and a level backfill, Rankine Theory was used to calculate the passive earth pressure coefficient at Abutment No. 2. Recommended passive earth pressure coefficients for the integral abutments and in-line wingwalls are provided in the table below.

Passive Earth Pressure Coefficients for Abutments and In-line Wingwalls	
Location	K_p
Abutment No. 1	7.21
Abutment No. 2	3.25

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. Conventional French Drains are the preferred system compared to other systems.

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

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

7.4 Settlement and Embankment Stability

The project calls for the vertical alignment of the new structure to be approximately 7 inches higher at Abutment No. 1 and 10 inches higher at Abutment No. 2. Additionally, the Abutment No. 2 approach will be raised by up to 12 inches. The bridge approach embankments will be constructed using granular borrow placed over loose to medium dense granular fill overlying primarily medium dense to 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 minimal 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, Fort Fairfield has a design freezing index (DFI) of approximately 2600 F-degree days. Fill soils are anticipated to be present at the abutments and embankments, either as silty fill or granular fill. Based on the coarse-grained fill with a water content of 20 percent, the estimated depth of frost penetration is approximately 7.5 feet. It is recommended that any foundation bearing on soils be embedded 7.5 feet for frost protection.

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

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

7.6 Seismic Design Considerations

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

Parameter	Design Value
Peak Ground Acceleration (PGA)	0.080g
Acceleration Coefficient (A_S)	0.128g
S_{DS} (Period = 0.2 sec)	0.287g
S_{D1} (Period = 1.0 sec)	0.125g
Site Class	D
Seismic Zone	1

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

8.0 Construction Recommendations and Considerations

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

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

Based on a Q1.1 water level of El. 354.88, a cofferdam will likely be necessary to successfully dewater and construct the abutments. Wood chips were noted in BB-FFPB-103 within the existing fill. Wood chips indicate the presence of wood debris or timber and may obstruct the installation of a cofferdam. Additionally, a previous structure at the bridge was supported on stone-filled log crib abutments. Wood or stone obstructions may need to be removed by conventional excavation methods.

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

9.0 CLOSURE

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

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

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

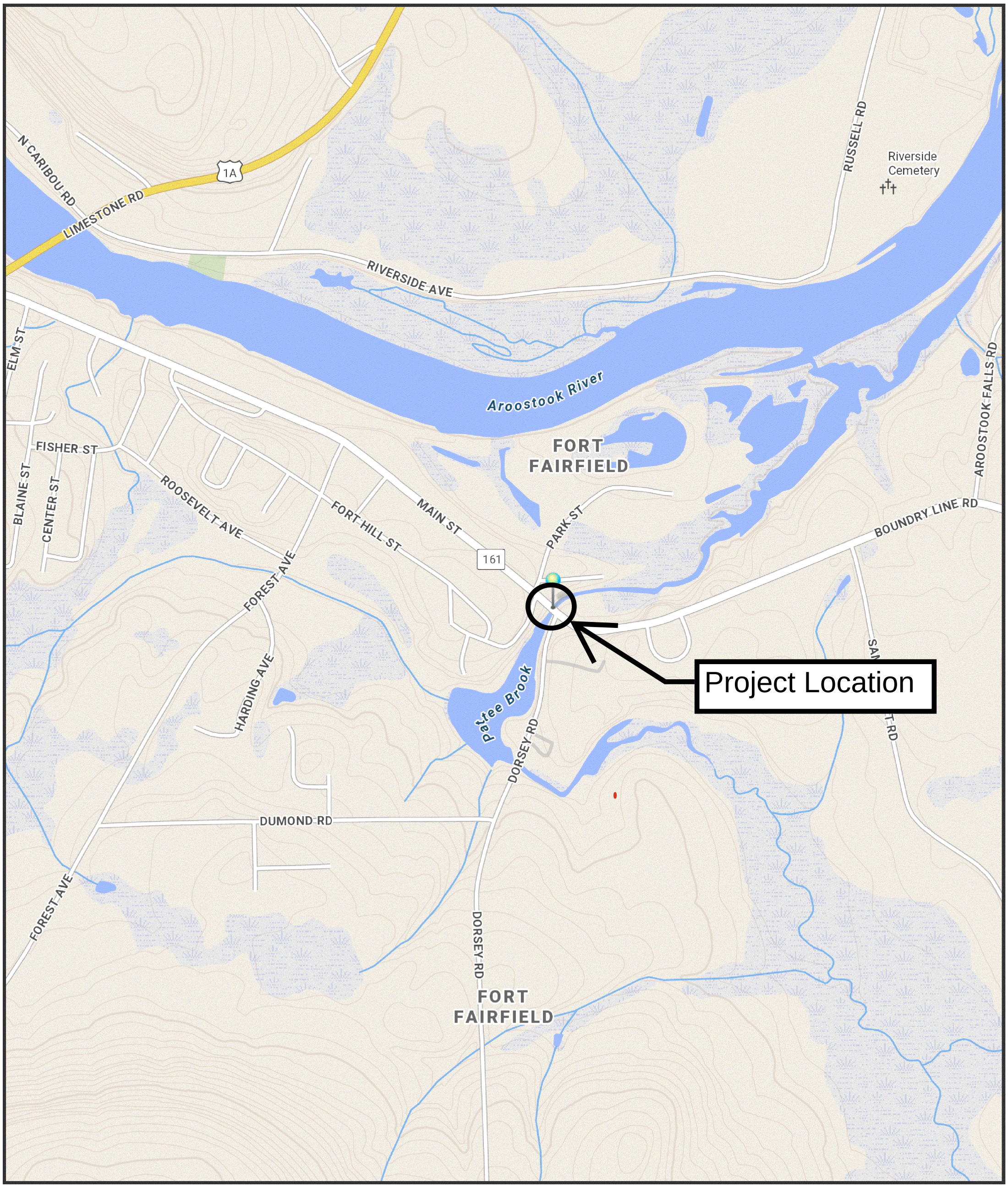
Sheets

Date:8/12/2024

Division: GEOTECH

Username: Terry.White

FORT FAIRFIELD



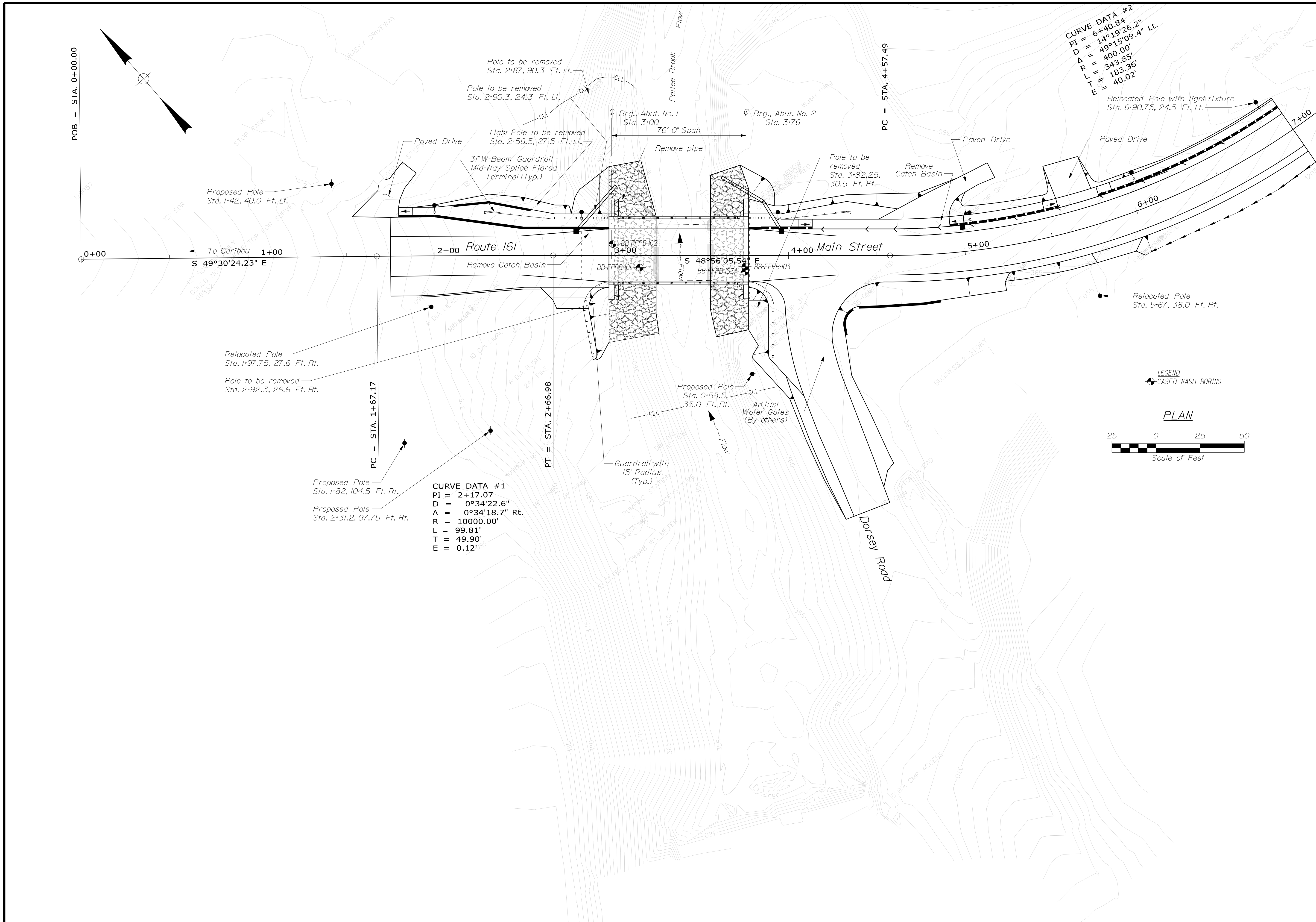
The Maine Department of Transportation provides this publication for information only. Reliance upon this information is at user risk. It is subject to revision and may be incomplete depending upon changing conditions. The Department assumes no liability if injuries or damages result from this information.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

0.25 Miles

1 inch = 0.28 miles

Date: 8/12/2024
Time: 7:22:40 AM

SHEET NUMBER	PUDDLE DOCK BRIDGE	STATE OF MAINE DEPARTMENT OF TRANSPORTATION
1	PATTEE BROOK	2545300
OF 5	FORT FAIRFIELD AROOSTOOK COUNTY	WIN
	LOCATION MAP	BRIDGE NO. 2691 25453.00 BRIDGE PLANS



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2545300

WIN 25453.00

BRIDGE PLANS

BRIDGE NO. 2691

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

SIGNATURE

P.E. NUMBER

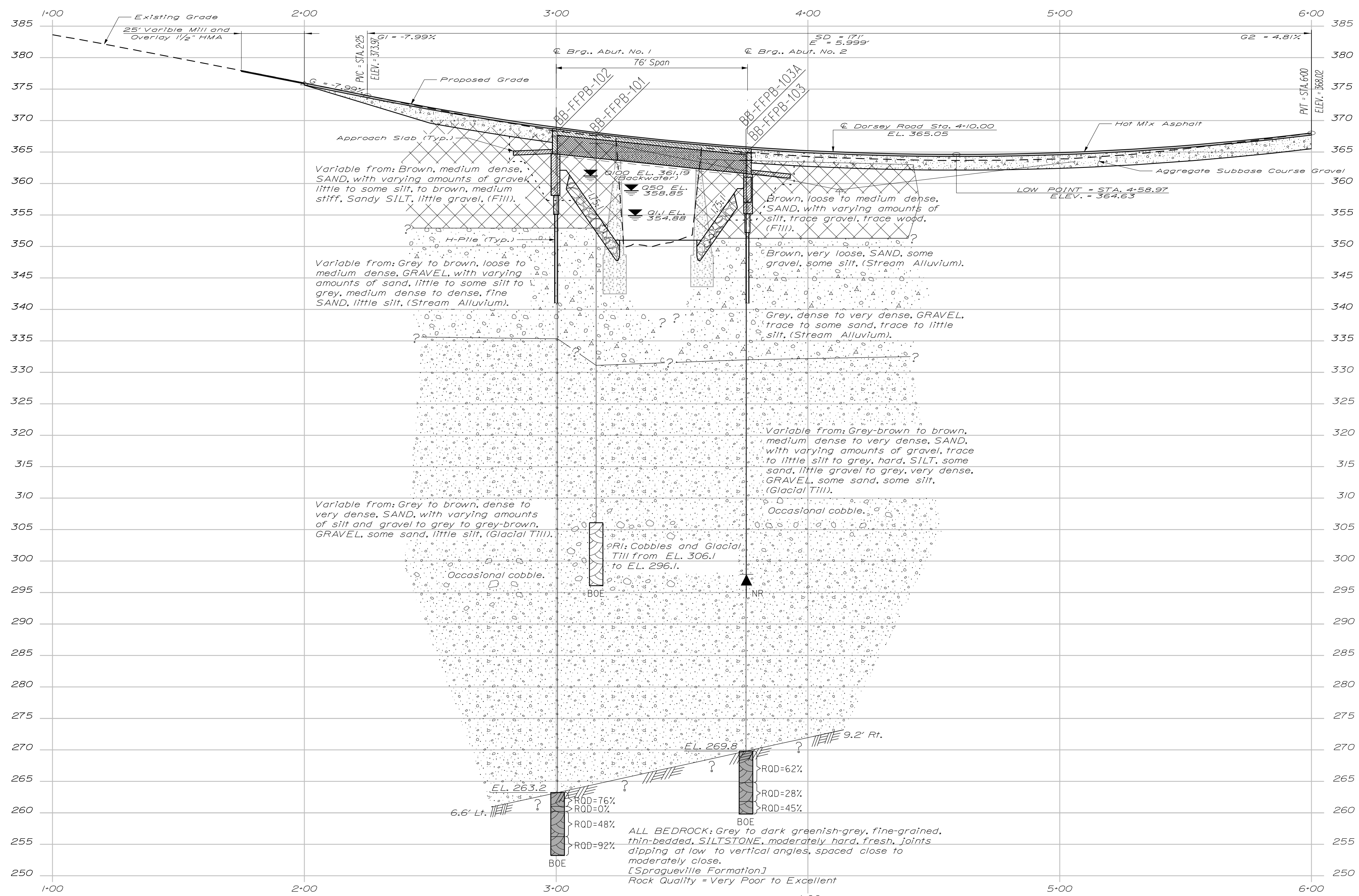
DATE

PUDDLE DOCK BRIDGE
PATTEE BROOK
FORT FAIRFIELD AROOSTOOK COUNTY

BORING LOCATION PLAN

SHEET NUMBER
2
OF 5

Filename: ...\00\GEOTECH\MSTA\006_BLP1.dgn Division: GEOTECH Username: Nathan.P.Pukay Date:8/12/2024



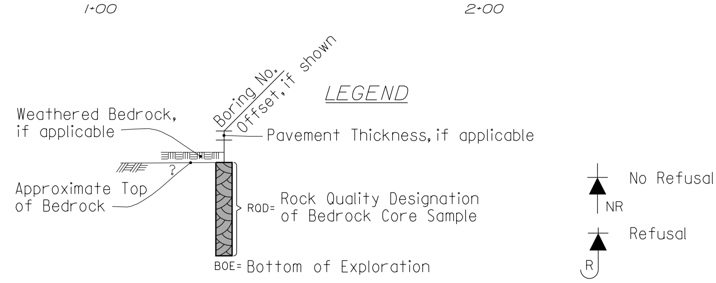
PROFILE

HORIZ 25 0 25 50

VERT 10 0 10 20

SCALE

LEGEND



Notes: 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.

"Varying Amounts" term = Portion is 0 to 50 percent of Total.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2545300

WIN
25453.00

BRIDGE PLANS

BRIDGE NO. 2691

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

SIGNATURE

P.E. NUMBER

DATE

PUDDLE DOCK BRIDGE
PATTEE BROOK
FORT FAIRFIELD AROOSTOOK COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

3

OF 5

Filename: ...\MSTA\008_BORING LOGS1.dgn Division: GEOTECH Username: Nathan.P.Pukay Date:11/1/2024

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

Project: Puddle Dock Bridge #2691 carries Route 161 over Pattee Brook
Location: Fort Fairfield, Maine

Boring No.: BB-FFPB-101
WIN: 25453.00

Driller:	S.W. Cole	Elevation (ft.)	367.6	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"
Date Start/Finish:	5/31/2022-6/1/2022	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	3+15.9, 6.7 ft Rt.	Casing ID/OD:	NW(3.0"/3.5"), HW(4.0"/4.5")	Water Level*:	8.0 ft bgs.
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 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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA		14" HMA.	
								366.4	1.2	
5	1D	24/11	5.00 - 7.00	5/4/9/4	13	20			Brown, damp, medium dense, Gravelly SAND, little silt, (Fill).	
10	2D	24/15	10.00 - 12.00	4/3/5/5	8	12	41		Brown, damp, medium dense, SAND, some silt, some gravel, (Fill).	G#241522 A-2-4, SM WC=11.6%
							41			
							48			
							38			
							40			
15	3D	24/4	15.00 - 17.00	3/2/2/1	4	6	10	352.6	15.0 Grey, wet, loose, Sandy GRAVEL, some silt, (Stream Alluvium).	
							17			
							18			
							22			
							10			
20							10			
	4D	24/3	21.00 - 23.00	5/3/4/2	7	11	16		Similar to 3D, except medium dense.	
							24			
							22			
							17			
25	5D	24/11	25.00 - 27.00	7/10/6/6	16	24	43		Grey-brown, wet, medium dense, Sandy GRAVEL, some silt, (Stream Alluvium).	
							45			
							37			
							60			
							88			
30	MD	24/0	30.00 - 32.00	11/11/10/15	21	32	46			
							42			
							45	334.6	33.0	
	6D	24/14	33.00 - 35.00	11/9/9/9	18	27	50		Grey, wet, medium dense, fine SAND, little silt, (Stream Alluvium).	G#337506 A-2-4, SM WC=20.2%
							52			
35	7D/A	24/20	35.00 - 37.00	5/10/18/27	28	42	76		7D (35.0-36.5 ft bgs.) Similar to 6D, except dense.	
							146	331.1	36.5 7D/A (36.5-37.0 ft bgs.) Brown-grey, wet, dense, Gravelly SAND, some silt, (Glacial Till).	
							132			
							251			
							174			
40	8D	24/15	40.00 - 42.00	14/16/36/32	52	79	38		Brown-grey, wet, very dense, SAND, some silt, trace gravel, (Glacial Till). Roller Coned ahead from 40.0-45.0 ft bgs.	G#337507 A-1-b, SM WC=12.7%
							50			
							45			
							64			
							129			
45	9D	24/17	45.00 - 47.00	28/37/27/37	64	97	59		Brown, wet, very dense, Silty SAND, little gravel, (Glacial Till). Roller Coned ahead from 45.0-50.0 ft bgs.	
							87			
							125			
							118			
							158			
50	10D	24/19	50.00 - 52.00	18/24/31/50	55	83	OPEN HOLE		Brown, wet, very dense, SAND, some silt, trace gravel, (Glacial Till).	
55	11D	24/21	55.00 - 57.00	25/29/39/38	68	103			Similar to 10D.	
60	12D	18/17	60.00 - 61.50	30/46/55	101	153			Brown, wet, very dense, SAND, some silt, trace rock fragments. Set in NW casing and drove to 61.5 ft bgs.	
	R1						NQ-2	306.1	61.5 Cored from 61.5-71.5 ft bgs through cobbles and glacial till. Top of Rock not encountered.	
65										
70								296.1	71.5 Bottom of Exploration at 71.5 feet below ground surface. Hole Abandoned due to stuck casing. 15.0 ft of NW(3") Casing was left in hole from 46.5 ft bgs (El. 321.1) to 61.5 ft bgs (El. 306. 1).	
75										

Remarks:
1) Auto Hammer #367
2) 15 feet of 3" casing (NW) abandoned in hole from 46.5 BGS (El. 321.1) to 61.5 BGS (El. 306.1)

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

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

Project: Puddle Dock Bridge #2691 carries Route 161 over Pattee Brook
Location: Fort Fairfield, Maine

Boring No.: BB-FFPB-102
WIN: 25453.00

Driller:	S.W. Cole	Elevation (ft.)	368.3	Auger ID/OD:	5" Solid Stem
Operator:	Kevin/Brian	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	Wilder/Pukay	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/15/2022-8/16/2022	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	3+00.6, 6.6 ft Lt.	Casing ID/OD:	NW(3.0"/3.5"), HW(4.0"/4.5")	Water Level*:	16.0 ft bgs.
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 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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	367.5	9" HMA. 0.8	
5	1D	24/15	5.00 - 7.00	4/7/7/5	14	21			Brown, damp, medium dense, SAND, some silt, some gravel, (Fill).	
10	2D	24/14	10.00 - 12.00	2/2/3/2	5	8			Brown, wet, medium stiff, Sandy SILT, little gravel, (Fill).	G#337508 A-4, CL WC=14.9%
15	3D	24/14	15.00 - 17.00	4/8/8/6	16	24		352.8	15.5 Brown, wet, medium dense, GRAVEL, some sand, little silt, (Stream Alluvium).	G#337509 A-1-b, GM WC=16.8%
20	4D	24/19	20.00 - 22.00	4/3/2/3	5	8	37		Grey, wet, loose, Sandy GRAVEL, some silt (Stream Alluvium).	G#337510 A-1-b, GM WC=26.0%
							46			
							59			
							77			
							68			
25	5D	24/15	25.00 - 27.00	13/9/7/13	16	24	72		Grey, wet, medium dense, GRAVEL, some sand, little silt, (Stream Alluvium).	
							58			
							49			
							46			
							49			
30	6D	24/14	30.00 - 32.00	14/8/7/7	15	23	46		Grey, wet, medium dense, Sandy GRAVEL, little silt, (Stream Alluvium).	
							43			
							51	335.3	33.0	
							83			
	7D	24/15	34.00 - 36.00	12/21/20/14	41	62	30		Grey, wet, very dense, SAND, some gravel, little silt, (Glacial Till).	G#337511 A-1-b, SM WC=11.1%
35							OPEN HOLE			
40	8D	24/13	40.00 - 42.00	17/12/10/15	22	33			Similar to 7D, except dense.	
45	9D	24/20	45.00 - 47.00	13/15/25/49	40	61			Brown, wet, very dense, SAND, some silt, little gravel, (Glacial Till).	G#337512 A-2-4, SM WC=13.7%
50	10D	18/16	50.00 - 51.50	36/39/65	104	158			Similar to above.	
55	11D	18/15	55.00 - 56.50	31/43/61	104	158			Brown, wet, very dense, SAND, little silt, little gravel, (Glacial Till).	
60								306.3	62.0	
65	12D	6/5	65.00 - 65.50	70(6")	---				Grey-brown, wet, very dense, GRAVEL, some sand, little silt, (Glacial Till).	
70										
									Occasional Cobble.	
75										

Remarks:
1) 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: Puddle Dock Bridge #2691 carries Route 161 over Pattee Brook
Location: Fort Fairfield, Maine

Boring No.: BB-FFPB-102
WIN: 25453.00

Driller:	S.W. Cole	Elevation (ft.)	368.3	Auger ID/OD:	5" Solid Stem
Operator:	Kevin/Brian	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	Wilder/Pukay	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/15/2022-8/16/2022	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	3+00.6, 6.6 ft Lt.	Casing ID/OD:	NW(3.0"/3.5"), HW(4.0"/4.5")	Water Level*:	16.0 ft bgs.
Hammer Efficiency Factor: 0.91		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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WOR/C = Weight of Rods or Casing
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WC = Water Content, percent
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PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
75	13D	24/18	75.00 - 77.00	42/43/42/38	85	129			Similar to 12D, except grey.	
80										
									Occasional Cobble.	
85	14D	18/16	85.00 - 86.50	22/25/63	88	133			Grey, wet, very dense, SAND, some gravel, little silt, (Glacial Till).	
90										
95	15D	24/14	95.00 - 97.00	9/9/16/23	25	38			Grey, wet, dense, Silty SAND, little gravel, (Glacial Till).	
100										
105	R1	26.4/24	105.10 - 107.30	RQD = 76%			NQ-2	263.2	Cobble at 104.8 ft bgs. Roller Coned ahead to 105.1 ft bgs. 105.1 Top of bedrock at Elev. 263.2 ft. R1: Bedrock: Grey to dark greenish-grey, fine-grained, thin-bedded, SILTSTONE, moderately hard, fresh, steeply dipping joints, closely spaced, with some quartz or calcite infilling. [Spragueville Formation] Rock Quality = Good. R1: Core Times (min:sec) 105.1-106.1 ft (2:01) 106.1-107.1 ft (2:04) 107.1-107.3 ft (1:04) Core Blocked 92% Recovery	
	R2	9.6/7	107.30 - 108.10	RQD = 0%					R2: Bedrock: Similar to R1 except with a vertical fracture throughout run. Fracture plane is fresh with minor iron oxide staining. [Spragueville Formation] Rock Quality = Very Poor. R2: Core Times (min:sec) 107.3-108.1 ft (2:32) Core Blocked 70% Recovery	
	R3	48/48	108.10 - 112.10	RQD = 48%					R3: Bedrock: Grey to dark greenish-grey, fine-grained, thin-bedded, SILTSTONE, moderately hard, fresh, vertical joints, closely spaced, with some quartz or calcite infilling, fracture planes are fresh with minor oxide staining, core becomes more competent throughout run. [Spragueville Formation] Rock Quality = Poor. R3: Core Times (min:sec) 108.1-109.1 ft (2:42) 109.1-110.1 ft (3:02) 110.1-111.1 ft (2:46) 111.1-112.1 ft (3:00) 100% Recovery	
110										
	R4	36/35	112.10 - 115.10	RQD = 92%					R4: Bedrock: Grey to dark greenish-grey, fine-grained, thin-bedded, SILTSTONE, moderately hard, fresh, joints dipping at moderate angles, spaced moderately close. [Spragueville Formation] Rock Quality = Excellent. R4: Core Times (min:sec) 112.1-113.1 ft (1:59) 113.1-114.1 ft (2:17) 114.1-115.1 ft (2:20) 97% Recovery	
115								253.2	115.1 Bottom of Exploration at 115.1 feet below ground surface.	
120										
125										
130										
135										
140										
145										
150										

Remarks:
1) 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.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
2545300
WIN 25453.00
BRIDGE PLANS
BRIDGE NO. 2691

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		
CHECKED-REVIEWED		
DESIGN2-DETAILED2	N. PUKAY	T. WHITE OCT 2024
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

PUDDLE DOCK BRIDGE
PATTEE BROOK
FORT FAIRFIELD AROOSTOOK COUNTY

BORING LOGS

SHEET NUMBER
4
OF 5

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

Project: Puddle Dock Bridge #2691 carries Route 161 over Pattee Brook
Location: Fort Fairfield, Maine

Boring No.: BB-FFPB-103
WIN: 25453.00

Driller:	S.W. Cole	Elevation (ft.)	364.9	Auger ID/OD:	5" Solid Stem
Operator:	Kevin/Brian	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	Wilder/Pukay	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/17,22/2022	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	3+75.6, 6.2 ft Rt.	Casing ID/OD:	HW(4.0"/4.5")	Water Level*:	13.0 ft bgs.
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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA		14" HMA.	
								363.7	1.2	
5	1D	24/19	5.00 - 7.00	4/3/4/4	7	11			Brown, dry, medium dense, SAND, some silt, trace gravel, (Fill).	G#337513 A-2-4, SM WC=14.5%
10	2D	24/19	10.00 - 12.00	4/2/2/2	4	6			Brown, moist, loose, Silty SAND, trace gravel, trace wood, (Fill).	G#337514 A-4, SM WC=37.3%
								351.4	13.5	
15	3D	24/6	15.00 - 17.00	2/2/1/2	3	5	HP		Brown, wet, very loose, SAND, some gravel, some silt, (Stream Alluvium).	
							11			
							33			
							56			
							43			
20	MD	24/0	20.00 - 22.00	14/9/2/2	11	17	45			
							34			
							35			
							62			
							50	340.9	24.0	
25	4D	24/13	25.00 - 27.00	15/16/11/10	27	41	48		Grey, wet, dense, GRAVEL, some sand, little silt, (Stream Alluvium).	
							62			
							67			
							34			
							47			
30	5D	24/4	30.00 - 32.00	10/18/36/20	44	67	56		Grey, wet, very dense, GRAVEL, trace sand, trace silt, (Stream Alluvium).	
							77			
							47			
							69	331.9	33.0	
	6D	24/19	34.00 - 36.00	8/15/21/18	36	55	30		Grey-brown, wet, very dense, Gravelly SAND, little silt, (Glacial Till).	G#337515 A-1-b, SM WC=11.7%
35							91			
							98			
							67			
							65			
	7D	24/14	39.00 - 41.00	8/8/9/12	17	26	26		Brown, wet, medium dense, SAND, trace gravel, trace silt, (Glacial Till).	
40							42			
							68			
							77			
							84			
							73			
45	8D	24/15	45.00 - 47.00	16/17/22/17	39	59	74		Brown, wet, very dense, SAND, some gravel, little silt, (Glacial Till).	
							99			
							90			
							113			
							122			
50	9D	24/16	50.00 - 52.00	22/21/20/25	41	62	97		Similar to 8D.	
							116			
							121			
							129			
							116		Occasional Cobble.	
55	10D	24/15	55.00 - 57.00	17/19/30/49	49	74	a57 OPEN HOLE		a57 blows for 0.5 ft. Brown, wet, very dense, Gravelly SAND, trace silt, (Glacial Till).	G#337516 A-1-b, SW-SM WC=8.1%
60										
65	11D	24/18	65.00 - 67.00	22/31/34/39	65	99			Brown, wet, very dense, SAND, little gravel, little silt, (Glacial Till).	
								297.9	67.0 Bottom of Exploration at 67.0 feet below ground surface. Hole Abandoned. 10.0 ft of broken HW(4") casing abandoned in hole from 57. 0 ft bgs (El. 307.9) to 67.0 ft bgs (El. 297.9).	
70										
75										

Remarks:
1) Auto Hammer #367
2) 10.0 ft of broken HW(4") casing abandoned in hole from 57.0 ft bgs (El. 307.9) to 67.0 ft bgs (El.297.9).
3) HP = Hydraulic Push

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-FFPB-103

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

Project: Puddle Dock Bridge #2691 carries Route 161 over Pattee Brook
Location: Fort Fairfield, Maine

Boring No.: BB-FFPB-103A
WIN: 25453.00

Driller:	S.W. Cole	Elevation (ft.)	364.8	Auger ID/OD:	5" Solid Stem
Operator:	Kevin/Brian	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	Wilder/Pukay	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/22/2022-8/23/2022	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	3+75.4, 9.2 ft Rt.	Casing ID/OD:	HW(4.0"/4.5")	Water Level*:	13.0 ft bgs.
Hammer Efficiency Factor: 0.91		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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MV = Unsuccessful Field Vane Shear Test Attempt
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SSA = Solid Stem Auger
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WOH = Weight of 140lb. Hammer
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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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA		14" HMA.	
								363.6	1.2 Reference BB-FFPB-103 for samples up to and including 65 ft bgs.	
5										
10										
								351.3	13.5	
15							HP			
							HP			
							38			
							83			
							66			
20							27			
							35			
							42			
							71			
							152			
25							109			
							75			
							48			
							36			
							78			
30							52			
							46			
							63	331.8	33.0	
							74			
							98			
35							47			
							68			
							70			
							113			
							96			
40							OPEN HOLE			
45										
50										
55										
60										
65										
70										
75										

Remarks:
1) Auto Hammer #367
2) HP = Hydraulic Push

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-FFPB-103A

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

Project: Puddle Dock Bridge #2691 carries Route 161 over Pattee Brook
Location: Fort Fairfield, Maine

Boring No.: BB-FFPB-103A
WIN: 25453.00

Driller:	S.W. Cole	Elevation (ft.)	364.8	Auger ID/OD:	5" Solid Stem
Operator:	Kevin/Brian	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	Wilder/Pukay	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/22/2022-8/23/2022	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	3+75.4, 9.2 ft Rt.	Casing ID/OD:	HW(4.0"/4.5")	Water Level*:	13.0 ft bgs.
Hammer Efficiency Factor: 0.91		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
75	1D	24/18	75.00 - 77.00	12/13/22/32	35	53			Grey, wet, hard, SILT, some sand, little gravel, (Glacial Till).	
80									ARTESIAN water pressure at 82.0-85.0 ft bgs.	
85										
90	2D	24/15	90.00 - 92.00	17/18/22/34	40	61			Grey, wet, very dense, GRAVEL, some sand, some silt, (Glacial Till).	
95								269.8	95.0	
	R1	60/58	95.00 - 100.00	RQD = 82%			NQ-2		Top of Bedrock at Elev. 269.8 ft. R1: Bedrock: Grey to dark greenish-grey, fine-grained, thin-bedded, SILTSTONE, moderately hard, fresh, joints dip at low to moderate angles, spaced moderately close, with rock flour evident on fracture planes, some quartz or calcite infilling. [Spragueville Formation] Rock Quality = Good. R1: Core Times (min:sec) 95.0-96.0 ft (3:01) 96.0-97.0 ft (3:04) 97.0-98.0 ft (3:00) 98.0-99.0 ft (3:01) 99.0-100.0 ft (3:03) 97% Recovery	
100	R2	36/36	100.00 - 103.00	RQD = 28%					R2: Bedrock: Grey to dark greenish-grey, fine-grained, thin-bedded, SILTSTONE, moderately hard, fresh, joints are moderately dipping and fresh with minor iron oxide staining, fracture zone near middle of run contains rock flour on fracture planes. [Spragueville Formation] Rock Quality = Poor. R2: Core Times (min:sec) 100.0-101.0 ft (3:28) 101.0-102.0 ft (2:45) 102.0-103.0 ft (3:45) Core Blocked 100% Recovery	
	R3	24/24	103.00 - 105.00	RQD = 88%					R3: Bedrock: Grey to dark greenish-grey, fine-grained, thin-bedded, SILTSTONE, moderately hard, fresh, one steeply dipping fracture through most of the run, fracture plane is fresh. [Spragueville Formation] Rock Quality = Good. R3: Core Times (min:sec) 103.0-104.0 ft (3:46) 104.0-105.0 ft (3:50) 100% Recovery	
105								259.8	105.0 Bottom of Exploration at 105.0 feet below ground surface.	
110										
115										
120										
125										

Remarks:
1) Auto Hammer #367
2) HP = Hydraulic Push

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-FFPB-103A

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
2545300
BRIDGE NO. 2691
WIN 25453.00
BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		
CHECKED-REVIEWED		
DESIGN2-DETAILED2	N. PUKAY	T. WHITE	OCT 2024
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

PUDDLE DOCK BRIDGE
PATTEE BROOK
FORT FAIRFIELD AROOSTOOK COUNTY

BORING LOGS

SHEET NUMBER
5
OF 5

Filename: ...\MSTA\009_BORING LOGS2.dgn Division: GEOTECH Username: Nathan.P.Pukay Date:11/1/2024

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

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

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

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

MODIFIED BURMISTER SYSTEM

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Density of Cohesionless Soils	Standard Penetration Resistance N_{60}-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_{60}-Value (blows per foot)	Approximate Undrained Shear Strength (psf)	Field Guidelines
Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily penetrates
Soft	2 - 4	250 - 500	Thumb easily penetrates
Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort
Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort
Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail
Hard	>30	over 4000	Indented by thumbnail with difficulty

Rock Quality Designation (RQD):

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

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

Rock Quality Based on RQD

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

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

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Puddle Dock Bridge #2691 carries Route 161 over Pattee Brook
Location: Fort Fairfield, Maine

Boring No.: BB-FFPB-101
WIN: 25453.00

Driller:	S.W. Cole	**Elevation (ft.)**	367.6	**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"
Date Start/Finish:	5/31/2022-6/1/2022	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+15.9, 6.7 ft Rt.	**Casing ID/OD:**	NW(3.0"/3.5"), HW(4.0"/4.5")	**Water Level*:**	8.0 ft bgs.
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			14" HMA.	
								366.4		1.2	
5	1D	24/11	5.00 - 7.00	5/4/9/4	13	20				Brown, damp, medium dense, Gravelly SAND, little silt, (Fill).	
10	2D	24/15	10.00 - 12.00	4/3/5/5	8	12	41			Brown, damp, medium dense, SAND, some silt, some gravel, (Fill).	G#241522 A-2-4, SM WC=11.6%
							41				
							48				
							38				
							40				
15	3D	24/4	15.00 - 17.00	3/2/2/1	4	6	10	352.6		15.0 Grey, wet, loose, Sandy GRAVEL, some silt, (Stream Alluvium).	
							17				
							18				
							22				
							10				
20							10				
	4D	24/3	21.00 - 23.00	5/3/4/2	7	11	16			Similar to 3D, except medium dense.	
							24				
							22				
							17				
25											

Remarks:

1) Auto Hammer #367
2) 15 feet of 3" casing (NW) abandoned in hole from 46.5 BGS (El. 321.1) to 61.5 BGS (El. 306.1)

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: Puddle Dock Bridge #2691 carries Route 161 over Pattee Brook
Location: Fort Fairfield, Maine

Boring No.: BB-FFPB-101
WIN: 25453.00

Driller:	S.W. Cole	**Elevation (ft.)**	367.6	**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"
Date Start/Finish:	5/31/2022-6/1/2022	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+15.9, 6.7 ft Rt.	**Casing ID/OD:**	NW(3.0"/3.5"), HW(4.0"/4.5")	**Water Level*:**	8.0 ft bgs.
Hammer Efficiency Factor:	0.91	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
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	24/11	25.00 - 27.00	7/10/6/6	16	24	43			Grey-brown, wet, medium dense, Sandy GRAVEL, some silt, (Stream Alluvium).	
							45				
							37				
							60				
							88				
30	MD	24/0	30.00 - 32.00	11/11/10/15	21	32	46				
							42				
							45	334.6		33.0	
	6D	24/14	33.00 - 35.00	11/9/9/9	18	27	50			Grey, wet, medium dense, fine SAND, little silt, (Stream Alluvium).	G#337506 A-2-4, SM WC=20.2%
							52				
35	7D/A	24/20	35.00 - 37.00	5/10/18/27	28	42	76			7D (35.0-36.5 ft bgs.) Similar to 6D, except dense.	
							146	331.1		36.5	
							132			7D/A (36.5-37.0 ft bgs.) Brown-grey, wet, dense, Gravelly SAND, some silt, (Glacial Till).	
							251				
							174				
40	8D	24/15	40.00 - 42.00	14/16/36/32	52	79	38			Brown-grey, wet, very dense, SAND, some silt, trace gravel, (Glacial Till). Roller Coned ahead from 40.0-45.0 ft bgs.	G#337507 A-1-b, SM WC=12.7%
							50				
							45				
							64				
							129				
45	9D	24/17	45.00 - 47.00	28/37/27/37	64	97	59			Brown, wet, very dense, Silty SAND, little gravel, (Glacial Till). Roller Coned ahead from 45.0-50.0 ft bgs.	
							87				
							125				
							118				
50							158				

Remarks:

1) Auto Hammer #367
2) 15 feet of 3" casing (NW) abandoned in hole from 46.5 BGS (El. 321.1) to 61.5 BGS (El. 306.1)

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: Puddle Dock Bridge #2691 carries Route 161 over Pattee Brook
Location: Fort Fairfield, Maine

Boring No.: BB-FFPB-101
WIN: 25453.00

Driller:	S.W. Cole	**Elevation (ft.)**	367.6	**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"
Date Start/Finish:	5/31/2022-6/1/2022	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+15.9, 6.7 ft Rt.	**Casing ID/OD:**	NW(3.0"/3.5"), HW(4.0"/4.5")	**Water Level*:**	8.0 ft bgs.
Hammer Efficiency Factor:	0.91	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon 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.
50	10D	24/19	50.00 - 52.00	18/24/31/50	55	83	OPEN HOLE			Brown, wet, very dense, SAND, some silt, trace gravel, (Glacial Till).	
55	11D	24/21	55.00 - 57.00	25/29/39/38	68	103				Similar to 10D.	
60	12D	18/17	60.00 - 61.50	30/46/55	101	153				Brown, wet, very dense, SAND, some silt, trace rock fragments. Set in NW casing and drove to 61.5 ft bgs.	
	R1						NQ-2	306.1		61.5 Cored from 61.5-71.5 ft bgs through cobbles and glacial till. Top of Rock not encountered.	
65											
70								296.1		71.5 **Bottom of Exploration at 71.5 feet below ground surface.** Hole Abandoned due to stuck casing. 15.0 ft of NW(3") Casing was left in hole from 46.5 ft bgs (El. 321.1) to 61.5 ft bgs (El. 306. 1).	
75											

Remarks:

1) Auto Hammer #367
2) 15 feet of 3" casing (NW) abandoned in hole from 46.5 BGS (El. 321.1) to 61.5 BGS (El. 306.1)

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Puddle Dock Bridge #2691 carries Route 161 over Pattee Brook
Location: Fort Fairfield, Maine

Boring No.: BB-FFPB-102
WIN: 25453.00

Driller:	S.W. Cole	**Elevation (ft.)**	368.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Wilder/Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/15/2022-8/16/2022	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+00.6, 6.6 ft Lt.	**Casing ID/OD:**	NW(3.0"/3.5"), HW(4.0"/4.5")	**Water Level*:**	16.0 ft bgs.
Hammer Efficiency Factor: 0.91		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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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 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			9" HMA.	
								367.6		0.8	
5	1D	24/15	5.00 - 7.00	4/7/7/5	14	21				Brown, damp, medium dense, SAND, some silt, some gravel, (Fill).	
10	2D	24/14	10.00 - 12.00	2/2/3/2	5	8				Brown, wet, medium stiff, Sandy SILT, little gravel, (Fill).	G#337508 A-4, CL WC=14.9%
15	3D	24/14	15.00 - 17.00	4/8/8/6	16	24		352.8		15.5 Brown, wet, medium dense, GRAVEL, some sand, little silt, (Stream Alluvium).	G#337509 A-1-b, GM WC=16.8%
20	4D	24/19	20.00 - 22.00	4/3/2/3	5	8	37			Grey, wet, loose, Sandy GRAVEL, some silt (Stream Alluvium).	G#337510 A-1-b, GM WC=26.0%
							46				
							59				
							77				
							68				
25											

Remarks:

1) Auto Hammer #367

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Puddle Dock Bridge #2691 carries Route 161 over Pattee Brook
Location: Fort Fairfield, Maine

Boring No.: BB-FFPB-102
WIN: 25453.00

Driller:	S.W. Cole	**Elevation (ft.)**	368.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Wilder/Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/15/2022-8/16/2022	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+00.6, 6.6 ft Lt.	**Casing ID/OD:**	NW(3.0"/3.5"), HW(4.0"/4.5")	**Water Level*:**	16.0 ft bgs.
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
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
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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
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WC = Water Content, percent
LL = Liquid Limit
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G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/15	25.00 - 27.00	13/9/7/13	16	24	72			Grey, wet, medium dense, GRAVEL, some sand, little silt, (Stream Alluvium).	
							58				
							49				
							46				
							49				
30	6D	24/14	30.00 - 32.00	14/8/7/7	15	23	46			Grey, wet, medium dense, Sandy GRAVEL, little silt, (Stream Alluvium).	
							43				
							51				
							83	335.3		33.0	
	7D	24/15	34.00 - 36.00	12/21/20/14	41	62	30			Grey, wet, very dense, SAND, some gravel, some silt, (Glacial Till).	G#337511 A-1-b, SM WC=11.1%
35							OPEN HOLE				
40	8D	24/13	40.00 - 42.00	17/12/10/19	22	33				Similar to 7D, except dense.	
45	9D	24/20	45.00 - 47.00	13/15/25/49	40	61				Brown, wet, very dense, SAND, some silt, little gravel, (Glacial Till).	G#337512 A-2-4, SM WC=13.7%
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Puddle Dock Bridge #2691 carries Route 161 over Pattee Brook
Location: Fort Fairfield, Maine

Boring No.: BB-FFPB-102
WIN: 25453.00

Driller:	S.W. Cole	**Elevation (ft.)**	368.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Wilder/Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/15/2022-8/16/2022	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+00.6, 6.6 ft Lt.	**Casing ID/OD:**	NW(3.0"/3.5"), HW(4.0"/4.5")	**Water Level*:**	16.0 ft bgs.
Hammer Efficiency Factor:	0.91	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140 lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
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	10D	18/16	50.00 - 51.50	36/39/65	104	158				Similar to above.	
55	11D	18/15	55.00 - 56.50	31/43/61	104	158				Brown, wet, very dense, SAND, little silt, little gravel, (Glacial Till).	
60											
								306.3		62.0	
65	12D	6/5	65.00 - 65.50	70(6")	---					Grey-brown, wet, very dense, GRAVEL, some sand, little silt, (Glacial Till).	
70											
75										Occasional Cobble.	

Remarks:

1) Auto Hammer #367

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Puddle Dock Bridge #2691 carries Route 161 over Pattee Brook	**Location:**	Fort Fairfield, Maine	**Boring No.:**	BB-FFPB-102
				WIN:	25453.00

Driller:	S.W. Cole	**Elevation (ft.)**	368.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Wilder/Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/15/2022-8/16/2022	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+00.6, 6.6 ft Lt.	**Casing ID/OD:**	NW(3.0"/3.5"), HW(4.0"/4.5")	**Water Level*:**	16.0 ft bgs.
Hammer Efficiency Factor:	0.91	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140 lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
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.
75	13D	24/18	75.00 - 77.00	42/43/42/38	85	129				Similar to 12D, except grey.	
80											
										Occasional Cobble.	
85	14D	18/16	85.00 - 86.50	22/25/63	88	133				Grey, wet, very dense, SAND, some gravel, little silt, (Glacial Till).	
90											
95	15D	24/14	95.00 - 97.00	9/9/16/23	25	38				Grey, wet, dense, Silty SAND, little gravel, (Glacial Till).	
100											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Puddle Dock Bridge #2691 carries Route 161 over Pattee Brook
Location: Fort Fairfield, Maine

Boring No.: BB-FFPB-102
WIN: 25453.00

Driller:	S.W. Cole	**Elevation (ft.)**	368.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Wilder/Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/15/2022-8/16/2022	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+00.6, 6.6 ft Lt.	**Casing ID/OD:**	NW(3.0"/3.5"), HW(4.0"/4.5")	**Water Level*:**	16.0 ft bgs.
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
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
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C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
100											
105	R1	26.4/24	105.10 - 107.30	RQD = 76%			NQ-2	263.2		Cobble at 104.8 ft bgs. Roller Coned ahead to 105.1 ft bgs. 105.1	
										Top of bedrock at Elev. 263.2 ft. R1: Bedrock: Grey to dark greenish-grey, fine-grained, thin-bedded, SILTSTONE, moderately hard, fresh, steeply dipping joints, closely spaced, with some quartz or calcite infilling. [Spragueville Formation] Rock Quality = Good. R1: Core Times (min:sec) 105.1-106.1 ft (2:01) 106.1-107.1 ft (2:04) 107.1-107.3 ft (1:04) Core Blocked 92% Recovery	
	R2	9.6/7	107.30 - 108.10	RQD = 0%						R2: Bedrock: Similar to R1 except with a vertical fracture throughout run. Fracture plane is fresh with minor iron oxide staining. [Spragueville Formation] Rock Quality = Very Poor. R2: Core Times (min:sec) 107.3-108.1 ft (2:32) Core Blocked 70% Recovery	
	R3	48/48	108.10 - 112.10	RQD = 48%						R3: Bedrock: Grey to dark greenish-grey, fine-grained, thin-bedded, SILTSTONE, moderately hard, fresh, vertical joints, closely spaced, with some quartz or calcite infilling, fracture planes are fresh with minor oxide staining, core becomes more competent throughout run. [Spragueville Formation] Rock Quality = Poor. R3: Core Times (min:sec) 108.1-109.1 ft (2:42) 109.1-110.1 ft (3:02) 110.1-111.1 ft (2:46) 111.1-112.1 ft (3:00) 100% Recovery	
110											
	R4	36/35	112.10 - 115.10	RQD = 92%						R4: Bedrock: Grey to dark greenish-grey, fine-grained, thin-bedded, SILTSTONE, moderately hard, fresh, joints dipping at moderate angles, spaced moderately close. [Spragueville Formation] Rock Quality = Excellent. R4: Core Times (min:sec) 112.1-113.1 ft (1:59)	
115								253.2			
120											
125											

Remarks:

1) Auto Hammer #367

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Puddle Dock Bridge #2691 carries Route 161 over Pattee Brook	**Location:**	Fort Fairfield, Maine	**Boring No.:**	BB-FFPB-102
				WIN:	25453.00
Driller:	S.W. Cole	**Elevation (ft.)**	368.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Wilder/Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/15/2022-8/16/2022	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+00.6, 6.6 ft Lt.	**Casing ID/OD:**	NW(3.0"/3.5"), HW(4.0"/4.5")	**Water Level*:**	16.0 ft bgs.
Hammer Efficiency Factor:	0.91	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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MD = Unsuccessful Split Spoon Sample Attempt
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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.
125										113.1-114.1 ft (2:17) 114.1-115.1 ft (2:20) 97% Recovery 115.1 **Bottom of Exploration at 115.1 feet below ground surface.**	
130											
135											
140											
145											
150											

Remarks:

1) Auto Hammer #367

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Puddle Dock Bridge #2691 carries Route 161 over Pattee Brook
Location: Fort Fairfield, Maine

Boring No.: BB-FFPB-103
WIN: 25453.00

Driller:	S.W. Cole	**Elevation (ft.)**	364.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Wilder/Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/17,22/2022	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	3+75.6, 6.2 ft Rt.	**Casing ID/OD:**	HW(4.0"/4.5")	**Water Level*:**	13.0 ft bgs.
Hammer Efficiency Factor: 0.91		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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
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G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			14" HMA.	
								363.7		1.2	
5	1D	24/19	5.00 - 7.00	4/3/4/4	7	11				Brown, dry, medium dense, SAND, some silt, trace gravel, (Fill).	G#337513 A-2-4, SM WC=14.5%
10	2D	24/19	10.00 - 12.00	4/2/2/2	4	6				Brown, moist, loose, Silty SAND, trace gravel, trace wood, (Fill).	G#337514 A-4, SM WC=37.3%
								351.4		13.5	
15	3D	24/6	15.00 - 17.00	2/2/1/2	3	5	HP			Brown, wet, very loose, SAND, some gravel, some silt, (Stream Alluvium).	
							11				
							33				
							56				
							43				
20	MD	24/0	20.00 - 22.00	14/9/2/2	11	17	45				
							34				
							35				
							62				
							50	340.9		24.0	
25											

Remarks:

1) Auto Hammer #367
2) 10.0 ft of broken HW(4") casing abandoned in hole from 57.0 ft bgs (El. 307.9) to 67.0 ft bgs (El.297.9).
3) HP = Hydraulic Push

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Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Puddle Dock Bridge #2691 carries Route 161 over Pattee Brook
Location: Fort Fairfield, Maine

Boring No.: BB-FFPB-103
WIN: 25453.00

Driller:	S.W. Cole	**Elevation (ft.)**	364.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Wilder/Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/17,22/2022	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	3+75.6, 6.2 ft Rt.	**Casing ID/OD:**	HW(4.0"/4.5")	**Water Level*:**	13.0 ft bgs.
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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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.
25	4D	24/13	25.00 - 27.00	15/16/11/10	27	41	48			Grey, wet, dense, GRAVEL, some sand, little silt, (Stream Alluvium).	
							62				
							67				
							34				
							47				
30	5D	24/4	30.00 - 32.00	10/18/36/20	44	67	56			Grey, wet, very dense, GRAVEL, trace sand, trace silt, (Stream Alluvium).	
							77				
							47				
							69	331.9		33.0	
	6D	24/19	34.00 - 36.00	8/15/21/18	36	55	30			Grey-brown, wet, very dense, Gravelly SAND, little silt, (Glacial Till).	G#337515 A-1-b, SM WC=11.7%
35							91				
							98				
							67				
							65				
	7D	24/14	39.00 - 41.00	8/8/9/12	17	26	26			Brown, wet, medium dense, SAND, trace gravel, trace silt, (Glacial Till).	
40							42				
							68				
							77				
							84				
							73				
45	8D	24/15	45.00 - 47.00	16/17/22/17	39	59	74			Brown, wet, very dense, SAND, some gravel, little silt, (Glacial Till).	
							99				
							90				
							113				
50							122				

Remarks:

1) Auto Hammer #367
2) 10.0 ft of broken HW(4") casing abandoned in hole from 57.0 ft bgs (El. 307.9) to 67.0 ft bgs (El.297.9).
3) HP = Hydraulic Push

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Puddle Dock Bridge #2691 carries Route 161 over Pattee Brook
Location: Fort Fairfield, Maine

Boring No.: BB-FFPB-103
WIN: 25453.00

Driller:	S.W. Cole	**Elevation (ft.)**	364.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Wilder/Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/17,22/2022	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	3+75.6, 6.2 ft Rt.	**Casing ID/OD:**	HW(4.0"/4.5")	**Water Level*:**	13.0 ft bgs.
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
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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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	24/16	50.00 - 52.00	22/21/20/25	41	62	97			Similar to 8D.	
							116				
							121				
							129				
							116			Occasional Cobble.	
55	10D	24/15	55.00 - 57.00	17/19/30/49	49	74	a57 OPEN HOLE			a57 blows for 0.5 ft. Brown, wet, very dense, Gravelly SAND, trace silt, (Glacial Till).	G#337516 A-1-b, SW-SM WC=8.1%
60											
65	11D	24/18	65.00 - 67.00	22/31/34/39	65	99				Brown, wet, very dense, SAND, little gravel, little silt, (Glacial Till).	
67.0								297.9		**Bottom of Exploration at 67.0 feet below ground surface.** Hole Abandoned. 10.0 ft of broken HW(4") casing abandoned in hole from 57. 0 ft bgs (El. 307.9) to 67.0 ft bgs (El. 297.9).	
70											
75											

Remarks:

1) Auto Hammer #367
2) 10.0 ft of broken HW(4") casing abandoned in hole from 57.0 ft bgs (El. 307.9) to 67.0 ft bgs (El.297.9).
3) HP = Hydraulic Push

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Puddle Dock Bridge #2691 carries Route 161 over Pattee Brook **Location:** Fort Fairfield, Maine	**Boring No.:** BB-FFPB-103A **WIN:** 25453.00

Driller:	S.W. Cole	**Elevation (ft.)**	364.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Wilder/Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/22/2022-8/23/2022	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+75.4, 9.2 ft Rt.	**Casing ID/OD:**	HW(4.0"/4.5")	**Water Level*:**	13.0 ft bgs.
Hammer Efficiency Factor: 0.91		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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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
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
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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			14" HMA.	
								363.6		1.2	
										Reference BB-FFPB-103 for samples up to and including 65 ft bgs.	
5											
10											
								351.3		13.5	
15							HP				
							HP				
							38				
							83				
							66				
20							27				
							35				
							42				
							71				
							152				
25											

Remarks:

1) Auto Hammer #367
2) HP = Hydraulic Push

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Puddle Dock Bridge #2691 carries Route 161 over Pattee Brook **Location:** Fort Fairfield, Maine	**Boring No.:** BB-FFPB-103A **WIN:** 25453.00

Driller:	S.W. Cole	**Elevation (ft.)**	364.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Wilder/Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/22/2022-8/23/2022	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+75.4, 9.2 ft Rt.	**Casing ID/OD:**	HW(4.0"/4.5")	**Water Level*:**	13.0 ft bgs.
Hammer Efficiency Factor:	0.91	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							109				
							75				
							48				
							36				
							78				
30							52				
							46				
							63				
							74	331.8		33.0	
							98				
35							47				
							68				
							70				
							113				
							96				
40							OPEN HOLE				
45											
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Puddle Dock Bridge #2691 carries Route 161 over Pattee Brook
Location: Fort Fairfield, Maine

Boring No.: BB-FFPB-103A

WIN: 25453.00

Driller:	S.W. Cole	**Elevation (ft.)**	364.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Wilder/Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/22/2022-8/23/2022	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+75.4, 9.2 ft Rt.	**Casing ID/OD:**	HW(4.0"/4.5")	**Water Level*:**	13.0 ft bgs.
Hammer Efficiency Factor:	0.91	**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.
50											
55											
60											
65											
70											
75											

Remarks:

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US CUSTOMARY UNITS

Project: Puddle Dock Bridge #2691 carries Route 161 over Pattee Brook
Location: Fort Fairfield, Maine

Boring No.: BB-FFPB-103A

WIN: 25453.00

Driller:	S.W. Cole	**Elevation (ft.)**	364.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Wilder/Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/22/2022-8/23/2022	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+75.4, 9.2 ft Rt.	**Casing ID/OD:**	HW(4.0"/4.5")	**Water Level*:**	13.0 ft bgs.
Hammer Efficiency Factor:	0.91	**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.
75	1D	24/18	75.00 - 77.00	12/13/22/32	35	53				Grey, wet, hard, SILT, some sand, little gravel, (Glacial Till).	
80											
										ARTESIAN water pressure at 82.0-85.0 ft bgs.	
85											
90	2D	24/15	90.00 - 92.00	17/18/22/34	40	61				Grey, wet, very dense, GRAVEL, some sand, some silt, (Glacial Till).	
95	R1	60/58	95.00 - 100.00	RQD = 82%			NQ-2	269.8		95.0 Top of Bedrock at Elev. 269.8 ft. R1: Bedrock: Grey to dark greenish-grey, fine-grained, thin-bedded, SILTSTONE, moderately hard, fresh, joints dip at low to moderate angles, spaced moderately close, with rock flour evident on fracture planes, some quartz or calcite infilling. [Spragueville Formation] Rock Quality = Good. R1: Core Times (min:sec) 95.0-96.0 ft (3:01) 96.0-97.0 ft (3:04)	
100											

Remarks:

1) Auto Hammer #367
2) HP = Hydraulic Push

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:** Puddle Dock Bridge #2691 carries Route 161 over Pattee Brook **Location:** Fort Fairfield, Maine	**Boring No.:** BB-FFPB-103A **WIN:** 25453.00

Driller:	S.W. Cole	**Elevation (ft.)**	364.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Kevin/Brian	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Wilder/Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/22/2022-8/23/2022	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+75.4, 9.2 ft Rt.	**Casing ID/OD:**	HW(4.0"/4.5")	**Water Level*:**	13.0 ft bgs.
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 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.
100	R2	36/36	100.00 - 103.00	RQD = 28%						97.0-98.0 ft (3:00) 98.0-99.0 ft (3:01) 99.0-100.0 ft (3:03) 97% Recovery R2: Bedrock: Grey to dark greenish-grey, fine-grained, thin-bedded, SILTSTONE, moderately hard, fresh, joints are moderately dipping and fresh with minor iron oxide staining, fracture zone near middle of run contains rock flour on fracture planes. [Spragueville Formation] Rock Quality = Poor. R2: Core Times (min:sec) 100.0-101.0 ft (3:28) 101.0-102.0 ft (2:45) 102.0-103.0 ft (3:45) Core Blocked 100% Recovery	
	R3	24/24	103.00 - 105.00	RQD = 88%						R3: Bedrock: Grey to dark greenish-grey, fine-grained, thin-bedded, SILTSTONE, moderately hard, fresh, one steeply dipping fracture through most of the run, fracture plane is fresh. [Spragueville Formation] Rock Quality = Good. R3: Core Times (min:sec) 103.0-104.0 ft (3:46) 104.0-105.0 ft (3:50) 100% Recovery	
105								259.8		105.0 **Bottom of Exploration at 105.0 feet below ground surface.**	
110											
115											
120											
125											

Remarks:

1) Auto Hammer #367
2) HP = Hydraulic Push

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

Puddle Dock Bridge #2691 Carries Route 161 Over Pattee Brook

Fort Fairfield, ME

Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-FFPB-102	R1	105.1-107.3	26	24	20	76	SILTSTONE	1
BB-FFPB-102	R2	107.3-108.1	10	7	0	0	SILTSTONE	1
BB-FFPB-102	R3	108.1-112.1	48	48	23	48	SILTSTONE	1+2
BB-FFPB-102	R4	112.1-115.1	36	35	33	92	SILTSTONE	2
BB-FFPB-103A	R1	95.0-100.0	60	58	49	82	SILTSTONE	3
BB-FFPB-103A	R2	100.1-103.0	36	36	10	28	SILTSTONE	4
BB-FFPB-103A	R3	103.0-105.0	24	24	21	88	SILTSTONE	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.

3. Transition between core runs is marked by wooden blocks.

Appendix C

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Fort Fairfield Work Number: 25453.00

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-FFPB-101, 2D	3+15.9	6.7 Rt.	10.0-12.0	241522	1	11.6			SM	A-2-4	II
BB-FFPB-101, 6D	3+15.9	6.7 Rt.	33.0-35.0	337506	1	20.2			SM	A-2-4	II
BB-FFPB-101, 8D	3+15.9	6.7 Rt.	40.0-42.0	337507	1	12.7			SM	A-1-b	II
BB-FFPB-102, 2D	3+00.6	6.6 Lt.	10.0-12.0	337508	2	14.9			CL	A-4	IV
BB-FFPB-102, 3D	3+00.6	6.6 Lt.	15.0-17.0	337509	2	16.8			GM	A-1-b	I
BB-FFPB-102, 4D	3+00.6	6.6 Lt.	20.0-22.0	337510	2	26.0			GM	A-1-b	I
BB-FFPB-102, 7D	3+00.6	6.6 Lt.	34.0-36.0	337511	2	11.1			SM	A-1-b	II
BB-FFPB-102, 9D	3+00.6	6.6 Lt.	45.0-47.0	337512	2	13.7			SM	A-2-4	II
BB-FFPB-103, 1D	3+75.6	6.2 Rt.	5.0-7.0	337513	3	14.5			SM	A-2-4	II
BB-FFPB-103, 2D	3+75.6	6.2 Rt.	10.0-12.0	337514	3	37.3			SM	A-4	III
BB-FFPB-103, 6D	3+75.6	6.2 Rt.	34.0-36.0	337515	3	11.7			SM	A-1-b	II
BB-FFPB-103, 10D	3+75.6	6.2 Rt.	55.0-57.0	337516	3	8.1			SW-SM	A-1-b	0

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.

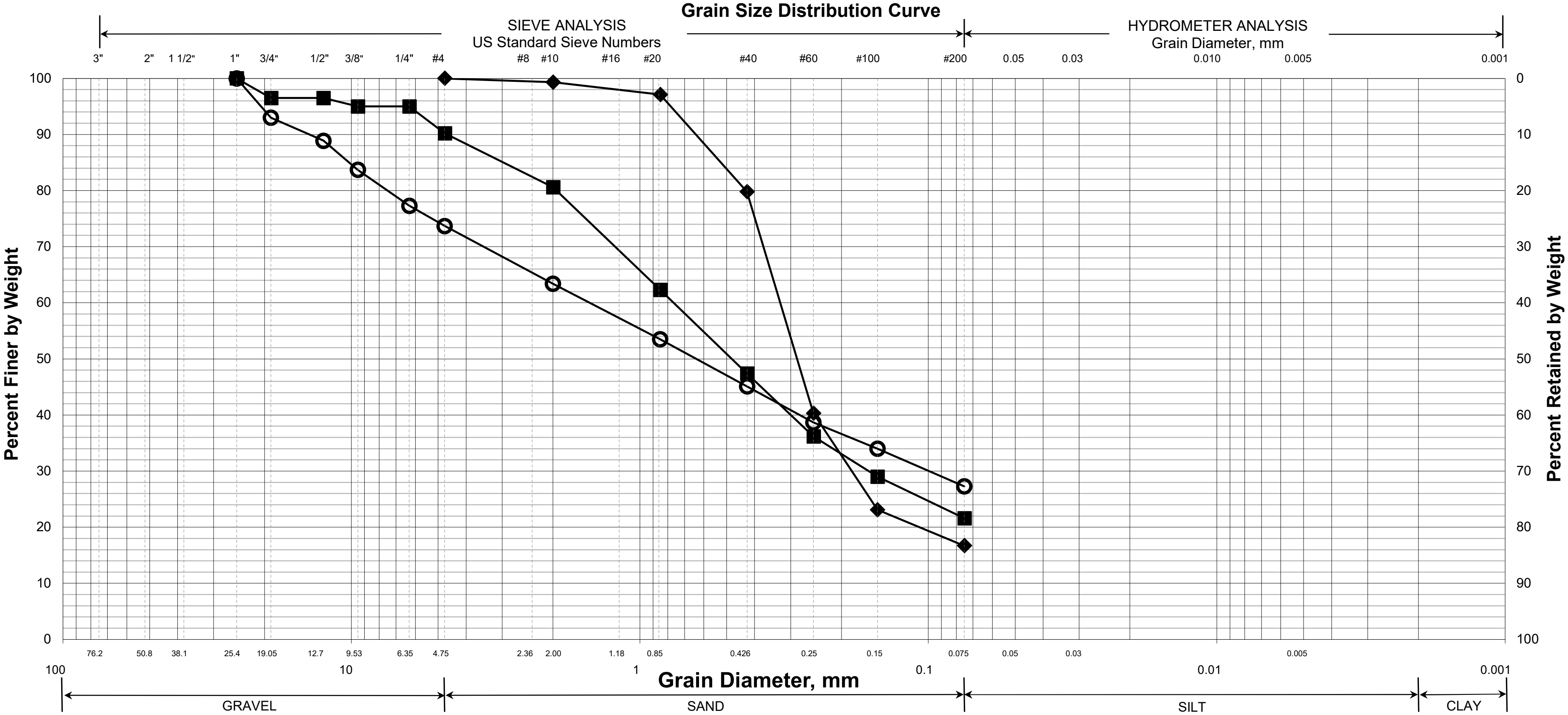
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

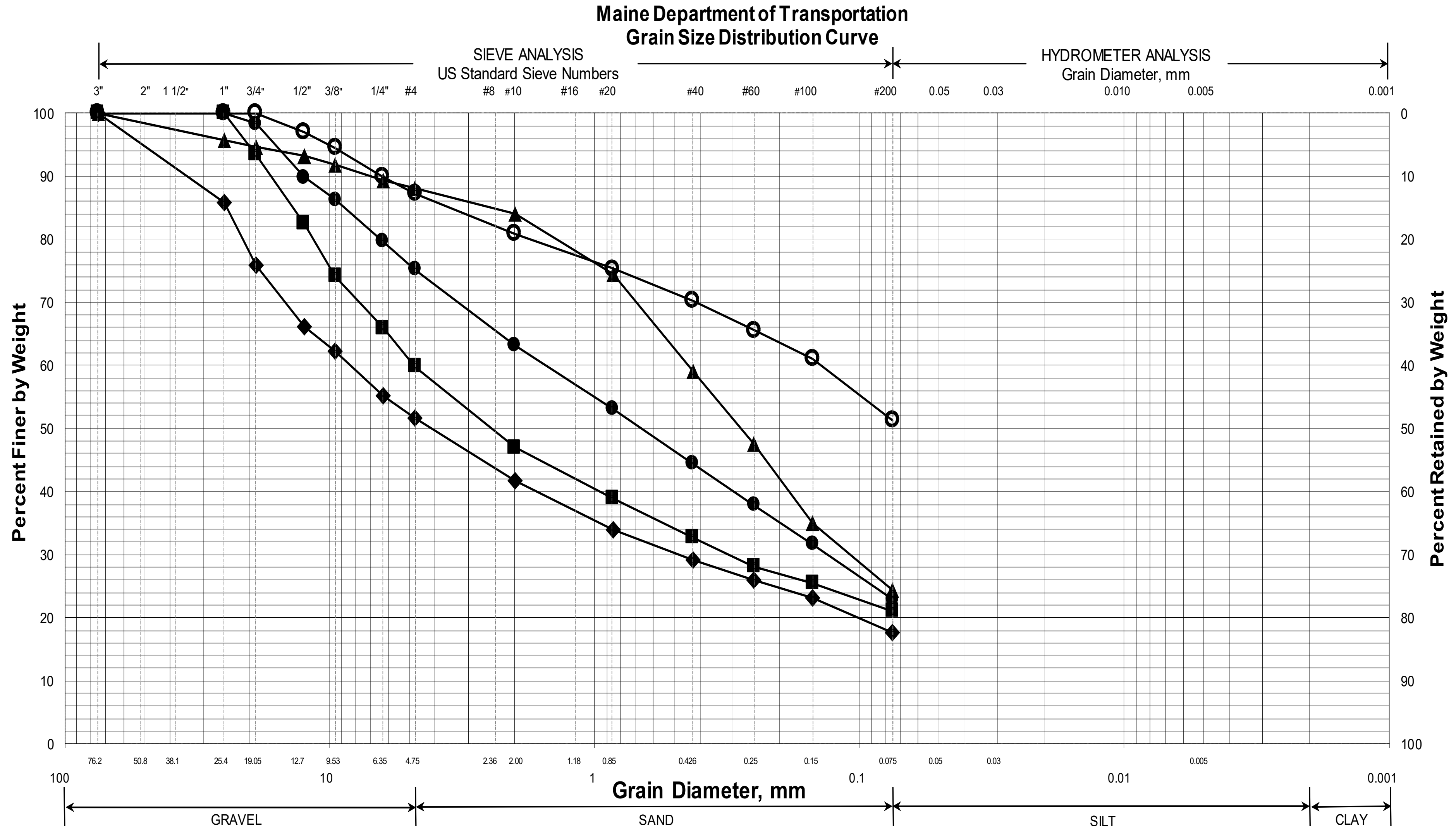


UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-FFPB-101/2D	3+15.9	6.7 RT	10.0-12.0	SAND, some silt, some gravel.	11.6			
◆	BB-FFPB-101/6D	3+15.9	6.7 RT	33.0-35.0	SAND, little silt.	20.2			
■	BB-FFPB-101/8D	3+15.9	6.7 RT	40.0-42.0	SAND, some silt, trace gravel.	12.7			
●									
▲									
×									

WIN
025453.00
Town
Fort Fairfield
Reported by/Date
WHITE, TERRY A 10/31/2022

SHEET 1



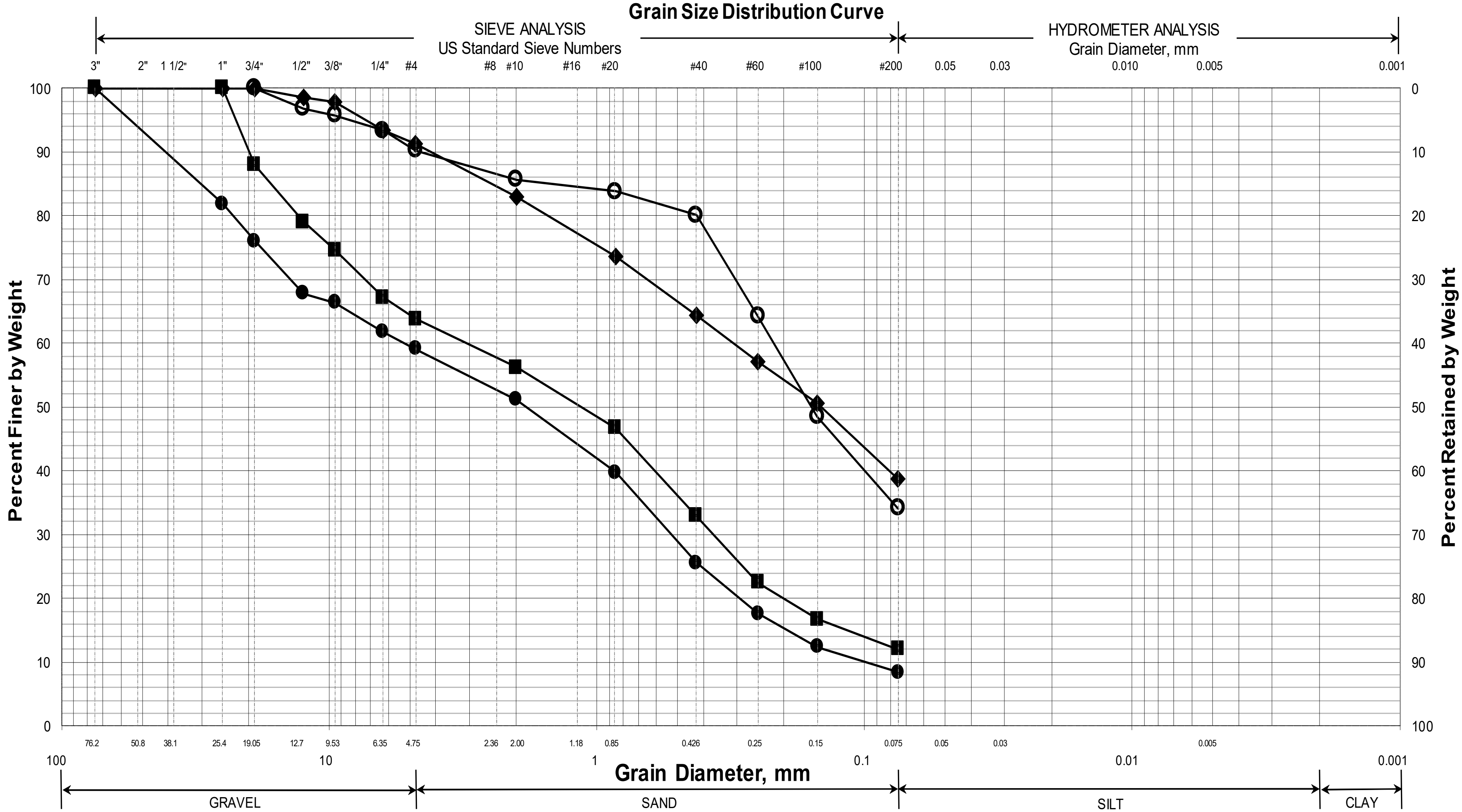
UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-FFPB-102/2D	3+00.6	6.6 LT	10.0-12.0	Sandy SILT, little gravel.	14.9			
◆	BB-FFPB-102/3D	3+00.6	6.6 LT	15.0-17.0	GRAVEL, some sand, little silt.	16.8			
■	BB-FFPB-102/4D	3+00.6	6.6 LT	20.0-22.0	Sandy GRAVEL, some silt.	26			
●	BB-FFPB-102/7D	3+00.6	6.6 LT	34.0-36.0	SAND, some gravel, some silt.	11.1			
▲	BB-FFPB-102/9D	3+00.6	6.6 LT	45.0-47.0	SAND, some silt, little gravel.	13.7			
×									

WIN	
025453.00	
Town	
Fort Fairfield	
Reported by/Date	
WHITE, TERRY A	10/31/2022

SHEET 2

Maine Department of Transportation
Grain Size Distribution Curve



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-FFPB-103/1D	3+75.6	6.2 RT	5.0-7.0	SAND, some silt, trace gravel.	14.5			
◆	BB-FFPB-103/2D	3+75.6	6.2 RT	10.0-12.0	Silty SAND, trace gravel.	37.3			
■	BB-FFPB-103/6D	3+75.6	6.2 RT	34.0-36.0	Gravelly SAND, little silt.	11.7			
●	BB-FFPB-103/10D	3+75.6	6.2 RT	55.0-57.0	Gravelly SAND, trace silt.	8.1			
▲									
×									

WIN	
025453.00	
Town	
Fort Fairfield	
Reported by/Date	
WHITE, TERRY A	10/31/2022

SHEET 3

Appendix D

Calculations

Driven H-Pile Resistance

Design of H-piles

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

Bedrock Properties

BB-FFPB-102, R1 RQD = 76%, R2 RQD = 0%, R3 RQD = 92%
Rock Type: SILTSTONE (moderately hard), fresh

BB-FFPB-103A, R1 RQD = 82%, R2 RQD = 28%, R3 RQD = 88%
Rock Type: SILTSTONE (moderately hard), fresh

Siltstone Co = 1,400-17,000 psi

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

For Design Purposes: RQD = 50%, Co = 8500 psi

Pile Properties

Use the following piles: 14x89, 14x117

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

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

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

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

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

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

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

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

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

LRFD Article 6.9.4.1.1 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:

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

LRFD Eq. 6.9.4.1.1-1

then:

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

Factored Axial Structural Resistance for the Strength Limit State

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

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

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

LRFD 10.7.3.2.3 - Piles Driven to Hard Rock -

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

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

Nominal Structural Resistance Previously Calculated:

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

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

$$\phi_c := 0.5$$

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

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

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

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

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

14x89
14x117

Drivability Analyses

Ref: LRFD Article 10.7.8

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

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

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

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

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

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

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

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

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

GRLWeap Soil and Pile Model Assumptions

Abutment #1:
Based on proposed bottom of footing of elevation 358.1 at abutment #1, the estimated pile length will be approx. 95 feet. Assume contractor drives pile lengths of 100 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. 280 kips as skin friction based on local experience in similar deposits.

Abutment #2:
Based on proposed bottom of footing of elevation 355.2 at abutment #2, the estimated pile length will be approx. 86 feet. Assume contractor drives pile lengths of 95 ft (extra length accommodates for attachment of dynamic testing equipment, embedment into abutment, variation in bedrock surface).

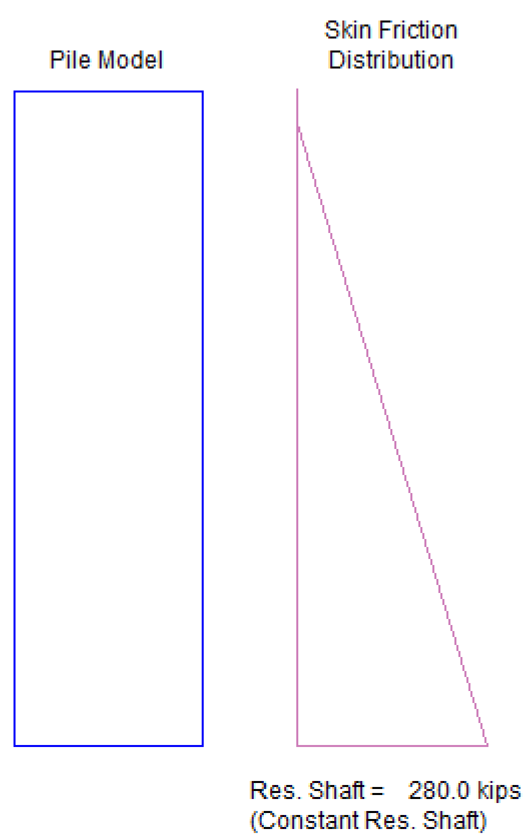
Use constant shaft resistances so that GRLWeap will assign approx. 250 kips as skin friction based on local experience in similar deposits.

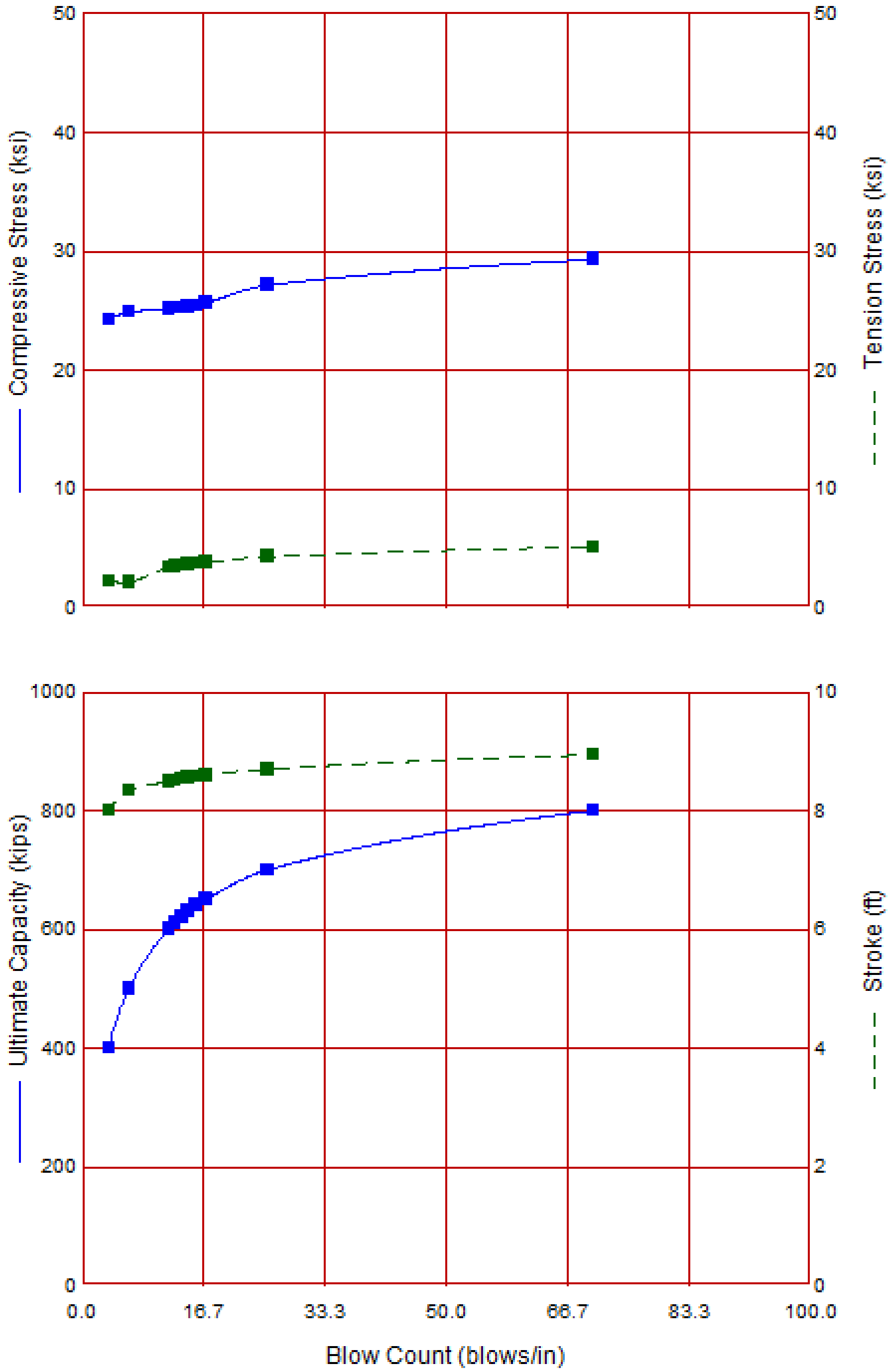
Abutment 1, Pile Size is 14 x 89, APE D19-42 Hammer

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	0.070	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	100.00	ft
Pile Penetration	94.90	ft
Pile Top Area	26.10	in2





Compressive Stress (ksi)
Tension Stress (ksi)
Ultimate Capacity (kips)
Stroke (ft)
Blow Count (blows/in)
0.0
16.7
33.3
50.0
66.7
83.3
100.0

Maine DOT
25453 Fort Fairfield 14x89 ABT #1 D19-42

19-Apr-2024
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	24.18	2.20	3.7	8.01	21.12
500.0	24.85	2.09	6.4	8.34	21.96
600.0	25.15	3.37	11.9	8.49	22.37
610.0	25.21	3.47	12.7	8.51	22.43
620.0	25.25	3.57	13.6	8.54	22.48
630.0	25.33	3.65	14.4	8.56	22.62
640.0	25.36	3.73	15.6	8.57	22.62
650.0	25.59	3.81	16.9	8.60	22.67
700.0	27.11	4.26	25.4	8.70	23.02
800.0	29.29	5.09	70.2	8.94	23.63

Limit to 15 bpi

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

Strength Limit State

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

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

Extreme and
Service Limit States

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

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

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

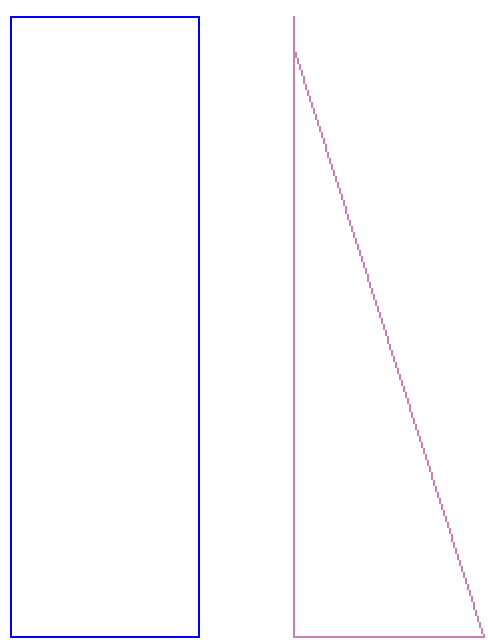
The 14x89 pile can be driven to the resistances below with a APE D25-42 hammer at fuel setting 4 (100% of max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE D 25-42

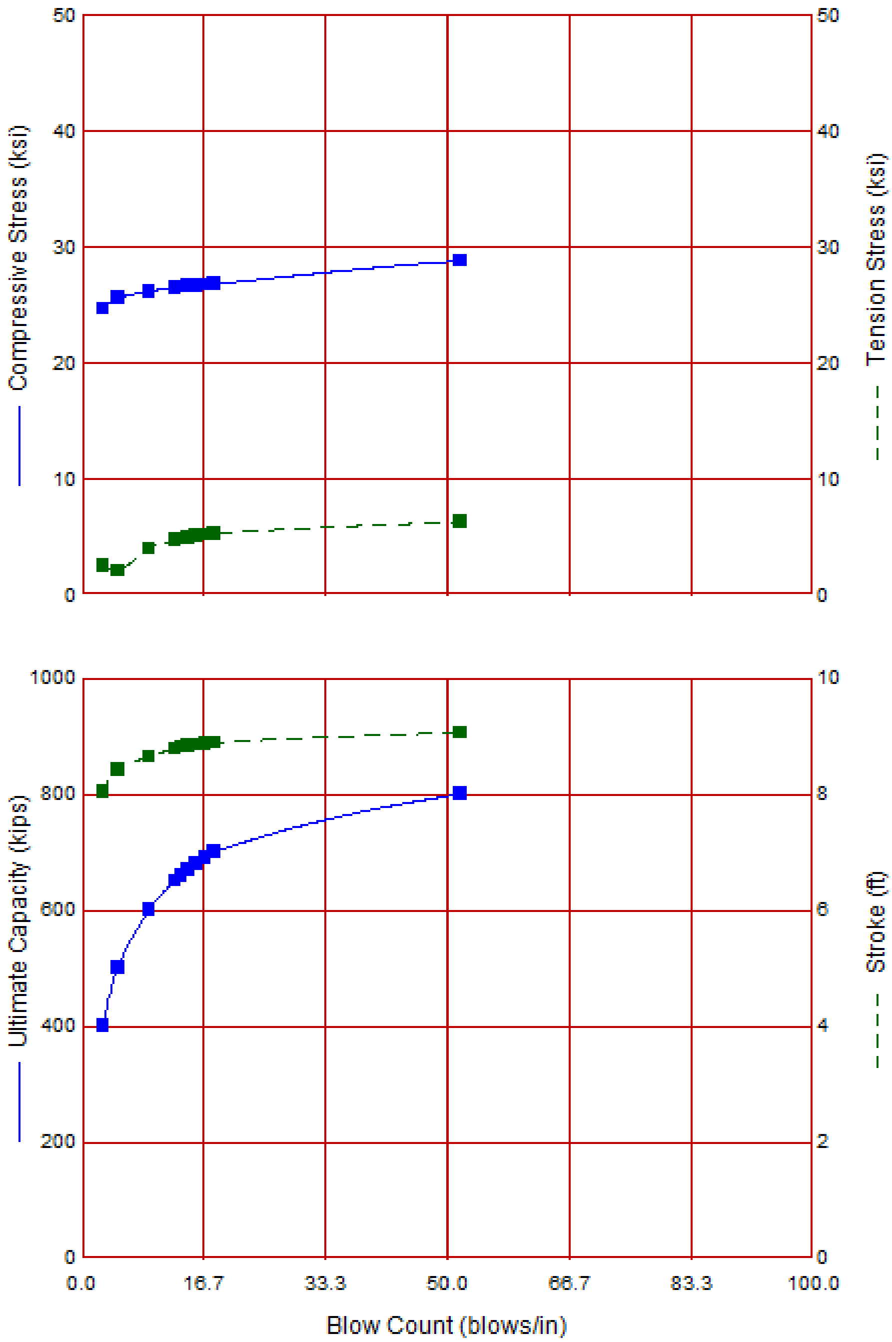
Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1425 (100%)	psi
Helmet Weight	3.00	kips
Hammer Cushion	34825	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.070	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	100.00	ft
Pile Penetration	94.90	ft
Pile Top Area	26.10	in2

Pile Model

Skin Friction Distribution



Res. Shaft = 280.0 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
16.7
33.3
50.0
66.7
83.3
100.0
Blow Count (blows/in)

Maine DOT
25453 Fort Fairfield 14x89 ABT #1 D25-42

19-Apr-2024
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	24.67	2.54	2.8	8.05	24.84
500.0	25.61	2.11	4.8	8.42	26.21
600.0	26.20	4.03	9.1	8.65	27.02
650.0	26.54	4.74	12.7	8.78	27.52
660.0	26.61	4.87	13.5	8.81	27.63
670.0	26.66	4.98	14.4	8.83	27.74
680.0	26.71	5.10	15.6	8.85	27.78
690.0	26.74	5.21	16.8	8.87	27.83
700.0	26.83	5.31	18.0	8.89	27.96
800.0	28.82	6.30	51.6	9.06	28.54

Limit to 15 bpi

$$R_{ndr} := 670 \cdot kip$$

Strength Limit State

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

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

Extreme and
Service Limit States

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

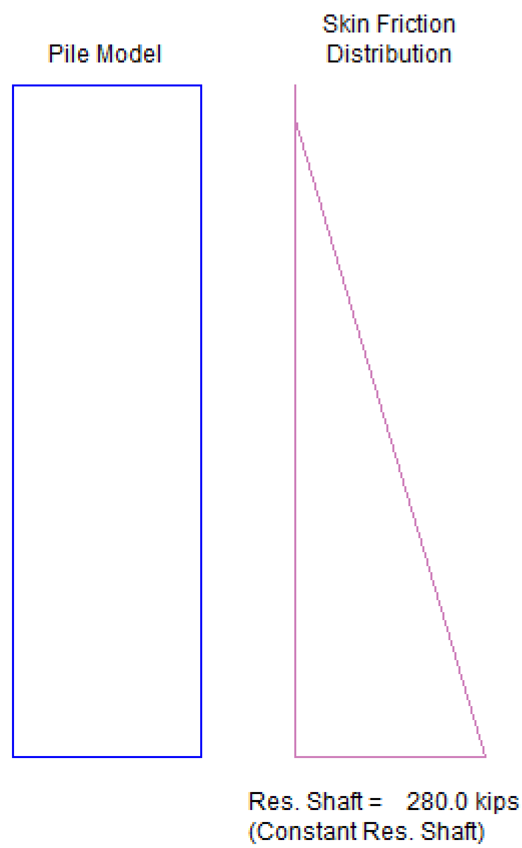
$$\boxed{R_{dr} = 670 \cdot kip}$$

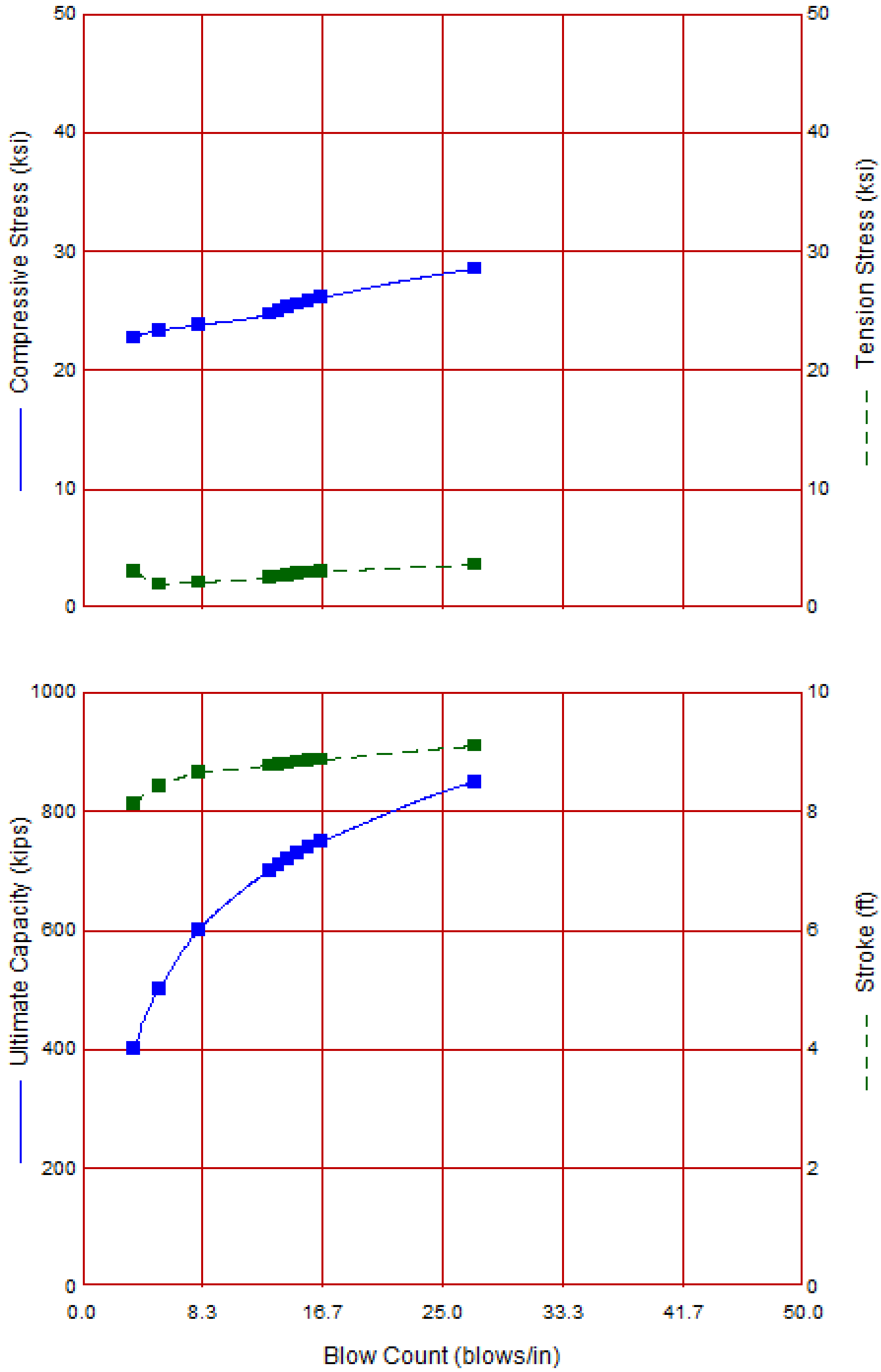
Abutment 1, Pile Size is 14 x 117, APE D19-42 Hammer

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

APE D 19-42

Ram Weight	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	0.070	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	100.00	ft
Pile Penetration	94.90	ft
Pile Top Area	34.40	in2




Compressive Stress (ksi)
Tension Stress (ksi)
Ultimate Capacity (kips)
Stroke (ft)
Blow Count (blows/in)
0.0
8.3
16.7
25.0
33.3
41.7
50.0

Maine DOT
25453 Fort Fairfield 14x117 ABT#1 D19-42

19-Apr-2024
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	22.73	3.10	3.6	8.12	20.02
500.0	23.33	2.00	5.4	8.42	20.82
600.0	23.79	2.15	8.1	8.65	21.45
700.0	24.71	2.57	13.0	8.77	21.73
710.0	24.99	2.68	13.6	8.79	21.78
720.0	25.26	2.78	14.2	8.81	21.89
730.0	25.54	2.87	14.9	8.84	21.94
740.0	25.82	2.96	15.7	8.86	22.00
750.0	26.09	3.03	16.6	8.87	22.02
850.0	28.50	3.63	27.3	9.10	22.70

Limit to 15 bpi

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

Strength Limit State

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

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

Extreme and
Service Limit States

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

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

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

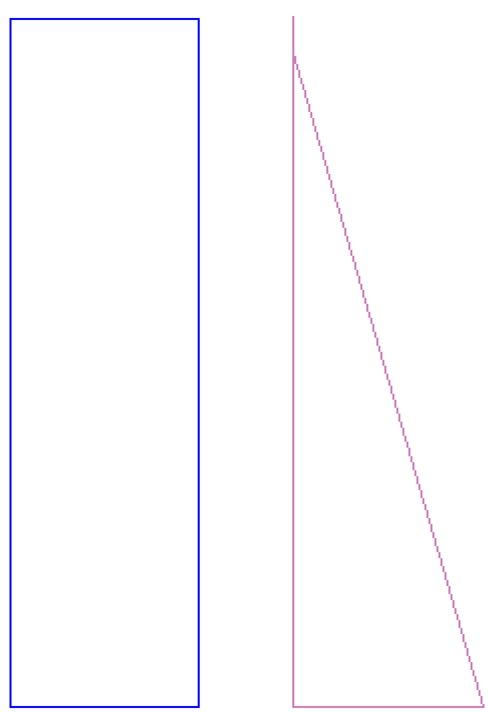
The 14x117 pile can be driven to the resistances below with a APE D25-42 hammer at fuel setting 4 (100% of max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE D 25-42

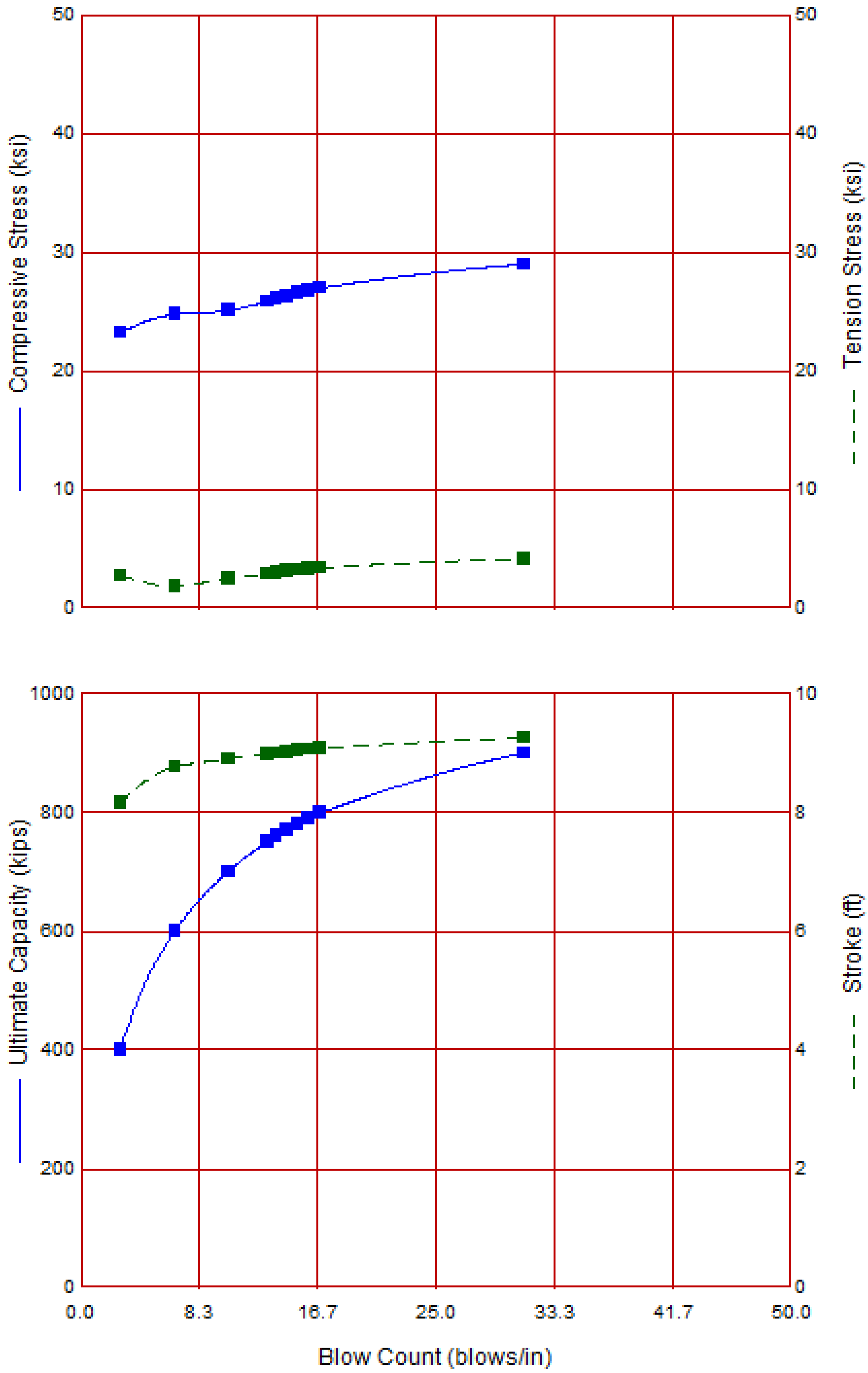
Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1425 (100%)	psi
Helmet Weight	3.00	kips
Hammer Cushion	34825	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.070	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	100.00	ft
Pile Penetration	94.90	ft
Pile Top Area	34.40	in2

Pile Model

Skin Friction Distribution



Res. Shaft = 280.0 kips
(Constant Res. Shaft)


50
40
30
20
10
0
Compressive Stress (ksi)
Tension Stress (ksi)
1000
800
600
400
200
0
10
8
6
4
2
Ultimate Capacity (kips)
Stroke (ft)
0.0
8.3
16.7
25.0
33.3
41.7
50.0
Blow Count (blows/in)

Maine DOT 19-Apr-2024
25453 Fort Fairfield 14x117 ABT#1 D25-42 GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	23.27	2.74	2.8	8.16	23.23
600.0	24.77	1.91	6.6	8.77	25.34
700.0	25.10	2.52	10.4	8.90	25.84
750.0	25.85	2.94	13.1	8.99	26.13
760.0	26.10	3.04	13.8	9.00	26.19
770.0	26.33	3.14	14.5	9.02	26.22
780.0	26.60	3.24	15.3	9.04	26.28
790.0	26.79	3.33	16.0	9.06	26.39
800.0	27.01	3.43	16.9	9.08	26.45
900.0	29.01	4.19	31.2	9.26	27.08

Limit to 15 bpi

$R_{ndr} := 770 \cdot kip$

Strength Limit State

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

$\boxed{R_{fdr} = 501 \cdot kip}$

Extreme and
Service Limit States

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

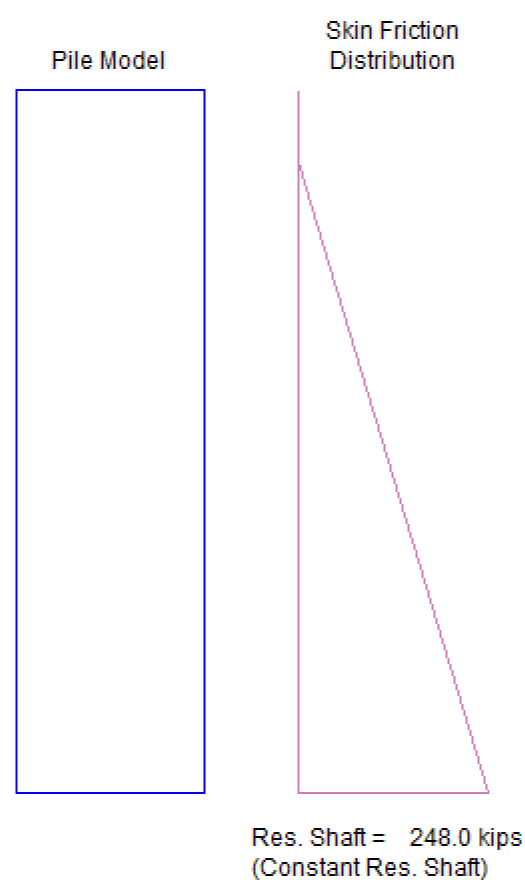
$\boxed{R_{dr} = 770 \cdot kip}$

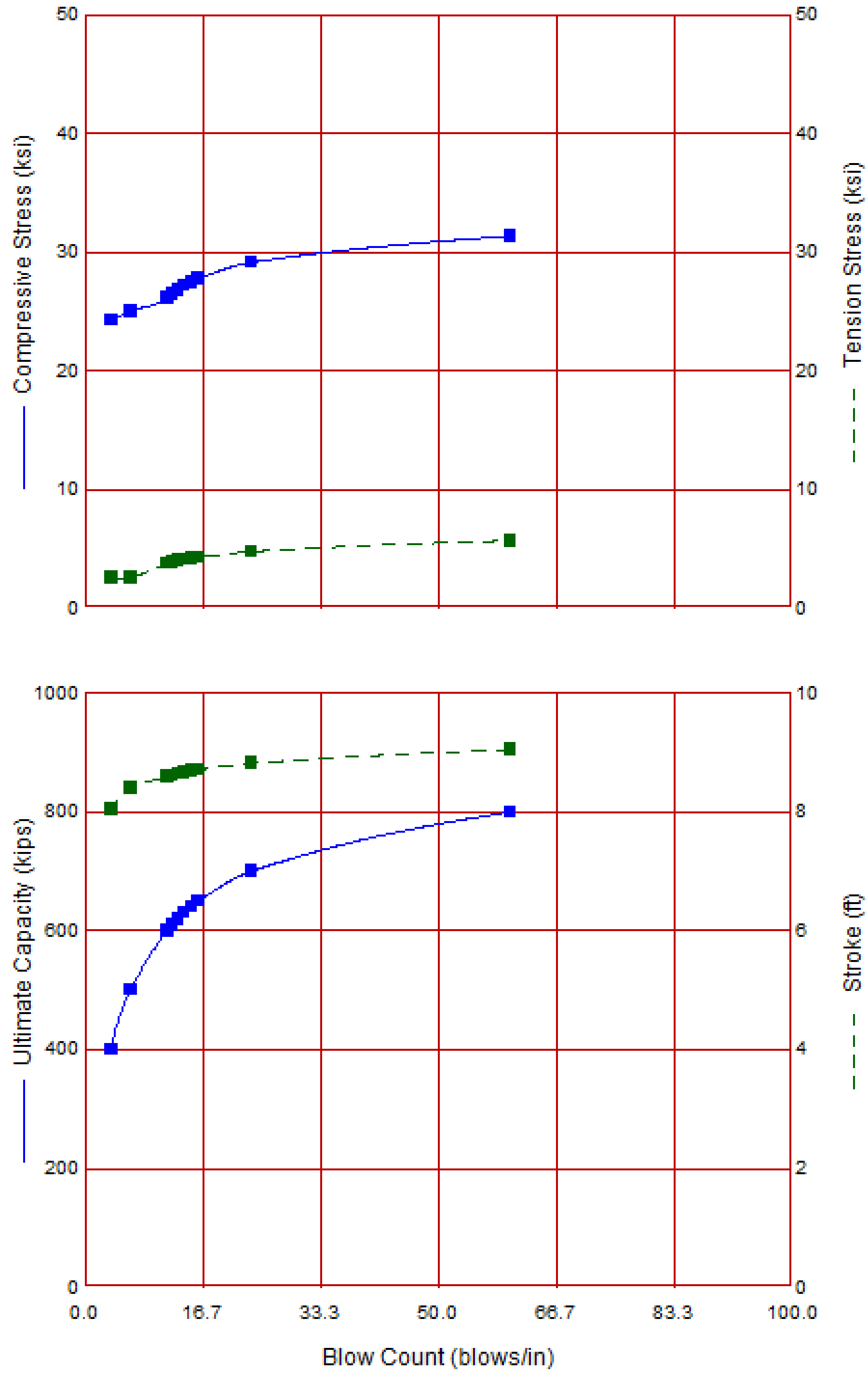
Abutment 2, Pile Size is 14 x 89, APE D19-42 Hammer

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

APE D 19-42

Ram Weight	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	0.070	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	95.00	ft
Pile Penetration	85.40	ft
Pile Top Area	26.10	in2




Compressive Stress (ksi)
Tension Stress (ksi)
50
40
30
20
10
0
Ultimate Capacity (kips)
Stroke (ft)
1000
800
600
400
200
0
10
8
6
4
2
0.0
16.7
33.3
50.0
66.7
83.3
100.0
Blow Count (blows/in)

Maine DOT
25453 Fort Fairfield 14x89 ABT #2 D19-42

19-Apr-2024
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	24.29	2.63	3.8	8.05	21.31
500.0	25.04	2.57	6.5	8.41	22.26
600.0	26.14	3.80	11.7	8.59	22.74
610.0	26.49	3.91	12.4	8.61	22.83
620.0	26.80	4.02	13.2	8.64	22.90
630.0	27.19	4.11	14.0	8.66	22.98
640.0	27.45	4.21	15.1	8.69	23.00
650.0	27.77	4.31	16.1	8.71	23.09
700.0	29.15	4.80	23.6	8.82	23.41
800.0	31.36	5.71	60.1	9.05	24.03

Limit to 15 bpi

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

Strength Limit State

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

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

Extreme and
Service Limit States

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

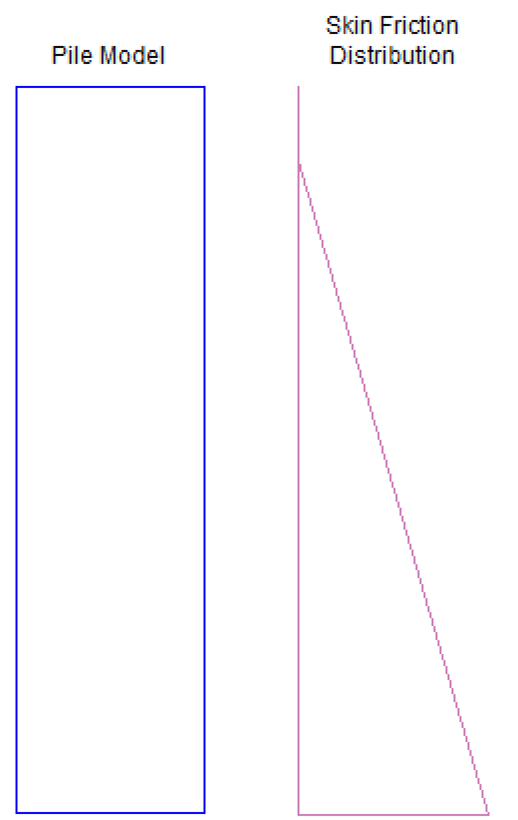
$\boxed{R_{dr} = 630 \cdot kip}$

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

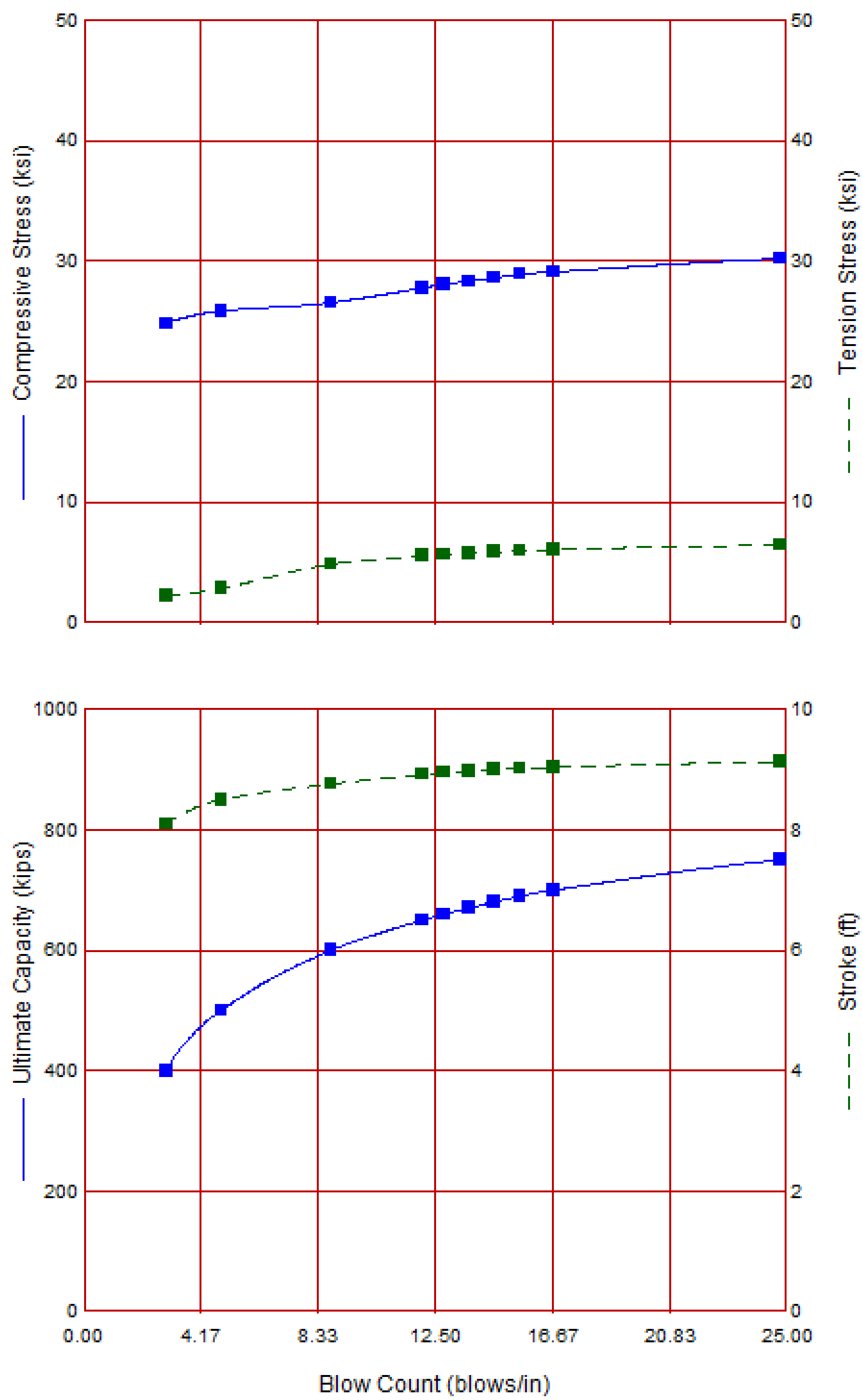
The 14x89 pile can be driven to the resistances below with a APE D25-42 hammer at fuel setting 4 (100% of max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE D 25-42

Ram Weight	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	95.00	ft
Pile Penetration	85.40	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)
0.00
4.17
8.33
12.50
16.67
20.83
25.00

Maine DOT
25453 Fort Fairfield 14x89 ABT #2 D25-42

19-Apr-2024
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	24.83	2.23	2.9	8.09	25.18
500.0	25.87	2.85	4.9	8.50	26.61
600.0	26.56	4.86	8.8	8.76	27.61
650.0	27.78	5.54	12.0	8.92	28.16
660.0	28.09	5.66	12.8	8.95	28.29
670.0	28.34	5.77	13.7	8.97	28.32
680.0	28.61	5.87	14.5	9.00	28.44
690.0	28.92	5.97	15.5	9.02	28.55
700.0	29.11	6.08	16.7	9.04	28.58
750.0	30.22	6.44	24.8	9.13	28.89

Limit to 15 bpi

$$R_{ndr} := 680 \cdot kip$$

Strength Limit State

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

$$\boxed{R_{fdr} = 442 \cdot kip}$$

Extreme and
Service Limit States

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

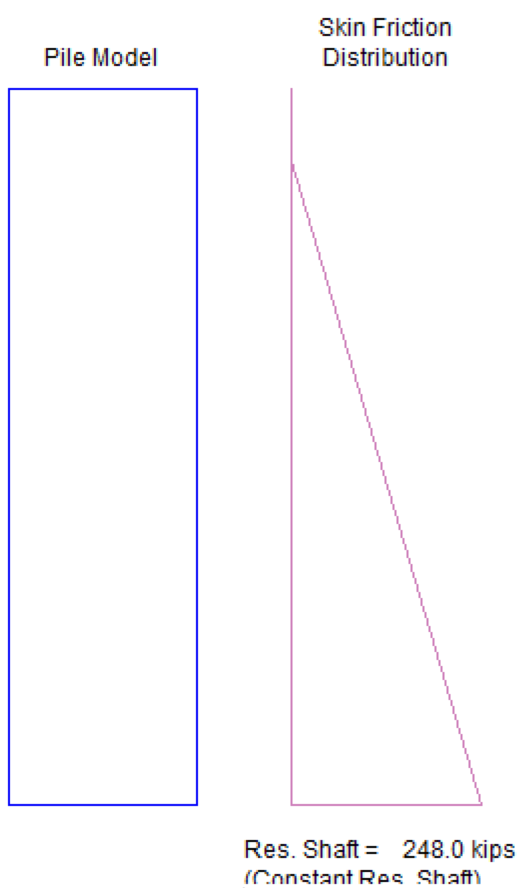
$$\boxed{R_{dr} = 680 \cdot kip}$$

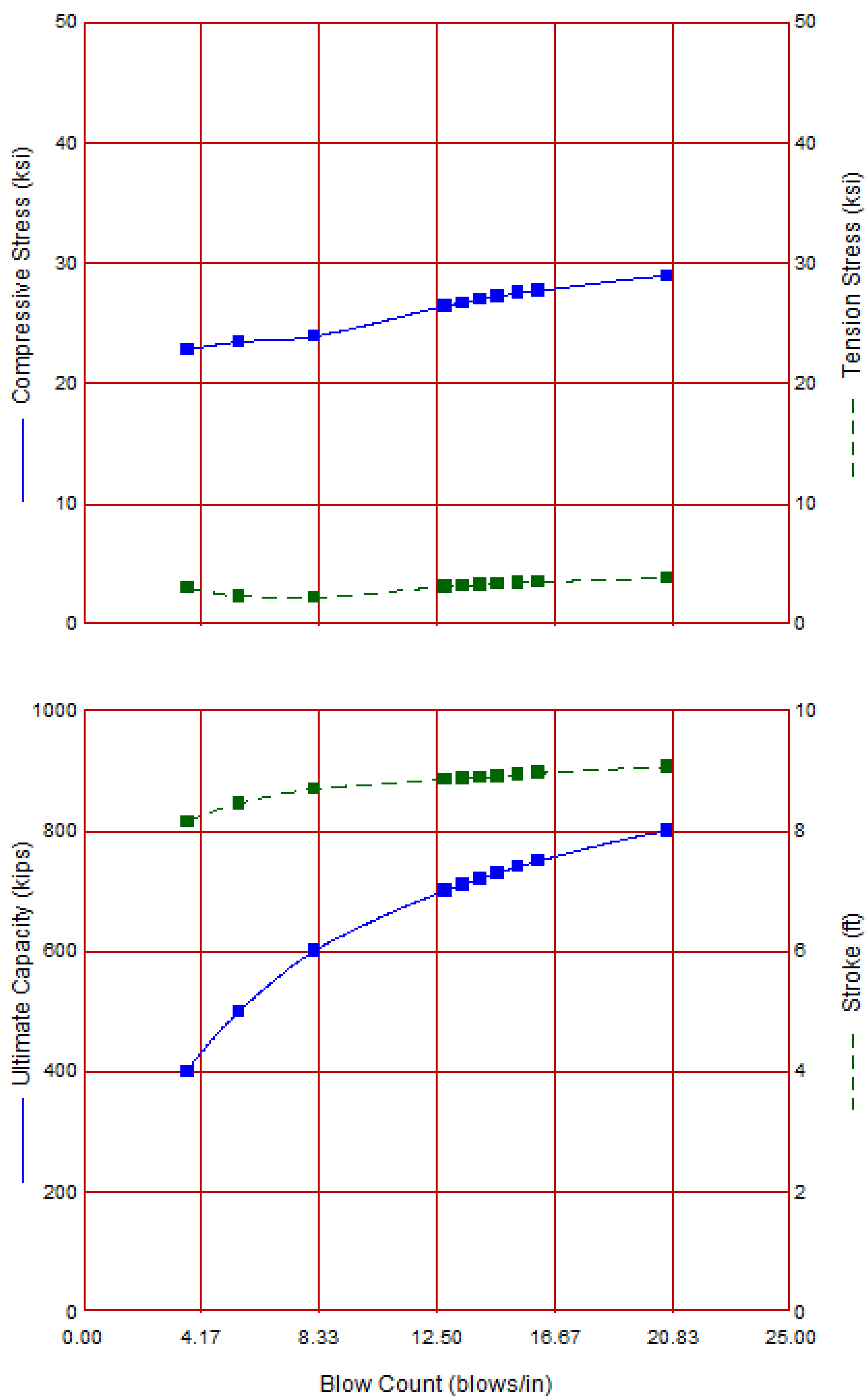
Abutment 2, Pile Size is 14 x 117, APE D19-42 Hammer

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

APE D 19-42

Ram Weight	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	0.070	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	95.00	ft
Pile Penetration	85.40	ft
Pile Top Area	34.40	in2




50
40
30
20
10
0
Compressive Stress (ksi)
Tension Stress (ksi)
1000
800
600
400
200
0
10
8
6
4
2
Ultimate Capacity (kips)
Stroke (ft)
0.00
4.17
8.33
12.50
16.67
20.83
25.00
Blow Count (blows/in)

Maine DOT
25453 Fort Fairfield 14x117 ABT#2 D19-42

19-Apr-2024
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	22.80	3.03	3.7	8.14	20.16
500.0	23.45	2.37	5.5	8.46	21.00
600.0	23.91	2.23	8.2	8.69	21.64
700.0	26.40	3.13	12.8	8.85	22.04
710.0	26.70	3.22	13.4	8.87	22.10
720.0	27.00	3.30	14.0	8.89	22.15
730.0	27.24	3.37	14.7	8.91	22.23
740.0	27.55	3.44	15.4	8.93	22.30
750.0	27.73	3.51	16.1	8.96	22.37
800.0	28.95	3.84	20.6	9.06	22.64

Limit to 15 bpi

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

Strength Limit State

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

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

Extreme and
Service Limit States

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

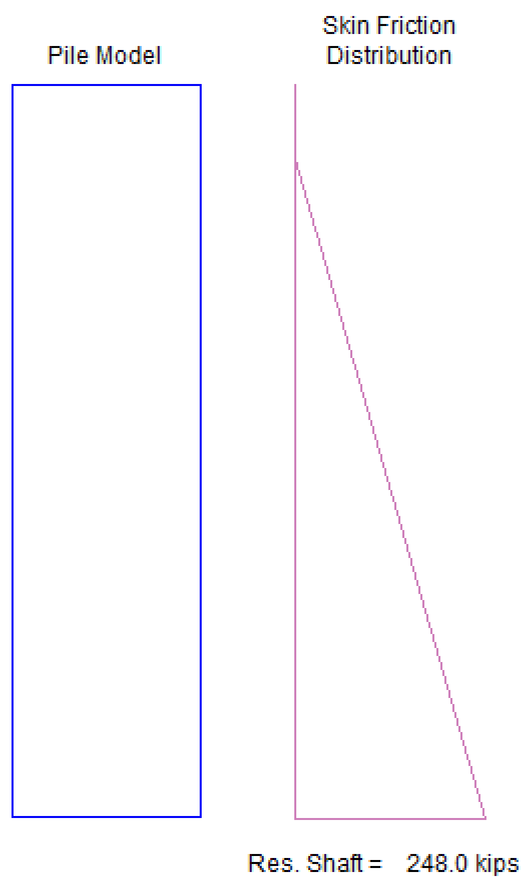
$\boxed{R_{dr} = 730 \cdot kip}$

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

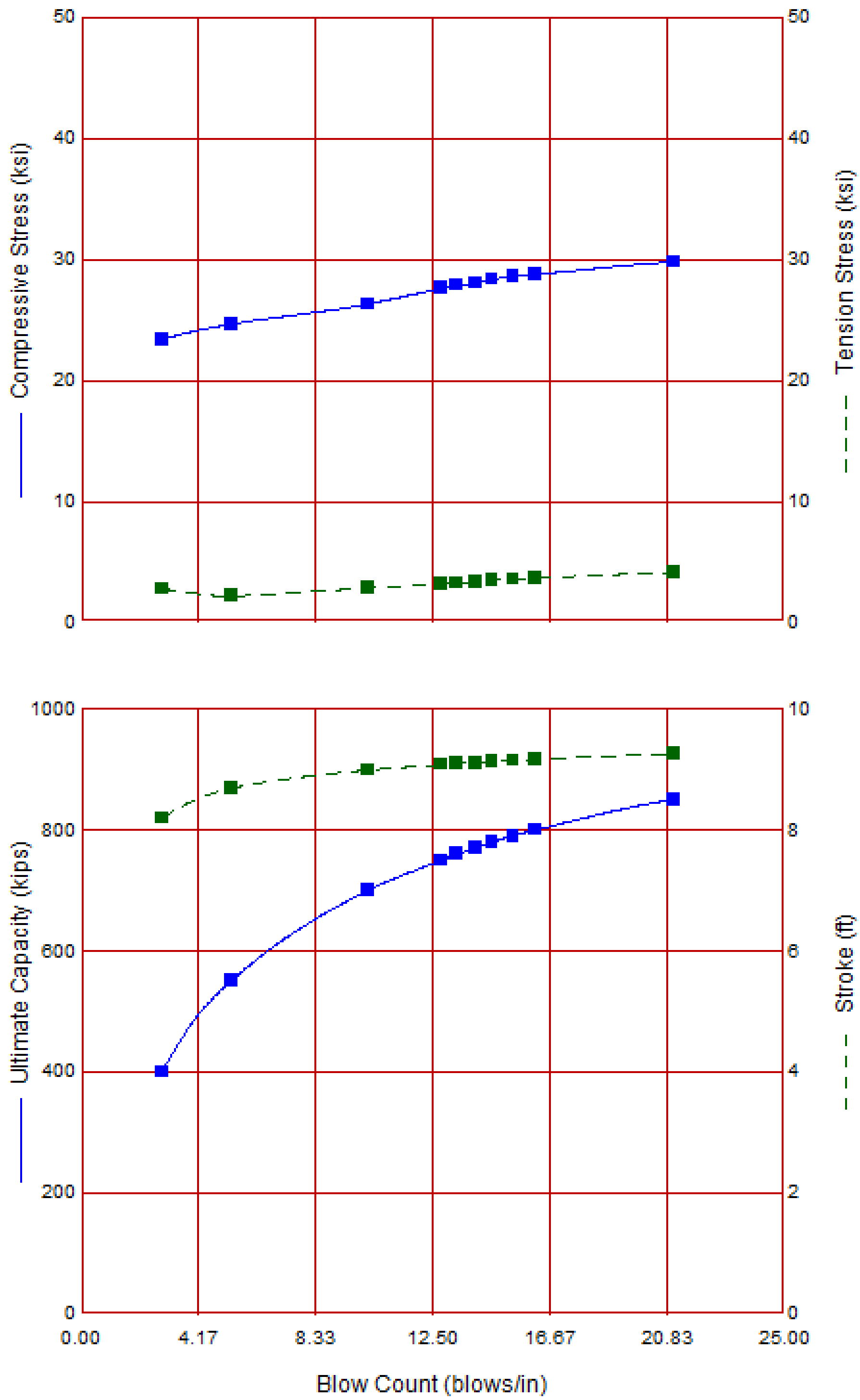
The 14x117 pile can be driven to the resistances below with a APE D25-42 hammer at fuel setting 4 (100% of max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE D 25-42

Ram 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	95.00	ft
Pile Penetration	85.40	ft
Pile Top Area	34.40	in2



(Constant Res. Shaft)



Maine DOT
25453 Fort Fairfield 14x117 ABT#2 D25-42

19-Apr-2024
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	23.40	2.75	2.9	8.20	23.58
550.0	24.64	2.17	5.3	8.69	25.21
700.0	26.32	2.83	10.2	8.99	26.23
750.0	27.60	3.17	12.8	9.08	26.47
760.0	27.86	3.27	13.3	9.10	26.59
770.0	28.04	3.37	14.0	9.11	26.63
780.0	28.35	3.47	14.6	9.14	26.75
790.0	28.59	3.56	15.4	9.15	26.80
800.0	28.76	3.66	16.2	9.17	26.85
850.0	29.80	4.11	21.1	9.26	27.17

Limit to 15 bpi

$R_{ndr} := 780 \cdot kip$

Strength Limit State

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

$\boxed{R_{fdr} = 507 \cdot kip}$

Extreme and
Service Limit States

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

$\boxed{R_{dr} = 780 \cdot kip}$

25453.00 Fort Fairfield - Puddle Dock Bridge #2691
GRL WEAP INPUT + RESULT SUMMARY
NPP 4/12/24

	Abutment	Pile Size	Pile Length	Pile Penetration	Hammer	Fuel Setting	Shaft Quake	Toe Quake	Shaft Damping	Toe Damping	Skin Friction	Ultimate Capacity	Max Comp Stress	Max Tension Stress	Blows/In	Stroke	Energy
Abutment #1 14x89 APE D19-42	1	HP 14x89	100	94.9	APE D19-42	3	0.10	0.04	0.05	0.15	280	590	23.52	3.03	14.5	7.71	19.40
	1	HP 14x89	100	94.9	APE D19-42	3	0.10	0.07	0.05	0.15	280	590	23.59	3.04	14.9	7.75	19.49
	1	HP 14x89	100	94.9	APE D19-42	3	0.10	0.10	0.05	0.15	280	580	23.47	2.82	14.6	7.69	19.37
	1	HP 14x89	100	94.9	APE D19-42	4	0.10	0.04	0.05	0.15	280	640	26.05	3.81	14.8	8.62	22.72
	1	HP 14x89	100	94.9	APE D19-42	4	0.10	0.07	0.05	0.15	280	630	25.33	3.65	14.4	8.56	22.62
	1	HP 14x89	100	94.9	APE D19-42	4	0.10	0.10	0.05	0.15	280	620	25.17	3.53	14.5	8.49	22.38
Abutment #1 14x89 APE D25-42	1	HP 14x89	100	94.9	APE D25-42	3	0.10	0.04	0.05	0.15	280	610	24.29	3.75	14.0	7.89	22.75
	1	HP 14x89	100	94.9	APE D25-42	3	0.10	0.07	0.05	0.15	280	610	24.15	3.90	14.8	7.84	22.60
	1	HP 14x89	100	94.9	APE D25-42	3	0.10	0.10	0.05	0.15	280	600	24.00	3.33	14.5	7.78	22.43
	1	HP 14x89	100	94.9	APE D25-42	4	0.10	0.04	0.05	0.15	280	680	27.45	5.15	14.7	8.89	27.93
	1	HP 14x89	100	94.9	APE D25-42	4	0.10	0.07	0.05	0.15	280	670	26.66	4.98	14.4	8.83	27.74
	1	HP 14x89	100	94.9	APE D25-42	4	0.10	0.10	0.05	0.15	280	660	26.50	4.43	14.5	8.77	27.50
Abutment #1 14x117 APE D19-42	1	HP14x117	100	94.9	APE D19-42	3	0.10	0.04	0.05	0.15	280	680	23.29	2.48	14.4	7.94	18.83
	1	HP14x117	100	94.9	APE D19-42	3	0.10	0.07	0.05	0.15	280	670	22.24	2.54	14.8	7.88	18.65
	1	HP14x117	100	94.9	APE D19-42	3	0.10	0.10	0.05	0.15	280	660	22.29	2.47	14.8	7.90	18.76
	1	HP14x117	100	94.9	APE D19-42	4	0.10	0.04	0.05	0.15	280	740	26.49	2.82	14.6	8.89	22.08
	1	HP14x117	100	94.9	APE D19-42	4	0.10	0.07	0.05	0.15	280	730	25.54	2.87	14.9	8.84	21.94
	1	HP14x117	100	94.9	APE D19-42	4	0.10	0.10	0.05	0.15	280	710	23.98	2.88	14.8	8.75	21.68
Abutment #1 14x117 APE D25-42	1	HP14x117	100	94.9	APE D25-42	3	0.10	0.04	0.05	0.15	280	710	23.64	2.33	14.8	8.06	21.52
	1	HP14x117	100	94.9	APE D25-42	3	0.10	0.07	0.05	0.15	280	700	22.93	2.37	14.9	8.01	21.43
	1	HP14x117	100	94.9	APE D25-42	3	0.10	0.10	0.05	0.15	280	680	22.74	2.67	14.5	7.95	21.19
	1	HP14x117	100	94.9	APE D25-42	4	0.10	0.04	0.05	0.15	280	790	27.81	3.31	14.9	9.10	26.46
	1	HP14x117	100	94.9	APE D25-42	4	0.10	0.07	0.05	0.15	280	770	26.33	3.14	14.5	9.02	26.22
	1	HP14x117	100	94.9	APE D25-42	4	0.10	0.10	0.05	0.15	280	750	25.21	3.14	14.3	8.95	25.99
Abutment #2 14x89 APE D19-42	2	HP 14x89	95	85.4	APE D19-42	3	0.10	0.04	0.05	0.15	248	600	25.14	3.34	14.9	7.83	19.85
	2	HP 14x89	95	85.4	APE D19-42	3	0.10	0.07	0.05	0.15	248	590	24.00	3.49	14.8	7.76	19.66
	2	HP 14x89	95	85.4	APE D19-42	3	0.10	0.10	0.05	0.15	248	580	23.55	3.74	14.9	7.70	19.52
	2	HP 14x89	95	85.4	APE D19-42	4	0.10	0.04	0.05	0.15	248	650	28.48	4.26	14.9	8.76	23.25
	2	HP 14x89	95	85.4	APE D19-42	4	0.10	0.07	0.05	0.15	248	630	27.19	4.11	14.0	8.66	22.98
	2	HP 14x89	95	85.4	APE D19-42	4	0.10	0.10	0.05	0.15	248	620	25.61	4.24	14.2	8.60	22.82
Abutment #2 14x89 APE D25-42	2	HP 14x89	95	85.4	APE D25-42	3	0.10	0.04	0.05	0.15	248	630	25.86	5.02	15.0	8.06	23.46
	2	HP 14x89	95	85.4	APE D25-42	3	0.10	0.07	0.05	0.15	248	620	24.61	4.82	14.9	8.00	23.28
	2	HP 14x89	95	85.4	APE D25-42	3	0.10	0.10	0.05	0.15	248	600	24.33	4.37	14.1	7.90	22.91
	2	HP 14x89	95	85.4	APE D25-42	4	0.10	0.04	0.05	0.15	248	690	29.93	6.24	14.5	9.07	28.65
	2	HP 14x89	95	85.4	APE D25-42	4	0.10	0.07	0.05	0.15	248	680	28.61	5.87	14.5	9.00	28.44
	2	HP 14x89	95	85.4	APE D25-42	4	0.10	0.10	0.05	0.15	248	670	26.94	5.68	14.8	8.92	28.21
Abutment #2 14x117 APE D19-42	2	HP14x117	95	85.4	APE D19-42	3	0.10	0.04	0.05	0.15	248	690	25.11	2.76	14.8	8.03	19.09
	2	HP14x117	95	85.4	APE D19-42	3	0.10	0.07	0.05	0.15	248	670	23.86	2.80	14.6	7.94	18.91
	2	HP14x117	95	85.4	APE D19-42	3	0.10	0.10	0.05	0.15	248	650	22.41	2.80	14.2	7.95	18.91
	2	HP14x117	95	85.4	APE D19-42	4	0.10	0.04	0.05	0.15	248	750	28.36	3.45	14.8	9.00	22.42
	2	HP14x117	95	85.4	APE D19-42	4	0.10	0.07	0.05	0.15	248	730	27.24	3.37	14.7	8.91	22.23
	2	HP14x117	95	85.4	APE D19-42	4	0.10	0.10	0.05	0.15	248	710	25.32	3.33	14.8	8.82	21.98
Abutment #2 14x117 APE D25-42	2	HP14x117	95	85.4	APE D25-42	3	0.10	0.04	0.05	0.15	248	720	25.51	2.94	14.7	8.17	21.99
	2	HP14x117	95	85.4	APE D25-42	3	0.10	0.07	0.05	0.15	248	700	23.98	2.77	14.4	8.09	21.76
	2	HP14x117	95	85.4	APE D25-42	3	0.10	0.10	0.05	0.15	248	690	23.03	2.96	15.0	8.04	21.63
	2	HP14x117	95	85.4	APE D25-42	4	0.10	0.04	0.05	0.15	248	800	29.71	3.87	14.7	9.21	26.98
	2	HP14x117	95	85.4	APE D25-42	4	0.10	0.07	0.05	0.15	248	780	28.35	3.47	14.6	9.14	26.75
	2	HP14x117	95	85.4	APE D25-42	4	0.10	0.10	0.05	0.15	248	760	25.99	3.39	14.8	9.06	26.45

Hammer Information:

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

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

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

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.

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

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

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

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

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

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

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

Abutment Backfill Angles

α = Angle of fill slope to the horizontal

Angles computed based on roadway elevation change 25 feet behind the centerline of the abutments

$\text{Rise}_{ABT1} := 1.5\text{ft}$ $\text{Rise}_{ABT2} := -0.6\text{ft}$ $\text{Run} := 25\text{ft}$

$$\alpha_{ABT1} := \text{atan}\left(\frac{\text{Rise}_{ABT1}}{\text{Run}}\right) = 3.43\ \text{deg}$$ Abutment No. 1

$$\alpha_{ABT2} := \text{atan}\left(\frac{\text{Rise}_{ABT2}}{\text{Run}}\right) = -1.37\ \text{deg}$$ Abutment No. 2

Integral Abutment - Passive Earth Pressure - Coulomb Theory (Abutment No. 1)

α_{ABT1} = Angle of fill slope to the horizontal at Abutment No. 1 $\alpha_{ABT1} = 3.43\ \text{deg}$

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

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

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

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

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

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

$$K_{p\_coulomb} := \frac{\sin(\beta - \phi')^2}{\sin(\beta)^2 \cdot \sin(\beta + \delta') \cdot \left(1 - \sqrt{\frac{\sin(\phi' + \delta') \cdot \sin(\phi' + \alpha_{ABT1})}{\sin(\beta + \delta') \cdot \sin(\beta + \alpha_{ABT1})}}\right)^2}$$

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

$K_{p\_coulomb} = 7.21$

Recommend K=7.21 at Abutment No. 1

Integral Abutment and Wingwall - Passive Earth Pressure - Rankine Theory

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

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

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

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

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

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

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

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

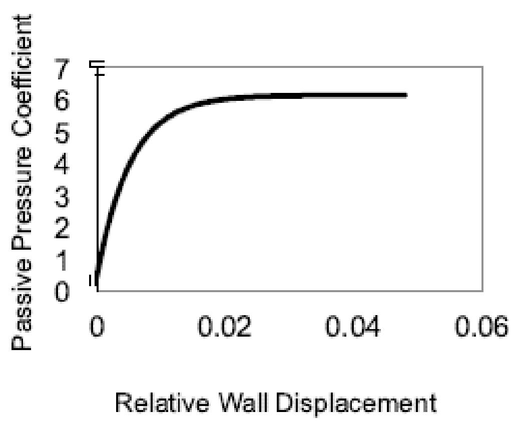


Figure 3.10.8-1: Plot of Passive Pressure Coefficient, K, vs. Relative Wall Displacement, δ_T/H.

Thermal displacement at each abutment: $\delta := 0.22in$

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

Relative wall displacement: $x := \frac{\delta}{h}$ $x = 0.0017$

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

$\boxed{K = 1.98}$ **< K_{p_rank} of 3.25, therefore recommend K=3.25 at Abutment No. 2**

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	$\boxed{\text{17 to 22}}$	0.31 to 0.40
• Silty sand, gravel or sand mixed with silt or clay	17	0.31
• Fine sandy silt, nonplastic silt	14	0.25
Various structural materials:		
• Masonry on masonry, igneous and metamorphic rocks:		
o dressed soft rock on dressed soft rock	35	0.70
o dressed hard rock on dressed soft rock	33	0.65
o dressed hard rock on dressed hard rock	29	0.55
• Masonry on wood in direction of cross grain	26	0.49
• Steel on steel at sheet pile interlocks	17	0.31

3.11.5.4—Passive Lateral Earth Pressure Coefficient, k_p

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

For cohesive soils, passive pressures may be estimated by:

C3.11.5.4

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

Table 7.9 (*Continued*)

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

7.12 Coulomb's Passive Earth Pressure

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

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

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

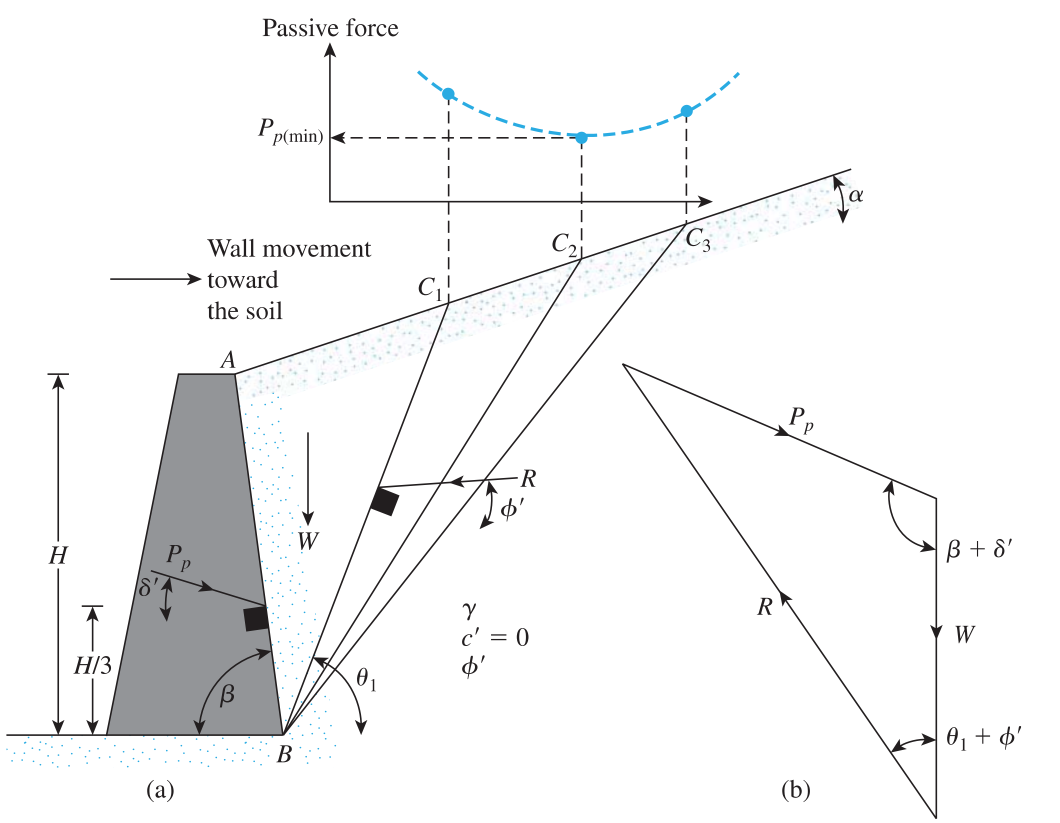


Figure 7.25 Coulomb's passive pressure

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

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

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

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

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

where

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

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

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

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

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

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

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

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

Again, at $z = 3$ m,

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

Hence,

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

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

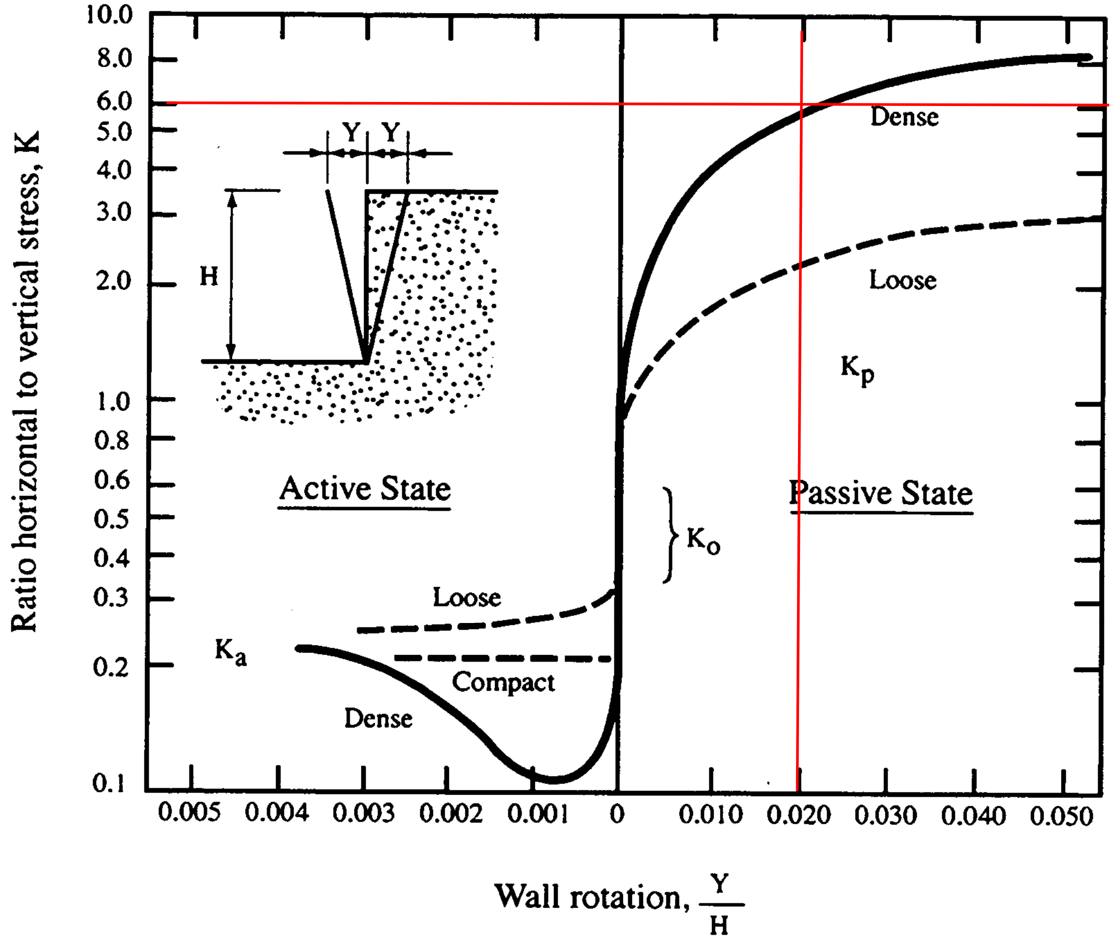
$$\sigma'_p = \gamma z K_p \tag{7.65}$$

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).

allowed on the bridge before pouring the abutment diaphragm. In such cases, the Load Factors for Construction Loads shall be taken as per Article 3.4.2 of the *AASHTO LRFD Bridge Design Specifications*.

3.10.7 Superstructure Design Methodology

The connection between the beams and the abutment shall be assumed to be simply supported for superstructure design and analysis. It is recognized that, in some cases, it may be desirable to take advantage of the frame action in the superstructure design by assuming some degree of fixity. This, however, requires careful engineering judgment. Due to the uncertainty in the degree of fixity, frame action shall not be used to reduce design moments in the beams.

3.10.8 Pile Cap and Abutment Diaphragm Design

The superstructure is assumed to transfer moment, and vertical and horizontal forces due to all applicable loads, at the time when the rigid connection with the abutment is achieved. The effects of skew, curvature, thermal expansion of the superstructure, reveal, and grade are considered.

The design provisions below are conservative because the pile cap and the abutment diaphragm are very rigid members, therefore all loads shall be uniformly distributed across the abutment.

For the integral abutments constructed in two stages as specified above, the abutment shall be designed for the following two cases:

1. The pile cap is designed to resist all vertical loads including live load. It is assumed to act as a continuous beam supported by piles. The analysis can be simplified by assuming the pile cap acting as a simple span between piles and then taking 80% of simple span moments to account for continuity. Shears may be taken equal to simple span shears.

2. The entire abutment wall (the combined height of the pile cap and the abutment diaphragm) is designed to resist the earth pressure due to the backfill material, assuming the wall to act as a horizontal continuous beam supported on the girders, i.e., with spans equal to the girder spacing along the skew (if any).

The abutments should be kept as short as possible to reduce the magnitude of soil pressure developed. A minimum of 3'-0" for inspection access shall be provided. A minimum fill cover over the bottom of the abutment of 3'-0" is desirable. It is recommended to have abutments of equal height due to the fact that a difference in abutment heights causes more movements to take place at the shorter abutment. Abutments of unequal height shall be designed by balancing the earth pressure consistent with the magnitude of the displacement at each abutment.

The magnitude of lateral earth pressure developed by the backfill is dependent on the relative wall displacement, δ_T/H, and may be considered to develop between full passive and at-rest earth pressure. The backfill force shall be determined based on the movement-dependent coefficient of earth pressure (K). Results from full scale wall tests performed by UMASS[1] show reasonable agreement between the predicted average passive earth pressure response of MassDOT's standard compacted gravel borrow and the curves of K versus δ_T/H for dense sand found in design manuals DM-7[2] and NCHRP[3]. For the design of integral abutments, the coefficient of horizontal earth pressure when using compacted gravel borrow backfill shall be estimated using the equation:

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

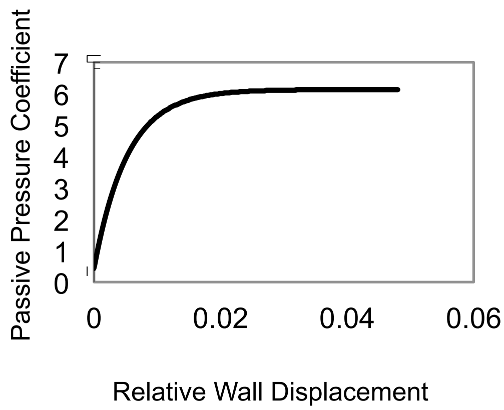


Figure 3.10.8-1: Plot of Passive Pressure Coefficient, K, vs. Relative Wall Displacement, δ_T/H.

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

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

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

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

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

Frost Depth

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

From Design Freezing Index Map: Fort Fairfield, Maine
DFI = 2600 degree-days.
Fine-Grained Fill w=15% (BB-FFPB-102, 2D)
Coarse-Grained Fill w=20% (BB-FFPB-101 2D; BB-FFPB-103 1D; BB-FFPB-103 2D)

Fine-Grained Fill

For DFI = 2600, Fine-Grained Soil, w=10%

d=Depth of Frost Penetration

$d_1 := 77.5\text{in} \qquad w_1 := 10\%$

For DFI = 2600, Fine-Grained Soil, w=20%

$d_2 := 66.5\text{in} \qquad w_2 := 20\%$

Interpolate for DFI = 2600, Fine-Grained Soil, w=15%

$w_3 := 15\%$

$$d_{fine} := d_1 + (w_3 - w_1)\cdot\frac{(d_2 - d_1)}{(w_2 - w_1)}$$

$\boxed{d_{fine} = 72\cdot in} \qquad \boxed{d_{fine} = 6\cdot ft}$ **for Fine-Grained Fill**

Coarse-Grained Fill

For DFI = 2600, Coarse-Grained Soil, w=20%

$\boxed{d_{coarse} := 89.9\text{in}} \qquad \boxed{d_{coarse} = 7.5\cdot ft}$ **for Coarse-Grained Fill**

Recommend any foundation bearing on soils be embedded **7.5 feet** for frost protection.

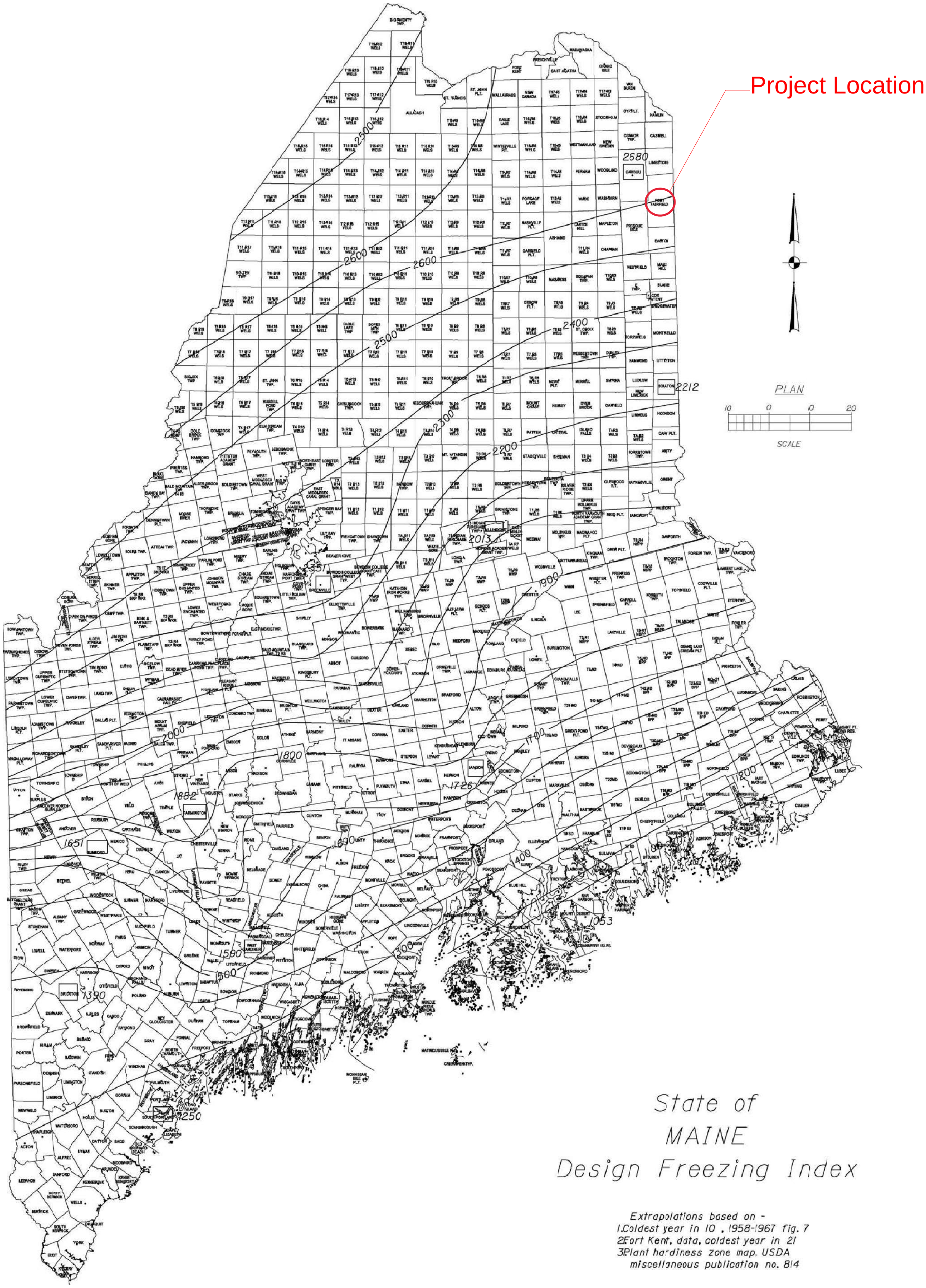


Figure 5-1 Maine Design Freezing Index Map

5.2 General

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-FFPB-102			
Depth	**N_{60}**	**di**	**di/N**
5	21	10	0.48
10	8	5.5	0.69
15	24	4.5	0.19
20	8	5	0.63
25	24	5	0.21
30	23	3	0.13
34	62	7	0.11
40	33	5	0.15
45	61	5	0.08
50	100	5	0.05
55	100	7	0.07
65	100	13	0.13
75	100	10	0.10
85	100	10	0.10
95	38	5	0.13
SUM		**100**	**3.24**
		di/di/N	**30.84**

BB-LBS-103/103A			
Depth	**N_{60}**	**di**	**di/N**
5	11	10	0.91
10	6	3.5	0.58
15	5	6.5	1.30
20	17	4	0.24
25	41	6	0.15
30	67	3	0.04
34	55	6	0.11
39	26	6	0.23
45	59	5	0.08
50	62	5	0.08
55	74	5	0.07
65	99	10	0.10
75	53	15	0.28
90	61	10	0.16
95	100	5	0.05
SUM		**100**	**4.39**
		di/di/N	**22.78**

SUM	**Nav.**	**26.81**

15 < Nav. < 50 bpf

Conclusion: Site Class D

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

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Abutment No. 1 and 2 Seismic Parameters

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 46.765500
Longitude = -067.816639
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.080	PGA - Site Class B
0.2	0.179	Ss - Site Class B
1.0	0.052	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
Latitude = 46.765500
Longitude = -067.816639
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	0.128	As - Site Class D
0.2	0.287	SDs - Site Class D
1.0	0.125	SD1 - Site Class D

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