MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

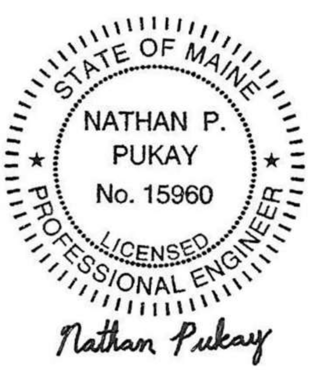
ST. MARY'S BRIDGE
CASTONGUAY ROAD OVER VIOLETTE BROOK
VAN BUREN, MAINE

Prepared by:

Nathan Pukay, P.E.
Transportation Engineer II

Reviewed by:

Laura Krusinski, P.E.
Senior Geotechnical Engineer



Aroostook County
WIN 26083.00

Soils Report 2025-25
Bridge No. 5309

Federal Project No. 2608300
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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 St. Mary's Bridge which carries Castonguay Road over Violette Brook in Van Buren, Maine. This report presents the subsurface information obtained at the site during the subsurface investigations, geotechnical design recommendations, and construction recommendations for the new substructures.

The existing St. Mary's Bridge was constructed in 1958 and is a steel structural plate pipe arch with a 26-foot span and a 7-foot 6-inch rise founded on concrete footings bearing on soil. According to the 2023 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the condition rating of St. Mary's Bridge is a 2 (critical condition) due to it being scour critical with a deep scour hole observed at the upstream end of the southeast footing. The culvert is otherwise in poor condition due to a corner of the pipe folding inward at the downstream end, along with cracking and moderate scaling of the concrete footings.

Available as-built drawings indicate a previous structure at the bridge consisted of a timber bridge comprised of log crib abutments.

The proposed replacement structure consists of a 72-foot, single-span, precast concrete Northeast Extreme Tee (NEXT) F-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 and vertical alignment that will approximately match the existing.

Traffic will be maintained with an off-site detour using State and State-aid roads.

2.0 GEOLOGIC SETTING

St. Mary's Bridge carries Castonguay Road over Violette Brook as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of the Van Buren Quadrangle, Maine, Open-File No. 78-9 (1978), indicates the surficial soils in the vicinity of the bridge project consist of stream alluvium, glacial-stream deposits and glacial till. Stream alluvium consists of sand, gravel, and silt deposited on flood plains and stream beds by postglacial streams. Glacial-stream deposits consist of sand and gravel deposited by meltwater streams and currents during melting of the Late Wisconsinan glacier. 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 calcareous pelite of the New Sweden Formation.

3.0 SUBSURFACE INVESTIGATION

Two test borings were drilled to explore subsurface conditions at the site. Boring BB-VBVB-101 was drilled near the location of proposed Abutment No. 1 and boring BB-VBVB-102 was drilled at the centerline of proposed Abutment No. 2. The boring locations are shown on Sheet 2 – Boring Location Plan.

The borings were drilled by the MaineDOT Drill Crew in November 2023 and June and July 2024. 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 November 2023. All N-values discussed in this report are corrected N-values computed by applying an average energy transfer of 0.962 to the raw field N-values. This hammer efficiency factor (0.962) and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring logs.

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

4.0 LABORATORY TESTING

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

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

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings generally consisted of Fill, 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 7 to 12 feet. The fill materials encountered consisted of:

- Brown, SAND, some gravel, some silt.

Corrected SPT N-values in the Fill unit ranged from 10 to 43 blows per foot (bpf) indicating the fill is loose to dense in consistency.

One grain size analysis performed on a sample recovered from the Fill unit indicated the material is classified as A-1-b under the AASHTO Soil Classification System and SM under the Unified Soil Classification System (USCS). The natural water content of the sample tested was 6 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 20 to 24 feet. The deposit was variable and consisted of:

- Brown, Sandy GRAVEL, little to some silt;
- Brown, SAND, some silt, trace to some gravel; and
- Cobbles.

Corrected SPT N-values within the Stream Alluvium deposit ranged from 22 to greater than 50 bpf, indicating the deposit is medium dense to very dense in consistency. One grain size analysis conducted on a sample of the deposit indicated the material is classified as A-1-b under the AASHTO Soil Classification System and GM under the USCS. The natural water content of the sample tested was 10 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 98 to 113 feet. The Glacial Till varied from:

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

Corrected SPT N-values within the fine-grained Glacial Till ranged from 27 to greater than 50 bpf indicating the fine-grained subunits are very stiff to hard in consistency.

Corrected SPT N-values within the coarse-grained Glacial Till ranged from 24 to greater than 50 bpf indicating those subunits are medium dense to very dense in consistency.

Three grain size analyses performed on samples recovered from the deposit resulted in the material being classified as A-4 and A-2-4 under the AASHTO Soil Classification System and SC-SM, ML and SM under the USCS. The natural water content of the samples tested ranged from 9 to 12 percent.

5.4 Bedrock

Bedrock was encountered and cored in both 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)
BB-VBVB-101	16+58	9.0 Rt	134.4	399.4	0, 20, 58
BB-VBVB-102	17+12	7.0 Lt	139.7	394.2	20, 0, 26

Bedrock at the site consisted of grey to dark grey, very fine to fine-grained, PHYLLITE, moderately hard, slightly weathered to fresh, with iron oxide staining on some fracture planes, joints dipping at moderate to vertical angles, closely spaced, with highly fractured zones and some calcite infilling. The RQD of the bedrock cores ranged from 0 to 58 percent, corresponding to a Rock Quality of very poor to fair.

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

5.5 Groundwater

Groundwater was measured at depths ranging from 9 to 10 feet below the roadway surface upon completion of the borings. Note that water was introduced into the boreholes during drilling operations and the measured levels may not represent stabilized groundwater elevations. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, river levels and construction activities.

Artisan water pressure was observed at a depth of 125 feet below the roadway surface while drilling both borings.

6.0 Foundation Alternatives

Due to the depth of bedrock and the chosen span length, integral abutments founded on driven piles was the preferred substructure design due to cost, ease of construction, and reduced maintenance costs.

7.0 Geotechnical Design Considerations and Recommendations

The following sections provide geotechnical design considerations and recommendations for H-pile supported integral abutments which is the proposed substructure type for the St. Mary's 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 In-Place Pile Lengths[1] (feet)
Abutment No. 1	523.8	399.4	127
Abutment No. 2	524.1	394.2	132

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

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

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

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

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

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

7.1.1 Axial Pile Resistance – Strength Limit State

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

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

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

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

Strength Limit State Factored Axial Pile Resistance					
Pile Section	Structural Resistance[1] ϕ_c=0.50 (kips)	Controlling Geotechnical Resistance[2] ϕ_c=0.50 (kips)	Drivability Resistance[3] φ_{dyn} = 0.65 (kips)		Governing Axial Pile Resistance[6] (kips)
HP 14 x 89	652	652	377[4]	397[5]	377[4]
HP 14 x 117	860	860	436[4]	462[5]	436[4]

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

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[1] Structural resistances were calculated for a braced pile segment in pure axial compression, using a resistance factor, ϕ_c, for severe driving conditions. Factored structural resistances should be calculated for upper and lower unbraced pile segments based upon L-Pile results using a resistance factor of $\phi_c = 0.70$ 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; Abutments 1 and 2 are the same for the HP 14x89 pile section. Abutment 2 controls for the HP 14x117 pile section.

[5] Drivability resistance based on a APE D25-42 pile hammer at Fuel Setting 4; Abutment 2 pile controls.

[6] Drivability evaluations were performed for both Abutments No.1 and 2 piles. Resistances for the HP 14x89 and HP 14x117 pile sections based on a APE D19-42 pile hammer govern.

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

7.1.2 Axial Pile Resistance – Service and Extreme Limit State

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

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

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

Service and Extreme Limit State Factored Axial Pile Resistance					
Pile Section	Structural Resistance[1] $\phi = 1.0$ (kips)	Controlling Geotechnical Resistance[2] $\phi = 1.0$ (kips)	Drivability Resistance[3] $\phi = 1.0$ (kips)		Governing Axial Pile Resistance[6] (kips)
HP 14 x 89	1,305	1,305	580[4]	610[5]	580[4]
HP 14 x 117	1,720	1,720	670[4]	710[5]	670[4]

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

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

[3] Drivability analyses were performed to determine the pile resistance that might be achieved considering available diesel hammers. Nominal drivability resistances were determined based on 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. Abutments 1 and 2 are the same for the HP 14x89 pile section. Abutment 2 controls for the HP 14x117 pile section.

[5] Drivability resistance based on a APE D25-42 Pile Hammer at Fuel Setting 4; Abutment 2 pile controls.

[6] Drivability evaluations performed for both Abutments No.1 and 2 piles. Resistances for the HP 14x89 and HP 14x117 pile sections based on a APE D19-42 pile hammer govern.

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

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

7.1.3 Lateral Pile Resistance/Behavior

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

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

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

LPile Input Parameters Abutment No. 1						
Soil Layer	Soil/Rock Model	Top Elevation of Layer (ft)	Layer Thickness (ft)	γ_e[1] (pcf)	ϕ'[2] (deg)	k_s[3] (pci)
Granular Borrow	Reese Sand	533	7	125	32	90
Granular Borrow	Reese Sand	526	4	63	32	60
Stream Alluvium	Reese Sand	522	24	68	34	60
Glacial Till	Reese Sand	498	44	78	36	125
Glacial Till	Reese Sand	454	54	83	38	125

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

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	534	10	125	32	90
Stream Alluvium	Reese Sand	524	17	73	36	90
Glacial Till	Reese Sand	507	68	73	36	125
Glacial Till	Reese Sand	439	45	83	38	125

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. In general, when the calculated ratio of lateral movement to wall height exceeds 0.004, a passive earth pressure coefficient can be estimated using MassDOT LRFD Bridge Design Manual Figure 3.10.8-1 (reproduced in Appendix D – Calculations). The thermal movement at each abutment was estimated by the bridge designer to be 0.246 inch, resulting in an estimated ratio of thermal expansion to abutment height (δ/H) of 0.0021. Therefore, Rankine Theory is recommended to determine the passive earth pressure coefficient. Using Rankine Theory, a lateral earth pressure coefficient of 3.25 is recommended assuming a δ/H of 0.0021 and a level backfill (see Appendix D – Calculations).

A load factor for passive earth pressure is not specified in LRFD. For purposes of the integral abutment backwall reinforcing steel design, use a maximum load factor (γ_{EH}) of 1.50 to calculate factored passive earth pressures.

Additional lateral earth pressure due to live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for abutments if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination of the surcharge load, is permitted per LRFD Article

3.11.6.5. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from the table, below:

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

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

7.3 Abutment Sections

The abutment design shall include a drainage system behind the abutment to intercept any groundwater. Drainage behind the structure shall be in accordance with MaineDOT BDG Section 5.4.2.13. 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 horizontal and vertical alignment of the new structure approximately match the existing. The bridge approach embankments will be constructed using granular borrow placed over loose to dense granular fill overlying primarily 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, Van Buren has a design freezing index (DFI) of approximately 2600 F-degree days. The anticipated coarse-grained fill soil was assigned a water content of 10%. These components correlate to a frost depth of 9.1 feet. Any foundation bearing on soils shall be embedded 9.1 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)	0.085g
Acceleration Coefficient (A_S)	0.137g
S_{DS} (Period = 0.2 sec)	0.309g
S_{D1} (Period = 1.0 sec)	0.133g
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 encountered in the stream alluvium and 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, then they may be cleared by conventional excavation methods, pre-augering, predrilling, spudding, use of rock chisels, or down-hole hammers.

Driven H-pile may reach the required nominal capacity within the glacial till. If this occurs, the pile driving criteria should be carried out for 6 consecutive inches. The geotechnical engineer will review the pile logs to confirm the depth of penetration is acceptable. If the depth of penetration is not acceptable, the contractor will be responsible to advance the pile further, which may include, but is not limited to, modifying the pile driving equipment, excavation, or predrilling.

Based on a Q1.1 water level of El. 526.32, a cofferdam will likely be necessary to successfully dewater and construct the abutments. A previous structure at the bridge was supported on log crib abutments. Wood obstructions may need to be removed by conventional excavation methods.

The new integral abutments will be constructed behind the existing concrete footings. 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 St. Mary's Bridge in Van Buren, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied.

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

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

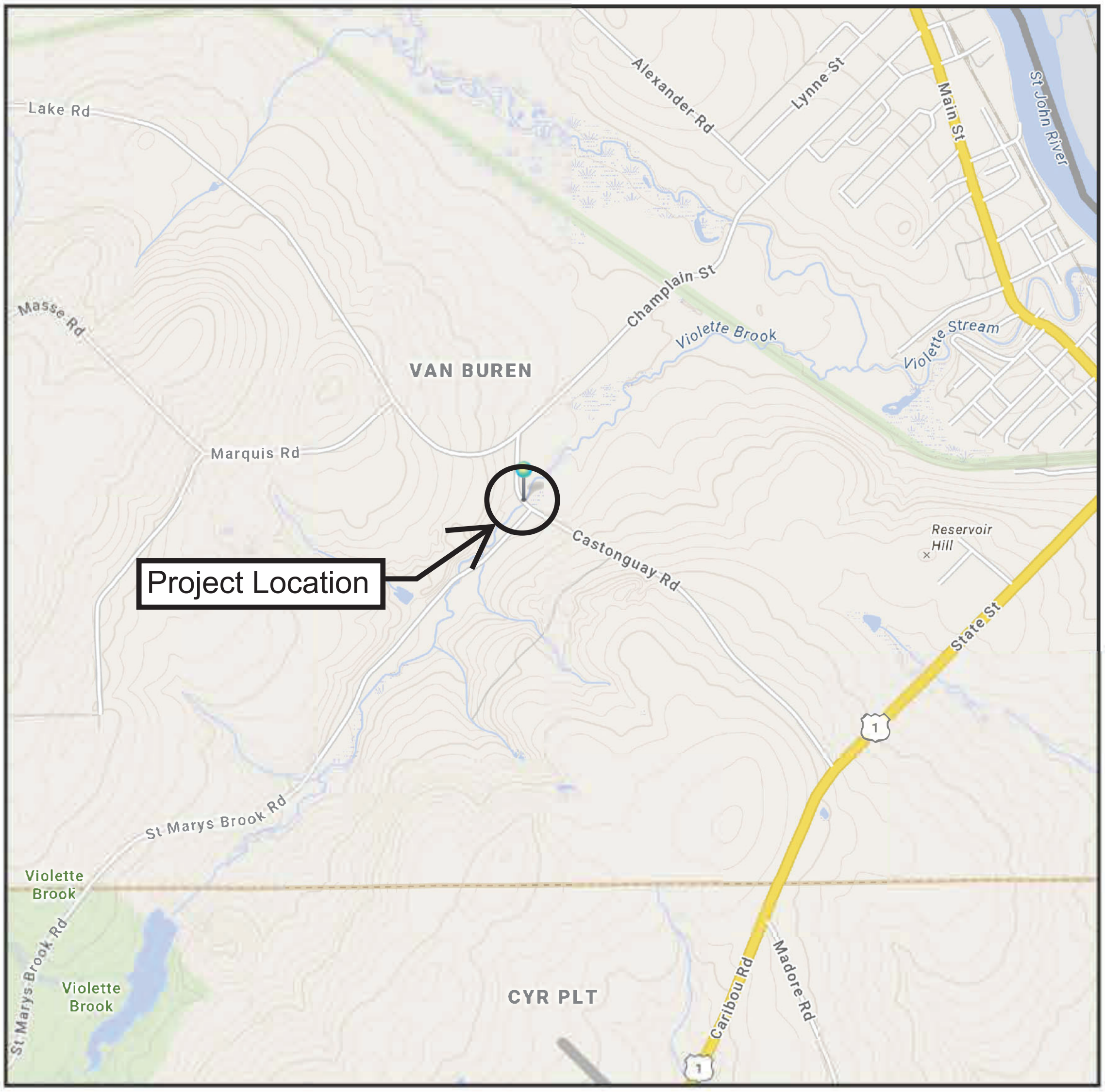
Sheets

DATE:2025/08/04

DIVISION:GEOTECH

USERNAME:TERRY.WHITE

VAN BUREN, MAINE



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0.5 Miles

1 inch =0.57 miles

Date: 8/4/2025
Time: 7:11:17 AM

SHEET NUMBER 1 OF 5	ST. MARY'S BRIDGE #5309 VIOLETTE BROOK VAN BUREN LOCATION MAP	STATE OF MAINE DEPARTMENT OF TRANSPORTATION 2608300 WIN 26083.00 BRIDGE PLANS

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2608300

WIN
26083.00

BRIDGE PLANS

BRIDGE NO. 5309

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		
CHECKED-REVIEWED		
DESIGN2-DETAILED2	NPUKAY	
DESIGN3-DETAILED3	TSWHITE	OCT 2025
REVISIONS 1		OCT 2025
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

VAN BUREN
ST. MARY'S BRIDGE

BORING LOCATION PLAN

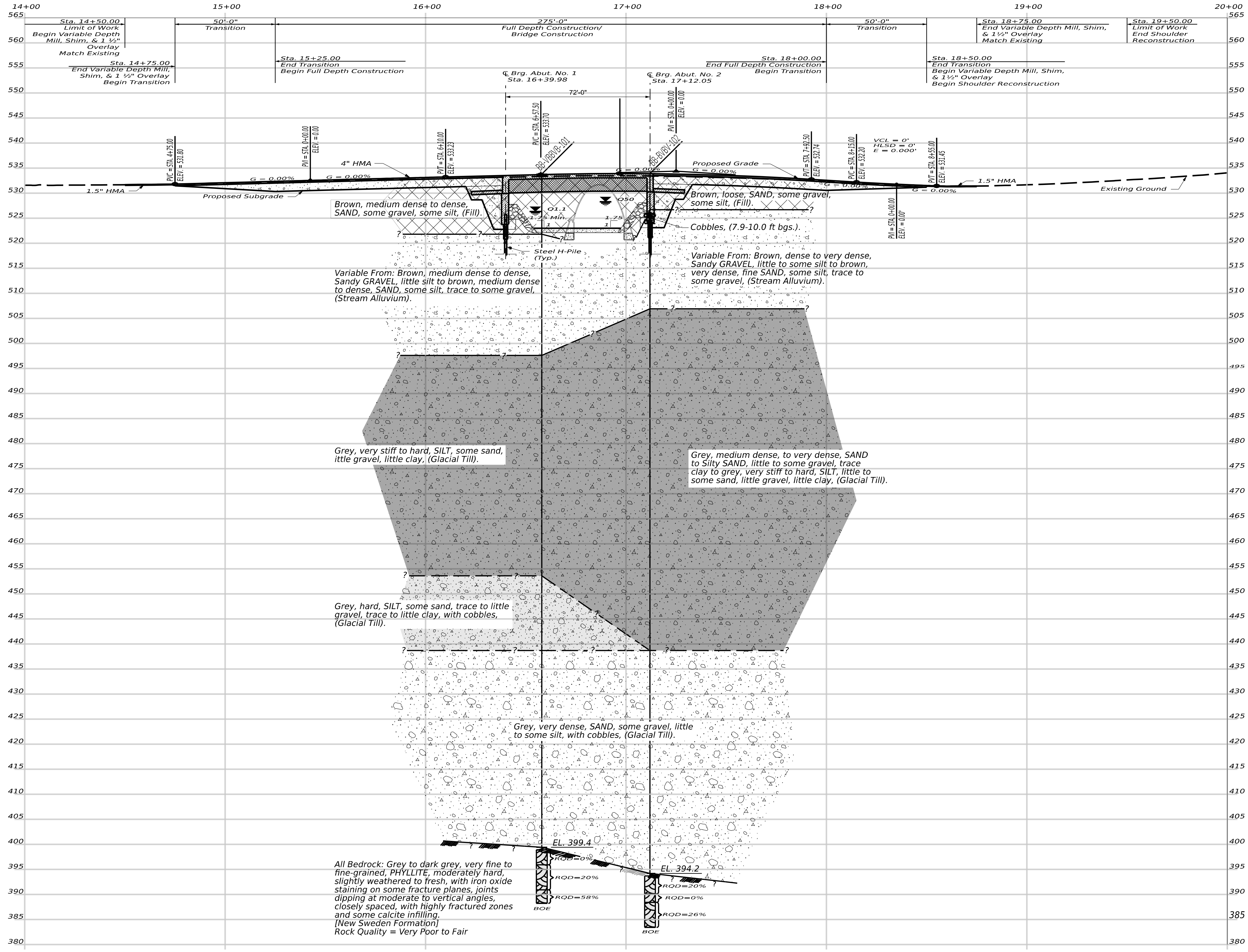
SHEET NUMBER

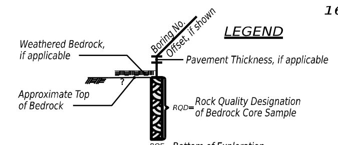
2

OF 5

Username: Nathan.P.Pukay Date: 12/1/2025

Filename: C:\Users\Nathan.P.Pukay\State of Maine\MaineDOT Work - 26083_00\GEOTECH\Boring_Location_Plan1.dgn





PROFILE

HORIZ 25 0 25 50
VERT 10 0 10 20
SCALE

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

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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
2608300
WIN 26083.00
BRIDGE PLANS
BRIDGE NO. 5309

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		
CHECKED-REVIEWED	N.PUKAY	
DESIGN2-DETAILED2	T.WHITE	OCT 2025
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

VAN BUREN
ST. MARY'S BRIDGE
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
3
OF 5

Username: Nathan.P.Pukay Date: 12/1/2025

Filename: C:\Users\Terry.White\State of Maine\MaineDOT Work - WIN\26083_00\GEOTECH\Geotech_ISP.dgn

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: St. Mary's Bridge #5309 carries Castonguay Road over Violette
Location: Van Buren, Maine

Boring No.: BB-VBVB-101
WIN: 26083.00

Driller:	MaineDOT	Elevation (ft.)	533.8	Auger ID/OD:	5" Solid Stem
Operator:	Daggett/Andrle	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	N. Pukay/B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	11/13/23-11/29/23	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	16+58, 9.0 ft Rt.	Casing ID/OD:	NW(3.0"/3.5")	Water Level*:	9.5 ft bgs.
Hammer Efficiency Factor: 0.962		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	533.6	3" HMA. (0.3)	
5	1D	24/19	5.00 - 7.00	12/13/14/16	27	43			Brown, moist, dense, SAND, some gravel, some silt, (Fill).	G#380935 A-1-b, SM WC=5.6%
10	2D	24/16	10.00 - 12.00	3/7/12/16	19	30	8		Brown, wet, medium dense, SAND, some gravel, some silt, (Fill).	
							49			
							84	521.8	(12.0)	
							72			
							82			
15	3D	24/18	15.00 - 17.00	14/9/12/11	21	34	24		Brown, wet, dense, Sandy GRAVEL, little silt, (Stream Alluvium).	G#380936 A-1-b, GM WC=10.1%
							26			
							36			
							42			
							50			
20	4D	24/9	20.00 - 22.00	13/10/7/13	17	27	30		Similar to 3D, except medium dense.	
							32			
							55			
							50			
							42			
25	5D	24/4	25.00 - 27.00	10/7/7/7	14	22	26		Brown, wet, medium dense, Sandy GRAVEL, little silt, (Stream Alluvium).	
							23			
							46			
							45			
							44			
30	6D	24/18	30.00 - 32.00	9/9/14/18	23	37	29		Brown, wet, dense, SAND, some silt, some gravel, (Stream Alluvium).	
							60			
							76			
							104			
							117			
35	7D/A	24/12	35.00 - 37.00	10/10/7/20	17	27	42	497.8	7D (Top 6") Brown, wet, medium dense, fine SAND, some silt, trace gravel, (Stream Alluvium). (36.0) 7D/A (Bottom 6") Brown, wet, very stiff, SILT, some sand, little gravel, (Glacial Till).	
							75			
							100			
							93			
							86			
40	8D	24/18	40.00 - 42.00	18/12/17/19	29	46	RC		Similar to 7D/A, except hard.	
45	9D	24/8	45.00 - 47.00	14/14/18/17	32	51	70		Grey, wet, hard, SILT, some sand, little gravel, little clay, (Glacial Till).	
							71			
							102			
							102			
							135			
50	10D	24/12	50.00 - 52.00	9/16/21/20	37	59	105		Similar to 9D.	
							172			
							214			
							245			
							OPEN HOLE			
55	11D	24/18	55.00 - 57.00	9/14/22/32	36	58			Grey, wet, hard, SILT, some sand, little gravel, little clay, (Glacial Till).	
60	12D	24/12	60.00 - 62.00	13/12/16/33	28	45			Grey, wet, hard, SILT, some sand, little gravel, little clay, (Glacial Till).	G#380937 A-4, SC-SM WC=10.0%
65	13D	24/2	65.00 - 67.00	13/14/15/18	29	46			Grey, wet, hard, SILT, some sand, little gravel, little clay, (Glacial Till).	
70	14D	24/8	70.00 - 72.00	15/10/13/12	23	37			Similar to 13D.	
75										

Remarks:

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

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

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS
Project: St. Mary's Bridge #5309 carries Castonguay Road over Violette
Location: Van Buren, Maine
Boring No.: BB-VBVB-101
WIN: 26083.00

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75	15D	24/8	75.00 - 77.00	9/9/9/15	18	29			Grey, wet, very stiff, SILT, some sand, little gravel, little clay, (Glacial Till).	
80	16D	24/8	80.00 - 82.00	13/27/26/24	53	85		453.8	(80.0) Grey, wet, hard, SILT, some sand, little clay, little gravel, (Glacial Till). Cobble at 82.2 ft bgs.	
85	17D	24/12	85.00 - 87.00	37/39/51/59	90	144			Grey, wet, hard, SILT, some sand, trace gravel, (Glacial Till). Cobble at 89.5 ft bgs.	G#380938 A-4, ML WC=11.7%
90	18D	14.4/8	90.00 - 91.20	32/56/50(2.4")	---		NQ-2		Similar to 17D. Cored from 91.2-94.9 ft bgs. through cobbles and glacial till.	
95	19D	12/6	95.00 - 96.00	48/86	---		OPEN HOLE	438.9	(94.9) Grey-brown, wet, very dense, SAND, some silt, some gravel, (Glacial Till). Occasional Cobble.	
100	20D	12/6	100.00 - 101.00	55/76	---				Grey-brown, wet, very dense, SAND, some gravel, little silt, (Glacial Till).	
105	21D	2.4/1	105.00 - 105.20	50(2.4")	---				Similar to 20D.	
110	22D	3/1	110.00 - 110.25	70(3")	---				Grey-brown, wet, very dense, SAND, some gravel, little silt, (Glacial Till).	
115	23D	6/4	115.70 - 116.20	59	---				Grey-brown, wet, very dense, SAND, some silt, some gravel, (Glacial Till).	
120										
125	24D	4.8/4	125.00 - 125.40	92(4.8")	---				Similar to 23D. Artisan water pressure at 125 ft bgs.	G#380939 A-2-4, SM WC=9.3%
130	25D	4.8/4.8	130.00 - 130.40	75(4.8")	---				Grey-brown, wet, very dense, SAND, some silt, some gravel, (Glacial Till).	
135	R1	36/24	134.90 - 137.90	RQD = 0%			NQ-2	399.4	(134.4) Top of Bedrock at Elev. 399.4 ft. Roller Coned ahead to 134.9 ft bgs. R1: Bedrock: Grey to dark grey, very fine to fine-grained, PHYLLITE, moderately hard, slightly weathered with iron oxide stained joint faces, near vertical foliation paralleling bedding, highly fractured. [New Sweden Formation] Rock Quality = Very Poor R1: Core times (min:sec) 134.9-135.9 ft (2:45) 135.9-136.9 ft (5:16) 136.9-137.9 ft (5:31) Core Blocked 67% Recovery	
	R2	60/59	137.90 - 142.90	RQD = 20%					R2: Bedrock: Grey to dark grey, very fine to fine-grained, PHYLLITE, moderately hard, slightly weathered joint faces, near vertical foliation paralleling bedding, highly fractured, with bands of calcite infilling, then grey to dark grey, very fine to fine-grained, PHYLLITE, moderately hard, fresh, near vertical foliation paralleling bedding, steeply dipping joints, closely spaced. [New Sweden Formation] Rock Quality = Very Poor R2: Core Times (min:sec) 137.9-138.9 ft (3:03) 138.9-139.9 ft (4:21) 139.9-140.9 ft (3:51) 140.9-141.9 ft (6:20) 141.9-142.9 ft (7:04) 98% Recovery	
	R3	36/36	142.90 - 145.90	RQD = 58%				387.9	R3: Bedrock: Grey to dark grey, very fine to fine-grained, PHYLLITE, moderately hard, fresh, near vertical foliation paralleling bedding, calcite infused lenses trending across foliation, steeply dipping joints, closely spaced.	
150										

Remarks:

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

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

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS
Project: St. Mary's Bridge #5309 carries Castonguay Road over Violette
Location: Van Buren, Maine
Boring No.: BB-VBVB-101
WIN: 26083.00

Depth (ft.)	Visual Description and Remarks
150	[New Sweden Formation] Rock Quality = Fair R3: Core Times (min:sec) 142.9-143.9 ft (2:45) 143.9-144.9 ft (2:59) 144.9-145.9 ft (3:24) 100% Recovery (145.9) Bottom of Exploration at 145.9 feet below ground surface.
155	
160	
165	
170	
175	

Remarks:

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

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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
2608300
WIN 26083.00
BRIDGE NO. 5309
BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		
CHECKED-REVIEWED		
DESIGN2-DETAILED2	N.PUKAY	T.WHITE OCT 2025
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

VAN BUREN ST. MARY'S BRIDGE
BORING LOGS

SHEET NUMBER 4 OF 5

Username: Nathan.P.Pukay Date: 12/1/2025

Filename: C:\Users\Nathan.P.Pukay\State of Maine\MaineDOT Work - 26083_00\GEOTECH\Boring_Log_Plan1.dgn

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

Project: St. Mary's Bridge #5309 carries Castonguay Road over Violette
Location: Van Buren, Maine

Boring No.: BB-VBVB-102
WIN: 26083.00

Driller:	MaineDOT	Elevation (ft.)	533.9	Auger ID/OD:	5" Solid Stem
Operator:	Daggett/Andrle	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	N. Pukay/B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	6/24/2024, 7/2/2024	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	17+12, 7.0 ft Lt.	Casing ID/OD:	HW(4/4.5"), NW(3/3.5")	Water Level*:	8.5 ft bgs.
Hammer Efficiency Factor:	0.962	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	533.5	5" HMA. —0.4	
5	1D	20.4/11	5.00 - 6.70	5/3/3/50(2.4")	6	10			Brown, moist, loose, SAND, some gravel, some silt, (Fill).	
								526.9	Cobble at 6.7 ft bgs. —7.0	
									Multiple cobbles from 7.9-10.0 ft bgs.	
10	2D	24/2	10.00 - 12.00	27/18/20/34	38	61	RC		Brown, wet, very dense, Sandy GRAVEL, some silt, (Stream Alluvium). Set in HW casing at 10.0 ft bgs.	
15	3D	24/8	15.00 - 17.00	11/13/14/19	27	43	87		Brown, wet, dense, Sandy GRAVEL little silt, (Stream Alluvium).	
							86			
							104			
							88			
							84			
20	4D	24/6	20.00 - 22.00	13/13/18/16	31	50	34		Brown, wet, dense, Sandy GRAVEL, some silt, (Stream Alluvium).	
							81			
							80			
							84			
							140			
25	5D	24/11	25.00 - 27.00	11/14/20/25	34	55	41		Brown, wet, very dense, fine SAND, some silt, (Stream Alluvium).	
							137			
							240	506.9	—27.0	
							201			
							209			
30	6D	24/8	30.00 - 32.00	13/15/16/15	31	50	OPEN HOLE		Grey wet, dense, Silty SAND, some gravel, (Glacial Till).	
35	7D	24/7	35.00 - 37.00	10/20/21/20	41	66			Grey, wet, very dense, SAND, some gravel, some silt, (Glacial Till).	
45	MD	3.6/0	45.00 - 45.30	70(3.6")	---				Similar to 7D in wash water.	
55	8D	24/1	55.00 - 57.00	7/7/10/13	17	27			Grey, wet, medium dense, SAND, some silt, little gravel, (Glacial Till).	
65	9D	24/13	65.00 - 67.00	7/10/13/19	23	37			Grey, wet, hard, SILT, some sand, little gravel, little clay, (Glacial Till).	
75										

Remarks:

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

Page 1 of 3
Boring No.: BB-VBVB-102

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS
Project: St. Mary's Bridge #5309 carries Castonguay Road over Violette
Location: Van Buren, Maine
Boring No.: BB-VBVB-102
WIN: 26083.00

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75	10D	24/11	75.00 - 77.00	7/7/10/12	17	27			Grey, wet, very stiff, SILT, little clay, little sand, (Glacial Till). Set NW casing at 75.0 ft bgs.	
85	11D	24/8	85.00 - 87.00	4/7/8/10	15	24			Grey, wet, medium dense, Silty, fine SAND, trace clay, (Glacial Till).	
95	12D	24/11	95.00 - 97.00	23/24/25/26	49	79		438.9	—95.0 Grey, wet, very dense, Silty SAND, little gravel, (Glacial Till).	
100									Drilling slowed significantly at approximately 100 ft bgs. 700-1000 psi down pressure required to advance hole. Occasional cobble.	
105	13D	10.8/5	105.00 - 105.90	37/64(4.8")	---				Grey, wet, very dense, SAND, some silt, some gravel, (Glacial Till).	
115	14D	12/7	115.00 - 116.00	33/70	---				Similar to 13D. Occasional cobble.	
125									Artesian water pressure at 125.0 ft bgs. Occasional cobble.	
140	R1	43.2/43.2	140.20 - 143.80	RQD = 20%			NQ-2	394.2	—139.7 Top of Bedrock at Elev. 394.2 ft. Roller coned ahead to 140.2 ft bgs. R1: Bedrock: Grey to dark grey, very fine to fine-grained, PHYLLITE, moderately hard, slightly weathered joint faces, joints dipping at steep to vertical angles, closely spaced, with fractured zones. [New Sweden Formation] Rock Quality = Very Poor R1: Core Times (min:sec) 140.2-141.2 ft (4:14) 141.2-142.2 ft (5:59) 142.2-143.2 ft (5:04) 143.2-143.8 ft (7:33) core blocked 100% Recovery	
	R2	26.4/17	143.80 - 145.50	RQD = 0%					R2: Bedrock: Grey to dark grey, very fine to fine-grained, PHYLLITE, moderately hard, slightly weathered, highly fractured. [New Sweden Formation] Rock Quality = Very Poor R2: Core Times (min:sec) 143.8-144.2 ft (3:48) 144.2-145.2 ft (3:11) 145.2-145.5 ft (5:27) core blocked	
145	R3	60/60	145.50 - 150.50	RQD = 26%						
150										

Remarks:

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

Page 2 of 3
Boring No.: BB-VBVB-102

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
150								383.4	64% Recovery R3: Bedrock: Grey to dark grey, very fine to fine-grained, PHYLLITE, moderately hard, slightly weathered joint faces, joints dipping at steep to moderate angles, closely spaced. [New Sweden Formation] Rock Quality = Poor R3: Core Times (min:sec) 145.5-146.5 ft (3:19) 146.5-147.5 ft (2:52) 147.5-148.5 ft (3:48) 148.5-149.5 ft (2:41) 149.5-150.5 ft (3:43) 100% Recovery —150.5 Bottom of Exploration at 150.5 feet below ground surface.	
155										
160										
165										
170										
175										

Remarks:

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

Page 3 of 3
Boring No.: BB-VBVB-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
2608300
WIN 26083.00
BRIDGE PLANS
BRIDGE NO. 5309

SIGNATURE
P.E. NUMBER
DATE

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		
CHECKED-REVIEWED	N.PUKAY	
DESIGN2-DETAILED2	T.WHITE	OCT 2025
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

VAN BUREN
ST. MARY'S BRIDGE
BORING LOGS

SHEET NUMBER
5
OF 5

Username: Nathan.P.Pukay Date: 12/1/2025

Filename: C:\Users\Nathan.P.Pukay\State of Maine\MaineDOT Work - 26083_00\GEOTECH\Boring_Log_Plan2.dgn

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

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

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

MODIFIED BURMISTER SYSTEM

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Density of Cohesionless Soils	Standard Penetration Resistance N_{60}-Value (blows per foot)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

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

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

Rock Quality Designation (RQD):

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

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

Rock Quality Based on RQD

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

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: St. Mary's Bridge #5309 carries Castonguay Road over Violette Brook
Location: Van Buren, Maine

Boring No.: BB-VBVB-101
WIN: 26083.00

Driller:	MaineDOT	**Elevation (ft.)**	533.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay/B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	11/13/23-11/29/23	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	16+58, 9.0 ft Rt.	**Casing ID/OD:**	NW(3.0"/3.5")	**Water Level*:**	9.5 ft bgs.
Hammer Efficiency Factor:	0.962	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	533.6		3" HMA. 0.3	
5	1D	24/19	5.00 - 7.00	12/13/14/16	27	43				Brown, moist, dense, SAND, some gravel, some silt, (Fill).	G#380935 A-1-b, SM WC=5.6%
10	2D	24/16	10.00 - 12.00	3/7/12/16	19	30	8			Brown, wet, medium dense, SAND, some gravel, some silt, (Fill).	
							49				
							84	521.8		12.0	
							72				
							82				
15	3D	24/10	15.00 - 17.00	14/9/12/11	21	34	24			Brown, wet, dense, Sandy GRAVEL, little silt, (Stream Alluvium).	G#380936 A-1-b, GM WC=10.1%
							26				
							36				
							42				
							50				
20	4D	24/9	20.00 - 22.00	13/10/7/13	17	27	30			Similar to 3D, except medium dense.	
							32				
							55				
							50				
							42				
25											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: St. Mary's Bridge #5309 carries Castonguay Road over Violette Brook

Location: Van Buren, Maine

Boring No.: BB-VBVB-101

WIN: 26083.00

Driller:	MaineDOT	**Elevation (ft.)**	533.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay/B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	11/13/23-11/29/23	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	16+58, 9.0 ft Rt.	**Casing ID/OD:**	NW(3.0"/3.5")	**Water Level*:**	9.5 ft bgs.
Hammer Efficiency Factor:	0.962	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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WOH = Weight of 140 lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
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$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
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T_V = Pocket Torvane Shear Strength (psf)
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LL = Liquid Limit
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G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/4	25.00 - 27.00	10/7/7/7	14	22	26			Brown, wet, medium dense, Sandy GRAVEL, little silt, (Stream Alluvium).	
							23				
							46				
							45				
							44				
30	6D	24/10	30.00 - 32.00	9/9/14/18	23	37	29			Brown, wet, dense, SAND, some silt, some gravel, (Stream Alluvium).	
							60				
							76				
							104				
							117				
35	7D/A	24/12	35.00 - 37.00	10/10/7/20	17	27	42			7D (Top 6") Brown, wet, medium dense, fine SAND, some silt, trace gravel, (Stream Alluvium).	
							75	497.8		36.0 — 7D/A (Bottom 6") Brown, wet, very stiff, SILT, some sand, little gravel, (Glacial Till).	
							100				
							93				
							86				
40	8D	24/10	40.00 - 42.00	18/12/17/19	29	46	RC			Similar to 7D/A, except hard.	
							↓				
							↓				
							↓				
							↓				
45	9D	24/8	45.00 - 47.00	14/14/18/17	32	51	70			Grey, wet, hard, SILT, some sand, little gravel, little clay, (Glacial Till).	
							71				
							102				
							102				
50							135				

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: St. Mary's Bridge #5309 carries Castonguay Road over Violette Brook
Location: Van Buren, Maine

Boring No.: BB-VBVB-101
WIN: 26083.00

Driller:	MaineDOT	**Elevation (ft.)**	533.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay/B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	11/13/23-11/29/23	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	16+58, 9.0 ft Rt.	**Casing ID/OD:**	NW(3.0"/3.5")	**Water Level*:**	9.5 ft bgs.
Hammer Efficiency Factor:	0.962	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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WOR/C = Weight of Rods or Casing
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$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
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N-uncorrected = Raw Field SPT N-value
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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	10D	24/12	50.00 - 52.00	9/16/21/20	37	59	105			Similar to 9D.	
							172				
							214				
							245				
							OPEN HOLE				
55	11D	24/10	55.00 - 57.00	9/14/22/32	36	58				Grey, wet, hard, SILT, some sand, little gravel, little clay, (Glacial Till).	
60	12D	24/12	60.00 - 62.00	13/12/16/33	28	45				Grey, wet, hard, SILT, some sand, little gravel, little clay, (Glacial Till).	G#380937 A-4, SC-SM WC=10.0%
65	13D	24/2	65.00 - 67.00	13/14/15/18	29	46				Grey, wet, hard, SILT, some sand, little gravel, little clay, (Glacial Till).	
70	14D	24/8	70.00 - 72.00	15/10/13/12	23	37				Similar to 13D.	
75											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: St. Mary's Bridge #5309 carries Castonguay Road over Violette Brook
Location: Van Buren, Maine

Boring No.: BB-VBVB-101
WIN: 26083.00

Driller:	MaineDOT	**Elevation (ft.)**	533.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay/B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	11/13/23-11/29/23	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	16+58, 9.0 ft Rt.	**Casing ID/OD:**	NW(3.0"/3.5")	**Water Level*:**	9.5 ft bgs.
Hammer Efficiency Factor:	0.962	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75	15D	24/8	75.00 - 77.00	9/9/9/15	18	29				Grey, wet, very stiff, SILT, some sand, little gravel, little clay, (Glacial Till).	
80	16D	24/8	80.00 - 82.00	13/27/26/24	53	85		453.8		80.0 — Grey, wet, hard, SILT, some sand, little clay, little gravel, (Glacial Till). Cobble at 82.2 ft bgs.	
85	17D	24/12	85.00 - 87.00	37/39/51/59	90	144				Grey, wet, hard, SILT, some sand, trace gravel, (Glacial Till).	G#380938 A-4, ML WC=11.7%
90	18D	14.4/8	90.00 - 91.20	32/56/50(2.4")	---		NQ-2			Cobble at 89.5 ft bgs. Similar to 17D. Cored from 91.2-94.9 ft bgs. through cobbles and glacial till.	
95	19D	12/6	95.00 - 96.00	48/86	---		OPEN HOLE	438.9		94.9 — Grey-brown, wet, very dense, SAND, some silt, some gravel, (Glacial Till). Occasional Cobble.	
100											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: St. Mary's Bridge #5309 carries Castonguay Road over Violette Brook
Location: Van Buren, Maine

Boring No.: BB-VBVB-101
WIN: 26083.00

Driller:	MaineDOT	**Elevation (ft.)**	533.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay/B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	11/13/23-11/29/23	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	16+58, 9.0 ft Rt.	**Casing ID/OD:**	NW(3.0"/3.5")	**Water Level*:**	9.5 ft bgs.
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
100	20D	12/6	100.00 - 101.00	55/76	---					Grey-brown, wet, very dense, SAND, some gravel, little silt, (Glacial Till).	
105	21D	2.4/1	105.00 - 105.20	50(2.4")	---					Similar to 20D.	
110	22D	3/1	110.00 - 110.25	70(3")	---					Grey-brown, wet, very dense, SAND, some gravel, little silt, (Glacial Till).	
115	23D	6/4	115.70 - 116.20	59	---					Grey-brown, wet, very dense, SAND, some silt, some gravel, (Glacial Till).	
120											
125											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: St. Mary's Bridge #5309 carries Castonguay Road over Violette Brook
Location: Van Buren, Maine

Boring No.: BB-VBVB-101
WIN: 26083.00

Driller:	MaineDOT	**Elevation (ft.)**	533.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay/B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	11/13/23-11/29/23	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	16+58, 9.0 ft Rt.	**Casing ID/OD:**	NW(3.0"/3.5")	**Water Level*:**	9.5 ft bgs.
Hammer Efficiency Factor:	0.962	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
125	24D	4.8/4	125.00 - 125.40	92(4.8")	---					Similar to 23D. Artisan water pressure at 125 ft bgs.	G#380939 A-2-4, SM WC=9.3%
130	25D	4.8/4.8	130.00 - 130.40	75(4.8")	---					Grey-brown, wet, very dense, SAND, some silt, some gravel, (Glacial Till).	
										134.4	
	R1	36/24	134.90 - 137.90	RQD = 0%			NQ-2	399.4		Top of Bedrock at Elev. 399.4 ft. Roller Coned ahead to 134.9 ft bgs. R1: Bedrock: Grey to dark grey, very fine to fine-grained, PHYLLITE, moderately hard, slightly weathered with iron oxide stained joint faces, near vertical foliation paralleling bedding, highly fractured. [New Sweden Formation] Rock Quality = Very Poor R1: Core times (min:sec) 134.9-135.9 ft (2:45) 135.9-136.9 ft (5:16) 136.9-137.9 ft (5:31) Core Blocked 67% Recovery	
135											
	R2	60/59	137.90 - 142.90	RQD = 20%						R2: Bedrock: Grey to dark grey, very fine to fine-grained, PHYLLITE, moderately hard, slightly weathered joint faces, near vertical foliation paralleling bedding, highly fractured, with bands of calcite infilling, then grey to dark grey, very fine to fine-grained, PHYLLITE, moderately hard, fresh, near vertical foliation paralleling bedding, steeply dipping joints, closely spaced. [New Sweden Formation] Rock Quality = Very Poor R2: Core Times (min:sec) 137.9-138.9 ft (3:03) 138.9-139.9 ft (4:21) 139.9-140.9 ft (3:51) 140.9-141.9 ft (6:20) 141.9-142.9 ft (7:04) 98% Recovery	
140											
	R3	36/36	142.90 - 145.90	RQD = 58%						R3: Bedrock: Grey to dark grey, very fine to fine-grained, PHYLLITE, moderately hard, fresh, near vertical foliation paralleling	
145								387.9			
150											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: St. Mary's Bridge #5309 carries Castonguay Road over Violette Brook
Location: Van Buren, Maine

Boring No.: BB-VBVB-101
WIN: 26083.00

Driller:	MaineDOT	**Elevation (ft.)**	533.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay/B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	11/13/23-11/29/23	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	16+58, 9.0 ft Rt.	**Casing ID/OD:**	NW(3.0"/3.5")	**Water Level*:**	9.5 ft bgs.
Hammer Efficiency Factor:	0.962	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
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N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
150										bedding, calcite infused lenses trending across foliation, steeply dipping joints, closely spaced. [New Sweden Formation] Rock Quality = Fair R3: Core Times (min:sec) 142.9-143.9 ft (2:45) 143.9-144.9 ft (2:59) 144.9-145.9 ft (3:24) 100% Recovery	
										145.9	
										Bottom of Exploration at 145.9 feet below ground surface.	
155											
160											
165											
170											
175											

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Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: St. Mary's Bridge #5309 carries Castonguay Road over Violette Brook
Location: Van Buren, Maine

Boring No.: BB-VBVB-102

WIN: 26083.00

Driller:	MaineDOT	**Elevation (ft.)**	533.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay/B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/24/2024, 7/2/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	17+12, 7.0 ft Lt.	**Casing ID/OD:**	HW(4/4.5"), NW(3/3.5")	**Water Level*:**	8.5 ft bgs.
Hammer Efficiency Factor:	0.962	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	533.5		5" HMA. ——— 0.4	
5	1D	20.4/11	5.00 - 6.70	5/3/3/50(2.4")	6	10				Brown, moist, loose, SAND, some gravel, some silt, (Fill).	
								526.9		Cobble at 6.7 ft bgs. ——— 7.0	
										Multiple cobbles from 7.9-10.0 ft bgs.	
10	2D	24/2	10.00 - 12.00	27/18/20/34	38	61	RC			Brown, wet, very dense, Sandy GRAVEL, some silt, (Stream Alluvium). Set in HW casing at 10.0 ft bgs.	
15	3D	24/8	15.00 - 17.00	11/13/14/19	27	43	87			Brown, wet, dense, Sandy GRAVEL little silt, (Stream Alluvium).	
							86				
							104				
							88				
							84				
20	4D	24/6	20.00 - 22.00	13/13/18/16	31	50	34			Brown, wet, dense, Sandy GRAVEL, some silt, (Stream Alluvium).	
							81				
							80				
							84				
25							140				

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Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: St. Mary's Bridge #5309 carries Castonguay Road over Violette Brook
Location: Van Buren, Maine

Boring No.: BB-VBVB-102

WIN: 26083.00

Driller:	MaineDOT	**Elevation (ft.)**	533.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay/B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/24/2024, 7/2/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	17+12, 7.0 ft Lt.	**Casing ID/OD:**	HW(4/4.5"), NW(3/3.5")	**Water Level*:**	8.5 ft bgs.
Hammer Efficiency Factor:	0.962	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/11	25.00 - 27.00	11/14/20/25	34	55	41			Brown, wet, very dense, fine SAND, some silt, (Stream Alluvium).	
							137				
							240	506.9		27.0	
							201				
							209				
30	6D	24/8	30.00 - 32.00	13/15/16/15	31	50	OPEN HOLE			Grey wet, dense, Silty SAND, some gravel, (Glacial Till).	
35	7D	24/7	35.00 - 37.00	10/20/21/20	41	66				Grey, wet, very dense, SAND, some gravel, some silt, (Glacial Till).	
40											
45	MD	3.6/0	45.00 - 45.30	70(3.6")	---					Similar to 7D in wash water.	
50											

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Boring No.: BB-VBVB-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: St. Mary's Bridge #5309 carries Castonguay Road over Violette Brook
Location: Van Buren, Maine

Boring No.: BB-VBVB-102
WIN: 26083.00

Driller:	MaineDOT	**Elevation (ft.)**	533.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay/B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/24/2024, 7/2/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	17+12, 7.0 ft Lt.	**Casing ID/OD:**	HW(4/4.5"), NW(3/3.5")	**Water Level*:**	8.5 ft bgs.
Hammer Efficiency Factor:	0.962	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50											
55	8D	24/1	55.00 - 57.00	7/7/10/13	17	27				Grey, wet, medium dense, SAND, some silt, little gravel, (Glacial Till).	
60											
65	9D	24/13	65.00 - 67.00	7/10/13/19	23	37				Grey, wet, hard, SILT, some sand, little gravel, little clay, (Glacial Till).	
70											
75											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	St. Mary's Bridge #5309 carries Castonguay Road over Violette Brook	**Boring No.:**	BB-VBVB-102		
Location:	Van Buren, Maine	**WIN:**	26083.00		
Driller:	MaineDOT	**Elevation (ft.)**	533.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay/B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/24/2024, 7/2/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	17+12, 7.0 ft Lt.	**Casing ID/OD:**	HW(4/4.5"), NW(3/3.5")	**Water Level*:**	8.5 ft bgs.
Hammer Efficiency Factor:	0.962	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75	10D	24/11	75.00 - 77.00	7/7/10/12	17	27				Grey, wet, very stiff, SILT, little clay, little sand, (Glacial Till). Set NW casing at 75.0 ft bgs.	
80											
85	11D	24/8	85.00 - 87.00	4/7/8/10	15	24				Grey, wet, medium dense, Silty, fine SAND, trace clay, (Glacial Till).	
90											
95	12D	24/11	95.00 - 97.00	23/24/25/26	49	79		438.9		95.0 Grey, wet, very dense, Silty SAND, little gravel, (Glacial Till).	
100											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: St. Mary's Bridge #5309 carries Castonguay Road over Violette Brook
Location: Van Buren, Maine

Boring No.: BB-VBVB-102
WIN: 26083.00

Driller:	MaineDOT	**Elevation (ft.)**	533.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay/B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/24/2024, 7/2/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	17+12, 7.0 ft Lt.	**Casing ID/OD:**	HW(4/4.5"), NW(3/3.5")	**Water Level*:**	8.5 ft bgs.
Hammer Efficiency Factor:	0.962	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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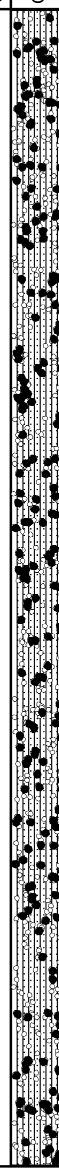
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
100										Drilling slowed significantly at approximately 100 ft bgs. 700-1000 psi down pressure required to advance hole.	
										Occasional cobble.	
105	13D	10.8/5	105.00 - 105.90	37/64(4.8")	---					Grey, wet, very dense, SAND, some silt, some gravel, (Glacial Till).	
110											
115	14D	12/7	115.00 - 116.00	33/70	---					Similar to 13D.	
										Occasional cobble.	
120											
125											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: St. Mary's Bridge #5309 carries Castonguay Road over Violette Brook
Location: Van Buren, Maine

Boring No.: BB-VBVB-102
WIN: 26083.00

Driller:	MaineDOT	**Elevation (ft.)**	533.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay/B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/24/2024, 7/2/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	17+12, 7.0 ft Lt.	**Casing ID/OD:**	HW(4/4.5"), NW(3/3.5")	**Water Level*:**	8.5 ft bgs.
Hammer Efficiency Factor:	0.962	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
125										Artisan water pressure at 125.0 ft bgs. Occasional cobble.	
130											
135											
								394.2		139.7	
140	R1	43.2/43.2	140.20 - 143.80	RQD = 20%			NQ-2			Top of Bedrock at Elev. 394.2 ft. Roller coned ahead to 140.2 ft bgs. R1: Bedrock: Grey to dark grey, very fine to fine-grained, PHYLLITE, moderately hard, slightly weathered joint faces, joints dipping at steep to vertical angles, closely spaced, with fractured zones. [New Sweden Formation] Rock Quality = Very Poor R1: Core Times: (min:sec) 140.2-141.2 ft (4:14) 141.2-142.2 ft (5:59) 142.2-143.2 ft (5:04) 143.2-143.8 ft (7:33) core blocked 100% Recovery	
	R2	20.4/17	143.80 - 145.50	RQD = 0%							
145	R3	60/60	145.50 - 150.50	RQD = 26%						R2: Bedrock: Grey to dark grey, very fine to fine-grained, PHYLLITE, moderately hard, slightly weathered, highly fractured. [New Sweden Formation] Rock Quality = Very Poor R2: Core Times (min:sec)	
150											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: St. Mary's Bridge #5309 carries Castonguay Road over Violette Brook
Location: Van Buren, Maine

Boring No.: BB-VBVB-102
WIN: 26083.00

Driller:	MaineDOT	**Elevation (ft.)**	533.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Andrle	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay/B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/24/2024, 7/2/2024	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	17+12, 7.0 ft Lt.	**Casing ID/OD:**	HW(4/4.5"), NW(3/3.5")	**Water Level*:**	8.5 ft bgs.
Hammer Efficiency Factor:	0.962	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
150								383.4		143.8-144.2 ft (1:48) 144.2-145.2 ft (3:11) 145.2-145.5 ft (5:27) core blocked 83% Recovery R3: Bedrock: Grey to dark grey, very fine to fine-grained, PHYLLITE, moderately hard, slightly weathered joint faces, joints dipping at steep to moderate angles, closely spaced. [New Sweden Formation] Rock Quality = Poor R3: Core Times (min:sec) 145.5-146.5 ft (3:19) 146.5-147.5 ft (2:52) 147.5-148.5 ft (3:02) 148.5-149.5 ft (2:41) 149.5-150.5 ft (3:43) 100% Recovery 150.5 **Bottom of Exploration at 150.5 feet below ground surface.**	
155											
160											
165											
170											
175											

Remarks:

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

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

Appendix B

Rock Core Photographs

MaineDOT

St. Mary's Bridge #5309 Carries Castonguay Rd. Over Violette Brook

Van Buren, ME

Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-VBVB-101	R1	134.9-137.9	36	24	0	0	PHYLLITE	1
BB-VBVB-101	R2	137.9-142.9	60	59	12	20	PHYLLITE	2
BB-VBVB-101	R3	142.9-145.9	36	36	21	58	PHYLLITE	3

Notes: 1. "Box row" indicates the section of the box where the core run is contained: 1 = top, 3 = bottom.
2. Top of each core run is on the left and increases with depth to the right.

MaineDOT

St. Mary's Bridge #5309 Carries Castonguay Rd. Over Violette Brook

Van Buren, ME

Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-VBVB-102	R1	140.2-143.8	43.2	43.2	9	20	PHYLLITE	1
BB-VBVB-102	R2	143.8-145.5	20.4	17	0	0	PHYLLITE	2
BB-VBVB-102	R3	145.5-150.5	60	60	16	26	PHYLLITE	4

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

Appendix C

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Van Buren Work Number: 26083.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification Unified	AASHTO	Frost
BB-VBVB-101, 1D	16+58	9.0 Rt.	5.0-7.0	380935	1	5.6			SM	A-1-b	II
BB-VBVB-101, 3D	16+58	9.0 Rt.	15.0-17.0	380936	1	10.1			GM	A-1-b	I
BB-VBVB-101, 12D	16+58	9.0 Rt.	60.0-62.0	380937	1	10.0			SC-SM	A-4	III
BB-VBVB-101, 17D	16+58	9.0 Rt.	85.0-87.0	380938	1	11.7			ML	A-4	IV
BB-VBVB-101, 24D	16+58	9.0 Rt.	125.0-125.4	380939	1	9.3			SM	A-2-4	II

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

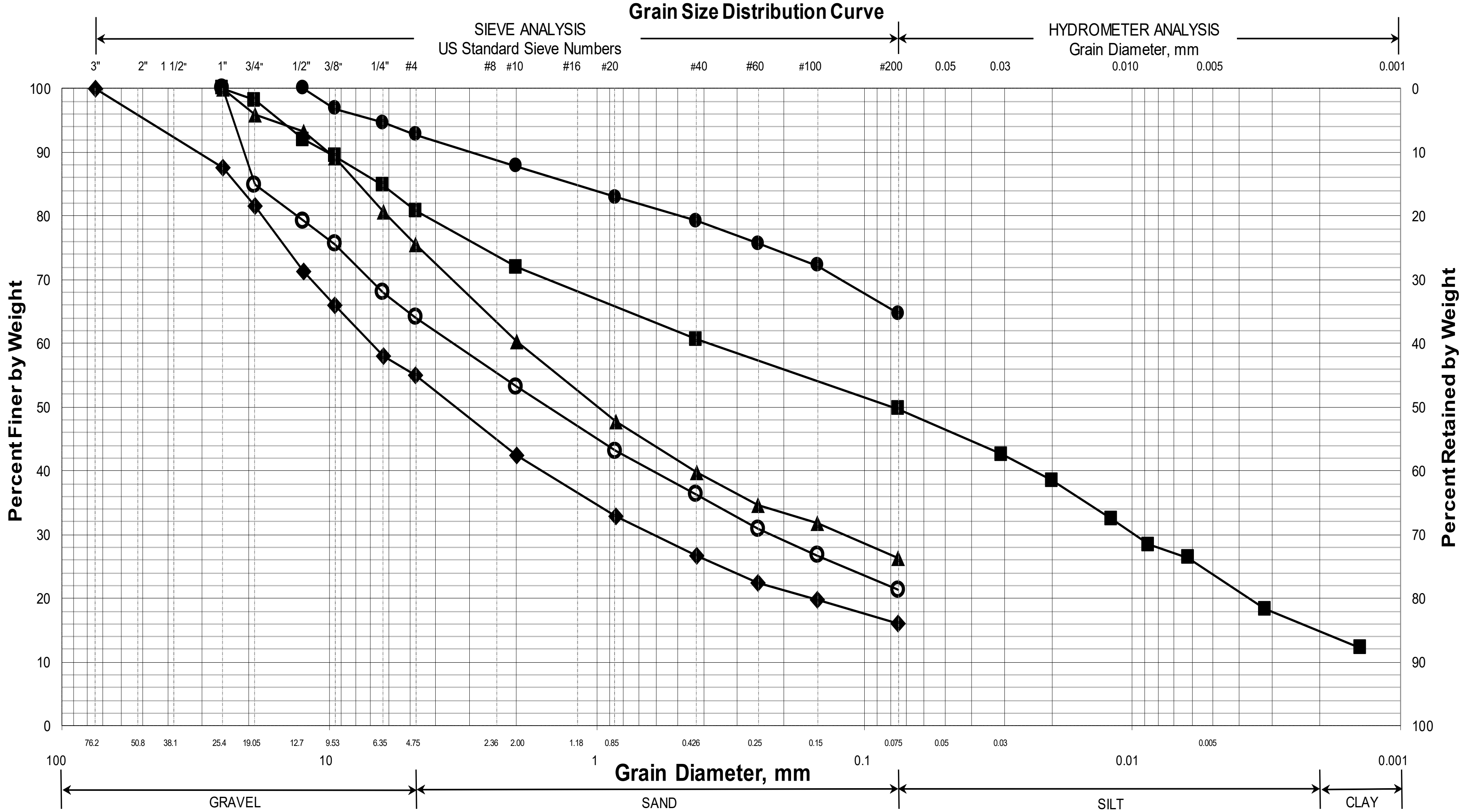
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

Maine Department of Transportation
Grain Size Distribution Curve



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-VBVB-101/1D	16+58	9.0 RT	5.0-7.0	SAND, some gravel, some silt.	5.6			
◆	BB-VBVB-101/3D	16+58	9.0 RT	15.0-17.0	Sandy GRAVEL, little silt.	10.1			
■	BB-VBVB-101/12D	16+58	9.0 RT	60.0-62.0	SILT, some sand, little gravel, little clay.	10			
●	BB-VBVB-101/17D	16+58	9.0 RT	85.0-87.0	SILT, some sand, trace gravel.	11.7			
▲	BB-VBVB-101/24D	16+58	9.0 RT	125.0-125.4	SAND, some silt, some gravel	9.3			
×									

WIN	
026083.00	
Town	
Van Buren	
Reported by/Date	
WHITE, TERRY A	2/22/2024

SHEET 1

Appendix D

Calculations

Driven H-Pile Resistance

Design of H-piles

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

Bedrock Properties

BB-VBVB-101, R1 RQD = 0%, R2 RQD = 20%, R3 RQD = 58%
Rock Type: PHYLLITE (moderately hard, slightly weathered to fresh)

BB-VBVB-102, R1 RQD = 20%, R2 RQD = 0%, R3 RQD = 26%
Rock Type: PHYLLITE (moderately hard, slightly weathered)

Siltstone Co = 3,500-35,000 psi

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

For Design Purposes: RQD = 20%, Co = 5,000 psi

Pile Properties

Use the following piles: 14x89, 14x117

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

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

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

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

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

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

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

Check For Slender Members

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

LRFD eq. 6.9.4.2.1-1

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

From Table 6.9.4.2.1-1:

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

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

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

For 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 P_e, LRFD eq. 6.9.4.1.2-1

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

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

LRFD eq. 6.9.4.1.2-1

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

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

$$\frac{P_o}{P_e} = \begin{pmatrix} 8.529 \times 10^{-6} \\ 8.247 \times 10^{-6} \end{pmatrix}$$

If Po/Pe < or = 2.25, then:

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

LRFD Eq. 6.9.4.1.1-1

then:

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

Factored Axial Structural Resistance for the Strength Limit State

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

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

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

LRFD 10.7.3.2.3 - Piles Driven to Hard Rock -

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

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

Nominal Structural Resistance Previously Calculated:

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

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

$$\phi_c := 0.5$$

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

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

14x89
14x117

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

$$\phi_c := 1.0$$ LRFD 6.5.5

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

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

14x89
14x117

Drivability Analyses

Ref: LRFD Article 10.7.8

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

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

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

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

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

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

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

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

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

GRLWeap Soil and Pile Model Assumptions

Abutment #1:
Based on a proposed bottom of footing of elevation 523.8 at Abutment No. 1, and assuming the pile are driven to bedrock at elevation 399.4, the estimated pile length will be approximately 125 feet. Assume the contractor drives 130-foot pile (extra length accommodates for attachment of dynamic testing equipment, embedment into the abutment and variation in the bedrock surface).

Use constant shaft resistances so that GRLWeap will assign approxately 150 kips as skin friction for an HP14x89 pile and 160 kips for an HP14x117 pile, based on local experience in similar deposits.

Abutment #2:
Based on a proposed bottom of footing of elevation 524.1 at Abutment No. 2, and assuming the pile are driven to bedrock at elevation 394.2, the estimated pile length will be approximately 130 feet. Assume the contractor drives 135-foot pile (extra length accommodates for attachment of dynamic testing equipment, embedment into the abutment and variation in the bedrock surface).

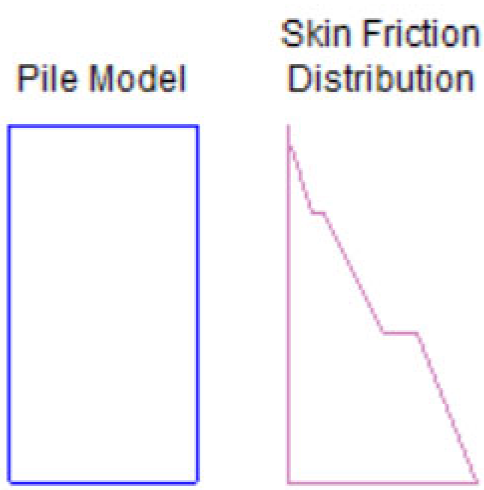
Use constant shaft resistances so that GRLWeap will assign approxately 160 kips as skin friction for an HP14x89 pile and 170 kips for an HP14x117 pile, based on local experience in similar deposits.

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

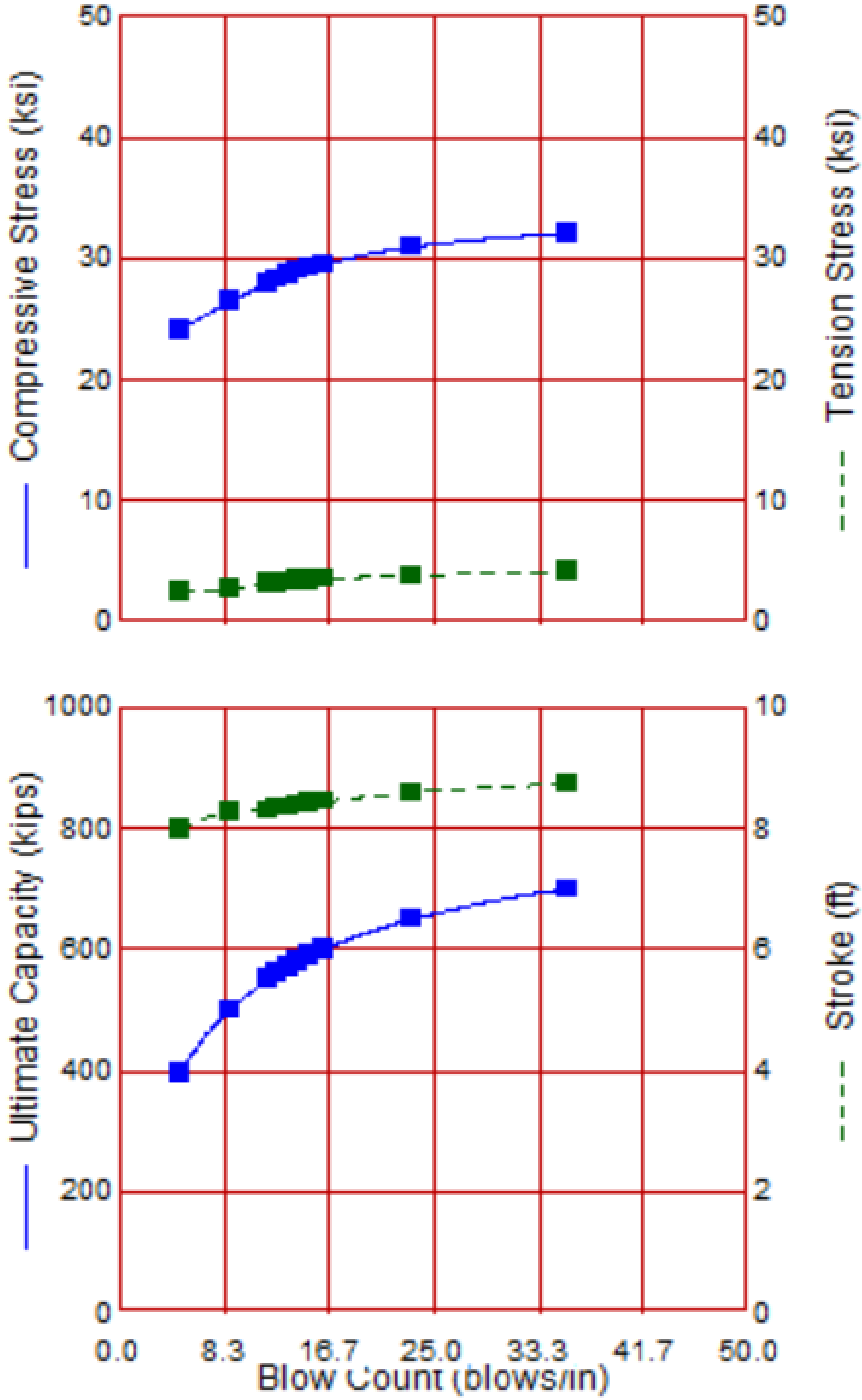
The 14x89 pile can be driven to the resistances below with an APE D19-42 hammer at fuel setting 4 (100% of Max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE D 19-42

Ram Weight	4.19	kips
Efficiency	0.800	
Pressure	1710 (100%)	psi
Helmet Weight	3.00	kips
Hammer Cushic	34825	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.070	in
Skin Damping	0.100	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	130.00	ft
Pile Penetration	124.40	ft
Pile Top Area	26.10	in2



Res. Shaft = 150.1 kips
(Constant Res. Shaft)


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

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
395.0	24.05	2.50	4.8	7.98	22.12
500.0	26.38	2.73	8.8	8.27	22.85
550.0	27.95	3.14	11.8	8.31	23.00
560.0	28.32	3.22	12.6	8.34	23.09
570.0	28.60	3.29	13.4	8.36	23.12
580.0	28.97	3.36	14.1	8.40	23.27
590.0	29.26	3.43	15.1	8.43	23.32
600.0	29.53	3.50	16.2	8.45	23.36
650.0	30.91	3.81	23.2	8.60	23.77
700.0	32.05	4.09	35.7	8.74	24.16

Limit to 15 bpi

$R_{ndr} := 580 \cdot kip$

Strength Limit State

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

$R_{fdr} = 377 \cdot kip$

Extreme and
Service Limit States

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

$R_{dr} = 580 \cdot kip$

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

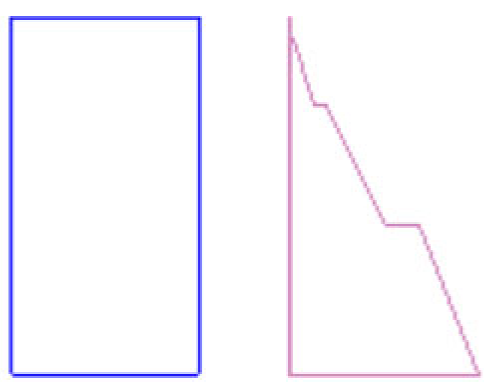
The 14x89 pile can be driven to the resistances below with a APE D25-42 hammer at fuel setting 4 (100% of max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE D 25-42

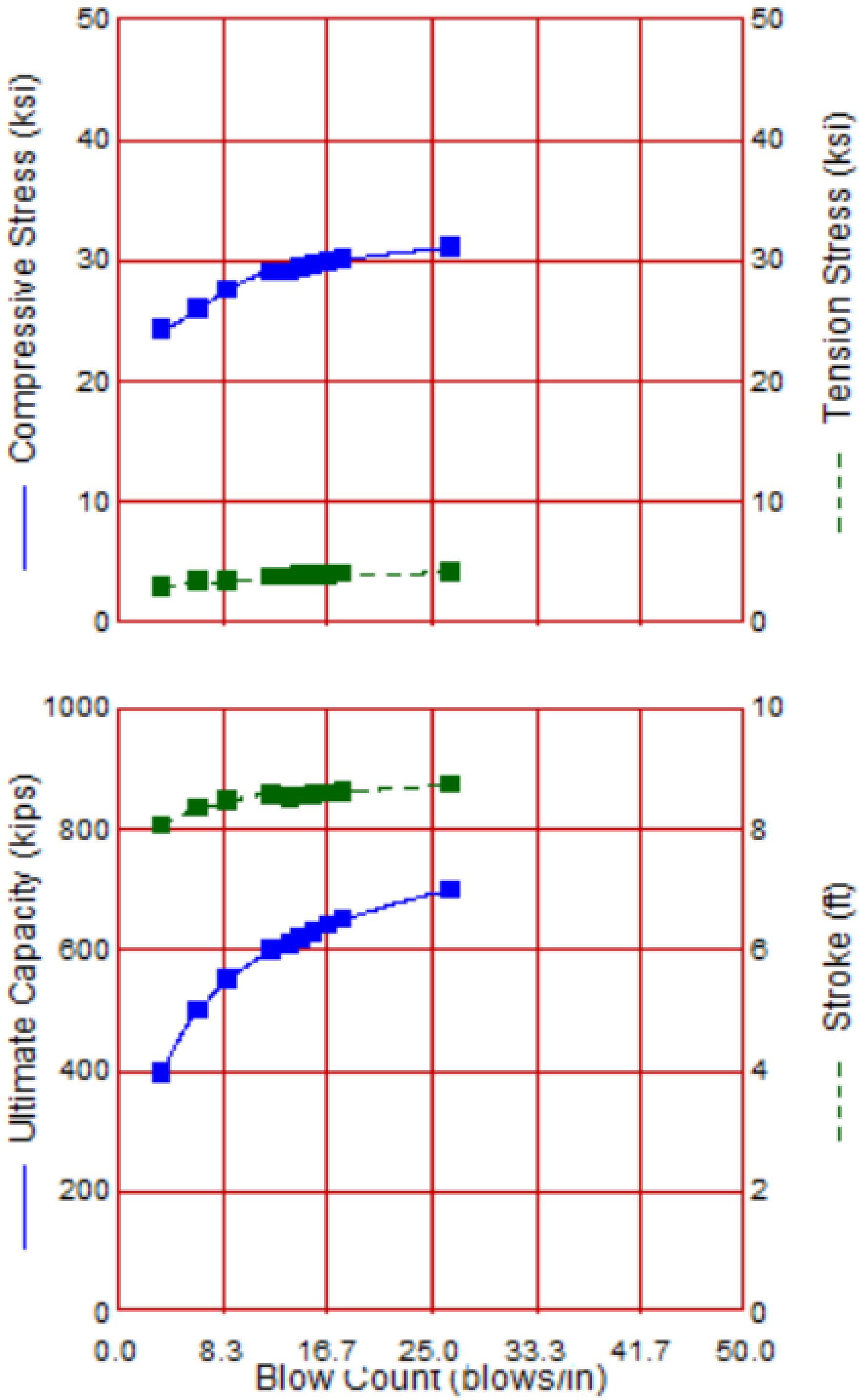
Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1425 (100%)	psi
Helmet Weight	3.00	kips
Hammer Cushic	34825	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.070	in
Skin Damping	0.100	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	130.00	ft
Pile Penetration	124.40	ft
Pile Top Area	26.10	in2

Pile Model

Skin Friction Distribution



Res. Shaft = 150.1 kips
(Constant Res. Shaft)


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

Maine DOT 17-Jul-2025
26083 Van Buren 14x89 ABT #1 D25-42 GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
395.0	24.28	2.91	3.6	8.05	26.61
500.0	25.95	3.39	6.4	8.34	27.72
550.0	27.59	3.44	8.7	8.46	28.14
600.0	29.02	3.73	12.3	8.58	28.59
610.0	28.99	3.78	13.8	8.51	28.33
620.0	29.32	3.83	14.7	8.55	28.49
630.0	29.60	3.88	15.7	8.57	28.56
640.0	29.83	3.93	16.9	8.60	28.63
650.0	30.06	3.98	18.0	8.62	28.77
700.0	31.12	4.18	26.5	8.74	29.20

Limit to 15 bpi

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

Strength Limit State

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

$R_{fdr} = 403 \cdot kip$

Extreme and
Service Limit States

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

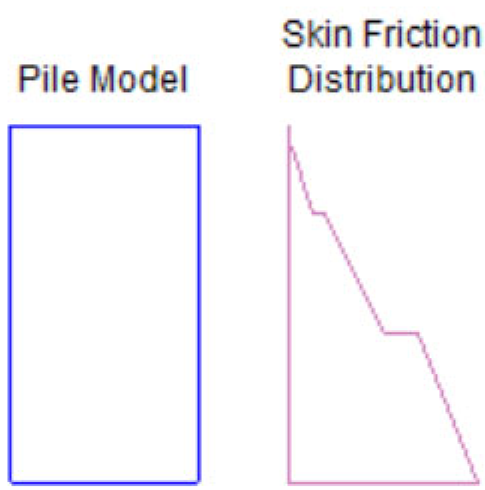
$R_{dr} = 620 \cdot kip$

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

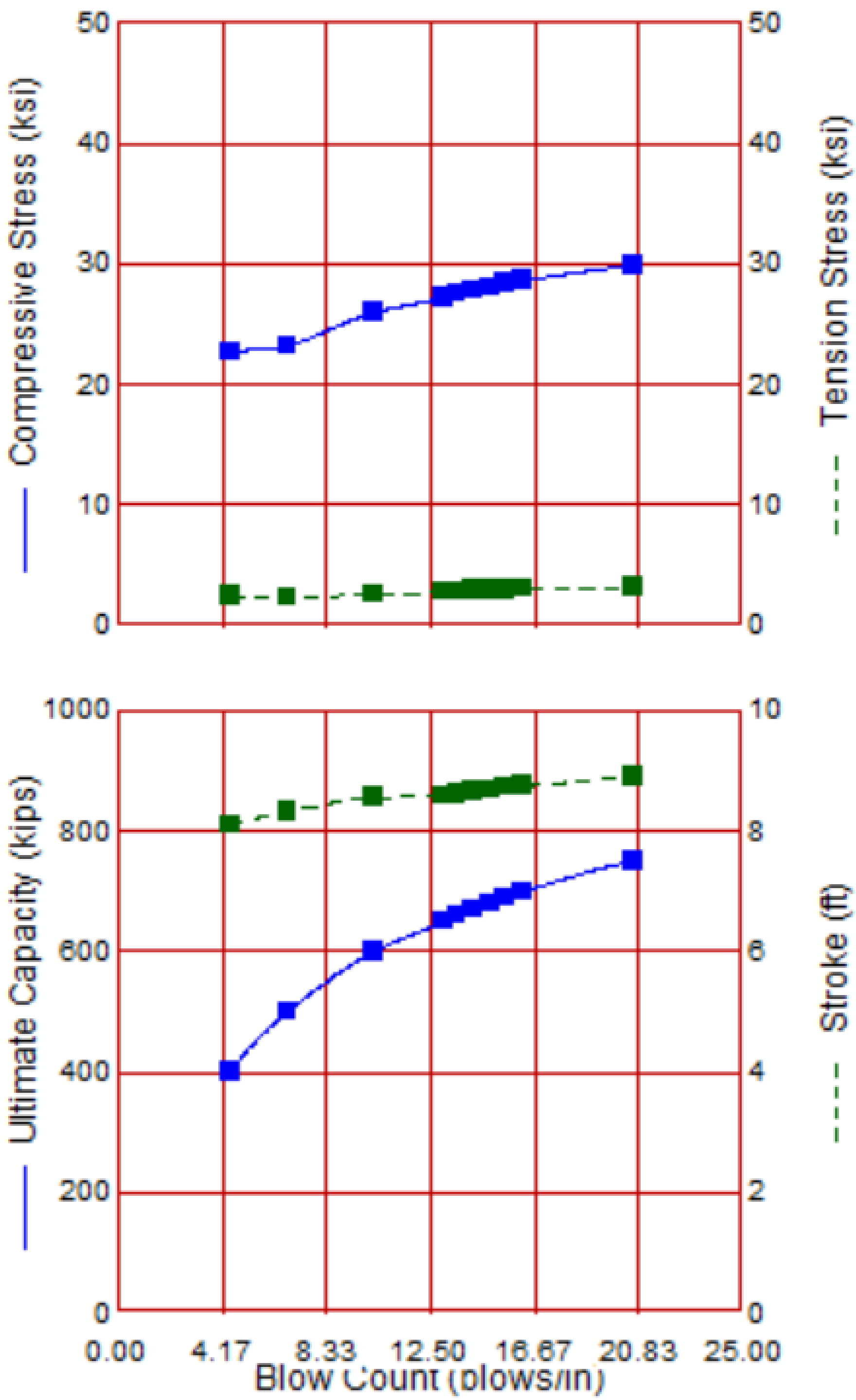
The 14x117 pile can be driven to the resistances below with a APE D19-42 hammer at fuel setting 4 (100% of max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE D 19-42

Ram Weight	4.19	kips
Efficiency	0.800	
Pressure	1710 (100%)	psi
Helmet Weight	3.00	kips
Hammer Cushic	34825	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.070	in
Skin Damping	0.100	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	130.00	ft
Pile Penetration	124.40	ft
Pile Top Area	34.40	in2



Res. Shaft = 160.0 kips
(Constant Res. Shaft)


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

Maine DOT — 17-Jul-2025
26083 Van Buren 14x117 ABT #1 D19-42 — GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	22.70	2.38	4.5	8.10	20.76
500.0	23.17	2.35	6.8	8.33	21.40
600.0	25.93	2.54	10.3	8.56	22.03
650.0	27.19	2.77	13.1	8.60	22.10
660.0	27.49	2.84	13.6	8.63	22.23
670.0	27.78	2.89	14.2	8.66	22.31
680.0	28.04	2.94	14.9	8.69	22.37
690.0	28.35	2.99	15.5	8.73	22.51
700.0	28.60	3.03	16.2	8.75	22.58
750.0	29.84	3.18	20.6	8.90	22.98

Limit to 15 bpi

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

Strength Limit State

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

$R_{fdr} = 442 \cdot kip$

Extreme and
Service Limit States

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

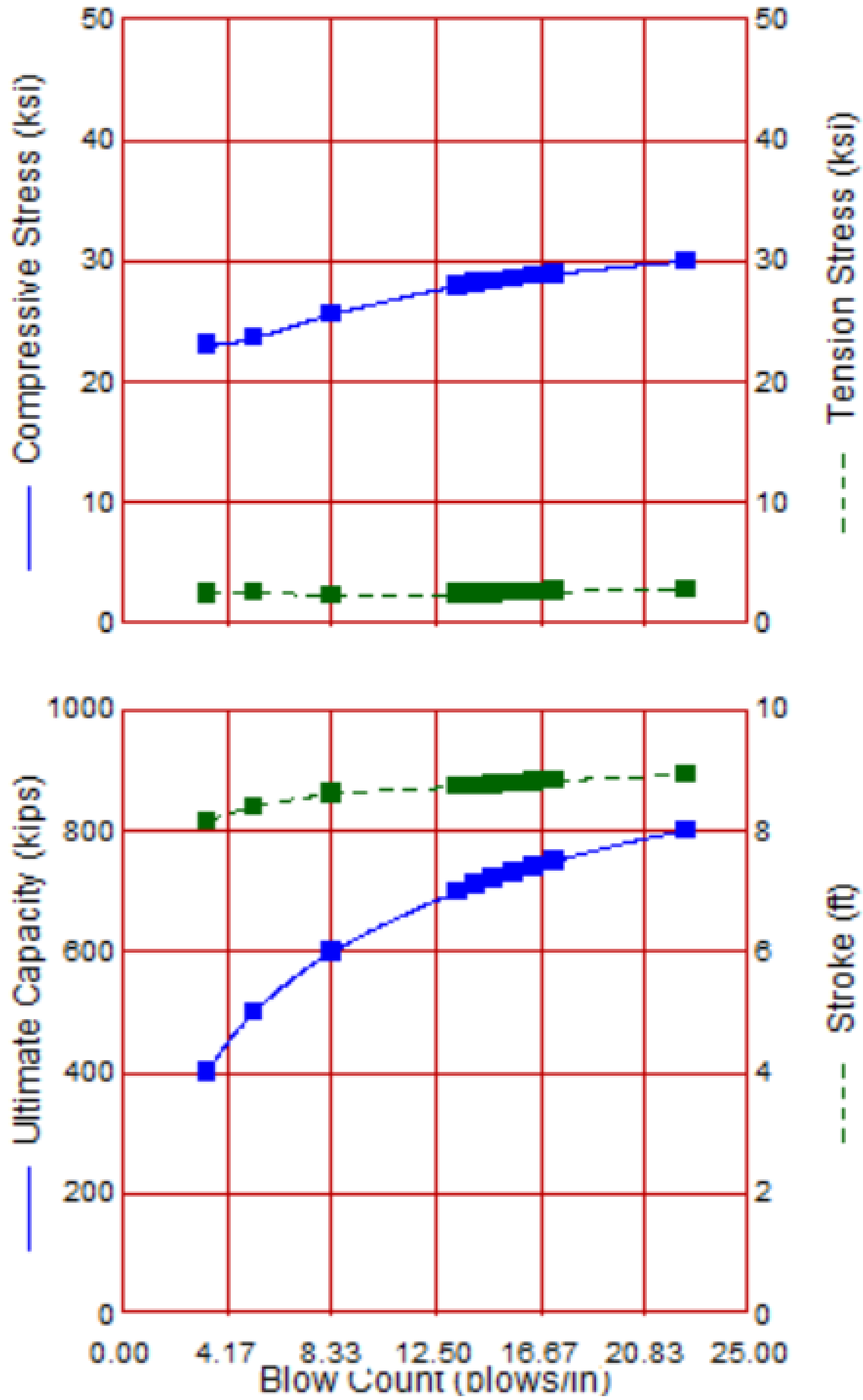
$R_{dr} = 680 \cdot kip$

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

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

APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1425 (100%)	psi
Helmet Weight	3.00	kips
Hammer Cushic	34825	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.070	in
Skin Damping	0.100	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	130.00	ft
Pile Penetration	124.40	ft
Pile Top Area	34.40	in2



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

Maine DOT
26083 Van Buren 14x117 ABT #1 D25-42

17-Jul-2025
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	22.99	2.46	3.4	8.16	24.63
500.0	23.61	2.55	5.3	8.40	25.47
600.0	25.57	2.36	8.4	8.61	26.22
700.0	27.91	2.41	13.4	8.73	26.66
710.0	28.13	2.45	14.1	8.74	26.71
720.0	28.32	2.49	14.8	8.76	26.75
730.0	28.50	2.54	15.6	8.78	26.79
740.0	28.73	2.59	16.4	8.80	26.84
750.0	28.91	2.64	17.2	8.83	26.94
800.0	29.95	2.83	22.5	8.93	27.33

Limit to 15 bpi

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

Strength Limit State

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

$R_{fdr} = 468 \cdot kip$

Extreme and
Service Limit States

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

$R_{dr} = 720 \cdot kip$

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

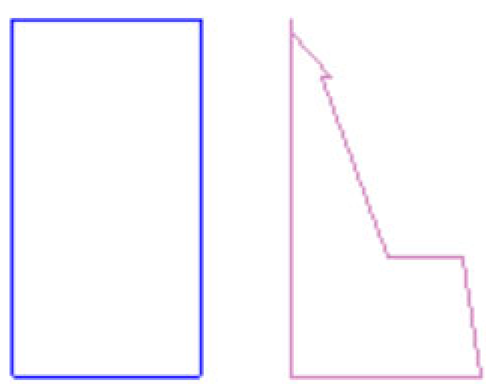
The 14x89 pile can be driven to the resistances below with a APE D19-42 hammer at fuel setting 4 (100% of max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE D 19-42

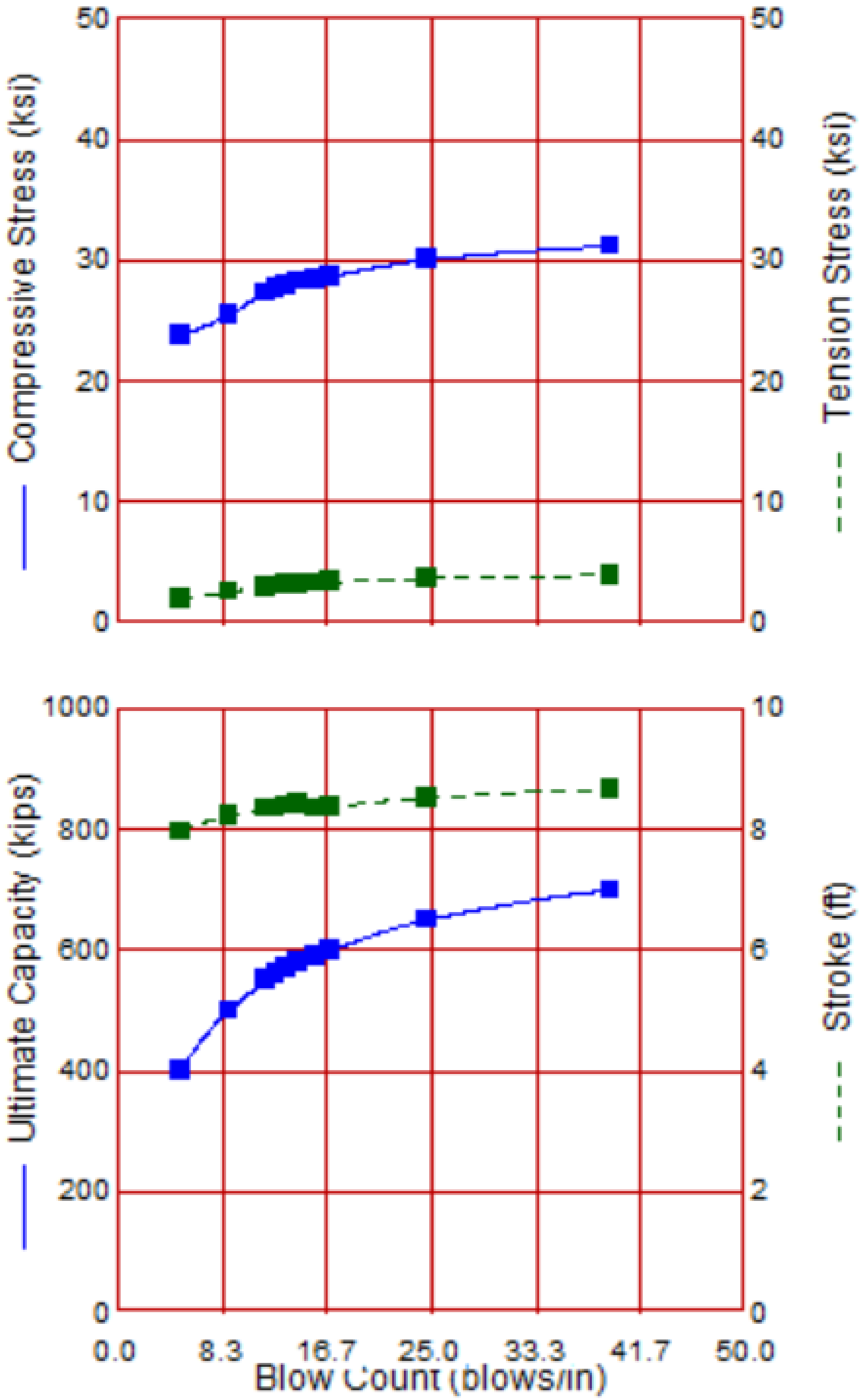
Ram Weight	4.19	kips
Efficiency	0.800	
Pressure	1710 (100%)	psi
Helmet Weight	3.00	kips
Hammer Cushic	34825	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.070	in
Skin Damping	0.100	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	135.00	ft
Pile Penetration	129.90	ft
Pile Top Area	26.10	in2

Pile Model

Skin Friction
Distribution



Res. Shaft = 160.0 kips
(Constant Res. Shaft)


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

Maine DOT
26083 Van Buren 14x89 ABT #2 D19-42

17-Jul-2025
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	23.80	2.01	5.0	7.96	21.95
500.0	25.51	2.53	8.9	8.23	22.68
550.0	27.31	2.99	11.9	8.34	23.05
560.0	27.62	3.07	12.7	8.36	23.09
570.0	27.95	3.15	13.5	8.40	23.18
580.0	28.24	3.22	14.4	8.42	23.23
590.0	28.33	3.29	15.9	8.36	23.04
600.0	28.66	3.35	16.9	8.38	23.14
650.0	30.03	3.67	24.6	8.53	23.54
700.0	31.19	3.94	39.3	8.67	23.92

Limit to 15 bpi

$R_{ndr} := 580 \cdot kip$

Strength Limit State

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

$R_{fdr} = 377 \cdot kip$

Extreme and
Service Limit States

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

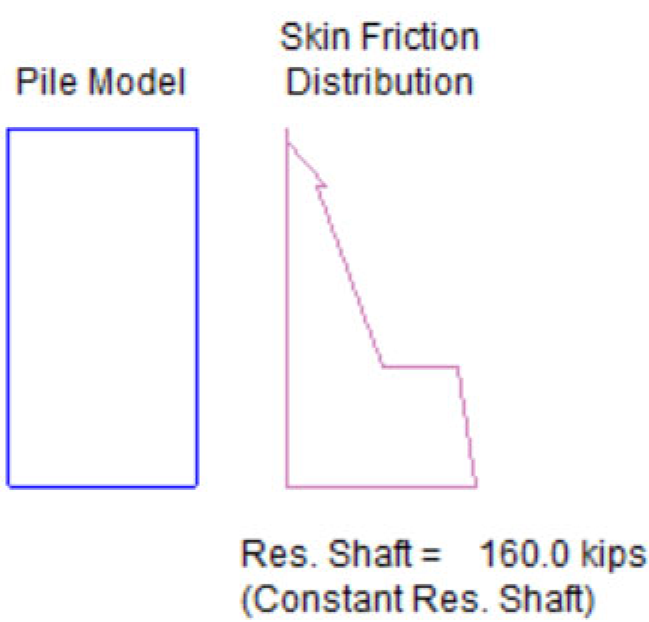
$R_{dr} = 580 \cdot kip$

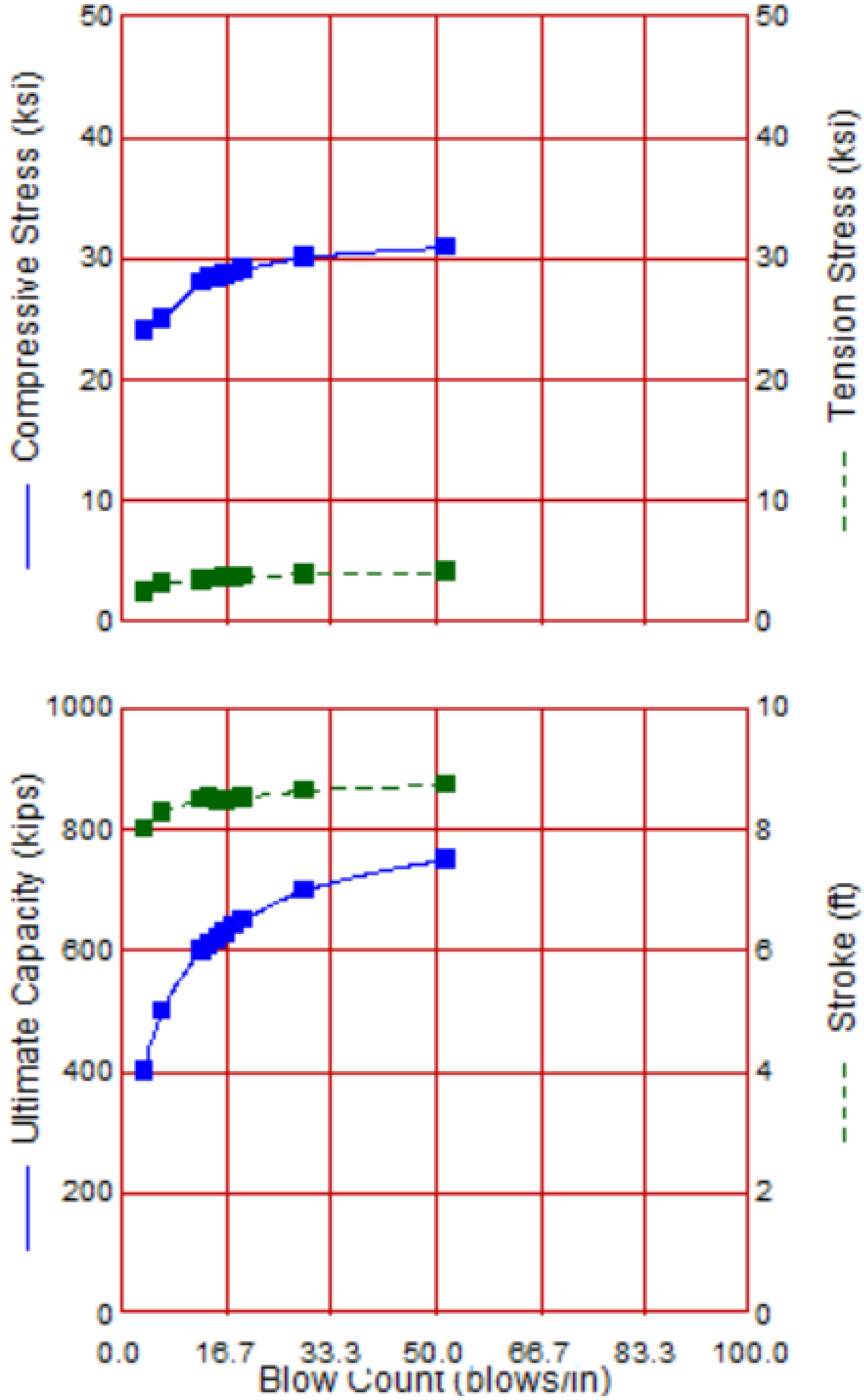
Abutment 2, Pile Size is 14 x 89, APE D25-42 Hammer

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

APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1425 (100%)	psi
Helmet Weight	3.00	kips
Hammer Cushic	34825	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.070	in
Skin Damping	0.100	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	135.00	ft
Pile Penetration	129.90	ft
Pile Top Area	26.10	in2




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

Maine DOT — 17-Jul-2025
26083 Van Buren 14x89 ABT #2 D25-42 — GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	23.96	2.51	3.8	8.00	26.40
500.0	24.99	3.12	6.6	8.27	27.41
600.0	28.08	3.47	13.0	8.50	28.26
610.0	28.38	3.54	14.0	8.52	28.37
620.0	28.31	3.58	15.6	8.46	28.11
630.0	28.59	3.63	16.7	8.48	28.23
640.0	28.81	3.68	18.0	8.50	28.29
650.0	29.06	3.73	19.4	8.53	28.37
700.0	30.10	3.95	29.4	8.64	28.81
750.0	30.90	4.14	51.7	8.74	29.19

Limit to 15 bpi

$$R_{ndr} := 610 \cdot kip$$

Strength Limit State

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

$$R_{fdr} = 397 \cdot kip$$

Extreme and
Service Limit States

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

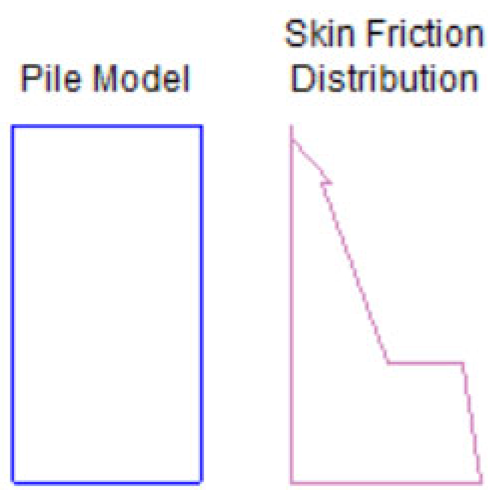
$$R_{dr} = 610 \cdot kip$$

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

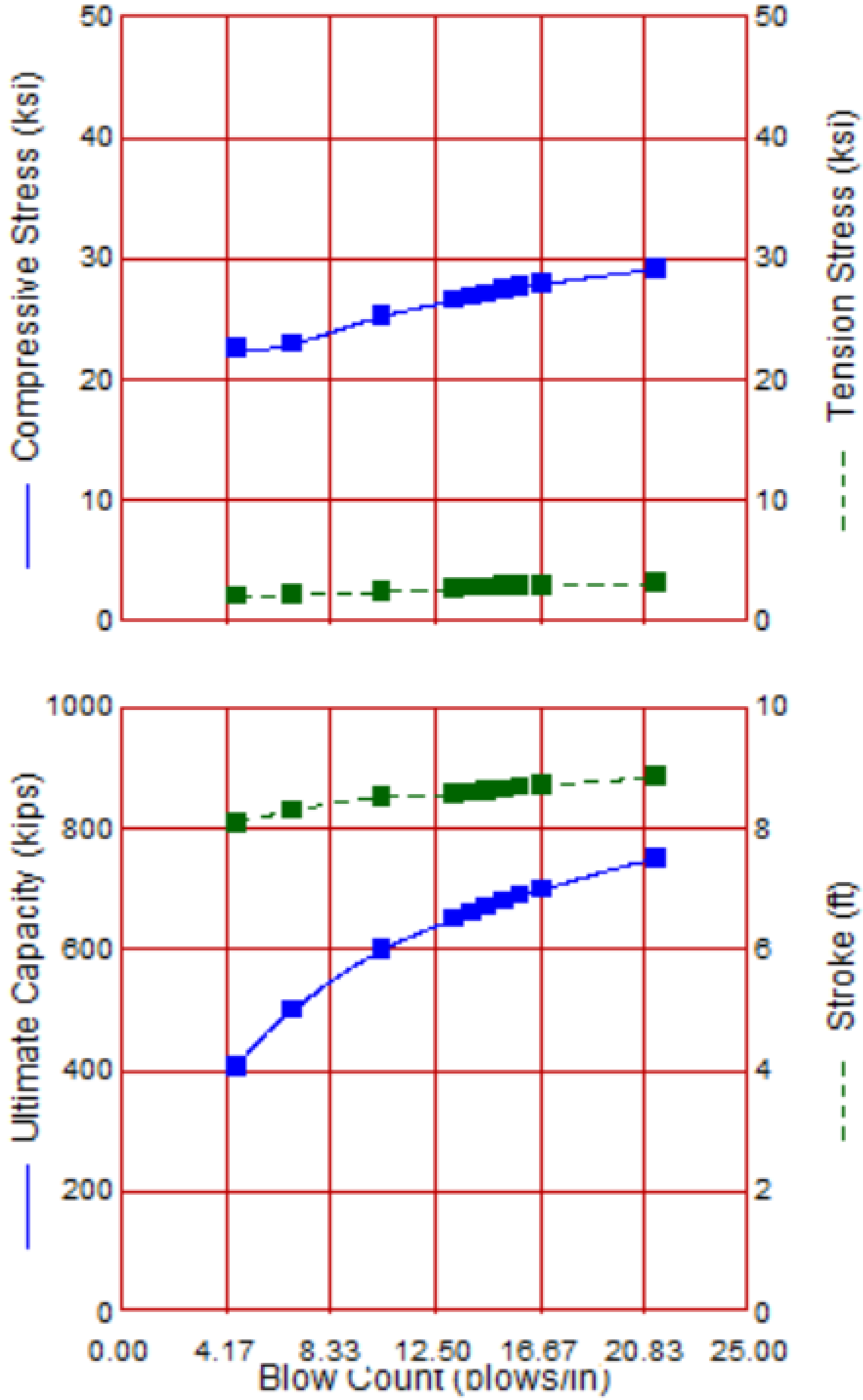
The 14x117 pile can be driven to the resistances below with a APE D19-42 hammer at fuel setting 4 (100% of max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE D 19-42

Ram Weight	4.19	kips
Efficiency	0.800	
Pressure	1710 (100%)	psi
Helmet Weight	3.00	kips
Hammer Cushic	34825	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.070	in
Skin Damping	0.100	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	135.00	ft
Pile Penetration	129.90	ft
Pile Top Area	34.40	in2



Res. Shaft = 170.1 kips
(Constant Res. Shaft)



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

Maine DOT 17-Jul-2025
26083 Van Buren 14x117 ABT #2 D19-42 GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
405.0	22.50	2.06	4.6	8.09	20.71
500.0	22.95	2.17	6.8	8.30	21.32
600.0	25.24	2.43	10.4	8.53	21.89
650.0	26.55	2.72	13.3	8.56	22.01
660.0	26.81	2.77	13.9	8.60	22.07
670.0	27.11	2.82	14.6	8.63	22.15
680.0	27.38	2.87	15.3	8.65	22.23
690.0	27.68	2.92	15.9	8.69	22.36
700.0	27.92	2.95	16.8	8.71	22.38
750.0	29.11	3.10	21.3	8.85	22.80

Limit to 15 bpi

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

Strength Limit State

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

$R_{fdr} = 436 \cdot kip$

Extreme and
Service Limit States

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

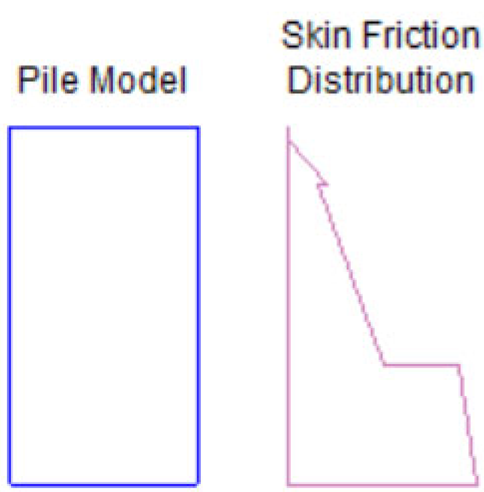
$R_{dr} = 670 \cdot kip$

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

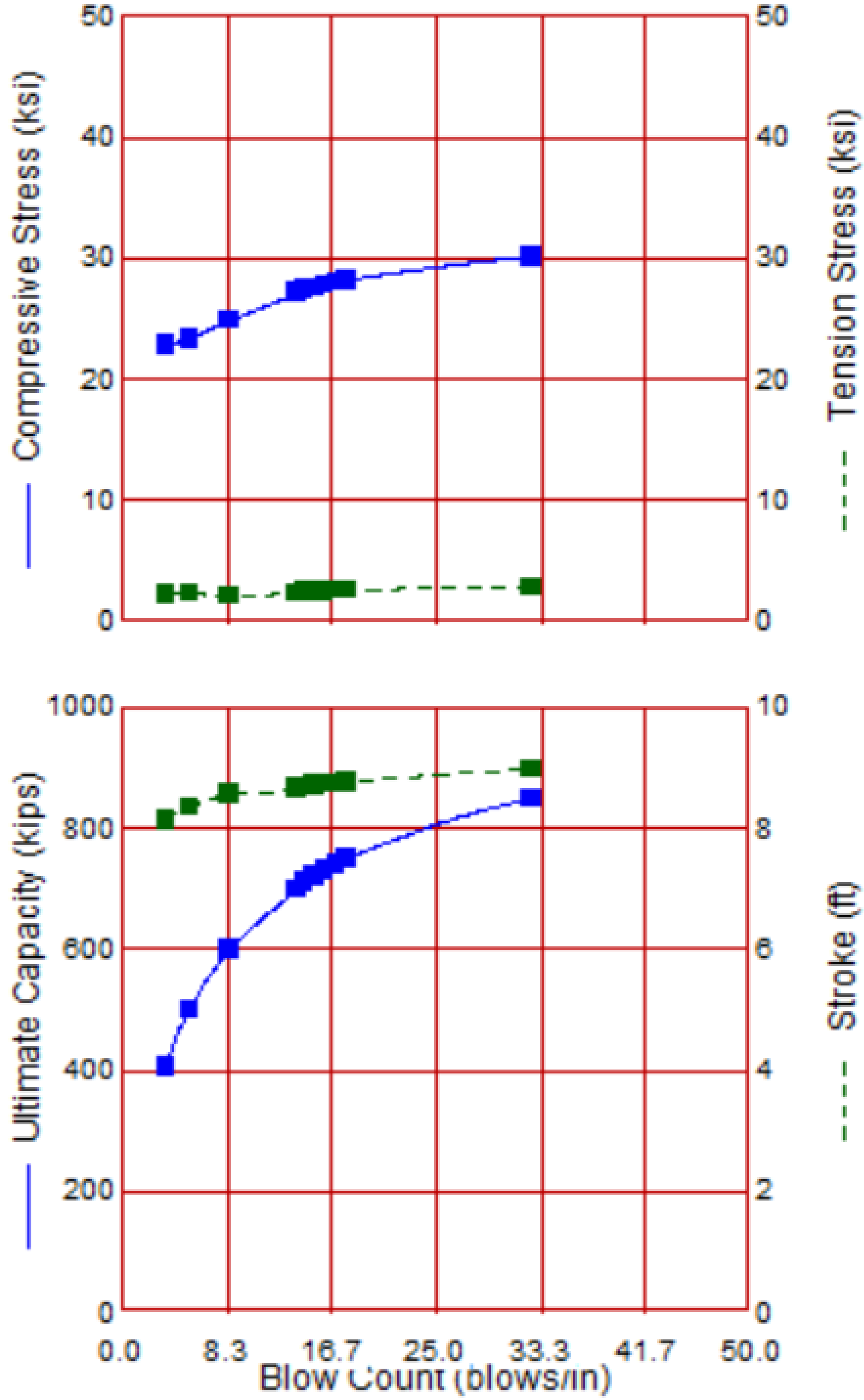
The 14x117 pile can be driven to the resistances below with a APE D25-42 hammer at fuel setting 4 (100% of max) and 3.0 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1425 (100%)	psi
Helmet Weight	3.00	kips
Hammer Cushic	34825	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.070	in
Skin Damping	0.100	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	135.00	ft
Pile Penetration	129.90	ft
Pile Top Area	34.40	in2



Res. Shaft = 170.1 kips
(Constant Res. Shaft)


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

Maine DOT
26083 Van Buren 14x117 ABT #2 D25-42

17-Jul-2025
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
405.0	22.82	2.20	3.5	8.14	24.51
500.0	23.34	2.29	5.4	8.36	25.26
600.0	24.86	2.08	8.6	8.57	26.00
700.0	27.13	2.35	14.0	8.66	26.33
710.0	27.38	2.41	14.5	8.69	26.49
720.0	27.58	2.46	15.3	8.71	26.53
730.0	27.84	2.51	16.2	8.73	26.58
740.0	28.04	2.55	17.1	8.74	26.62
750.0	28.21	2.60	17.9	8.76	26.74
850.0	30.14	2.83	32.6	8.98	27.53

Limit to 15 bpi

$R_{ndr} := 710 \cdot kip$

Strength Limit State

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

$R_{fdr} = 462 \cdot kip$

Extreme and
Service Limit States

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

$R_{dr} = 710 \cdot kip$

26083 Van Burren - St. Mary's Bridge #5309

GRL WEAP INPUT + RESULT SUMMARY

Created By: NPP 7/17/25

Checked By: LK 7/28/25

	Abutment	Pile Size	Pile Length (ft)	Pile Penetration (ft)	Hammer	Fuel Setting	Shaft Quake (in)	Toe Quake (in)	Shaft Damping (s/ft)	Toe Damping (s/ft)	Skin Friction (kips)	Ultimate Capacity (kips)	Max Comp Stress (ksi)	Max Tension Stress (ksi)	Blows/In	Stroke (ft)	Energy (kips-ft)
Abutment #1 14x89 APE D19-42	1	HP 14x89	95	90	APE D19-42	3	0.10	0.10	0.10	0.15	150	540	25.70	3.55	14.5	7.75	19.89
	1	HP 14x89	95	90	APE D19-42	3	0.10	0.20	0.10	0.15	150	480	23.12	2.93	13.8	7.57	19.54
	1	HP 14x89	95	90	APE D19-42	3	0.10	0.30	0.10	0.15	150	440	22.66	1.86	14.0	7.37	19.15
	1	HP 14x89	95	90	APE D19-42	3	0.13	0.33	0.13	0.07	150	460	22.79	3.72	15.0	7.48	19.39
	1	HP 14x89	95	90	APE D19-42	4	0.10	0.10	0.10	0.15	150	590	29.14	4.02	14.6	8.71	23.43
	1	HP 14x89	95	90	APE D19-42	4	0.10	0.20	0.10	0.15	150	520	24.90	3.28	14.0	8.38	22.71
	1	HP 14x89	95	90	APE D19-42	4	0.10	0.30	0.10	0.15	150	480	24.60	1.78	14.5	8.26	22.49
	1	HP 14x89	95	90	APE D19-42	4	0.13	0.33	0.13	0.07	150	500	24.71	3.83	14.6	8.36	22.72
	1	HP 14x89	130	124.4	APE D19-42	4	0.10	0.04	0.10	0.15	150	600	29.98	3.76	14.5	8.49	23.50
	1	HP 14x89	130	124.4	APE D19-42	4	0.10	0.07	0.10	0.15	150	580	28.97	3.36	14.1	8.40	23.27
Abutment #1 14x89 APE D25-42	1	HP 14x89	95	90	APE D25-42	3	0.10	0.10	0.10	0.15	150	580	25.85	5.08	14.7	8.03	23.93
	1	HP 14x89	95	90	APE D25-42	3	0.10	0.20	0.10	0.15	150	530	23.66	3.46	14.9	7.75	23.02
	1	HP 14x89	95	90	APE D25-42	3	0.10	0.30	0.10	0.15	150	490	23.11	1.81	14.2	7.57	22.58
	1	HP 14x89	95	90	APE D25-42	3	0.13	0.33	0.13	0.07	150	510	23.27	2.47	14.7	7.67	22.99
	1	HP 14x89	95	90	APE D25-42	4	0.10	0.10	0.10	0.15	150	640	29.93	5.89	14.6	9.06	29.16
	1	HP 14x89	95	90	APE D25-42	4	0.10	0.20	0.10	0.15	150	590	26.36	4.18	14.9	8.79	28.43
	1	HP 14x89	95	90	APE D25-42	4	0.10	0.30	0.10	0.15	150	540	25.54	2.26	14.1	8.48	27.52
	1	HP 14x89	95	90	APE D25-42	4	0.13	0.33	0.13	0.07	150	570	25.82	3.18	14.7	8.64	28.17
	1	HP 14x89	130	124.4	APE D25-42	4	0.10	0.04	0.10	0.15	150	630	30.42	4.41	14.1	8.62	28.74
	1	HP 14x89	130	124.4	APE D25-42	4	0.10	0.07	0.10	0.15	150	620	29.32	3.83	14.7	8.55	28.49
Abutment #1 14x117 APE D19-42	1	HP14x117	95	90	APE D19-42	3	0.10	0.10	0.10	0.15	160	610	23.92	2.63	14.5	7.87	18.95
	1	HP14x117	95	90	APE D19-42	3	0.10	0.20	0.10	0.15	160	530	21.77	2.46	14.4	7.67	18.53
	1	HP14x117	95	90	APE D19-42	3	0.10	0.30	0.10	0.15	160	470	21.32	2.40	14.6	7.47	18.06
	1	HP14x117	95	90	APE D19-42	3	0.13	0.33	0.13	0.07	160	480	21.38	4.61	14.3	7.53	18.21
	1	HP14x117	95	90	APE D19-42	4	0.10	0.10	0.10	0.15	160	670	27.30	2.80	14.8	8.83	22.26
	1	HP14x117	95	90	APE D19-42	4	0.10	0.20	0.10	0.15	160	580	23.52	2.47	15.0	8.53	21.53
	1	HP14x117	95	90	APE D19-42	4	0.10	0.30	0.10	0.15	160	510	23.16	2.32	14.4	8.35	21.18
	1	HP14x117	95	90	APE D19-42	4	0.13	0.33	0.13	0.07	160	530	23.27	4.34	14.3	8.45	21.42
	1	HP14x117	130	124.4	APE D19-42	4	0.10	0.04	0.10	0.15	160	710	29.31	3.04	15.0	8.83	22.78
	1	HP14x117	130	124.4	APE D19-42	4	0.10	0.07	0.10	0.15	160	680	28.04	2.94	14.9	8.69	22.37
Abutment #1 14x117 APE D25-42	1	HP14x117	95	90	APE D25-42	3	0.10	0.10	0.10	0.15	160	660	24.32	2.69	15.0	8.13	22.27
	1	HP14x117	95	90	APE D25-42	3	0.10	0.20	0.10	0.15	160	570	22.25	2.50	14.3	7.83	21.41
	1	HP14x117	95	90	APE D25-42	3	0.10	0.30	0.10	0.15	160	510	21.68	1.38	13.5	7.63	20.93
	1	HP14x117	95	90	APE D25-42	3	0.13	0.33	0.13	0.07	160	530	21.76	3.13	13.9	7.69	21.22
	1	HP14x117	95	90	APE D25-42	4	0.10	0.10	0.10	0.15	160	730	28.36	3.06	14.7	9.17	27.27
	1	HP14x117	95	90	APE D25-42	4	0.10	0.20	0.10	0.15	160	640	24.85	2.93	14.7	8.87	26.43
	1	HP14x117	95	90	APE D25-42	4	0.10	0.30	0.10	0.15	160	570	24.06	1.86	14.0	8.55	25.58
	1	HP14x117	95	90	APE D25-42	4	0.13	0.33	0.13	0.07	160	600	24.25	3.41	14.3	8.68	26.04
	1	HP14x117	130	124.4	APE D25-42	4	0.10	0.04	0.10	0.15	160	740	29.44	3.25	14.5	8.84	27.02
	1	HP14x117	130	124.4	APE D25-42	4	0.10	0.07	0.10	0.15	160	720	28.32	2.49	14.8	8.76	26.75

	Abutment	Pile Size	Pile Length (ft)	Pile Penetration (ft)	Hammer	Fuel Setting	Shaft Quake (in)	Toe Quake (in)	Shaft Damping (s/ft)	Toe Damping (s/ft)	Skin Friction (kips)	Ultimate Capacity (kips)	Max Comp Stress (ksi)	Max Tension Stress (ksi)	Blows/In	Stroke (ft)	Energy (kips-ft)
Abutment #2 14x89 APE D19-42	2	HP 14x89	110	105.2	APE D19-42	3	0.10	0.10	0.10	0.15	160	530	24.36	3.17	14.3	7.62	19.67
	2	HP 14x89	110	105.2	APE D19-42	3	0.10	0.20	0.10	0.15	160	480	22.76	2.41	14.4	7.44	19.20
	2	HP 14x89	110	105.2	APE D19-42	3	0.10	0.30	0.10	0.15	160	440	22.36	1.53	14.0	7.26	18.76
	2	HP 14x89	110	105.2	APE D19-42	3	0.13	0.33	0.13	0.07	160	450	22.49	3.44	14.0	7.35	18.95
	2	HP 14x89	110	105.2	APE D19-42	4	0.10	0.10	0.10	0.15	160	580	27.56	3.48	15.0	8.49	22.96
	2	HP 14x89	110	105.2	APE D19-42	4	0.10	0.20	0.10	0.15	160	520	24.63	3.06	14.4	8.31	22.47
	2	HP 14x89	110	105.2	APE D19-42	4	0.10	0.30	0.10	0.15	160	480	24.24	1.40	14.7	8.12	22.03
	2	HP 14x89	110	105.2	APE D19-42	4	0.13	0.33	0.13	0.07	160	500	24.44	3.47	15.0	8.23	22.32
	2	HP 14x89	135	129.9	APE D19-42	4	0.10	0.04	0.10	0.15	160	590	28.75	3.44	14.3	8.40	23.17
	2	HP 14x89	135	129.9	APE D19-42	4	0.10	0.07	0.10	0.15	160	580	28.24	3.22	14.4	8.42	23.23
Abutment #2 14x89 APE D25-42	2	HP 14x89	110	105.2	APE D25-42	3	0.10	0.10	0.10	0.15	160	550	23.61	3.96	13.6	7.74	23.10
	2	HP 14x89	110	105.2	APE D25-42	3	0.10	0.20	0.10	0.15	160	520	23.08	2.92	14.8	7.59	22.68
	2	HP 14x89	110	105.2	APE D25-42	3	0.10	0.30	0.10	0.15	160	480	22.47	1.14	13.7	7.39	22.03
	2	HP 14x89	110	105.2	APE D25-42	3	0.13	0.33	0.13	0.07	160	500	22.65	2.54	15.0	7.49	22.33
	2	HP 14x89	110	105.2	APE D25-42	4	0.10	0.10	0.10	0.15	160	620	27.71	4.67	14.9	8.69	28.20
	2	HP 14x89	110	105.2	APE D25-42	4	0.10	0.20	0.10	0.15	160	570	25.34	3.46	14.5	8.45	27.42
	2	HP 14x89	110	105.2	APE D25-42	4	0.10	0.30	0.10	0.15	160	530	24.91	1.71	13.7	8.30	27.02
	2	HP 14x89	110	105.2	APE D25-42	4	0.13	0.33	0.13	0.07	160	550	25.09	2.62	13.8	8.40	27.35
	2	HP 14x89	135	129.9	APE D25-42	4	0.10	0.04	0.10	0.15	160	620	29.15	4.04	14.1	8.49	28.23
	2	HP 14x89	135	129.9	APE D25-42	4	0.10	0.07	0.10	0.15	160	610	28.38	3.54	14.0	8.52	28.37
Abutment #2 14x117 APE D19-42	2	HP14x117	110	105.2	APE D19-42	3	0.10	0.10	0.10	0.15	170	610	23.23	2.58	14.9	7.82	18.88
	2	HP14x117	110	105.2	APE D19-42	3	0.10	0.20	0.10	0.15	170	530	21.50	2.15	14.8	7.58	18.25
	2	HP14x117	110	105.2	APE D19-42	3	0.10	0.30	0.10	0.15	170	470	21.06	2.15	14.3	7.37	17.75
	2	HP14x117	110	105.2	APE D19-42	3	0.13	0.33	0.13	0.07	170	480	21.23	4.19	14.1	7.48	18.00
	2	HP14x117	110	105.2	APE D19-42	4	0.10	0.10	0.10	0.15	170	660	26.07	2.79	15.0	8.67	21.87
	2	HP14x117	110	105.2	APE D19-42	4	0.10	0.20	0.10	0.15	170	580	23.35	2.50	15.0	8.49	21.43
	2	HP14x117	110	105.2	APE D19-42	4	0.10	0.30	0.10	0.15	170	510	22.87	2.05	14.2	8.25	20.89
	2	HP14x117	110	105.2	APE D19-42	4	0.13	0.33	0.13	0.07	170	530	23.08	4.04	14.3	8.38	21.18
	2	HP14x117	135	129.9	APE D19-42	4	0.10	0.04	0.10	0.15	170	700	28.37	2.94	14.8	8.75	22.55
	2	HP14x117	135	129.9	APE D19-42	4	0.10	0.07	0.10	0.15	170	670	27.11	2.82	14.6	8.63	22.15
Abutment #2 14x117 APE D25-42	2	HP14x117	110	105.2	APE D25-42	3	0.10	0.10	0.10	0.15	170	630	22.41	2.31	14.3	7.85	21.48
	2	HP14x117	110	105.2	APE D25-42	3	0.10	0.20	0.10	0.15	170	570	21.83	2.20	15.0	7.71	21.06
	2	HP14x117	110	105.2	APE D25-42	3	0.10	0.30	0.10	0.15	170	510	21.31	1.53	13.8	7.52	20.61
	2	HP14x117	110	105.2	APE D25-42	3	0.13	0.33	0.13	0.07	170	530	21.43	3.10	14.4	7.60	20.85
	2	HP14x117	110	105.2	APE D25-42	4	0.10	0.10	0.10	0.15	170	710	26.60	2.56	14.8	8.88	26.51
	2	HP14x117	110	105.2	APE D25-42	4	0.10	0.20	0.10	0.15	170	620	24.04	2.43	14.2	8.58	25.53
	2	HP14x117	110	105.2	APE D25-42	4	0.10	0.30	0.10	0.15	170	570	23.48	1.63	14.9	8.36	24.92
	2	HP14x117	110	105.2	APE D25-42	4	0.13	0.33	0.13	0.07	170	590	23.79	3.29	14.1	8.53	25.46
	2	HP14x117	135	129.9	APE D25-42	4	0.10	0.04	0.10	0.15	170	730	28.51	2.94	14.4	8.76	26.70
	2	HP14x117	135	129.9	APE D25-42	4	0.10	0.07	0.10	0.15	170	710	27.38	2.41	14.5	8.69	26.49

Created By: NPP 7/14/25

Checked By: LK 7/28/25

Quake and Damping Summary from CAPWAP Analyses - Pile Ending in Very Dense Soil

WIN	Town	Bridge Name	Bridge Number	Pile Location	Pile ID	EOD/BOR	Shaft Quake (in)	Toe Quake (in)	Shaft Damping (s/ft)	Toe Damping (s/ft)
22627.01	Milo	Pleasant River Bridge	3244	Abutment No. 1	S-1	EOD	0.11	0.40	0.11	0.09
22627.01	Milo	Pleasant River Bridge	3244	Abutment No. 1	S-1	BOR	0.22	0.30	0.10	0.03
22627.01	Milo	Pleasant River Bridge	3244	Abutment No. 2	N-10	EOD	0.11	0.36	0.10	0.08
22627.01	Milo	Pleasant River Bridge	3244	Abutment No. 2	N-10	BOR	0.18	0.36	0.13	0.11
22627.01	Milo	Pleasant River Bridge	3244	Pier	PN-5	EOD	0.26	0.54	0.05	0.03
22627.01	Milo	Pleasant River Bridge	3244	Pier	PN-5	BOR	0.03	0.43	0.09	0.10
20502.01	Milo	Old Toll Bridge	2867	Abutment No. 2	2A	BOR	0.04	0.40	0.10	0.04
20502.01	Milo	Old Toll Bridge	2867	Abutment No. 2	2F	BOR	0.06	0.36	0.11	0.04
20502.01	Milo	Old Toll Bridge	2867	Abutment No. 2	2C	EOD	0.08	0.43	0.15	0.05
20502.01	Milo	Old Toll Bridge	2867	Abutment No. 2	2B	EOD	0.09	0.36	0.16	0.03
20502.01	Milo	Old Toll Bridge	2867	Abutment No. 2	2B	BOR	0.04	0.38	0.10	0.03
18915.00	Brewer	Eaton Brook Bridge	6645	Abutment No. 1	1-1	EOD	0.10	0.23	0.16	0.08
18915.00	Brewer	Eaton Brook Bridge	6645	Abutment No. 1	1-1	BOR	0.09	0.26	0.16	0.08
18915.00	Brewer	Eaton Brook Bridge	6645	Abutment No. 1	1-5	EOD	0.08	0.17	0.15	0.10
18915.00	Brewer	Eaton Brook Bridge	6645	Abutment No. 1	1-5	BOR	0.08	0.24	0.15	0.08
18915.00	Brewer	Eaton Brook Bridge	6645	Abutment No. 2	2-1	EOD	0.20	0.27	0.12	0.07
18915.00	Brewer	Eaton Brook Bridge	6645	Abutment No. 2	2-1	BOR	0.10	0.28	0.11	0.10
18915.00	Brewer	Eaton Brook Bridge	6645	Abutment No. 2	2-5	EOD	0.10	0.18	0.15	0.10
18915.00	Brewer	Eaton Brook Bridge	6645	Abutment No. 2	2-5	BOR	0.10	0.13	0.20	0.05
Average EOD Values (highlighted rows)							0.13	0.33	0.13	0.07

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

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

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

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

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

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

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

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

4.4.8.2.3 Tolerable Movement

Refer to Article 4.4.7.2.3.

4.4.9 Overall Stability

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

26083.00 Van Buren - St. Mary's Bridge #5309
Created By: NPP 3/13/25
Checked By: LK 7/11/25

PDA Shaft Resistance for Projects with Similar Soils

WIN	Town	Bridge Name	Pile Location	Pile Penetration (ft)	PDA Measured Shaft Resistance EOD (kips)	PDA Measured Shaft Resistance BOR (kips)	Notes
18915	Brewer	Eaton Brook	ABT #1	49.4	210	260	Tip in Glacial Till with spoon refusals per BB-BEB-201
18915	Brewer	Eaton Brook	ABT #1	46.5	230	215	Tip in Glacial Till with spoon refusals per BB-BEB-202
18915	Brewer	Eaton Brook	ABT #2	34.0	155	180	Tip in hard Glacial Till per BB-BEB-104
18915	Brewer	Eaton Brook	ABT #2	54.0	400	403	Tip in Glacial Till with spoon refusals per BB-BEB-204A
20502	Milo	Old Toll	ABT #1	53.7	200	220	
20503	Milo	Old Toll	Pier	42.8	230	190	
20504	Milo	Old Toll	ABT #2	40.0	n/a	170	Tip in Glacial Till
20504	Milo	Old Toll	ABT #2	39.8	n/a	160	Tip in Glacial Till
20504	Milo	Old Toll	ABT #2	38.0	120	n/a	Tip in Glacial Till
20504	Milo	Old Toll	ABT #2	35.6	160	160	Tip in Glacial Till
22627	Milo	Pleasant River	ABT #1	99.6	140	160	Tip in Glacial Till
22627	Milo	Pleasant River	Pier	103.8	160	190	Tip in Glacial Till
22627	Milo	Pleasant River	ABT #2	101.1	150	200	Tip in Glacial Till
20503	Brownville	Brownville Junction	ABT #2	61.5	210	315	
22250	Macwahoc PLT	Kingman Road Bridge	ABT #1	48.3	150	100	
22250	Macwahoc PLT	Kingman Road Bridge	ABT #2	47.2	80	30	
22250	Macwahoc PLT	Kingman Road Bridge	ABT #2	50.9	150	130	

GRLWEAP Shaft Resistance Inputs		
Pile Size	Abutment No. 1	Abutment No. 2
14x89	150	160
14x117	160	170

Objective:

Estimate shaft resistance for GRLWEAP using the effective stress β-Method

Given:

1. Limited lab data and SPT N-values.

References:

1. Project soil models
2. FHWA-NHI-16-009 Design and Construction of Driven Pile Foundations - Volume I, July 2016

β Coefficient Reference Table:

Table 7-9 Approximate Range of β and N_t Coefficients (after Fellenius 2014)

Soil Type	ϕ'	β	N_t
Clay	25-30	0.15-0.35	3-30
Silt	28-34	0.25-0.50	20-40
Sand	32-40	0.30-0.90	30-150
Gravel	35-45	0.35-0.80	60-300

Symbol Definitions:

h = height of soil layer
γ = effective unit weight
ϕ = friction angle
β = Bjerrum-Burland beta coefficent
σ = effective stress at midpoint of soil layer
f = unit shaft resistance
R_s = total shaft resistance

Abutment No. 1

Assumptions:

Pile Properties:

Section: HP 14x89

Depth: $d := 13.80\text{in}$

Flange Width: $b := 14.70\text{in}$

Pile Length (BOF to Top of Competent Bedrock): $L_{pile} := 124.4\text{ft}$

Pile Perimeter - Plugged Pile

$p := 2 \cdot d + 2 \cdot b$

$p = 57 \cdot \text{in}$ $p = 4.75 \cdot \text{ft}$

Unit Weight of Water

$\gamma_{water} := 62.4\text{pcf}$

Layer 0 - Approach Layer

Soil description: Granular Fill

Layer properties:

$h_0 := 9.6ft$

$\gamma_0 := 125pcf$

$\phi_0 := 32$

Surcharge:

$s_0 := 0.5 \cdot h_0 \cdot \gamma_0$

Note: Approach layer surcharge reduced by 50% due to unequal backfill in front and behind the abutment.

$s_0 = 600 \cdot psf$

Layer 1 - Fill (Submerged)

Soil description: Granular Fill

Layer properties:

$h_1 := 2.0ft$

$\gamma_1 := 125pcf - \gamma_{water} = 62.6 \cdot pcf$

$\phi_1 := 32$

$\beta_1 := 0.30$

Effective Stress at Layer Midpoint:

$\sigma_1 := s_0 + 0.5 \cdot h_1 \cdot \gamma_1$

$\sigma_1 = 663 \cdot psf$

Unit Shaft Resistance at Layer Midpoint:

$f_1 := \beta_1 \cdot \sigma_1$

$f_1 = 199 \cdot psf$

Pile Surface Area Within Layer:

$A_1 := h_1 \cdot p$

$A_1 = 9.5 \cdot ft^2$

Layer 2 - Stream Alluvium

Soil description: medium dense, Sandy GRAVEL, little to some silt

Layer properties:

$h_2 := 24.0ft$

$\gamma_2 := 130pcf - \gamma_{water} = 67.6 \cdot pcf$

$\phi_2 := 34$

$\beta_2 := 0.35$

Effective Stress at Layer Midpoint:

$\sigma_2 := \sigma_1 + 0.5 \cdot h_2 \cdot \gamma_2$

$\sigma_2 = 1474 \cdot psf$

Unit Shaft Resistance at Layer Midpoint:

$f_2 := \beta_2 \cdot \sigma_2$

$f_2 = 516 \cdot psf$

Pile Surface Area Within Layer:

$A_2 := h_2 \cdot p$

$A_2 = 114 \cdot ft^2$

Layer 3 - Glacial Till (Upper)

Soil description: hard, SILT, some sand, little gravel, little clay

Layer properties:

$h_3 := 44.0ft$

$\gamma_3 := 140pcf - \gamma_{water} = 77.6 \cdot pcf$

$\phi_3 := 36$

$\beta_3 := 0.4$

Effective Stress at Layer Midpoint:

$\sigma_3 := \sigma_2 + 0.5 \cdot h_3 \cdot \gamma_3$

$\sigma_3 = 3181 \cdot psf$

Unit Shaft Resistance at Layer Midpoint:

$f_3 := \beta_3 \cdot \sigma_3$

$f_3 = 1272 \cdot psf$

Pile Surface Area Within Layer:

$A_3 := h_3 \cdot p$

$A_3 = 209 \cdot ft^2$

Layer 4 - Glacial Till (Lower)

Soil description: very dense, SAND, some silt, some gravel

Layer properties:

$h_4 := 54.4\text{ft}$

$\gamma_4 := 145\text{pcf} - \gamma_{water} = 82.6 \cdot \text{pcf}$

$\phi_4 := 38$

$\beta_4 := 0.6$

Effective Stress at Layer Midpoint:

$\sigma_4 := \sigma_3 + 0.5 \cdot h_4 \cdot \gamma_4$

$\sigma_4 = 5428 \cdot \text{psf}$

Unit Shaft Resistance at Layer Midpoint:

$f_4 := \beta_4 \cdot \sigma_4$

$f_4 = 3257 \cdot \text{psf}$

Pile Surface Area Within Layer:

$A_4 := h_4 \cdot p$

$A_4 = 258.4 \cdot \text{ft}^2$

Total Shaft Resistance

$R_s := f_1 \cdot A_1 + f_2 \cdot A_2 + f_3 \cdot A_3 + f_4 \cdot A_4$

$R_s = 1168 \cdot \text{kip}$

Abutment No. 2

Assumptions:

Pile Properties:

Section: HP 14x89

Depth: $d := 13.80 in$

Flange Width: $b := 14.70 in$

Pile Length (BOF to Top of Competent Bedrock): $L_{pile} := 129.9 ft$

Pile Perimeter - Plugged Pile

$p := 2 \cdot d + 2 \cdot b$

$p = 57 \cdot in$ $p = 4.75 \cdot ft$

Unit Weight of Water

$\gamma_{water} := 62.4 pcf$

Layer 0 - Approach Layer

Soil description: Granular Fill

Layer properties:

$h_0 := 9.8 ft$

$\gamma_0 := 125 pcf$

$\phi_0 := 32$

Surcharge:

$s_0 := 0.5 \cdot h_0 \cdot \gamma_0$

Note: Approach layer surcharge reduced by 50% due to unequal backfill in front and behind the abutment.

$s_0 = 613 \cdot psf$

Layer 1 - Stream Alluvium

Soil description: very dense, Sandy GRAVEL, little to some silt

Layer properties:

$h_1 := 17.2ft$

$\gamma_1 := 135pcf - \gamma_{water} = 72.6 \cdot pcf$

$\phi_1 := 36$

$\beta_1 := 0.4$

Effective Stress at Layer Midpoint:

$\sigma_1 := s_0 + 0.5 \cdot h_1 \cdot \gamma_1$

$\sigma_1 = 1237 \cdot psf$

Unit Shaft Resistance at Layer Midpoint:

$f_1 := \beta_1 \cdot \sigma_1$

$f_1 = 495 \cdot psf$

Pile Surface Area Within Layer:

$A_1 := h_1 \cdot p$

$A_1 = 81.7 \cdot ft^2$

Layer 2 - Glacial Till (Upper)

Soil description: dense, Silty SAND, little to some gravel

Layer properties:

$h_2 := 68ft$

$\gamma_2 := 135pcf - \gamma_{water} = 72.6 \cdot pcf$

$\phi_2 := 36$

$\beta_2 := 0.4$

Effective Stress at Layer Midpoint:

$\sigma_2 := \sigma_1 + 0.5 \cdot h_2 \cdot \gamma_2$

$\sigma_2 = 3705 \cdot psf$

Unit Shaft Resistance at Layer Midpoint:

$f_2 := \beta_2 \cdot \sigma_2$

$f_2 = 1482 \cdot psf$

Pile Surface Area Within Layer:

$A_2 := h_2 \cdot p$

$A_2 = 323 \cdot ft^2$

Layer 3 - Glacial Till (Lower)

Soil description: very dense, SAND, some silt, some gravel

Layer properties:

$h_3 := 44.7ft$

$\gamma_3 := 145pcf - \gamma_{water} = 82.6 \cdot pcf$

$\phi_3 := 38$

$\beta_3 := 0.60$

Effective Stress at Layer Midpoint:

$\sigma_3 := \sigma_2 + 0.5 \cdot h_3 \cdot \gamma_3$

$\sigma_3 = 5551 \cdot psf$

Unit Shaft Resistance at Layer Midpoint:

$f_3 := \beta_3 \cdot \sigma_3$

$f_3 = 3331 \cdot psf$

Pile Surface Area Within Layer:

$A_3 := h_3 \cdot p$

$A_3 = 212.3 \cdot ft^2$

Total Shaft Resistance

$R_s := f_1 \cdot A_1 + f_2 \cdot A_2 + f_3 \cdot A_3$

$R_s = 1226 \cdot kip$

Conclusion

Override estimated shaft resistance (by Beta Method) with shaft resistances measured during PDA testing on projects with similar soil profiles. See table labeled "PDA Shaft Resistance for Projects with Similar Soils"

Earth Pressure

Earth Pressure:

Backfill engineering strength parameters

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

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

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

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

Abutment Backfill Angles

α = Angle of fill slope to the horizontal

Assume backfill is level at both abutments

$\alpha := 0\text{deg}$

Integral Abutment - Passive Earth Pressure - Coulomb Theory

α = Angle of fill slope to the horizontal $\alpha := 0\text{deg}$

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

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

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

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

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

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

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

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

$K_{p\_coulomb} = 6.02$

Integral Abutment and Wingwall - Passive Earth Pressure - Rankine Theory

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

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

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

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

$K_{p\_rank} = 3.25$

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

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

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

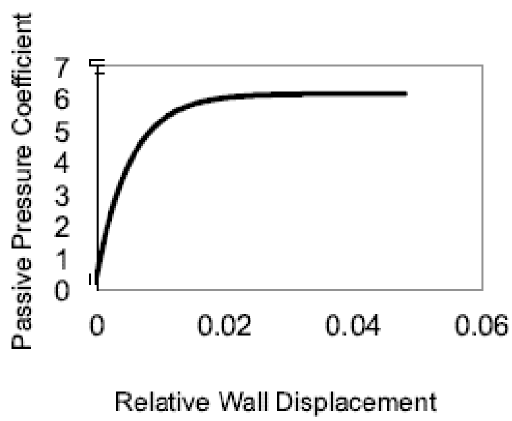


Figure 3.10.8-1: Plot of Passive Pressure Coefficient, K, vs. Relative Wall Displacement, δ_T/H.

Compute Relative Wall Displacement

Abutment Displacement (Expansion): $\delta := 0.246in$ Provided by HNTB

Abutment height: $h := 10.0ft$ $h = 120 \cdot in$

Relative wall displacement: $x := \frac{\delta}{h}$ $x = 0.0021$

Calculate MassDOT Passive Pressure Coefficient

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

$K = 2.27$ **< K_{p_rank} of 3.25, therefore recommend K=3.25 for both Abutments**

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

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

3.11.5.4—Passive Lateral Earth Pressure Coefficient, k_p

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

For cohesive soils, passive pressures may be estimated by:

C3.11.5.4

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

Table 7.9 (*Continued*)

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

7.12 Coulomb's Passive Earth Pressure

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

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

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

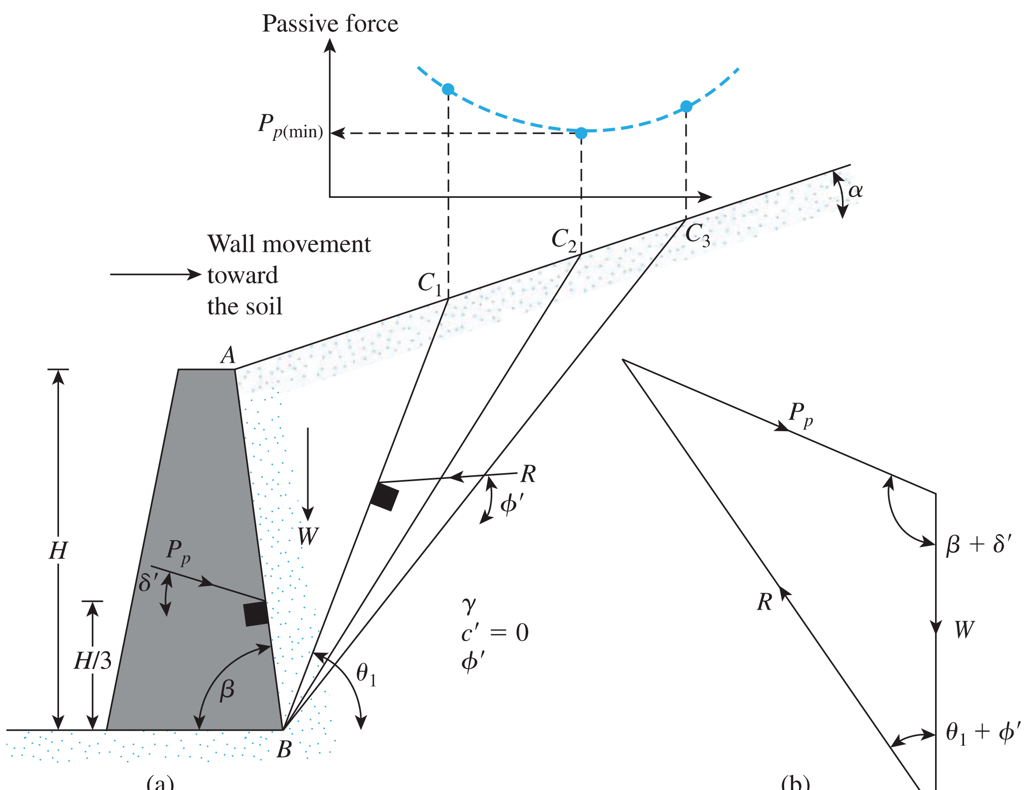


Figure 7.25 Coulomb's passive pressure

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

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

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

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

$$P_p = \tfrac{1}{2}\gamma H^2 K_p \qquad (7.70)$$

where

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

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

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

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

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

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

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

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

Again, at $z = 3$ m,

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

Hence,

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

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

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

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

■

7.11 Rankine Passive Earth Pressure: Vertical Backface and Inclined Backfill

Granular Soil

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

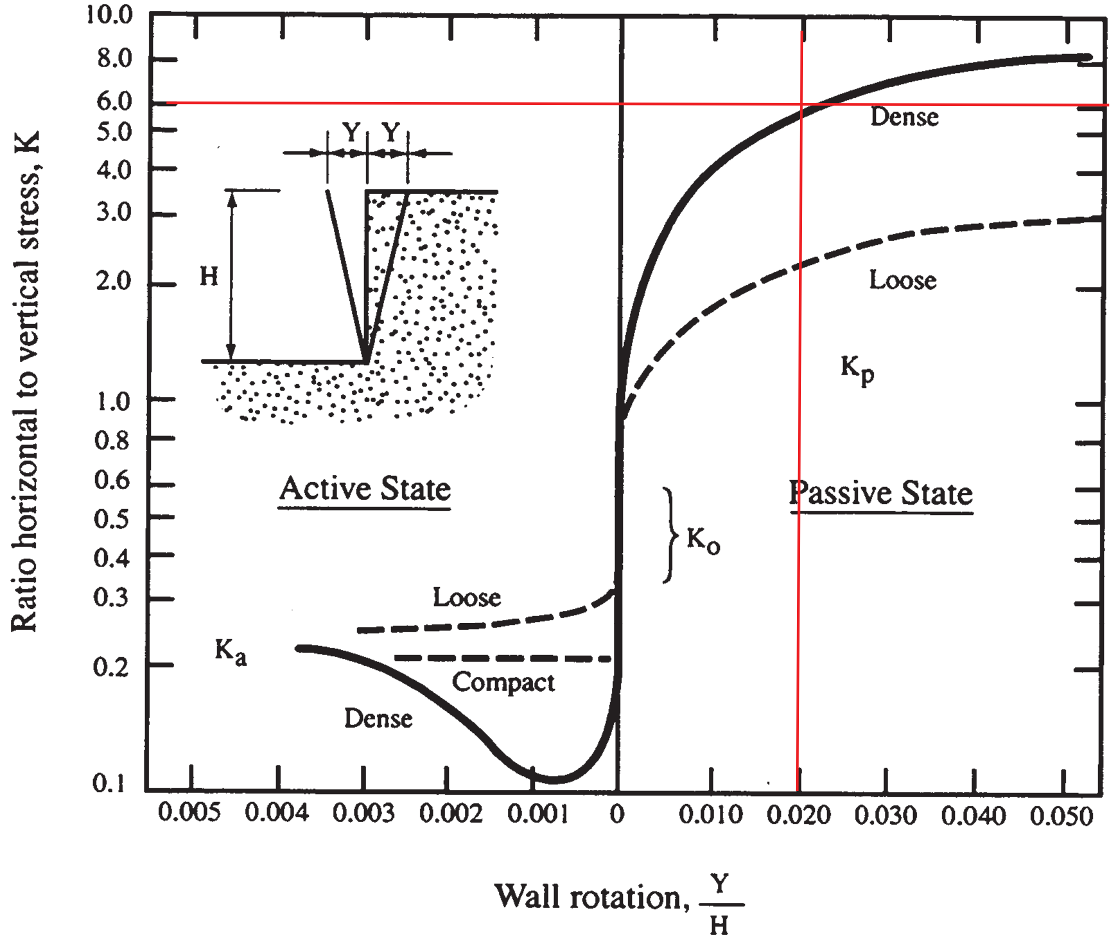
$$\sigma'_p = \gamma z K_p \tag{7.65}$$

and the passive force is

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

where

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



Magnitude of Wall Rotation to Reach Failure

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

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

Frost Depth

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

From Design Freezing Index Map: Van Buren, Maine
DFI = 2600 degree-days.
Coarse-Grained Fill w=10% (BB-VBVB-101, 1D)

Coarse-Grained Fill

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

$d_{coarse} := 109.1in$ $d_{coarse} = 9.1 \cdot ft$ d=Depth of Frost Penetration

Recommend any foundation bearing on soil be embedded **9.1 feet** for frost protection.

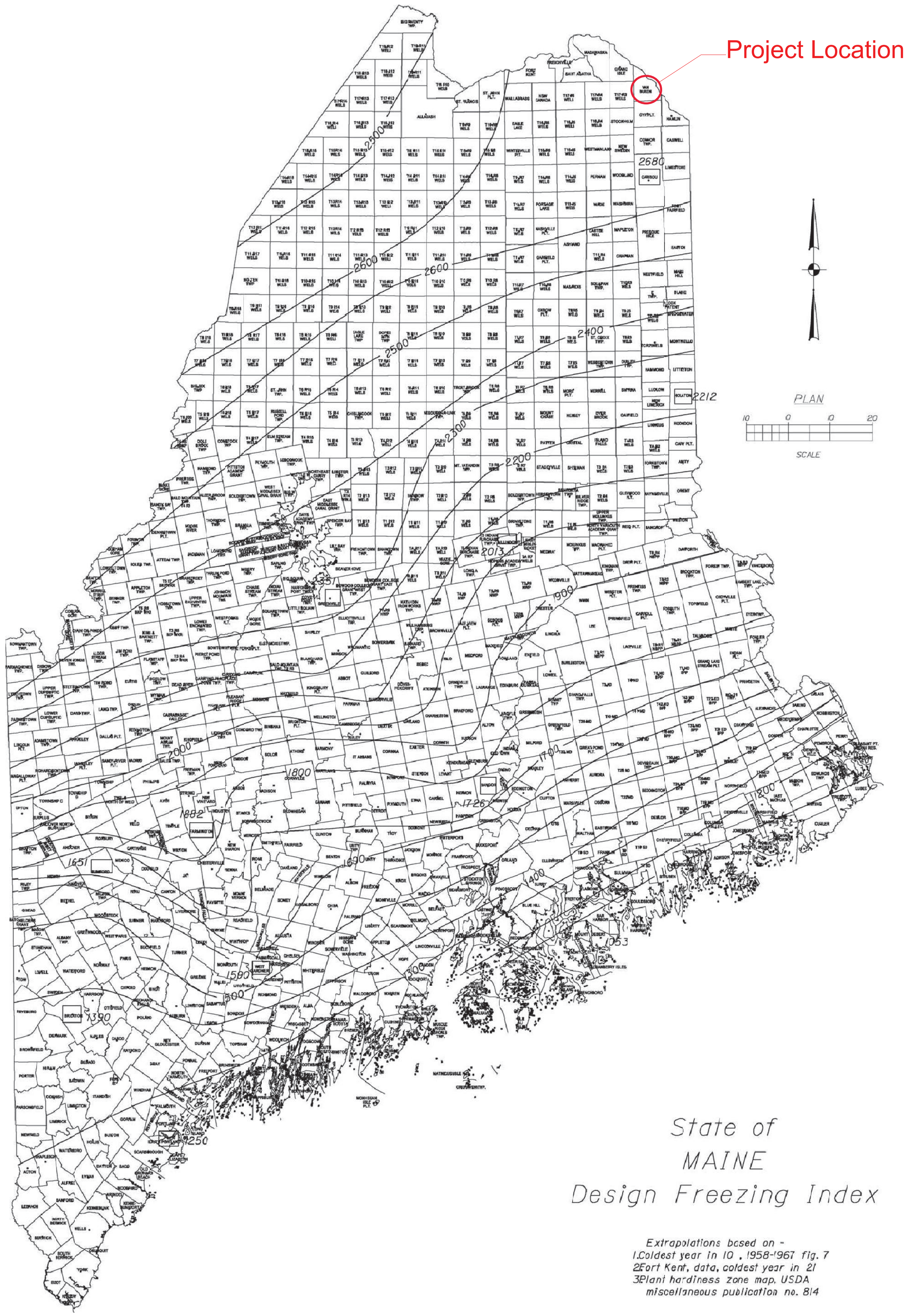


Figure 5-1 Maine Design Freezing Index Map

5.2 General

5.2.1 Frost

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

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

Table 5-1 Depth of Frost Penetration

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

Seismic Parameters

BB-VBVB-101			
Depth	**N_{60}**	**di**	**di/N**
5	43	7	0.16
10	30	5	0.17
15	34	8	0.24
20	27	5	0.19
25	22	5	0.23
30	37	5	0.14
35	27	1	0.04
40	46	9	0.20
45	51	5	0.10
50	59	5	0.08
55	58	5	0.09
60	45	5	0.11
65	46	5	0.11
70	37	5	0.14
75	29	5	0.17
80	85	5	0.06
85	100	5	0.05
90	100	5	0.05
95	100	5	0.05
SUM		**100**	**2.35**
		di/di/N	**42.55**

BB-VBVB-102			
Depth	**N_{60}**	**di**	**di/N**
5	10	7	0.70
10	61	8	0.13
15	43	5	0.12
20	50	5	0.10
25	55	2	0.04
30	50	8	0.16
35	66	10	0.15
45	50	10	0.20
55	27	10	0.37
65	37	10	0.27
75	27	10	0.37
85	24	10	0.42
95	79	5	0.06
SUM		**100**	**3.09**
		di/di/N	**32.40**

SUM	**Nav.**	**37.48**

15 < Nav. < 50 bpf

Conclusion: Site Class D

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

Van Buren, St. Mary's Bridge #5309
WIN 26083.00
March 10, 2025

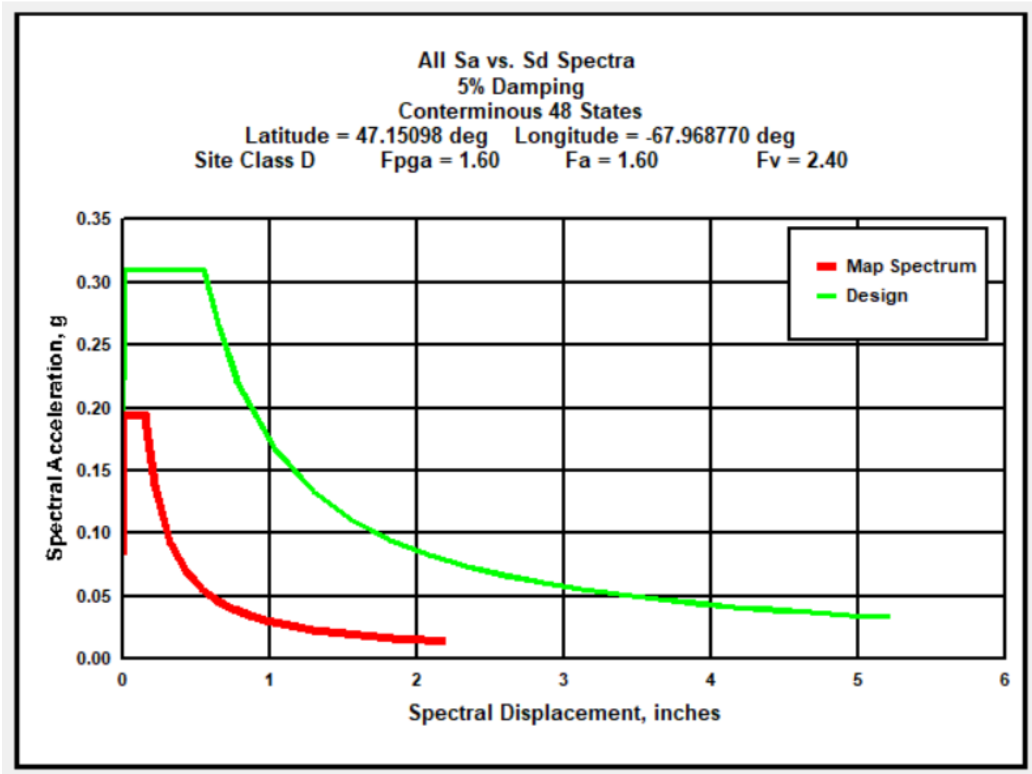
Abutment No. 1 and 2 Seismic Parameters

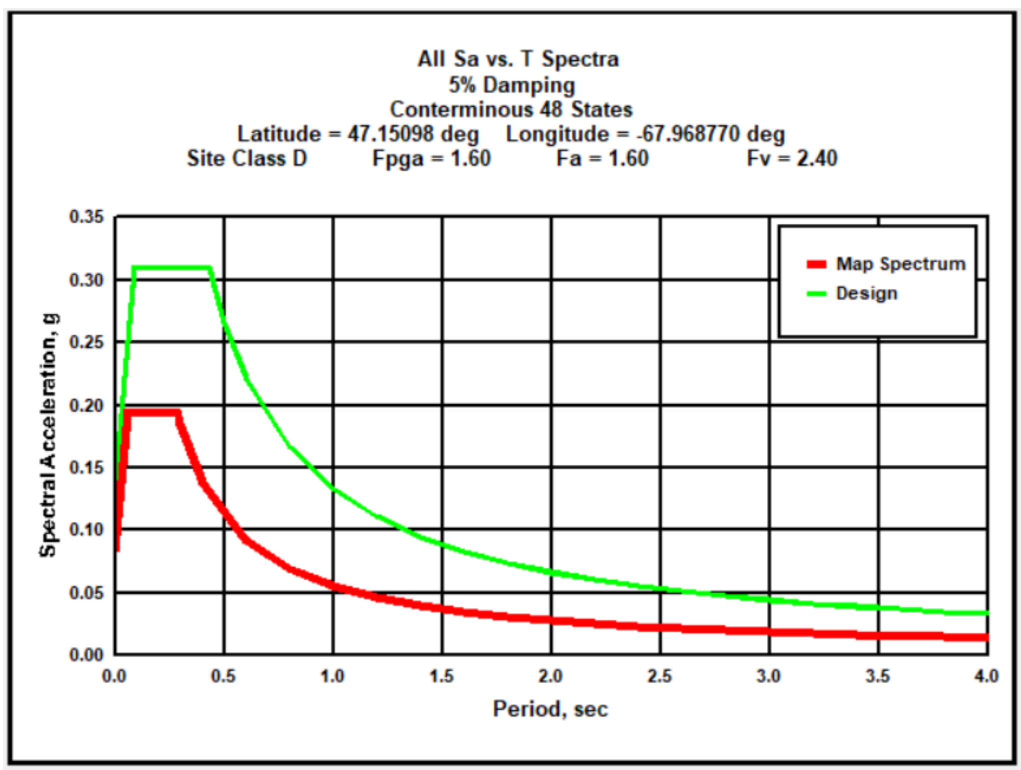
Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 47.150983
Longitude = -067.968767
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.085	PGA - Site Class B
0.2	0.193	Ss - Site Class B
1.0	0.055	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
Latitude = 47.150983
Longitude = -067.968767
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.137	As - Site Class D
0.2	0.309	SDs - Site Class D
1.0	0.133	SD1 - Site Class D


All Sa vs. Sd Spectra
5% Damping
Conterminous 48 States
Latitude = 47.15098 deg Longitude = -67.968770 deg
Site Class D Fpga = 1.60 Fa = 1.60 Fv = 2.40
Map Spectrum
Design
Spectral Acceleration, g
Spectral Displacement, inches
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
0
1
2
3
4
5
6


All Sa vs. T Spectra
5% Damping
Conterminous 48 States
Latitude = 47.15098 deg Longitude = -67.968770 deg
Site Class D Fpga = 1.60 Fa = 1.60 Fv = 2.40
Map Spectrum
Design
Spectral Acceleration, g
Period, sec
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0