Maine Department of Transportation
Bridge Program
Geotechnical Section
Augusta, Maine

Geotechnical Design Report

For the Replacement of:

Ice House Bridge
State Route 201A over Gilman Brook
Anson, Maine

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1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and provide geotechnical design recommendations for the replacement of Ice House Bridge which carries State Routes 201A and 8 over Gilman Brook in Anson, 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 older portions of the existing structure were constructed in 1923 and consist of a mix of stacked and grouted stone abutments with a buried concrete slab superstructure. The structure was widened in 1946 with a similar buried concrete slab on mass concrete abutments, with a floor slab across the entire stream

According to the 2018 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 scaling of the concrete surfaces and bulging, cracked, or missing stones in the masonry portions. The FHWA Sufficiency Rating of the existing bridge is 31.1, which classifies the existing structure as Structurally Deficient.

The proposed replacement structure consists of a 40-foot single span precast concrete NEXT-D 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 Ice House Bridge will be designed to match the existing horizontal alignment and closely match the existing vertical profile. A single lane temporary detour upstream of the project will maintain one lane of alternating traffic during construction of the replacement structure.

2.0 GEOLOGIC SETTING

The existing structure carries State Route 201A over Gilman Brook as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of the Anson Quadrangle, Maine, Open-file No. 86-28 (1986), indicates the surficial soils in the vicinity of the bridge project consist of glacial marine deposits, which accumulated on the ocean floor during the late-glacial marine submergence of lowland areas in southern and eastern Maine. These soils are generally comprised of silt, clay, sand, and minor amounts of gravel. The most common component is the clayey silt known as the Presumpscot Formation, but sand is very abundant in some areas. The unit also may contain small areas of till, sand, and gravel that are not completely covered by the marine sediment. The MGS Surficial Geology Map of the Anson Quadrangle shows nearby contacts to alluvial deposits related to stream terraces of the Kennebec River.

The MGS Maine Geologic Map Series GM-7 (1979) contains the Geologic Map of Anson Quadrangle, which cites the bedrock at the project site as the Lower Metaconglomerate Member of the Sangerville Formation. The Lower Metaconglomerate Member includes graywacke, metasandstone, metasiltstone, and metapelite of the main formation.

3.0 SUBSURFACE INVESTIGATION

Two test borings were drilled to explore subsurface conditions at the site. Boring BB-AGB-101 was drilled south of the existing bridge and boring BB-AGB-102 was drilled north of the existing bridge in July 2017. The boring locations are shown on Sheet 2 – Boring Location Plan. 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 4 – Boring Logs.

Borings were performed by using a combination of solid stem auger, cased wash boring, and rock coring techniques. Soil samples were typically obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The drill rig used in the subsurface investigation was equipped with an automatic hammer to drive the split spoon. The hammer was calibrated per ASTM D 4633 "Standard Test Method for Energy Measurement for Dynamic Penetrometers" in March 2017. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.677. The hammer efficiency factor (0.677) 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 logged the subsurface conditions encountered. A MaineDOT 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 two 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 and on Sheet 4 -Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings generally consisted of granular fill, relic topsoil, riverbed gravel and cobbles and glacial till. The fill unit and subsurface soil deposits are underlain by metamorphic bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs. A generalized subsurface profile is shown on

Sheet 3 – Interpretive Subsurface Profile. The following paragraphs discuss the subsurface conditions encountered:

5.1 Fill

A layer of fill material was encountered in both borings. The thickness of the fill unit was approximately 8.6 to 14.2 feet at the boring locations. The fill encountered generally consisted of:

- Black to brown, gravel, some sand, little to trace silt;
- Red-brown, silty sand, trace gravel;
- Grey-brown, mottled, sand, some silt;
- Brown, sandy gravel, little silt; and
- Reddish brown, sand, some silt, trace gravel.

Corrected SPT N-values in the fill unit ranged from 3 to 60 blows per foot (bpf) indicating the fill is very loose to very dense in consistency. Three grain size analyses performed on samples recovered from within the fill unit resulted in A-2-4 and A-4 material classifications according to the AASHTO Soil Classification System and a SM classification according to the Unified Soil Classification System (USCS). The natural water content of the samples tested ranged from approximately 16 to 28 percent.

5.2 Relic Topsoil and Old Fill

Encountered in the borings was a layer of relic topsoil and old fill materials. The thickness of the unit was approximately 1.0 to 4.7 feet at the boring locations. The soils encountered generally consisted of:

- Brown, sandy silt, trace gravel, with peat and wood;
- Red-tan, sandy silt, trace wood.

Corrected SPT N-values in the unit were 3 to 36 bpf indicating the consistency of layer ranges from soft to hard.

5.3 Riverbed Gravel and Cobbles

Encountered in the borings was a deposit of stream alluvium. The thickness of the deposit was approximately 1.2 to 4.0 feet at the boring locations. The alluvial deposit generally consisted or riverbed gravels and cobbles. Based on drilling behavior, the layer is very dense.

One grain size analysis performed on a sample recovered from the riverbed gravel deposit resulted in a material classification of A-1-a according to the AASHTO Soil Classification System and GW-GM according to the USCS. The natural water content of the sample tested was approximately 10 percent.

5.4 Glacial Till

Glacial till was encountered beneath the fill materials in the borings. The thickness ranged from approximately 19.7 to 31.0 feet. The glacial till generally consisted of:

- Dark grey, silty gravel, some sand;
- Dark grey, silt, trace sand, trace gravel, trace clay;
- Dark grey to grey sand, some silt, some to trace gravel, little to trace clay;
- Dark grey, gravelly silt, some sand;
- Dark grey to grey, gravel, some sand, some to little silt, trace gravel; and
- Grey, gravelly sand, some to little silt.

Several SPT attempts failed to advance the split spoon the required 18 inches without delivering excessive blows. Corrected SPT N-values from successful SPT tests within the glacial till deposit ranged from 29 to greater than 100 bpf indicating the glacial till is medium dense to very dense or hard in consistency. Five grain size analyses performed on samples recovered from within the glacial till deposit resulted in A-1-b, A-2-4, and A-4 material classifications according to the AASHTO Soil Classification System and GM, SM, SC-SM, and CL classifications according to the USCS. The natural water contents of the samples tested ranged from approximately 6 to 20 percent.

5.5 Bedrock

Bedrock was encountered and cored in both borings. Table 1 summarizes the approximate initial bedrock core depths, elevations, and RQD's.

Boring	Station	Offset (feet)	Approximate Depth to Initial Bedrock Core (feet)	Approximate Initial Core Elevation (feet)	RQD (%)
BB-AGB-101	104+89.4	7.8 Lt	40.0	216.7	58
BB-AGB-102	105+35.6	10.8 Rt	46.3	209.8	8

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

The bedrock recovered from the borings is identified generally as green-grey, aphanitic to medium grained, meta argillaceous sandstone, with quartz veins, hard, typically fresh, breaks are typically at high angles, undulating, rough, fresh to discolored, moderately close with mud infilling. The RQD of the bedrock cores ranged from 8 to 80 percent correlating to a rock mass quality of very poor to good. Drilling behavior inferred that approximately 4 to 10 inches of weathered bedrock overlies the more intact bedrock surface. Detailed bedrock descriptions and the RQD of each core run are provided on the boring logs in Appendix A – Boring Logs and on Sheet 4 – Boring Logs.

5.6 Groundwater

Groundwater was measured at 8.8 feet bgs in boring BB-AGB-101 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

A precast concrete box culvert and an integral abutment bridge with precast concrete superstructure were considered during preliminary design. Construction of the box culvert alternative is more challenging at this location because of the difficulty of dewatering the excavation and the existing buried utilities within the excavation limits. An integral abutment bridge avoids many of the construction difficulties while providing improved hydraulic performance. Therefore, the integral abutment bridge with 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 Ice House 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 12x53, 12x74, 14x73, 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. Abutments No. 1 and 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	Boring Offset	Approximate Pile Tip Elevation (feet)	Estimated Pile Lengths[1] (feet)
Abutment No. 1	247.0	BB-AGB-101	Left	216	33
Abutment No. 2	247.0	BB-AGB-102	Right	209	40

[1] Includes an additional 2 feet of pile length for embedment in the abutment

Table 2 – Estimated Pile Lengths for Abutments

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 or variations in the bedrock surface.

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

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 five 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 Table 3 is for the lower braced pile segment, using a resistance factor, $\phi_c = 0.50$ for severe driving conditions. The factored structural resistances for the approximated upper unbraced segments use an axial resistance factor $\phi_c = 0.70$ for combined axial and flexure are not provided in Table 3 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 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 piles subject to severe driving conditions are provided in Table 3.

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 the H-pile shapes at the strength limit states for are provided in Table 3. Supporting calculations are provided in Appendix C – Calculations.

Pile Section	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 12x53	387[4]	387[4]	291	291
HP 12x74	545	545	367	367
HP 14x73	535[4]	535[4]	365	365
HP 14 x 89	652	652	421	421
HP 14 x 117	860	860	538 (587)[5]	538 (587)[5]

Table 3– Factored Axial Compressive Resistances for 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

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[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, 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.

[4] Does not consider resistance factors of slender elements. 12x53 and 14x73 H-pile sections may require additional reductions for slenderness. HP 12x53 and 14x73 sections do not comply with LRFD slenderness requirements and generally should be avoided for simplified pile design methods, (ref: Integral Abutment Bridge Design Guidelines, VTrans Structures Section, 2008).

[5] Drivability resistance based on a Delmag D19-42. Drivability resistance with a Delmag D36-32 shown in parentheses.

severe driving conditions applied. However, the estimated factored axial pile resistances from the drivability analyses for the 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 pile design are the drivability resistances provided in the rightmost column "Governing Axial Pile Resistance (kips)" in Table 3 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.

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 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 $\phi = 1.0$ with the exception of uplift of piles, for which the resistance factor, φ_{up}, shall be 0.80 or less per LRFD Article 10.5.5.3.2.

The nominal axial geotechnical pile resistance of 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 five H-pile sections for the extreme and service limit states are provided in Table 4. Supporting documentation is provided in Appendix C – Calculations.

Pile Section	Factored Axial Pile Resistance Extreme and Service Limit State			
	Structural Resistance[6] $\phi = 1.0$ (kips)	Controlling Geotechnical Resistance $\phi = 1.0$ (kips)	Drivability Resistance $\phi = 1.0$ (kips)	Governing Axial Pile Resistance (kips)
HP 12x53	775[7]	775[7]	447	447
HP 12x74	1090	1090	564	564
HP 14x73	1070[7]	1070[7]	562	562
HP 14 x 89	1305	1305	648	648
HP 14 x 117	1720	1720	827 (903)[8]	827 (903)[8]

Table 4 – Factored Axial Compressive Resistances for 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 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 are the resistances provided in the rightmost column “Governing Axial Pile Resistance (kips)” in Table 4. The maximum applied factored axial pile load for piles at the extreme and service limit states should not exceed the governing factored pile resistance shown in Table 4 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.

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[6] 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, $\phi = 1.0$.

[7] Does not consider resistance factors of slender elements. 12x53 and 14x73 H-pile sections may require additional reductions. HP 12x53 and 14x73 sections do not comply with LRFD slenderness requirements and generally should be avoided for simplified pile design methods, (ref: Integral Abutment Bridge Design Guidelines, VTrans Structures Section, 2008).

[8] 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 5. In general, the models developed should emulate the soil at the site by using the soil layers (referenced in Table 5 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 5, 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	256.7 – 247.0	Above	125	90	32°
Loose, Fine Sand	247.0 – 241.0	Below	68	20	28°
Dense to very dense, Glacial Till	241.0 – 217.0	Below	85	125	36°

Table 5 – 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 a wave equation analysis 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. Minimum 24-hour restrike tests are recommended to verify time-dependent loss of pile resistance does not occur. If a loss in pile resistance does occur, the driving criteria shall be adjusted. Restrikes or additional dynamic tests may be required as part of the pile field quality control program should pile behavior vary radically between adjacent piles, should pile behavior indicate a pile is refusing on a boulder or in a cobble layer above bedrock, should the pile tip be not firmly embedded in bedrock, or if piles "walk" out of position.

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

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

7.2 Integral Abutment 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 6:

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

Table 6 – 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 In-line Wingwalls

In-line cantilevered wingwalls may be used in conjunction with the integral abutments. The wingwalls shall be designed for all relevant strength, service, and extreme limit states and load combinations 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, and collision loads, as well as, creep, temperature, and shrinkage deformations. The design of in-line wingwalls shall account for the additional bending stresses resulting from the wingwall being cantilevered off the abutment. These additional bending stresses may require wingwalls longer than 10 feet to be independently supported.

In-line wingwalls shall be designed for passive earth pressure. Calculation of passive earth pressures may assume a Rankine passive earth pressure coefficient, K_p, of 3.25 assuming small wingwall movements. See Appendix C – Calculations for supporting documentation. A load factor for passive earth pressure is not specified in LRFD; use a maximum load factor (γ_{EH}) of 1.50 to calculate factored passive earth pressures.

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

7.4 Settlement

The approach fills encountered in the test borings include very loose fill materials. The north approach fill soils where found to be underlain by approximately 5 feet of soft silt (relic topsoil). To mitigate potential settlement, we recommend that very loose fill soils and soft silt be excavated to El. 243 and replaced with Granular Borrow – Material for Underwater Backfill. Prior to placing the Granular Borrow, the exposed subgrade should be thoroughly compacted. With these provisions, post-construction settlement of the approaches is anticipated to be minimal. Construction loads could introduce elastic settlements, however these settlements will occur relatively quickly.

Any settlement of the piles 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, Anson has a design freezing index (DFI) of approximately 1800 F-degree days. The anticipated coarse-grained fill material was assigned a water content of 15%. These components correlate to a frost depth of 6.9 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Madison, Maine has an air DFI from the Modberg database of approximately 1847 F-degree days. Madison was selected because it lies along the same isoline as Anson and Anson is not available in the Modberg database. A water content of 15% was used. These components correlate to a frost depth of approximately 7.3 feet.

Based on the MaineDOT BDG methodology it is recommended that foundations bearing on coarse-grained soils be designed with an embedment of approximately 6.9 feet for frost protection. For supporting calculations see Appendix C – Calculations.

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

7.6 Scour and Riprap

Project requirements will include reconstruction of the streambed over the concrete slab in the existing streambed. Therefore, grain size parameters for scour analyses are not provided in this report.

However, 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. The existing concrete slab in the streambed will remain and the armored slopes in front of the abutments will toe-in behind the existing concrete slab. 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. Heavy riprap shall conform to MaineDOT Standard Specification 703.28 – Heavy 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 7:

Parameter	Design Value
Peak Ground Acceleration (PGA)	0.076g
Acceleration Coefficient (A_S)	0.122g
S_{DS} (Period = 0.2 sec)	0.260g
S_{D1} (Period = 1.0 sec)	0.115g
Site Class	D
Seismic Zone	1

Table 7 – 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

The new integral abutments will be constructed behind the existing abutments avoiding placement of fills in the river. 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.

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 Ice House Bridge in Anson, 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.

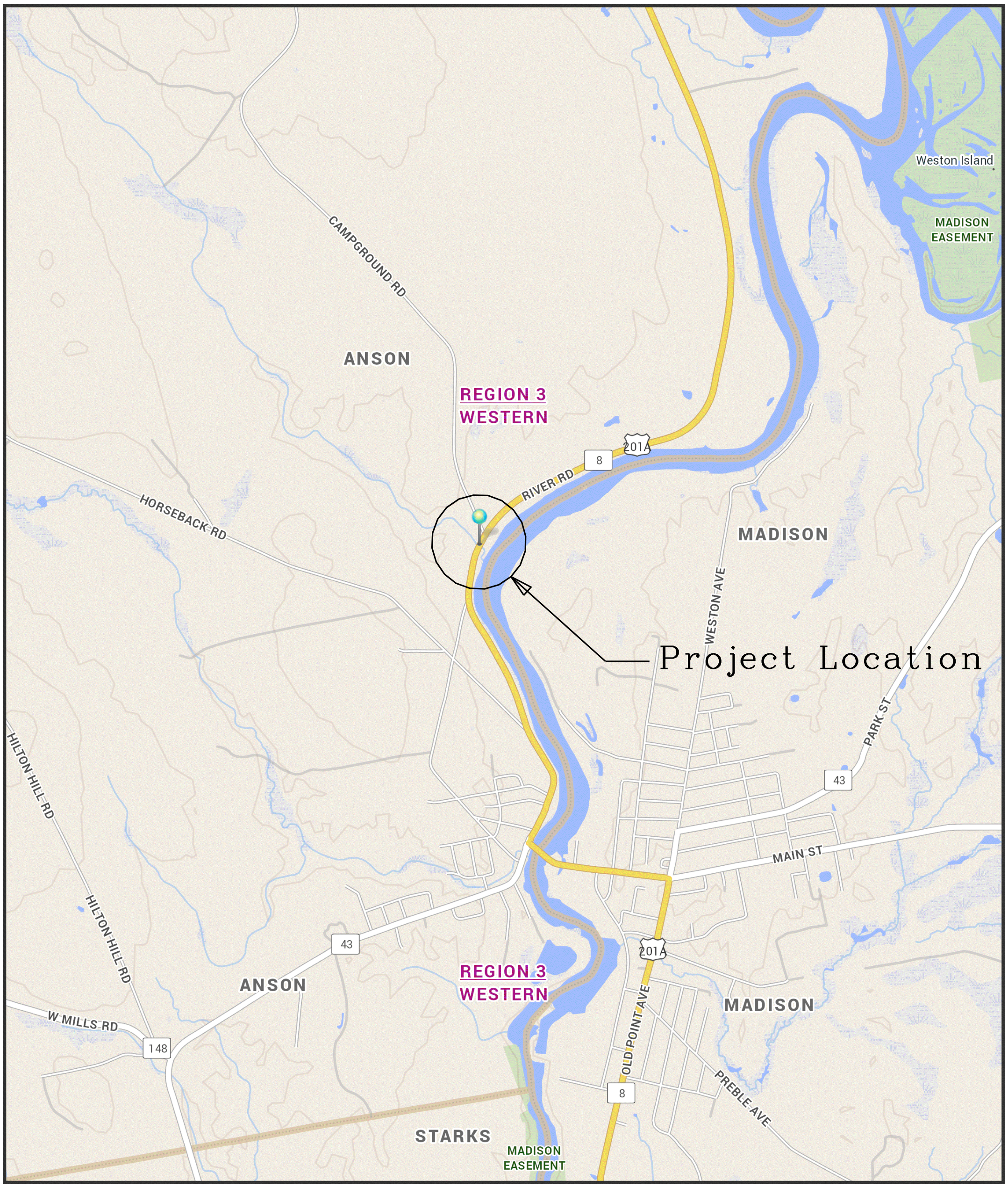
Sheets

Date:2/12/2019

Division: GEOTECH

Username: Terry.White

ANSON, MAINE



0.5 Miles

1 inch = 0.57 miles

Date: 2/12/2019
Time: 12:15:42 PM

SHEET NUMBER

1

OF 4

ICE HOUSE BRIDGE
GILMAN BROOK

ANSON SOMERSET COUNTY

LOCATION MAP

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

021657.00

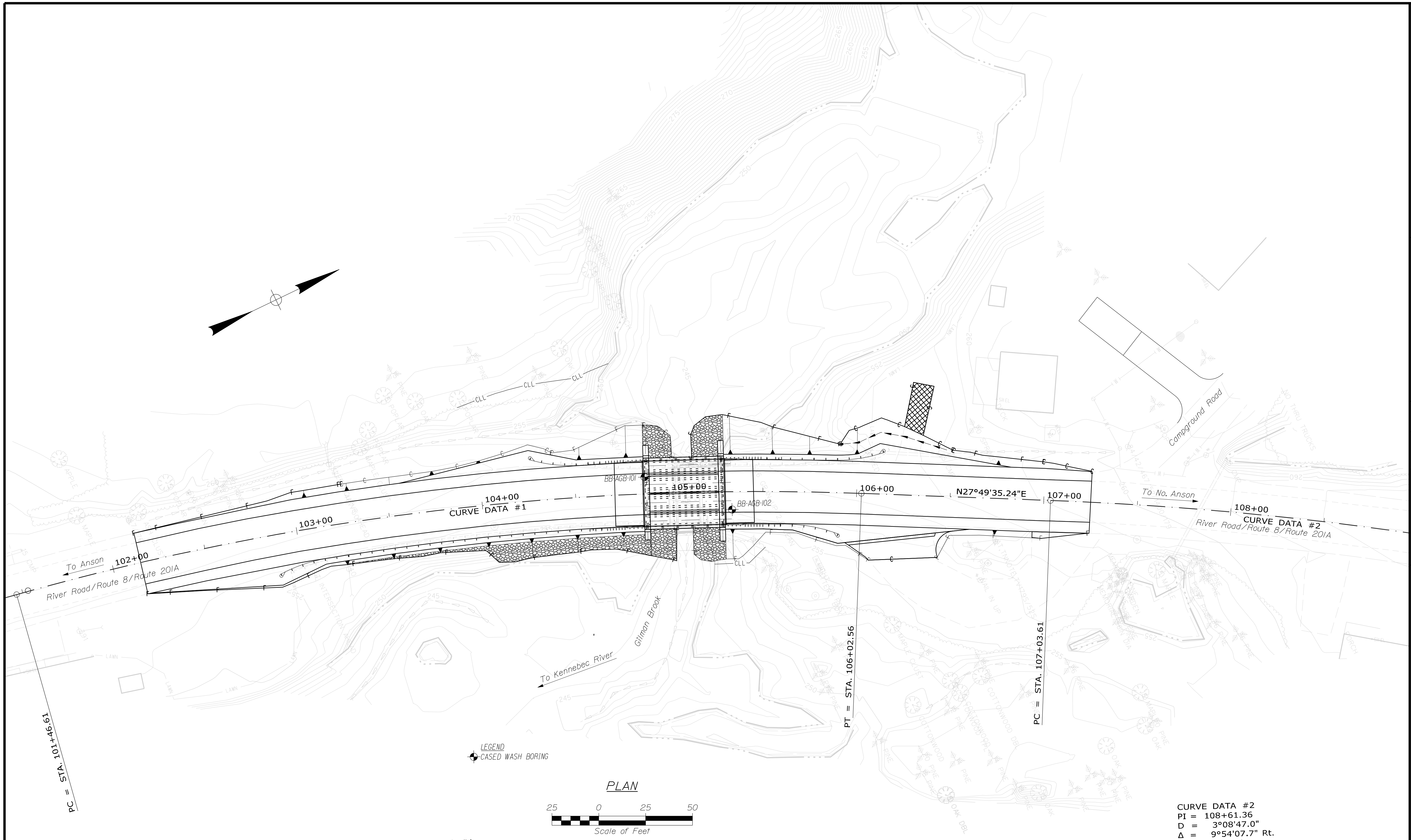
WIN

BRIDGE NO. 3726

21657.00

BRIDGE PLANS

Filename: ...\00\GEOTECH\MSTA\006_BLP1.dgn Division: GEOTECH Username: Laura.Krusinski Date:9/16/2019



LEGEND

CASED WASH BORING

PLAN

CURVE DATA #1
PI = 103+76.47
D = 3°56'16.3"
Δ = 17°57'17.7" Rt.
R = 1455.00'
L = 455.96'
T = 229.86'
E = 18.05'

CURVE DATA #2
PI = 108+61.36
D = 3°08'47.0"
Δ = 9°54'07.7" Rt.
R = 1821.00'
L = 314.71'
T = 157.75'
E = 6.82'

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

021657.00

BRIDGE NO. 3726

WIN
21657.00

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		
CHECKED-REVIEWED		
DESIGN2-DETAILED2	B.SLAVEN	T.WHITE FEB 2019
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

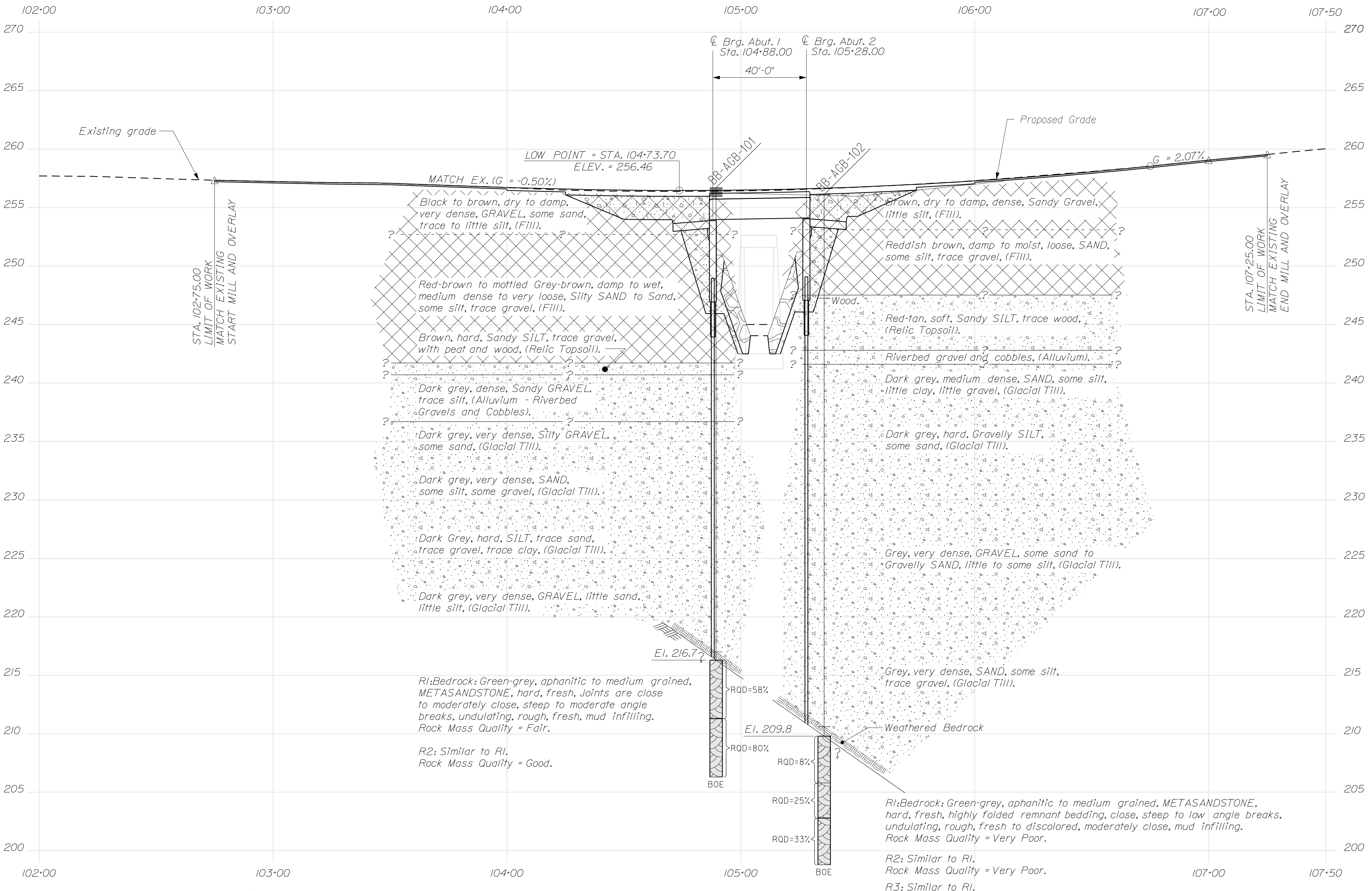
DATE

ICE HOUSE BRIDGE
GILMAN BROOK
ANSON SOMERSET COUNTY

BORING LOCATION PLAN

SHEET NUMBER
2
OF 4

Filename: ...\00\GEOTECH\MSTA\007_ISP1.dgn Division: GEOTECH Username: Laura.Krusinski Date:9/16/2019



Black to brown, dry to damp, very dense, GRAVEL, some sand, trace to little silt, (Fill).

Red-brown to mottled Grey-brown, damp to wet, medium dense to very loose, Silty SAND to Sand, some silt, trace gravel, (Fill).

Brown, hard, Sandy SILT, trace gravel, with peat and wood, (Relic Topsoil).

Dark grey, dense, Sandy GRAVEL, trace silt, (Alluvium - Riverbed Gravels and Cobbles).

Dark grey, very dense, Silty GRAVEL, some sand, (Glacial Till).

Dark grey, very dense, SAND, some silt, some gravel, (Glacial Till).

Dark Grey, hard, SILT, trace sand, trace gravel, trace clay, (Glacial Till).

Dark grey, very dense, GRAVEL, little sand, little silt, (Glacial Till).

R1:Bedrock: Green-grey, aphanitic to medium grained, METASANDSTONE, hard, fresh, Joints are close to moderately close, steep to moderate angle breaks, undulating, rough, fresh, mud infilling. Rock Mass Quality = Fair.

R2: Similar to R1. Rock Mass Quality = Good.

Brown, dry to damp, dense, Sandy Gravel, little silt, (Fill).

Reddish brown, damp to moist, loose, SAND, some silt, trace gravel, (Fill).

Red-tan, soft, Sandy SILT, trace wood, (Relic Topsoil).

Riverbed gravel and cobbles, (Alluvium).

Dark grey, medium dense, SAND, some silt, little clay, little gravel, (Glacial Till).

Dark grey, hard, Gravelly SILT, some sand, (Glacial Till).

Grey, very dense, GRAVEL, some sand to Gravelly SAND, little to some silt, (Glacial Till).

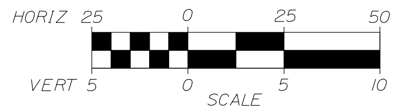
Grey, very dense, SAND, some silt, trace gravel, (Glacial Till).

R1:Bedrock: Green-grey, aphanitic to medium grained, METASANDSTONE, hard, fresh, highly folded remnant bedding, close, steep to low angle breaks, undulating, rough, fresh to discolored, moderately close, mud infilling. Rock Mass Quality = Very Poor.

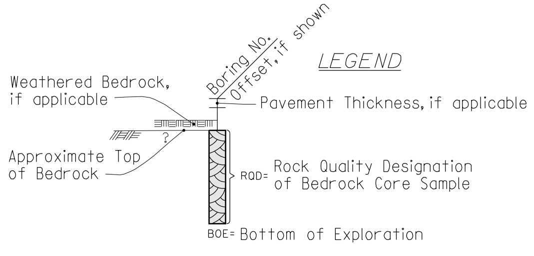
R2: Similar to R1. Rock Mass Quality = Very Poor.

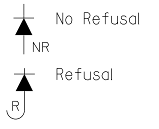
R3: Similar to R1. Rock Mass Quality = Poor.

PROFILE



LEGEND





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

021657.00

WIN
21657.00

BRIDGE NO. 3726

BRIDGE PLANS

PROJ. MANAGER	BY	DATE
DESIGN-DETAILED		
CHECKED-REVIEWED		
DESIGN2-DETAILED2	B.SLAVEN T.WHITE	FEB 2019
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

ICE HOUSE BRIDGE
GILMAN BROOK
ANSON SOMERSET COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
3
OF 4

Filename: ...\MSTA\008_BORING LOGS1.dgn   Division: GEOTECH   Username: Laura.Krusinski   Date:9/16/2019

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Ice House Bridge #3726 carries Routes 8/201A (River Road) over
Location: Anson, Maine

Boring No.: BB-AGB-101
WIN: 21657.00

Driller:	New England Boring Co.	Elevation (ft.)	256.7	Auger ID/OD:	5" Solid Stem
Operator:	Enos/Share	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	Be Schonewald	Rig Type:	Mobile B53 Rubber Track	Hammer Wt./Fall:	140/30"
Date Start/Finish:	7/5/2017-7/6/2017	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	104+89.4, 7.8 ft Lt.	Casing ID/OD:	HW & NW	Water Level*:	8.8 ft bgs, open
Hammer Efficiency Factor: 0.677		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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows
0							SSA
	1D	24/12	1.00 - 3.00	23/29/24/31	53	60	
5	2D	24/20	5.00 - 7.00	6/7/5/3	12	14	
10	3D	24/18	10.00 - 12.00	WOH/2/1/1	3	3	23
							22
							31
							32
							32
15	4D/A	24/11	15.00 - 17.00	3/12/20/13	32	36	22
							85
							78
							300
20	5D	24/12	20.00 - 22.00	14/20/40/39	60	68	RC
25	6D	24/17	25.00 - 27.00	35/30/39/61	69	78	
30	7D	21.6/19	30.00 - 31.80	70/63/71/50(4")	134	151	
35	8D	3.6/3.6	35.00 - 35.30	150(4")	---		
40	R1	60/60	40.00 - 45.00	RQD = 58%			NQ-2
45	R2	60/60	45.40 - 50.40	RQD = 80%			

Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
	9" HMA.	
256.0	0.8	
	Black to brown, dry to damp, very dense, GRAVEL, some fine to coarse sand, trace to little silt, (Fill).	
252.7	4.0	
	Red-brown, damp to moist, medium dense, Silty fine to medium SAND, trace fine gravel, trace coarse sand, (Fill).	G#270357 A-4, SM WC=18.0%
	Grey-brown, mottled, moist to wet, very loose, fine SAND, some silt, (Fill).	G#270358 A-2-4, SM WC=28.1%
241.7	15.0	
	4D (15.0-16.0 ft bgs) Brown, hard, fine to medium Sandy SILT, trace gravel, with peat and wood, (Relic Topsoil).	
240.7	16.0	
	4D/A (16.0-17.0 ft bgs) Dark grey, dense, fine to coarse Sandy GRAVEL, trace silt, (Alluvium - Riverbed Gravel).	G#270359 A-1-a, GW-GM WC=10.2%
	Very boney: difficult to penetrate with roller cone. Riverbed Gravels and Cobbles based on drilling behavior.	
236.7	20.0	
	Dark grey, very dense, Silty GRAVEL, some fine to coarse sand, (Glacial Till).	
	Dark grey, very dense, SAND, some silt, some gravel, (Glacial Till).	G#270360 A-2-4, SM WC=8.7%
	Dark grey, hard, SILT, trace sand, trace gravel, trace clay, (Glacial Till) .	G#270361 A-4, CL WC=20.2%
	Dark grey, very dense, GRAVEL, little fine to coarse sand, little silt, (Glacial Till).	
217.0	39.7	
	Weathered Bedrock. Drove NW Casing to 40.0 ft bgs.	
216.7	40.0	
	Top of Bedrock at Elev. 216.7 ft. R1; Bedrock: Green-grey, aphanitic to medium grained, META ARGILLACEOUS SANDSTONE, occasional quartz veins; hard, fresh, remnant bedding not observed. Joints are close to moderately close, steep to moderate angle breaks, undulating, rough, typically fresh, typically moderately close, with mud infilling. [SANGERVILLE FORMATION] Rock Mass Quality = Fair. R1: Core Times (min:sec) 40.0-41.0 ft (3:00) 41.0-42.0 ft (2:35) 42.0-43.0 ft (2:25) 43.0-44.0 ft (2:25) 44.0-45.0 ft (2:30) 100% recovery Loss pressure from 45.0-45.4 ft bgs, re-ream R1 to 45.4 ft bgs. R2: Bedrock: Similar to R1. Solid piece below 46.8 ft bgs. Rock Mass Quality = Good. R2: Core Times (min:sec) 45.4-46.4 ft (2:10) 46.4-47.4 ft (2:20) 47.4-48.4 ft (2:45) 48.4-49.4 ft (3:15) 49.4-50.4 ft (3:40) 100% Recovery	
206.3	50.4	
	Bottom of Exploration at 50.4 feet below ground surface.	

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-AGB-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Ice House Bridge #3726 carries Routes 8/201A (River Road) over
Location: Anson, Maine

Boring No.: BB-AGB-102
WIN: 21657.00

Driller:	New England Boring Co.	Elevation (ft.)	256.1	Auger ID/OD:	5" Solid Stem
Operator:	Enos/Share	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	Be Schonewald	Rig Type:	Mobile B53 Rubber Track	Hammer Wt./Fall:	140/30"
Date Start/Finish:	7/6/2017-7/7/2017	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	105+35.6, 10.8 ft Rt.	Casing ID/OD:	HW & NW	Water Level*:	None Observed
Hammer Efficiency Factor: 0.677		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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows
0							SSA
	1D	24/17	1.00 - 3.00	23/21/22/20	43	49	
5	2D	24/10	5.00 - 7.00	3/2/4/2	6	7	6
							7
							12
							52
							46
10	3D	24/1	10.00 - 12.00	1/1/2/3	3	3	31
							34
							51
							138
							RC
15	4D	24/7	15.00 - 17.00	22/18/8/10	26	29	107
							64
							119
							74
							94
20	5D	24/9	20.00 - 22.00	6/16/13/47	29	33	65
							122
							205
							RC
25	MD	2.4/0	25.00 - 25.20	50(2")	--		
30	MD	2.4/0	30.00 - 30.20	120(2")	---		
	6D	6/4	31.00 - 31.50	365(6")	---		
35	7D	12/12	35.00 - 36.00	79/127	---		
40	8D	20.4/ 20.4	40.00 - 41.70	50/50/93/50(2")	143	161	RC
	R1	48/47	46.30 - 50.30	RQD = 8%			NQ-2
50	R2	36/35	50.30 - 53.30	RQD = 25%			
	R3	48/47	53.30 - 57.30	RQD = 33%			

Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
	Brown, dry to damp, dense, fine to coarse Sandy GRAVEL, little silt. Asphalt in tip of spoon; Strong Petroleum Odor, (Fill).	
253.1	3.0	
	Reddish brown, damp to moist, loose, fine to medium SAND, some silt, trace gravel, trace coarse sand, (Fill).	G#270362 A-2-4, SM WC=15.6%
247.5	8.6	
	Wood at 8.6 ft bgs based on drilling behavior.	
	Red-tan, soft, fine Sandy SILT, trace wood, trace coarse sand, (Relic Topsoil).	
242.8	13.3	
	Riverbed gravel and cobbles, (Alluvium).	
241.6	14.5	
	Dark grey, medium dense, SAND, some silt, little clay, little gravel, (Glacial Till).	G#270363 A-4, SC-SM WC=9.1%
	Dark grey, hard, Gravelly SILT, some fine to coarse sand, layered, (Glacial Till).	
	Dark grey, Gravel. Silty fine SAND in tip of spoon.	
	Grey, very dense, GRAVEL, some sand, little silt, (Glacial Till).	G#270364 A-1-b, GM WC=6.3%
	Grey, very dense, Gravelly fine to medium SAND, little to some silt; layered, (Glacial Till).	
	Grey, very dense, fine to medium SAND, some silt, trace fine gravel, (Glacial Till).	G#270365 A-2-4, SM WC=14.4%
210.6	45.5	
	Weathered Bedrock.	
209.8	46.3	
	Top of Bedrock at Elev. 209.8 ft. R1: Bedrock: Green-grey, aphanitic to medium grained, META ARGILLACEOUS SANDSTONE, quartz veins, hard, typically fresh, some remnant bedding visible and highly folded. Close, typically high angle with minor low angle breaks; undulating, rough, fresh to discolored, moderately close, with mud infilling, highly broken at 46.3-47.1 ft bgs. Rock Mass Quality = Very Poor. [SANGERVILLE FORMATION] R1: Core Times (min:sec) 46.3-47.0 ft (---) 47.0-48.0 ft (3:05) 48.0-49.0 ft (2:55) 49.0-50.0 ft (3:05) 50.0-50.3 ft (---) 98% Recovery R2: Bedrock: Similar to R1. Highly broken 53.0-53.3 ft bgs. Rock Mass Quality = Very Poor. R2: Core Times (min:sec) 50.3-51.3 ft (2:10) 51.3-52.3 ft (2:20) 52.3-53.3 ft (2:50) 97% Recovery R3: Bedrock: Similar to R1. Highly broken 55.6-56.0 ft bgs. Near vertical fracture terminates at 56.7 ft bgs. Rock Mass Quality = Poor. No core times taken. 98% Recovery	
198.8	57.3	
	Bottom of Exploration at 57.3 feet below ground surface.	

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-AGB-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

021657.00

WIN
21657.00

BRIDGE PLANS

BRIDGE NO. 3726

		BY	DATE
PROJ. MANAGER	--------		
DESIGN-DETAILED	--------	--------	--------
CHECKED-REVIEWED	--------	--------	--------
DESIGN2-DETAILED2	B.SLAVEN	T.WHITE	APR 2019
DESIGN3-DETAILED3	--------	--------	--------
REVISIONS 1	--------		--------
REVISIONS 2	--------		--------
REVISIONS 3	--------		--------
REVISIONS 4	--------		--------
FIELD CHANGES	--------		--------

SIGNATURE

P.E. NUMBER

DATE

ICE HOUSE BRIDGE
GILMAN BROOK
ANSON   SOMERSET COUNTY

BORING LOGS

SHEET NUMBER
4
OF 4

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):

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

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

October 2016

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Ice House Bridge #3726 carries Routes 8/ 201A (River Road) over Gilman Brook
Location: Anson, Maine

Boring No.: BB-AGB-101
WIN: 21657.00

Driller:	New England Boring Co.	**Elevation (ft.)**	256.7	**Auger ID/OD:**	5" Solid Stem
Operator:	Enos/Share	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	Mobile B53 Rubber Track	**Hammer Wt./Fall:**	140/30"
Date Start/Finish:	7/5/2017-7/6/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	104+89.4, 7.8 ft Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	8.8 ft bgs, open
Hammer Efficiency Factor:	0.677	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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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 Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows				
0							SSA	256.0		9" HMA. 0.8	
	1D	24/12	1.00 - 3.00	23/29/24/31	53	60				Black to brown, dry to damp, very dense, GRAVEL, some fine to coarse sand, trace to little silt, (Fill).	
5	2D	24/20	5.00 - 7.00	6/7/5/3	12	14				Red-brown, damp to moist, medium dense, Silty fine to medium SAND, trace fine gravel, trace coarse sand, (Fill).	G#270357 A-4, SM WC=18.0%
10	3D	24/18	10.00 - 12.00	WOH/2/1/1	3	3	23			Grey-brown, mottled, moist to wet, very loose, fine SAND, some silt, (Fill).	G#270358 A-2-4, SM WC=28.1%
							22				
							31				
							32				
							32				
15	4D/A	24/11	15.00 - 17.00	3/12/20/13	32	36	22	241.7 240.7		15.0 4D (15.0-16.0 ft bgs) Brown, hard, fine to medium Sandy SILT, trace gravel, with peat and wood, (Relic Topsoil). 16.0	
										4D/A (16.0-17.0 ft bgs) Dark grey, dense, fine to coarse Sandy GRAVEL, trace silt, (Alluvium - Riverbed Gravel).	G#270359 A-1-a, GW-GM WC=10.2%
							85				
							78			Very boney; difficult to penetrate with roller cone. Riverbed Gravels and Cobbles based on drilling behavior.	
							300				
20	5D	24/12	20.00 - 22.00	14/20/40/39	60	68	RC	236.7		20.0 Dark grey, very dense, Silty GRAVEL, some fine to coarse sand, (Glacial Till).	
25											

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: Ice House Bridge #3726 carries Routes 8/ 201A (River Road) over Gilman Brook
Location: Anson, Maine

Boring No.: BB-AGB-101
WIN: 21657.00

Driller:	New England Boring Co.	**Elevation (ft.)**	256.7	**Auger ID/OD:**	5" Solid Stem
Operator:	Enos/Share	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	Mobile B53 Rubber Track	**Hammer Wt./Fall:**	140/30"
Date Start/Finish:	7/5/2017-7/6/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	104+89.4, 7.8 ft Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	8.8 ft bgs, open
Hammer Efficiency Factor:	0.677	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows				
25	6D	24/17	25.00 - 27.00	35/30/39/61	69	78				Dark grey, very dense, SAND, some silt, some gravel, (Glacial Till).	G#270360 A-2-4, SM WC=8.7%
30	7D	21.6/19	30.00 - 31.80	70/63/71/50(4")	134	151				Dark grey, hard, SILT, trace sand, trace gravel, trace clay, (Glacial Till) .	G#270361 A-4, CL WC=20.2%
35	8D	3.6/3.6	35.00 - 35.30	150(4")	---					Dark grey, very dense, GRAVEL, little fine to coarse sand, little silt, (Glacial Till).	
								217.0 216.7		39.7 Weathered Bedrock. Drove NW Casing to 40.0 ft bgs. 40.0	
40	R1	60/60	40.00 - 45.00	RQD = 58%			NQ-2			Top of Bedrock at Elev. 216.7 ft. R1; Bedrock: Green-grey, aphanitic to medium grained, META ARGILLACEOUS SANDSTONE, occasional quartz veins; hard, fresh, remnant bedding not observed. Joints are close to moderately close, steep to moderate angle breaks, undulating, rough, typically fresh, typically moderately close, with mud infilling. [SANGERVILLE FORMATION] Rock Mass Quality = Fair. R1: Core Times (min:sec) 40.0-41.0 ft (3:00) 41.0-42.0 ft (2:35) 42.0-43.0 ft (2:25) 43.0-44.0 ft (2:25) 44.0-45.0 ft (2:30) 100% recovery Loss pressure from 45.0-45.4 ft bgs, re-ream R1 to 45.4 ft bgs.	
45	R2	60/60	45.40 - 50.40	RQD = 80%						R2: Bedrock: Similar to R1. Solid piece below 46.8 ft bgs. Rock Mass Quality = Good. R2: Core Times (min:sec)	
50											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Ice House Bridge #3726 carries Routes 8/ 201A (River Road) over Gilman Brook
Location: Anson, Maine

Boring No.: BB-AGB-101
WIN: 21657.00

Driller:	New England Boring Co.	**Elevation (ft.)**	256.7	**Auger ID/OD:**	5" Solid Stem
Operator:	Enos/Share	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	Mobile B53 Rubber Track	**Hammer Wt./Fall:**	140/30"
Date Start/Finish:	7/5/2017-7/6/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	104+89.4, 7.8 ft Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	8.8 ft bgs, open
Hammer Efficiency Factor:	0.677	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50							V	206.3		45.4-46.4 ft (2:10) 46.4-47.4 ft (2:20) 47.4-48.4 ft (2:45) 48.4-49.4 ft (3:15) 49.4-50.4 ft (3:40) 100% Recovery 50.4 **Bottom of Exploration at 50.4 feet below ground surface.**	
55											
60											
65											
70											
75											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Ice House Bridge #3726 carries Routes 8/ 201A (River Road) over Gilman Brook
Location: Anson, Maine

Boring No.: BB-AGB-102

WIN: 21657.00

Driller:	New England Boring Co.	**Elevation (ft.)**	256.1	**Auger ID/OD:**	5" Solid Stem
Operator:	Enos/Share	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	Mobile B53 Rubber Track	**Hammer Wt./Fall:**	140/30"
Date Start/Finish:	7/6/2017-7/7/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	105+35.6, 10.8 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed

Hammer Efficiency Factor: 0.677 **Hammer Type:** Automatic ☒ Hydraulic ☐ Rope & Cathead ☐

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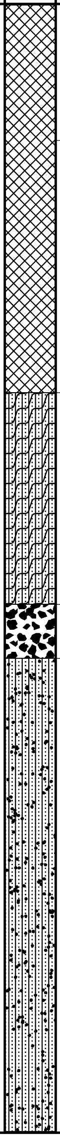

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample Information										
0							SSA				
	1D	24/17	1.00 - 3.00	23/21/22/20	43	49				Brown, dry to damp, dense, fine to coarse Sandy GRAVEL, little silt. Asphalt in tip of spoon; Strong Petroleum Odor, (Fill).	
5	2D	24/10	5.00 - 7.00	3/2/4/2	6	7	6			Reddish brown, damp to moist, loose, fine to medium SAND, some silt, trace gravel, trace coarse sand, (Fill).	G#270362 A-2-4, SM WC=15.6%
							7				
							12				
							52	247.5		8.6 Wood at 8.6 ft bgs based on drilling behavior.	
							46				
10	3D	24/1	10.00 - 12.00	1/1/2/3	3	3	31			Red-tan, soft, fine Sandy SILT, trace wood, trace coarse sand, (Relic Topsoil).	
							34				
							51				
							138	242.8		13.3 Riverbed gravel and cobbles, (Alluvium).	
							RC	241.6		14.5	
15	4D	24/7	15.00 - 17.00	22/18/8/10	26	29	107			Dark grey, medium dense, SAND, some silt, little clay, little gravel, (Glacial Till).	G#270363 A-4, SC-SM WC=9.1%
							64				
							119				
							74				
							94				
20	5D	24/9	20.00 - 22.00	6/16/13/47	29	33	65			Dark grey, hard, Gravelly SILT, some fine to coarse sand, layered, (Glacial Till).	
							122				
							205				
							RC				
25											

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Ice House Bridge #3726 carries Routes 8/ 201A (River Road) over Gilman Brook **Location:** Anson, Maine	**Boring No.:**	BB-AGB-102
		WIN:	21657.00

Driller:	New England Boring Co.	**Elevation (ft.)**	256.1	**Auger ID/OD:**	5" Solid Stem
Operator:	Enos/Share	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	Mobile B53 Rubber Track	**Hammer Wt./Fall:**	140/30"
Date Start/Finish:	7/6/2017-7/7/2017	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	105+35.6, 10.8 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.677	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows				
25	MD	2.4/0	25.00 - 25.20	50(2")	--					Dark grey, Gravel. Silty fine SAND in tip of spoon.	
30	MD	2.4/0	30.00 - 30.20	120(2")	---						
	6D	6/4	31.00 - 31.50	365(6")	---					Grey, very dense, GRAVEL, some sand, little silt, (Glacial Till).	G#270364 A-1-b, GM WC=6.3%
35	7D	12/12	35.00 - 36.00	79/127	---					Grey, very dense, Gravelly fine to medium SAND, little to some silt; layered, (Glacial Till).	
40	8D	20.4/20.4	40.00 - 41.70	50/50/93/50(2")	143	161	RC			Grey, very dense, fine to medium SAND, some silt, trace fine gravel, (Glacial Till).	G#270365 A-2-4, SM WC=14.4%
45								210.6		45.5 Weathered Bedrock.	
	R1	48/47	46.30 - 50.30	RQD = 8%			NQ-2	209.8		46.3 Top of Bedrock at Elev. 209.8 ft. R1: Bedrock: Green-grey, aphanitic to medium grained, META ARGILLACEOUS SANDSTONE, quartz veins, hard, typically fresh, some remnant bedding visible and highly folded. Close, typically high angle with minor low angle breaks; undulating, rough, fresh to discolored, moderately close, with mud infilling, highly broken at 46.3-47.1 ft bgs.	
50											

Remarks:

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Hammer Efficiency Factor:	0.677	**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.
50	R2	36/35	50.30 - 53.30	RQD = 25%						Rock Mass Quality = Very Poor. [SANGERVILLE FORMATION] R1: Core Times (min:sec) 46.3-47.0 ft (---) 47.0-48.0 ft (3:05) 48.0-49.0 ft (2:55) 49.0-50.0 ft (3:05) 50.0-50.3 ft (---) 98% Recovery	
	R3	48/47	53.30 - 57.30	RQD = 33%						R2: Bedrock: Similar to R1. Highly broken 53.0-53.3 ft bgs. Rock Mass Quality = Very Poor. R2: Core Times (min:sec) 50.3-51.3 ft (2:10) 51.3-52.3 ft (2:20) 52.3-53.3 ft (2:50) 97% Recovery	
55											
								198.8		R3: Bedrock: Similar to R1. Highly broken 55.6-56.0 ft bgs. Near vertical fracture terminates at 56.7 ft bgs. Rock Mass Quality = Poor. No core times taken. 98% Recovery	
										57.3	
										Bottom of Exploration at 57.3 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.

Appendix B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Anson Work Number: 21657.00

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-AGB-101, 2D	104+89.4	7.8 Lt.	5.0-7.0	270357	1	18.0			SM	A-4	III
BB-AGB-101, 3D	104+89.4	7.8 Lt.	10.0-12.0	270358	1	28.1			SM	A-2-4	II
BB-AGB-101, 4D/A	104+89.4	7.8 Lt.	16.0-17.0	270359	1	10.2			GW-GM	A-1-a	0
BB-AGB-101, 6D	104+89.4	7.8 Lt.	25.0-27.0	270360	1	8.7			SM	A-2-4	II
BB-AGB-101, 7D	104+89.4	7.8 Lt.	30.0-31.8	270361	1	20.2			CL	A-4	IV
BB-AGB-102, 2D	105+35.6	10.8 Rt.	5.0-7.0	270362	2	15.6			SM	A-2-4	II
BB-AGB-102, 4D	105+35.6	10.8 Rt.	15.0-17.0	270363	2	9.1			SC-SM	A-4	III
BB-AGB-102, 6D	105+35.6	10.8 Rt.	31.0-31.5	270364	2	6.3			GM	A-1-b	I
BB-AGB-102, 8D	105+35.6	10.8 Rt.	40.0-41.7	270365	2	14.4			SM	A-2-4	II

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

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

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

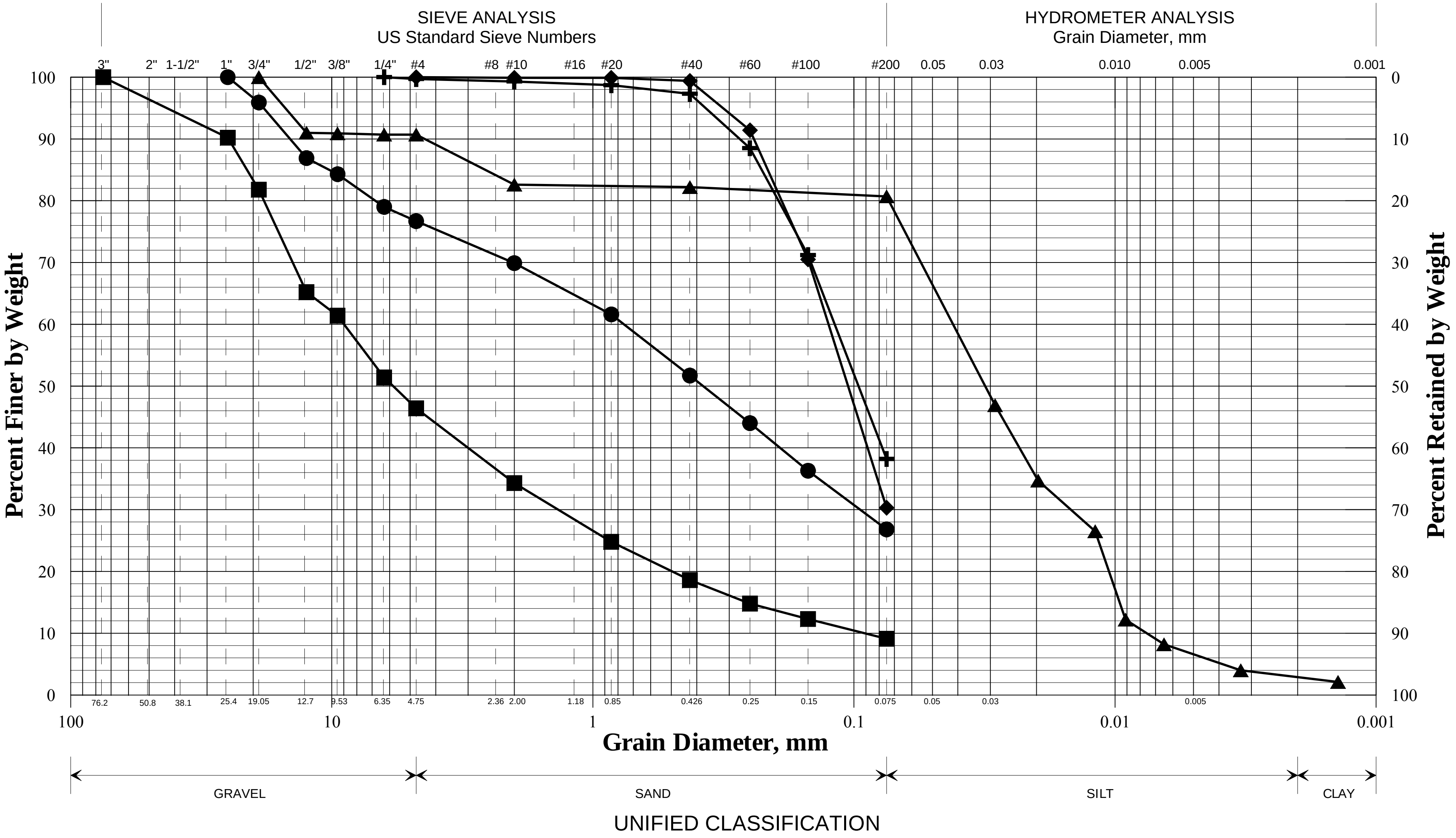
LL = Liquid limit as determined by AASHTO T 89-96 and/or ASTM D 4318-98

NP = Non Plastic

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

State of Maine Department of Transportation

GRAIN SIZE DISTRIBUTION CURVE

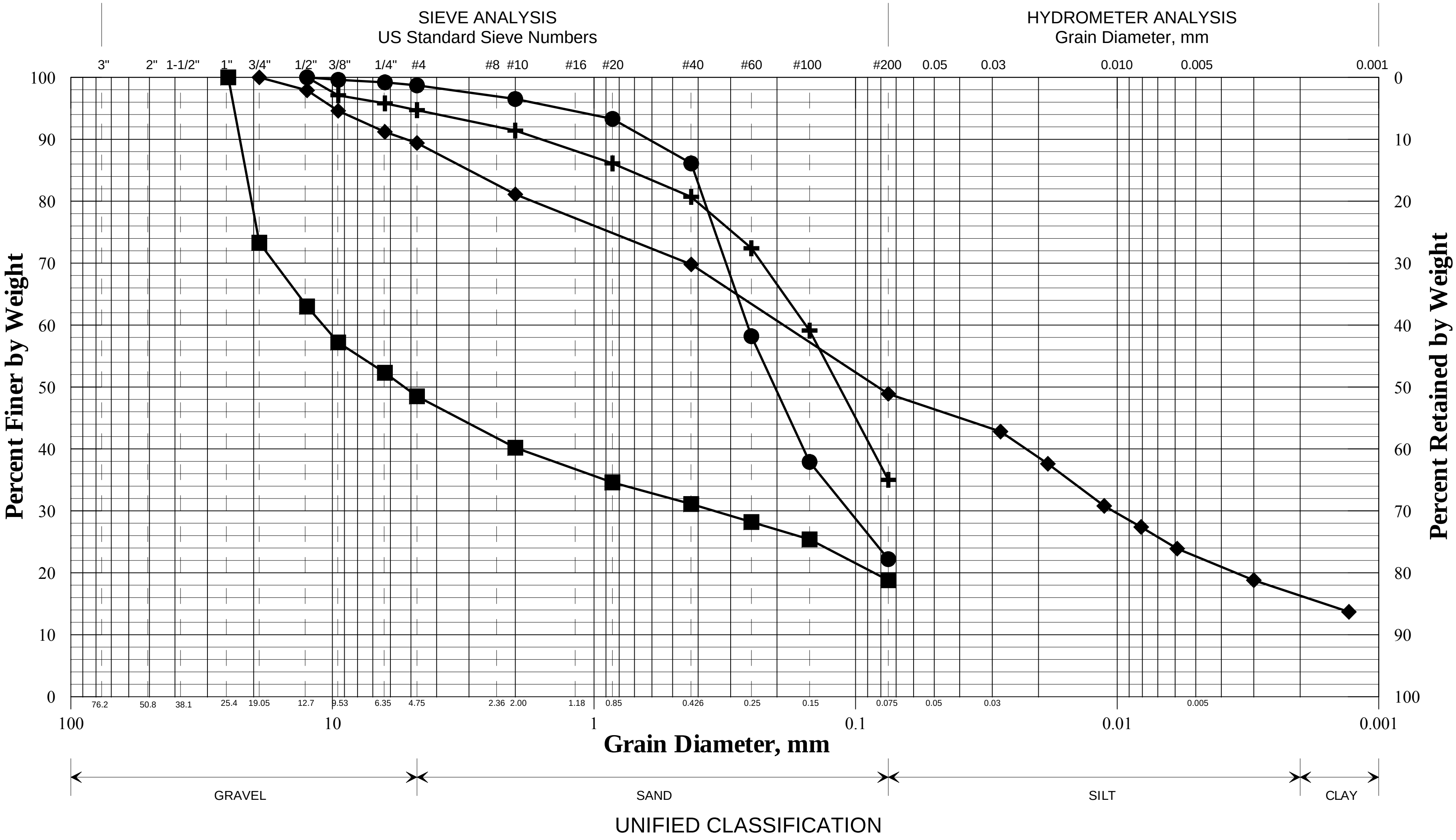


	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-AGB-101/2D	104+89.4	7.8 LT	5.0-7.0	Silty SAND, trace gravel.	18.0			
◆	BB-AGB-101/3D	104+89.4	7.8 LT	10.0-12.0	SAND, some silt,	28.1			
■	BB-AGB-101/4DA	104+89.4	7.8 LT	16.0-17.0	Sandy GRAVEL, trace silt.	10.2			
●	BB-AGB-101/6D	104+89.4	7.8 LT	25.0-27.0	SAND, some silt, some gravel.	8.7			
▲	BB-AGB-101/7D	104+89.4	7.8 LT	30.0-31.8	SILT, trace sand, trace gravel, trace clay.	20.2			
✖									

WIN	
021657.00	
Town	
Anson	
Reported by/Date	
WHITE, TERRY A	7/27/2017

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-AGB-102/2D	105+35.6	10.8 RT	5.0-7.0	SAND, some silt, trace gravel.	15.6			
◆	BB-AGB-102/4D	105+35.6	10.8 RT	15.0-17.0	SAND, some silt, little clay, little gravel.	9.1			
■	BB-AGB-102/6D	105+35.6	10.8 RT	31.0-31.5	GRAVEL, some sand, little silt.	6.3			
●	BB-AGB-102/8D	105+35.6	10.8 RT	40.0-41.7	SAND, some silt, trace gravel.	14.4			
▲									
✖									

WIN	
021657.00	
Town	
Anson	
Reported by/Date	
WHITE, TERRY A	7/27/2017

Appendix C

Calculations

H-Pile Resistance

Design of H-piles

Reference: AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017.

Bedrock Properties

Rock Type: Metasandstone.

ϕ = 27-34 (AASHTO LRFD Table C.10.4.6.4-1);
Co = 9,700 - 25,000 psi (AASHTO Standard Specifications for Bridges 17th Edition, Table 4.4.8.2.2)

For Design Purposes, use bedrock data from BB-AGB-102: RQD = 8% and an assumed Unconfined Compressive Strength of 10,000 psi.

Pile Properties

Use the following piles: 12x53, 12x74, 14x73, 14x89, 14x117

$$A_s := \begin{pmatrix} 15.5 \\ 21.8 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot in^2 \qquad d := \begin{pmatrix} 11.78 \\ 12.13 \\ 13.6 \\ 13.83 \\ 14.21 \end{pmatrix} \cdot in \qquad b := \begin{pmatrix} 12.045 \\ 12.215 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot in$$

$$A_{box} := \overrightarrow{(d \cdot b)} \qquad A_{box} = \begin{pmatrix} 141.89 \\ 148.168 \\ 198.356 \\ 203.232 \\ 211.516 \end{pmatrix} \cdot in^2$$

12x53
12x74
14x73 Note: All matrices set up in this order
14x89
14x117

r_s= radius of gyration $\qquad r_s := \begin{pmatrix} 2.86 \\ 2.92 \\ 3.49 \\ 3.53 \\ 3.59 \end{pmatrix} \cdot in$

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

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

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

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

$$\boxed{P_o = \begin{pmatrix} 775 \\ 1090 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot \text{kip}}$$

Slender element reduction factor, Q, may be required to reduce resistance for 12x53 and 14x73 H-pile sections per LFRD 6.9.4.2.

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 $E := 29000 \cdot \text{ksi}$

K = effective length factor $K_{eff} := 2.1$ LRFD Table C4.6.2.5-1
(assume plastic hinge develops, K selected per Vtrans Design Procedure)

l = "unbraced" length $l_{unbraced\_top} := 4.9 \cdot \text{ft}$

LRFD eq. 6.9.4.1.2-1

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

$$P_e = \begin{pmatrix} 2380 \\ 3489 \\ 4893 \\ 6105 \\ 8322 \end{pmatrix} \cdot \text{kip}$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 3.071 \\ 3.201 \\ 4.573 \\ 4.678 \\ 4.839 \end{pmatrix}$$

If Pe/Po > or = 0.44, then:

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

LRFD Eq. 6.9.4.1.1-1

then:

$$\boxed{P_n = \begin{pmatrix} 676 \\ 956 \\ 976 \\ 1193 \\ 1577 \end{pmatrix} \cdot \text{kip}}$$

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

$$P_r := \phi_{cu} \cdot P_n$$

Factored structural compressive resistance, P_r

this applies to all pile sizes

$$\boxed{P_r = \begin{pmatrix} 473 \\ 669 \\ 683 \\ 835 \\ 1104 \end{pmatrix} \cdot kip}$$

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 assumed pinned at the top and bottom. (see also LRFD Table C4.6.2.5-1)

l = "unbraced" length $l_{unbraced\_mid} := 9.4 \cdot ft$

Assumed unbraced length for middle zone. Actual determined from LPile.

LRFD eq. 6.9.4.1.2-1

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

$$P_e = \begin{pmatrix} 2852 \\ 4181 \\ 5863 \\ 7316 \\ 9973 \end{pmatrix} \cdot kip$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 3.68 \\ 3.836 \\ 5.48 \\ 5.606 \\ 5.798 \end{pmatrix}$$

If Pe/Po > or = 0.44, then:

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

LRFD Eq. 6.9.4.1.1-1

then:

this applies to all pile sizes

$$\boxed{P_n = \begin{pmatrix} 692 \\ 977 \\ 991 \\ 1211 \\ 1600 \end{pmatrix} \cdot kip}$$

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 $P_r := \phi_{cu} \cdot P_n$

Factored structural compressive resistance, P_r

this applies to all pile sizes

$$\boxed{P_r = \begin{pmatrix} 484 \\ 684 \\ 694 \\ 848 \\ 1120 \end{pmatrix} \cdot \text{kip}}$$

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 $E := 29000 \cdot \text{ksi}$

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

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

LRFD eq. 6.9.4.1.2-1

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

$$P_e = \begin{pmatrix} 6 \times 10^7 \\ 9 \times 10^7 \\ 1 \times 10^8 \\ 2 \times 10^8 \\ 2 \times 10^8 \end{pmatrix} \cdot \text{kip}$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 7.696 \times 10^4 \\ 8.022 \times 10^4 \\ 1.146 \times 10^5 \\ 1.172 \times 10^5 \\ 1.213 \times 10^5 \end{pmatrix}$$

If Pe/Po > or = 0.44, then:

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

LRFD Eq. 6.9.4.1.1-1

then:

this applies to all pile sizes

$$\boxed{P_n = \begin{pmatrix} 775 \\ 1090 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot \text{kip}}$$

Factored Axial Structural Resistance for the Strength Limit State

Resistance factor for H-pile in pure compression, severe driving conditions, per LRFD 6.5.4.2 for the case where pile tip is necessary

$\phi_c := 0.5$

The Factored Structural Resistance (Pr) per LRFD 6.9.2.1-1 is

$P_r := \phi_c \cdot P_n$

Factored structural compressive resistance, P_r

$$P_r = \begin{pmatrix} 387 \\ 545 \\ 535 \\ 652 \\ 860 \end{pmatrix} \cdot \text{kip}$$

LRFD 10.7.3.2.3 - Piles Driven to Hard Rock -

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

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

Nominal Structural Resistance Previously Calculated:

$$P_n = \begin{pmatrix} 775 \\ 1090 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot \text{kip}$$

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

$\phi_c := 0.5$

$P_r := \phi_c \cdot P_n$

$$P_r = \begin{pmatrix} 387 \\ 545 \\ 535 \\ 652 \\ 860 \end{pmatrix} \cdot \text{kip}$$

12x53
12x74
14x73
14x89
14x117

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

$$\phi_c := 1.0$$

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

$$\boxed{P_{r\_ee} = \begin{pmatrix} 775 \\ 1090 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot kip}$$

12x53
12x74
14x73
14x89
14x117

Nominal and Factored Axial Geotechnical Resistance of HP piles

Geotechncial axial pile resistance for pile 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

$$q_{u\_1} := 10000 \cdot psi$$

Geotechnical tip resistance.

$$q_{p\_2} := 2.5 \cdot q_{u\_1} \qquad \boxed{q_{p\_2} = 3600 \cdot ksf}$$

Nominal geotechnical tip resistance, R_p

$$R_p := \overrightarrow{\left(q_{p\_2} \cdot A_s\right)} \qquad \boxed{R_p = \begin{pmatrix} 388 \\ 545 \\ 535 \\ 653 \\ 860 \end{pmatrix} \cdot kip}$$

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)

$R_{r\_p2} := \phi_{stat} \cdot R_p$

$$\boxed{R_{r\_p2} = \begin{pmatrix} 174 \\ 245 \\ 241 \\ 294 \\ 387 \end{pmatrix} \cdot kip}$$

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)

$R_{r\_p2} := \phi \cdot R_p$

$$\boxed{R_{r\_p2} = \begin{pmatrix} 388 \\ 545 \\ 535 \\ 653 \\ 860 \end{pmatrix} \cdot kip}$$

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)

$R_{r\_p2} := \phi_{ee} \cdot R_p$

$$\boxed{R_{r\_p2} = \begin{pmatrix} 388 \\ 545 \\ 535 \\ 653 \\ 860 \end{pmatrix} \cdot kip}$$

Drivability Analyses

Ref: LRFD Article 10.7.8

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

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

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

$\sigma_{dr} = 45 \cdot \text{ksi}$ 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

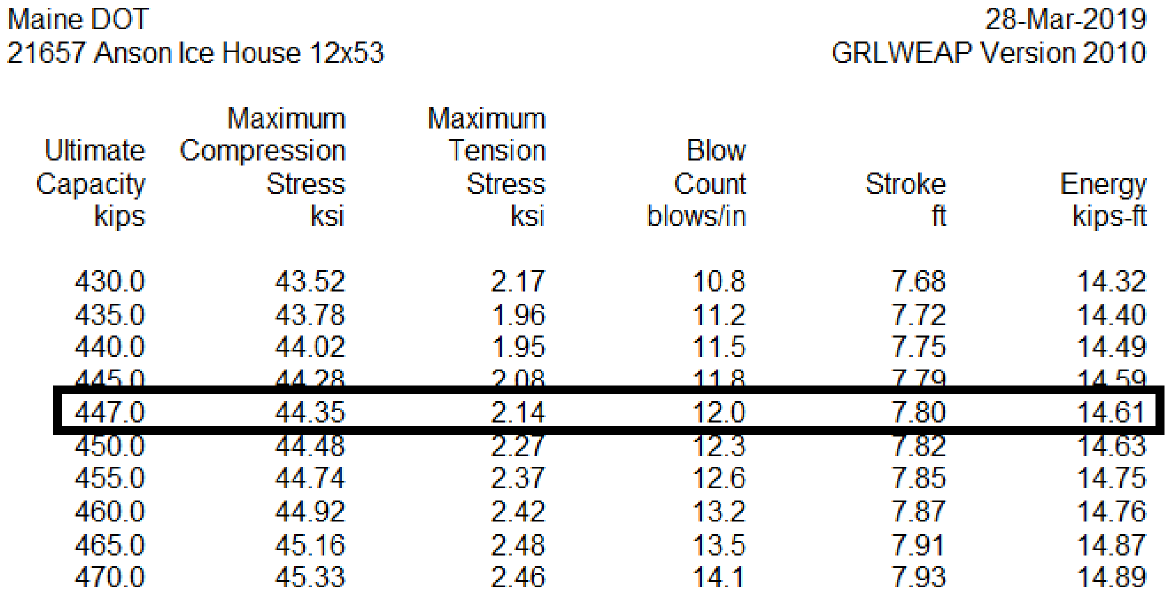
GRLWeap Soil and Pile Model Assumptions

Based on Table 2 of this Report, estimated pile lengths will be approx. 30 to 37 ft. Assume contractor drives pile lengths of 40 ft (extra length accommodates for attachment of dynamic testing equipment, embedment into abutment, variation in bedrock surface).

Use proportional shaft resistances so that GRLWeap will assign approx. 10% of the ultimate capacities as skin friction consistent with observed dynamic test results in similar glacial till deposits. (Ref: WIN 18958 Day Block, Pembroke Stream Bridge)

Pile Size is 12 x 53

The 12x53 pile can be driven to the resistances below with a D 19-42 hammer at fuel setting -3 (73% of Max) and 1.9 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:



Maine DOT
21657 Anson Ice House 12x53

28-Mar-2019
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
430.0	43.52	2.17	10.8	7.68	14.32
435.0	43.78	1.96	11.2	7.72	14.40
440.0	44.02	1.95	11.5	7.75	14.49
445.0	44.28	2.08	11.8	7.79	14.59
447.0	44.35	2.14	12.0	7.80	14.61
450.0	44.48	2.27	12.3	7.82	14.63
455.0	44.74	2.37	12.6	7.85	14.75
460.0	44.92	2.42	13.2	7.87	14.76
465.0	45.16	2.48	13.5	7.91	14.87
470.0	45.33	2.46	14.1	7.93	14.89

Limit stress to 45 ksi

$$R_{ndr} := 447 \cdot kip$$

Strength Limit State

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

$$\boxed{R_{fdr} = 291 \cdot kip}$$

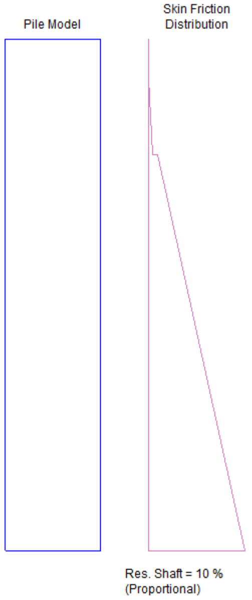
Extreme and
Service Limit States

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

$$\boxed{R_{dr} = 447 \cdot kip}$$

DELMAG D 19-42

Ram Weight	4.00	kips
Efficiency	0.800	
Pressure	1165 (73%)	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	40.00	ft
Pile Penetration	37.00	ft
Pile Top Area	15.50	in2



Pile Size is 12 x 74

The 12x74 pile can be driven to the resistances below with a D 19-42 hammer at fuel setting -2 (81% of max) and 1.9 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:

```
Maine DOT                                                      28-Mar-2019
21657 Anson Ice House 12x74                          GRLWEAP Version 2010

               Maximum     Maximum
 Ultimate  Compression     Tension        Blow
 Capacity       Stress      Stress       Count      Stroke      Energy
     kips          ksi         ksi   blows/in          ft     kips-ft

    500.0        41.02        3.52        11.1        8.29       15.83
    510.0        41.47        3.45        11.7        8.35       15.97
    520.0        41.86        3.10        12.3        8.41       16.09
    530.0        42.27        2.81        12.8        8.46       16.23
    540.0        42.68        2.86        13.4        8.54       16.43
    550.0        43.14        3.31        14.0        8.59       16.56
    560.0        43.51        3.44        14.8        8.65       16.69
    564.0        43.67        3.45        15.0        8.68       16.77
    570.0        43.88        3.31        15.4        8.71       16.82
    575.0        44.04        3.25        15.7        8.73       16.89
```

Limit to 15 bpi

$R_{ndr} := 564 \cdot kip$

Strength Limit State

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

$\boxed{R_{fdr} = 367 \cdot kip}$

Extreme and
Service Limit States

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

$\boxed{R_{dr} = 564 \cdot kip}$

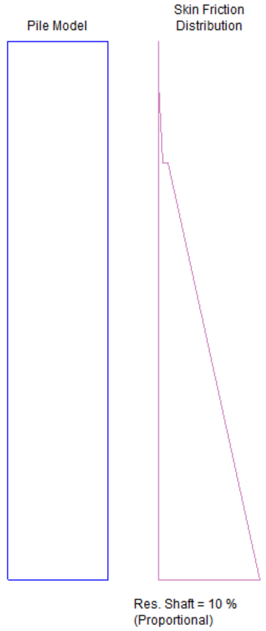
```
DELMAG  D 19-42

Ram Weight                  4.00  kips
Efficiency                 0.800
Pressure             1295 (81%)  psi

Helmet Weight               1.90  kips
Hammer Cushion             60155  kips/in
COR of H.C.                0.800

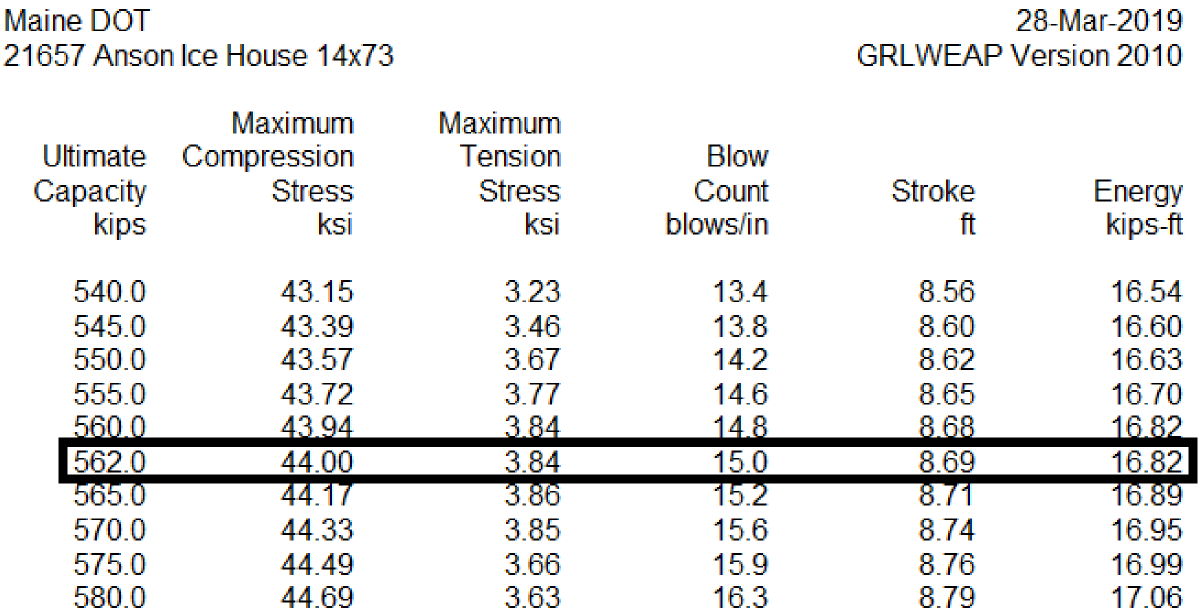
Skin Quake                 0.100  in
Toe Quake                  0.040  in
Skin Damping               0.050  sec/ft
Toe Damping                0.150  sec/ft

Pile Length                40.00  ft
Pile Penetration           37.00  ft
Pile Top Area              21.80  in2
```



Pile Size is 14 x 73

The 14x73 pile can be driven to the resistances below with a D 19-42 at fuel setting -2 (81% of max) hammer and 1.9 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:



Maine DOT
21657 Anson Ice House 14x73

28-Mar-2019
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
540.0	43.15	3.23	13.4	8.56	16.54
545.0	43.39	3.46	13.8	8.60	16.60
550.0	43.57	3.67	14.2	8.62	16.63
555.0	43.72	3.77	14.6	8.65	16.70
560.0	43.94	3.84	14.8	8.68	16.82
562.0	44.00	3.84	15.0	8.69	16.82
565.0	44.17	3.86	15.2	8.71	16.89
570.0	44.33	3.85	15.6	8.74	16.95
575.0	44.49	3.66	15.9	8.76	16.99
580.0	44.69	3.63	16.3	8.79	17.06

Limit blow counts to 15 bpi.

$$R_{ndr} := 562 \cdot kip$$

Strength Limit State

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

$$\boxed{R_{fdr} = 365 \cdot kip}$$

Extreme and Service Limit States

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

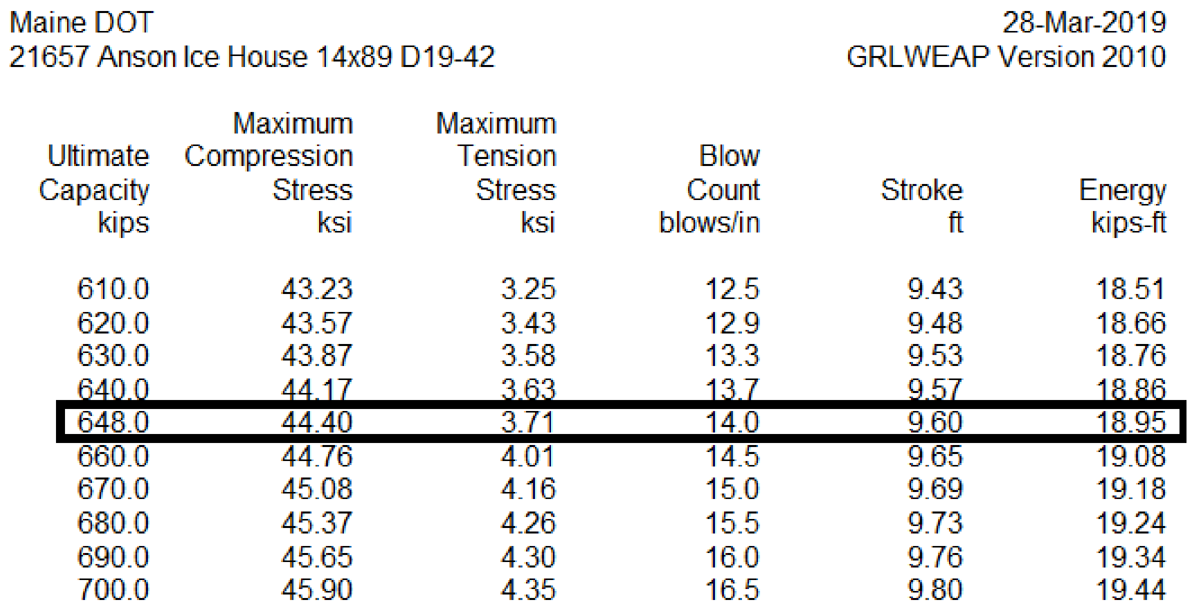
$$\boxed{R_{dr} = 562 \cdot kip}$$

DELMAG D 19-42

Ram Weight	4.00	kips
Efficiency	0.800	
Pressure	1295 (81%)	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	40.00	ft
Pile Penetration	37.00	ft
Pile Top Area	21.40	in2

Pile Size is 14 x 89

The 14x89 pile can be driven to the resistances below with a D 19-42 hammer at -1 fuel setting (90% of max) and 1.9 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:



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Maine DOT                                                    28-Mar-2019
21657 Anson Ice House 14x89 D19-42                  GRLWEAP Version 2010

             Maximum      Maximum
Ultimate  Compression     Tension       Blow
Capacity       Stress      Stress      Count      Stroke      Energy
    kips          ksi         ksi   blows/in          ft     kips-ft

   610.0        43.23        3.25       12.5        9.43       18.51
   620.0        43.57        3.43       12.9        9.48       18.66
   630.0        43.87        3.58       13.3        9.53       18.76
   640.0        44.17        3.63       13.7        9.57       18.86
   648.0        44.40        3.71       14.0        9.60       18.95
   660.0        44.76        4.01       14.5        9.65       19.08
   670.0        45.08        4.16       15.0        9.69       19.18
   680.0        45.37        4.26       15.5        9.73       19.24
   690.0        45.65        4.30       16.0        9.76       19.34
   700.0        45.90        4.35       16.5        9.80       19.44
```

Limiting driving stress to 45 ksi

$R_{ndr} := 648 \cdot kip$

Strength Limit State

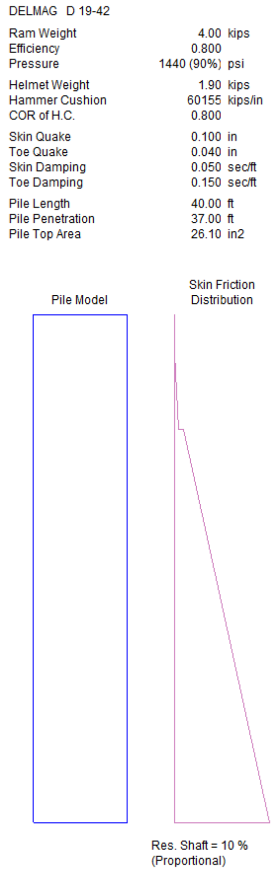
$R_{fdr} := R_{ndr} \cdot \phi_{dyn}$

$\boxed{R_{fdr} = 421 \cdot kip}$

Extreme and Service Limit States

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

$\boxed{R_{dr} = 648 \cdot kip}$



```
DELMAG  D 19-42

Ram Weight                4.00  kips
Efficiency               0.800
Pressure           1440 (90%)  psi

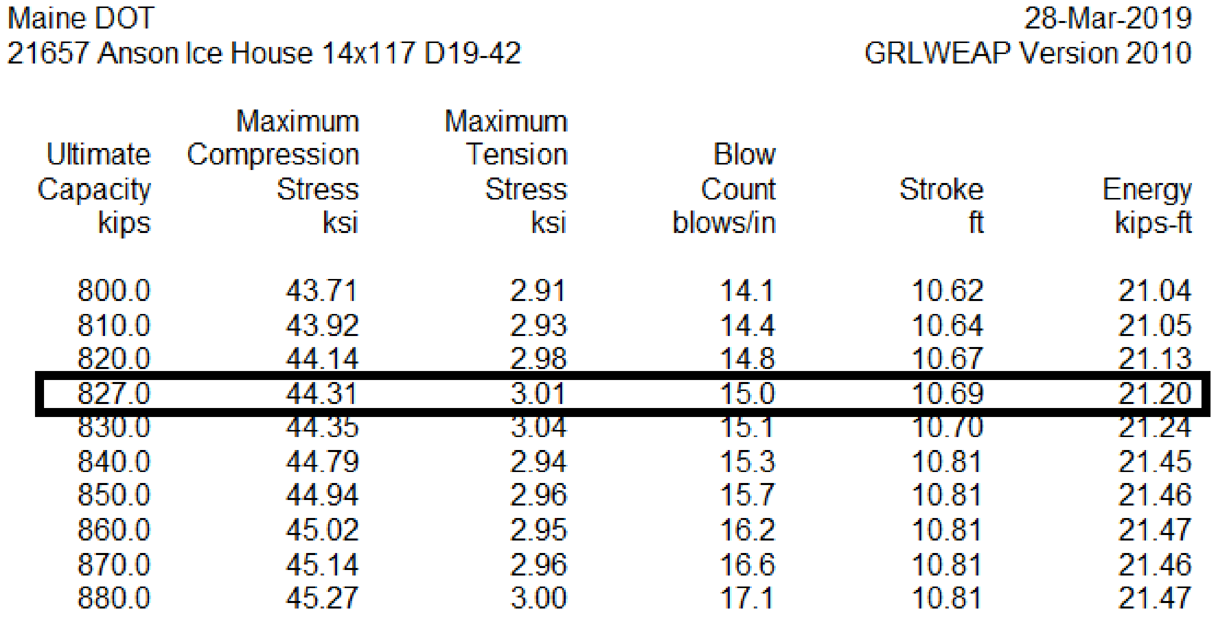
Helmet Weight             1.90  kips
Hammer Cushion           60155  kips/in
COR of H.C.              0.800

Skin Quake               0.100  in
Toe Quake                0.040  in
Skin Damping             0.050  sec/ft
Toe Damping              0.150  sec/ft

Pile Length              40.00  ft
Pile Penetration         37.00  ft
Pile Top Area            26.10  in2
```

Pile Size is 14 x 117

The 14x117 pile can be driven to the resistances below 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
21657 Anson Ice House 14x117 D19-42

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Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
800.0	43.71	2.91	14.1	10.62	21.04
810.0	43.92	2.93	14.4	10.64	21.05
820.0	44.14	2.98	14.8	10.67	21.13
827.0	44.31	3.01	15.0	10.69	21.20
830.0	44.35	3.04	15.1	10.70	21.24
840.0	44.79	2.94	15.3	10.81	21.45
850.0	44.94	2.96	15.7	10.81	21.46
860.0	45.02	2.95	16.2	10.81	21.47
870.0	45.14	2.96	16.6	10.81	21.46
880.0	45.27	3.00	17.1	10.81	21.47

Limit blows to 15 bpi

$$R_{ndr} := 827 \cdot kip$$

Strength Limit State

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

$$\boxed{R_{fdr} = 538 \cdot kip}$$

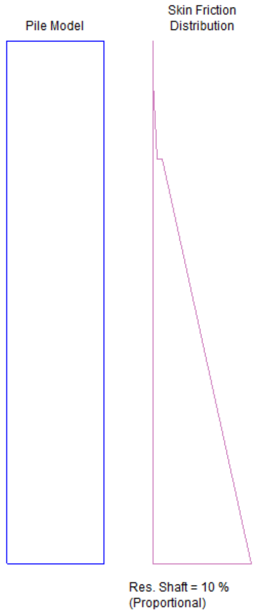
Extreme and Service Limit States

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

$$\boxed{R_{dr} = 827 \cdot kip}$$

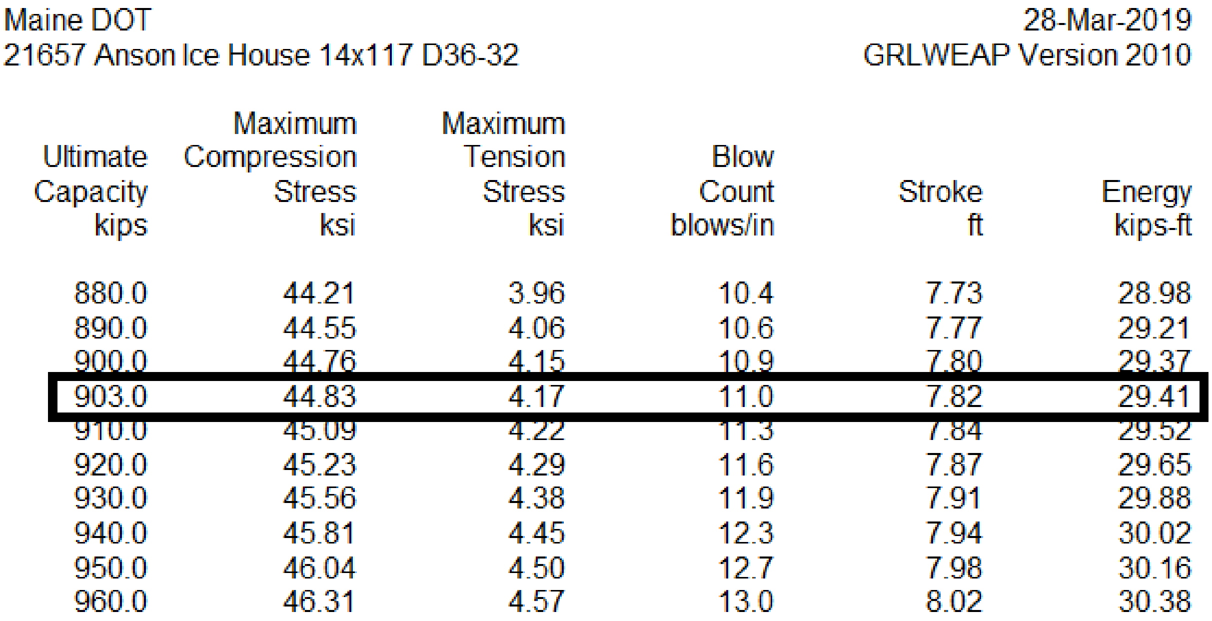
DELMAG D 19-42

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	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	40.00	ft
Pile Penetration	37.00	ft
Pile Top Area	34.40	in2



Pile Size is 14 x 117

The 14x117 pile can be driven to the resistances below with a D 36-32 hammer at 73% of max (-3) fuel setting and 3.2 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:



Maine DOT
21657 Anson Ice House 14x117 D36-32

28-Mar-2019
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
880.0	44.21	3.96	10.4	7.73	28.98
890.0	44.55	4.06	10.6	7.77	29.21
900.0	44.76	4.15	10.9	7.80	29.37
903.0	44.83	4.17	11.0	7.82	29.41
910.0	45.09	4.22	11.3	7.84	29.52
920.0	45.23	4.29	11.6	7.87	29.65
930.0	45.56	4.38	11.9	7.91	29.88
940.0	45.81	4.45	12.3	7.94	30.02
950.0	46.04	4.50	12.7	7.98	30.16
960.0	46.31	4.57	13.0	8.02	30.38

Limit stress to 45 ksi

$$R_{ndr} := 903 \cdot kip$$

Strength Limit State

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

$$\boxed{R_{fdr} = 587 \cdot kip}$$

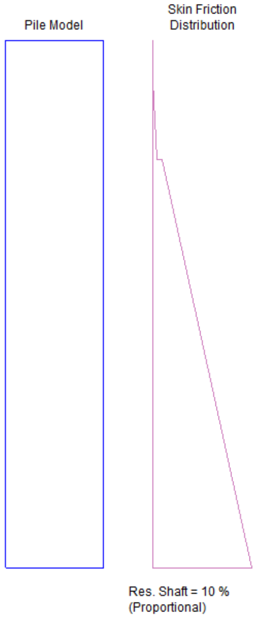
Extreme and Service Limit States

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

$$\boxed{R_{dr} = 903 \cdot kip}$$

DELMAG D 36-32

Ram Weight	7.93	kips
Efficiency	0.800	
Pressure	1095 (73%)	psi
Helmet Weight	3.20	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	40.00	ft
Pile Penetration	37.00	ft
Pile Top Area	34.40	in2



LPile Parameters

Development of soil model for LPile

OBJECTIVE
Estimate soil parameters for lateral pile analyses.

Given:
1) 100-series boring logs and lab data.

Assumptions:
1) Assume the groundwater table is at Elevation 247.0.
2) MaineDOT Bridge Design Guide (BDG) Soil Type 4 will be used for integral abutment backfill.
3) Piles are driven to, or within, bedrock.

LPile Soil Model

1) The design soil layers are delineated as depicted on the attached annotated boring log, 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. 256.7 - 241.0

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

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

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

Soil Layer No. 2 (Submerged, loose, fine Sand) El. 247.0 - 241.0

Design N_{60} = 3 bpf

Sample BB-AGB-101;3D

Internal Angle of Friction

$\boxed{\phi_2 := 28}$

Lambe and Whitman, Fig 11.14, correlation between friction angle and pentration resistance

Dry Unit Weight

Loose angular silty sand = 102 pcf

$\gamma_{2\text{dry}} := 102\text{pcf}$

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

Natural water content at saturated state:

$w_{2\text{sat}} := .28$

sample BB-AGB-101;3D

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

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

Saturated Unit Weight

$\gamma_{2\text{saturated}} = 131 \cdot \text{pcf}$

Effective Unit Weight

weight of water = γ_w =62.4 pcf

$\gamma_w := 62.4\text{pcf}$

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

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

$\boxed{\gamma'_2 = 68 \cdot \text{pcf}}$

Representative constant giving the variation of soil modulus with depth, k:
Medium dense sand below water table for static loading = 20 pci

Technical Manual
LPile 2016
p. 96

Soil Layer No. 3 (Submerged, Glacial Till) El. 241.0 - 217.0

Design N_{60} = 33 bpf

Sample BB-AGB-102;5D

Internal Angle of Friction

$\boxed{\phi_3 := 36}$

Lambe and Whitman, Fig 11.14, correlation between friction angle and pentration resistance

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:

$w_{3sat} := .10$

sample BB-AGB-101;4D/A

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

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

$\gamma_{3saturated} = 147 \cdot pcf$

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).

$\boxed{\gamma'_3 = 85 \cdot pcf}$

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

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	**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

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

Cantilevered Wingwall Live Load Surcharge
At-Rest Earth Pressure - Rankine Theory

$K_o := 1 - \sin(\phi')$

Das, Principles of Geotechnical Engineering 7th Ed. p 427 Eq. 13.5

$\boxed{K_o = 0.47}$

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:		
○ dressed soft rock on dressed soft rock	35	0.70
○ dressed hard rock on dressed soft rock	33	0.65
○ 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*)

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

7.12 Coulomb's Passive Earth Pressure

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

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

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

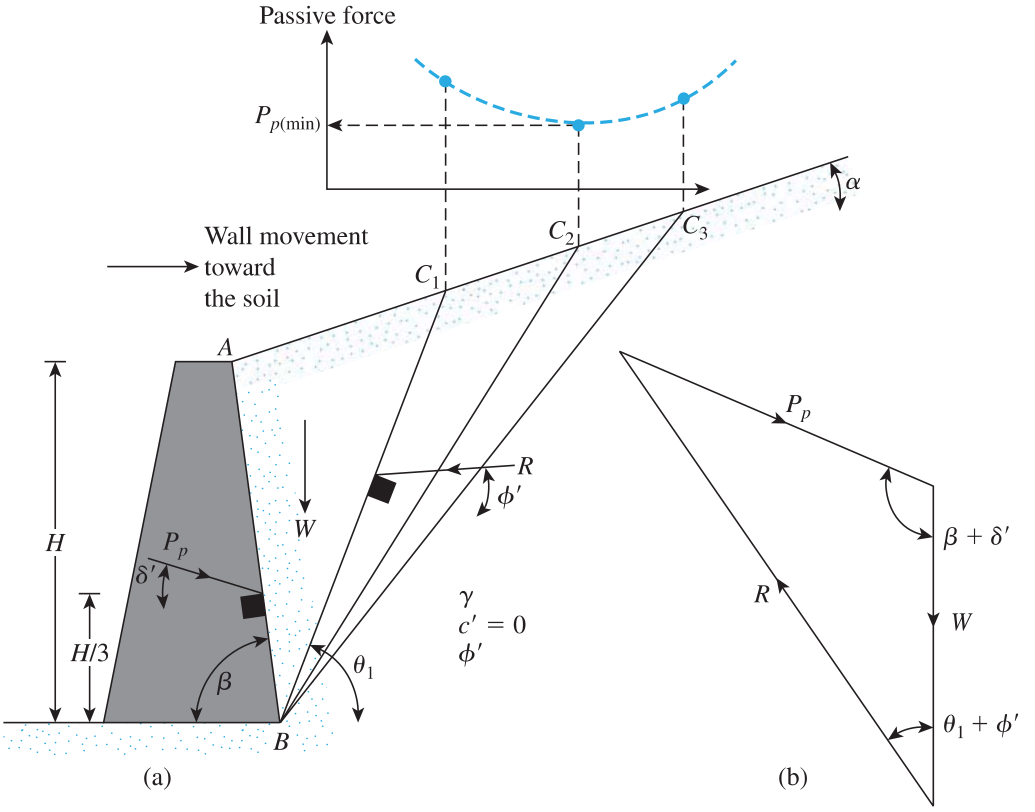


Figure 7.25 Coulomb's passive pressure

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,

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

Hence,

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

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

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

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

■

7.11 Rankine Passive Earth Pressure: Vertical Backface and Inclined Backfill

Granular Soil

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

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

and the passive force is

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

where

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

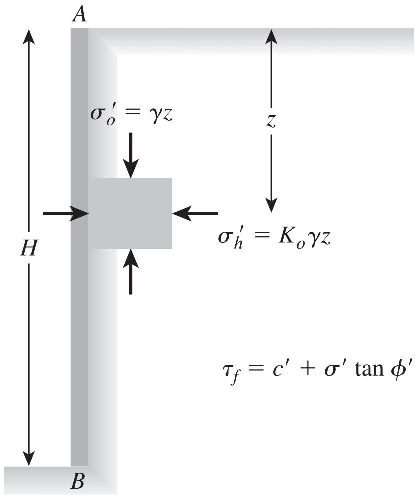


Figure 13.3 Earth pressure at rest

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

Anson
21657.00 Ice House Br #3726

Frost Penetration Analysis

B.Slaven
April 2019
Check by : __LK 4/2019

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

From Design Freezing Index Map: **Anson, Maine**
DFI = 1800 degree-days.
Case 1 - coarse grained granular fill soils W=15% (assumed).

For DFI = 1800

at w=20% $d_1 := 74.5\text{in}$

at w=10% $d_2 := 90.1\text{in}$

Depth of Frost Penetration

$$d := \frac{d_2 + d_1}{2} \qquad d = 82.3 \cdot \text{in} \qquad d = 6.9 \cdot \text{ft}$$

Method 2 - ModBerg Software

Examine foundations placed on coarse grained fill soils

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

```
                  --- ModBerg Results ---

Project Location: Madison, Maine
Air Design Freezing Index              = 1847 F-days
N-Factor                                         = 0.80
Surface Design Freezing Index          = 1478 F-days
Mean Annual Temperature                = 42.4 deg F
Design Length of Freezing Season       = 136 days
------------------------------------------------------------------------------------------
Layer
#:Type      t        w%       d        Cf       Cu       Kf       Ku       L
------------------------------------------------------------------------------------------
1-Coarse    87.7     15.0     125.0    31       40       2.9      1.8      2,700
------------------------------------------------------------------------------------------
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 f

  Total Depth of Frost Penetration = 7.3 ft = 87.7 in.
```

Recommendation: 6.9 feet for design of foundations constructed on coarse grained soils

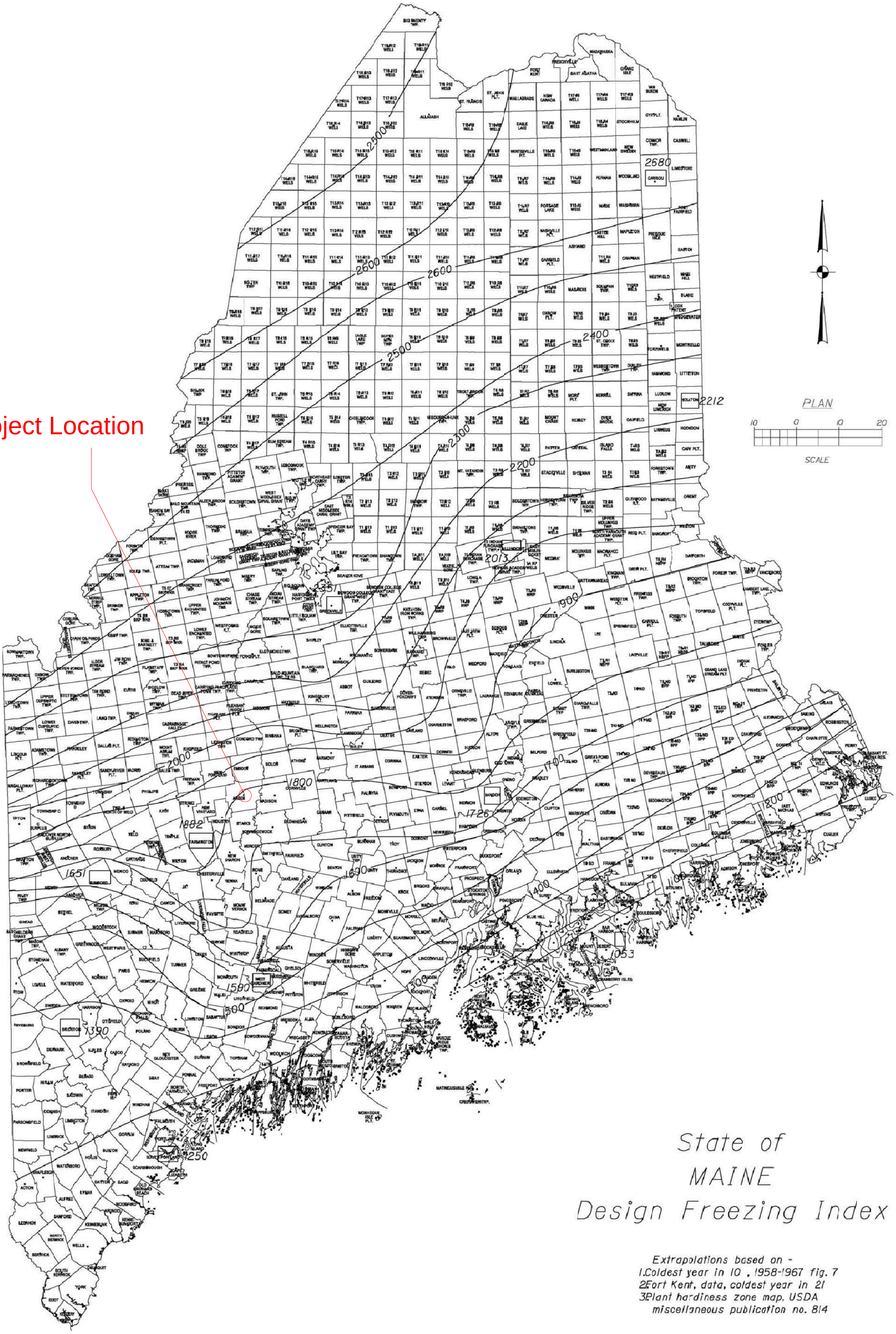


Figure 5-1 Maine Design Freezing Index Map

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

Anson Ice House Br
21657.00

Seismic Site Classification

B. Slaven
March 2019
Check by: _LK 4/2019_

BB-AGB-101				
Depth	**N_{60}**		**di**	**di/N**
1.5	60		1.5	0.03
5.5	14		4	0.29
11.5	3		6	2.00
15.5	36		4	0.11
20.5	68		5	0.07
25.5	78		5	0.06
30.5	100		5	0.05
35.5	100		5	0.05
40.0	100	**Bedrock**	4.5	0.05
100	100		60	0.60
SUM			**100**	**3.30**

di/di/N **30.26**

BB-AGB-102				
Depth	**102 N_{60}**		**di**	**di/N**
1.5	49		1.5	0.03
5.5	7		4	0.57
10.5	3		5	1.67
15.5	29		5	0.17
20.5	33		5	0.15
25.5	100		5	0.05
30.5	100		5	0.05
35.5	100		5	0.05
40.5	100		5	0.05
45.5	100	**Bedrock**	5	0.05
100	100		54.5	0.55
SUM			**100**	**3.39**

di/di/N **29.52**

SUM	**Nav.**	**29.89**

$15 < N_{av.} < 50$

Conclusion: Site Class D

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

Anson Ice House Br #3726 Seismic Parameters B. Slaven
21657.00 March 2019
Check by: LK 4/2019

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 44.813333
Longitude = -069.892222
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.076	PGA - Site Class B
0.2	0.163	Ss - Site Class B
1.0	0.048	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
Latitude = 44.813333
Longitude = -069.892222
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.122	As - Site Class D
0.2	0.260	SDs - Site Class D
1.0	0.115	SD1 - Site Class D