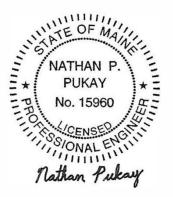
MAINE DEPARTMENT OF TRANSPORTATION BRIDGE PROGRAM GEOTECHNICAL SECTION AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

PUDDLE DOCK BRIDGE STATE ROUTE 161 OVER PATTEE BROOK FORT FAIRFIELD, MAINE



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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 (N₆₀) are shown on the boring logs.

Bedrock was cored in the borings using NQ-2" core barrels and the Rock Quality Designation (RQD) of the cores calculated. The MaineDOT geotechnical engineer selected the boring locations and drilling methods, designated type and depth of sampling techniques, 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 Hpile 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

<u>Structural Resistance.</u> Preliminary estimates of the factored structural axial resistance of two Hpile 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).

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

<u>Drivability Analyses</u>. Drivability analyses were performed to determine the pile resistance that might be achieved considering available diesel hammers. LRFD 10.7.8 limits driving stresses to 0.90 fy, 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)	Resis ⁻ φ _{dyn} =	ubility tance ³ = 0.65 ps)	Governing Axial Pile Resistance ⁶ (kips)			
HP 14 x 89	652	652	409 ⁴	4365	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	Resist \$\$	ıbility tance ³ 1.0 ps)	Governing Axial Pile Resistance ⁶ (kips)						
HP 14 x 89	1,305	1,305	630 ⁴	670 ⁵	630 ⁴				
HP 14 x 117									

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 LayerTop Elevation ModelLayer Thickness 								
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_0 , 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 (As)	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.

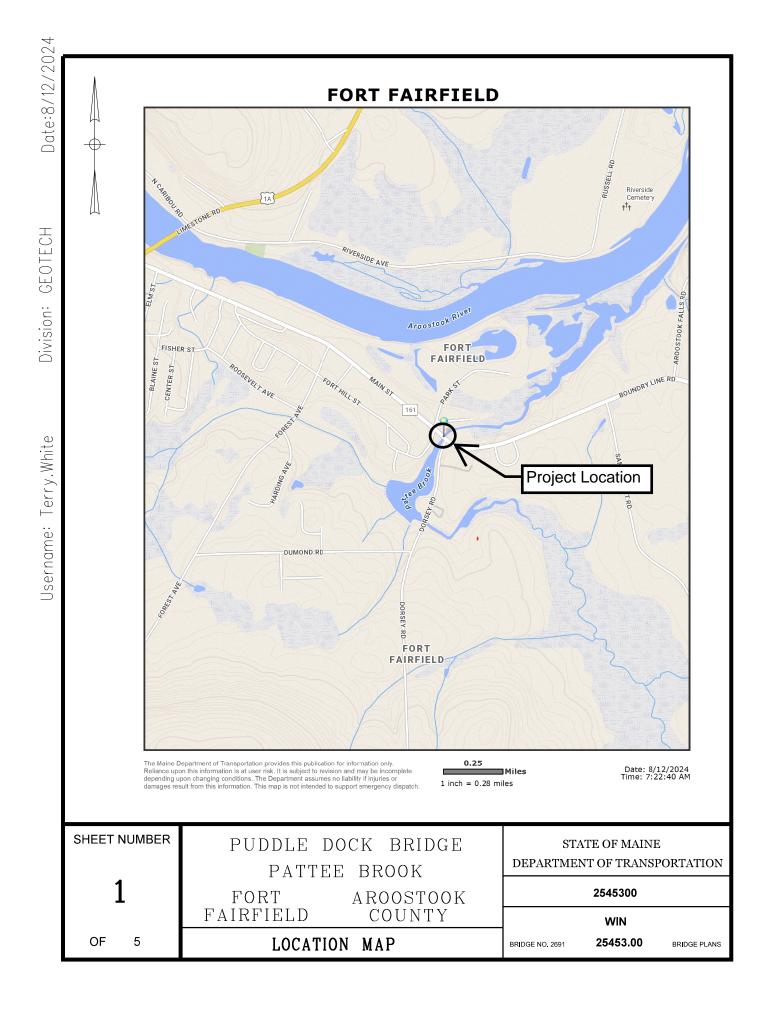
9.0 CLOSURE

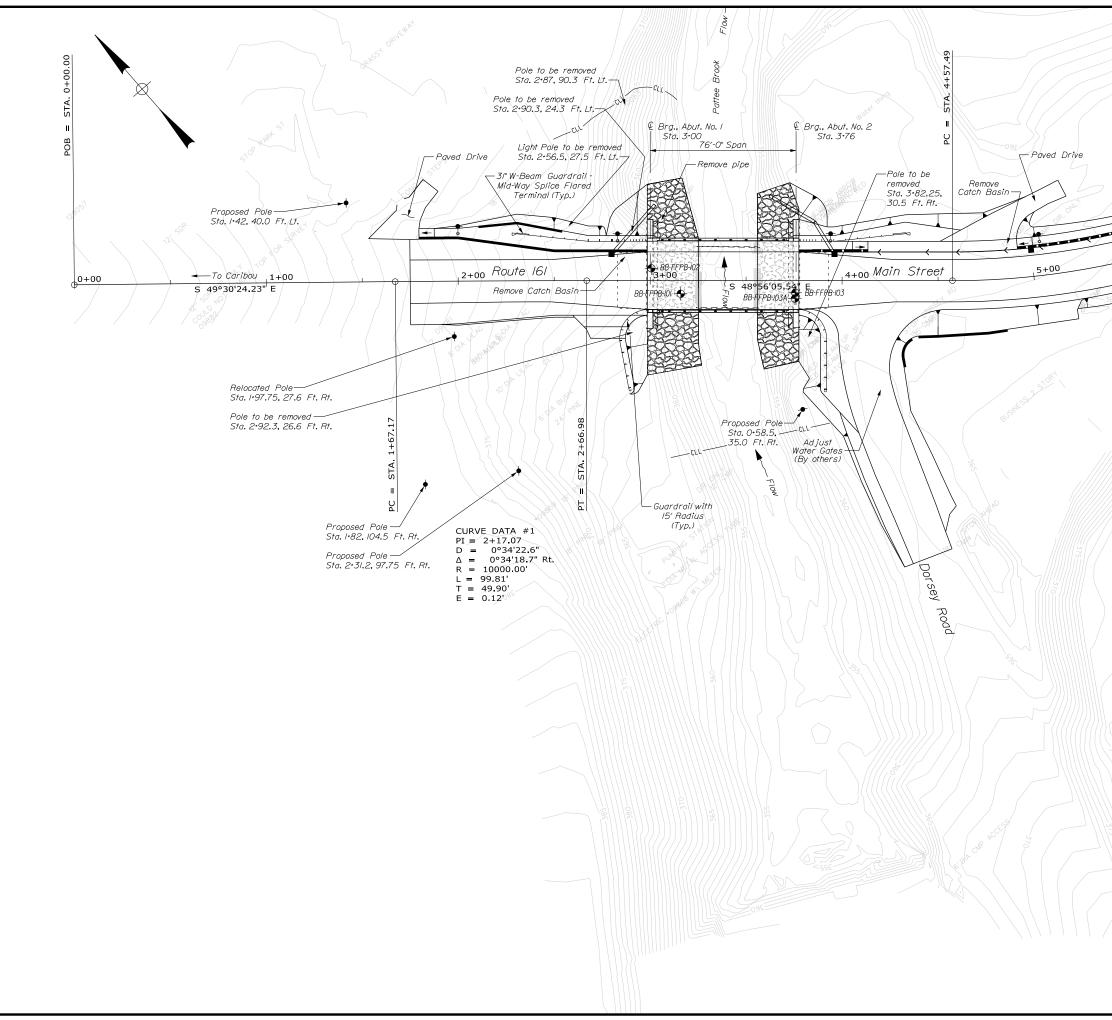
This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Puddle Dock Bridge in Fort Fairfield, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed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

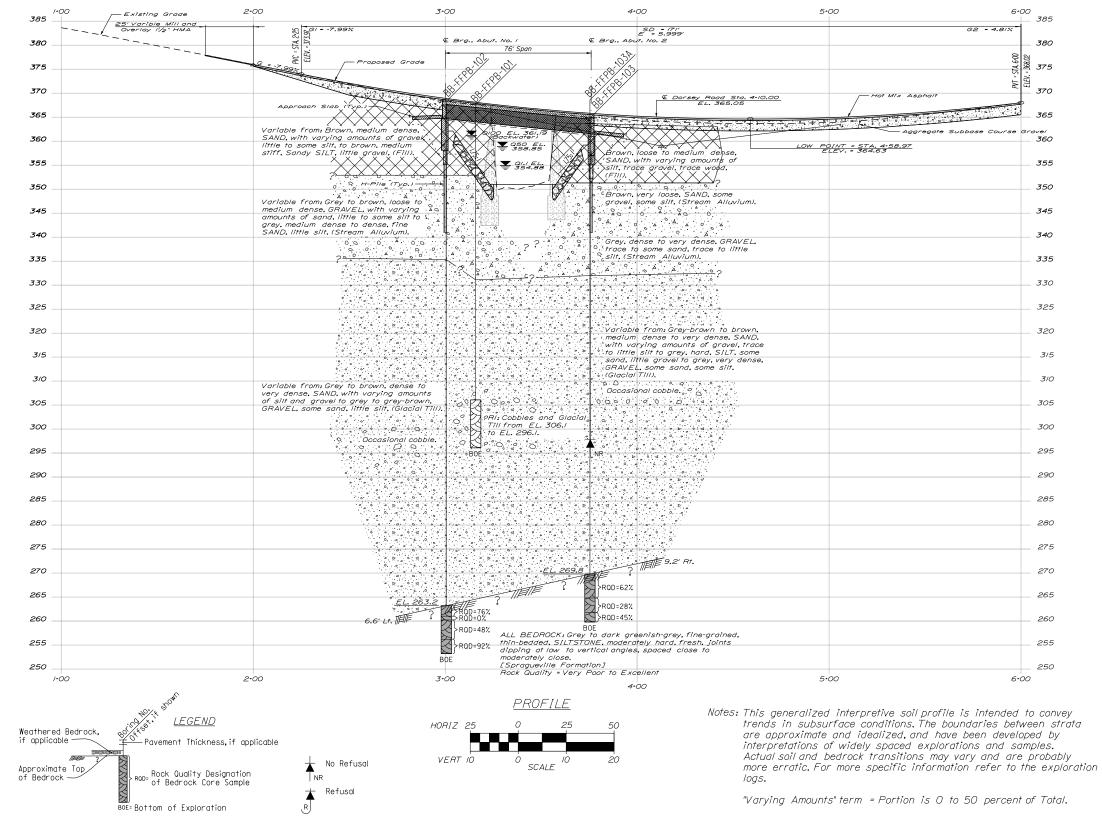
It is recommended that a geotechnical engineer be provided the opportunity for a review of the final design and specifications in order that the earthwork and foundation recommendations and construction considerations presented in this report are properly interpreted and implemented in the design and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

<u>Sheets</u>





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- 80													-											S	DEPARTME				
- 85	14	ID	18/1	6	85.00 - 86.50		22/2	/63		18	133		-	والمراجع المتحالية المحالية ال	Occastonal Cobbie Gray, wet, vary c Glactal Ttill,		AND, some	igravel+ I	ittie stit.										
- 90 - 95	15	iD	24/1	4	95.00 - 97.00		9/9/1	5/23		:5	38				Crey, wat. dense. Tfill.	, Sīlty S	SAND, 1711	tle gravel	• (Glacia)						A THE TO E	DIGNALUKE	P.E. NUMBER		
-100														5 - 17 - 15 - 15 - 5 - 5 - 5 - 5 - 5 - 5 - 5 -										BY DATE		WHITE 0CT 2024	P.E.	-	
- 105	R	2	26.4/ 9.6/ 48/4	,	105.10 - 107.30 - 108.10 - 108.10 - 112.10		ROD = ROD =	ox				N0-2	- 263. 	2 11/11/11/11/11/11/11/11/11/11/11/11/11/	Cobbie of 104.8 Roller Creat offect Top of bedrock of Ribedrock: Crep thin-bedded. Sill dipping joints. calcite infilling [Sprogues/lie for Rock Quality = Co Rib Core Times in 105.1-106.1 ft (2 106.1-101.3 ft (1 92% Recovery) R2: Bedrock: Simil frocture throughout	t Elev. 2 y to dark TSTONE, m closely g. rmation] and. minisec) 2:01) 2:04) 1:04) Cor	263.2 ft. k greenist moderately spaced. m	n-grey, fî y hard, fr wîth some	105.1 ne-grained, esh, steepi: quartz or ritcai fresh with					MANAGER		2 N. PUKAY T.	ETAILED3	2	5
-115	R	•	36/3	5	112.10 -		ROD =	92%				V	253.	2	R2: Bedrock: Simi frobury mouther (Scropeville for Rock Quality = W R2: Core ITmes (n 107, Recovery R3: Bedrock: Core Hnin-beddes, SLI vertical joints cuicite Infiling a (Berock: Core R3: Core ITmes (n 108, -100, 1 + 12) (Scropeville for R3: Core ITmes (n 108, -100, 1 + 12) (109, -1-10, 1 + 12) (109, -1-10, 1 + 12) (109, -1-10, 1 + 12) (101, -111, 1 + 12)	y ta dark ISTONE, m closely g, fractu core bec	k greenist moderately y spaced ure planes comes more							PROJ. MAN	DESIGN-DETAILED CHECKED-REVIEWE		-	RE VISIONS	RF VISIONS
- 120													-		110.1-111.1 ff (2) 111.1-12.1 ff (3) 100% Recovery Min-bedded, Sill- dioping at moderc [Sprogueville for Rock Quality = E R41 Core Times (1) 112.1-13.1 ff (1) 113.1-114.1 ff (2) 97% Recovery Bottom of Expl	y to dork TSTONE, m ate angle rmation] Excellent minisec) 1159) 2117) 2120)	k greenist moderately es spaced		115 1					BRIDGE				(0
-130																								DOCK BR	EE BR(1	AIVUL		
-135																								PUDDI, F		1 -		(((חעח
-145																										горт гл			
			-							1														s	HE	ΕT	NU	ME	31
1)			mer R		ent opprov	filore t	oundor l	a batwa	in set	1,08	u from	alt lone	noy be	grodus			Page 2	? of 2		1						Z	4		
					been mode 17mb mboe				diile	16 6 7a	1 0.0	eundeol	ner flui	tuat fe	is may eccur due to cendit	lons other	Borin	ng No.:	BB-FFPB	-102						OF	5		

Orfii			S.₩. Cole Kevîn/Brîan			ivat fon tunt	(ft.)	364 NAV			Auger 10/00: Sompler:	5" Soild Ste Standard Spi	
Logge	od By:		N fider/Puka 8/17-22/202	у	RIG	туре:	Methods	Die	dr ích	D-50 h Boring	Hammer Wt./Fall Core Barrel:		3000
8or ir	ng Loca	tion:	3+75.6. 6.2	ft Rt.	Cas	sing 10	/00:	HNK	4.0*/4	.5")	Water Level®:	13.0 ft bgs.	
Hamme Defini 0 = \$~	ar Effi Tiona:	oîency Fo n Sonnoi≁	ictor: 0.91	R = Reck SSL = 4-	Har Care Sar	mer Ty ole Auger	De1	Autom Su =	Peck/Re	Hydroulic wolded Field Vone Undroined to to Vone Undroined Shear Stren	Rope & Cathead E hear Strength (per) 7,	- Pocket Torvone Sheer BC = Boter Centent,	- Strength room
NO - U U - Tri NU - 1	neuccese In Boll	n Sample ful Spift S fube Sample ful Thin Ma	peen Somple At	R = Rock SSA = Sei RC = Roi RC = Roi Attempt 100H = 104 et Penetromater 100P/C Attempt 100H = 10	llow Ster ler Cone tent of 1	4016. N	-	ap . N-un Hone	Uncenti Correcte Mr Effic	Hydrouric Li melded Field vone Undrofned 1 to Vone Undrofned Shior Streng Indi Colpressive Strength (sau ed = Rev Field SPT N-volut Centry Foctor = Rig Specific) -uncorrected Corrected for Hom r Efficiency Foctor/601(H-u	ra Innual Califoration Val	LL = Lfould Linit PL = Ploatic Linit us Pl = Ploaticity in	dex
v - 17 W - U	eld Vole	Sheer Test ful Fleid V	II Tube Sompte PP = Pock ome Shear Test	et Penetromater MOR/C Attends MOIP = No Sample Information	= Nelight alight of	of Rods One Peru	er Castr en	4 NEO NEO	= SPT N-	uncorrected Corrected for Ho r Eff7c7ency Foster/601.00-u	mar Efficiency corrected	G = Grein Size Aneiyei C = Censeifdotion Teat	
()	No.		Depth	é a	rected			s	8	Vienai Pr	scription and Pro	orka	Laborat Testir Result AASHT
Depth (f)	Somple N	en. /Rec.	omole De ft.)	Shear (/6 Shear (/6 (pst) or R00 ()	+-uncorre		Casing Blows	[evation	aphic L	Visual De	scription and Rem	- ^o	AASHT and hiffed (
8 0	š	2	ŝ	9225P	ż	N60	SSA SSA	<u>5</u>	5	14" HMA.			
								363.7				1.2	
5	10	24/19	5.00 - 7.00	4/3/4/4	7	11				Brown, dry, medium de gravel, (fill),	nse, SAND, some a	illt. trace	G#3375 A-2-4. NC=14
10	20	24/19	10.00 - 12.00	4/2/2/2	4	6				Brown, moist, loose, wood, (Fill).	STITY SAND. Troom	gravel, trace	G#3375 A-4. NC=37.
							$\sqrt{1}$	351.4					
15							IV						
.,	30	24/6	15.00 - 17.00	2/2/1/2	3	5	HP			Brown, wet, very loos (Stream Alluvium).	e: SAND: some gro	ivel, some sîlt.	
		<u> </u>			-		11	1					
			-			-	55	1					
20							43	1					
	MD	24/0	20.00 - 22.00	14/9/2/2	11	17	45 34						
					-		34 35						
ľ							62	340.9					
25			25.00 -				50			Grey, wet, dense, GRA	VEL, some sand,		
	4D	24/13	25.00 - 27.00	15/16/11/10	27	41	48 62			Grey, wet, dense, GRA (Stream Alluvium).			
							67		÷.				
							34		1999.				
30	5D	24/4	30.00 -	10/18/36/20	44	67	47		11000		. GRAVEL. trace :	and, trace silt,	
	50	24/4	30.00 - 32.00	10/18/36/20		67	77			(Stream Alluvium).			
							47	331.9					
			34.00 -				69			Grey-brown, wet, very sfit, (Glacial Till),	dense. Gravelly		G#3375
35	60	24/19	36.00	8/15/21/18	36	55	30 91		1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	sîit, (Giacîai Tîii).			A-1-b. WC=11.
							98						
							67						
	70	24/14	39.00 - 41.00	8/8/9/12	17	26	65 26		ť.	Brown, wet, medium de silt, (Glacial IIII).	nse, SAND, trace	gravel, trace	
40			41100				42						
							68						
							84						
45							73		1				
•	8D	24/15	45.00 - 47.00	16/17/22/17	39	59	74			Brown, wet, very dens stit, (Glacial Itil).	e⊷ SAND⊷ some gr	ovel. Ifttle	
						-	99 90	-					
						-	90	1					
50							122	1		1 -1-1			
-	9D	24/16	50.00 - 52.00	22/21/20/25	41	62	97 116			Similar to 80.			
					-		116						
							129	1					
55			55.00 -			_	116			Occastonal Cabble. 957 blows for 0.5 ft.			G#3375
	100	24/15	55.00 - 57.00	17/19/30/49	49	74	OPEN HOLE	1		057 blows far 0.5 ft. Brown, wet, very dens (Glacfal Tfll),	e, Gravelly SAND.	trace silt.	м-1-Б. 9 WC=8.
		L			L	L	Ľ	1					
60						-	\vdash						
					-		\vdash		P	1			
								1					
	-				-								
65	110	24/18	65.00 - 67.00	22/31/34/39	65	99	V	1		Brown, wet, very dens slit, (Giaciai Till),	e, SAND, ITTTIe	grovel, lîttle	
	_	L	61.00		Ľ	L	L	297.9	14 A.A.A.A.A.A.A.A.A.A.A.A.A.A.A.A.A.A.A.				
		1						,		Bottom of Exploration Hole Abandoned. 10.0 abandoned in hole fro ft bgs (El. 297.9).	at 67.0 feet beig ft of broken HW m 57.0 ft bgs (8	w ground surface. 4°) casing 1. 307.9) to 67.0	
		<u> </u>			-		-	1	1	ft bgs (El. 297.9).			
70							-	1	1				
l								1	1				
					-								
					-	-		1					
75 Remor		mme: #101			·	ı	l	·	·				
2) 1	-UTO HO	of broke	n HW(4") oo	sina abandoned in	hole f	rom 57		or (5)	. 307.	9) to 67.0 ft bgs (E).	97.91.		

Oper o Logge	tort		S.W. Cole Kevîn/Brîan Wîlder/Puka		Dat	tum: 3 Type:	(ft.)	NAV	D88 drich D	-50	Auger 10/00: Sampler: Hammer Wt+/Fall	5" Soild S Standard S 140#/30"	plît Spe
Date		Finishi	8/22/2022-8 3+75.4, 9.2	/23/2022	Ori		Method	Cas	ad Wash 4.0*/4.	Bar Ing	Core Barrel: Noter Level*:	NO-2"	s.
					Hot	nor Tu	0.01	Autom	atic El	Hydroulic 🗆	Rope & Cotheod	* Pocket Torvone Shi	o. Kr Strengt
0 = Sp 100 = U U = Th	IT Spec	n Sample dui Spitt S Tube Sample	peon Somple An	SSA = Se tempt HSA = Ho RC = Res	illa Stem illa Stem illa Cane	Auger Auger		2.0	uncentin Corrected	Vane Undrained Shoor S ed Coloressive Strength = Raw Freid SP1 N-valu	Strength (psf) 5 (kaf) Je	IC = Bater Content, p LL = Liquid Limit PL = Picetic Limit	percent
NU - U Y - F7 NY - U	ald Vene	dul Thin Bo Shear Test dul Fleid V	II Tuba Somple PP = Pock one Shear Teat	R = Rock SSA = Sa SSA = Sa RC = Roj Attempt ICH = Ne et Penatrometer VCR/C Attempt ICH = I	light of 1 = Velight	of Rode One Pers	amer er Cost	Home Neo Neo	SPT N-L	ency factor - Rig Speci noerrected Carrected fe Efficiency Factor/601	Rope & Catheod fined Shear Strength (pef) t strength (pef) t start the strength (per) to the strength the strength (strength) to the strength this strength (strength) to the strength this strength (strength) to the strength (stre	ue Pi = Plosticity) = Grain Size Ansiy C = Consolidation Te	index s7s s7
_		÷	Depth	É _	,				•				Labara Testi Resul AASH
н (1 1	ole No.	/Rec.	Sample Dec (ft.)	Blows (/6 Shear Strength (bsf) or ROD (3)	N-uncorrected		2 2	Elevation (ft.)	ata Le	Visua	i Description and Rem	orka	Resul AASH and Unified
c Depth I	Somple	Per.	Song ft,	Blow Sheo Stre Stre Cpsf	N-N	N60	Castng Biows	Elev (ft.	Graph	14" HMA.			Unified
							SSA	363.6			-103 for samples up f		2-
										65 ft bgs.		o and meroding	
								-					
10													
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							1/	351.3				13.:	5-
15							V						
		<u> </u>					HP HP						
		-			-	-	HP 38						
					L	L	83	1					
20 -							66						
						-	27	-					
							42						
							71						
25 -							152						
							109 75						
							48						
ľ							36						
30 -							78						
							52 46						
							63		- 22 - 10 - 22 - 2 10 - 22 - 2				
							74	331.8				-33.1	0.
35 -							98		1.1				
							47 68						
							70						
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40 -							96						
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75 Remor	K51				<u> </u>	L		<u> </u>	BUN				1
		anner #361 araulic F											

filer: erator: ogged By		S.W. Cole Kevin/Brian	y	E I Da	i ON I evation tum: g Type: Tilling	(ft.)	364 NAV	.8 088 drīch I	Bridge #2651 corriss Boring No.: ver Pottes Brock Teld. Molne MIN: Auger 10/00: Sompler: -50 Hommer Wt./Foll Boring Core Borreit:	BB-FFPB-103A 25453:00 5" Solid Stem Standard Split Scen 140#/30" No-2"		ORTATION	
mmer Ef Inffrenss Spirt Sp - Unaucce	cation: fictency Fo wen Sample seaful Spift S	3+75.4. 9.2 Inctor: 0.91 Inctor: 0.91 In Tube Sample PP = Peck	ft Rt. R = Reck SSA = Se	Ho Core So Ind Stem Low Ste Low Ste		201	Automo Su = Sul I	4.0"/4 peck/Re ab) = LC Uncenf1 korrecte wr Effic = SPT N- = (Howe	5") Water Level*: Hydraulic Rope & Catheod C	13.0 ft bgs. Pocket lervone Sneor Strength (psf) = Noter Content, percent = Loude Lot	STATE OF MAINE	RANSP(
Somple No.	Pen./Rec. (In.)	Somple Depth (ft.)	Blows (/6 In. Shear Strength (psf) or ROD (%)	N-Uncorrected	N60	Castrig Bilows	Eleverton (ft.)	Graphic Log	Visual Description and Rem	and Unified Class	ATE OI	IT OF]	
	24/18	75.00 - 77.00	12/13/22/32	35	53			an an an Air an Air Air an Air an	Grey, met hord, SLLT, some sond, 119 (Gioclof Thi), ARTESIAN water pressure at \$2.0-85.0 f		ST	DEPARTMENT OF TRANSPORTATION	
		90.00 -							Crey, wet, very censs. CRAVEL, some so (Giocle ITII),	c com al (s			
20 	60/58	92.00 92.00 95.00 - 100.00	17/18/22/34 ROD = 82%	40	61	N0-2	269.8					SIGNATURE	
R2	36/36	100.00 - 103.00	ROD = 28%						Teo of Bearcoix of Line, 263.6 41. Bit Bearcoix for yo gone, generator-way offic and the second second second second of an intermediate of the second second of second second second second second second second second second second second Ret Core Times (efficiency) Ret C	fine-grained.	DATE	SIG	
R3	24/24	103.00 - 105.00	ROD = 88%			V	259.8		31 A Recovery R21 Bearcock: Crey te dork greenfah-gre thin-backed, S1L151064, moderotely how atolining, fracture zon neor middle of rock flour on fracture piones. [Sproguevile formition] R22 Core Times (mitisec) 100.0-010.0 4 (3128) 100.0-010.0 4 (3128) 100.0-100.0 5 (3128) 100.0 100.0 5 (3128) 1	freshi joints nor iron oxide un contains	8		
									1005 Recovery ESI Basrook Dray to dark green (an-yrk thin-baseds, SI (S100k, moeranter) har steely data for forcture through (spragew) lie formation) Rot duality is doed. 183 (S-106, 0) 184 (S-106, 0) 185 (S-106, 0	fine-grained. fresh, one the run.	MANAGER	D-REV	Ż
											PROJ. M		~ ~
													1100 1
	Hommer #36 Hydrou i to f		ota boundor fas batveen						Page 7 of 7		DOCK BRIDGE	BROOK	NOCEDCC

MAINE	TO DE LE CAL	KANSPUKIAHUN						_	00 BRIDGE PLANS	
STATE OF MAINE		DEPARIMENT OF IKANSPOKIATION		JEAE200	6407			MIM	BRIDGE NO 2691 25453 00	
		SIGNATURE			E. NUMBER			ATT: A	AIE	
DATE		S	OCT 2024		Ē					
BY			T. WHITE C							
PROJ. MANAGER	DESIGN-DETAILED	CHECKED-REVIEWED	DESIGN2-DETAILED2 N. PUKAY T	DESIGN3-DETAILED3	REVISIONS 1		REVISIONS 2	REVISIONS 3	REVISIONS 4	FIELD CHANGES
PUDDLE DOCK BRIDGE		PATTEE BROOK		_						FIE
s	н	EE			L IL	IV	ЛE	ЗE	R	
)					
		(DF	-	5					

<u>Appendix A</u>

Boring Logs

	UNIFIE	ED SOIL C	LASSIFIC	ATION SYSTEM		MODIFIED B	URMISTER S	YSTEM
			GROUP					
MAJ COARSE- GRAINED SOILS	GRAVELS	CLEAN GRAVELS (little or no	GW GP	TYPICAL NAMES Well-graded gravels, gravel- sand mixtures, little or no fines. Poorly-graded gravels, gravel	tr li	<u>tive Term</u> race ittle ome . Sandy, Clayey)	<u>Porti</u>	<u>on of Total (%)</u> 0 - 10 11 - 20 21 - 35 36 - 50
	alf of coan er than No size)	fines)		sand mixtures, little or no fines.	(0.g.	TERMS		3
is larger ize)	(more than half of coarse fraction is larger than No. 4 sieve size)	GRAVEL WITH FINES (Appreciable amount of	GM GC	Silty gravels, gravel-sand-silt mixtures. Clayey gravels, gravel-sand-clay mixtures.	sieve): Includes (´ Clayey or Gravelly penetration resista	<u>soils</u> (more than half o 1) clean gravels; (2) S y sands. Density is ra ance (N-value).	ilty or Clayey gravels ated according to star	nan No. 200 s; and (3) Silty, ndard
(more than half of material is larger than No. 200 sleve size)		fines) CLEAN	SW	Well-graded sands, Gravelly	<u>Cohesion</u> Very	<u>isity of</u> <u>nless Soils</u> / loose pose		enetration Resistance ue (blows per foot) 0 - 4 5 - 10
e than hal than No.	SANDS	SANDS (little or no	SP	sands, little or no fines Poorly-graded sands, Gravelly	De	m Dense ense Dense		11 - 30 31 - 50 > 50
(mor	(more than half of coarse fraction is smaller than No. 4 sieve size)	fines)		sand, little or no fines.	Fine-grained soil	ls (more than half of n 1) inorganic and orgar		an No. 200
	ire than h∉ on is small sieve s	SANDS WITH FINES	SM	Silty sands, sand-silt mixtures	or Silty clays; and strength as indica	., ,,	<u>Approximate</u>	ording to undrained shear
	(mo fracti	(Appreciable amount of fines)	SC	Clayey sands, sand-clay mixtures.	Consistency of Cohesive soils	SPT N ₆₀ -Value	<u>Undrained</u> <u>Shear</u> Strength (psf)	<u>Field</u> <u>Guidelines</u>
			ML	Inorganic silts and very fine sands, rock flour, Silty or Clayey fine sands, or Clayey silts with	Very Soft Soft Medium Stiff	WOH, WOR, WOP, <2 2 - 4 5 - 8	0 - 250 250 - 500 500 - 1000	Fist easily penetrates Thumb easily penetrates Thumb penetrates with
FINE- GRAINED	SILTS AN	ID CLAYS	CL	slight plasticity. Inorganic clays of low to medium plasticity, Gravelly clays, Sandy	Stiff Very Stiff	9 - 15 16 - 30	1000 - 2000 2000 - 4000	moderate effort Indented by thumb with great effort Indented by thumbnail
SOILS	(liquid limit l	ess than 50)	OL	clays, Silty clays, lean clays. Organic silts and organic Silty	Hard Rock Quality Des	>30 signation (RQD):	over 4000	Indented by thumbnail with difficulty
ial is e size)				clays of low plasticity.	RQD (%) =	sum of the lengths *Minimu	of intact pieces of length of core ad im NQ rock core (*	lvance
(more than half of material is smaller than No. 200 sieve size)	SILTS AN	ID CLAYS	СН	Inorganic silts, micaceous or diatomaceous fine Sandy or Silty soils, elastic silts. Inorganic clays of high		Rock Quality Ba <u>Rock Quality</u> Very Poor Poor	<u>RQD (%)</u> ≤25 26 - 50	
(more the smaller tha	(liquid limit gr	eater than 50)	ОН	plasticity, fat clays. Organic clays of medium to high plasticity, organic silts.	Desired Rock C	Fair Good Excellent Dbservations (in th	51 - 75 76 - 90 91 - 100 nis order, if appli	cable):
		ORGANIC	Pt	Peat and other highly organic soils.	Color (Munsell Texture (aphan Rock Type (gra		tc.) one, etc.)	
Desired Se	il Observet	tions (in thi	s order if	annlicable):			ht, moderate, mod	l. severe, severe, etc.)
Color (Muns Moisture (dr Density/Cor Texture (find Name (Sand Gradation (sell color ch ry, damp, m nsistency (fr e, medium, d, Silty San well-graded on-plastic, s ayering, frac ell, moderat n (weak, mo rigin (till, ma	art) oist, wet) om above ri coarse, etc. d, Clay, etc. , poorly-grad slightly plast ctures, crack ely, loosely, oderate, or s	ght hand s) , including ded, unifor ic, modera (s, etc.) etc.,) strong)	portions - trace, little, etc.) m, etc.) tely plastic, highly plastic)	Formation (Wat RQD and correl ref: ASTM D6 Site Characte Recovery (inch/ Rock Core Rate	35-55 deg., stee -spacing (very clos close - 1-3 feet, -tightness (tight, op -infilling (grain size terville, Ellsworth, C lation to rock quality 032 and FHWA NH erization, Table 4-12 /inch and percentag e (X.X ft - Y.Y ft (mi	ep - 55-85 deg., ve ee - <2 inch, close wide - 3-10 feet, v pen, or healed) , color, etc.) Cape Elizabeth, etc y (very poor, poor, 1I-16-072 GEC 5 - 2 ge) n:sec))	very wide >10 feet) c.) etc.) Geotechnical
Key	y to Soil a	Geotechi	<i>nical</i> Sec Descrip	tions and Terms	Sample Cont WIN Bridge Name Boring Numbe Sample Numb Sample Depth	er ber	Requirements: Blow Counts Sample Recove Date Personnel Initia	ery

g g	N	Aaine	-	artment Soil/Rock Expl	of Transporta	ation	1		-	161 o	ver Pat	e Brook	FFPB-101
Service Stain Datum: XAV08s Sampler: Human Spit Syon us BardPiritabilit: 3/10/22-01/2022 Drilling Method: Cack Weak Berling: Keek Berling: <				US CUSTOM/	ARY UNITS				aliO	n. For	raiffie	WIN:	5453.00
orgen d yr N. Nator Big Type: Dickich D-50 Hammer WL-Fill: N0-27 with generality A. 2010224 (1) 20224 (1) 2023 (1) 201024 (1) 2027 Dickich D-50 Hammer WL-Fill: N0-27 with generality A. 2010224 (1) 2023 (1) 201024 (1) 2027 Dickich D-50 Hammer WL-Fill: N0-27 with generality A. 2010224 (1) 2024 (1) 2024 A. 2010224 (1) 20	Drille	er:		S.W. Cole		Elev	vatior	ו (ft.))	367	6	Auger ID/OD: 5" Solid St	em
Bits Burger 11:00:00:00:00:00:00:00:00:00:00:00:00:0)per	ator:		Kevin/Brian		Dat	um:			NA	VD88	Sampler: Standard S	plit Spoon
Original Control NUMBER 1000:	.ogg	jed By:		N. Pukay		Rig	Туре	:		Die	lrich D	50 Hammer Wt./Fall: 140#/30"	
Bit Market Type: Automatic B Hydralic C Kop: 4 Callead C Get Boon Gamps 6. 46 Cot Societ 5. 46 Cot S	Date	Start/Fi	nish:	5/31/2022-6/1/	/2022	Dril	ling N	leth	od:	Cas	ed Was	Boring Core Barrel: NQ-2"	
Note: Picture	Borii	ng Loca	tion:	3+15.9, 6.7 ft	Rt.	Cas	ing II	D/OD):	NW	(3.0"/3	5"), HW(4.0"/4.5") Water Level*: 8.0 ft bgs.	
• (a) No. Status Bit Status (a) Status Bit Status			ciency F	actor: 0.91				Тур	e:				
0 0	0 = Sp /ID = 1 /I = Th /IU = 1 / = Fie	blit Spoon S Unsuccess hin Wall Tu Unsuccess eld Vane S	ful Split Sp be Sample ful Thin Wa hear Test,	all Tube Sample A PP = Pocket Per	SSA = Solid ppt HSA = Hollo RC = Roller ttempt WOH = Wei netrometer WOR/C = W	I Stem Ai ow Stem Cone ight of 14 Veight of	uger Auger Юlb. Ha Rods o	or Casi	r ing	S _{u(la} q _p = N-ur Ham N ₆₀	ab) = Lal Unconfi correcte mer Effi = SPT N	Vane Undrained Shear Strength (psf) WC = Water Content, ed Compressive Strength (ksf) LL = Liquid Limit = Raw Field SPT N-value PL = Plastic Limit ency Factor = Rig Specific Annual Calibration Value PI = Plasticity Index uncorrected Corrected for Hammer Efficiency G = Grain Size Analy	percent
Image:					Sample Information					ı —	4		Laborato
Image:	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	Testing Results AASHTC and Unified Cla
Image: space of the space	0											14" HMA.	
1D 24/11 5.00 - 7.00 5/49/4 13 20 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1										366.4			1.2-
1D 24/11 5.00 - 7.00 5/49/4 13 20 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1													
1D 24/11 5.00 - 7.00 5/49/4 13 20 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1								-	-				
1D 24/11 5.00 - 7.00 5/49/4 13 20 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1													
1D 24/11 5.00 - 7.00 5/49/4 13 20 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1													
Image: Same site some site some gravel. (Fill) GPAI A224, WC-1 Image: Same site some gravel. (Fill) GPAI A224, WC-1 Image: Same site some gravel. (Fill) GPAI A224, WC-1 Image: Same site some gravel. (Fill) GPAI A224, WC-1 Image: Same site some gravel. (Fill) GPAI A224, WC-1 Image: Same site some gravel. (Fill) GPAI A224, WC-1 Image: Same site some gravel. (Fill) GPAI A224, WC-1 Image: Same site some gravel. (Fill) GPAI A224, WC-1 Image: Same site some gravel. (Fill) GPAI A224, WC-1 Image: Same site some site some site some gravel. (Fill) GPAI A224, WC-1 Image: Same site some some site some some some some some some some som	5 -	1D	24/11	5.00 7.00	5/4/0/4	12	20	-				Brown, damp, medium dense, Gravelly SAND, little silt, (Fill).	
2D 24/15 10:00 - 12:00 4/3/5/5 8 12 41 1 1 1 41 41 2 1 1 41 41 1 1 44 44 1 1 44 48 2 1 1 44 3 1 1 40 3 1 1 40 3 1 1 40 3 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 <td></td> <td>ID</td> <td>24/11</td> <td>5.00 - 7.00</td> <td>5/4/9/4</td> <td>15</td> <td>20</td> <td>-</td> <td></td> <td></td> <td></td> <td></td> <td></td>		ID	24/11	5.00 - 7.00	5/4/9/4	15	20	-					
2D 24/15 10:00 - 12:00 4/3/5/5 8 12 41 1 1 1 41 41 2 1 1 41 41 1 1 44 44 1 1 44 48 2 1 1 44 3 1 1 40 3 1 1 40 3 1 1 40 3 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 <td></td>													
2D 24/15 10:00 - 12:00 4/3/5/5 8 12 41 1 1 1 41 41 2 1 1 41 41 1 1 44 44 1 1 44 48 2 1 1 44 3 1 1 40 3 1 1 40 3 1 1 40 3 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 <td></td>													
2D 24/15 10:00 - 12:00 4/3/5/5 8 12 41 1 1 1 41 41 2 1 1 41 41 1 1 44 44 1 1 44 48 2 1 1 44 3 1 1 40 3 1 1 40 3 1 1 40 3 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td>+</td> <td></td> <td></td> <td></td> <td></td> <td></td>								+					
2D 24/15 10:00 - 12:00 4/3/5/5 8 12 41 1 1 1 41 41 2 1 1 41 41 1 1 44 44 1 1 44 48 2 1 1 44 3 1 1 40 3 1 1 40 3 1 1 40 3 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td>+</td> <td>+/-</td> <td></td> <td></td> <td></td> <td></td>								+	+/-				
2D 24/15 10:00 - 12:00 4/3/5/5 8 12 41 1 1 1 41 41 2 1 1 41 41 1 1 44 44 1 1 44 48 2 1 1 44 3 1 1 40 3 1 1 40 3 1 1 40 3 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 <td>10 -</td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td> \</td> <td>\bigvee</td> <td></td> <td></td> <td></td> <td></td>	10 -							\	\bigvee				
Image:	10	2D	24/15	10.00 - 12.00	4/3/5/5	8	12	4	41			Brown, damp, medium dense, SAND, some silt, some gravel, (Fill)	. G#24152 A-2-4, SN
Image: Second state in the state of the state in the state. Image: Second state in the state. Second state.									11				WC=11.6
A matrix 322 4/4 15.00 - 17.00 3/2/2/1 4 6 10 A matrix 322 6 matrix A matrix 320 6 matrix A m								-	+1				
3 1								4	48				
3D 24/4 15.00 · 17.00 3/2/2/1 4 6 10 1 1 17 1 17 1 16 1 1 1 17 1 16 10 1 1 10									38				
3D 24/4 15.00 · 17.00 3/2/2/1 4 6 10 1 1 17 1 17 1 16 1 1 1 17 1 16 10 1 1 10									10				
3D 24/4 15.00 - 17.00 3/2/2/1 4 6 10 1 1 1 17 18 17 18 19 10 18 10<	15 -								+0	352.6			5.0-
image: image		3D	24/4	15.00 - 17.00	3/2/2/1	4	6		10			Grey, wet, loose, Sandy GRAVEL, some silt, (Stream Alluvium).	
i i									17				
i i								<u> </u>	18		580151 H		
i i i i <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td>-</td> <td></td> <td></td> <td></td> <td></td> <td></td>								-					
a) iiiiiiiiiiiiiiiiiiiiiiiiiiiiiiiiiiii								1	22				
Image: state of the state									10				
4D 24/3 21.00 - 23.00 5/3/4/2 7 11 16 Image: Image	20 -								10				
4D 24/3 21.00 - 23.00 5/3/4/2 7 11 16 1 1 1 24 24 1 10 10 10 1 1 1 24 24 24 10 10 10 10 10 1 1 1 1 22 10 17 10								-				Similar to 3D, except medium dense.	
Image:		4D	24/3	21.00 - 23.00	5/3/4/2	7	11		16			-	
i i									24				
i i									22				
amarks: INMEDIAN) Auto Hammer #367) 15 feet of 3" casing (NW) abandoned in hole from 46.5 BGS (El. 321.1) to 61.5 BGS (El. 306.1) atification lines represent approximate boundaries between soil types; transitions may be gradual. Page 1 of 3 //ater level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other Page 1 of 3								+					
) Auto Hammer #367) 15 feet of 3" casing (NW) abandoned in hole from 46.5 BGS (El. 321.1) to 61.5 BGS (El. 306.1) atification lines represent approximate boundaries between soil types; transitions may be gradual. /ater level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other	25								17		間間		
/ater level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other	1) A	uto Ham		(NW) abandone	d in hole from 46.5 BG	S (El. 32	21.1) t	o 61.	5 BG	S (El. 3	06.1)		
/ater level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other	tratifi	cation line:	s represent	approximate bour	ndaries between soil types; t	ransition	s may t	be gra	dual.			Page 1 of 3	
han those present at the time measurements were made. Boring No.: BB-FFPB-101	Wate	er level rea	dings have	been made at time	es and under conditions stat	ted. Grou	undwat	er fluc	tuatio	ns may c	ccur du	to conditions other	

N	/laine	e Dep	artment	of Transport	atio	n	Project:			Bridge #2691 carries Route	Boring No.:	BB-FF	PB-101
			Soil/Rock Exp US CUSTOM/				Locatio			ee Brook ld, Maine	WIN:	254	52.00
			03 00310101	<u>ART UNITS</u>							WIN:	234.	53.00
Drille	er:		S.W. Cole		Ele	vation	n (ft.)	367.	5		Auger ID/OD:	5" Solid Stem	
Oper	ator:		Kevin/Brian		Dat	tum:		NAV	/D88		Sampler:	Standard Split	Spoon
.ogg	ed By:		N. Pukay		Rig	ј Туре	:	Died	rich D	-50	Hammer Wt./Fall:	140#/30"	
Date	Start/Fi	nish:	5/31/2022-6/1	/2022	Dri	lling N	lethod:			h Boring	Core Barrel:	NQ-2"	
Borir	ng Loca	tion:	3+15.9, 6.7 ft	Rt.	_	sing IC		NW	3.0"/3	5"), HW(4.0"/4.5")	Water Level*:	8.0 ft bgs.	
Definit D = Sp /ID = I /ID = I /IU = I /IU = I	ions: Ilit Spoon S Jnsuccess in Wall Tu Jnsuccess Id Vane S	Sample ful Split Sp be Sample ful Thin Wa hear Test,	oon Sample Atten all Tube Sample A PP = Pocket Pe ane Shear Test Att	RC = Rolle WOH = We netrometer WOR/C = V	Core Sam d Stem A ow Stem r Cone eight of 1- Weight of	nple Auger Auger Auger 40 lb. Ha f Rods o	r Casing	S _{u(la} q _p = N-un Hami N ₆₀ :	Peak/R b) = Lat Unconfi correcte ner Effi = SPT N	Hydraulic □ amolded Field Vane Undrained She vane Undrained Shear Strength (hed Compressive Strength (ksf) d = Raw Field SPT N-value ciency Factor = Rig Specific Annual -uncorrected Corrected for Hamme ner Efficiency Factor/60%)*N-uncor	psf) WC = LL = PL = I Calibration Value PI = I er Efficiency G = 0	Pocket Torvane She = Water Content, per Liquid Limit Plastic Limit Plasticity Index Grain Size Analysis Consolidation Test	
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or ROD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)	Graphic Log	Visual De	scription and Remarks		Laborato Testing Results, AASHTC and Unified Cla
25	5D	24/11	25.00 - 27.00	7/10/6/6	16	24	43			Grey-brown, wet, medium	dense, Sandy GRAVEL, s	ome silt, (Stream	
				-			45			Alluvium).			
									200 000 000 000				
							37						
							60						
30 -							88		9 2 6 1 9 60 1 60 1 60	5			
,0	MD	24/0	30.00 - 32.00	11/11/10/15	21	32	46						
							42		6 6 6 6 6 6 6 6 6 6 6 6 6 6 6 6 6 6 6				
							45		0 9 9 9 9 9 9 9 9 9 9 9 9 9 9 9 9 9 9 9				
	6D	24/14	33.00 - 35.00	11/9/9/9	18	27	50	334.6		Grey, wet, medium dense, f	fine SAND, little silt, (Stre	— — — —33.0 eam Alluvium).	G#33750 A-2-4, SI
							52						WC=20.2
35 -	7D/A	24/20	35.00 - 37.00	5/10/18/27	28	42	76		1000 0000 1000 0000 1000 0000	7D (35.0-36.5 ft bgs.) Simi	lar to 6D, except dense.		
							146	331.1		7D/A (36.5-37.0 ft bgs.) Br	own-grey, wet, dense, Gra	avelly SAND,	
							132			some silt, (Glacial Till).			
							251						
10							174						
40 -	8D	24/15	40.00 - 42.00	14/16/36/32	52	79	38			Brown-grey, wet, very dens Till).	se, SAND, some silt, trace	gravel, (Glacial	G#33750 A-1-b, Sl
							50			Roller Coned ahead from 4	0.0-45.0 ft bgs.		WC=12.7
							45	1					
							64						
							129						
45 -	9D	24/17	45.00 - 47.00	28/37/27/37	64	97	59			Brown, wet, very dense, Sil Roller Coned ahead from 4		Blacial Till).	
							87			Koner Coneu aneau from 4	5.0-50.0 It bgs.		
									1. 1. 1.				
							125						
							118						
50							158			4			

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.	Page 2 of 3
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.	Boring No.: BB-FFPB-101

Ι	Maine	e Dep	artment	of Transport	ation		Project:			Bridge #2691 carries Route	Boring No.:	BB-FF	PB-101
			Soil/Rock Exp US CUSTOM				Locatio			ee Brook ld, Maine	WIN:	2545	53.00
Drill	or:		S.W. Cole		Elevat	lion	(#+)	367.	6		Auger ID/OD:	5" Solid Stem	
	rator:		Kevin/Brian		Datum		(11.)		7 D88		Sampler:	Standard Split	Spoon
	ged By:		N. Pukay		Rig Ty				rich D	-50	Hammer Wt./Fall:	140#/30"	Spoon
	Start/Fi	nish:	5/31/2022-6/1	/2022		-	ethod:	Case	d Wasl	n Boring	Core Barrel:	NQ-2"	
Bori	ng Loca	tion:	3+15.9, 6.7 ft	Rt.	Casin	g ID	/OD:	NW	3.0"/3	5"), HW(4.0"/4.5")	Water Level*:	8.0 ft bgs.	
		ciency F	actor: 0.91		Hamm		Туре:	Automa		Hydraulic 🗆	Rope & Cathead □		
MD = U = TI MU = V = Fi	plit Spoon S Unsuccess hin Wall Tu Unsuccess eld Vane S	ful Split Sp be Sample ful Thin Wa hear Test,	oon Sample Atter III Tube Sample A PP = Pocket Pe <u>ne Shear Test At</u>	SSA = Sol mpt HSA = Ho RC = Rolle Attempt WOH = W enetrometer WOR/C =	Core Sample id Stem Auge low Stem Auge or Cone eight of 140 lk Weight of Roo Veight of One	er ger b. Ha ds or	Casing	S _{u(la} q _p = N-un Hami N ₆₀ :	b) = Lab Unconfir correcte ner Effic = SPT N	emolded Field Vane Undrained She Vane Undrained Shear Strength (ned Compressive Strength (ksf) d = Raw Field SPT N-value isiency Factor = Rig Specific Annual -uncorrected Corrected for Hamme ner Efficiency Factor/60%)*N-uncor	psf) WC LL PL I Calibration Value PI er Efficiency G =	= Pocket Torvane She C = Water Content, per = Liquid Limit = Plastic Limit = Plasticity Index = Grain Size Analysis = Consolidation Test	
		÷	1		pe								Laboratory Testing
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log		scription and Remark		Results/ AASHTO and Unified Class.
50	10D	24/19	50.00 - 52.00	18/24/31/50	55 8	83	OPEN HOLE			Brown, wet, very dense, SA	AND, some silt, trace gr	avel, (Glacial Till).	
- 55 -	11D	24/21	55.00 - 57.00	25/29/39/38	68 1	03				Similar to 10D.			
- 60 -										Brown, wet, very dense, SA	AND, some silt, trace ro	ck fragments.	
	12D	18/17	60.00 - 61.50	30/46/55	101 1	53	v			Set in NW casing and drove			
	R1						NQ-2	306.1		Cored from 61.5-71.5 ft bg Rock not encountered.		61.5- lacial till. Top of	
- 65 -													
- 70 -													
								296.1		Bottom of Exploration Hole Abandoned due to stu left in hole from 46.5 ft bgs		/(3") Casing was	
1) A 2) 1 Stratif	ication lines	3" casing	approximate bou	ed in hole from 46.5 BC ndaries between soil types:	transitions m	nay b	e gradual.			s to conditions other	Page 3 of 3		
		-	ime measuremen					, -			Boring No	b.: BB-FFPB	-101

1	Maine	e Dep	artment	of Transporta	tion	l	Proj	ect:			Bridge #2691 carries Route	Boring No.:	BB-FF	PB-102
			Soil/Rock Expl				Loca	atio			ee Brook ld, Maine			
			US CUSTOM/	ARY UNITS								WIN:	2543	53.00
Drill	er:		S.W. Cole		Elev	/ation	(ft.)		368.	3		Auger ID/OD:	5" Solid Stem	
Ope	rator:		Kevin/Brian		Dati		()			/D88		Sampler:	Standard Split	Spoon
<u> </u>	ged By:		Wilder/Pukay		Rig	Туре			Died	rich D-	50	Hammer Wt./Fall:	140#/30"	1
	Start/Fi	nish:	8/15/2022-8/1	6/2022		ling N		d:	Case	d Wasl	1 Boring	Core Barrel:	NQ-2"	
Bori	ng Loca	tion:	3+00.6, 6.6 ft	Lt.	Cas	ing ID)/OD:		NW	(3.0"/3.	5"), HW(4.0"/4.5")	Water Level*:	16.0 ft bgs.	
Ham	mer Effi	ciency F	actor: 0.91		Han	nmer	Туре	:	Automa			Rope & Cathead □		
Defini D = S	tions: plit Spoon :	Sample		R = Rock Co SSA = Solid	ore Samp Stem Au	ole uger					emolded Field Vane Undrained She Vane Undrained Shear Strength (Pocket Torvane She Water Content, per	
MD =	Unsuccess		oon Sample Atter		w Stem				q _D =	Únconfir	ned Compressive Strength (ksf) d = Raw Field SPT N-value	LL =	Liquid Limit Plastic Limit	
MU =	Unsuccess	ful Thin Wa	all Tube Sample A PP = Pocket Per	ttempt WOH = Weig	ght of 14			a	Ham	mer Effic	iency Factor = Rig Specific Annual -uncorrected Corrected for Hamme	Calibration Value PI = I	Plasticity Index Grain Size Analysis	
			ine Shear Test Att	wo1P = We				9			her Efficiency Factor/60%)*N-uncor		Consolidation Test	
		-		Sample Information	70									Laboratory
	ö	Ē.	epth	ii.) %	scted					bo				Testing Results/
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected		0		Elevation (ft.)	Graphic Log	Visual De	scription and Remarks		AASHTO
epth	amp	l/.ne	amp (;	ows near trent sf) - RQ	oun-	N ₆₀	Casing	ows	evat t.)	rapł				and Unified Class.
	ů	ď	й£	ଌୢ୰ଊୡୢଌ	ż	ž	Ű	B	Ξŧ	G	9" HMA.			
Ŭ							SS	A	367.6	*****	9 HMA.			
- 5 -											Brown, damp, medium dens	se, SAND, some silt, som	ne gravel, (Fill).	
	1D	24/15	5.00 - 7.00	4/7/7/5	14	21					-			
- 10 -	210	24/14	10.00 12.00	2/2/2/2	5	0		_			Brown, wet, medium stiff, S	Sandy SILT, little gravel,	(Fill).	G#337508
	2D	24/14	10.00 - 12.00	2/2/3/2	5	8								A-4, CL WC=14.9%
15														
- 15 ·	3D	24/14	15.00 - 17.00	4/8/8/6	16	24			352.8				15.5-	G#337509
			+ +								Brown, wet, medium dense. Alluvium).	, GRAVEL, some sand, li	ttle silt, (Stream	A-1-b, GM
														WC=16.8%
							$ \rangle$	1						
							\square	1						
							+	\vdash						
- 20 -								1			Grey, wet, loose, Sandy GR	AVEL some silt (Stream	Alluvium)	G#337510
	4D	24/19	20.00 - 22.00	4/3/2/3	5	8	3'	7			orcy, wet, toose, Salidy OK	AT THE, SOME SHE (SHEAM	, a sinu v luiii <i>j</i> .	A-1-b, GM
							4	6						WC=26.0%
										ही थे। इ.स. स. स. स.				
							5	9						
							7	7		SIC 196				
							6	8		900 P010				
25 Rem	arks:				[副船路	1			
		mer #367												
	MIC FIAID	mei #30/												
Stratit	ication line	s represent	approximate bour	ndaries between soil types; tr	ansitions	s may b	e grad	ual.				Page 1 of 6		
		-		es and under conditions state	ed. Grou	undwate	er flucti	uatio	ns may o	ccur due	to conditions other	Doring No.	• DD DDD	102
thar	those pres	sent at the t	ime measurement	ts were made.								Boring No.	. вв-ггрв	-102

I	Maine	e Dep	artment	of Transport	atio	n	Project			Bridge #2691 carries Route	Boring No.:	BB-FF	PB-102
			Soil/Rock Exp				Locatio			tee Brook eld, Maine			
		<u> </u>	US CUSTOM/	ARY UNITS						,	WIN:	2545	53.00
Drill	er.		S.W. Cole		Fle	vation	(ft)	368	3		Auger ID/OD:	5" Solid Stem	
	rator:		Kevin/Brian		_	tum:	(14)		VD88		Sampler:	Standard Split	Spoon
⊢÷–	ged By:		Wilder/Pukay		_	a Type			drich E	9-50	Hammer Wt./Fall:	140#/30"	Spoon
<u> </u>	Start/Fi	nish	8/15/2022-8/1	6/2022	-		lethod:			sh Boring	Core Barrel:	NQ-2"	
	ng Loca		3+00.6, 6.6 ft		_	sing I				5.5"), HW(4.0"/4.5")	Water Level*:	16.0 ft bgs.	
			actor: 0.91	Li.	_	mmer			hatic \boxtimes		Rope & Cathead	10.0 ft bgs.	
Defini		CIENCY F	actor. 0.91	R = Rock C			Type.			Hydraulic emolded Field Vane Undrained She	1	Pocket Torvane She	ar Strength (psf)
	plit Spoon : Unsuccess		oon Sample Atten	npt SSA = Soli HSA = Holl				S _u (ab) = La	b Vane Undrained Shear Strength (ined Compressive Strength (ksf)		 Water Content, per Liquid Limit 	cent
U = T	nin Wall Tu	ibe Sample	•	RC = Rolle	r Cone	-		N-u	ncorrect	ed = Raw Field SPT N-value	PL =	Plastic Limit	
V = Fi	eld Vane S	Shear Test,	II Tube Sample A PP = Pocket Pe	netrometer WOR/C = \	Neight o	f Rods o	r Casing	N60	= SPT I	ciency Factor = Rig Specific Annua N-uncorrected Corrected for Hamme	er Efficiency G = G	Plasticity Index Grain Size Analysis	
MV =	Unsuccess	ful Field Va	ne Shear Test Att	tempt WO1P = W Sample Information	eight of	One Per	son	N ₆	<u>= (Ham</u>	mer Efficiency Factor/60%)*N-unco	rrected C = C	Consolidation Test	
		$\widehat{}$	1		σ				1				Laboratory
	ö	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psť) or RQD (%)	N-uncorrected				og				Testing Results/
Depth (ft.)	Sample No.	Rec		gth 2D (corre		<u>م</u>	Elevation	Graphic Log	visual De	scription and Remarks		AASHTO
epth	dme	l/.ue	d mg ()	ows near reng sf) RC	oun-	N ₆₀	Casing Blows	eva	rapt				and Unified Class.
25	ů	ď	йĘ	ଌ୕୵୰୰ଌ	z	z	U m m	Ξŧ		Crow wat madium dance i	CDAVEL come cond litt	la cilt (Stucom	
25	5D	24/15	25.00 - 27.00	13/9/7/13	16	24	72			Grey, wet, medium dense, Alluvium).	GRAVEL, some sand, int	ie siit, (Stream	
							58		848) 6489				
1								-					
							49			þ 9			
							46						
								-		1 ž			
- 30 -							49		e Zeo Geo				
50	6D	24/14	30.00 - 32.00	14/8/7/7	15	23	46			Grey, wet, medium dense, Alluvium).	Sandy GRAVEL, little sil	t, (Stream	
							43		er X . er e				
							43	-	i c i o	Ş			
							51	0.00	86 P			22.0	
							83	335.	80 Pe 3			33.0	
								-		Grey, wet, very dense, SAN	ND, some gravel, some sil	t, (Glacial Till).	G#337511
- 35 -	7D	24/15	34.00 - 36.00	12/21/20/14	41	62	30						A-1-b, SM
55							OPEN			4			WC=11.1%
							HOLE	-					
								4					
								-					
10													
- 40 -	8D	24/13	40.00 - 42.00	17/12/10/19	22	33				Similar to 7D, except dense	2.		
	-							-					
1													
1							+	-					
1						L		4					
1													
- 45 -	9D	24/20	45.00 - 47.00	13/15/25/49	40	61		1		Brown, wet, very dense, SA	AND, some silt, little grav	el, (Glacial Till).	G#337512
1	90	24/20	+5.00 - 47.00	13/13/23/47	+0	01		-					A-2-4, SM WC=13.7%
1													
1								1					
1								-					
1													
1													
50 Rem	arks:	1				L							
		mer #367											
	sato riam	uner #30/											
1													
Stratif	ication line	s represent	approximate bour	ndaries between soil types;	transitio	ns may h	e gradual				Page 2 of 6		
											1 age 2 01 0		
		-	been made at tim	es and under conditions sta ts were made.	ited. Gro	oundwate	er fluctuati	ons may	occur du	e to conditions other	Boring No.	: BB-FFPR	-102
uial	anose pres	כייי מי נוופ נ	e measuremen	to word made.									102

Ι	Maine	e Dep	artment	of Transport	atio	n	Proje				Bridge #2691 carries Route	Boring No.:	BB-FF	PB-102
			Soil/Rock Exp JS CUSTOM				Locat				ee Brook ld, Maine	WIN:	2545	53.00
Drill	er:		S.W. Cole		Ele	vatior	n (ft.)		368.3	;		Auger ID/OD:	5" Solid Stem	
	rator:		Kevin/Brian		_	tum:	. ()		NAV			Sampler:	Standard Split	Spoon
· ·	ged By:		Wilder/Pukay		-	ј Туре	:			rich D	-50	Hammer Wt./Fall:	140#/30"	~F
	Start/Fi	nish:	8/15/2022-8/1	6/2022	_		lethod				h Boring	Core Barrel:	NQ-2"	
	ng Loca		3+00.6, 6.6 ft		_	sing II					5"), HW(4.0"/4.5")	Water Level*:	16.0 ft bgs.	
	_		actor: 0.91		_	-	Type:		itoma		Hydraulic 🗆	Rope & Cathead		
Defini	tions:			R = Rock C	ore Sam	nple			S _u = F	Peak/Re	emolded Field Vane Undrained She	ear Strength (psf) T _V	= Pocket Torvane She	
MD = U = TI MU = V = Fi	hin Wall Tu Unsuccess eld Vane S	ful Split Sp be Sample ful Thin Wa hear Test,	oon Sample Atten II Tube Sample A PP = Pocket Pe <u>ne Shear Test Att</u>	RC = Rolle ttempt WOH = We netrometer WOR/C = V	ow Stem r Cone eight of 14 Veight of	40 lb. Haf Rods o	r Casing		$q_p = l$ N-unc Hamn N ₆₀ =	Ínconfii orrecte ter Effic SPT N	 vane Undrained Shear Strength (hed Compressive Strength (ksf) d = Raw Field SPT N-value ciency Factor = Rig Specific Annua uncorrected Corrected for Hammener Efficiency Factor/60%)*N-unco 	LL PL Calibration Value PI er Efficiency G :	C = Water Content, pero = Liquid Limit = Plastic Limit = Plasticity Index = Grain Size Analysis = Consolidation Test	cent
				Sample Information										Laboratory
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Rlows		(ft.)	Graphic Log	Visual De	scription and Remark	s	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			306.3		Brown, wet, very dense, S <i>i</i>	ND, little silt, little gra	vel, (Glacial Till).	
- 65 - - 70 -	12D	6/5	65.00 - 65.50	70(6")							Grey-brown, wet, very den Till).	se, GRAVEL, some san	d, little silt, (Glacial	
	arks:	mer #367									Occasional Cobble.			
Stratif	ication lines	s represent	approximate bou	ndaries between soil types;	transitior	ns may b	oe gradua	al.				Page 3 of 6		
		-	been made at tim me measuremen	es and under conditions sta ts were made.	ted. Gro	oundwat	er fluctua	itions r	may oc	cur due	e to conditions other	Boring No	o.: BB-FFPB	-102

N	Aaino	e Depa	artment	of Transport	ation	l	Proje	ect:			Bridge #2691 carries Route	Boring No.:	BB-FF	PB-102
			Soil/Rock Exp JS CUSTOM/				Loca	tior			tee Brook eld, Maine	WIN:	2545	53.00
Drille			C.W. Cala		Elev	ation	(64.)		269.5				5" C -1: 1 Ctores	
	ator:		S.W. Cole Kevin/Brian		Datu		(n.)		368.3 NAV			Auger ID/OD: Sampler:	5" Solid Stem Standard Split	Spoon
· ·	ged By:		Wilder/Pukay		-	туре			Died		0-50	Hammer Wt./Fall:	140#/30"	opoon
	Start/Fi	nish:	8/15/2022-8/1	6/2022	_		letho	d:			sh Boring	Core Barrel:	NQ-2"	
Bori	ng Loca	tion:	3+00.6, 6.6 ft	Lt.	-	-)/OD:				3.5"), HW(4.0"/4.5")	Water Level*:	16.0 ft bgs.	
Ham	mer Effi	ciency F	actor: 0.91		Ham	nmer	Type:		Automa	tic 🛛	Hydraulic 🗆	Rope & Cathead □		
MD = U = Th MU = V = Fie	olit Spoon Unsuccess hin Wall Tu Unsuccess eld Vane S	ful Split Spo be Sample ful Thin Wa hear Test,	oon Sample Atten II Tube Sample A PP = Pocket Pe ne Shear Test Att	RC = Rolle ttempt WOH = We netrometer WOR/C = V	d Stem Au ow Stem A r Cone eight of 140 Veight of F	iger Auger 0 lb. Ha Rods o	r Casin	g	S _{u(lat} q _p = U N-unc Hamn N ₆₀ =) = La Jnconf orrecte ner Eff SPT I	temolded Field Vane Undrained Sh b Vane Undrained Shear Strength i ined Compressive Strength (ksf) ad = Raw Field SPT N-value ciciency Factor = Rig Specific Annua N-uncorrected Corrected for Hamm mer Efficiency Factor/60%)'N-unco	(psf) WC LL = PL = I Calibration Value PI = er Efficiency G =	Pocket Torvane Shea = Water Content, pero Liquid Limit • Plastic Limit Plasticity Index Grain Size Analysis Consolidation Test	ent
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing	Blows	Elevation (ft.)	Graphic Log		escription and 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 gree	у		
- 80 -											Occasional Cobble.			
- 85 -	14D	18/16	85.00 - 86.50	22/25/63	88	133					Grey, wet, very dense, SAI	ND, some gravel, little sil	t, (Glacial Till).	
- 90 -	15D	24/14	95.00 - 97.00	9/9/16/23	25	38					Grey, wet, dense, Silty SA	ND, little gravel, (Glacial	Till).	
								\square		2 • ¥				
								Π						
100 Rem	arks:													
		mer #367												
Stratifi	cation line	s represent	approximate bour	ndaries between soil types;	transitions	s may b	e grad	ual.				Page 4 of 6		
* Wate	er level rea	dings have		es and under conditions sta					ns may oc	cur du	e to conditions other	-	.: BB-FFPB	-102
												•		

N	Iain	e Depa	artment	of Transporta	tion	Project		Bridge #2691 carries Route	Boring No.:	BB-FF	PB-102
			Soil/Rock Exp JS CUSTOM	•		Locatio	161 over Patt on: Fort Fairfie		WIN:	2545	3.00
Drille			S.W. Cole		Elevatio	n (ft)	368.3		Auger ID/OD:	5" Solid Stem	
Oper			Kevin/Brian		Datum:		NAVD88		Sampler:	Standard Split S	Spoon
	ed By:		Wilder/Pukay	,	Rig Type		Diedrich D	-50	Hammer Wt./Fall:	140#/30"	spoon
	Start/Fi	nish	8/15/2022-8/1		Drilling		Cased Was		Core Barrel:	NQ-2"	
	ng Loca		3+00.6, 6.6 ft		Casing I			5"), HW(4.0"/4.5")	Water Level*:	16.0 ft bgs.	
	•		actor: 0.91		Hammer		Automatic 🛛	Hydraulic 🗆	Rope & Cathead □	6	
Definit				R = Rock Co			S _u = Peak/Re	emolded Field Vane Undrained Sh	ear Strength (psf) T _V =	Pocket Torvane Shea = Water Content, perc	
MD = 0 U = Th MU = 0 V = Fie	Jnsuccess in Wall Tu Jnsuccess eld Vane S	sful Split Spo be Sample sful Thin Wa Shear Test,	oon Sample Atter II Tube Sample A PP = Pocket Pe ne Shear Test At	RC = Roller Mutempt WOH = Weig whether WOR/C = W WO1P = Weig WO1P = Weig	w Stem Auger	or Casing	q _p = Unconfii N-uncorrecte Hammer Effic N ₆₀ = SPT N	 Vane Undrained Shear Strength (hed Compressive Strength (ksf) d = Raw Field SPT N-value iency Factor = Rig Specific Annua -uncorrected Corrected for Hammer efficiency Factor/60%)*N-unco 	LL = PL = I Calibration Value PI = er Efficiency G =	Liquid Limit Plastic Limit Plasticity Index Grain Size Analysis Consolidation Test	
				Sample Information	70						Laboratory
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected N ₆₀	Casing Blows	Elevation (ft.) Graphic Log	Visual De	escription and Remarks		Testing Results/ AASHTO and Unified Class.
100											
- 105 -			105.10 -				263.2	Cobble at 104.8 ft bgs. Roller Coned ahead to 105.	1 ft bas		
	R1	26.4/24	107.30	RQD = 76%		NQ-2		\	-		
								Top of bedrock at Elev. 26 R1: Bedrock: Grey to dark		ed, thin-bedded,	
	R2	9.6/7	107.30 -	RQD = 0%				SILTSTONE, moderately h	hard, fresh, steeply dippin		
	K2	9.0/7	108.10 108.10 -	KQD = 070				spaced, with some quartz o [Spragueville Formation]	r calcite infilling.		
	R3	48/48	112.10	RQD = 48%				Rock Quality = Good.			
								R1: Core Times (min:sec) 105.1-106.1 ft (2:01)			
- 110 -								106.1-107.1 ft (2:04) 107.1-107.3 ft (1:04) Core	Disakad		
								92% Recovery	DIOCKEU		
								R2: Bedrock: Similar to R1	except with a vertical fra	cture throughout	
	R4	36/35	112.10 -	RQD = 92%			1 []]	run. Fracture plane is fresh	with minor iron oxide sta		
			115.10					[Spragueville Formation] Rock Quality = Very Poor.			
								R2: Core Times (min:sec)			
								107.3-108.1 ft (2:32) Core 70% Recovery	Blocked		
- 115 -						¥	- 253.2 (XXXI) -	R3: Bedrock: Grey to dark SILTSTONE, moderately I with some quartz or calcite minor oxide staining, core [Spragueville Formation] Rock Quality = Poor. R3: Core Times (min:sec) 108.1-109.1 ft (2:42) 109.1-110.1 ft (3:02)	hard, fresh, vertical joints, infilling, fracture planes	closely spaced, are fresh with	
- 120 -								110.1-111.1 ft (2:46) 111.1-112.1 ft (3:00) 100% Recovery			
							1	R4: Bedrock: Grey to dark	greenish-grev. fine-grain	ed, thin-bedded.	
							1	SILTSTONE, moderately h	hard, fresh, joints dipping		
								angles, spaced moderately [Spragueville Formation]	ciose.		
								Rock Quality = Excellent.			
							1	R4: Core Times (min:sec) 112.1-113.1 ft (1:59)			
125 Rem	arks:										
1) A Stratifi	uto Ham	-		ndaries between soil types; tr	-	-			Page 5 of 6		
		-	been made at tim me measuremen	nes and under conditions state ts were made.	d. Groundwa	ter fluctuati	ons may occur due	e to conditions other	Boring No	: BB-FFPB-	-102

I	Main	e Dep	artment	of Tran	sporta	tion		Project:			Bridge #2691 carries Route	Boring No.:	BB-FF	PB-102
		-	Soil/Rock Exp		-			Locatio			ee Brook Id, Maine			
			US CUSTOM	IARY UNITS								WIN:	254	53.00
Drill	er:		S.W. Cole			Elev	ation	(ft.)	368.3	3		Auger ID/OD:	5" Solid Stem	
Ope	rator:		Kevin/Brian			Datu	ım:		NAV	/D88		Sampler:	Standard Split	Spoon
Log	ged By:		Wilder/Pukay	y		Rig	Туре		Died	rich D	-50	Hammer Wt./Fall:	140#/30"	
	Start/F		8/15/2022-8/					lethod:	Case	d Was	h Boring	Core Barrel:	NQ-2"	
	ng Loc		3+00.6, 6.6 ft	t Lt.		-	-	D/OD:			.5"), HW(4.0"/4.5")	Water Level*:	16.0 ft bgs.	
Ham Defini		ficiency	Factor: 0.91		R = Rock Co			Туре:	Automa S _{II} =		Hydraulic emolded Field Vane Undrained She	Rope & Cathead \Box ear Strength (psf) T_V	= Pocket Torvane She	ar Strength (psf)
	plit Spoor Unsucces		poon Sample Atte	:	SSA = Solid HSA = Hollo	Stem Au	ger		Sulla	b) = Lat	vane Undrained Shear Strength (ned Compressive Strength (ksf)	psf) WC	= Water Content, per = Liquid Limit	
		ube Samplessful Thin W	e /all Tube Sample /		RC = Roller WOH = Weig) lb. Ha	ammer	N-unc	correcte	d = Raw Field SPT N-value ciency Factor = Rig Specific Annua		= Plastic Limit = Plasticity Index	
V = Fi	ield Vane	Shear Test	PP = Pocket Pe ane Shear Test A	enetrometer	WOR/C = W WO1P = We				N ₆₀ = N ₆₀ =	= SPT N = (Hamr	I-uncorrected Corrected for Hamme ner Efficiency Factor/60%)*N-unco	er Efficiency G = rrected C =	Grain Size Analysis Consolidation Test	
		1		Sample Infor	mation									Laboratory
	Ġ	(in.)	Sample Depth (ft.)	Û.	()	N-uncorrected				bc				Testing
(I t.)	Sample No.	Pen./Rec. (in.)	e De	Blows (/6 in.) Shear Strength	D (%	orre		5	ion	Graphic Log	Visual De	scription and Remarks	3	Results/ AASHTO
Depth (ft.)	ampl	H.ne	ampl	ows near trenc	RO	-nuc	N60	Casing Blows	Elevation (ft.)	raph				and Unified Class.
125	ű	ď	ů E		20	Ż	z	00	Ξŧ	U	113.1-114.1 ft (2:17)			
											114.1-115.1 ft (2:20) 97% Recovery			
											ļ	at 115.1 feet below gro		
											Bottom of Exploration	at 115.1 leet below gro	Junu surrace.	
- 130 -														
- 135 -														
- 140 -														
		1												
- 145 -						\rightarrow								
<u>150</u> <u>Rem</u>	l arks:	1		1				I	1		I			I
1) A	Auto Har	nmer #36	7											
0				understa de l				1 .				Dama C af C		
				undaries between				-	ne mov c	our du	a to conditions other	Page 6 of 6		
			time measuremen		numons state	a. Grou	nuwate	= nucluatio	по пау 00	Jour au	e to conditions other	Boring No	b.: BB-FFPB	-102

S.W. Cole Kevin/Brian Wilder/Pukay 8/17,22/2022 3+75.6, 6.2 ft / Factor: 0.91		Dat Rig	-	. /	Die	.9 VD88 1rich D-	Auger ID/OD: 5" Solid Stem Sampler: Standard Split Split 50 Hammer Wt./Fall: 140#/30"	boon
Kevin/Brian Wilder/Pukay 8/17,22/2022 3+75.6, 6.2 ft		Dat Rig Dril	um: Type: ling M	:	NA Die	VD88	Sampler: Standard Split Sp	oon
Wilder/Pukay 8/17,22/2022 3+75.6, 6.2 ft		Rig Dril	Type: ling M		Die			0001
8/17,22/2022 3+75.6, 6.2 ft		Dril	ling M			Inch D.		
3+75.6, 6.2 ft	Rt.	_	-	ictiliou.		ad Wash	Boring Core Barrel: N/A	
	Kt.	Ous		ν/OD·		(4.0"/4.		
Factor. 0.91		Han	nmer '		Autom		Hydraulic Rope & Cathead	
Spoon Sample Atten ole Wall Tube Sample A st, PP = Pocket Pe Vane Shear Test Att	RC = Roller WOH = Wei netrometer WOR/C = W	ore Sam I Stem A ow Stem Cone ight of 14 /eight of	ple uger Auger 40lb. Hai Rods or	mmer r Casing	S _u = S _{u(I} q _p = N-u Han N ₆₀	Peak/Re ab) = Lab Unconfir corrected mer Effic = SPT N	molded Field Vane Undrained Shear Strength (psf) T _v = Pocket Torvane Shear Vane Undrained Shear Strength (psf) WC = Water Content, percer ed Compressive Strength (ksf) LL = Liquid Limit I = Raw Field SPT N-value PL = Plastic Limit ency Factor = Rig Specific Annual Calibration Value PI = Plasticity Index uncorrected Corrected for Hammer Efficiency G = Grain Size Analysis	
		ncorrected		ing /s	ation	ohic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and
San (ft.)	Blow Shee or R(psf)	-n N	N60	Cas Blow	(ft.)	Gra	μ	nified Clas
				SSA			14" HMA. 	
9 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%
					-			
9 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=37.39
					351.4		13.5- Brown, wet, very loose, SAND, some gravel, some silt, (Stream	
5 15.00 - 17.00	2/2/1/2	3	5	HP			Alluvium).	
				11 33	-			
_				56				
				43				
) 20.00 - 22.00	14/9/2/2	11	17	45 34				
				35				
				62	340.9			
				50				
	Vane Shear Test At Image: Constraint of the second seco	Vane Shear Test Attempt WO1P = W Sample Information (i, i, j) (i, j) (i	Vane Shear Test Attempt WO1P = Weight of C Sample Information Page (1) Page (2) Page (2)	Vane Shear Test Attempt W01P = Weight of One Per Sample Information Page Page	W01P = Weight of One Person Sample Information 400 $()$	Vane Shear Test Attempt W01P = Weight of One Person Ngo Sample Information 90 00	Vane Shear Test Attempt W01P = Weight of One Person Ng0 = (Harm Sample Information 0	Sample Information Sige - Hammer Efficiency Eacord/03/11%-unoproceed C = Consolidation Test Image: Sample Information Image: Sampl

			Soil/Rock Expl			-	Locatio	161 ov	er Patt	e Brook d Maine	FPB-103
			US CUSTOMA	ARY UNITS						WIN: <u>254</u>	53.00
Drille	er:		S.W. Cole		Ele	vation	(ft.)	364.	Ð	Auger ID/OD:5" Solid Ster	ı
Oper	ator:		Kevin/Brian		Da	tum:		NAV	'D88	Sampler: Standard Spl	t Spoon
ogg	ed By:		Wilder/Pukay		Rig	у Туре		Died	rich D	50 Hammer Wt./Fall: 140#/30"	
Date	Start/Fi	nish:	8/17,22/2022		Dri	lling N	lethod:	Case	d Was	Boring Core Barrel: N/A	
Boriı	ng Loca	tion:	3+75.6, 6.2 ft	Rt.	Ca	sing ID)/OD:	HW	4.0"/4	5") Water Level*: 13.0 ft bgs.	
		ciency F	actor: 0.91			mmer	Туре:	Automa		Hydraulic 🗆 Rope & Cathead 🗆	0
/ID = J = Th /IU = / = Fie	lit Spoon Jnsuccess in Wall Tu Jnsuccess eld Vane S	sful Split Sp be Sample sful Thin Wa Shear Test,	oon Sample Atterr all Tube Sample A PP = Pocket Per une Shear Test Att	ttempt WOH = W work = WOR/C =	id Stem A llow Sterr er Cone eight of 1 Weight o	Auger n Auger 40 lb. Ha f Rods o	r Casing	S _{u(la} q _p = N-un Hami N ₆₀ :	_{o)} = Lab Jnconfii correcte ner Effic = SPT N	$ \begin{array}{llllllllllllllllllllllllllllllllllll$	ercent
		1		Sample Information	-	1	1				Laborator
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	Testing Results/ AASHTO and Unified Cla
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				
									61 2 5 8 1		
							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		e ge		
							47				
							69	331.9			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 Til). G#33751: A-1-b, SN
35 -							91				WC=11.7
							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 _	arks:										1

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.	Page 2 of 3
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.	Boring No.: BB-FFPB-103

1	Maine	e Depa	artment	of Transpor	tation		Project:	Puddl	e Dock	Bridge #2691 carries Route	Boring No.:	BB-FF	PB-103
			Soil/Rock Expl	oration Log			Location			ee Brook Id, Maine			
		<u> </u>	JS CUSTOMA	ARY UNITS			Location	1. Pon	Panne	iu, mane	WIN:	2545	53.00
Drill	er:		S.W. Cole		Eleva	tion	(ft.)	364.	9		Auger ID/OD:	5" Solid Stem	
Ope	rator:		Kevin/Brian		Datun	n:		NA	/D88		Sampler:	Standard Split	Spoon
Log	ged By:		Wilder/Pukay		Rig T	ype:	:	Died	lrich D	-50	Hammer Wt./Fall:	140#/30"	-
	Start/Fi	nish:	8/17,22/2022				lethod:	Case	d Was	h Boring	Core Barrel:	N/A	
Bori	ng Loca	tion:	3+75.6, 6.2 ft	Rt.	Casin	-			(4.0"/4	-	Water Level*:	13.0 ft bgs.	
			actor: 0.91		Hamn	-		Autom		Hydraulic 🗆	Rope & Cathead □		
Defini D = S MD = U = T MU = V = Fi	tions: plit Spoon \$ Unsuccess hin Wall Tu Unsuccess jeld Vane S	Sample ful Split Spo be Sample ful Thin Wa hear Test,	oon Sample Attern II Tube Sample At PP = Pocket Per ne Shear Test Att	SSA = So ppt HSA = Ho RC = Rol ttempt WOH = V netrometer WOR/C =	Core Sample blid Stem Auge blow Stem Au ler Cone Veight of 140 I = Weight of Ro Weight of One	er iger b. Ha	ammer r Casing	S _u = S _{u(la} q _p = N-un Ham N ₆₀	Peak/Re b) = Lab Unconfin correcte mer Effic = SPT N	emolded Field Vane Undrained Sh v Vane Undrained Shear Strength (hed Compressive Strength (ksl) d = Raw Field SPT N-value ciency Factor = Rig Specific Annua -uncorrected Corrected for Hamm ner Efficiency Factor/60%)*N-unco	ear Strength (psf) T _V = (psf) WC LL = PL = I Calibration Value PI = er Efficiency G =	Pocket Torvane She = Water Content, per Liquid Limit = Plastic Limit Plasticity Index Grain Size Analysis Consolidation Test	
		Ŷ			1				1				Laboratory Testing
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)	Graphic Log	Visual De	escription and Remarks		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			Occasional Cobble.			
- 55 -	105	0.175	55 00 55 0	17/10/00/110		.	116			^a 57 blows for 0.5 ft.			G#337516
	10D	24/15	55.00 - 57.00	17/19/30/49	49	74	a57 OPEN HOLE			Brown, wet, very dense, G	ravelly SAND, trace silt,	(Glacial Till).	A-1-b, SW-SM WC=8.1%
							HOLE						
- 60 -									 				
- 65 -										Drown wat your dance S	AND little energy little a	ilt (Clasici Till)	
	11D	24/18	65.00 - 67.00	22/31/34/39	65	99				Brown, wet, very dense, SA	AND, Inthe gravel, inthe s	ilit, (Glacial Till).	
								297.9				67.0	
								291.9		Bottom of Exploration Hole Abandoned. 10.0 ft of	n at 67.0 feet below grou of broken HW(4") casing	ind surface.	
										from 57. 0 ft bgs (El. 307.9) to 67.0 ft bgs (El. 297.9).	
- 70 -													
 	arks:												
1) A 2) 1	Auto Ham 0.0 ft of t			andoned in hole from	57.0 ft bgs (El. 3	607.9) to 6	i7.0 ft b	gs (El.2	297.9).			
Stratif	ication lines	s represent	approximate bour	ndaries between soil type	s; transitions n	nay b	e gradual.				Page 3 of 3		
			been made at tim	es and under conditions s s were made.	tated. Ground	dwate	er fluctuation	ns may c	ccur due	e to conditions other	Boring No	: BB-FFPB	-103
uncil													100

Ι	Main	e Dep	artment	of Transport	ation		Projec				Bridge #2691 carries Route	Boring No.:	BB-FFF	PB-103A
			Soil/Rock Exp	-			Locati				ee Brook ld, Maine		05.4	a 00
			US CUSTOM	IARY UNITS								WIN:	2545	53.00
Drill	er:		S.W. Cole		Eleva	atior	n (ft.)	3	364.8			Auger ID/OD:	5" Solid Stem	
Ope	rator:		Kevin/Brian		Datu	m:		l	NAV	D88		Sampler:	Standard Split	Spoon
	ged By:		Wilder/Pukay		Rig T					rich D-		Hammer Wt./Fall:	140#/30"	
	Start/F		8/22/2022-8/2		_	-	lethod:				n Boring	Core Barrel:	NQ-2"	
	ng Loca		3+75.4, 9.2 ft	t Rt.	_	-	D/OD:			4.0"/4.		Water Level*:	13.0 ft bgs.	
Defini		iciency	Factor: 0.91	R = Rock C	Core Sample	е	Туре:	5	tomat S _u = F	Peak/Re	Hydraulic molded Field Vane Undrained She	Rope & Cathead ear Strength (psf) T _v	= Pocket Torvane She	ar Strength (psf)
	plit Spoon Unsucces:		ooon Sample Atte	mpt SSA = Soli HSA = Holl				c	q _p = L	Inconfir	Vane Undrained Shear Strength (ed Compressive Strength (ksf)	LL	C = Water Content, per = Liquid Limit	cent
		ube Sample sful Thin W	all Tube Sample /	Attempt RC = Rolle		lb. Ha	ammer	H	Hamm	er Effic	d = Raw Field SPT N-value iency Factor = Rig Specific Annua	I Calibration Value PI	= Plastic Limit = Plasticity Index	
			PP = Pocket Pe ane Shear Test A					1 1	N ₆₀ = N ₆₀ =	SPT N- (Hamm	-uncorrected Corrected for Hamme ner Efficiency Factor/60%)*N-unco	er Efficiency G rrected C	= Grain Size Analysis = Consolidation Test	
		1		Sample Information										Laboratory
		Pen./Rec. (in.)	Sample Depth (ft.)	(·u (9	N-uncorrected					Log				Testing Results/
Depth (ft.)	Sample No.	kec.	e De	Blows (/6 in.) Shear Strength (psf) or RQD (%)	orre		5	.u		ic Lo	Visual De	scription and Remark	s	AASHTO
epth	amp	en./F	ampl	ows near trenç sf) · RQ	-nuc	N ₆₀	Casing Blows	evat	(ft.)	Graphic				and Unified Class.
0	, М	ď	ů E	ස්ග්හිලිවි	ż	ž	Ű		£	Ċ	14" HMA.			
÷							SSA				14 IIWIA.			
								36	63.6		Reference BB-FFPB-103 f	or samples up to and inc		

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- 20 -							27							
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							35	_						
							42		,					
							71		•	89 a 189				
							152	1						
25 Rem	arks:						152			194193				
		nmer #367	7											
		Iraulic Pu												
04	iooti "		topprovide t	undering between 194	trongiti							Page 4 of 5		
				undaries between soil types;						ourd	to conditions other	Page 1 of 5		
			been made at tir time measuremer	nes and under conditions sta nts were made.	ileu. Groun	uwat	er nuctuat	uuns m	iay OC	cur düê		Boring No	o.: BB-FFPB	-103A

N	Aaine	-	Soil/Rock Exp		atior	n		1	161 ov	/er	ock Bridge #2 Pattee Brook irfield, Maine	691 carries Route	Boring No.: WIN:		PB-103A 53.00
Drille	er:		S.W. Cole		Ele	vation	(ft.)		364.	8			Auger ID/OD:	5" Solid Stem	
	ator:		Kevin/Brian			tum:	,		NAV		88		Sampler:	Standard Split	Spoon
	ed By:		Wilder/Pukay	r	Rig	Туре					h D-50		Hammer Wt./Fall:	140#/30"	1
	Start/Fi	nish:	8/22/2022-8/2		_	lling N		d:	Case	ed V	Wash Boring		Core Barrel:	NQ-2"	
Borir	ng Loca	tion:	3+75.4, 9.2 ft	Rt.	_	sing ID)"/4.5")		Water Level*:	13.0 ft bgs.	
lamı	mer Effi	ciency l	Factor: 0.91		Har	mmer	Туре	: A	utoma	atic	⊠ H	ydraulic 🗆	Rope & Cathead □		
MD = l J = Th MU = l / = Fie	olit Spoon S Unsuccess in Wall Tu Unsuccess old Vane S	ful Split Sp be Sample ful Thin W hear Test,	all Tube Sample A PP = Pocket Pe ane Shear Test At	RC = Rolle Attempt WOH = We enetrometer WOR/C = W ttempt WO1P = W	d Stem A low Stem or Cone eight of 14 Weight of	Auger Auger 40 lb. Ha	r Casin		S _{u(la} q _p = N-un Hami N ₆₀ :	b) = Unc corr mer = SI	E Lab Vane Undr confined Compre rected = Raw Fie Efficiency Facto PT N-uncorrecte	eld Vane Undrained S rained Shear Strength essive Strength (ksf) eld SPT N-value or = Rig Specific Annu d Corrected for Hamr cy Factor/60%)*N-unc	n (psf) Wo LL PL ual Calibration Value PI ner Efficiency G	= Pocket Torvane She C = Water Content, per = Liquid Limit = Plastic Limit = Plasticity Index = Grain Size Analysis = Consolidation Test	
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected		ing	'S	Elevation (ft.)		Graphic Log	Visual D	escription and Remark	S	Laborator Testing Results/ AASHTO and
80 25	Sam	Pen	Sam (ft.)	Blow She Stre (psf	n-N	N ₆₀	Casing		(ft.)	ទ្ប	nn Q <u>a</u>				Unified Cla
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ł							30	6			8 69 6 6 8 8 8 8 8 8 8 8 8 8 8 8 8 8 8 8				
30							78	8		000					
50							52	2		900					
							40	_		6					
							63 74		331.8					33.0	F
							98	_							
35 -							47	7							
							68	8		. 1					
							70	0							
							11	_							
40							90 OPI	_							
							НО			• X•					
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Rema 1) A		mer #367 raulic Pus								<u></u>					
tratific	cation lines	s represen	t approximate bou	indaries between soil types;	transition	ns may b	e grad	ual.					Page 2 of 5		
			e been made at tim time measuremen	nes and under conditions stants were made.	ated. Gro	oundwate	er fluctu	uations	may o	сси	r due to conditio	ns other		o.: BB-FFPB	-103A

1	Main	e Dep	artment	of Transporta	ation	L	Project			ck Bridge #2691 carries Route	Boring No.:	BB-FFP	B-103A
			Soil/Rock Exp				Locatio			attee Brook field, Maine		2545	2 00
			US CUSTOM	ARY UNITS							WIN:	2545	53.00
Drill	er:		S.W. Cole		Elev	ation	(ft.)	36	4.8		Auger ID/OD:	5" Solid Stem	
Оре	rator:		Kevin/Brian		Datu	ım:		NA	VD8	3	Sampler:	Standard Split	Spoon
Log	ged By:		Wilder/Pukay	/	Rig	Туре		Di	edrich	D-50	Hammer Wt./Fall:	140#/30"	
	Start/F		8/22/2022-8/2		_	-	ethod:			ash Boring	Core Barrel:	NQ-2"	
	ng Loca		3+75.4, 9.2 ft	Rt.	_	ing ID				/4.5")	Water Level*:	13.0 ft bgs.	
Ham Defini		iciency	Factor: 0.91	R = Rock C			Туре:	Autor		Hydraulic /Remolded Field Vane Undrained She	Rope & Cathead	= Pocket Torvane Shea	ar Strength (psf)
D = S MD = U = T MU = V = F	plit Spoon Unsucces hin Wall Tu Unsucces jeld Vane S	sful Split Sp ube Sample sful Thin W Shear Test,	poon Sample Atte e /all Tube Sample / PP = Pocket Pe ane Shear Test A	SSA = Solid mpt HSA = Hollc RC = Roller Attempt WOH = Wei enetrometer WOR/C = W ttempt WO1P = Wei	Stem Au ow Stem A Cone ght of 140 /eight of F	iger Auger D Ib. Ha Rods or	Casing	S _{u(} qp N-u Hai N ₆ i	lab) = = Unco incorre mmer E) = SP	Lab Vane Undrained Shear Strength (nfined Compressive Strength (ksf) cted = Raw Field SPT N-value fifciency Factor = Rig Specific Annua r N-uncorrected Corrected for Hamme <u>mmere Efficiency Factor/60%)'N-unco</u>	psf) WC LL = PL = I Calibration Value PI = er Efficiency G =	= Water Content, perc = Liquid Limit = Plastic Limit • Plasticity Index Grain Size Analysis Consolidation Test	
		<u> </u>		Sample Information	σ			1	-				Laboratory
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation	Granhie Log	visual De	scription and Remarks	;	Testing Results/ AASHTO and Unified Class.
50								-					
- 55 -								-					
								-					
- 60 -								_					
- 65 -								-					
								-					
- 70 -								-					
									9.				
		1			-+			1					
75 Rem	arks:												
1) A 2) H	Auto Ham HP = Hyd	nmer #36′ Iraulic Pu	sh								Demo 2 - 4 5		
				undaries between soil types; t							Page 3 of 5		
			e been made at tin time measuremer	nes and under conditions stat hts were made.	ed. Grou	ndwate	er fluctuati	ons may	occur	due to conditions other	Boring No	.: BB-FFPB	-103A

N	/laine	e Depa	artment	of Transporta	ation	L	Proj	ect:			Bridge #2691 carries Route	Boring No.:	BB-FFF	PB-103A
			Soil/Rock Expl				Loca	atio			ee Brook ld, Maine			
		<u>l</u>	JS CUSTOMA	RY UNITS			LUUG	atiol		. anne	a, mane	WIN:	2545	53.00
Drille	er:		S.W. Cole		Elev	ation	(ft.)		364.8	3		Auger ID/OD:	5" Solid Stem	
Oper	ator:		Kevin/Brian		Datu	ım:			NAV	D88		Sampler:	Standard Split	Spoon
Logg	ed By:		Wilder/Pukay		Rig	Туре	:		Died	rich D-	50	Hammer Wt./Fall:	140#/30"	-
	Start/Fi	nish:	8/22/2022-8/23	3/2022	-	ing N		d:	Case	d Wast	Boring	Core Barrel:	NQ-2"	
	ng Loca		3+75.4, 9.2 ft I		-	ing ID				4.0"/4.	0	Water Level*:	13.0 ft bgs.	
	-				-	mer							15.0 ft 0gs.	
Definiti		ciency F	actor: 0.91	R = Rock Co			Type	•	Automa Su =		Hydraulic molded Field Vane Undrained She	Rope & Cathead \Box ear Strength (psf) $T_{y} =$	Pocket Torvane She	ar Strength (psf)
MD = l U = Th MU = l	in Wall Tu Jnsuccess	ful Split Spo be Sample ful Thin Wa	oon Sample Attem Il Tube Sample At	sSA = Solid pt HSA = Hollo RC = Roller tempt WOH = Wei	Stem Au ow Stem A Cone ght of 140	iger Auger 0 lb. Ha			S _{u(lal} q _p = l N-uno Hamr) = Lab Jnconfir orrected ner Effic	Vane Undrained Shear Strength (ned Compressive Strength (ksf) d = Raw Field SPT N-value iency Factor = Rig Specific Annual	psf) WC = LL = PL = I Calibration Value PI = I	Water Content, pero Liquid Limit Plastic Limit Plasticity Index	
			PP = Pocket Per ne Shear Test Atte					ng			-uncorrected Corrected for Hamme ner Efficiency Factor/60%)*N-unco		Grain Size Analysis Consolidation Test	
			S	ample Information										Laboratory
		Û.	Ę	$\widehat{}$	eq					_				Laboratory Testing
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing	Blows	Elevation (ft.)	Graphic Log	Visual De	scription and Remarks		Results/ AASHTO and Unified Class.
75	1D	24/18	75.00 - 77.00	12/13/22/32	35	53					Grey, wet, hard, SILT, som	e sand, little gravel, (Gla	cial Till).	
· 80 -											ARTESIAN water pressure	at 82.0-85.0 ft bgs.		
. 85 -														
										2				
90 -	2D	24/15	90.00 - 92.00	17/18/22/34	40	61					Grey, wet, very dense, GRA	AVEL, some sand, some s	ilt, (Glacial Till).	
95			05.00						269.8				95.0-	
.100	R1	60/58	95.00 - 100.00	RQD = 82%				2-2			Top of Bedrock at Elev. 26 R1: Bedrock: Grey to dark SILTSTONE, moderately h angles, spaced moderately of planes, some quartz or calc [Spragueville Formation] Rock Quality = Good. R1: Core Times (min:sec) 95.0-96.0 ft (3:01) 96.0-97.0 ft (3:04)	greenish-grey, fine-graine nard, fresh, joints dip at lo close, with rock flour evid	d, thin-bedded, w to moderate	
Rema	arks:													
1) A	uto Ham	mer #367 aulic Pusl	1											
Stratif	nation line	reprocent	approvimate have	daries between coll times t	rancition	mour	o ara-	luol				Page 4 of 5		
		-		daries between soil types; tr		-	-					Faye 4 OF 5		
		-	been made at time me measurement	es and under conditions stat s were made.	ed. Grou	Indwate	er fluct	uatio	ns may oo	cur due	to conditions other	Boring No.	: BB-FFPB	-103A

I	Main	e Dep	artment	of Transpor	tation	Project			Bridge #2691 carries Route	Boring No.:	BB-FFF	PB-103A
			Soil/Rock Exp			Locatio			ee Brook ld, Maine		0.5.4	
			US CUSTOM	<u>ARY UNITS</u>						WIN:	2545	53.00
Drill	er:		S.W. Cole		Elevatio	on (ft.)	364	.8		Auger ID/OD:	5" Solid Stem	
Оре	rator:		Kevin/Brian		Datum:		NA	VD88		Sampler:	Standard Split	Spoon
Log	ged By:		Wilder/Pukay	7	Rig Typ	e:	Die	drich D	-50	Hammer Wt./Fall:	140#/30"	
	Start/F		8/22/2022-8/2			Method:			h Boring	Core Barrel:	NQ-2"	
	ng Loca		3+75.4, 9.2 ft	Rt.	Casing			/(4.0"/4.	,	Water Level*:	13.0 ft bgs.	
Defini	tions:		factor: 0.91		Core Sample	r Type:	Autom Su =		Hydraulic emolded Field Vane Undrained Sho	Rope & Cathead ear Strength (psf) T _v =	= Pocket Torvane She	ar Strength (psf)
	plit Spoon Unsucces		oon Sample Atter		olid Stem Auger	r			Vane Undrained Shear Strength (ned Compressive Strength (ksf)		= Water Content, per Liquid Limit	cent
		ube Sample sful Thin W	all Tube Sample A	RC = Rol Attempt WOH = V	ler Cone Veight of 140 lb.	Hammer	N-u	ncorrecte	d = Raw Field SPT N-value siency Factor = Rig Specific Annua		= Plastic Limit Plasticity Index	
V = Fi	eld Vane S	Shear Test,	PP = Pocket Pe ane Shear Test At	enetrometer WOR/C =	 Weight of Rods Weight of One P 				-uncorrected Corrected for Hamme ner Efficiency Factor/60%)*N-unco		Grain Size Analysis Consolidation Test	
		1		Sample Information	N		-					Laboratory
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	Casing Blows	Elevation (ft.)	Graphic Log	Visual De	scription and Remarks	;	Testing Results/ AASHTO and
	Sar	Per	Sar (ft.)	She Stre (pst or F	NeO N	Blo as B	(ft.)	Gra				Unified Class.
100	R2	36/36	100.00 - 103.00	RQD = 28%				90	97.0-98.0 ft (3:00) 98.0-99.0 ft (3:01)			
			100.00				1		99.0-100.0 ft (3:03) 97% Recovery			
					<u> </u>		-			anaanish C	ad this 1-31 1	
			102.00				-		R2: Bedrock: Grey to dark SILTSTONE, moderately h	hard, fresh, joints are more	derately dipping	
	R3	24/24	103.00 - 105.00	RQD = 88%				1990	and fresh with minor iron of run contains rock flour on f		one near middle of	
								<i>M</i>	[Spragueville Formation]	F		
- 105 -						V	259.8	erne	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			
							1		R3: Bedrock: Grey to dark SILTSTONE, moderately h			
							-		through most of the run, fra		FF8	
- 110 -							-		[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			
							1		J	at 105 0 faat balaw and		
							-		Bottom of Exploration	at 105.0 feet below gro	und surface.	
							-					
- 115 -												
- 115 -												
					$\left \right $		-					
							-					
- 120 -		1					1					
					$\left \right $		-					
		<u> </u>				_	4					
		1					1					
125 Rem	arks:											
		nmer #367										
		Iraulic Pus										
Stratif	ication line	es represent	approximate bou	Indaries between soil types	s; transitions may	/ be gradual.				Page 5 of 5		
* Wat	er level rea	adings have	been made at tin	nes and under conditions s	tated. Groundw	ater fluctuati	ons may	occur due	e to conditions other	Dent M		102.4
than	those pre	sent at the	time measuremer	nts were made.						Boring No	BB-FFPB	-103A

<u>Appendix B</u>

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.

<u>Appendix C</u>

Laboratory Test Results

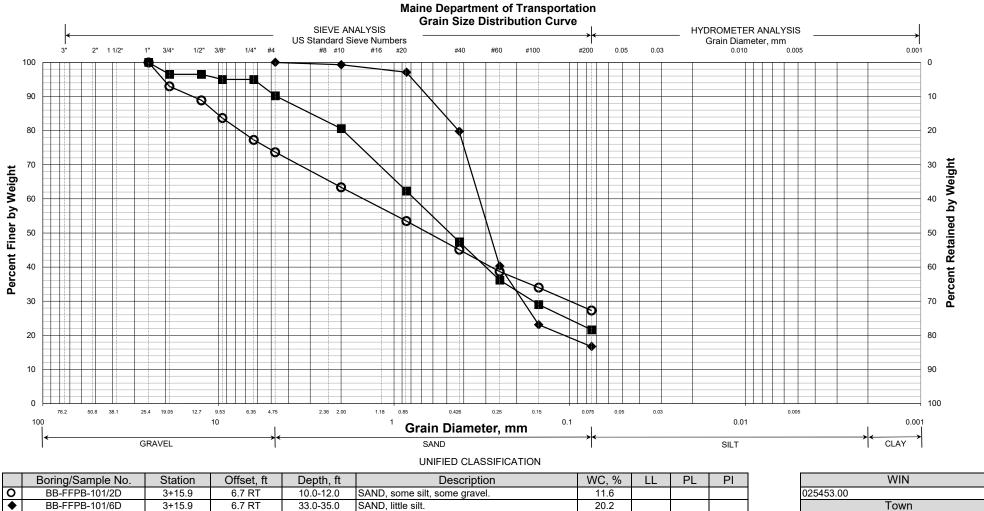
State of Maine - Department of Transportation Laboratory Testing Summary Sheet

Town(s):	Fort F	-	eld				ımt	ber	: 254	53.00	
Boring & Sample	Station	Offset	Depth	Reference	G.S.D.C.		L.L.	P.I.	-	ssificatio	
Identification Number	(Feet)	(Feet)	(Feet)	Number	Sheet	%			Unified	AASHTO	
BB-FFPB-101, 2D	3+15.9	6.7 Rt.	10.0-12.0	241522	1	11.6			SM	A-2-4	11
BB-FFPB-101, 6D	3+15.9	6.7 Rt.	33.0-35.0	337506	1	20.2			SM	A-2-4	
BB-FFPB-101, 8D	3+15.9	6.7 Rt.	40.0-42.0	337507	1	12.7			SM	A-1-b	
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	
BB-FFPB-102, 9D	3+00.6	6.6 Lt.	45.0-47.0	337512	2	13.7			SM	A-2-4	
BB-FFPB-103, 1D	3+75.6	6.2 Rt.	5.0-7.0	337513	3	14.5			SM	A-2-4	
BB-FFPB-103, 2D	3+75.6	6.2 Rt.	10.0-12.0	337514	3	37.3			SM	A-4	
BB-FFPB-103, 6D	3+75.6	6.2 Rt.	34.0-36.0	337515	3	11.7			SM	A-1-b	
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 th is followed by the '	'Frost Suscep	tibility Rat	ing" from zero	o (non-frost s	usceptible	e) to Cla	ass IV	(highl	ly frost su	sceptible)	
The "Frost Sus											
SDC = Grain Size Distributer SSDC = water content as deter			-				122-03	(пеар	pioved 19	90)	

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



12.7

SAND, some silt, trace gravel.

• • ×

BB-FFPB-101/8D

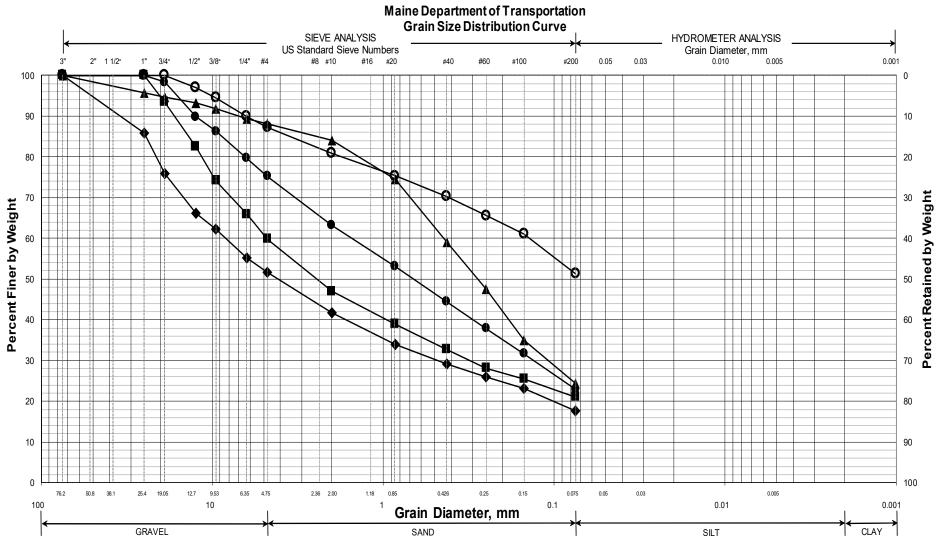
3+15.9

6.7 RT

40.0-42.0

025453.00	
To	own
Fort Fairfield	
Reporte	d by/Date
WHITE, TERRY A	10/31/2022

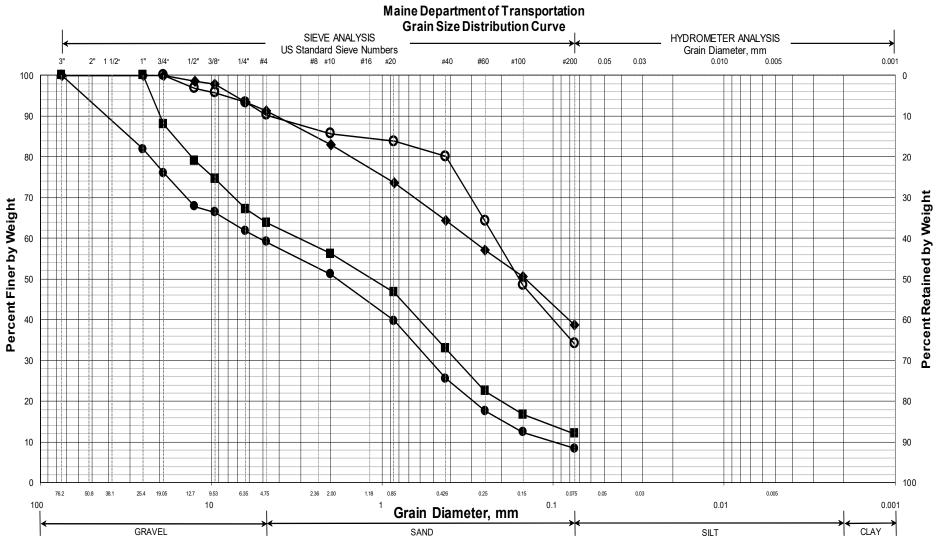
SHEET 1



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
0	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			
×									

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



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
0	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			
\bullet	BB-FFPB-103/10D	3+75.6	6.2 RT	55.0-57.0	Gravelly SAND, trace silt.	8.1			
×									

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

SHEET 3

<u>Appendix D</u>

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

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

Use the following piles: 14x89, 14x117

(34.4) (14.2) (14.9) (0.805)	$A_g := \binom{26.1}{34.4} \cdot in^2$	$\mathbf{d} := \begin{pmatrix} 13.8\\ 14.2 \end{pmatrix} \cdot \mathbf{i} \mathbf{n}$	$\mathbf{b} := \begin{pmatrix} 14.7\\ 14.9 \end{pmatrix} \cdot \mathbf{in}$	$t_{f} := \begin{pmatrix} 0.615\\ 0.805 \end{pmatrix} in$	$t_w := t_f$
------------------------------	--	---	---	---	--------------

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

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

Pile yield strength

r_s= radius of gyration

 $F_y := 50 \cdot ksi$

 $\mathbf{r}_{s} := \begin{pmatrix} 3.53 \\ 3.59 \end{pmatrix} \cdot \mathbf{in}$

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

E = Elastic Modulus E := 29000 · 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

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

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 $\lambda_{rf} = 13.487$ $b_f := 0.5 \cdot b$ $b_f = \begin{pmatrix} 7.35 \\ 7.45 \end{pmatrix} \cdot in$ $\frac{b_f}{t_f} = \begin{pmatrix} 11.951 \\ 9.255 \end{pmatrix}$ Both H-pile sizes are nonslender for flange membersFor webs: $\lambda_{rw} := 1.09 \sqrt{\frac{E}{F_y}}$ where $b_w =$ Web height/distance between flanges $\lambda_{rw} = 26.251$ $b_w := d - 2 \cdot t_f$ $b_w = \begin{pmatrix} 12.57 \\ 12.59 \end{pmatrix} \cdot in$ $\frac{b_w}{t_w} = \begin{pmatrix} 20.439 \\ 15.64 \end{pmatrix}$ Both H-Pile sizes are nonslender for web members

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

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

Nominal Axial Structural Resistance

Determine equivalent yield resistance $P_o := F_y \cdot A_g$ 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 Pe, LRFD eq. 6.9.4.1.2-1

K = effective length factor

 $K_{eff} := 0.65$

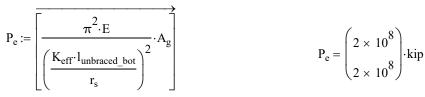
LRFD Table C4.6.2.5-1. Use K=0.65 for assumed segment in pure compression. Fixed top and bottom

I = "unbraced" length

 $l_{unbraced\ bot}\coloneqq 0.1\!\cdot\!ft$

Assume in pure compression

LRFD eq. 6.9.4.1.2-1



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:
$$P_{n} := \begin{pmatrix} \frac{P_{o}}{P_{e}} \\ 0.658 \\ P_{o} \end{pmatrix}$$
 LRFD Eq. 6.9.4.1.1-1

then:

this applies to all pile sizes

л	(1305)	1.
$P_n =$	(1720)	•k1p

Factored Axial Structural Resistance for the Strength Limit State

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

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

D	(652)	1.1.
$P_r =$	860	·кıр

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

$$\phi_{c} := 0.5$$
$$P_{r} := \phi_{c} \cdot P_{n}$$



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
 $P_{r_{e}ee} := \phi_{c} \cdot P_{n}$
 $P_{r_{e}ee} = \begin{pmatrix} 1305\\ 1720 \end{pmatrix} \cdot kip$ 14x89
14x117

Drivability Analyses

Ref: LRFD Article 10.7.8

For steel piles in compression or tension, driving stresses are limited to 90% of $f_{\rm v}$

φ _{da} := 1.0	Resistance factor from LRFD Table 10.5.5.2.3-1, Drivablity Analysis, steel piles
$\sigma_{dr} \coloneqq 0.90 \cdot 50 \cdot (ksi)$	$\cdot \phi_{da}$

 $\sigma_{dr} = 45 \cdot ksi$ 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 drivability 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.

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

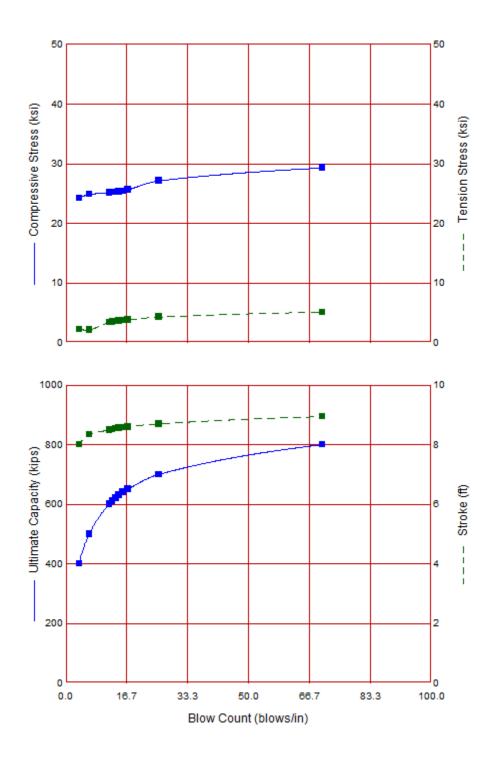
Use constant shaft resistances so that GRLWeap will assign approx. 250 kips as skin friction based on local experience in similar deposits.

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
Pile Model	Skin Friction Distribution

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



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

19-Apr-2024 GRLWEAP Version 2010

Iltimate apacity 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
0.008	29.29	5.09	70.2	8.94	23.63

Limit to 15 bpi

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

Strength Limit State

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

 $R_{fdr} = 409 \cdot kip$

Extreme and Service Limit States

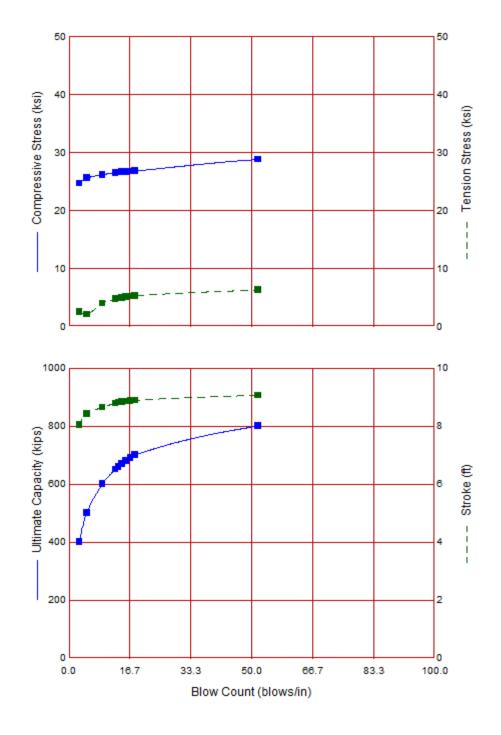
 $R_{dr} := R_{ndr} \cdot \varphi$

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	
Effic	n Weight iency soure	5.51 kips 0.800 1425 (100%) psi
Harr	net Weight nmer Cushion R of H.C.	3.00 kips 34825 kips/in 0.800
Toe Skin	Quake Quake Damping Damping	0.100 in 0.070 in 0.050 sec/ft 0.150 sec/ft
Pile	Length Penetration Top Area	100.00 ft 94.90 ft 26.10 in2
	Pile Model	Skin Friction Distribution

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



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

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

R_{fdr} = 436⋅kip

 $\begin{array}{l} \mbox{Extreme and} \\ \mbox{Service Limit States} \\ R_{dr} \coloneqq R_{ndr} \cdot \varphi \end{array}$

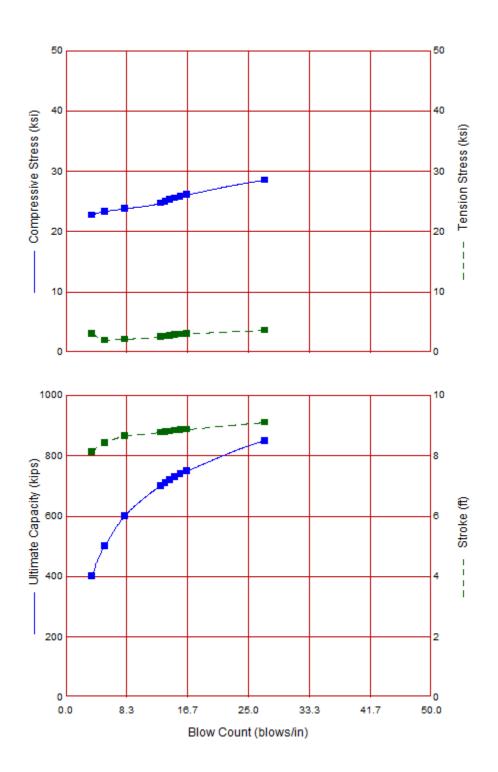
 $R_{dr} = 670 \cdot kip$

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:



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



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

 $R_{fdr} \coloneqq R_{ndr} \cdot \varphi_{dyn}$

 $R_{fdr} = 474 \cdot kip$

Extreme and Service Limit States $R_{dr} \coloneqq R_{ndr} \cdot \varphi$

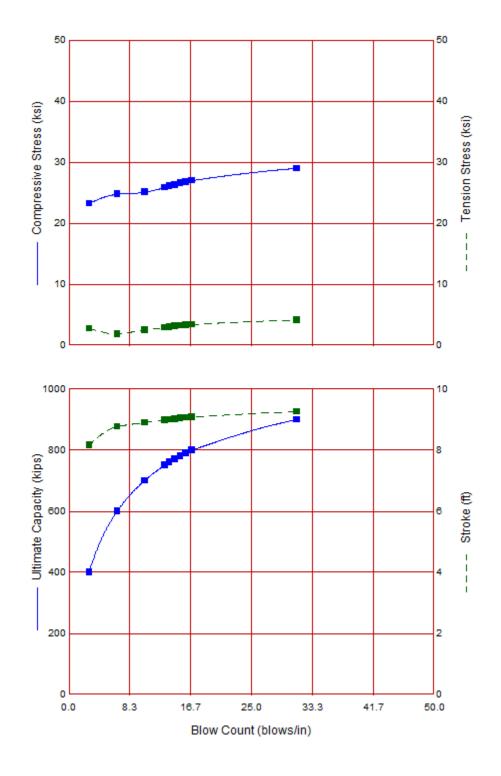
R_{dr} = 730∙kip

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 1425 (100%) psi Pressure 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 Skin Friction Distribution Pile Model

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



Maine DOT 25453 Fort Fairfield 14x117 ABT#1 D25-42

19-Apr-2024 GRLWEAP Version 2010

	imate pacity 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
– E	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
	0.008	27.01	3.43	16.9	9.08	26.45
1	900.0	29.01	4.19	31.2	9.26	27.08

Limit to 15 bpi

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

Strength Limit State

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

R_{fdr} = 501 · kip

Extreme and Service Limit States $R_{dr} := R_{ndr} \cdot \varphi$

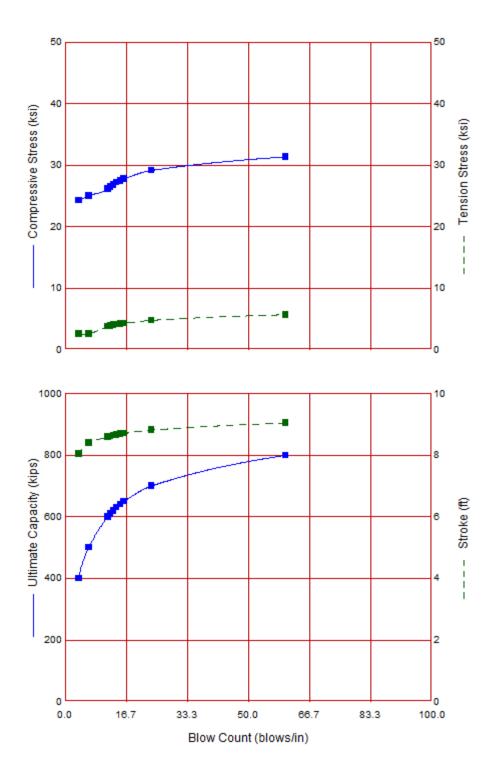
R_{dr} = 770 ⋅ kip

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

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

APE D 19-42 Ram Weight Efficiency Pressure Helmet Weight Hammer Cushion COR of H.C. Skin Quake Toe Quake Skin Damping Toe Damping Pile Length Pile Penetration Pile Top Area		4.19 kips 0.800 1710 (100%) psi 3.00 kips 34825 kips/in 0.800 0.100 in 0.070 in 0.050 sec/ft 0.150 sec/ft 95.00 ft 85.40 ft 26.10 in2		
F	'ile Model	Skin Fr Distrib		

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



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
0.008	31.36	5.71	60.1	9.05	24.03

Limit to 15 bpi

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

Strength Limit State

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

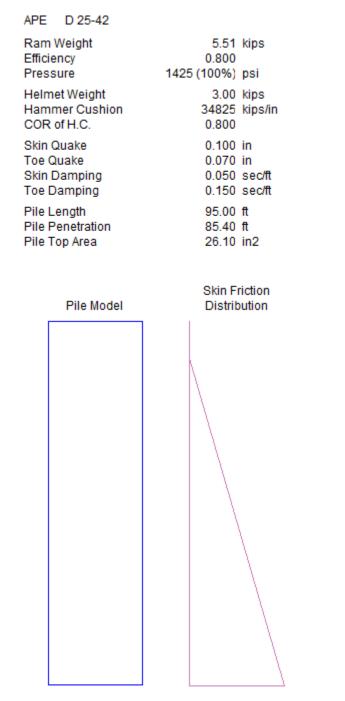
R_{fdr} = 409·kip

Extreme and Service Limit States $R_{dr} \coloneqq R_{ndr} \cdot \varphi$

R_{dr} = 630·kip

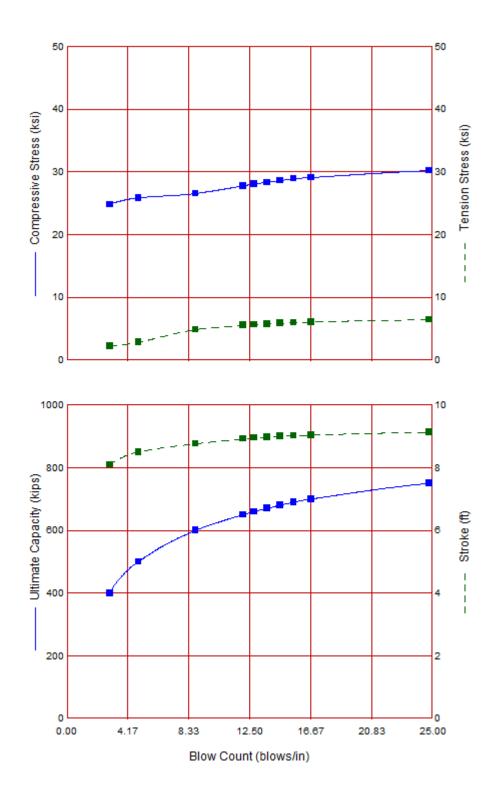
Abutment 2, Pile Size is 14 x 89, APE D25-42 Hammer

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:



Res. Shaft = 248.0 kips (Constant Res. Shaft) 21 of 29

(constant too, onary



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

 $R_{fdr} \coloneqq R_{ndr} \cdot \varphi_{dyn}$

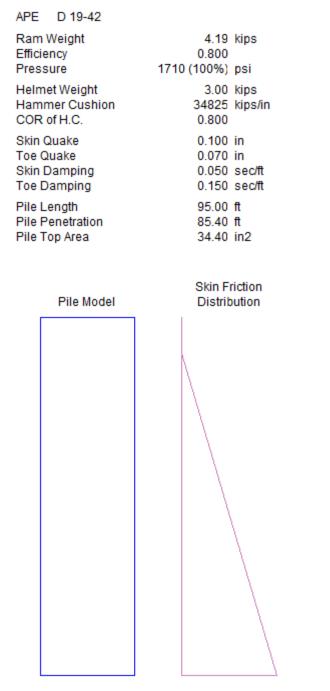
$$R_{fdr} = 442 \cdot kip$$

Extreme and Service Limit States $R_{dr} \coloneqq R_{ndr} \cdot \varphi$

 $R_{dr} = 680 \cdot kip$

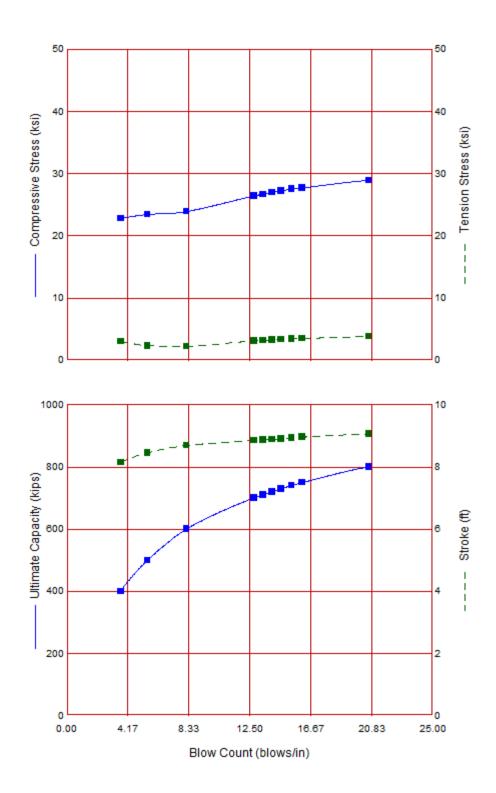
Abutment 2, Pile Size is 14 x 117, APE D19-42 Hammer

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:



Res. Shaft = 248.0 kips (Constant Res. Shaft) 24 of 29

(constant too, onary



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

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

R_{fdr} = 474 · kip

Extreme and Service Limit States $R_{dr} \coloneqq R_{ndr} \cdot \varphi$

R_{dr} = 730·kip

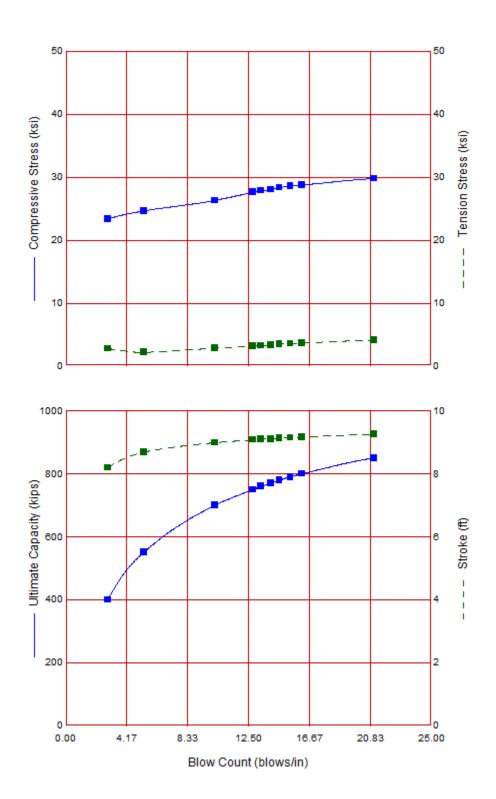
Abutment 2, Pile Size is 14 x 117, APE D25-42 Hammer

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 Efficiency Pressure Helmet Weight Hammer Cushion COR of H.C. Skin Quake Toe Quake Skin Damping Toe Damping Pile Length Pile Penetration Pile Top Area	5.51 kips 0.800 1425 (100%) psi 3.00 kips 34825 kips/in 0.800 0.100 in 0.070 in 0.050 sec/ft 0.150 sec/ft 95.00 ft 85.40 ft 34.40 in2
Pile Model	Skin Friction Distribution

Res. Shaft = 248.0 kips 27 of 29

(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

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

 $R_{fdr} = 507 \cdot kip$

Extreme and Service Limit States

 $R_{dr} \coloneqq R_{ndr} \cdot \varphi$

R_{dr} = 780·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
	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
Abutment #1 14x89	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
APE D19-42	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
	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
Abutment #1 14x89	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
APE D25-42	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
	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
Abutment #1 14x117	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
APE D19-42	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
	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
Abutment #1 14x117	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
APE D25-42	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
	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
Abutment #2 14x89	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
APE D19-42	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
	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
Abutment #2 14x89	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
APE D25-42	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
	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
Abutment #2 14x117	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
APE D19-42	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
	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
Abutment #2 14x117	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
APE D25-42	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 Infor	mation	
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		

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

Quartzmonzonite

Schist

Syenite

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

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

⁽²⁾Not including oil shale.

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

(4.4.8.2.2-2)

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):

$$E_{\rm m} = \alpha_{\rm E} E_{\rm o}$$
 (4.4.8.2.2-3)

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

19,000-23,000

1,400-21,000

26,000-62,000

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 .

2,700-3,300

3,800- 9,000

200-3,000

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

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Earth Pressure

Earth Pressure:

Backfill engineering strength parameters

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

Unit weight	$\gamma_1 \coloneqq 125 \cdot \text{pcf}$
Internal friction angle	$\varphi' \coloneqq 32 \cdot \text{deg}$
Cohesion	$c_1 := 0 \cdot psf$

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

 $\begin{aligned} \text{Rise}_{\text{ABT1}} &\coloneqq 1.5\text{ft} & \text{Rise}_{\text{ABT2}} &\coloneqq -0.6\text{ft} & \text{Run} &\coloneqq 25\text{ft} \\ \alpha_{\text{ABT1}} &\coloneqq \text{atan} \left(\frac{\text{Rise}_{\text{ABT1}}}{\text{Run}} \right) &= 3.43 \text{ deg} & \text{Abutment No. 1} \end{aligned}$

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

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

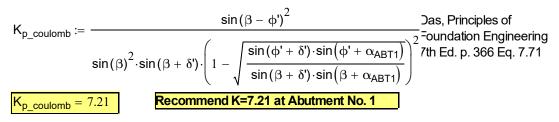
α_{ABT1} = Angle of fill slope to the horizontal at Abutment No. 1	$\alpha_{ABT1}=3.43\text{deg}$
$\varphi_1 = Angle of internal friction$	$\phi' = 32 \cdot \text{deg}$
β = Angle of back face of wall to the horizontal	$\beta := 90 \cdot \text{deg}$

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)

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



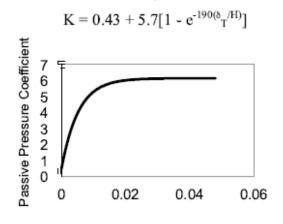
Integral Abutment and Wingwall - Passive Earth Pressure - Rankine Theory

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 $\alpha > 0$.

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

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

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



Relative Wall Displacement

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

Thermal displacement at each abutment: $\delta := 0.22$ inAbutment height:h := 11 ft $h = 132 \cdot in$ Relative wall displacement: $\mathbf{x} := \frac{\delta}{h}$ $\mathbf{x} = 0.0017$ $\mathbf{K} := 0.43 + 5.7 \cdot [1 - \exp[-190(\mathbf{x})]]$

< K_{p rank} of 3.25, therefore recommend K=3.25 at Abutment No. 2

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

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

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.

c' /yz							
ϕ' (deg)	α (deg)	0.025	0.050	0.100	0,500		
30	0	3.087	3.173	3.346	4.732		
	5	3.042	3.129	3.303	4.674		
	10	2.907	2.996	3.174	4.579		
	15	2.684	2.777	2.961	4.394		

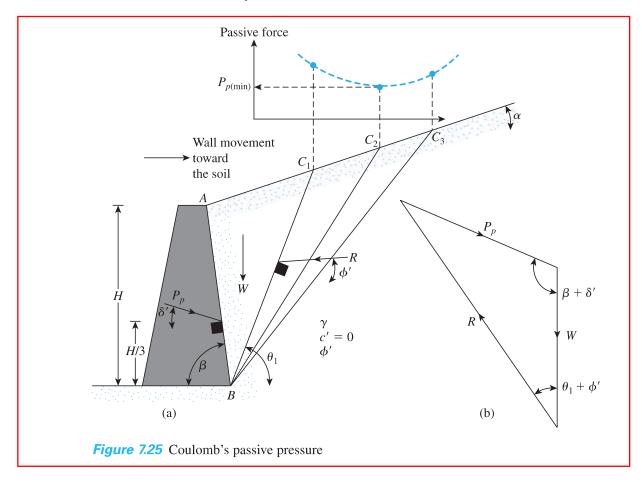
7.12

Coulomb's Passive Earth Pressure

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

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

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



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

Table 7.10	Values of K	[from Eq.	(7.71)] for β =	= 90° and $\alpha = 0^\circ$
------------	-------------	-----------	--------------------------	------------------------------

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

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

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

where

7.13

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

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

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

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

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 kN/m²

Hence,

$$\sigma'_{p} = \sigma'_{o}K_{p(2)} + 2c'_{2}\sqrt{K_{p(2)}} = 40.49(2.56) + (2)(10)(1.6)$$

= 135.65 kN/m²

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

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

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

7.11

Rankine Passive Earth Pressure: Vertical Backface and Inclined Backfill

Granular Soil

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

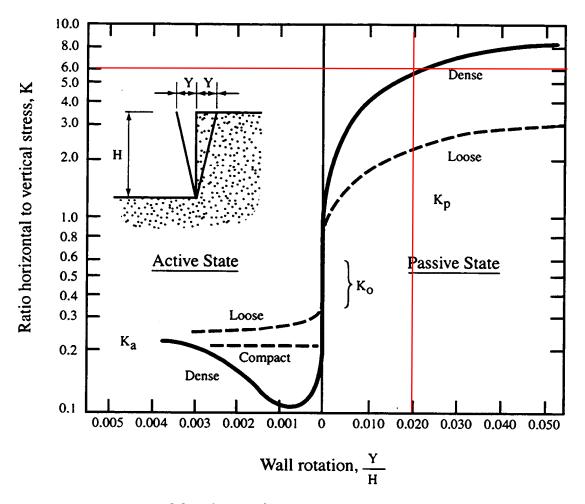
$$\sigma_p' = \gamma z K_p \tag{7.65}$$

and the passive force is

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

where

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



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

Magnitude of Wall Rotation to Reach Failure

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:

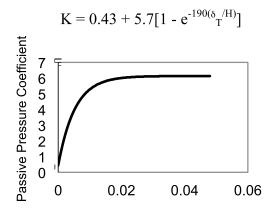
- 1. The pile cap is designed to resist all vertical loads including live load. It is assumed to act as a continuous beam supported by piles. The analysis can be simplified by assuming the pile cap acting as a simple span between piles and then taking 80% of simple span moments to account for continuity.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



Relative Wall Displacement

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.

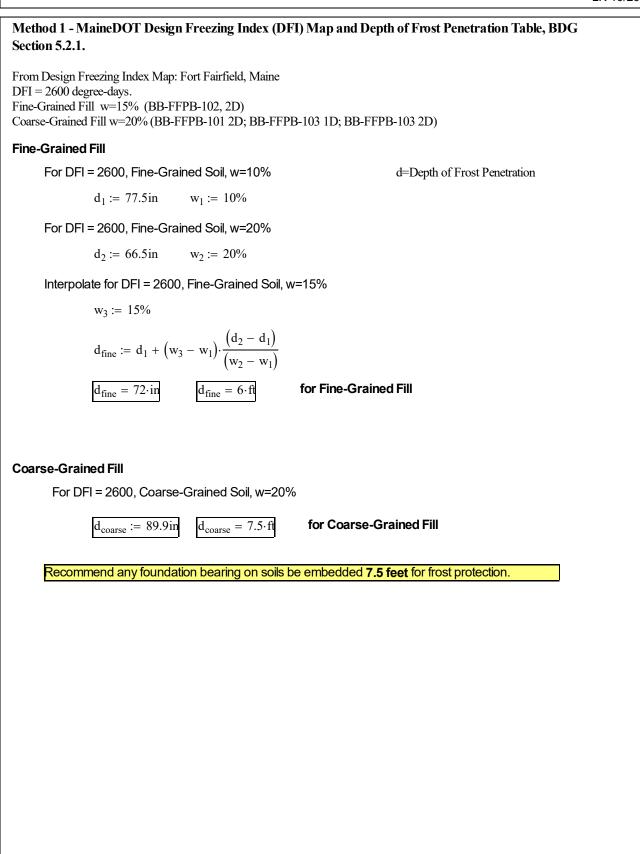
The longitudinal reinforcement of the pile cap shown in Chapter 12 of Part II of this Bridge Manual represents an upper-bound for the required reinforcement assuming the girders are located at the positions that produce maximum effects on the pile cap and assuming a conservative value of other dead loads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

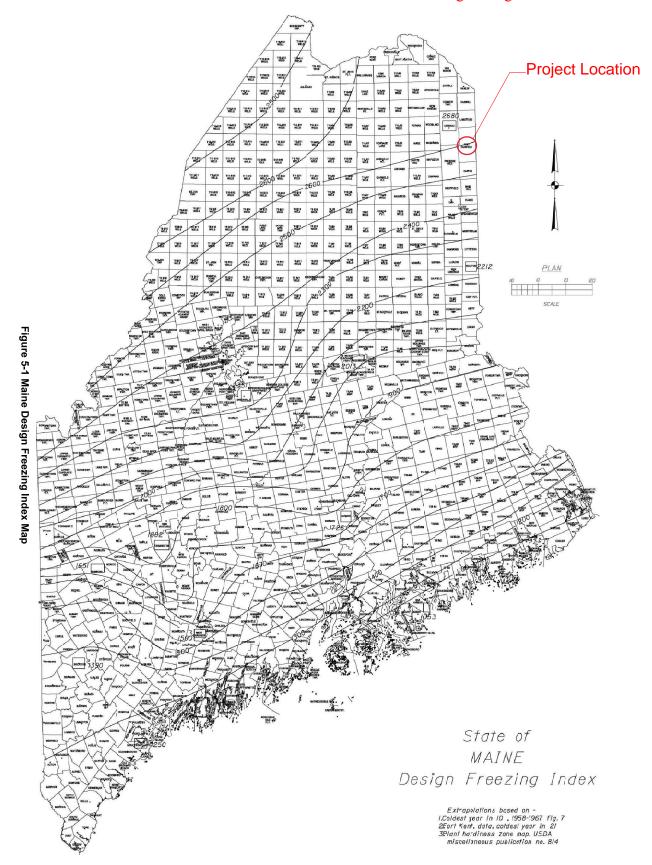
L-shaped connection reinforcing bars indicated in the standard drawings of Chapter 12 of Part II and Chapter 2 of Part III of this Bridge Manual shall be provided to transfer the maximum expected connection moment between the abutment and the superstructure. These bars shall be #6 @ 9" for girders up 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



MaineDOT Bridge Design Guide



5.2 General

MaineDOT Bridge Design Guide

5.2.1 Frost

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

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

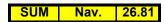
Design	Frost Penetration (in)					
Freezing	Co	arse Grair	red Fine Grained		d	
Index	w=10%	w=20%	w=30%	w=10%	w=20%	w=30%
1000	66.3	55.0	47.5	47.1	40.7	36.9
1100	69.8	57.8	49.8	49.6	42.7	38.7
1200	73.1	60.4	52.0	51.9	44.7	40.5
1300	76.3	63.0	54.3	54.2	46.6	42.2
1400	79.2	65.5	56.4	56.3	48.5	43.9
1500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

Table 5-1 Depth of Frost Penetration

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				



15 < Nav. < 50 bpf **Conclusion: Site Class D** Site Classification per LRFD Table C3.10.3.1-1 - Method B Fort Fairfield, Puddle Dock Bridge #2691 WIN 25453.00 March 25, 2024

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 Sa (sec) (g) 0.0 0.080 PGA - Site Class B 0.2 0.179 Ss - Site Class B 0.052 S1 - Site Class B 1.0 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 Sa (sec) (g) 0.0 0.128 As - Site Class D 0.2 0.287 SDs - Site Class D 0.125 SD1 - Site Class D 1.0

