MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

BOG STREAM BRIDGE
STATE ROUTE 192 OVER BOG STREAM
NORTHFIELD, MAINE

Prepared by:
Brandon Slaven
Assistant Geotechnical Engineer

Reviewed by:
Laura Krusinski, P.E.
Senior Geotechnical Engineer

Washington County
WIN 21699.00

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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 Bog Stream Bridge which carries State Route 192 over Bog Stream in Northfield, Maine. This report presents the subsurface information obtained at the site during the subsurface investigation, geotechnical recommendations, and geotechnical design parameters for the design of the new substructures.

The existing structure was originally constructed in 1940. The superstructure consists of a concrete tee beam superstructure with a twenty-six-foot span. The substructure consists of mass concrete abutments with spread footings founded on soil. The bridge was widened one in 2015. According to the 2017 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the bridge deck and superstructure are in poor condition (rating of 4) and the substructure is in fair condition (rating of 5) with extensive map cracking with isolated delamination. The FHWA Sufficiency Rating of the existing bridge is 46.4.

The proposed replacement structure consists of a 62-foot single span precast concrete NEXT beam superstructure founded on H-pile supported integral abutments. The existing structure will be removed and 1.75H:1V (horizontal:vertical) riprap slopes will be placed in front of the new integral abutments. The new Bog Stream Bridge will be designed to match the existing horizontal alignment and closely match the existing vertical profile. A single lane of alternating traffic will maintain traffic on a temporary detour upstream of the project.

2.0 GEOLOGIC SETTING

The existing structure carries State Route 192 over Bog Stream as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of the Wesley Quadrangle, Maine, Open-file No. 86-26 (1986), indicates the surficial soils in the vicinity of the bridge project consist of thin surficial deposits and bedrock outcrops with nearby contacts to glacial till. Glacial till is a heterogeneous mixture of sand, silt, clay and stones. These soils generally overly bedrock, but may overlie, or include, sand and gravel.

The Bedrock Geologic Map of the Wesley 15’ Quadrangle, MGS (1981), cites the bedrock at the project site as Bog Lake Granite, which is a biotite granite with oval quartz phenocrysts.

3.0 SUBSURFACE INVESTIGATION

Three test borings explored subsurface conditions at the site. Borings BB-NBS-101 and BB-NBS-103 were drilled west of the existing bridge in June 2018. Boring BB-NBS-102 was drilled east of the existing bridge in May 2017. The boring locations are shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 3 – 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 in 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 two drill rigs used in the subsurface investigation were equipped with an automatic hammer to drive the split spoon. The hammers were calibrated per ASTM D4633 "Standard Test Method for Energy Measurement for Dynamic Penetrometers." All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.928 for the westerly borings and 0.873 for the easterly boring. The hammer efficiency factors (0.928 and 0.873) and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring logs.

Bedrock was cored using an NQ-2" core barrel and the Rock Quality Designation (RQD) of the core calculated. A geotechnical engineer or a Northeast Transportation Technician Certification Program (NETTCP) Certified Subsurface Inspector logged the subsurface conditions encountered. The geotechnical engineer selected the boring locations and drilling methods, designated type and depth of sampling techniques, reviewed boring logs, and identified field testing requirements. The borings were located in the field using taped measurements at the completion of the drilling program.

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 consisted of seven standard grain size analyses with natural water content and one grain size analyses with hydrometer and natural moisture content. The results of soil tests are included as Appendix B – Laboratory Test Results. Moisture content information and other soil test results are also shown on the boring logs provided in Appendix A – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings generally consisted of granular fill material and glacial till. The fill unit and subsurface soils are underlain by igneous bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs. A generalized subsurface profile is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile. The following paragraphs discuss the subsurface conditions encountered:

5.1 Fill

Encountered in the borings was a layer of fill material. The thickness of the fill unit was approximately 7.5 to 9.5 feet at the boring locations. The fill encountered generally consisted of:

- Brown, damp, gravelly sand, trace silt, occasional cobbles; and

- Brown, damp to wet, sand, some to little gravel, trace silt.

Corrected SPT N-values in the fill unit ranged from 5 to 32 blows per foot (bpf) indicating the fill is loose to dense in consistency. One SPT test refused on a presumed cobble within the unit after 50 blows advanced the sampler six inches. One grain size analysis performed on a sample recovered from within the fill unit resulted in an A-1-b material classification according to the AASHTO Soil Classification System and SW-SM classification according to the Unified Soil Classification System (USCS). The natural water content of the sample tested was approximately 7 percent.

5.2 Glacial Till

Glacial till was encountered beneath the fill materials in the borings. The thickness ranged from approximately 34.4 to 69.0 feet. The glacial till generally consisted of:

- Grey, wet, sand, some gravel, some to little silt;
- Grey, wet, silty sand, some to trace gravel;
- Grey, wet, sandy silt, some to trace gravel; trace small cobbles;
- Grey, wet, gravelly sand, trace silt;
- Grey, wet, sand, some silt, little clay, little gravel;
- Cobbles; and
- Boulders.

Corrected SPT N-values of the glacial till deposit ranged from 26 to greater than 100 bpf indicating the glacial till is medium dense to very dense in consistency. Seven grain size analyses performed on samples recovered from within the glacial till deposit resulted in A-1-a, A-1-b, and A-4 material classifications according to the AASHTO Soil Classification System and SM, SW-SM, and SC-SM classifications according to the USCS. The natural water contents of the samples tested ranged from approximately 8 to 14 percent.

5.3 Bedrock

Bedrock was encountered and cored in borings BB-NBS-101, BB-NBS-102, and BB-NBS-103. Table 1 summarizes the approximate initial bedrock core depths, elevations, and RQD's. The roller cone penetrated the upper 10 feet of bedrock in BB-NBS-101 and no bedrock sample was recovered from El. 112.2 to 102.2. The roller cone also penetrated the upper 1 foot of bedrock in BB-NBS-102 and no bedrock sample was recovered from El. 75.6 to 74.6.

Boring	Station	Offset (feet)	Approximate Depth to Initial Bedrock Core (feet)	Approximate Initial Core Elevation (feet)	RQD (%)
BB-NBS-101	12+17	7.9 Lt	52.4	102.2	43
BB-NBS-102	12+89.3	6.2 Rt	80.0	74.6	0
BB-NBS-103	12+36	7.4 Rt	47.0	107.7	55

Table 1 – Summary of Approximate Initial Bedrock Core Depth, Elevation, and RQD

The bedrock recovered from the borings is generally identified as white to grey, medium grained, amphibole granite, hard, slightly weathered, joints dipping at horizontal to steep angles, very close to moderately close, tight. The RQD of the bedrock cores ranged from 0 to 80 percent correlating to a rock mass quality of very poor to good. Detailed bedrock descriptions and the RQD of each core run are provided on the boring logs in Appendix A – Boring Logs and on Sheet 3 – Boring Logs.

5.4 Groundwater

Groundwater was measured between 4.0 and 9.2 feet bgs during the subsurface investigation. Note that water was introduced into the borehole during drilling operations. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, river levels, and construction activities.

6.0 Foundation Alternatives

Bridge replacement alternatives in the Preliminary Design Report (PDR), February 2017, considered weathering steel and precast concrete superstructures with pile-supported integral abutments and a buried concrete arch on strip footings. Concrete arch spread footings require a deeper excavation when compared to pile-supported bridges because of the need to embed the footings for scour, bearing resistance, and frost. The deeper excavations required for the concrete arch alternative resulted in higher costs; therefore, a pile-supported integral bridge with a precast concrete superstructure was selected as the preferred bridge replacement alternative.

7.0 Geotechnical Design Considerations and Recommendations

The following sections provide geotechnical design considerations and recommendations for H-pile supported integral bridge abutments, which are the proposed substructures for the Bog Stream Bridge replacement project.

7.1 Integral Abutment H-Piles

Abutments No. 1 and No. 2 will be integral abutments founded on a single row of driven H-piles. The piles shall be end bearing on or within bedrock and driven to the required resistance. 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. Abutment No. 1 and Abutment No. 2 piles require driving pile points conforming to MaineDOT Standard Specification 711.10 to protect pile tips and improve penetration.

Pile lengths at the proposed abutments may be estimated based on Table 2:

Location	Approximate Bottom Elevation of Proposed Abutment (feet)	Boring	Offset	Approximate Pile Tip Elevation (feet)	Estimated Pile Lengths (feet)
Abutment No. 1	144.0	BB-NBS-101	Left	110.0	34.0
		BB-NBS-103	Right	110.0	34.0
Abutment No. 2	144.0	BB-NBS-102	Right	110.0	34.0

Table 2 – Estimated Pile Lengths for Integral Abutments No. 1 and No. 2

The estimated pile lengths in Table 2 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, additional pile length needed for embedment in the abutment or pile cap, or variations in the bedrock surface.

A minimum of 10 feet of additional pile length for Abutment No. 2 piles should be specified on the Contract Plans to reduce construction delays should piles not achieve the required resistance at the anticipated tip elevation.

7.1.1 Strength Limit State Design

The design of pile foundations bearing on bedrock at the strength limit state shall consider;

- compressive axial geotechnical resistance of individual piles,
- drivability resistance of individual piles,
- structural resistance of individual piles in axial compression, and
- structural resistance of individual 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. The pile group resistance after scour due to the design flood shall provide adequate foundation resistance using the resistance factors given in this section.

Per AASHTO LRFD Bridge Design Specifications 8th Edition with interim revisions through 2018 (LRFD) Article 6.5.4.2, at the strength limit state, the axial resistance factor ϕ_c = 0.50 (severe driving conditions) shall be applied to the structural compressive resistance of the pile. Since the H-piles will be subjected to lateral loading, the piles shall also be checked for combined axial compression and flexure as prescribed in LRFD Articles 6.9.2.2 and 6.15.2. This design axial load may govern the design. Per LRFD Article 6.5.4.2, at the strength limit state, the axial resistance factor ϕ_c = 0.70 and the flexural resistance factor ϕ_f = 1.0 shall be applied to the combined axial and flexural resistance of the pile in the interaction equation (LRFD Eq. 6.9.2.2-1 or -2). H-piles shall also be analyzed for fixity using LPile® v2016 (LPile) software, or similar.

Structural Resistance. The nominal axial compressive structural resistance (P_n) for piles loaded in compression shall be as specified in LRFD Article 6.9.4.1. Preliminary estimates of the structural axial resistance of two H-pile sections were calculated for approximated upper and lower unbraced pile segments, and for the lower braced pile segment. The controlling resistance shown in Tables 3 and 4 is for the lower braced pile segment, using a resistance factor, ϕ_c = 0.50 for severe driving conditions. The factored structural resistances for the approximated upper unbraced segments use an axial resistance factor ϕ_c = 0.70 for combined axial and flexure are not provided in Tables 3 and 4 because these did not govern. Supporting calculations are provided in Appendix C – Calculations. The unbraced pile lengths (ℓ) and effective length factors (K) in these evaluations have been assumed. It is the responsibility of the structural engineer to calculate the nominal axial structural compressive resistance (P_n) based on unbraced lengths (ℓ) and effective length factors (K) determined from LPile.

Geotechnical Resistance. The nominal axial geotechnical resistance of Abutment No. 1 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 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 compressive resistances for Abutment No. 1 piles subject to severe driving conditions are provided in Table 3.

The nominal axial geotechnical resistance estimates of Abutment No. 2 piles at the strength limit state was calculated using the guidance in LRFD Article 10.7.3.8.6 and utilizing the Nordlund static pile resistance method with SPT-Meyerhof method limiting the side resistance below the pile's critical depth of 20 pile diameters for very dense glacial till. Dynamic testing shall establish driving criteria and measure actual nominal pile resistance; therefore, the reliability of the nominal pile resistance is dependent upon the reliability of the dynamic testing and a resistance factor, ϕ_{dyn}, of 0.65 applies as recommended by LRFD Article C10.5.5.2.3. The resulting controlling factored geotechnical compressive resistances for Abutment No. 2 piles are provided in Table 4.

Drivability Analyses. Drivability analyses were performed to determine the pile resistance that might be achieved considering available diesel hammers. The maximum driving stresses in the pile, assuming the use of 50 ksi steel, shall be less than 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 two H-pile shapes at the strength limit states for Abutment No. 1 are provided in Table 3. Supporting calculations are provided in Appendix C – Calculations.

Pile Section	Abutment No. 1 Factored Axial Pile Resistance Strength Limit State			
	Structural Resistance[1] $\phi_c = 0.50$ (kips)	Controlling Geotechnical Resistance[2] $\phi = 0.50$ (kips)	Drivability Resistance[3] $\varphi_{dyn} = 0.65$ (kips)	Governing Axial Pile Resistance (kips)
HP 14 x 89	652	600	419	419
HP 14 x 117	860	793	511 (595)[4]	511 (595)[4]

Table 3– Factored Axial Compressive Resistances for Abutment No. 1 H-Piles at Strength Limit States – Driven Piles End Bearing on Bedrock

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, the estimated factored axial pile resistances from the drivability analyses for the Abutment No. 1 H-pile sections are less than the controlling factored axial geotechnical and structural resistance per LRFD Article 10.7.3.2.3. Therefore, drivability governs and the recommended governing resistances for Abutment No. 1 pile design are the drivability resistances provided in the rightmost column "Governing Axial Pile Resistance (kips)" in Table 3 above.

A summary of the calculated factored axial compressive structural, geotechnical, and drivability resistances of two H-pile shapes at the strength limit states for Abutment No. 2 are provided in Table 4. Supporting calculations are provided in Appendix C – Calculations.

[1] Structural resistances were calculated for approximated upper and lower unbraced pile segments and the lower braced pile segment. Controlling value shown here is for a braced segment in pure compression using a resistance factor, ϕ_c=0.50, for severe driving conditions. The factored structural resistances for the upper segments use a resistance factor of ϕ_c=0.70 for combined axial loading and bending are not shown here, but are provided in Appendix C – Calculations.
[2] Based on guidance in LRFD Article 10.7.3.2.3., *Piles Driven to Hard Rock.*
[3] Uses a resistance factor, $\varphi_{dyn} = 0.65$, assuming the driving criteria is established by dynamic testing, and quality control by dynamic testing of at least two piles per site condition and no less than two percent of production piles.
[4] Drivability resistance based on a Delmag D19-42. Drivability resistance with a Delmag D36-32 shown in parentheses.

Pile Section	Abutment No. 2 Factored Axial Pile Resistance Strength Limit State			
	Structural Resistance[5] $\phi_c = 0.50$ (kips)	Controlling Geotechnical Resistance[6] $\varphi_{dyn} = 0.65$ (kips)	Drivability Resistance[7] $\varphi_{dyn} = 0.65$ (kips)	Governing Axial Pile Resistance (kips)
HP 14 x 89	652	298	325	298
HP 14 x 117	860	354	344 (647)[8]	354

Table 4 – Factored Axial Compressive Resistances for Abutment No. 2 H-Piles at Strength Limit States - Driven Piles Friction and End Bearing in Glacial Till with Pile Tip at Approx. Elev. 110

The estimated factored axial pile resistances from the static pile analyses for the Abutment No. 2 H-pile sections are less than the axial pile resistances from the drivability analyses and the structural resistances with a resistance factor for severe driving conditions applied; therefore, the static pile geotechnical resistance governs and the recommended governing resistances for Abutment No. 2 pile design are the controlling geotechnical resistances provided in the rightmost column "Governing Axial Pile Resistance (kips)" in Table 4 above.

The maximum applied factored axial pile load for the strength limit states should not exceed the governing factored pile resistance shown in Table 3 for Abutment No. 1 and Table 4 for Abutment No. 2.

7.1.2 Service and Extreme Limit State Design

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 considering changes in soil conditions due to scour due to the design flood (Q_{100}). For the service limit state, resistance factors of $\phi = 1.0$ should be used in accordance with LRFD Article 10.5.5.1. The

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[5] Structural resistances were calculated for approximated upper and lower unbraced pile segments and the lower braced pile segment. Controlling value shown here is for a braced segment in pure compression using a resistance factor, ϕ_c=0.50, for severe driving conditions. The factored structural resistances for the upper segments use a resistance factor of ϕ_c=0.70 for combined axial loading and bending are not shown here, but are provided in Appendix C – Calculations.
[6] Based on guidance in LRFD Article 10.7.3.8.6, *Static analysis.* Static Resistance calculated using Driven 1.2 software (FHWA). Driven applies a limiting tip resistance suggested by Meyerhof. Nordlund Method used for shaft resistance with side friction resistance limited by SPT-Meyerhof below the critical depth of 20D. Resistance factor, φ_{dyn} = 0.65, selected assuming dynamic testing establishes driving criteria and measures nominal pile resistance in the field.
[7] Uses a resistance factor, φ_{dyn} = 0.65, assuming the driving criteria is established by dynamic testing, and quality control by dynamic testing of at least two piles per site condition and no less than two percent of production piles.
[8] Drivability resistance based on a Delmag D19-42. Drivability resistance with a Delmag D36-32 shown in parentheses.

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 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, ice loads, debris loads, and certain hydraulic events. Extreme limit state design shall also check that the nominal pile foundation resistance remaining after scour due to the check flood (Q_{500}) can support the extreme limit state loads. Resistance factors for extreme limit states, per LRFD Article 10.5.5.3, shall be taken as ϕ = 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 nominal axial geotechnical pile resistance of Abutment No. 1 piles at the service and extreme limit state was calculated using the guidance in LRFD Article 10.7.3.2.3. The calculated factored axial structural, geotechnical, and drivability resistances of two H-pile sections for the extreme and service limit states are provided in Table 5. Supporting documentation is provided in Appendix C – Calculations.

Pile Section	Abutment No. 1 Factored Axial Pile Resistance Extreme and Service Limit State			
	Structural Resistance[9] ϕ = 1.0 (kips)	Controlling Geotechnical Resistance ϕ = 1.0 (kips)	Drivability Resistance ϕ = 1.0 (kips)	Governing Axial Pile Resistance (kips)
HP 14 x 89	1199	1199	644	644
HP 14 x 117	1585	1585	786 (915)[10]	786 (915)[10]

Table 5 – Factored Axial Compressive Resistances for Abutment No. 1 H-Piles at Service and Extreme Limit States - Driven Piles End Bearing on Bedrock

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, the estimated factored axial pile resistances from the drivability analyses for the Abutment No. 1 H-pile sections are less than the controlling factored axial structural resistance per LRFD Article 10.7.3.2.3 and the nominal structural resistances. Therefore, drivability controls, and the recommended governing resistances for Abutment No. 1 pile design are the resistances provided in the rightmost column “Governing Axial Pile Resistance (kips)” in Table 5. The maximum applied factored axial pile load for Abutment No. 1 piles at the extreme and service limit states should not exceed the governing factored pile resistance shown in Table 5 above.

[9] Normal conditions consider no soil loss due to scour. Nominal compressive resistances were calculated for upper and lower unbraced pile segments and the lower braced pile segment. Controlling value shown here is for the lower unbraced pile segment, using a resistance factor, ϕ = 1.0.
[10] Drivability resistance based on a Delmag D19-42. Drivability resistance with a Delmag D36-32 shown in parentheses.

The nominal axial geotechnical pile resistance of Abutment No. 2 piles at the service and extreme limit state was calculated using the guidance in LRFD Article 10.7.3.8.6. The calculated factored axial structural, geotechnical, and drivability resistances of two H-pile sections for the extreme and service limit states are provided in Table 6. Supporting documentation is provided in Appendix C – Calculations.

Pile Section	Abutment No. 2 Factored Axial Pile Resistance Extreme and Service Limit State			
	Structural Resistance[11] $\phi = 1.0$ (kips)	Controlling Geotechnical Resistance $\phi = 1.0$ (kips)	Drivability Resistance $\phi = 1.0$ (kips)	Governing Axial Pile Resistance (kips)
HP 14 x 89	1305	459	500	459
HP 14 x 117	1720	545	529 (995)[12]	545

Table 6 – Factored Axial Compressive Resistances for Abutment No. 2 H-Piles at Service and Extreme Limit States - Driven Piles Friction and End Bearing in Glacial Till with Pile Tip at Approx. Elev. 110

The estimated factored axial pile resistances from the static pile analyses for the Abutment No. 2 H-pile sections are less than the axial pile resistances from the drivability analyses and the structural resistances. Therefore, the static pile resistance governs and the recommended governing resistances for Abutment No. 2 pile design are the controlling geotechnical resistances provided in the rightmost column "Governing Axial Pile Resistance (kips)" in Table 6. The maximum applied factored axial pile load for the extreme and service limit states should not exceed the governing factored pile resistance shown in Table 6 above.

7.1.3 Lateral Pile Resistance/Behavior

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

A series of lateral pile resistance analyses should be performed to evaluate pile behavior at the abutments using LPile, or similar, software. 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.

[11] Normal conditions consider no soil loss due to scour. Nominal compressive resistances were calculated for upper and lower unbraced pile segments and the lower braced pile segment. Controlling value shown here is for the lower unbraced pile segment, using a resistance factor, $\phi = 1.0$.
[12] Drivability resistance based on a Delmag D19-42. Drivability resistance with a Delmag D36-32 shown in parentheses.

Recommended geotechnical parameters for generation of soil-resistance (p-y) curves in lateral pile analyses are provided in Table 7. In general, the models developed should emulate the soil at the site by using the soil layers (referenced in Table 7 by elevations), and appropriate structural parameters and pile-head boundary conditions for the pile section(s) being analyzed. Other geotechnical parameters that are not provided in Table 7, but are required by the lateral pile resistance software, can be provided by the project geotechnical engineer upon request.

Soil Layer	Approx. Elevation of Soil Layer (feet)	Water Table Condition	Effective Unit Weight (lbs/ft^3)	k_s (lb/in^3)	Internal Angle of Friction
Medium dense, Granular Borrow.	155.0 – 144.0	Above/ Below	113	Top: 90 Bot: 60	32°
Very dense, Glacial Till.	144.0 – 110.0	Below	83	125	43°

Table 7 – Soil Parameters for Generation of Soil-Resistance (p-y) Curves

7.1.4 Driven Pile Resistance and Pile Quality Control

The contract plans shall require the contractor to perform two wave equation analyses (one for each abutment) of the proposed pile-hammer system and conduct dynamic pile load tests with signal matching at each abutment. 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. 24-hour (minimum) restrike tests are required at both abutments because piles may “walk” out of position due to sloping rock at Abutment No. 1 and piles end bearing in glacial till may “relax” at Abutment No. 2.

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 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. Stub abutments shall be designed to resist all lateral earth loads, vehicular loads, dead and live loads, and lateral forces transferred through the integral superstructure. The design of the integral abutment at the strength limit state shall consider reinforced-concrete structural design. Strength limit state design shall also consider changes in foundation conditions and foundation resistance after scour due to the design (Q_{100}) flood.

A resistance factor (ϕ) of 1.0 shall be used to assess abutment design at the service limit state, including: settlement, excessive horizontal movement, and movement resulting after scour due to the design (Q_{100}) flood. The overall stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, ϕ, of 0.65.

Extreme limit state design of integral abutments supported on H-piles shall include pile structural resistance, pile geotechnical resistance, pile resistance in combined axial and flexure, and overall stability. Resistance factors for extreme limit state shall be taken as 1.0. Extreme limit state design shall also check that the nominal foundation resistance remaining after scour due to the check (Q_{500}) flood can support the extreme limit state loads with a resistance factor of 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: angle of internal friction (ϕ) of 32 degrees, total unit weight (γ) of 125 pcf, and a soil-concrete interface friction angle (δ) of 20 degrees.

Integral abutment sections shall be designed to withstand a lateral earth load equal to the passive pressure state. Calculation of passive earth pressures should assume a Coulomb passive earth pressure coefficient, K_p, of 6.73. Developing full passive pressure assumes that the ratio of lateral abutment movement to abutment height (y/H) exceeds 0.005. If the calculated displacements are significantly less than that required to develop full passive pressure the designer may consider using the Rankine passive earth pressure coefficient of 3.25. A load factor for passive earth pressure is not specified in LRFD. For purposes of the integral abutment backwall reinforcing steel design, use a maximum load factor (γ_{EH}) of 1.50 to calculate factored passive earth pressures.

Additional lateral earth pressure due to live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for abutments if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination of the surcharge load, is permitted per LRFD Article 3.11.6.5. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 8:

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

Table 8 – Equivalent Height of Soil for Estimating Live Load Surcharge on Abutments

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

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

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

7.3 Return (U-Shaped) Wingwalls

U-shaped return wingwalls will be used in conjunction with the integral abutments. The wingwalls will range from approximately 10 to 19 feet in length and constructed monolithically with the abutments. Monolithic U-shaped wingwalls that are not cantilevered may be designed to be pile supported. A chamfer, typically 1 foot, should be used between the abutment and the wingwalls to minimize concrete shrinkage cracking caused by the abrupt change in thickness at the connection. The wingwalls are essentially retaining walls and shall be designed for all relevant load combinations at the strength, service, and extreme limit states specified in LRFD Articles 3.4.1, 11.5.5 and 11.6. The walls shall be designed to resist lateral earth pressures, vehicular loads, collision loads, and creep and temperature and shrinkage deformations. The wingwalls shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) per LRFD Article 3.11.6.4.

For design of the wingwalls, two load cases shall be considered. The first load case is where the wingwall is subjected to passive earth pressure to account for the bridge moving laterally and pushing the wingwall into the fill. This load case is considered under strength limit state. Calculation of passive earth pressures may assume a Rankine passive earth pressure coefficient, Kp, of 3.25 assuming small wingwall movements. A load factor for passive earth pressure is not specified in LRFD. A maximum load factor (γ_{EH}) of 1.50 is recommended to calculate factored passive earth pressures.

The second load case considers that the wingwall is subjected to active pressure and to collision loads on wall mounted bridge rail. This load case is considered under the extreme limit state. Calculation of active earth pressure shall use the Rankine active earth pressure coefficient, Ka, of 0.31 assuming a level backslope. See Appendix C – Calculations for supporting documentation.

There are no bearing resistance considerations for U-shaped wingwalls that are slope-tapered and designed to cantilever off of the abutment section. However, the wall sections shall be embedded a minimum of 5.3 feet for frost protection as discussed in Section 7.5 below.

Alternatively, U-shaped wingwalls constructed monolithically with the abutments may be designed to be pile supported. Refer to Sections 7.1.1 and 7.1.2 for pile design recommendations.

7.4 Settlement

The approximately 7.5 to 9.5-foot-thick fill unit encountered in the test borings is loose to dense in consistency. These coarse-grained materials undergo elastic, immediate, compression in response to an increase of vertical overburden pressure. The project calls for the vertical alignment of the new structure to closely match the existing structure. No significant increase in overburden pressure is anticipated. Any settlement that may take place should be small and occur relatively quickly. Construction loads could introduce elastic settlements and these settlements are also anticipated to be small and occur relatively quickly. Post construction settlement should be negligible with proper compaction of any replaced fill materials used during construction.

LRFD Article C10.7.2.3.1 states that settlement assessments of pile groups adequately embedded into dense granular soils may be waived when the equivalent footing for the pile group is located within the dense granular soils. Any settlement of the Abutment No. 2 pile group bearing within dense glacial till is anticipated to be minimal. Any settlement of the Abutment No. 1 pile group bearing on bedrock will be due to axial compression of the foundation piles and is anticipated to be minimal.

7.5 Frost Protection

Pile-supported integral abutments shall be embedded a minimum of 4.0 feet for frost protection per MaineDOT BDG Figure 5-2.

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, Northfield has a design freezing index (DFI) of approximately 1300 F-degree days. The anticipated coarse-grained fill material was assigned a water content of 20%. These components correlate to a frost depth of 5.3 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Brunswick, Maine has an air DFI from the Modberg database of approximately 1276 F-degree days. Brunswick was selected because it lies along the same isoline as Northfield and Northfield is not available in the Modberg database. A water content of 20% was used. These components correlate to a frost depth of approximately 5.8 feet.

Based on the MaineDOT BDG methodology it is recommended that foundations bearing on coarse-grained soils be designed with an embedment of approximately 5.3 feet for frost protection. See Appendix C – Calculations for supporting calculations.

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

7.6 Scour and Riprap

Grain size analyses were performed on samples of the glacial till deposit to generate grain size curves for determining parameters to be used in scour analyses. Sample BB-NBS-102;5D was assumed to be similar in nature to the soils likely to be exposed to scour conditions. The following streambed grain size parameters can be used in scour analyses:

- Average diameter of particle at 50 percent passing, D_{50} = 3.2 mm (coarse sand).
- Average diameter of particle at 95 percent passing, D_{95} = 44.5 mm (fine gravel).

The grain size curves are included in Appendix B – Laboratory Test Results.

The consequences of changes in foundation conditions resulting from the design (Q_{100}) and check (Q_{500}) floods for scour shall be considered at the strength and extreme limit states, respectively. Design at the strength limit state should consider loss of lateral and vertical support due to scour. Design at the extreme limit state should check that the nominal foundation resistance due to the check flood (Q_{500}) event is no less that the extreme limit state loads. At the service limit state, the design shall limit movements and ensure overall stability considering scour at the design load.

For scour protection of the pile supported abutments, the PDR indicates the bridge approach slopes and the abutment slopes will be armored with riprap. Refer to MaineDOT BDG Section 2.3.11.3 for information regarding scour design. Typically, the top of the riprap is located at, or above, the Q_{50} elevation.

Plain riprap shall conform to MaineDOT Standard Specification 703.26 – Plain and Hand Laid Riprap. The toe of the riprap section shall be constructed at least 1 foot below the streambed elevation. The riprap section shall be underlain by a 1-foot thick layer of bedding material conforming MaineDOT Standard Specification 703.19 and Class 1 nonwoven erosion control geotextile per MaineDOT Standard Details 610(02) and 610(03).

7.7 Seismic Design Considerations

The United States Geological Survey Seismic Design CD (Version 2.1) provided with the LRFD Manual, 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 Table 9:

Parameter	Design Value
Peak Ground Acceleration (PGA)	0.078g
Acceleration Coefficient (A_S)	0.09g
S_{DS} (Period = 0.2 sec)	0.19g
S_{D1} (Period = 1.0 sec)	0.07g
Site Class	C
Seismic Zone	1

Table 9 – Seismic Design Parameters

In conformance with LRFD Article 4.7.4 seismic analysis is not required for bridges in Seismic Zone 1 or 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.

See Appendix C – Calculations for supporting documentation.

7.8 Construction Considerations

Construction of the proposed structure will require pile driving. The contractor shall be responsible for excavating those portions of existing structure that conflict with the proposed piles by conventional excavation methods, pre-augering, predrilling, spudding, use of rock chisels, or down-hole hammers.

Numerous cobbles and boulders were encountered in the fill unit and glacial till soils. The contractor should assume the use of conventional excavation methods, pre-augering, predrilling, rock chisels, or down-hole hammers is necessary to clear obstructions and allow pile driving activities. The contractor should assume difficult pile driving even after the removal of obstructions. Care should be taken to drive H-piles within allowable tolerances without damaging the H-piles.

Excavations for the proposed abutments 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.

8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Bog Stream Bridge in Northfield, 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 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.

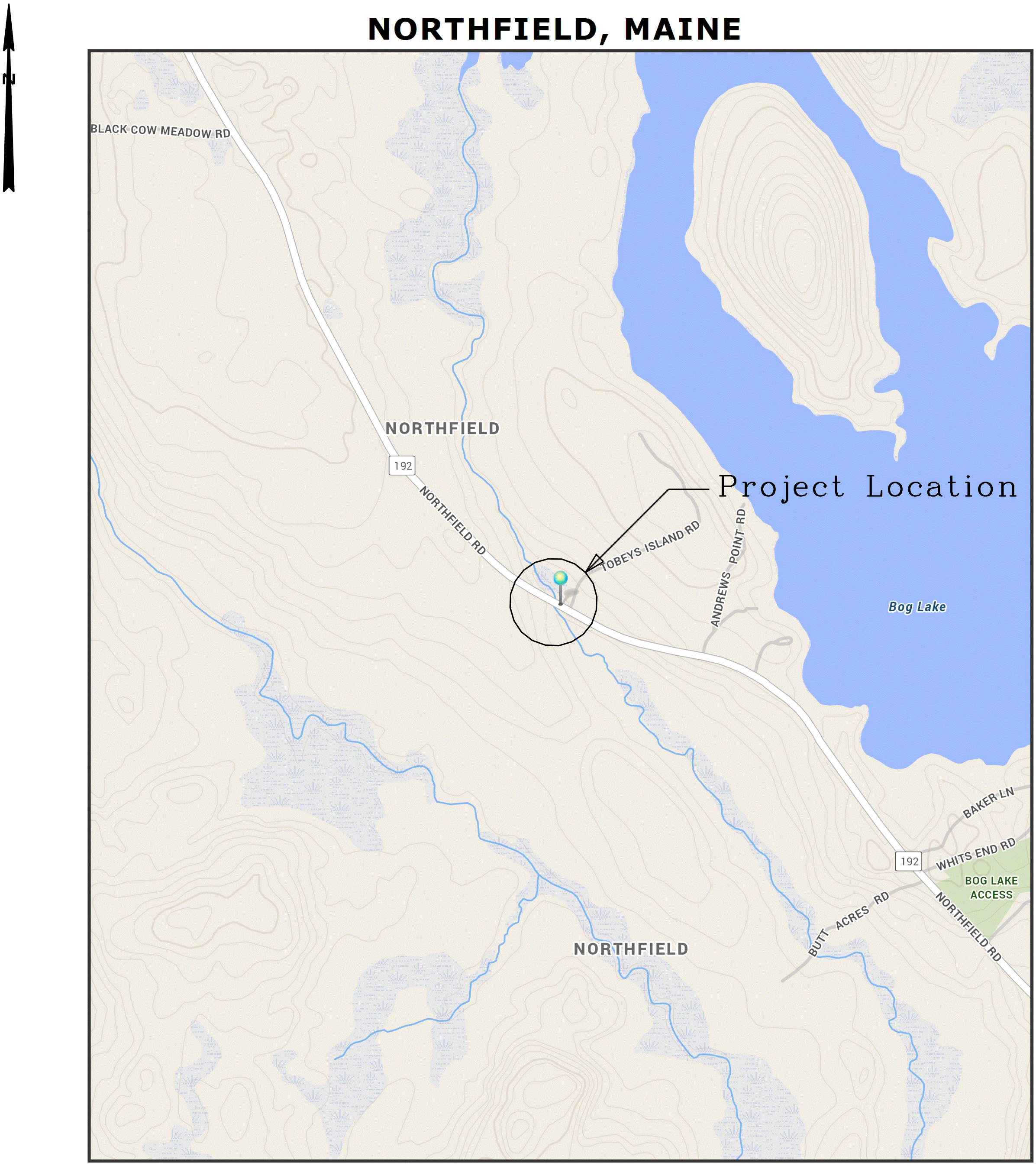
Sheets

Date:1/2/2018

Division: GEOTECH

Username: Terry.White

NORTHFIELD, MAINE

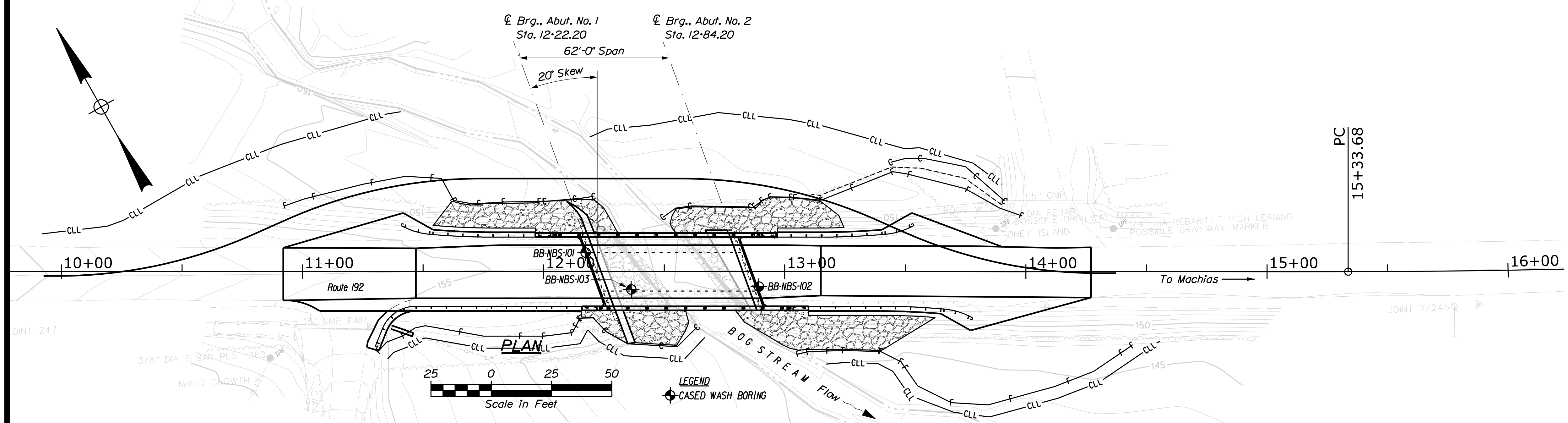


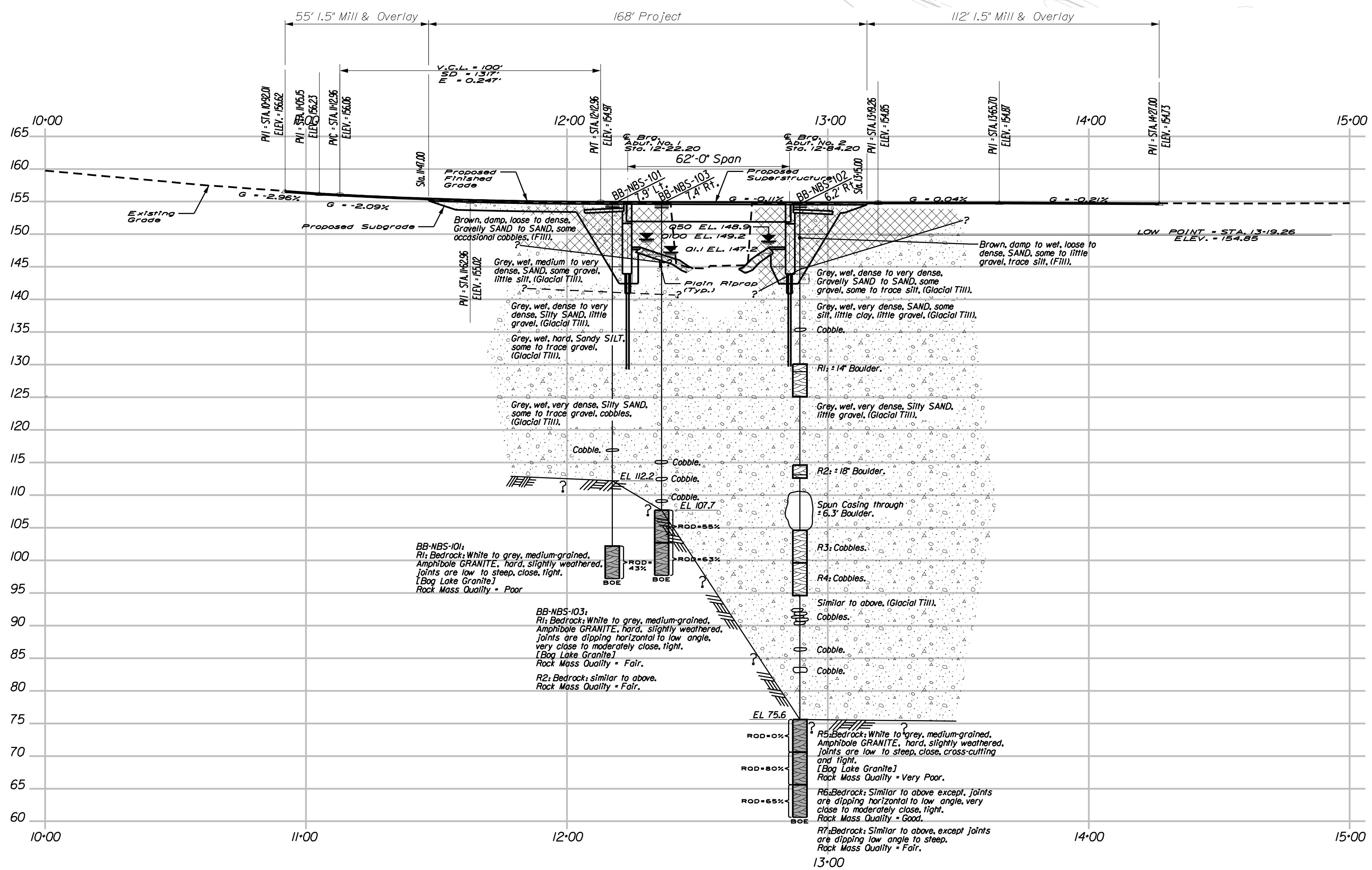
The Maine Department of Transportation provides this publication for information only. Reliance upon this information is at user risk. It is subject to revision and may be incomplete depending upon changing conditions. The Department assumes no liability if injuries or damages result from this information.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

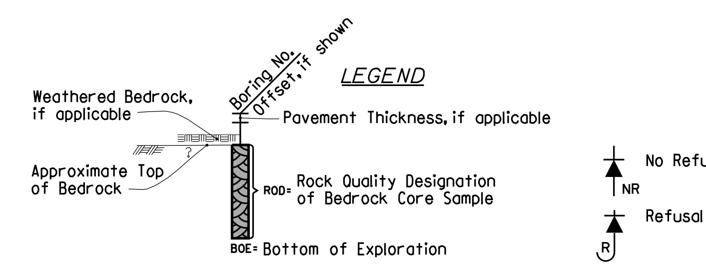
0.25 Miles
1 inch = 0.28 miles

Date: 1/2/2018
Time: 8:46:47 AM

SHEET NUMBER	BOG STREAM BRIDGE BOG STREAM	STATE OF MAINE DEPARTMENT OF TRANSPORTATION
1	NORTHFIELD AROOSTOOK COUNTY	STP-2169(900)
OF 3	LOCATION MAP	WIN 21699.00 BRIDGE NO. 3719 BRIDGE PLANS







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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2169(900)

WIN 21699.00

BRIDGE NO. 3719

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		
CHECKED-REVIEWED	T. WHITE	JUN 2018
DESIGN2-DETAILED2	B. SLOVEN	
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

BOG STREAM BRIDGE
BOG STREAM
NORTHFIELD WASHINGTON COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
2
OF 3

Filename: ...\GEOTECH\MSTA\007_BLP&SP1.dgn Division: GEOTECH Username: Brandon.Sloven Date:10/10/2018

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

Project: Bog Stream Bridge #3719 carries Route 192 over Bog Stream
Location: Northfield, Maine

Boring No.: BB-NBS-101
WIN: 21699.00

Driller:	MaineDOT	Elevation (ft.)	154.6	Auger ID/OD:	5" Solid Stem
Operator:	Daggett/Niles	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	6/13/2018; 07:00-16:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	12+17.4, 7.9 ft Lt.	Casing ID/OD:	HW-4"	Water Level*:	4.0 ft bgs.
Hammer Efficiency Factor: 0.928		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	154.1	5.5" HMA. (0.5)	
5	1D	12/10	5.00 - 6.00	8/50	---				Brown, damp, Gravelly SAND, trace silt, occasional cobbles, (Fill).	
								146.6	(8.0)	
10	2D	24/10	10.00 - 12.00	48/21/15/11	36	56	34		Grey, wet, very dense, SAND, some gravel, little silt, (Glacial Till).	G#296516 A-1-b, SM WC=14.1%
							99			
							80			
							74	141.1	(13.5)	
							65			
15	3D	24/14	15.00 - 17.00	5/9/11/17	20	31	67		Grey, wet, dense, Silty SAND, little gravel, (Glacial Till).	
							144			
							292			
							118			
							102			
20	4D	24/17	20.00 - 22.00	13/18/13/15	31	48	62		Grey, wet, hard, Sandy SILT, trace gravel, (Glacial Till). Roller Coned ahead to 25.0 ft bgs.	G#296517 A-4, SM WC=9.2%
							57			
							78			
							87			
							94	130.6	(24.0)	
25	5D	24/20	25.00 - 27.00	14/19/29/36	48	74	66		Grey, wet, very dense, Silty SAND, little gravel, (Glacial Till). Roller Coned ahead to 30.0 ft bgs.	
							70			
							71			
							73			
							66			
30	6D	24/19	30.00 - 32.00	9/18/20/23	38	59	67		Similar to above. Roller Coned ahead to 35.0 ft bgs.	
							83			
							86			
							97			
							96			
35	7D	24/14	35.00 - 37.00	11/22/24/28	46	71	96		Similar to above, except trace gravel. Roller Coned ahead to 40.0 ft bgs.	G#296518 A-4, SM WC=8.0%
							87			
							146		Cobble from 37.5-37.9 ft bgs.	
							184			
							RC	115.6	(39.0) Roller Coned ahead to 52.4 ft bgs.	
40	8D	24/15	40.00 - 42.00	16/46/46/50	92	142			Grey, wet, very dense, Silty SAND, some gravel, (Glacial Till).	
								112.2	(42.4) Probable Bedrock at 42.4 ft bgs.	
	R1	60/60	52.40 - 57.40	RQD = 43%			NQ-2	102.2	(52.4) R1: Bedrock: White to grey, medium-grained, Amphibole GRANITE, hard, slightly weathered, joints are low to steep, close, tight. [Bog Lake Granite] Rock Mass Quality = Poor. R1: Core Times (min:sec) 52.4-53.4 ft (5:02) 53.4-54.4 ft (4:52) 54.4-55.4 ft (5:03) 55.4-56.4 ft (5:20) 56.4-57.4 ft (5:10) 100% Recovery	
								97.2	(57.4) Bottom of Exploration at 57.4 feet below ground surface.	
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 1
Boring No.: BB-NBS-101

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

Project: Bog Stream Bridge #3719 carries Route 192 over Bog Stream
Location: Northfield, Maine

Boring No.: BB-NBS-102
WIN: 21699.00

Driller:	S. W. Cole Explorations, LLC	Elevation (ft.)	154.6	Auger ID/OD:	5" Solid Stem
Operator:	J. Lee	Datum:	NAVD88	Sampler:	Standard Split-Spoon
Logged By:	N. Strout	Rig Type:	Deidrich D50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	05/01/2017 - 05/02/2017	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	12+89.3, 6.2 ft Rt.	Casing ID/OD:	HW 4"/4.5" NW 3"/3.5"	Water Level*:	9.2' (on 05/02/2017)
Hammer Efficiency Factor: 0.873		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	154.1	6" of Pavement (0.5)	
	1D	24/8	1.00 - 3.00	11/13/9/7	22	32			Brown, damp, dense, SAND, some gravel, trace silt, (Fill).	
	2D	24/10	3.00 - 5.00	7/6/6/10	12	17			Similar to above, except medium dense.	
5	3D	24/14	5.00 - 7.00	6/4/2/2	6	9	8		Brown, moist, loose, SAND, little gravel, trace silt, (Fill).	
							15			
	4D	24/16	7.00 - 9.00	2/2/2/2	4	6	15		Brown, wet, loose, SAND, some gravel, trace silt, (Fill).	G#304379 A-1-b, SW-SM WC=7.2%
							21			
							18	144.6	(10.0)	
10	5D	24/9	10.00 - 12.00	15/11/16/16	27	39	31		Grey, wet, dense, Gravelly SAND, trace silt, (Glacial Till).	G#304380 A-1-a, SW-SM WC=10.3%
							68			
							54			
							53			
							46			
15	6D	24/13	15.00 - 17.00	9/17/20/26	37	54	24		Grey, wet, very dense, SAND, some gravel, some silt.	G#304381 A-1-b, SM WC=8.4%
							70	137.6	(17.0)	
	7D	24/12	17.00 - 19.00	33/30/27/29	57	83	100		Grey, wet, very dense, SAND, some silt, little clay, little gravel, (Glacial Till).	G#304382 A-4, SC-SM WC=8.0%
							157		±6" Cobble	
							211			
20	8D	24/22	20.00 - 22.00	20/47/45/49	92	134	111		Similar to above.	
							158			
							257			
							368			
	R1	60/14	24.50 - 29.50				241 NQ2	130.1	(24.5) ±14" Boulder	
							SPUN	125.1	(29.5)	
30	9D	24/24	30.00 - 32.00	20/25/32/44	57	83			Placed NW spin casing. Grey, wet, very dense, Silty SAND, little gravel, (Glacial Till).	
35	10D	24/24	35.00 - 37.00	21/26/40/43	66	96			Similar to above.	
40	R2	24/12	40.00 - 42.00				NQ2	114.6	(40.0) 18" Boulder.	
							SPUN	112.6	(42.0) ±6.3' Boulder	
50	R3	60/18	50.00 - 55.00				NQ2	104.6	(50.0) Advanced by rock core through boulder and frequent cobbles from 50 to 60 ft bgs.	
55	R4	60/10	55.00 - 60.00							
60	11D	17/9	60.00 - 61.42	27/47/100-5"	- -		SPUN	94.6	(60.0) Grey, wet, very dense, Silty SAND, little gravel, (Glacial Till). Frequent cobbles from 62 to 64.5 ft bgs.	
65	12D	16/8	65.00 - 66.33	42/68/100-4"	- -				Similar to above. ±6" Cobble.	
70	13D	8/7	70.00 - 70.67	25/100-3"	- -				Similar to above. ±10" Cobble.	
75										

Remarks:

Auto-hammer SN #362.
Casing driven using 140# auto-hammer with 30" drop.
Water level recorded prior to drilling on 05/02/2017 with casing to 45 ft bgs.

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

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

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

Project: Bog Stream Bridge #3719 carries Route 192 over Bog Stream
Location: Northfield, Maine

Boring No.: BB-NBS-102
WIN: 21699.00

(Header data as page 1: Driller S. W. Cole Explorations, LLC; Operator J. Lee; Logged By N. Strout; Date Start/Finish 05/01/2017 - 05/02/2017; Boring Location 12+89.3, 6.2 ft Rt.; Hammer Efficiency Factor: 0.873; Elevation (ft.) 154.6; Datum NAVD88; Rig Type Deidrich D50; Drilling Method Cased Wash Boring; Casing ID/OD HW 4"/4.5" NW 3"/3.5"; Auger ID/OD 5" Solid Stem; Sampler Standard Split-Spoon; Hammer Wt./Fall 140#/30"; Core Barrel NQ-2"; Water Level* 9.2' (on 05/02/2017))

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							OPEN			
								75.6	(79.0) Top of Bedrock at Elev 75.6 ft. Advanced by rollercone through Bedrock from 79 to 80 ft bgs.	
80	R5	48/11	80.00 - 84.00	RQD = 0%			NQ2		R5:Bedrock: White to grey, medium-grained, Amphibole GRANITE, hard, slightly weathered, joints are low to steep, close, cross-cutting and tight. [Bog Lake Granite] Rock Mass Quality = Very Poor. R5:Core Times (min:sec) 80.0-81.0 ft (3:16) 81.0-82.0 ft (3:54) 82.0-83.0 ft (4:43) 83.0-84.0 ft (4:27) 23% Recovery	
	R6	60/60	84.00 - 89.00	RQD = 80%				70.6	(84.0) R6:Bedrock: Similar to above except, joints are dipping horizontal to low angle, very close to moderately close, tight. Rock Mass Quality = Good. R6:Core Times (min:sec) 84.0-85.0 ft (5:00) 85.0-86.0 ft (4:02) 86.0-87.0 ft (4:13) 87.0-88.0 ft (4:43) 88.0-89.0 ft (4:50) 100% Recovery	
90	R7	60/60	89.00 - 94.00	RQD = 65%					R7:Bedrock: Similar to above, except joints are dipping low angle to steep. Rock Mass Quality = Fair. R7:Core Times (min:sec) 89.0-90.0 ft (3:41) 90.0-91.0 ft (3:35) 91.0-92.0 ft (3:10) 92.0-93.0 ft (3:15) 93.0-94.0 ft (4:01) 100% Recovery	
								60.6	(94.0) Bottom of Exploration at 94.0 feet below ground surface.	
100										

Remarks:

Auto-hammer SN #362.
Casing driven using 140# auto-hammer with 30" drop.
Water level recorded prior to drilling on 05/02/2017 with casing to 45 ft bgs.

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

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

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

Project: Bog Stream Bridge #3719 carries Route 192 over Bog Stream
Location: Northfield, Maine

Boring No.: BB-NBS-103
WIN: 21699.00

Driller:	MaineDOT	Elevation (ft.)	154.7	Auger ID/OD:	5" Solid Stem
Operator:	Daggett/Niles	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	6/14/2018; 07:00-15:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	12+36.3, 7.4 ft Rt.	Casing ID/OD:	HW-4"/NW-3"	Water Level*:	8.5 ft bgs.
Hammer Efficiency Factor: 0.928		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	154.0	8" HMA. (0.7)	
5	1D	24/12	5.00 - 7.00	2/1/2/2	3	5	19		Brown, damp, loose, SAND, some gravel, trace silt.	
								145.7	(9.0)	
10	2D	24/8	10.00 - 12.00	12/12/5/2	17	26	32		Grey, wet, medium dense, SAND, some gravel, little silt.	
								140.7	(14.0)	
15	3D	24/22	15.00 - 17.00	17/18/20/27	38	59	63		Grey, wet, very dense, Silty SAND, little gravel, (Glacial Till).	G#296519 A-4, SM WC=8.1%
20	4D	24/18	20.00 - 22.00	17/18/23/30	41	63	OPEN HOLE		Similar to above. Roller Coned ahead to 47.0 ft bgs.	
25	5D	24/15	25.00 - 27.00	12/28/39/44	67	104			Grey, wet, hard, fine to coarse Sandy SILT, some gravel, trace small cobbles.	
30	6D	24/20	30.00 - 32.00	12/12/25/37	37	57			Grey, wet, very dense, Silty SAND, little gravel, (Glacial Till).	
35	7D	24/19	35.00 - 37.00	8/16/23/33	39	60			Similar to above.	
40	8D	24/20	40.00 - 42.00	12/21/28/40	49	76			Cobble from 39.4-39.9 ft bgs. Similar to above. Cobble from 42.0-42.5 ft bgs.	
45	9D	4.8/4.8	45.00 - 45.41	50(4.8")	---		50		Grey, wet, very dense, Silty SAND, some gravel, (Glacial Till). Set in NW Casing at 45.0 ft bgs. Cobble from 45.4-45.8 ft bgs. Roller Coned ahead to 47.0 ft bgs.	
	R1	60/60	47.00 - 52.00	RQD = 55%			NQ-2	107.7	(47.0) Top of Bedrock at Elev. 107.7 ft. R1: Bedrock: White to grey, medium-grained, Amphibole GRANITE, hard, slightly weathered, joints are dipping horizontal to low angle, very close to moderately close, tight. [Bog Lake Granite] Rock Mass Quality = Fair. R1: Core Times (min:sec) 47.0-48.0 ft (5:50) 48.0-49.0 ft (3:12) 49.0-50.0 ft (3:16) 50.0-51.0 ft (3:40) 51.0-52.0 ft (4:02) 100% Recovery	
	R2	60/58	52.00 - 57.00	RQD = 63%					R2: Bedrock: similar to above. Rock Mass Quality = Fair. R2: Core Times (min:sec) 52.0-53.0 ft (3:05) 53.0-54.0 ft (3:25) 54.0-55.0 ft (5:00) 55.0-56.0 ft (5:00) 56.0-57.0 ft (5:45) 97% Recovery	
								97.7	(57.0) Bottom of Exploration at 57.0 feet below ground surface.	
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 1
Boring No.: BB-NBS-103

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2169(900)
WIN 21699.00
BRIDGE PLANS
BRIDGE NO. 3719

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		
CHECKED-REVIEWED		
DESIGN2-DETAILED2	B.SLAVEN	
DESIGN3-DETAILED3	T.WHITE	AUG 2018
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

BOG STREAM BRIDGE
BOG STREAM
NORTHFIELD WASHINGTON COUNTY

BORING LOGS

SHEET NUMBER
3
OF 3

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

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

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

MODIFIED BURMISTER SYSTEM

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

Rock Mass 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 mass quality (very poor, poor, etc.)
ref: ASTM D6032 and AASHTO Standard Specification for Highway Bridges, 17th Ed. Table 4.4.8.1.2A
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: Bog Stream Bridge #3719 carries Route 192 over Bog Stream
Location: Northfield, Maine

Boring No.: BB-NBS-101
WIN: 21699.00

Driller:	MaineDOT	**Elevation (ft.)**	154.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/13/2018; 07:00-16:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+17.4, 7.9 ft Lt.	**Casing ID/OD:**	HW-4"	**Water Level*:**	4.0 ft bgs.
Hammer Efficiency Factor:	0.928	**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	N60	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	154.1	5½" HMA. 0.5	
5	1D	12/10	5.00 - 6.00	8/50	---				Brown, damp, Gravelly SAND, trace silt, occasional cobbles, (Fill).	
								146.6	8.0	
10	2D	24/10	10.00 - 12.00	48/21/15/11	36	56	34		Grey, wet, very dense, SAND, some gravel, little silt, (Glacial Till).	G#296516 A-1-b, SM WC=14.1%
							99			
							80			
							74			
							65			
15	3D	24/14	15.00 - 17.00	5/9/11/17	20	31	67		Grey, wet, dense, Silty SAND, little gravel, (Glacial Till).	
							144			
							292			
							118			
							102			
20	4D	24/17	20.00 - 22.00	13/18/13/15	31	48	62		Grey, wet, hard, Sandy SILT, trace gravel, (Glacial Till).	G#296517 A-4, SM WC=9.2%
							57		Roller Coned ahead to 25.0 ft bgs.	
							78			
							87			
25							94			

Remarks:

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* 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: Bog Stream Bridge #3719 carries Route 192 over Bog Stream
Location: Northfield, Maine

Boring No.: BB-NBS-101
WIN: 21699.00

Driller:	MaineDOT	**Elevation (ft.)**	154.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/13/2018; 07:00-16:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+17.4, 7.9 ft Lt.	**Casing ID/OD:**	HW-4"	**Water Level*:**	4.0 ft bgs.
Hammer Efficiency Factor:	0.928	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
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WOH = Weight of 140 lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
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C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/20	25.00 - 27.00	14/19/29/36	48	74	66			Grey, wet, very dense, Silty SAND, little gravel, (Glacial Till). Roller Coned ahead to 30.0 ft bgs.	
							70				
							71				
							73				
							66				
30	6D	24/19	30.00 - 32.00	9/18/20/23	38	59	67			Similar to above. Roller Coned ahead to 35.0 ft bgs.	
							83				
							86				
							97				
							96				
35	7D	24/14	35.00 - 37.00	11/22/24/28	46	71	96			Similar to above, except trace gravel. Roller Coned ahead to 40.0 ft bgs.	G#296518 A-4, SM WC=8.0%
							87				
							146			Cobble from 37.5-37.9 ft bgs.	
							184				
							RC			Roller Coned ahead to 52.4 ft bgs.	
40	8D	24/15	40.00 - 42.00	16/46/46/50	92	142				Grey, wet, very dense, Silty SAND, some gravel, (Glacial Till).	
								112.2		42.4	
										Probable Bedrock at 42.4 ft bgs.	
45											
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Bog Stream Bridge #3719 carries Route 192 over Bog Stream
Location: Northfield, Maine

Boring No.: BB-NBS-101
WIN: 21699.00

Driller:	MaineDOT	**Elevation (ft.)**	154.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/13/2018; 07:00-16:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+17.4, 7.9 ft Lt.	**Casing ID/OD:**	HW-4"	**Water Level*:**	4.0 ft bgs.
Hammer Efficiency Factor:	0.928	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50											
	R1	60/60	52.40 - 57.40	RQD = 43%			NQ-2			R1: Bedrock: White to grey, medium-grained, Amphibole GRANITE, hard, slightly weathered, joints are low to steep, close, tight. [Bog Lake Granite] Rock Mass Quality = Poor. R1: Core Times (min:sec) 52.4-53.4 ft (5:02) 53.4-54.4 ft (4:52) 54.4-55.4 ft (5:03) 55.4-56.4 ft (5:20) 56.4-57.4 ft (5:10) 100% Recovery	
55											
								97.2		57.4	
										Bottom of Exploration at 57.4 feet below ground surface.	
60											
65											
70											
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: Bog Stream Bridge #3719 carries Route 192 over Bog Stream
Location: Northfield, Maine

Boring No.: BB-NBS-102
WIN: 21699.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	154.6	**Auger ID/OD:**	5" Solid Stem
Operator:	J. Lee	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	Deidrich D50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	05/01/2017 - 05/02/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+89.3, 6.2 ft Rt.	**Casing ID/OD:**	HW 4"/4.5" NW 3"/3.5"	**Water Level*:**	9.2' (on 05/02/2017)
Hammer Efficiency Factor:	0.873	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 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	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	154.1		6" of Pavement — 0.5	
	1D	24/8	1.00 - 3.00	11/13/9/7	22	32				Brown, damp, dense, SAND, some gravel, trace silt, (Fill).	
	2D	24/10	3.00 - 5.00	7/6/6/10	12	17				Similar to above, except medium dense.	
5	3D	24/14	5.00 - 7.00	6/4/2/2	6	9	8			Brown, moist, loose, SAND, little gravel, trace silt, (Fill).	
							15				
	4D	24/16	7.00 - 9.00	2/2/2/2	4	6	15			Brown, wet, loose, SAND, some gravel, trace silt, (Fill).	G#304379 A-1-b, SW-SM WC=7.2%
							21				
							18				
10	5D	24/9	10.00 - 12.00	15/11/16/16	27	39	31	144.6		10.0 — Grey, wet, dense, Gravelly SAND, trace silt, (Glacial Till).	G#304380 A-1-a, SW-SM WC=10.3%
							68				
							54				
							53				
							46				
15	6D	24/13	15.00 - 17.00	9/17/20/26	37	54	24			Grey, wet, very dense, SAND, some gravel, some silt.	G#304381 A-1-b, SM WC=8.4%
							70				
	7D	24/12	17.00 - 19.00	33/30/27/29	57	83	100	137.6		17.0 — Grey, wet, very dense, SAND, some silt, little clay, little gravel, (Glacial Till).	G#304382 A-4, SC-SM WC=8.0%
							157				
							211			±6" Cobble	
20	8D	24/22	20.00 - 22.00	20/47/45/49	92	134	111			Similar to above.	
							158				
							257				
							368				
25	R1	60/14	24.50 - 29.50				241	130.1		24.5	

Remarks:

Auto-hammer SN #362.
Casing driven using 140# auto-hammer with 30" drop.
Water level recorded prior to drilling on 05/02/2017 with casing to 45 ft bgs.

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: Bog Stream Bridge #3719 carries Route 192 over Bog Stream
Location: Northfield, Maine

Boring No.: BB-NBS-102

WIN: 21699.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	154.6	**Auger ID/OD:**	5" Solid Stem
Operator:	J. Lee	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	Deidrich D50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	05/01/2017 - 05/02/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+89.3, 6.2 ft Rt.	**Casing ID/OD:**	HW 4"/4.5" NW 3"/3.5"	**Water Level*:**	9.2' (on 05/02/2017)
Hammer Efficiency Factor:	0.873	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks		Laboratory Testing Results/ AASHTO and Unified Class.
25							NQ2			±14" Boulder		
							SPUN	125.1		Placed NW spin casing. Grey, wet, very dense, Silty SAND, little gravel, (Glacial Till).	29.5	
30	9D	24/24	30.00 - 32.00	20/25/32/44	57	83						
35	10D	24/24	35.00 - 37.00	21/26/40/43	66	96				Similar to above.		
40	R2	24/12	40.00 - 42.00				NQ2	114.6		18" Boulder.	40.0	
							SPUN	112.6			42.0	
										±6.3' Boulder		
45												
50												

Remarks:

Auto-hammer SN #362.
Casing driven using 140# auto-hammer with 30" drop.
Water level recorded prior to drilling on 05/02/2017 with casing to 45 ft bgs.

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: Bog Stream Bridge #3719 carries Route 192 over Bog Stream
Location: Northfield, Maine

Boring No.: BB-NBS-102
WIN: 21699.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	154.6	**Auger ID/OD:**	5" Solid Stem
Operator:	J. Lee	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	Deidrich D50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	05/01/2017 - 05/02/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+89.3, 6.2 ft Rt.	**Casing ID/OD:**	HW 4"/4.5" NW 3"/3.5"	**Water Level*:**	9.2' (on 05/02/2017)
Hammer Efficiency Factor:	0.873	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	R3	60/18	50.00 - 55.00				NQ2	104.6		50.0 Advanced by rock core through boulder and frequent cobbles from 50 to 60 ft bgs.	
55	R4	60/10	55.00 - 60.00								
60	11D	17/9	60.00 - 61.42	27/47/100-5"	- -		SPUN	94.6		60.0 Grey, wet, very dense, Silty SAND, little gravel, (Glacial Till).	
										Frequent cobbles from 62 to 64.5 ft bgs.	
65	12D	16/8	65.00 - 66.33	42/68/100-4"	- -					Similar to above.	
										±6" Cobble.	
70	13D	8/7	70.00 - 70.67	25/100-3"	- -					Similar to above. ±10" Cobble.	
75											

Remarks:

Auto-hammer SN #362.
Casing driven using 140# auto-hammer with 30" drop.
Water level recorded prior to drilling on 05/02/2017 with casing to 45 ft bgs.

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Bog Stream Bridge #3719 carries Route 192 over Bog Stream
Location: Northfield, Maine

Boring No.: BB-NBS-102
WIN: 21699.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	154.6	**Auger ID/OD:**	5" Solid Stem
Operator:	J. Lee	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	Deidrich D50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	05/01/2017 - 05/02/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+89.3, 6.2 ft Rt.	**Casing ID/OD:**	HW 4"/4.5" NW 3"/3.5"	**Water Level*:**	9.2' (on 05/02/2017)
Hammer Efficiency Factor:	0.873	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75							OPEN				
								75.6		79.0 Top of Bedrock at Elev 75.6 ft. Advanced by rollercone through Bedrock from 79 to 80 ft bgs.	
80	R5	48/11	80.00 - 84.00	RQD = 0%			NQ2			R5:Bedrock: White to grey, medium-grained, Amphibole GRANITE, hard, slightly weathered, joints are low to steep, close, cross-cutting and tight. [Bog Lake Granite] Rock Mass Quality = Very Poor. R5:Core Times (min:sec) 80.0-81.0 ft (3:16) 81.0-82.0 ft (3:54) 82.0-83.0 ft (4:43) 83.0-84.0 ft (4:27) 23% Recovery	
	R6	60/60	84.00 - 89.00	RQD = 80%				70.6		84.0	
85										R6:Bedrock: Similar to above except, joints are dipping horizontal to low angle, very close to moderately close, tight. Rock Mass Quality = Good. R6:Core Times (min:sec) 84.0-85.0 ft (5:00) 85.0-86.0 ft (4:02) 86.0-87.0 ft (4:13) 87.0-88.0 ft (4:43) 88.0-89.0 ft (4:50) 100% Recovery	
	R7	60/60	89.00 - 94.00	RQD = 65%							
90										R7:Bedrock: Similar to above, except joints are dipping low angle to steep. Rock Mass Quality = Fair. R7:Core Times (min:sec) 89.0-90.0 ft (3:41) 90.0-91.0 ft (3:35) 91.0-92.0 ft (3:10) 92.0-93.0 ft (3:15) 93.0-94.0 ft (4:01) 100% Recovery	
								60.6		94.0 **Bottom of Exploration at 94.0 feet below ground surface.**	
95											
100											

Remarks:

Auto-hammer SN #362.
Casing driven using 140# auto-hammer with 30" drop.
Water level recorded prior to drilling on 05/02/2017 with casing to 45 ft bgs.

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: Bog Stream Bridge #3719 carries Route 192 over Bog Stream Location: Northfield, Maine	Boring No.: BB-NBS-103 WIN: 21699.00

Driller:	MaineDOT	**Elevation (ft.)**	154.7	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/14/2018; 07:00-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+36.3, 7.4 ft Rt.	**Casing ID/OD:**	HW-4"/NW-3"	**Water Level*:**	8.5 ft bgs.
Hammer Efficiency Factor:	0.928	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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SSA = Solid Stem Auger
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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	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	154.0		8" HMA. 0.7	
5	1D	24/12	5.00 - 7.00	2/1/2/2	3	5	19			Brown, damp, loose, SAND, some gravel, trace silt.	
							10				
							19				
							18	145.7		9.0	
							56				
10	2D	24/8	10.00 - 12.00	12/12/5/2	17	26	32			Grey, wet, medium dense, SAND, some gravel, little silt.	
							55				
							36				
							57				
							147				
15	3D	24/22	15.00 - 17.00	17/18/20/27	38	59	63			Grey, wet, very dense, Silty SAND, little gravel, (Glacial Till).	G#296519 A-4, SM WC=8.1%
							63				
							56				
							68				
							62				
20	4D	24/18	20.00 - 22.00	17/18/23/30	41	63	OPEN HOLE			Similar to above. Roller Coned ahead to 47.0 ft bgs.	
25											

Remarks:

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Boring No.: BB-NBS-103

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Bog Stream Bridge #3719 carries Route 192 over Bog Stream **Location:** Northfield, Maine	**Boring No.:** BB-NBS-103 **WIN:** 21699.00

Driller:	MaineDOT	**Elevation (ft.)**	154.7	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/14/2018; 07:00-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+36.3, 7.4 ft Rt.	**Casing ID/OD:**	HW-4"/NW-3"	**Water Level*:**	8.5 ft bgs.
Hammer Efficiency Factor:	0.928	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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WOH = Weight of 140 lb. Hammer
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q_p = Unconfined Compressive Strength (ksf)
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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	24/15	25.00 - 27.00	12/28/39/44	67	104				Grey, wet, hard, fine to coarse Sandy SILT, some gravel, trace small cobbles.	
30	6D	24/20	30.00 - 32.00	12/12/25/37	37	57				Grey, wet, very dense, Silty SAND, little gravel, (Glacial Till).	
35	7D	24/19	35.00 - 37.00	8/16/23/33	39	60				Similar to above.	
										Cobble from 39.4-39.9 ft bgs.	
40	8D	24/20	40.00 - 42.00	12/21/28/40	49	76				Similar to above.	
										Cobble from 42.0-42.5 ft bgs.	
45	9D	4.89/4.8	45.00 - 45.41	50(4.8")	---		50			Grey, wet, very dense, Silty SAND, some gravel, (Glacial Till). Set in NW Casing at 45.0 ft bgs. Cobble from 45.4-45.8 ft bgs. Roller Coned ahead to 47.0 ft bgs.	
							83	107.7		47.0	
	R1	60/60	47.00 - 52.00	RQD = 55%			NQ-2			Top of Bedrock at Elev. 107.7 ft. R1: Bedrock: White to grey, medium-grained, Amphibole GRANITE, hard, slightly weathered, joints are dipping horizontal to low angle, very close to moderately close, tight. [Bog Lake Granite] Rock Mass Quality = Fair.	
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

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Location: Northfield, Maine

Boring No.: BB-NBS-103
WIN: 21699.00

Driller:	MaineDOT	**Elevation (ft.)**	154.7	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/14/2018; 07:00-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+36.3, 7.4 ft Rt.	**Casing ID/OD:**	HW-4"/NW-3"	**Water Level*:**	8.5 ft bgs.
Hammer Efficiency Factor:	0.928	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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WOH = Weight of 140 lb. Hammer
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$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
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N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
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LL = Liquid Limit
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C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										R1: Core Times (min:sec) 47.0-48.0 ft (5:50) 48.0-49.0 ft (3:12) 49.0-50.0 ft (3:16) 50.0-51.0 ft (3:40) 51.0-52.0 ft (4:02) 100% Recovery	
	R2	60/58	52.00 - 57.00	RQD = 63%						R2: Bedrock: similar to above. Rock Mass Quality = Fair. R2: Core Times (min:sec) 52.0-53.0 ft (3:05) 53.0-54.0 ft (3:25) 54.0-55.0 ft (5:00) 55.0-56.0 ft (5:00) 56.0-57.0 ft (5:45) 97% Recovery	
55								97.7		57.0	
										Bottom of Exploration at 57.0 feet below ground surface.	
60											
65											
70											
75											

Remarks:

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

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Appendix B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Northfield   Work Number: 21699.00

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-NBS-101, 2D	12+17.4	7.9 Lt.	10.0-12.0	296516	1	14.1			SM	A-1-b	II
BB-NBS-101, 4D	12+17.4	7.9 Lt.	20.0-22.0	296517	1	9.2			SM	A-4	III
BB-NBS-101, 7D	12+17.4	7.9 Lt.	35.0-37.0	296518	1	8.0			SM	A-4	III
BB-NBS-102, 4D	12+89.3	6.2 Rt.	7.0-9.0	304379	2	7.2			SW-SM	A-1-b	0
BB-NBS-102, 5D	12+89.3	6.2 Rt.	10.0-12.0	304380	2	10.3			SW-SM	A-1-a	0
BB-NBS-102, 6D	12+89.3	6.2 Rt.	15.0-17.0	304381	2	8.4			SM	A-1-b	II
BB-NBS-102, 7D	12+89.3	6.2 Rt.	17.0-19.0	304382	2	8.0			SC-SM	A-4	III
BB-NBS-103, 3D	12+36.3	7.4 Rt.	15.0-17.0	296519	1	8.1			SM	A-4	III

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

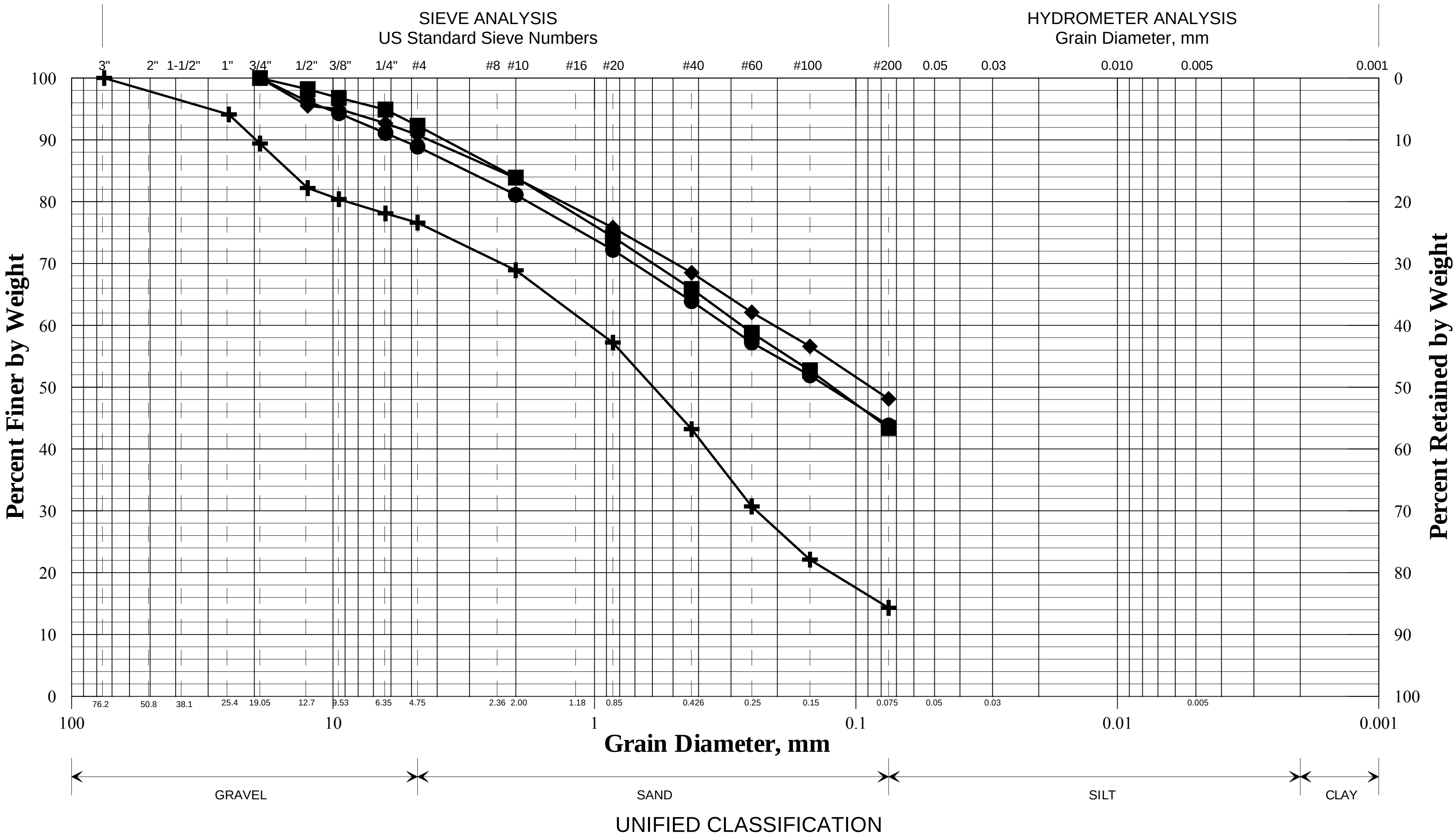
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

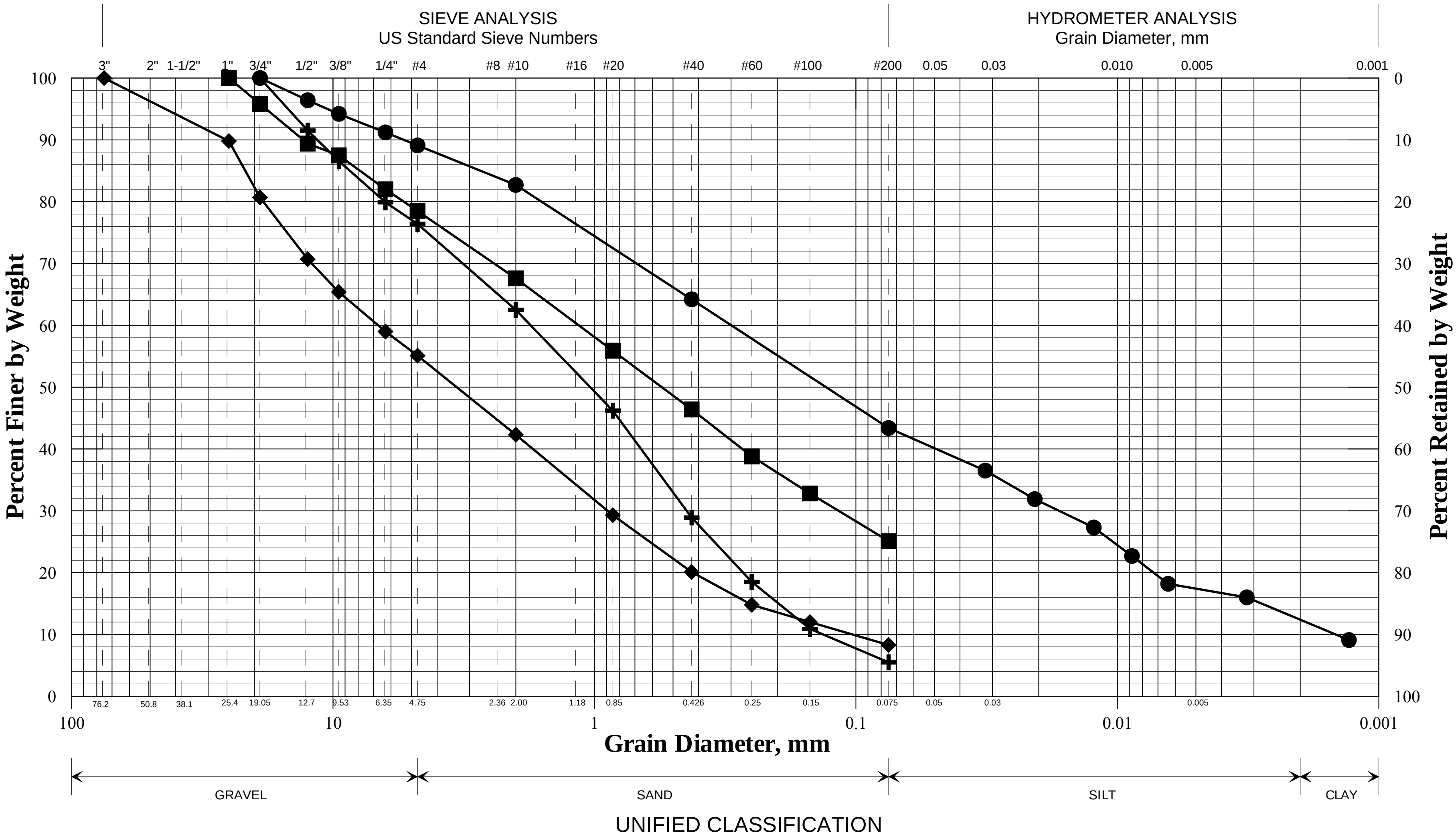


	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-NBS-101/2D	12+17.4	7.9 LT	10.0-12.0	SAND, some gravel, little silt.	14.1			
◆	BB-NBS-101/4D	12+17.4	7.9 LT	20.0-22.0	Sandy SILT, trace gravel.	9.2			
■	BB-NBS-101/7D	12+17.4	7.9 LT	35.0-37.0	Silty SAND, trace gravel.	8.0			
●	BB-NBS-103/3D	12+36.3	7.4 RT	15.0-17.0	Silty SAND, little gravel.	8.1			
▲									
✖									

WIN
021699.00
Town
Northfield
Reported by/Date
WHITE, TERRY A 7/13/2018

SHEET 1

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-NBS-102/4D	12+89.3	6.2 RT	7.0-9.0	SAND, some gravel, trace silt.	7.2			
◆	BB-NBS-102/5D	12+89.3	6.2 RT	10.0-12.0	Gravelly SAND, trace silt.	10.3			
■	BB-NBS-102/6D	12+89.3	6.2 RT	15.0-17.0	SAND, some silt, some gravel.	8.4			
●	HB-NBS-102/7D	12+89.3	6.2 RT	17.0-19.0	SAND, some silt, little clay, little gravel.	8.0			
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SHEET 2

Appendix C

Calculations

H-Pile Resistance

Design of H-piles

Reference: AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017 with 2018 Interim Revisions

Bedrock Properties

BB-NBS-103, R1 RQD = 55%
Rock Type: Granite, joints are horizontal to low angle.

ϕ = 34-40 (AASHTO LRFD Table C.10.4.6.4-1);
Co = 2,100 - 49,000 psi (AASHTO Standard Specifications for Bridges 17th Edition, Table 4.4.8.1.2B)

Pile Properties

Use the following piles: 14x89, 14x117

$$A_s := \begin{pmatrix} 26.1 \\ 34.4 \end{pmatrix} \cdot in^2 \qquad d := \begin{pmatrix} 13.80 \\ 14.20 \end{pmatrix} in \qquad b := \begin{pmatrix} 14.70 \\ 14.90 \end{pmatrix} in$$

Box Area

$$A_{box} := \overrightarrow{(d \cdot b)} \qquad A_{box} = \begin{pmatrix} 202.9 \\ 211.6 \end{pmatrix} \cdot in^2$$

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

Box Area Perimeter

$$A_{boxPerimeter} := d + d + b + b = \begin{pmatrix} 57 \\ 58.2 \end{pmatrix} \cdot in$$

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

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

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

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

Use LRFD Equation 6.9.2.1-1 Pr = ϕ Pn

Nominal Axial Structural Resistance

Determine equivalent yield resistance $P_o = QF_yA_s$ (LRFD 6.9.4.1.1)

$Q := 1.0$ LRFD Article 6.9.4.2

$P_o := Q \cdot F_y \cdot A_s$

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

14x89
14x117

A. Structural Resistance of upper "unbraced" segment of pile

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

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

K = effective length factor $K_{eff} := 1.2$

LRFD Table C4.6.2.5-1
(assume plastic hinge does not develop, K selected per Vtrans Design Procedure)

l = "unbraced" length $l_{unbraced\_Up} := 5.0 \cdot ft$

Assumed. LPile evaluations not performed at this time.

LRFD eq. 6.9.4.1.2-1

$$P_e := \overrightarrow{\left[\frac{\pi^2 \cdot E}{\left(\frac{K_{eff} \cdot l_{unbraced\_Up}}{r_s}\right)^2} \cdot A_s\right]}$$

$$P_e = \begin{pmatrix} 17957 \\ 24478 \end{pmatrix} \cdot kip$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 13.76 \\ 14.232 \end{pmatrix}$$

If Pe/Po > or = 0.44, then:

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

LRFD Eq. 6.9.4.1.1-1

then:

$$\boxed{P_{nUp} = \begin{pmatrix} 1266 \\ 1670 \end{pmatrix} \cdot kip}$$

this applies to all pile sizes

Factored Axial Structural Resistance at the Strength Limit State

Resistance factor for upper unbraced segments in combined compression and flexure per LRFD 6.5.4.2: $\phi_{cu} := 0.7$

The Factored Structural Resistance (P_r) per LRFD 6.9.2.1-1 is $P_r := \phi_{cu} \cdot P_{nUp}$

Factored structural compressive resistance, P_r

this applies to all pile sizes

$$\boxed{P_r = \begin{pmatrix} 886 \\ 1169 \end{pmatrix} \cdot kip}$$

B. Structural Resistance of lower "unbraced" segment of pile

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

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

K = effective length factor $K_{eff} := 1.0$

Per Vtrans Integral Abutment Design Guideline, use K=1 for 2nd unbraced segment that is pinned at the top and bottom. (see also LRFD Table C4.6.2.5-1) l = "unbraced" length $l_{unbraced\_Low} := 10.0 \cdot ft$ Assumed. LPile evaluations not performed at this time.

LRFD eq. 6.9.4.1.2-1

$$P_e := \overrightarrow{\left[\frac{\pi^2 \cdot E}{\left(\frac{K_{eff} \cdot l_{unbraced\_Low}}{r_s}\right)^2} \cdot A_s\right]}$$

$$P_e = \begin{pmatrix} 6464 \\ 8812 \end{pmatrix} \cdot kip$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 4.954 \\ 5.123 \end{pmatrix}$$

If Pe/Po > or = 0.44, then:

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

LRFD Eq. 6.9.4.1.1-1

then:

this applies to all pile sizes

$$\boxed{P_{nLow} = \begin{pmatrix} 1199 \\ 1585 \end{pmatrix} \cdot kip}$$

Factored Axial Structural Resistance at the Strength Limit State

Resistance factor for middle segment of H-pile in combined compression and flexure: $\phi_{cu} := 0.7$

The Factored Structural Resistance (P_r) per LRFD 6.9.2.1-1 is $P_r := \phi_{cu} \cdot P_{nLow}$

Factored structural compressive resistance, P_r

this applies to all pile sizes

$$\boxed{P_r = \begin{pmatrix} 839 \\ 1110 \end{pmatrix} \cdot kip}$$

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

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

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

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

l = "braced" length $l_{unbraced\_bot} := .1 \cdot ft$ Assume only the very tip is in pure compression

LRFD eq. 6.9.4.1.2-1

$$P_e := \overrightarrow{\left[\frac{\pi^2 \cdot E}{\left(\frac{K_{eff} \cdot l_{unbraced\_bot}}{r_s}\right)^2} \cdot A_s\right]}$$

$$P_e = \begin{pmatrix} 2 \times 10^8 \\ 2 \times 10^8 \end{pmatrix} \cdot kip$$

LRFD Article 6.9.4.1.1

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

If Pe/Po > or = 0.44, then:

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

LRFD Eq. 6.9.4.1.1-1

then:

this applies to all pile sizes

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

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

$$P_r := \phi_c \cdot P_{nBot}$$

Factored structural compressive resistance, P_r

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

LRFD 10.7.3.2.3 - Abutment No. 1 - 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 of Abutment No. 1 piles 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:

$$P_{nLow} = \begin{pmatrix} 1199 \\ 1585 \end{pmatrix} \cdot kip$$

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

$$\phi_c := 0.5$$

$$P_r := \phi_c \cdot P_{nLow}$$

$$\boxed{P_r = \begin{pmatrix} 600 \\ 793 \end{pmatrix} \cdot kip}$$

14x89
14x117

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

$$\phi_c := 1.0$$

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

$$P_{r\_ee} = \begin{pmatrix} 1199 \\ 1585 \end{pmatrix} \cdot kip$$ 14x89
14x117

Nominal and Factored Axial Geotechnical Resistance of HP piles

Geotechncial axial pile resistance for piles end bearing on rock determined by Intact Rock Method, proposed by Sandford, MaineDOT Transportation Research Division Technical Report 14-01, Phase 2 (January 2014), based on Rowe and Armitage (1987) equation cited by NCHRP Synthesis 360, Turner, (2006).

Nominal unit bearing resistance of pile point, Q_p

Design value of compressive strength of rock core

$$q_{u\_1} := 10960 \cdot psi$$

Assumed from UCT test results performed on similar granite from Lisbon, ME. UCT test restuls attached.

Geotechnical tip resistance.

$$q_{p\_2} := 2.5 \cdot q_{u\_1} \qquad q_{p\_2} = 3946 \cdot ksf$$

Nominal geotechnical tip resistance, R_p

$$R_p := \overrightarrow{\left(q_{p\_2} \cdot A_s\right)} \qquad R_p = \begin{pmatrix} 715 \\ 943 \end{pmatrix} \cdot kip$$

Factored Axial Geotechnical Compressive Resistance - *Strength Limit States*

Resistance factor, end bearing on rock Canadian Geotechnical Society method

$$\phi_{stat} := 0.45$$ LRFD Table 10.5.5.2.3-1

Factored Geotechnical Tip Resistance (R_r)

$$R_{r\_p2} := \phi_{stat} \cdot R_p \qquad R_{r\_p2} = \begin{pmatrix} 322 \\ 424 \end{pmatrix} \cdot kip$$

Factored Axial Geotechnical Compressive Resistance - *Service Limit States*

Resistance factor, end bearing on rock Canadian Geotechnical Society method

$$\phi := 1.0$$

Factored Geotechnical Tip Resistance (R_r)

$$R_{r\_p2} := \phi \cdot R_p \qquad \boxed{R_{r\_p2} = \begin{pmatrix} 715 \\ 943 \end{pmatrix} \cdot kip}$$

Factored Axial Geotechnical Compressive Resistance - *Extreme Limit States*

Resistance factor, end bearing on rock Canadian Geotechnical Society method

$$\phi_{ee} := 1.0$$

Factored Geotechnical Tip Resistance (R_r)

$$R_{r\_p2} := \phi_{ee} \cdot R_p \qquad \boxed{R_{r\_p2} = \begin{pmatrix} 715 \\ 943 \end{pmatrix} \cdot kip}$$

Drivability Analyses
Abutment No. 1 Piles - End Bearing on Bedrock
Abutment No. 2 Piles - Friction and End Bearing in Glacial Till

Ref: LRFD Article 10.7.8

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

$\phi_{da} := 1.0$ 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 5-15 blows per inch (bpi) per Section 501 (Note: 6-10 bpi is considered optimal for diesel hammers).

Compute the resistance that can be achieved in a drivablity 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

Estimated pile embedment will be approx. 34 ft. Assume contractor drives pile lengths of 45 ft (extra length accommodates for attachment of dynamic testing equipment, embedment into abutment, variation in tip elevation).

For piles driven to hard rock, use proportional shaft resistances so that GRLWeap will assign approx. 15% of the ultimate capacities as skin friction. For piles bearing in glacial till, use constant shaft resistance so that GRLWeap will assign approx. 336 Kips and 383 Kips, for 14x89 and 14x117, respectively, as skin friction, per Driven 1.2 and SPT-Meyerhof.

14 x 89 End Bearing on Hard Rock - Abut. 1

The 14x89 pile can be driven to the resistance below to hard rock with a D 19-42 hammer at max fuel setting and 1.9 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

Maine DOT — 20-Jul-2018
21699 Northfield 14x89 End — GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
610.0	43.68	2.92	10.7	10.32	21.47
620.0	43.96	2.94	11.1	10.35	21.54
630.0	44.21	2.96	11.5	10.39	21.68
640.0	44.49	2.96	11.9	10.42	21.75
644.0	44.64	2.92	12.0	10.44	21.77
650.0	44.82	2.92	12.3	10.46	21.82
660.0	45.12	2.88	12.7	10.51	21.94
670.0	45.38	2.83	13.0	10.54	22.03
680.0	45.63	2.73	13.5	10.58	22.10
690.0	45.91	3.05	13.9	10.62	22.16

DELMAG D 19-42

Ram Weight	4.00	kips
Efficiency	0.800	
Pressure	1600 (100%)	psi
Helmet Weight	1.90	kips
Hammer Cushion	60155	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	45.00	ft
Pile Penetration	34.00	ft
Pile Top Area	26.10	in2

Limiting driving stress to 45 ksi

$R_{ndr} := 644 \cdot kip$

Strength Limit State

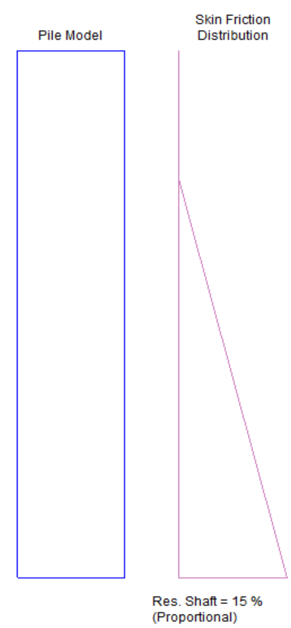
$R_{fdr} := R_{ndr} \cdot \phi_{dyn}$

$\boxed{R_{fdr} = 419 \cdot kip}$

Extreme and Service Limit States

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

$\boxed{R_{dr} = 644 \cdot kip}$



14 x 89 Friction and End Bearing in Glacial Till Pile - Abut. 2

The 14x89 pile can be driven to the resistance below to to bear in Glacial Till wtih a D 19-42 hammer at max fuel setting and 1.9 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

Maine DOT
21699 Northfield 14x89 Friction-Constant

02-Aug-2018
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
460.0	29.90	1.40	11.8	9.51	18.36
470.0	29.99	1.49	12.5	9.55	18.47
480.0	29.89	1.58	13.5	9.49	18.40
490.0	30.19	1.64	14.2	9.64	18.66
495.0	30.23	1.68	14.6	9.66	18.71
500.0	30.25	1.73	15.0	9.67	18.80
510.0	30.34	1.81	16.0	9.71	18.91
520.0	30.42	1.86	17.4	9.75	18.92
530.0	30.52	1.93	18.7	9.79	19.03
540.0	30.61	1.99	20.2	9.83	19.13

DELMAG D 19-42

Ram Weight	4.00	kips
Efficiency	0.800	
Pressure	1600 (100%)	psi
Helmet Weight	1.90	kips
Hammer Cushion	60155	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	45.00	ft
Pile Penetration	34.00	ft
Pile Top Area	26.10	in2

Limiting to 15 bpi

$R_{ndr} := 500 \cdot kip$

Strength Limit State

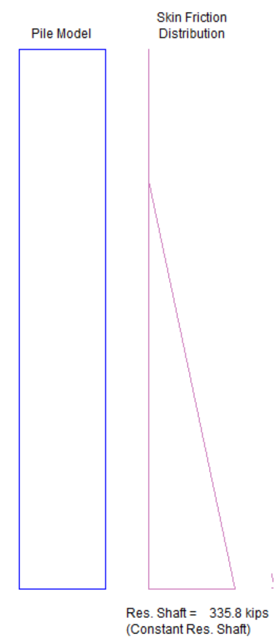
$R_{fdr} := R_{ndr} \cdot \phi_{dyn}$

$\boxed{R_{fdr} = 325 \cdot kip}$

Extreme and Service Limit States

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

$\boxed{R_{dr} = 500 \cdot kip}$



14 x 117 End Bearing on Hard Rock - Abut. 1 - D19-42

The 14x117 pile can be driven to the resistances below to bear on hard rock with a D 19-42 hammer at max fuel setting and 2.7 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

Maine DOT — 20-Jul-2018

21699 Northfield 14x117 End Hard Rock — GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
770.0	38.26	4.45	14.3	9.99	21.13
780.0	38.48	4.57	14.7	10.01	21.22
786.0	38.51	4.62	15.0	10.02	21.24
790.0	38.69	4.67	15.1	10.04	21.33
800.0	38.82	4.75	15.6	10.06	21.37
810.0	38.99	4.78	16.1	10.08	21.36
820.0	39.16	4.87	16.6	10.10	21.40
830.0	39.34	4.97	17.0	10.13	21.50
840.0	39.58	5.05	17.6	10.14	21.54
850.0	39.73	5.16	18.1	10.16	21.59

DELMAG D 19-42

Ram Weight	4.00	kips
Efficiency	0.800	
Pressure	1600 (100%)	psi
Helmet Weight	2.70	kips
Hammer Cushion	109975	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	45.00	ft
Pile Penetration	34.00	ft
Pile Top Area	34.40	in2

Limit blows to 15 bpi

$R_{ndr} := 786 \cdot kip$

Strength Limit State

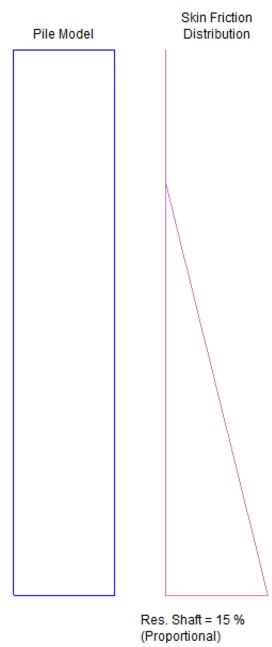
$R_{fdr} := R_{ndr} \cdot \phi_{dyn}$

$\boxed{R_{fdr} = 511 \cdot kip}$

Extreme and Service Limit States

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

$\boxed{R_{dr} = 786 \cdot kip}$



14 x 117 Friction and End Bearing Pile in Glacial Till - Abut. 2 - D19-42

The 14x117 pile can be driven to the resistances below to bear in glacial till with a D 19-42 hammer at max fuel setting and 2.7 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

Maine DOT 03-Aug-2018
Northfield 14x117 Friction-Const 19-42 GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
500.0	27.61	1.01	12.9	9.52	17.85
510.0	27.45	1.07	13.8	9.43	17.65
520.0	27.55	1.14	14.3	9.47	17.78
529.0	27.60	1.19	14.9	9.49	17.82
530.0	27.52	1.19	15.1	9.48	17.74
540.0	27.60	1.24	15.7	9.53	17.85
550.0	27.74	1.30	16.4	9.56	17.97
560.0	27.76	1.35	17.2	9.58	18.01
570.0	27.82	1.40	18.1	9.60	18.07
580.0	27.92	1.45	18.9	9.64	18.19

Limit blows to 15 bpi

$R_{ndr} := 529 \cdot kip$

Strength Limit State

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

$\boxed{R_{fdr} = 344 \cdot kip}$

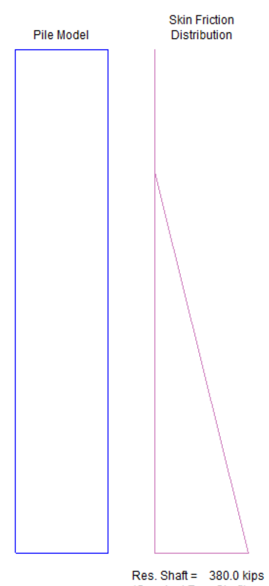
Extreme and Service Limit States

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

$\boxed{R_{dr} = 529 \cdot kip}$

DELMAG D 19-42

Ram Weight	4.00	kips
Efficiency	0.800	
Pressure	1600 (100%)	psi
Helmet Weight	1.90	kips
Hammer Cushion	60155	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	45.00	ft
Pile Penetration	34.00	ft
Pile Top Area	34.40	in2



14 x 117 End Bearing on Hard Rock - Abut. 1 - D36-32

The 14x117 pile can be driven to the resistances below to bear on hard rock with a D 36-32 hammer at 73% of max (-3) fuel setting and 2.7 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

Maine DOT — 23-Jul-2018

Northfield 14x117 End Hard Rock 36-32 — GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
830.0	42.65	4.21	10.0	7.66	29.97
860.0	43.45	4.42	11.0	7.76	30.37
890.0	44.10	4.53	12.0	7.84	30.80
910.0	44.55	4.57	12.8	7.90	31.11
915.0	44.70	4.61	13.0	7.91	31.12
920.0	44.80	4.61	13.2	7.92	31.25
930.0	45.02	4.64	13.6	7.95	31.39
940.0	45.24	4.67	14.0	7.98	31.53
950.0	45.45	4.69	14.5	8.01	31.68
960.0	45.37	4.70	15.3	7.97	31.45

DELMAG D 36-32

Ram Weight	7.93	kips
Efficiency	0.800	
Pressure	1095 (73%)	psi
Helmet Weight	2.70	kips
Hammer Cushion	109975	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	45.00	ft
Pile Penetration	34.00	ft
Pile Top Area	34.40	in2

Limit stress to 45 ksi

$$R_{ndr} := 915 \cdot kip$$

Strength Limit State

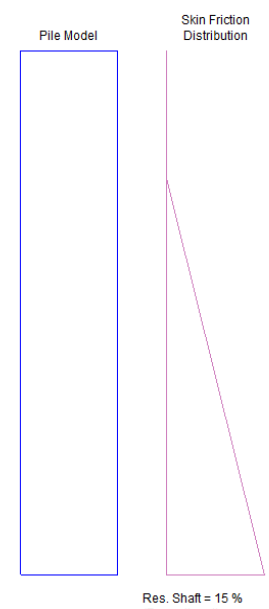
$$R_{fdr} := R_{ndr} \cdot \phi_{dyn}$$

$$\boxed{R_{fdr} = 595 \cdot kip}$$

Extreme and Service Limit States

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

$$\boxed{R_{dr} = 915 \cdot kip}$$



14 x 117 Friction and End Bearing Pile in Glacial Till - Abut. 2 - D36-32

The 14x117 pile can be driven to the resistances below to bear in glacial till with a D 36-32 hammer at max fuel setting and 2.7 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

Maine DOT
Northfield 14x117 Friction-Const 36-32

03-Aug-2018
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
900.0	39.66	2.67	10.4	10.36	46.58
950.0	40.20	2.91	12.6	10.55	47.34
960.0	40.31	3.02	13.0	10.58	47.64
970.0	40.39	3.04	13.6	10.61	47.70
975.0	40.50	3.08	13.8	10.64	47.88
980.0	40.45	3.13	14.1	10.65	47.97
985.0	40.54	3.12	14.5	10.67	47.97
990.0	40.65	3.18	14.6	10.69	48.20
995.0	40.60	3.22	14.9	10.70	48.28
1000.0	40.63	3.24	15.3	10.71	48.32

Limit blows to 15 bpi

$R_{ndr} := 995 \cdot kip$

Strength Limit State

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

$\boxed{R_{fdr} = 647 \cdot kip}$

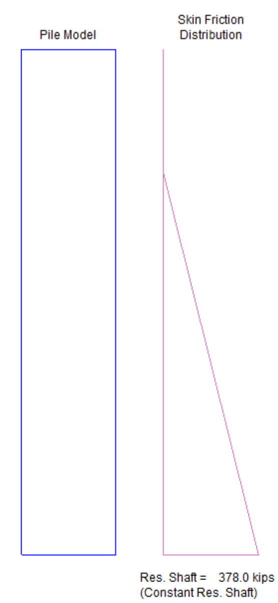
Extreme and Service Limit States

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

$\boxed{R_{dr} = 995 \cdot kip}$

DELMAG D 36-32

Ram Weight	7.93 kips
Efficiency	0.800
Pressure	1500 (100%) psi
Helmet Weight	2.70 kips
Hammer Cushion	109975 kips/in
COR of H.C.	0.800
Skin Quake	0.100 in
Toe Quake	0.100 in
Skin Damping	0.200 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	45.00 ft
Pile Penetration	34.00 ft
Pile Top Area	34.40 in2



Assumed Depth to Groundwater: 7 ft

Boring	Sample	Depth Interval	Soil Layer	Total Unit Weight (pcf)	Total Overburden Pressure[3] (psf)	Water Table Condition	Effective Overburden Pressure[3] (psf)	N_{60}	C_n[1]	$(N_1)_{60}$[2]
BB-NBS-102	1D	1-3	1	125	250	Above	250	32	1.70	54
BB-NBS-102	2D	3-5	1	125	500	Above	500	17	1.47	25
BB-NBS-102	3D	5-7	1	125	750	Above	750	9	1.33	12
BB-NBS-102	4D	7-9	2	142	1000	Below	938	6	1.26	8
BB-NBS-102	5D	10-12	3	145	1426	Below	1176	39	1.18	46
BB-NBS-102	6D	15-17	3	145	2151	Below	1589	54	1.08	58
BB-NBS-102	7D	17-19	3	145	2441	Below	1755	83	1.05	87
BB-NBS-102	8D	20-22	3	145	2876	Below	2002	134	1.00	134
BB-NBS-102	9D	30-32	3	145	4326	Below	2828	83	0.89	74
BB-NBS-102	10D	35-37	3	145	5051	Below	3241	96	0.84	81

[1] $C_n=[0.77*\log_{10}(40/\sigma'v)]$ LRFD 10.4.6.2.4-1 pg 10-17, limit C_n to 2.0 (σ'_v in KSF)

[2] $(N_1)_{60}=C_n*N_{60}$ LRFD 10.4.6.2.4-3 pg 10-17

[3] At middle of SPT interval

Pile Properties						
Size	Depth, D (in)	Flange (in)	Box Area (in^2) (Plugged Condition)	Half Area (in^2) (Half Plugged)	Steel Area, Ap (in^2)	Box Area Perimeter (in)
14x89	13.86	14.695	203.67	101.84	26.1	57.11
14x117	14.21	14.885	211.52	105.76	34.4	58.19

Nominal Meyerhof Tip Resistance												
							14x89			14x117		
Soil Layer	Design $(N_1)_{60}$[1]	Friction Angle[2] (Deg)	Elev.	Midpoint Elev.	Penetration, Db (ft)	Controling Limiting Tip Resistance, q_l[3] (ksf)	Db/D	Meyerhof Tip Resistance, q_p[4] (ksf)	Nominal Resistance, R_p[5] (Kips)	Db/D	Meyerhof Tip Resistance, q_p[3] (ksf)	Nominal Resistance, R_p[5] (Kips)
1	30	32	155-147	151	0	243	29.4	716		28.7	698	
2	8	32	147-144	145.5	0	60	29.4	177		28.7	173	
3	80	43	144-110	127	34	639	29.4	1882	116	28.7	1835	153

[1]Average of N60 values within a soil layer corrected for overburden pressure.

[2]Estimated based on the correlation of $(N_1)_{60}$ and phi provided in LRFD Table 10.4.6.2.4-1

[3]q_l limit is 8 times $(N_1)_{60}$ for sand. LRFD pg. 10-112 (in ksf)

[4]$(N_1)_{60}$*0.8(Db/D) LRFD 10.7.3.8.6g-1 pg. 10-122 (in ksf)

[5](q_l*Ap) LRFD 10.7.3.8.6a-3 pg 10-103

Nominal Meyerhof Skin Friction Resistance						
			14x89		14x117	
Soil Layer	Design $(N_1)_{60}$	Meyerhof, q_s[6] (ksf)	Nominal Shaft Resistance, R_s[7] (Kips)	Cumulative Shaft Resistance	Nominal Shaft Resistance, R_s[7] (Kips)	Cumulative Shaft Resistance
1	30	0.61	0		0	
2	8	0.15	0	0	0	0
3	80	1.60	259	259	263	263

[6]$(N_1)_{60}$/50 LRFD 10.7.3.8.6g-3 for nondisplacement piles

[7]$R_s=q_s*A_s$ LRFD 10.7.3.8.6a-4 pg. 10-103, A_s= Box Area Perimeter*Penetration

	Meyerhof Static Method		Nordlund Static Method (Driven	
	Total Nominal Resistance[8] (kips)	Total Factored Resistance[9] (Kips)	Total Nominal Resistance[8] (kips)	Total Factored Resistance[9] (Kips)
14x89	374	243	459	298
14x117	416	271	545	354

[8]R_p+R_s

[9]$R_r=\phi R_p+\phi R_s$, ϕ=0.65 (LRFD Table 10.5.5.2.3-1)for dynamic testing during installation

shall be corrected for the effects of overburden pressure determined as:

$$N1 = C_N N \qquad (10.4.6.2.4\text{-}1)$$

$N1$ = *SPT* blow count corrected for overburden pressure, σ'_v (blows/ft)
C_N = $[0.77 \log_{10}(40/\sigma'_v)]$, and $C_N < 2.0$
σ'_v = vertical effective stress (ksf)
N = uncorrected *SPT* blow count (blows/ft)

SPT N values should also be corrected for hammer efficiency, if applicable to the design method or correlation being used, determined as:

$$N_{60} = (ER/60\%)N \qquad (10.4.6.2.4\text{-}2)$$

where:

$N60$ = *SPT* blow count corrected for hammer efficiency (blows/ft)
ER = hammer efficiency expressed as percent of theoretical free fall energy delivered by the hammer system actually used
N = uncorrected *SPT* blow count (blows/ft)

When *SPT* blow counts have been corrected for both overburden effects and hammer efficiency effects, the resulting corrected blow count shall be denoted as $N1_{60}$, determined as:

$$N1_{60} = C_N N_{60} \qquad (10.4.6.2.4\text{-}3)$$

The drained friction angle of granular deposits should be determined based on the following correlation.

Table 10.4.6.2.4-1—Correlation of *SPT* $N1_{60}$ Values to Drained Friction Angle of Granular Soils (modified after Bowles, 1977)

$N1_{60}$	ϕ_f
<4	25–30
4	27–32
10	30–35
30	35–40
50	38–43

full grain size range of the soil to be included in the specimen. This may not always be possible, and if not possible, it should be recognized that the shear strength measured would likely be conservative.

A method using the results of *SPT* testing is presented. Other in-situ tests such as *CPT* and *DMT* may be used. For details on determination of ϕ_f from these tests, refer to Sabatini et al. (2002).

The use of automatic trip hammers is increasing. In order to use correlations based on standard rope and cathead hammers, the *SPT N* values must be corrected to reflect the greater energy delivered to the sampler by these systems.

Hammer efficiency (ER) for specific hammer systems used in local practice may be used in lieu of the values provided. If used, specific hammer system efficiencies shall be developed in general accordance with ASTM D4945 for dynamic analysis of driven piles or other accepted procedure.

The following values for ER may be assumed if hammer specific data are not available, e.g., from older boring logs:

ER = 60 percent for conventional drop hammer using rope and cathead
ER = 80 percent for automatic trip hammer

Corrections for rod length, hole size, and use of a liner may also be made if appropriate. In general, these are only significant in unusual cases or where there is significant variation from standard procedures. These corrections may be significant for evaluation of liquefaction. Information on these additional corrections may be found in Youd and Idriss (1997).

The $N1_{60}$-ϕ_f correlation used is modified after Bowles (1977). The correlation of Peck, Hanson, and Thornburn (1974) falls within the ranges specified. Experience should be used to select specific values within the ranges. In general, finer materials or materials with significant silt-sized material will fall in the lower portion of the range. Coarser materials with less than five percent fines will fall in the upper portion of the ranges. The geologic history and angularity of the particles may also need to be considered when selecting a value for ϕ_f.

Care should be exercised when using other correlations of *SPT* results to soil parameters. Some published correlations are based on corrected values ($N1_{60}$) and some are based on uncorrected values (N).

The designer should ascertain the basis of the correlation and use either $N1_{60}$ or N as appropriate.

Care should also be exercised when using *SPT* blow counts to estimate soil shear strength if in soils with coarse gravel, cobbles, or boulders. Large gravels, cobbles, or boulders could cause the *SPT* blow counts to be unrealistically high.

10.7.3.8.6g—Using SPT or CPT in Cohesionless Soils

These methods shall be applied only to sands and nonplastic silts.

The nominal unit tip resistance for the Meyerhof method, in ksf, for piles driven to a depth D_b into a cohesionless soil stratum shall be taken as:

$$q_p = \frac{0.8(N1_{60})D_b}{D} \leq q_\ell \qquad (10.7.3.8.6g\text{-}1)$$

where:

$N1_{60}$ = representative *SPT* blow count near the pile tip corrected for overburden pressure as specified in Article 10.4.6.2.4 (blows/ft)
D = pile width or diameter (ft)
D_b = depth of penetration in bearing strata (ft)
q_ℓ = limiting tip resistance taken as eight times the value of $N1_{60}$ for sands and six times the value of $N1_{60}$ for nonplastic silt (ksf)

The nominal side resistance of piles in cohesionless soils for the Meyerhof method, in ksf, shall be taken as:

- For driven displacement piles:

$$q_s = \frac{\bar{N}1_{60}}{25} \qquad (10.7.3.8.6g\text{-}2)$$

- For nondisplacement piles, e.g., steel H-piles:

$$q_s = \frac{\bar{N}1_{60}}{50} \qquad (10.7.3.8.6g\text{-}3)$$

where:

q_s = unit side resistance for driven piles (ksf)
$\bar{N}1_{60}$ = average corrected *SPT*-blow count along the pile side (blows/ft)

Tip resistance, q_p, for the Nottingham and Schmertmann method, in ksf, shall be determined as shown in Figure 10.7.3.8.6g-1.

In which:

$$q_p = \frac{q_{c1} + q_{c2}}{2} \qquad (10.7.3.8.6g\text{-}4)$$

where:

q_{c1} = average q_c over a distance of yD below the pile tip (path a-b-c); sum q_c values in both the downward (path a-b) and upward (path b-c)

C10.7.3.8.6g

In-situ tests are widely used in cohesionless soils because obtaining good quality samples of cohesionless soils is very difficult. In-situ test parameters may be used to estimate the tip resistance and side resistance of piles.

Two frequently used in-situ test methods for predicting pile axial resistance are the standard penetration test (*SPT*) method (Meyerhof, 1976) and the cone penetration test (*CPT*) method (Nottingham and Schmertmann, 1975).

Displacement piles, which have solid sections or hollow sections with a closed end, displace a relatively large volume of soil during penetration. Nondisplacement piles usually have relatively small cross-sectional areas, e.g., steel H-piles and open-ended pipe piles that have not yet plugged. Plugging occurs when the soil between the flanges in a steel H-pile or the soil in the cylinder of an open-ended steel pipe pile adheres fully to the pile and moves down with the pile as it is driven.

CPT may be used to determine:

- The cone penetration resistance, q_c, which may be used to determine the tip resistance of piles, and
- Sleeve friction, f_s, which may be used to determine the side resistance.

If a dynamic formula other than those provided herein is used, it shall be calibrated based on measured load test results to obtain an appropriate resistance factor, consistent with Article C10.5.5.2.

If a drivability analysis is not conducted, for steel piles, design stresses shall be limited as specified in Article 6.15.2.

Dynamic formulas should not be used when the required nominal resistance exceeds 600 kips.

As the required nominal bearing resistance increases, the reliability of dynamic formulas tends to decrease. The FHWA Gates formula tends to underpredict pile nominal resistance at higher resistances. The Engineering News formula tends to become unconservative as the nominal pile resistance increases. If other driving formulas are used, the limitation on the maximum driving resistance to be used should be based upon the limits for which the data is considered reliable, and any tendency of the formula to over or under predict pile nominal resistance.

10.7.3.8.6—Static Analysis

10.7.3.8.6a—General

Where a static analysis prediction method is used to determine pile installation criteria, i.e., for bearing resistance, the nominal pile resistance shall be factored at the strength limit state using the resistance factors in Table 10.5.5.2.3-1 associated with the method used to compute the nominal bearing resistance of the pile. The factored nominal bearing resistance of piles, R_R, may be taken as:

$$R_R = \varphi R_n \tag{10.7.3.8.6a-1}$$

or:

$$R_R = \varphi R_n = \varphi_{stat} R_p + \varphi_{stat} R_s \tag{10.7.3.8.6a-2}$$

in which:

$$R_p = q_p A_p \tag{10.7.3.8.6a-3}$$

$$R_s = q_s A_s \tag{10.7.3.8.6a-4}$$

where:

φ_{stat} = resistance factor for the bearing resistance of a single pile specified in Article 10.5.5.2.3
R_p = pile tip resistance (kips)
R_s = pile side resistance (kips)
q_p = unit tip resistance of pile (ksf)
q_s = unit side resistance of pile (ksf)
A_s = surface area of pile side (ft^2)
A_p = area of pile tip (ft^2)

C10.7.3.8.6a

While the most common use of static analysis methods is solely for estimating pile quantities, a static analysis may be used to establish pile installation criteria if dynamic methods are determined to be unsuitable for field verification of nominal bearing resistance. This is applicable on projects where pile quantities are relatively small, pile loads are relatively low, and/or where the setup time is long so that re-strike testing would require an impractical wait-period by the Contractor on the site, e.g., soft silts or clays where a large amount of setup is anticipated.

For use of static analysis methods for contract pile quantity estimation, see Article 10.7.3.3.

Table 10.5.5.2.3-1—Resistance Factors for Driven Piles

	Condition/Resistance Determination Method	Resistance Factor
Nominal Bearing Resistance of Single Pile—Dynamic Analysis and Static Load Test Methods, φ_{dyn}	Driving criteria established by successful static load test of at least one pile per site condition and dynamic testing* of at least two piles per site condition, but no less than 2% of the production piles	0.80
	Driving criteria established by successful static load test of at least one pile per site condition without dynamic testing	0.75
	Driving criteria established by dynamic testing* conducted on 100% of production piles	0.75
	Driving criteria established by dynamic testing,* quality control by dynamic testing* of at least two piles per site condition, but no less than 2% of the production piles	0.65
	Wave equation analysis, without pile dynamic measurements or load test but with field confirmation of hammer performance	0.50
	FHWA-modified Gates dynamic pile formula (End of Drive condition only)	0.40
	Engineering News (as defined in Article 10.7.3.8.5) dynamic pile formula (End of Drive condition only)	0.10
Nominal Bearing Resistance of Single Pile—Static Analysis Methods, φ_{stat}	Side Resistance and End Bearing: Clay and Mixed Soils	
	α-method (Tomlinson, 1987; Skempton, 1951)	0.35
	β-method (Esrig & Kirby, 1979; Skempton, 1951)	0.25
	λ-method (Vijayvergiya & Focht, 1972; Skempton, 1951)	0.40
	Side Resistance and End Bearing: Sand	
	Nordlund/Thurman Method (Hannigan et al., 2005)	0.45
	SPT-method (Meyerhof)	0.30
	CPT-method (Schmertmann)	0.50
	End bearing in rock (Canadian Geotech. Society, 1985)	0.45
Block Failure, φ_{b1}	Clay	0.60
Uplift Resistance of Single Piles, φ_{up}	Nordlund Method	0.35
	α-method	0.25
	β-method	0.20
	λ-method	0.30
	SPT-method	0.25
	CPT-method	0.40
	Static load test	0.60
	Dynamic test with signal matching	0.50
Group Uplift Resistance, φ_{ug}	All soils	0.50
Lateral Geotechnical Resistance of Single Pile or Pile Group	All soils and rock	1.0
Structural Limit State	Steel piles	See the provisions of Article 6.5.4.2
	Concrete piles	See the provisions of Article 5.5.4.2
	Timber piles	See the provisions of Article 8.5.2.2 and 8.5.2.3
Pile Drivability Analysis, φ_{da}	Steel piles	See the provisions of Article 6.5.4.2
	Concrete piles	See the provisions of Article 5.5.4.2
	Timber piles	See the provisions of Article 8.5.2.2
	In all three Articles identified above, use φ identified as "resistance during pile driving"	

* Dynamic testing requires signal matching, and best estimates of nominal resistance are made from a restrike. Dynamic tests are calibrated to the static load test, when available.

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename:
Project Name: 21699 Northfield   Project Date: 07/20/2018
Project Client: MaineDOT
Computed By: B Slaven
Project Manager:

PILE INFORMATION

Pile Type: H Pile - HP14X89
Top of Pile: 7.00 ft
Perimeter Analysis: Box
Tip Analysis: Pile Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	7.00 ft
	- Driving/Restrike	7.00 ft
	- Ultimate:	7.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesionless	8.00 ft	0.00%	125.00 pcf	32.0/0.0	Nordlund
2	Cohesionless	3.00 ft	0.00%	142.00 pcf	32.0/0.0	Nordlund
3	Cohesionless	24.00 ft	0.00%	145.00 pcf	43.0/0.0	Nordlund
4	Cohesionless	10.00 ft	0.00%	145.00 pcf	43.0/43.0	Nordlund

Limit Critical Depth to 20D (20*1.2 = 24')
Replace Nordlund method with SPT-Meyerhof from 24' bgs to pile tip
Ref: Navfac 7.2-193 and DAS pg 569

ULTIMATE - SKIN FRICTION

Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesionless	0.00 psf	0.00	N/A	0.00 Kips
6.99 ft	Cohesionless	0.00 psf	0.00	N/A	0.00 Kips
6.99 ft	Cohesionless	0.00 psf	0.00	N/A	0.00 Kips
7.00 ft	Cohesionless	875.00 psf	25.98	N/A	0.00 Kips
7.01 ft	Cohesionless	875.31 psf	25.98	N/A	0.02 Kips
7.99 ft	Cohesionless	905.99 psf	25.98	N/A	1.88 Kips
8.01 ft	Cohesionless	938.00 psf	25.98	N/A	1.92 Kips
10.99 ft	Cohesionless	1056.60 psf	25.98	N/A	8.53 Kips
11.01 ft	Cohesionless	1176.81 psf	34.92	N/A	8.61 Kips
20.01 ft	Cohesionless	1548.51 psf	34.92	N/A	75.67 Kips
29.01 ft	Cohesionless	1920.21 psf	34.92	N/A	174.92 Kips
34.99 ft	Cohesionless	2167.19 psf	34.92	N/A	258.66 Kips
35.01 ft	Cohesionless	3159.21 psf	34.92	N/A	258.97 Kips
44.01 ft	Cohesionless	3530.91 psf	34.92	N/A	411.86 Kips
44.99 ft	Cohesionless	3571.39 psf	34.92	N/A	430.45 Kips

ULTIMATE - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 ft	Cohesionless	0.00 psf	4.80	2.41 Kips	0.00 Kips
6.99 ft	Cohesionless	0.00 psf	4.80	2.41 Kips	0.00 Kips
6.99 ft	Cohesionless	0.00 psf	4.80	2.41 Kips	0.00 Kips
7.00 ft	Cohesionless	875.00 psf	4.80	2.41 Kips	0.28 Kips
7.01 ft	Cohesionless	875.63 psf	4.80	2.41 Kips	0.28 Kips
7.99 ft	Cohesionless	936.97 psf	4.80	2.41 Kips	0.30 Kips
8.01 ft	Cohesionless	938.40 psf	4.80	2.41 Kips	0.30 Kips
10.99 ft	Cohesionless	1175.60 psf	4.80	2.41 Kips	0.37 Kips
11.01 ft	Cohesionless	1177.23 psf	4.80	2.41 Kips	0.37 Kips
20.01 ft	Cohesionless	1920.63 psf	4.80	2.41 Kips	0.61 Kips
29.01 ft	Cohesionless	2664.03 psf	4.80	2.41 Kips	0.75 Kips
34.99 ft	Cohesionless	3157.97 psf	4.80	2.41 Kips	0.75 Kips
35.01 ft	Cohesionless	3159.63 psf	307.00	122.82 Kips	122.82 Kips
44.01 ft	Cohesionless	3903.03 psf	307.00	122.82 Kips	122.82 Kips
44.99 ft	Cohesionless	3983.97 psf	307.00	122.82 Kips	122.82 Kips

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
6.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
6.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
7.00 ft	0.00 Kips	0.28 Kips	0.28 Kips
7.01 ft	0.02 Kips	0.28 Kips	0.30 Kips
7.99 ft	1.88 Kips	0.30 Kips	2.18 Kips
8.01 ft	1.92 Kips	0.30 Kips	2.22 Kips
10.99 ft	8.53 Kips	0.37 Kips	8.91 Kips
11.01 ft	8.61 Kips	0.37 Kips	8.99 Kips
20.01 ft	75.67 Kips	0.61 Kips	76.28 Kips
29.01 ft	174.92 Kips	0.75 Kips	175.68 Kips
34.99 ft	258.66 Kips	0.75 Kips	259.41 Kips
35.01 ft	258.97 Kips	122.82 Kips	381.78 Kips
44.01 ft	~~411.86 Kips~~ 77 kips	122.82 Kips	534.67 Kips
44.99 ft	~~430.45 Kips~~	122.82 Kips	553.27 Kips

Limit Limit Critical Depth to 20D (20*1.2 = 24') + Pile head 11' bgs = 35 ft bgs.

Replace Nordlund method with SPT- Meyerhof from 35 ft bgs to pile tip

Meyerhof from 35-45: Qs

qs = N160/50 = 81/50 = 1.62 ksf LRFD 10.7.3.8.6g-3

Rs = qs*As = qs*Box Area Perimeter*Penetration LRFD 10.7.3.8.6a-4

Rs = 1.62 ksf + 57.11"/12" * 10' = 77 Kips

RsTot = 259 Kips + 77 Kips + = 336 Kips

Rtotal = RsTot + Rend_bearing = 336 Kips + 123 Kips = 459 Kips

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename:
Project Name: 21699 Northfield   Project Date: 07/20/2018
Project Client: MaineDOT
Computed By: B Slaven
Project Manager:

PILE INFORMATION

Pile Type: H Pile - HP14X117
Top of Pile: 7.00 ft
Perimeter Analysis: Box
Tip Analysis: Pile Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	7.00 ft
	- Driving/Restrike	7.00 ft
	- Ultimate:	7.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesionless	8.00 ft	0.00%	125.00 pcf	32.0/0.0	Nordlund
2	Cohesionless	3.00 ft	0.00%	142.00 pcf	32.0/0.0	Nordlund
3	Cohesionless	24.00 ft	0.00%	145.00 pcf	43.0/0.0	Nordlund
4	Cohesionless	10.00 ft	0.00%	145.00 pcf	43.0/43.0	Nordlund

Limit Critical Depth to 20D (20*1.4 = 24')
Replace Nordlund method with SPT-Meyerhof from 24' bgs to pile tip

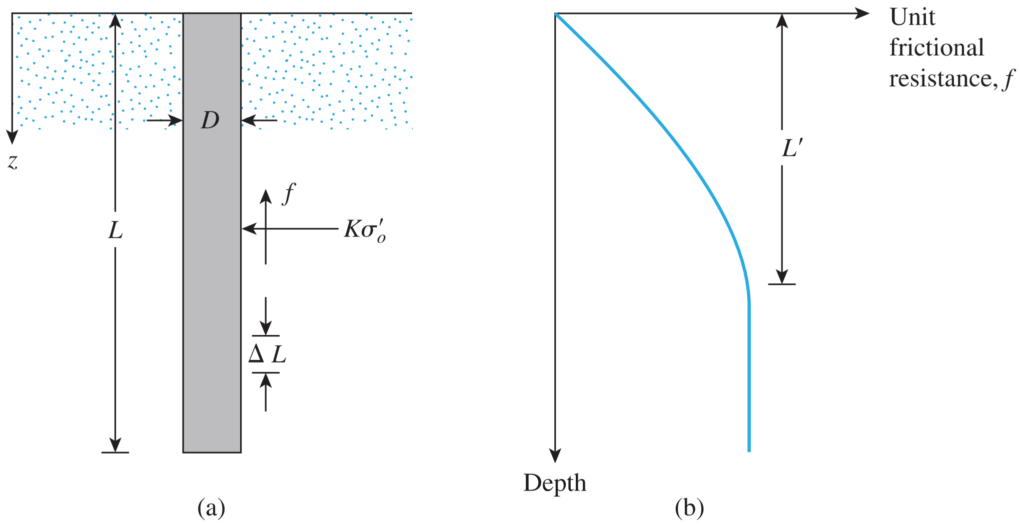


Figure 11.16 Unit frictional resistance for piles in sand

less linearly to a depth of L' and remains constant thereafter. The magnitude of the critical depth L' may be 15 to 20 pile diameters. A conservative estimate would be

Use L'=20D for

very dense glacial till.

$$L' \approx 15D \tag{11.40}$$

3. At similar depths, the unit skin friction in loose sand is higher for a high-displacement pile, compared with a low-displacement pile.
4. At similar depths, bored, or jetted, piles will have a lower unit skin friction compared with driven piles.

Taking into account the preceding factors, we can give the following approximate relationship for f (see Figure 11.16):

For $z = 0$ to L',

$$f = K\sigma'_o \tan\delta' \tag{11.41}$$

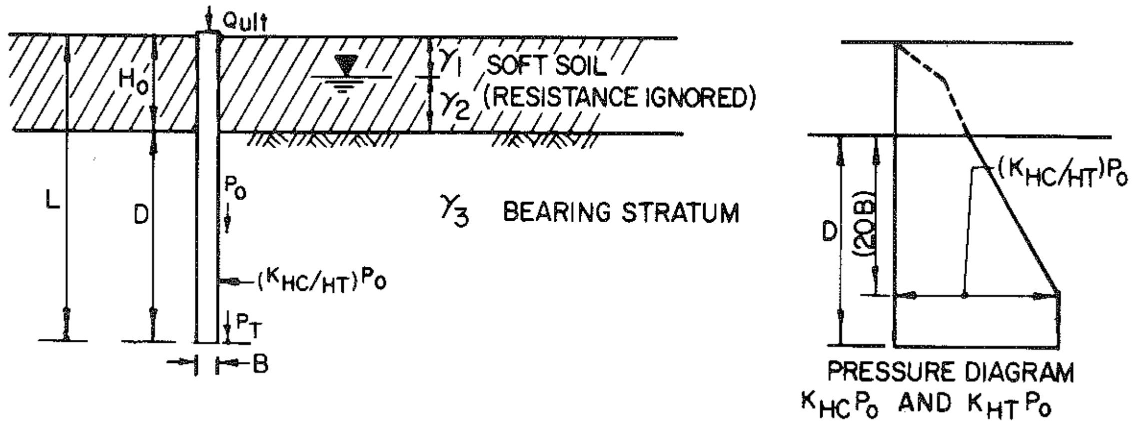
and for $z = L'$ to L,

$$f = f_{z=L'} \tag{11.42}$$

In these equations,

K = effective earth pressure coefficient
σ'_o = effective vertical stress at the depth under consideration
δ' = soil-pile friction angle

In reality, the magnitude of K varies with depth; it is approximately equal to the Rankine passive earth pressure coefficient, K_p, at the top of the pile and may be less than



(A) ULTIMATE LOAD CAPACITY IN COMPRESSION

$$Q_{ult} = P_T N_q A_T + \sum_{H=H_0}^{H=H_0+D} (K_{HC})(P_0)(\tan\delta)(S)$$

WHERE Q_{ult} = ULTIMATE LOAD CAPACITY IN COMPRESSION
P_T = EFFECTIVE VERTICAL STRESS AT PILE TIP (SEE NOTE 1)
N_q = BEARING CAPACITY FACTOR (SEE TABLE, FIGURE 1 CONTINUED)
A_T = AREA OF PILE TIP
K_{HC} = RATIO OF HORIZONTAL TO VERTICAL EFFECTIVE STRESS ON SIDE OF ELEMENT WHEN ELEMENT IS IN COMPRESSION.
P_0 = EFFECTIVE VERTICAL STRESS OVER LENGTH OF EMBEDMENT, D (SEE NOTE 1)
δ = FRICTION ANGLE BETWEEN PILE AND SOIL (SEE TABLE, FIGURE 1 CONTINUED)
S = SURFACE AREA OF PILE PER UNIT LENGTH

FOR CALCULATING Q_{all}, USE F_S OF 2 FOR TEMPORARY LOADS, 3 FOR PERMANENT LOADS. (SEE NOTE 2)

(B) ULTIMATE LOAD CAPACITY IN TENSION

$$T_{ult} = \sum_{H=H_0}^{H=H_0+D} (K_{HT})(P_0)(\tan\delta)(S)(H)$$

WHERE: T_{ult} = ULTIMATE LOAD CAPACITY IN TENSION, PULLOUT
K_{HT} = RATIO OF HORIZONTAL TO VERTICAL EFFECTIVE STRESS ON SIDE OF ELEMENT WHEN ELEMENT IS IN TENSION

FOR CALCULATING T_{all}, USE $F_S = 3$ ON T_{ult} PLUS THE WEIGHT OF THE PILE (W_P), THUS $T_{all} = \frac{T_{ult}}{3} + W_P$ (SEE NOTE 2)

NOTE-1: EXPERIMENTAL AND FIELD EVIDENCE INDICATE THAT BEARING PRESSURE AND SKIN FRICTION INCREASE WITH VERTICAL EFFECTIVE STRESS P_0 UP TO A LIMITING DEPTH OF EMBEDMENT, DEPENDING ON THE RELATIVE DENSITY OF THE GRANULAR SOIL AND POSITION OF THE WATER TABLE. BEYOND THIS LIMITING DEPTH (10B ± TO 40B ±) THERE IS VERY LITTLE INCREASE IN END BEARING, AND INCREASE IN SIDE FRICTION IS DIRECTLY PROPORTIONAL TO THE SURFACE AREA OF THE PILE. THEREFORE, IF D IS GREATER THAN 20 B, LIMIT P_0 AT THE PILE TIP TO THAT VALUE CORRESPONDING TO D = 20B.

NOTE-2: IF BUILDING LOADS AND SUBSURFACE CONDITION ARE WELL DOCUMENTED IN THE OPINION OF THE ENGINEER, A LESSER FACTOR OF SAFETY CAN BE USED BUT NOT LESS THAN 2.0 PROVIDED PILE CAPACITY IS VERIFIED BY LOAD TEST AND SETTLEMENTS ARE ACCEPTABLE.

FIGURE 1
Load Carrying Capacity of Single Pile in Granular Soils

LPile Parameters

Development of soil model for LPile

OBJECTIVE
Estimate soil parameters for lateral pile analyses.

Given:
1) Boring logs and lab data.

Assumptions:
1) Groundwater observations ranged from El. 150.6 to 145.4. Assume the average groundwater table is near the ordinary high water El. of 147.2 ft.
2) MaineDOT Bridge Design Guide (BDG) Soil Type 4 will be used for integral abutment backfill.
3) Piles at Abut No. 1 shall be driven to, or within, bedrock. Piles at Abut No. 2 shall be driven to approx. El. 110.

Soil Model

1) The design soil layers are delineated as depicted on the attached annotated boring logs, which indicates the top and bottom elevations of the soil layers based on differing engineering properties.

Soil Layer No. 1 (Granular Borrow for Underwater Backfill) El. 155.0 - 147.0

Internal Angle of Friction $\boxed{\phi_1 := 32\,deg}$

Soil Total Unit Weight $\boxed{\gamma_{1moist} := 125pcf}$

MaineDOT BDG Table 3-3

Representative constant giving the variation of soil modulus with depth, k:
Medium dense sand above water table for static loading = 90 pci

Technical Manual LPile 2016 p. 96

Height of layer 1:

$h_1 := 155.0ft - 147.0ft \quad h_1 = 8\cdot ft$

Assume the total (moist) unit weight of Soil Type 4 considers placement at the material's optimum moisture content. Based on density test results, optimum moisture content of granular borrow occurs between 6 and 10 percent. Assume 8 percent.

$w_{1opt} := .08 \qquad \gamma_w := 62.4pcf$

Dry unit weight

$$\gamma_{1dry} := \frac{\gamma_{1moist}}{1 + w_{1opt}} \qquad \gamma_{1dry} = 115.7\cdot pcf$$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships

Saturated Unit Weight

Natural water content at saturated state:
Loose uniform sand = 30%
Dense angular silty sand = 15%
Average Loose and Dense for Medium Dense:
Medium Dense angular silty sand: 23%

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 - Natural Moisture Content in a saturated state

$w_{1sat} := .23$

$$\gamma_{1saturated} := \gamma_{1dry}\cdot(1 + w_{1sat})$$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships

$\gamma_{1saturated} = 142\cdot pcf$

Soil Layer No. 2 (Submerged, Granular Borrow) El. 147.0 - 144.0

Internal Angle of Friction $\boxed{\phi_2 := 32deg}$

MaineDOT BDG Table 3-3

Soil Total Unit Weight $\gamma_{2moist} := 125pcf$

Height of layer 2:

$h_2 := 147.0ft - 144.0ft \quad h_2 = 3 \cdot ft$

Assume the total (moist) unit weight of Soil Type 4 considers placement at the material's optimum moisture content. Based on density test results from nearby projects, optimum moisture content of granular borrow occurs between 6 and 10 percent. Assume 8 percent.

$w_{2opt} := .08$

Dry unit weight

$\gamma_{2dry} := \frac{\gamma_{2moist}}{1 + w_{2opt}} \qquad \gamma_{2dry} = 115.7 \cdot pcf$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships

Saturated Unit Weight

Natural water content at saturated state:
Loose uniform sand = 30%
Dense angular silty sand = 15%
Average Loose and Dense for Medium Dense:
Medium Dense angular silty sand: 23%

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 - Natural Moisture Content in a saturated state

$w_{2sat} := .23$

$\gamma_{2saturated} := \gamma_{2dry} \cdot (1 + w_{2sat})$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships

$\gamma_{2saturated} = 142 \cdot pcf$

Effective Unit Weight

weight of water = γ_w =62.4 pcf

$\gamma_w := 62.4pcf$

$\gamma'_2 := \gamma_{2saturated} - \gamma_w$

$\gamma'_2 = 80 \cdot pcf$

Holtz and Kovacs, Intro to Geotechnical Eng. p. 18 Eq (2-11). Multiply density by gravity to arrive at weights

Representative constant for calculation of P-Y curve,
initial horizontal soil modulus, k_h:
Medium dense sand below water table with static loading = 60 pci

Technical Manual LPile 2016 p. 96

Combine Soil Layer 1 and Soil Layer 2 El. 155.0 - 144.0

LPile's layering correction to p-y curve algorithm returns an error when more than one soil layer is defined above the pile head. The recommended solution is to combine the soil layers above the pile head into a single layer with an effective unit weight that results in an equivalent effective vertical stress at the pile head and extend the combined layer to a minimum elevation of just below the top of the pile.

Effective Vertical stress at bottom of Soil Layer 1

$\sigma_{v1} := \gamma_{1moist} \cdot h_1 \qquad \sigma_{v1} = 1000 \cdot psf$

Holtz and Kovacs, Intro to Geotechnical Eng. p. 213 Eq (7-14b).

Effective Vertical stress at bottom of Soil Layer 2

$\sigma_{v2} := \gamma'_2 \cdot h_2 \qquad \sigma_{v2} = 240 \cdot psf$

Holtz and Kovacs, Intro to Geotechnical Eng. p. 213 Eq (7-15).

Total effective stress at top of pile

$\sigma_t := \sigma_{v1} + \sigma_{v2} \qquad \sigma_t = 1240 \cdot psf$

Holtz and Kovacs, Intro to Geotechnical Eng. p. 213 Eq (7-14c).

Equivalent weight

$$\gamma_{combined} := \frac{\sigma_t}{h_1 + h_2} \qquad \boxed{\gamma_{combined} = 112.7 \cdot pcf}$$

Soil Layer No. 3 (Submerged, Glacial Till) El. 144.0 - 110.0

Internal Angle of Friction

Design $(N_1)_{60}$ = 80 bpf $\qquad \boxed{\phi_3 := 43}$

Kulhawy and Mayne, Manual on Estimating Soil Properties p. 4-15: N vs. Phi

Height of layer 3:

$h_3 := 144.0ft - 110.0ft \qquad h_3 = 34 \cdot ft$

Dry Unit Weight

Dry, Dense Glacial Till = 134 pcf

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 - dry unit weight

$\gamma_{3dry} := 134pcf$

Saturated Unit Weight

Natural water content at saturated state:
Glacial Till: 8%

approximate water content of submerged samples BB-NBS-102;6D, 7D

$w_{3sat} := .08$

$\gamma_{3saturated} := \gamma_{3dry} \cdot (1 + w_{3sat})$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships

$\gamma_{3saturated} = 145 \cdot pcf$

Soil Effective Unit Weight

weight of water = γ_w =62.4 pcf

$\gamma_w := 62.4pcf$

$\gamma'_3 := \gamma_{3saturated} - \gamma_w$

Holtz and Kovacs, Intro to Geotechnical Eng. p. 15 Eq (2-11).

$\boxed{\gamma'_3 = 82.32 \cdot pcf}$

Representative constant giving the variation of soil modulus with depth, k:
Dense sand below water table for static loading = 125 pci

Technical Manual LPile 2016 p. 96

FG = 155.0

-11

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Bog Stream Bridge #3719 carries Route 192 over Bog Stream **Location:** Northfield, Maine	**Boring No.:** BB-NBS-102 **WIN:** 21699.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	154.6	**Auger ID/OD:**	5" Solid Stem
Operator:	J. Lee	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	Deidrich D50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	05/01/2017 - 05/02/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+89.3, 6.2 ft Rt.	**Casing ID/OD:**	HW 4"/4.5" NW 3"/3.5"	**Water Level*:**	9.2' (on 05/02/2017)
Hammer Efficiency Factor:	0.873	**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	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	154.1		6" of Pavement 0.5	
	1D	24/8	1.00 - 3.00	11/13/9/7	22	32				Brown, damp, dense, SAND, some gravel, trace silt, (Fill).	
	2D	24/10	3.00 - 5.00	7/6/6/10	12	17				Similar to above, except medium dense.	
		Layer 1 Moist, Granular Borrow 155-147									
5	3D	24/14	5.00 - 7.00	6/4/2/2	6	9	8			Brown, moist, loose, SAND, little gravel, trace silt, (Fill).	
							15				
	4D	24/16	7.00 - 9.00	2/2/2/2	4	6	15			Brown, wet, loose, SAND, some gravel, trace silt, (Fill).	G#304379 A-1-b, SW-SM WC=7.2%
		Layer 2 Submerged, Granular Borrow 147-144									
							18				
10	5D	24/9	10.00 - 12.00	15/11/16/16	27	39	31	144.6		10.0 Grey, wet, dense, Gravelly SAND, trace silt, (Glacial Till).	G#304380 A-1-a, SW-SM WC=10.3%
							68				
							54				
		Layer 3 Submerged, Glacial Till 144-110									
							46				
15	6D	24/13	15.00 - 17.00	9/17/20/26	37	54	24			Grey, wet, very dense, SAND, some gravel, some silt.	G#304381 A-1-b, SM WC=8.4%
							70	137.6		17.0	
	7D	24/12	17.00 - 19.00	33/30/27/29	57	83	100			Grey, wet, very dense, SAND, some silt, little clay, little gravel, (Glacial Till).	G#304382 A-4, SC-SM WC=8.0%
							157			±6" Cobble	
							211				
20	8D	24/22	20.00 - 22.00	20/47/45/49	92	134	111			Similar to above.	
							158				
							257				
							368				
25	R1	60/14	24.50 - 29.50				241	130.1		24.5	

Bot. of Abut = 144.0

0

Remarks:

Auto-hammer SN #362.
Casing driven using 140# auto-hammer with 30" drop.
Water level recorded prior to drilling on 05/02/2017 with casing to 45 ft bgs.

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: Bog Stream Bridge #3719 carries Route 192 over Bog Stream
Location: Northfield, Maine

Boring No.: BB-NBS-102
WIN: 21699.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	154.6	**Auger ID/OD:**	5" Solid Stem
Operator:	J. Lee	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	Deidrich D50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	05/01/2017 - 05/02/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+89.3, 6.2 ft Rt.	**Casing ID/OD:**	HW 4"/4.5" NW 3"/3.5"	**Water Level*:**	9.2' (on 05/02/2017)
Hammer Efficiency Factor:	0.873	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
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N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
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T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							NQ2			±14" Boulder	
							SPUN	125.1		29.5	
30	9D	24/24	30.00 - 32.00	20/25/32/44	57	83				Placed NW spin casing. Grey, wet, very dense, Silty SAND, little gravel, (Glacial Till).	
35	10D	24/24	35.00 - 37.00	21/26/40/43	66	96				Similar to above.	
								114.6		40.0	
40	R2	24/12	40.00 - 42.00				NQ2			18" Boulder.	
								112.6		42.0	
							SPUN				
										±6.3' Boulder	
45											
50											

Approx. anticipated pile tip El. 110

34

Remarks:

Auto-hammer SN #362.
Casing driven using 140# auto-hammer with 30" drop.
Water level recorded prior to drilling on 05/02/2017 with casing to 45 ft bgs.

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.

3.4 Construction Loads

The construction live load to be used for constructibility checks is 50 psf applied over the entire deck area. Consideration should be given to slab placement sequence for calculation of maximum force effects.

3.5 Railroad Loads

Railroad bridges should be designed according to the latest American Railroad Engineering and Maintenance-of-Way Association specifications (AREMA, 2002), with the Cooper live loading as determined by the railroad company.

3.6 Earth Loads

3.6.1 General

Earth pressures considered for wall and substructure design must use the appropriate soil weight shown in Table 3-3.

Table 3-3 Material Classification

Soil Type	Soil Description	Internal Angle of Friction of Soil, ϕ	Soil Total Unit Weight (pcf)	Coeff. of Friction, tan δ, Concrete to Soil	Interface Friction, Angle, Concrete to Soil δ
1	Very loose to loose silty sand and gravel Very loose to loose sand Very loose to medium density sandy silt Stiff to very stiff clay or clayey silt	29° *	100	0.35	19°
2	Medium density silty sand and gravel Medium density to dense sand Dense to very dense sandy silt	33°	120	0.40	22°
3	Dense to very dense silty sand and gravel Very dense sand	36°	130	0.45	24°
4	Granular underwater backfill Granular borrow	32°	125	0.45	24°
5	Gravel Borrow	36°	135	0.50	27°

* The value given for the internal angle of friction (ϕ) for stiff to very stiff silty clay or clayey silt should be used with caution due to the large possible variation with different moisture contents.

Table 3-6 Representative Values of *k* for Fine Sand Below the Water Table for Static and Cyclic Loading

Recommended *k*	Relative Density		
	Loose	Medium	Dense
MN/m^3 (pci)	5.4 (20.0)	16.3 (60.0)	34 (125.0)

Table 3-7 Representative Values of *k* for Fine Sand Above Water Table for Static and Cyclic Loading

Recommended *k*	Relative Density		
	Loose	Medium	Dense
MN/m^3 (pci)	6.8 (25.0)	24.4 (90.0)	61.0 (225.0)

If the sand profile is coarse or well-graded sand, the user may consider using a higher value of *k* that those suggested in the tables above. While experimental data for *k* in well-graded sands is poorly documented, use of values 10 to 50 percent higher may be appropriate in dense and very dense well-graded sands that do not contain any compressible minerals such as mica.

7. Fit the parabola between point *k* and point *m* as follows:

a. Compute the slope of the *p-y* curve between point *m* and point *u* using

$$m = \frac{p_u - p_m}{y_u - y_m} \quad \text{(3-62)}$$

b. Compute the power of the parabolic section using

$$n = \frac{p_m}{m y_m} \quad \text{(3-63)}$$

c. Compute the coefficient $\bar{C}$ using

$$\bar{C} = \frac{p_m}{y_m^{1/n}} \quad \text{(3-64)}$$

8. Compute the *y* value defining point *k* using

$$y_k = \left(\frac{\bar{C}}{kx} \right)^{\frac{n}{n-1}} \quad \text{(3-65)}$$

Compute the *p* value defining point *k* using

Das, Principles of Geotechnical Engineering 7th ed.

3.4 Various Unit-Weight Relationships

In Sections 3.2 and 3.3, we derived the fundamental relationships for the moist unit weight, dry unit weight, and saturated unit weight of soil. Several other forms of relationships that can be obtained for γ, γ_d, and γ_{sat} are given in Table 3.1. Some typical values of void ratio, moisture content in a saturated condition, and dry unit weight for soils in a natural state are given in Table 3.2.

Table 3.1 Various Forms of Relationships for γ, γ_d, and γ_{sat}

Moist unit weight (γ)		Dry unit weight (γ_d)		Saturated unit weight (γ_{sat})	
Given	**Relationship**	**Given**	**Relationship**	**Given**	**Relationship**
w, G_s, e	$\dfrac{(1+w)G_s\gamma_w}{1+e}$	γ, w	$\dfrac{\gamma}{1+w}$	G_s, e	$\dfrac{(G_s+e)\gamma_w}{1+e}$
S, G_s, e	$\dfrac{(G_s+Se)\gamma_w}{1+e}$	G_s, e	$\dfrac{G_s\gamma_w}{1+e}$	G_s, n	$[(1-n)G_s+n]\gamma_w$
w, G_s, S	$\dfrac{(1+w)G_s\gamma_w}{1+\dfrac{wG_s}{S}}$	G_s, n	$G_s\gamma_w(1-n)$	G_s, w_{sat}	$\left(\dfrac{1+w_{sat}}{1+w_{sat}G_s}\right)G_s\gamma_w$
w, G_s, n	$G_s\gamma_w(1-n)(1+w)$	G_s, w, S	$\dfrac{G_s\gamma_w}{1+\left(\dfrac{wG_s}{S}\right)}$	e, w_{sat}	$\left(\dfrac{e}{w_{sat}}\right)\left(\dfrac{1+w_{sat}}{1+e}\right)\gamma_w$
S, G_s, n	$G_s\gamma_w(1-n)+nS\gamma_w$	e, w, S	$\dfrac{eS\gamma_w}{(1+e)w}$	n, w_{sat}	$n\left(\dfrac{1+w_{sat}}{w_{sat}}\right)\gamma_w$
		γ_{sat}, e	$\gamma_{sat}-\dfrac{e\gamma_w}{1+e}$	γ_d, e	$\gamma_d+\left(\dfrac{e}{1+e}\right)\gamma_w$
		γ_{sat}, n	$\gamma_{sat}-n\gamma_w$	γ_d, n	$\gamma_d+n\gamma_w$
		γ_{sat}, G_s	$\dfrac{(\gamma_{sat}-\gamma_w)G_s}{(G_s-1)}$	γ_d, S	$\left(1-\dfrac{1}{G_s}\right)\gamma_d+\gamma_w$
				γ_d, w_{sat}	$\gamma_d(1+w_{sat})$

Table 3.2 Void Ratio, Moisture Content, and Dry Unit Weight for Some Typical Soils in a Natural State

Type of soil	Void ratio, e	Natural moisture content in a saturated state (%)	Dry unit weight, γ_d lb/ft^3	Dry unit weight, γ_d kN/m^3
Loose uniform sand	0.8	30	92	14.5
Dense uniform sand	0.45	16	115	18
Loose angular-grained silty sand	0.65	25	102	16
Dense angular-grained silty sand	0.4	15	121	19
Stiff clay	0.6	21	108	17
Soft clay	0.9–1.4	30–50	73–93	11.5–14.5
Loess	0.9	25	86	13.5
Soft organic clay	2.5–3.2	90–120	38–51	6–8
Glacial till	0.3	10	134	21

7.5 INTERGRANULAR OR EFFECTIVE STRESS

The concept of intergranular or *effective stress* was introduced in Sec. 6.2. By definition,

$$\sigma = \sigma' + u \tag{7-13}$$

where σ = total normal stress,
σ' = intergranular or effective normal stress, and
u = pore water or neutral pressure.

Both the total stress and pore water pressure may readily be estimated or calculated with knowledge of the densities and thicknesses of the soil layers and location of the ground water table. The effective stress cannot be measured; it can only be calculated!

The total vertical stress is called the *body stress* because it is generated by the mass (acted upon by gravity) in the body. To calculate the total vertical stress σ_v at a point in a soil mass, you simply sum up the densities of all the material (soil solids + water) above that point multiplied by the gravitational constant g, or

$$\sigma_v = \int_0^h \rho g dz \tag{7-14a}$$

If ρg is a constant throughout the depth, then

$$\sigma_v = \rho g h \tag{7-14b}$$

Typically, we divide the soil mass into n layers and evaluate the total stress incrementally for each layer or

$$\sigma_v = \sum_{i=1}^{n} \rho_i g z_i \tag{7-14c}$$

As an example, if a soil could have zero voids, then the total stress exerted on a particular plane would be the depth to the given point times the density of the material or, in this case, ρ_s times the gravitational constant g. If the soil were dry, then you would use ρ_d instead of ρ_s.

The neutral stress or pore water pressure is similarly calculated for static water conditions. It is simply the depth below the ground water table to the point in question, z_w, times the product of the density of water ρ_w and g, or

$$u = \rho_w g z_w \tag{7-15}$$

It is called the *neutral stress* because it has no shear component. Recall from fluid mechanics that by definition a liquid cannot support static shear stress. It has only normal stresses which act equally in all

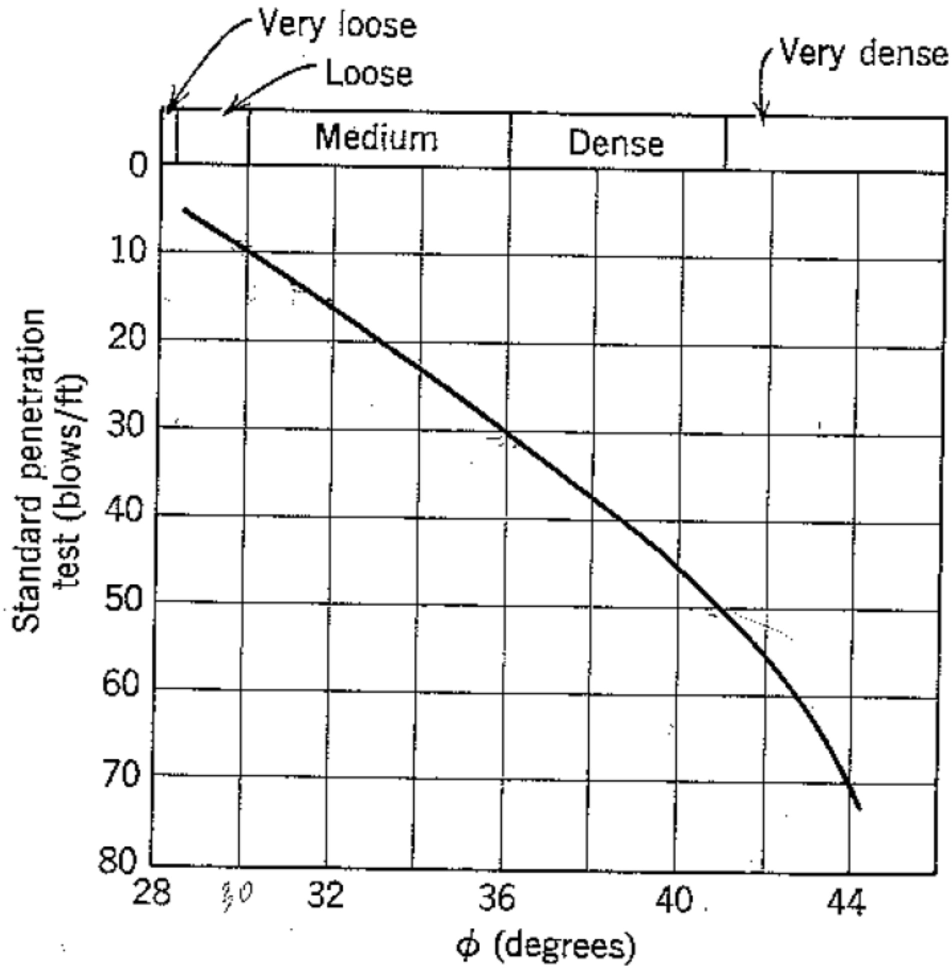

Fig. 11.14 Correlation between friction angle and penetration resistance (From Peck, Hanson, and Thornburn, 1953).

Earth Pressure

Earth Pressure:

Backfill engineering strength parameters

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

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

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

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

Integral Abutment - Passive Earth Pressure - Coulomb Theory

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

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

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

Use Coulomb for cases where interface friction is considered; typically gravity shaped structures, and integral abutments where the ratio of wall height to wall movement is .005 or greater. Coulomb should also be used when the fill slope is greater than horizontal.

For precast 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)

$\delta' := 19.5 \cdot \text{deg}$

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

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

$\boxed{K_{p\_coulomb} = 6.73}$

Integral Abutment and Wingwall - Passive Earth Pressure - Rankine Theory

Use Rankine only if the ratio of wall height to wall movement is significantly less than .005 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 $\alpha := 0 \cdot \text{deg}$

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

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

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

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

U-shaped Wingwalls - Active Earth Pressure

Active pressure acting parallel to the travelway is assumed to be resisted by the superstructure and can be neglected for butterfly walls. Design of U-shaped wingwalls shall consider active pressure acting perpendicular to the travelway.

Active Earth Pressure - Rankine Theory

- For cantilever walls with horizontal backslope

$$K_a := \tan\left[45 \cdot deg - \frac{(\phi')}{2}\right]^2$$

$$\boxed{K_a = 0.31}$$

- For a sloped backfill (2H:1V)

β = Angle of fill slope to the horizontal

$$\beta := 26.6 \cdot deg$$

$$K_{aslope} := \cos(\beta)\frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi')^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi')^2}}$$

$$\boxed{K_{aslope} = 0.46}$$

  - Pa is oriented at an angle of β to the vertical plane - See MaineDOT Bridge Design Guide Figure 3-3 attached.

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

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

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

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

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

where

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

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

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

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

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

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

Table 7.9 (*Continued*)

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

7.12 Coulomb's Passive Earth Pressure

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

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

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

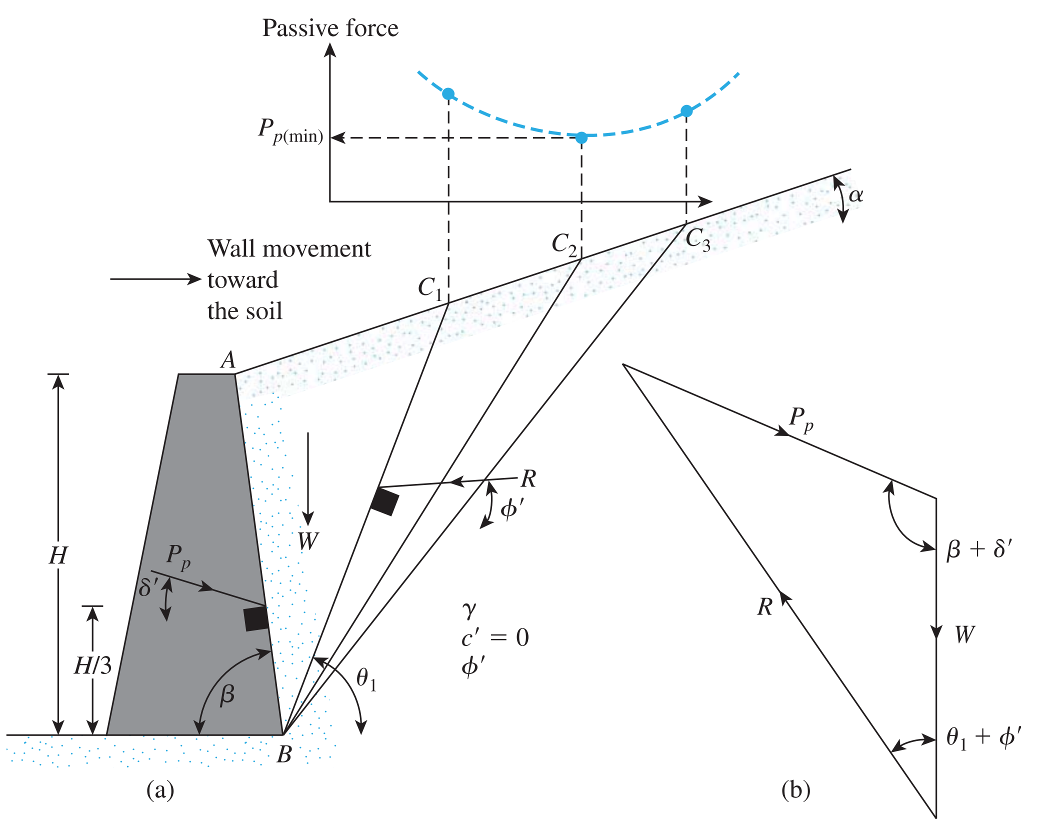

Figure 7.25 Coulomb's passive pressure

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

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

Again, at $z = 3$ m,

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

Hence,

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

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

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

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

■

7.11 Rankine Passive Earth Pressure: Vertical Backface and Inclined Backfill

Granular Soil

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

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

and the passive force is

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

where

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

Figure 3-2 Calculating β with Broken Backfill Surface

Rankine theory, as described in Section 3.6.5.2, may also be used for the design of yielding walls, for a simplified analysis (at the Structural Designer's option). The use of Rankine theory will result in a slightly more conservative design.

3.6.5.2 Rankine Theory

Rankine theory should be used for long-heeled cantilever walls. Refer to AASHTO LRFD Figure C3.11.5.3-1 (a) for the definition of a long heeled cantilever wall. For simplicity (at the Structural Designer's option), Rankine theory may also be used to compute lateral earth pressures on any yielding wall listed in 3.6.5.1 Coulomb Theory, although its use will result in a slightly more conservative design.

For these cases, interface friction between the wall backface and the backfill is not considered. Rankine earth pressure is applied to a plane extending vertically from the heel of the wall base, as shown in Figure 3-3.

For a horizontal backfill surface where $\beta = 0°$, the value of the coefficient of active earth pressure (Rankine), K_a, may be taken as:

$$\boxed{K_a = \tan^2\left(45° - \frac{\phi}{2}\right)}$$

where:

$\phi =$ angle of internal soil friction (degrees), taken from Table 3-3.

$\beta =$ angle of backfill to the horizontal (degrees), as shown in Figure 3-3.

For a sloped backfill surface where $\beta > 0°$, the coefficient of active earth pressure (Rankine), K_a, may be taken as:

$$\boxed{K_a = \cos\ \beta \cdot \frac{\cos\ \beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\ \beta + \sqrt{\cos^2\beta - \cos^2\phi}}}$$

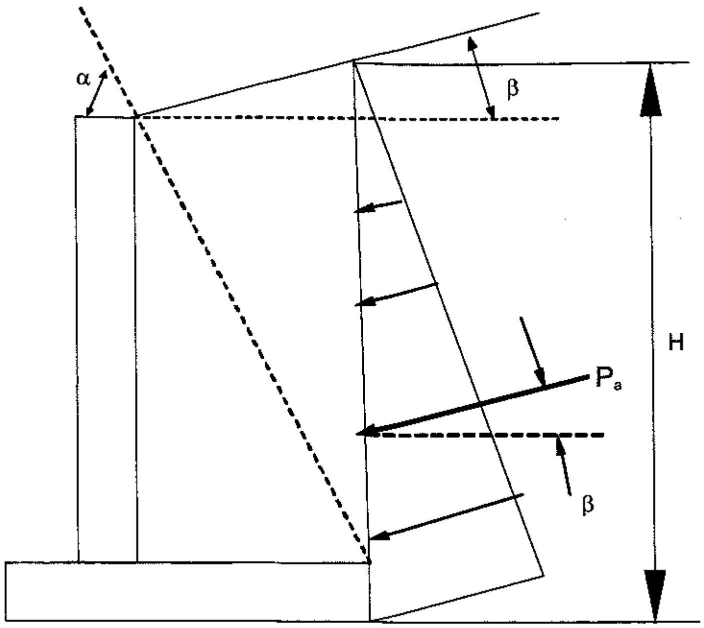

Figure 3-3 Rankine Theory

The resultant earth pressure force, P_a, is oriented at an angle, β, as shown if Figure 3-3. The resultant acts at a distance, H/3, from the base of the footing.

For situations with a broken backfill surface, the active earth pressure coefficient, Ka, may be determined using a β value adjusted per AASHTO LRFD Figures 3.11.5.8 -1 through 3, or substituted with β*, as shown in Figure 3-2.

3.6.6 Coulomb Passive Lateral Earth Pressure Coefficient

Values of the coefficient of passive lateral earth pressure, K_p, may be taken from Figures 3.11.5.4-1 and 2 in AASHTO LRFD or using Coulomb theory, as shown below:

$$K_p = \frac{\sin(\alpha - \phi)^2}{\sin\alpha^2 \cdot \sin(\alpha + \delta) \cdot \left(1 - \sqrt{\dfrac{\sin(\phi + \delta)\cdot\sin(\phi + \beta)}{\sin(\alpha + \delta)\cdot\sin(\alpha + \beta)}}\right)^2}$$

where:

α = angle (degrees) of back of wall to the horizontal as shown in Figure 3-1.

ϕ = angle of internal soil friction (degrees), taken from Table 3-3.

Das Principles of Geotechnical Engineering 7th ed., 2010

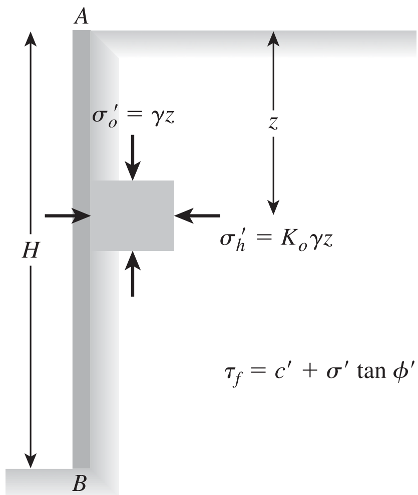

Figure 13.3
Earth pressure at rest

which shows a wall *AB* retaining a dry soil with a unit weight of γ. The wall is static. At a depth z,

$$\text{Vertical effective stress} = \sigma'_o = \gamma z$$
$$\text{Horizontal effective stress} = \sigma'_h = K_o \gamma z$$

So,

$$K_o = \frac{\sigma'_h}{\sigma'_o} = \text{at-rest earth pressure coefficient}$$

For coarse-grained soils, the coefficient of earth pressure at rest can be estimated by using the empirical relationship (Jaky, 1944)

$$K_o = 1 - \sin \phi' \tag{13.5}$$

where ϕ' = drained friction angle.

While designing a wall that may be subjected to lateral earth pressure at rest, one must take care in evaluating the value of K_o. Sherif, Fang, and Sherif (1984), on the basis of their laboratory tests, showed that Jaky's equation for K_o [Eq. (13.5)] gives good results when the backfill is loose sand. However, for a dense, compacted sand backfill, Eq. (13.5) may grossly underestimate the lateral earth pressure at rest. This underestimation results because of the process of compaction of backfill. For this reason, they recommended the design relationship

$$K_o = (1 - \sin \phi) + \left[\frac{\gamma_d}{\gamma_{d(\min)}} - 1\right]5.5 \tag{13.6}$$

where γ_d = actual compacted dry unit weight of the sand behind the wall
$\gamma_{d(\min)}$ = dry unit weight of the sand in the loosest state (Chapter 3)

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: **Northfield, Maine**
DFI = 1300 degree-days.
Case 1 - coarse grained granular fill soils W=20% (assumed).

For DFI = 1300

$d := 63.0 \cdot in$

Depth of Frost Penetration $d = 5.3 \cdot ft$

Method 2 - ModBerg Software

Examine foundations placed on coarse grained fill soils

Gardiner lies along the same Maine Design Freezing Index contour - use Gardiner data from Modberg's freezing index database.

```
                    ------------------------
               --- ModBerg Results ---
                    ----------------------
Project Location: Brunswick, Maine
Air Design Freezing Index           =  1276 F-days
N-Factor                            =  0.80
Surface Design Freezing Index       =  1021 F-days
Mean Annual Temperature            =  45.9 deg F
Design Length of Freezing Season =  118 days
---------------------------------------------------------
Layer
#:Type          t     w%     d      Cf  Cu   Kf   Ku     L
---------------------------------------------------------
1-Coarse     70.3 20.0  125.0  34  46   3.8  1.9  3,600
---------------------------------------------------------
t  = Layer thickness, in inches.
w% = Moisture content, in percentage of dry density.
d  = Dry density, in lbs/cubic ft.
Cf = Heat Capacity of frozen phase, in BTU/(cubic ft degree F).
Cu = Heat Capacity of thawed phase, in BTU/(cubic ft degree F).
Kf = Thermal conductivity in frozen phase, in BTU/(ft hr degree).
Ku = Thermal conductivity in thawed phase, in BTU/(ft hr degree).
L  = Latent heat of fusion, in BTU / cubic ft.
***********************************************************
  Total Depth of Frost Penetration = 5.86 ft = 70.3 in.
***********************************************************
```

Recommendation: 5.3 feet for design of foundations constructed on coarse grained soils

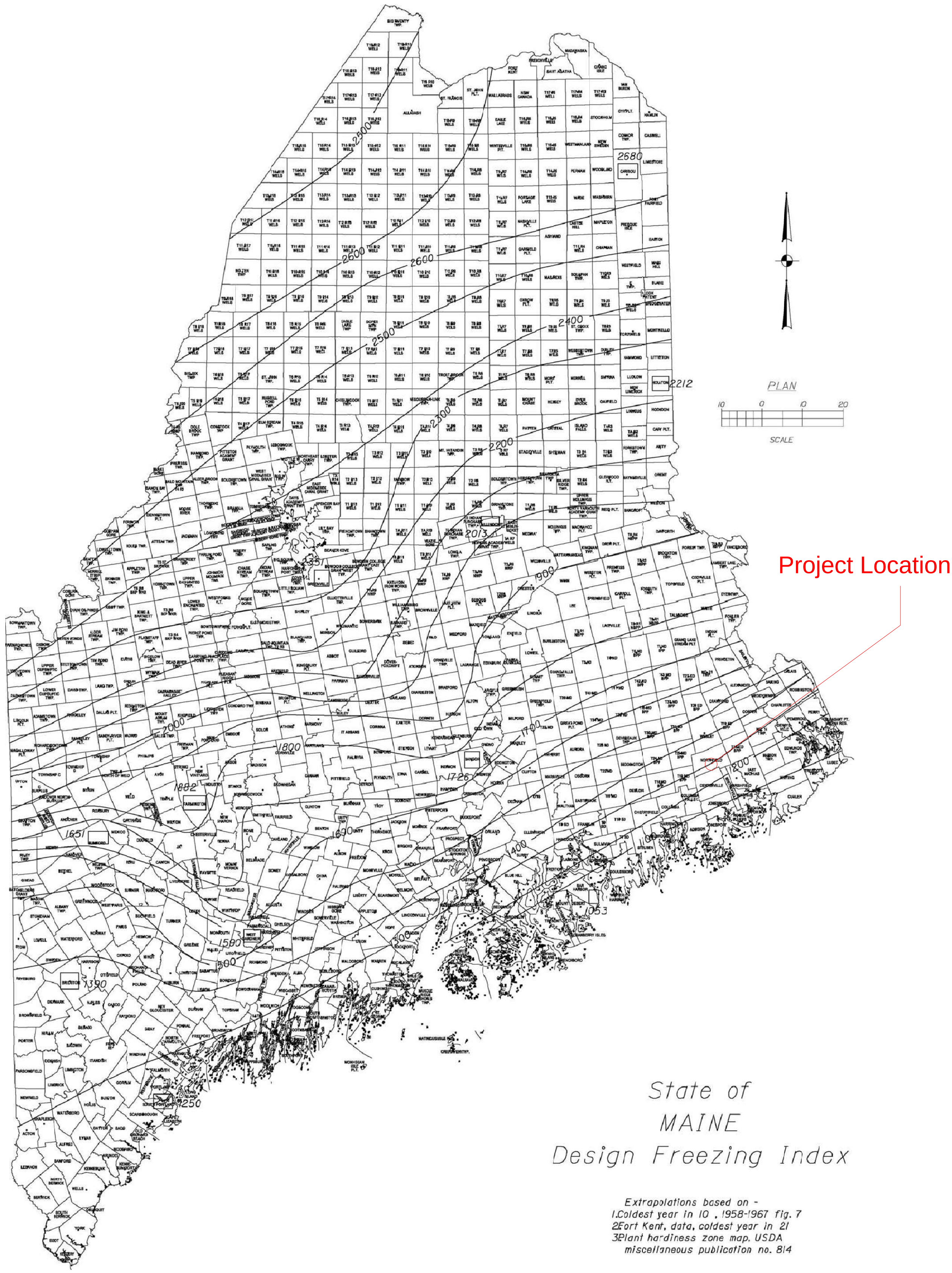

Figure 5-1 Maine Design Freezing Index Map

5.2 General

MaineDOT Bridge Design Guide

5.2.1 Frost

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

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

Table 5-1 Depth of Frost Penetration

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

Seismic Design

BB-NBS-101				
Depth	**N_{60}**		**di**	**di/N**
6	50		6	0.12
11	56		5	0.09
16	31		5	0.16
21	48		5	0.10
26	74		5	0.07
31	59		5	0.08
36	71		5	0.07
41	100		5	0.05
42.5	100	**Bedrock**	1.5	0.02
100	100		57.5	0.58
SUM			**100**	**1.34**

di/di/N **74.77**

BB-NBS-102				
Depth	**N_{60}**		**di**	**di/N**
2	32		2	0.06
4	17		2	0.12
6	9		2	0.22
8	6		2	0.33
11	39		3	0.08
16	54		5	0.09
18	83		2	0.02
21	100		3	0.03
31	83		10	0.12
36	96		5	0.05
79	100	**Bedrock**	43	0.43
100	100		21	0.21
SUM			**100**	**1.77**

di/di/N **56.44**

BB-NBS-103				
Depth	**102 N_{60}**		**di**	**di/N**
6	5		6	1.20
11	26		5	0.19
16	59		5	0.08
21	63		5	0.08
26	104		5	0.05
31	57		5	0.09
36	60		5	0.08
41	76		5	0.07
47	100	**Bedrock**	6	0.06
100	100		53	0.53
SUM			**100**	**2.43**

di/di/N **41.13**

Note:
N60 limited to 100 bpf.

SUM	Nav.	57.44

$N_{av.} > 50$

Conclusion: Site Class C

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

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 44.847220
Longitude = -067.594580
Site Class B
Data are based on a 0.05 deg grid spacing.

	Period (sec)	Sa (g)	
	0.0	0.078	PGA - Site Class B
	0.2	0.155	Ss - Site Class B
	1.0	0.041	S1 - Site Class B

2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
Latitude = 44.847220
Longitude = -067.594580
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class C - Fpga = 1.20, Fa = 1.20, Fv = 1.70
Data are based on a 0.05 deg grid spacing.

	Period (sec)	Sa (g)	
	0.0	0.093	As - Site Class C
	0.2	0.186	SDs - Site Class C
	1.0	0.070	SD1 - Site Class