

**MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE**

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

**ARMSTRONG ROAD BRIDGE
ARMSTRONG ROAD OVER INTERSTATE 95
WATERVILLE, MAINE**



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Soils Report 2020-38
Bridge No. 5815

Fed No. STP-2350(500)
October 7, 2020

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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 Armstrong Road Bridge which carries Rice Rips Road (previously named Armstrong Road) over Interstate 95 in Waterville, Maine. This report presents the subsurface information obtained at the site during the subsurface investigation, geotechnical design recommendations, and construction recommendations for the new substructures.

The existing Armstrong Road Bridge was constructed in 1959 and is a five span steel girder bridge. The abutments are founded on spread footings formed and cast on native glacial till. The four concrete piers are founded on spread footings on bedrock. The bridge underwent a major rehabilitation in 1992.

According to the 2018 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the bridge deck and the steel beams are in fair to good condition. The abutments are in good condition with only minor cracking of the concrete. The concrete piers are in poor condition with three of the four piers exhibiting heavy cracking and delamination; one pier has exposed rebar. The FHWA Sufficiency Rating of the bridge is 38.9, which classifies the existing bridge as Structurally Deficient.

The proposed replacement structure consists of a 300-foot, 2-span steel girder bridge with a composite cast-in-place deck, founded on pile-supported integral abutments and a center pier supported on a spread footing on bedrock. Due to shallow bedrock at proposed Abutment No. 1, the H-piles will be installed in bedrock sockets to provide sufficient pile length to control moments in the piles, and to prevent translation of the pile tip. Piles at proposed Abutment No. 2 will be driven to bedrock. 1.75H:1V (horizontal:vertical) riprap slopes will be placed in front of the new integral abutments.

The new bridge will be located on the same horizontal alignment as the existing bridge, with a decreased skew. An increase in the clearance of the new bridge over Interstate 95 will result in a grade raise in the approaches up to 2 feet.

The existing bridge will be closed during construction and traffic detoured onto local roads.

2.0 GEOLOGIC SETTING

The existing structure carries Rice Rips Road (formerly Armstrong Road) over Interstate 95 as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of the Waterville Quadrangle, Maine, Open-File No. 16-8 (2016), indicates the surficial soils in the vicinity of the bridge project consist the Presumpscot Formation with nearby contact to glacial deposits. The Presumpscot Formation is generally comprised of silt, clay, sand, and minor amounts of gravel. This soil deposit typically overlies an irregular surface of glacial till and may include areas of till exposed at the ground surface.

The MGS Bulletin No. 20, Stratigraphy, Structural Geology, and Metamorphism of the Waterville-Vassalboro Area, Maine (June 1968), cites the bedrock at the project site as a heavy-bedded, light-grey, slightly calcareous Wacke (Sandstone) and grey Phyllite, belonging to the Mayflower Hill Formation. Bedrock cores retrieved at the site are identified as Phyllite, Siltstone, Sandstone, and Slate.

3.0 SUBSURFACE INVESTIGATION

Six test borings were drilled to explore subsurface conditions at the site in April 2019. Borings BB-WRRR-101 and BB-WRRR-102 were drilled directly behind the existing westerly abutment. Borings BB-WRRR-103 and BB-WRRR-104 were drilled through the bridge deck, at the location of the proposed center pier. Borings BB-WRRR-105 and BB-WRRR-106 were drilled behind the existing easterly abutment. 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 Sheets 4 and 5 – Boring Logs.

Borings were performed by using 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 June 2018. All N-values discussed in this report are corrected N-values computed by applying an average energy transfer of 0.928. The hammer efficiency factor (0.928) 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 MaineDOT geotechnical engineer selected the boring locations and drilling methods, designated type and depth of sampling techniques, logged the subsurface conditions encountered in the borings, 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 and rock samples recovered from the test borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the project site. Laboratory testing on soil samples consisted of six standard grain size analyses with natural water content. Laboratory testing on bedrock samples consisted of three unconfined compressive strength with elastic moduli (ASTM D7012, Method D).

Soil laboratory testing was performed at the MaineDOT Lab in Bangor, Maine. Rock testing was performed by GeoTesting Expresss, Inc. in Acton Massachusetts.

The results of soil and rock tests are included in Appendix C – Laboratory Test Results. Moisture content information and other soil and bedrock test results are also presented on the boring logs provided in Appendix A – Boring Logs and on Sheets 4 and 5 -Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings generally consisted of fill and glacial till. The fill unit and native soil deposits are underlain by Phyllite, Sandstone, Siltstone, and Slate 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

Armstrong Road Bridge was built in concert with the construction of Interstate 95 in the late 1950's. Armstrong Road (now known as Rice Rips Road) was raised approximately 5 to 15 feet so that the Interstate could underpass the road. Building the Interstate required cuts in the native glacial till soils up to 14 feet. The native soils excavated for the construction of Interstate 95 were used to partially backfill the existing bridge abutments.

Fill materials were encountered in the borings drilled behind the existing abutments. The thickness of the fill unit was approximately 14 to 19 feet at the boring locations and it consisted of granular fill and reworked native glacial till soils excavated for the construction of Interstate 95.

The granular fill generally consisted of:

- Brown, SAND, some silt, little gravel;
- Brown, GRAVEL, some sand, some silt;
- Brown, Silty SAND, little gravel.

The reworked glacial till fill unit generally consisted of:

- Brown, Sandy GRAVEL, some silt;
- Brown, Sandy SILT, trace gravel;
- Brown, SAND, little to some silt, some decomposed rock fragments and gravel, trace wood fragments.

Corrected SPT N-values in the fill unit ranged from 6 to 20 blows per foot (bpf) indicating the fill is loose to medium dense in consistency. Five grain size analyses performed on samples recovered from within the fill unit resulted in A-2-4 and A-1-b material classifications according to the AASHTO Soil Classification System and SM and GM classifications according to the Unified Soil Classification System (USCS). The natural water content of the samples tested ranged from approximately 8 to 15 percent.

5.2 Glacial Till

Glacial till was encountered beneath the fill materials in the abutment borings and at the ground surface at the proposed central pier. The thickness ranged from approximately 5 to 10 feet thick behind the proposed abutments and 7 to 8 feet thick at the proposed pier. The variable glacial till consisted of:

- Brown, Silty SAND, little decomposed rock;
- Grey, decomposed BEDROCK, some silt;
- Brown, SILT, and little decomposed bedrock;
- Olive to olive-brown, Sandy SILT, little gravel, weathered rock.

Corrected SPT N-values from successful SPT tests within the glacial till deposit ranged from 22 to greater than 100 bpf indicating the glacial till is medium dense to very dense or very stiff to hard in consistency. One grain size analysis performed on a sample recovered from the glacial till deposit resulted in an A-4 material classification according to the AASHTO Soil Classification System and SM classification according to the USCS. The natural water content of the sample tested was approximately 13 percent.

5.3 Bedrock

Bedrock was encountered and cored in the six 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-WRRR-101	48+10.8	7.5 Rt	19.1	258.9	45, 81
BB-WRRR-102	48+17.3	8.7 Lt	20.2	258.1	53, 68
BB-WRRR-103	49+70.8	9.9 Rt	7.0	250.1	83, 95
BB-WRRR-104	49+82.9	9.8 Lt	8.4	247.9	78, 90
BB-WRRR-105	51+37.5	6.8 Rt	30.4	244.2	83, 82
BB-WRRR-106	51+46.5	8.4 Lt	28.2	246.1	40, 19

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

Bedrock consisted of grey to dark grey, fine grained, hard, fresh to moderately weathered, Phyllite, Sandstone, Siltstone and Slate. Joints are close to moderately close, tight to open, with no infilling, horizontal to steep, and planar. The beds of SLATE exhibit near vertical faults filled with quartz and quartz veins, and the joints are open in silt infilled. The Rock Quality Designations (RQD) ranged from 19 to 95 percent, which indicates variable rock quality (very poor to excellent).

Detailed bedrock descriptions and RQD of each core run are provided on the boring logs in Appendix A and on Sheets 4 and 5 – Boring Logs. Rock core photographs are provided in Appendix B – Rock Core Photographs.

Unconfined compressive strength (UCS) testing was conducted on three samples of bedrock, the results of which are summarized in the following table.

Summary of Unconfined Compressive Strength Testing					
Boring	Depth below top of rock (ft)	Unconfined Compressive Strength (psi)	Youngs's Modulus, E (ksi)	Unit Weight (pcf)	Rock Type
BB-WRRR-101	20-20.38	10,577	7,650,000	164	Phyllite
BB-WRRR-104	9.29-9.67	5,236	5,810,000	171	Sandstone
BB-WRRR-105	30.43-30.81	7,047	10,700,000	171	Phyllite

Table 2 – Summary of Unconfined Compressive Strength Testing

5.4 Groundwater

Groundwater was measured at 9.0 feet bgs in boring BB-WRRR-101 and 15.0 ft bgs in BB-WRRR-105 during the subsurface investigation. No groundwater was observed in the other four borings. Note that water was introduced into the boreholes during drilling operations. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, and construction activities.

6.0 FOUNDATION ALTERNATIVES

6.1 Abutment Foundations

Integral abutments supported on steel H-piles and full height, cantilever abutments with spread footings on bedrock are considered feasible foundation alternates. Based on ease of construction and cost considerations related to the excavation and earthwork required for full height abutments, the preferred alternative is an integral abutment on steel H-piles. Piles at Abutment No. 1 will be fitted with a steel plate welded to the base of pile and grouted into the bedrock sockets. Piles at Abutment No. 2 will be driven to the required nominal resistance on or within bedrock.

6.2 Pier Foundation

The existing four pier foundations consist of spread footings bearing on bedrock. The center pier of the new 2-span bridge will be constructed in the median of Interstate 95, between the existing piers. Bedrock was encountered at depths ranging from approximately 7 to 8.4 feet below the ground surface at the location of the proposed center pier. Braced excavations of this depth are considered technically feasible in the Interstate median. A spread footing foundation bearing on sound, intact bedrock is the preferred foundation type at the center pier.

7.0 GEOTECHNICAL DESIGN CONSIDERATIONS AND RECOMMENDATIONS

The following sections provide geotechnical design considerations and recommendations for H-pile supported integral bridge abutments and a wall pier bearing on a spread footing on bedrock, which are the proposed substructure types for the Armstrong Road Bridge replacement project.

7.1 Integral Abutment H-Piles

Abutments No. 1 and 2 will be integral abutments founded on a single row of H-piles. Due to the relatively shallow bedrock surface, H-piles at Abutment No. 1 will be installed in bedrock sockets to provide sufficient pile length to control moments and prevent translation of the pile tips. Piles at Abutment No. 2 will be driven to the required nominal resistance on or within bedrock.

Piles may be HP 14x89 or 14x117 depending on the factored design axial loads and ability to resist lateral loads. H-piles shall be 50 ksi, Grade A572 steel. Piles at Abutment No. 1 will be fitted with a steel plate welded to the pile web and flanges at the base of pile and grouted into the bedrock sockets. Abutment No. 2 piles require driving pile points conforming to MaineDOT Standard Specification 711.10 to protect pile tips and improve penetration into bedrock.

Pile lengths at the proposed abutments may be estimated based on Table 3.

Abutment	Approximate Bottom Elevation of Proposed Abutment (feet)	Approximate Top of Bedrock Elevation (feet)	Approximate Tip Elevation in Socket (feet)	Estimated Pile Lengths (feet)
Abutment No. 1 ¹	269.0	258	254	17
Abutment No. 2	266.0	244	n.a.	24

Table 3 – Estimated Pile Lengths

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

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

- compressive axial geotechnical resistance of piles,
- drivability resistance of piles (Abutment No. 2 only),
- structural resistance of piles in axial compression, and
- structural resistance of piles in combined axial loading and flexure.

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

Per AASHTO LRFD Bridge Design Specifications 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.

¹ Pile length based on depth to top of bedrock plus 4 feet of pile length in 5-foot bedrock socket. The basis of the bedrock socket is to extend the drill hole to rock quality classified as “fair” or better.

7.2 Axial Pile Resistance – Strength Limit State

Abutment No. 1 – West Abutment

Due to the shallow depth to bedrock at Abutment No. 1, H-piles will be installed in bedrock sockets to provide sufficient pile length to control moments in the pile and to prevent translation of the pile tips. To satisfy end bearing resistance, a steel plate will be welded to the pile web and flanges at the pile tip to provide an increased bearing area.

Rock socket depths should be investigated with lateral pile analyses using the parameters in Section 7.4 of this report.

Structural Resistance. Preliminary estimates of the factored structural axial resistance of two H-pile sections were calculated for approximated upper and lower unbraced pile segments and for the lower braced pile segment. The controlling resistance shown in Table 4 is for the lower braced pile segment, using a resistance factor, $\phi_c = 0.50$ for severe driving conditions. 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. *The structural capacity of the end plate also may govern the design.*

Geotechnical Resistance. The grouted rock socket detail will include a steel plate welded to the web and flanges of the pile tip and will be designed such that grout can be reliably placed below and around the pile tip and promote full, uniform load transfer to end bearing in bedrock. For this condition, the end bearing resistance was calculated as it would be for a drilled shaft bearing in bedrock. Using LRFD equation 10.8.3.5.4C-1 and an unconfined compressive strength of 4 ksi yields a nominal resistance of 10 ksi. Assuming a nominal bearing plate area of 144 square inches, less the area for a tremie pipe, the nominal geotechnical axial compressive resistance is 1,410 kips. The resulting factored static geotechnical resistances for two pile sections with steel end plates are provided in Table 4. For practical purposes we recommend a 14-inch by 14-inch square based plate with a tremie tube extending through and below the base plate.

A summary of the calculated factored axial compressive structural and geotechnical axial compressive resistances of the H-pile sections at the strength limit states are provided in Table 4 below.

Strength Limit State Factored Axial Pile Resistance – Abutment No. 1			
Pile Section	Structural Resistance ² $\phi_c=0.50$ (kips)	Static Geotechnical Resistance $\phi_{static} = 0.50^3$ (kips)	Governing Axial Pile Resistance (kips)
HP 14 x 89	652	704	652
HP 14 x 117	860	704	704

Table 4– Factored Axial Compressive Resistances for Strength Limit States – H-Piles with Steel End Plates Installed in Bedrock Sockets at Abutment No. 1

For the 14x89 pile modeled with a plate at the pile end, the factored structural axial compressive pile resistance is less than the factored geotechnical axial compressive resistance. For the 14x117 pile modeled with a plate at the pile end, the factored geotechnical axial compressive pile resistance is less than the factored structural axial compressive resistances. Therefore, the controlling and the recommended governing resistances for the Abutment No. 1 pile design are the resistances provided in the rightmost column “Governing Axial Pile Resistance (kips)” in Table 4.

The maximum factored axial pile load at Abutment No. 1 should not exceed the governing factored axial pile resistance shown in Table 4. In no instance should the H-piles with plates be loaded beyond their structural capacity. This evaluation is the responsibility of the structural engineer.

Abutment No. 2 – East Abutment

Structural Resistance. Preliminary estimates of the factored structural axial resistance of two H-pile sections were calculated for approximated upper and lower unbraced pile segments and for the lower braced pile segment. The controlling resistance shown in Table 5 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 5 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

² Structural resistances were calculated for a braced pile segment in pure axial compression, using a resistance factor, ϕ_c , for severe driving conditions. Factored structural resistances should be calculated for upper and lower unbraced pile segments and K-values based upon L-Pile results using a resistance factor of $\phi_c = 0.70$ for combined axial loading and bending. These resistances may be the controlling values.

³ Static geotechnical resistance was estimated using LRFD equation 10.8.3.5.4C-1 and an unconfined compressive strength of 4 ksi yielding a nominal resistance of 10 ksi; H-pile sections are modeled with a 12-inch by 12-inch steel plate welded across the pile tip for an increased bearing area.

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 L-Pile.

Geotechnical Resistance. The nominal axial geotechnical resistance of driven piles at the strength limit state was calculated using the guidance in LRFD Article 10.7.3.2.3, which states the nominal bearing resistance of piles driven to point bearing on hard rock shall not exceed the nominal structural pile resistances obtained from LRFD Article 6.9.4.1 with a resistance factor ϕ_c , of 0.50, for severe driving conditions applied. The resulting limiting factored geotechnical compressive resistances for piles are provided in Table 5.

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 driven H-piles at Abutment No 2, at the strength limit states for are provided in Table 5.

Strength Limit State Factored Axial Pile Resistance – Abutment No. 2				
Pile Section	Structural Resistance ⁴ $\phi_c=0.50$ (kips)	Controlling Geotechnical Resistance ⁵ $\phi_c=0.50$ (kips)	Drivability Resistance ⁶ $\phi_{dyn} = 0.65$ (kips)	Governing Axial Pile Resistance (kips)
HP 14 x 89	652	566	409	409
HP 14 x 117	860	749	481 (546) ⁵	481 (546) ⁷

Table 5– Factored Axial Compressive Resistances for Strength Limit States - Driven H-Piles End Bearing on Bedrock at Abutment No. 2

LRFD Article 10.7.3.2.3 states that the nominal axial compressive resistance of piles driven to hard rock is typically controlled by the structural resistance with a resistance factor for severe driving conditions applied. However, for the site conditions, the estimated factored axial pile resistances from the drivability analyses for the H-pile sections are less than the controlling factored axial compressive resistances. Local experience also supports the estimated factored resistances from the drivability analyses. Therefore, drivability controls and the recommended governing resistances for pile design are the resistances provided in the rightmost column “Governing Axial Pile Resistance (kips)” in Table 5.

The maximum applied factored axial pile load should not exceed the governing factored axial pile resistance shown in Table 5.

⁴ Structural resistances were calculated for a braced pile segment in pure axial compression, using a resistance factor, ϕ_c , for severe driving conditions. Factored structural resistances should be calculated for upper and lower unbraced pile segments based upon L-Pile results using a resistance factor of $\phi_c = 0.70$ for combined axial loading and bending. These resistances may be the controlling values.

⁵ Based on guidance in LRFD Article 10.7.3.2.3., *Piles Driven to Hard Rock*. The nominal axial geotechnical resistance in the strength limit state was calculated using the guidance in LRFD Article 10.7.3.2.3 which states the nominal bearing resistance of piles driven to point bearing on hard rock shall not exceed the nominal structural resistance values obtained from LRFD Article 6.9.4.1 with a resistance factor ϕ_c , of 0.50, for severe driving conditions applied.

⁶ Drivability analyses were performed to determine the pile resistance that might be achieved at Abutment No. 2 considering available diesel hammers. Nominal drivability resistances were determined based on a maximum driving stress of 45 ksi and a limiting driving criteria of 12 bpi (due to a slight risk of piles walking out of position before reaching the maximum allowable driving criteria of 15 bpi). 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.

⁷ Drivability resistance based on a Delmag D19-42. Drivability resistance with a Delmag D36-32 shown in parentheses.

7.3 Axial Pile Resistance – Service and Extreme Limit State

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

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

Abutment No. 1 – West Abutment

The calculated factored axial structural and geotechnical resistances, at the service and extreme limit state, of two H-pile sections modeled with a plate at the end, are provided in Table 6.

Service and Extreme Limit State Factored Axial Pile Resistance – Abutment No. 1			
Pile Section	Structural Resistance ⁸ $\phi = 1.0$ (kips)	Static Geotechnical Resistance ⁹ $\phi = 1.0$ (kips)	Governing Axial Pile Resistance $\phi = 1.0$ (kips)
HP 14 x 89	1,131	1,410	1,131
HP 14 x 117	1,498	1,410	1,410

Table 6– Factored Axial Compressive Resistances for Service and Extreme Limit States – H-Piles with Steel End Plates Installed in Bedrock Sockets – Abutment No. 1

The recommended governing resistances for extreme and service limit state pile design are the resistances provided in the rightmost column “Governing Axial Pile Resistance (kips)” in Table 6. The maximum applied factored axial pile load for the service and extreme limit states shall not exceed the governing factored axial pile resistance shown in Table 6.

⁸ Nominal structural resistances were calculated for upper and lower pile segments using approximated unbraced lengths and for a braced pile segment in pure axial compression. Nominal structural resistances should be recalculated for upper and lower unbraced pile segments in combined axial loading and bending. These resistances may be the controlling values.

⁹ Static geotechnical resistance was estimated using LRFD equation 10.8.3.5.4C-1 and an unconfined compressive strength of 4 ksi yielding a nominal resistance of 10 ksi; H-pile sections are modeled with a 12-inch by 12-inch steel plate welded across the pile tip for an increased bearing area.

Abutment No. 2 – East Abutment

The calculated factored axial structural, geotechnical and drivability resistances of two (2) H-pile sections for the service and extreme limit states are provided in Table 7.

Service and Extreme Limit State Factored Axial Pile Resistance				
Pile Section	Structural Resistance ¹⁰ $\phi = 1.0$ (kips)	Controlling Geotechnical Resistance ¹¹ $\phi = 1.0$ (kips)	Drivability Resistance ¹² $\phi = 1.0$ (kips)	Governing Axial Pile Resistance (kips)
HP 14 x 89	1,131	1,131	630	630
HP 14 x 117	1,498	1,498	740 (840) ⁵	740 (840) ¹³

Table 7– Factored Axial Compressive Resistances at Strength Limit States –
 Driven H-Piles End Bearing on Bedrock at Abutment No. 2

LRFD Article 10.7.3.2.3 states that the nominal axial compressive resistance of piles driven to hard rock is typically controlled by the structural resistance. However, the estimated factored axial pile resistances from the drivability analyses for the H-pile sections are less than the controlling factored axial geotechnical resistance and the structural resistance calculated for a braced pile segment. Therefore, drivability controls and the recommended governing resistances for pile design are the resistances provided in the rightmost column “Governing Axial Pile Resistance (kips)” in Table 7.

The maximum applied factored axial pile load for the service and extreme limit states shall not exceed the governing factored axial pile resistance shown in Table 7.

¹⁰ Nominal structural resistances were calculated for upper and lower pile segments using an approximated unbraced length and a braced pile segment in pure axial compression. Factored structural resistances should be recalculated for upper and lower unbraced pile segments in combined axial loading and bending, based on LPile results. These resistances may be the controlling values.

¹¹ Based on guidance in LRFD Article 10.7.3.2.3., *Piles Driven to Hard Rock*. The nominal axial geotechnical resistance in the strength limit state was calculated using the guidance in LRFD Article 10.7.3.2.3 which states the nominal bearing resistance of piles driven to point bearing on hard rock shall not exceed the nominal structural resistance values obtained from LRFD Article 6.9.4.1

¹² Drivability analyses were performed to determine the pile resistance that might be achieved at Abutment No. 2 considering available diesel hammers. Nominal drivability resistances were determined based on a limiting driving criteria of 12 bpi and a maximum driving stress of 45 ksi. These theoretical pile resistances may not be achievable if piles walk out of position before reaching the specified driving criteria.

7.4 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.

Recommended geotechnical parameters for generation of soil-resistance (p-y) curves in lateral pile analyses are provided in Tables 8 and 9. The models developed should emulate appropriate structural parameters and pile-head boundary conditions for the pile section(s) being analyzed.

LPile Input Parameters Abutment No. 1 (West) – Borings BB-WRRR-101, BB-WRRR-102						
Soil Layer	Soil/Rock Model	Top Elevation of Layer (ft)	Layer Thickness (ft)	γ_e^{24} (pcf)	ϕ^{25} (deg) / S_u^{26} (psf) / UC (psi)	k_s^{27} (pci) / ϵ_{50}^{28} / E_m^{29} (psi) / k_m^{30} / RQD (%)
Granular Borrow	Reese Sand	278	9	125	32°	225
Granular Fill	Reese Sand	269	6	73	32°	60
Glacial Till	Reese Sand	263	5	90	43°	125
Aggregate-filled Rock Socket	Reese Sand	258	2	63	34°	125
Grout-filled Rock Socket	Strong Rock (Vuggy Limestone)	256	3	88	4000 psi	–

Table 8 – Soil Parameters for Generation of Soil-Resistance (p-y) Curves for Abutment 1

²⁴ Effective Unit Weight.

²⁵ Effective Internal angle of friction.

²⁶ Undrained shear strength.

²⁷ Soil modulus constant.

²⁸ Strain at 50 percent of the ultimate stress.

²⁹ Initial Modulus of Rock Mass

³⁰ Strain Factor (for rock)

LPile Input Parameters Abutment No. 2 (East) – Borings BB-WRRR-105, BB-WRRR-106						
Soil Layer	Soil/Rock Model	Top Elevation of Layer (ft)	Layer Thickness (ft)	γ_e^{31} (pcf)	ϕ^{32} (deg) / S_u^{33} (psf) / UC (psi)	k_s^{34} (pci) / ϵ_{50}^{35} / E_m^{36} (psi) / k_m^{37} / RQD (%)
Granular Borrow	Reese Sand	276	10	125	32°	225
Fill	Reese Sand	266	4.7	62.3	33°	60
Fill (Stiff Silt)	Reese Sand	261.3	5	72.6	31°	90
Glacial Till (Hard Silt)	Reese Sand	256	10	88.6	43°	90

Table 9 – Soil Parameters for Generation of Soil-Resistance (p-y) Curves for Abutment 2

7.5 Driven Pile Quality Control

The contract plans shall require the contractor to perform a wave equation analysis of the proposed pile-hammer system and conduct dynamic pile load tests with signal matching at Abutment No. 2. The first pile driven 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 restrrike tests will be required to verify time-dependent loss of pile resistance does not occur. If a loss in pile resistance does occur, the driving criteria shall be adjusted. Restrikes or additional dynamic tests may be required as part of the pile field quality control program should pile behavior vary radically between adjacent piles, should 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.

³¹ Effective Unit Weight.

³² Effective Internal angle of friction.

³³ Undrained shear strength.

³⁴ Soil modulus constant.

³⁵ Strain at 50 percent of the ultimate stress.

³⁶ Initial Modulus of Rock Mass

³⁷ Strain Factor (for rock)

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.

7.6 Integral Abutment and Wingwall Design

Integral abutment sections shall be designed for all relevant strength, service, and extreme limit states and load combinations specified in LRFD Articles 3.4.1 and 11.5.5. A resistance factor (ϕ) of 1.0 shall be used to assess abutment design at the service limit state, including: settlement and excessive horizontal movement. The overall stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, ϕ , of 0.65. Resistance factors for extreme limit state shall be taken as 1.0.

The designer may assume Soil Type 4 (MaineDOT Bridge Design Guide (BDG) Section 3.6.1) for abutment backfill material soil properties. The backfill properties are as follows:

- Internal Friction Angle (ϕ) = 32°
- Total Unit Weight (γ) = 125 pcf
- Soil-Concrete Interface Friction Angle (δ) = 20°

Integral abutments and in-line wingwalls shall be designed to withstand a lateral earth load equal to the passive pressure state. Calculation of passive earth pressures should assume a passive earth pressure coefficient, K_p , of 6.02, assuming a ratio of lateral abutment movement to abutment height (y/H) of approximately 0.020 and a level backfill. If the calculated ratio of lateral movement to wall height is less than or greater than 0.020, a passive earth pressure coefficient can be estimated using Figure 3.10.8-1 from the MassDOT LRFD Bridge Design Manual. This figure is reproduced in Appendix D – Calculations. A load factor for passive earth pressure is not specified in LRFD. For purposes of the integral abutment backwall reinforcing steel design, use a maximum load factor (γ_{EH}) of 1.50 to calculate factored passive earth pressures.

Additional lateral earth pressure due to live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for abutments if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination of the surcharge load, is permitted per LRFD Article 3.11.6.5. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 10, below:

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

Table 10 – Equivalent Height of Soil for Estimating Live Load Surcharge on Abutments

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

7.7 Pier Foundation

The center pier for the new 2-span bridge will be constructed in the median of Interstate-95, between the two existing center piers. Bedrock was encountered at depths ranging from approximately 7 to 8.4 feet below the ground surface at the location of the proposed center pier. Braced excavations of this depth are considered technically feasible in the interstate median. A spread footing foundation bearing on sound, intact bedrock is the preferred pier foundation type.

The borings indicate that bedrock with an RQD of approximately 78 to 83 percent will be encountered at the bedrock surface at the pier location. The approximate bedrock surface is estimated to range from approximate El. 247 to 250. The thickness of the footing may be designed to vary in thickness to accommodate variations in the bedrock surface while maintaining a constant top of footing elevation shown on the final plans.

7.7.1 Pier Spread Footing Design

The pier spread footing shall be designed for all relevant strength, service and extreme limit state load combinations specified in AASHTO LRFD Articles 3.4.1 and 11.5.6.

The pier spread footings shall be investigated at the Strength and Extreme Limit states for:

- bearing resistance failure,
- eccentricity,
- lateral sliding,
- structural failure, and,
- overall stability.

Piers constructed within 30 feet of the edge of the roadway must be protected from vehicular collision with barriers as specified in LRFD Article 3.6.5.1. If barriers are not used, the pier shall be designed for an equivalent static force of 600 kips, assumed to act in a direction of 0

to 15 degrees with the edge of the pavement in a horizontal plane at a distance of 5 feet above the ground.

Pier Footing Bearing Resistance

Nominal and factored bearing resistances were calculated using a Rock Mass Rating (RMR)-based empirical correlation³⁸ in accordance with LRFD Articles C10.4.6.4 and 10.6.2.5.1. Whereas the current edition of LRFD does not include the RMR formulation, RMR was evaluated in accordance with Table 10.4.6.4-1 of the 2012 LRFD code.

Based on the available data and the RMR-based methodology, the calculated factored bearing resistance at the strength limit state is 31 ksf, assuming a resistance factor, ϕ_b , for spread footings on bedrock of 0.45. From a practical perspective we recommend that the footings have a minimum width of 3 feet. In no instance shall the bearing stress exceed the nominal structural resistance of the structural concrete which may be taken as $0.3f'c$.

At the service limit state, the footing should be designed for a maximum bearing resistance of 30 ksf. A service limit state bearing resistance 30 ksf should also be used for preliminary footing sizing as allowed in LRFD C10.6.2.1.

Factored Bearing Resistances for Pier Spread Footing					
Substructure	Assumed Bearing Material	Limit State	Resistance Factor, ϕ_b	Factored Bearing Resistance (ksf)	LRFD Reference
Center Pier	Sound, Intact Bedrock	Service	1.0	30	Article 10.5.5.1
		Strength	0.45	31	Table 10.5.5.2.2-1
		Extreme	0.8	54	Article C11.5.8

Table 11 – Factored Bearing Resistance of Pier Footing

The vertical bearing stress shall be calculated assuming a triangular or trapezoidal pressure distribution over an effective base as shown in LRFD Figure 11.6.3.2-2 for foundations on rock.

LRFD Article 10.6.2.4.4 indicates that footings bearing on rock with an RMR-based rock quality of Fair or better, and designed using LRFD methods, are generally anticipated to experience ½ inch or less of elastic settlement.

³⁸ “Foundations on Rock” by Duncan Wyllie.

Eccentricity

For spread footings on rock, the eccentricity of the resultant loads at the strength limit state shall not exceed 0.45 of the footing dimensions, in either direction. This eccentricity corresponds to the resultant of reaction forces falling within the middle nine-tenths (9/10) of the footing.

Eccentricity Requirements			
Substructure	Bearing Material	Location of Resultant Forces	LRFD Reference
Center Pier	Bedrock	Within the middle nine-tenths (9/10) of the base width	Article 11.6.3.3

Table 12 – Eccentricity Requirements

Sliding

For sliding analyses, a sliding resistance factor, ϕ_τ , of 0.80 shall be applied to the nominal sliding resistance of cast-in-place spread footings constructed on bedrock. Passive resistance shall be neglected in sliding and stability calculations.

Sliding Resistance Factors					
Substructure	Assumed Bearing Material	Condition	Limit State	Sliding Resistance Factor ϕ_τ	LRFD Reference
Center Pier	Bedrock	Cast-in-place concrete placed in dewatered cofferdam	Strength	0.80	Table 10.5.5.2.2-1
			Service	1.0	Article 10.5.5.1
			Extreme	1.0	Article 10.5.5.3

Table 13 – Sliding Resistance Factor Requirements

Assuming that the rock subgrade will be **prepared in-the-dry**, sliding computations for resistance of footings to lateral loads shall assume a maximum sliding resistance coefficient of 0.70 at the bedrock-to-concrete interface assuming a level bedrock surface. Refer to Table 12, below.

Substructure	Interface Materials	Limit States	Friction Angle, δ	Coefficient of Friction, $\tan \delta$ (dim.)	C ¹	LRFD Reference
Center Pier	Cast-in-place concrete to level, sound bedrock	All	35°	0.70	1.0	Table C3.11.5.3-1

1 Per LRFD Article 10.6.3.4 and Equation 10.6.3.4-2, C=1.0 for cast-in-place footings.

Table 14 – Sliding Coefficient of Friction

Overall Stability

For the service limit state, a resistance factor, ϕ , of 1.0 shall be used to assess spread footing design for settlement, horizontal movement, bearing resistance, sliding and eccentricity. The overall global stability of foundations is typically investigated at the Service I Load Combination and a resistance factor, ϕ , of 0.65. However, we do not anticipate shear failure along adversely oriented joint surfaces in the bedrock mass below the foundation.

7.8 Settlement and Embankment Stability

An increase in the clearance of the new bridge over Interstate 95 will result in a grade raise in the approach embankments up to 2 feet. The existing bridge approach embankments include loose fills. A portion of the loose fill material will be excavated to the subgrade elevation of the abutments/pile caps. However, approximately 2 feet of loose sand was encountered the below the pile cap subgrade for Abutment 1. To mitigate long term settlement, we recommend over-excavation to El. 266 at Abutment 1 and replacement with compacted granular borrow. Construction loads could introduce elastic settlements; however, these settlements will occur relatively quickly.

The bridge approach embankments will be constructed using granular borrow placed over medium dense and stiff embankment fills, dense glacial till and hard, silty glacial till. Conventional earth fill embankments constructed over dense and stiff soils, using MaineDOT Standard Specifications, with side slopes of 2H:1V or flatter, are expected to satisfy stability requirements. Slopes steeper than 2H:1V should be treated with riprap using MaineDOT standard details.

Settlement of the steel H-piles bearing on bedrock will be limited to elastic compression of the piles and is anticipated to be minimal.

7.9 Frost Protection

Foundations placed on soil should be designed with an appropriate embedment for frost protection. According to MaineDOT BDG Figure 5-1, Maine Design Freezing Index Map, Waterville has a design freezing index (DFI) of approximately 1600 F-degree days. The anticipated coarse-grained fill material was assigned a water content of 15%. These components correlate to a frost depth of 6.5 feet. It is recommended that any foundation bearing on soils be embedded 6.5 feet for frost protection.

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

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

7.10 Seismic Design Considerations

The United States Geological Survey Seismic Design CD (Version 2.1) provided with the 2014 LRFD Code (7th Edition), and LRFD Articles 3.10.3.1 and 3.10.6 were used to develop parameters for seismic design. Based on site coordinates, the software provided the recommended AASHTO Response Spectra for a 7 percent probability of exceedance in 75 years. These results are summarized in Table 15:

Parameter	Design Value
Peak Ground Acceleration (PGA)	0.077g
Acceleration Coefficient (A_S)	0.122g
S_{DS} (Period = 0.2 sec)	0.256g
S_{D1} (Period = 1.0 sec)	0.111g
Site Class	D
Seismic Zone	1

Table 15 – Seismic Design Parameters

In conformance with LRFD Table 4.7.4.3-1 seismic analysis is not required for multispan bridges in Seismic Zone 1. However, superstructure connections and minimum support length requirements shall be designed per LRFD Articles 3.10.9.2 and 4.7.4.4, respectively.

8.0 CONSTRUCTION RECOMMENDATIONS AND CONSIDERATIONS

8.1 Excavation and Dewatering

The new abutments will be constructed in the approximate location of the existing abutments and will require pile driving. The contractor shall be responsible for excavating the existing substructures and concrete approach slabs in their entirety. Boulders and cobbles that conflict with the proposed piles should be removed by conventional excavation methods, pre-augering, predrilling, spudding, use of rock chisels, or down-hole hammers.

Any loose, soft or unsuitable soil encountered at the subgrade level for both abutments shall be excavated in its entirety and replaced with Granular Borrow – Material for Underwater Backfill and the exposed subgrade then thoroughly compacted.

Excavation for the abutments is anticipated to be accomplished using sloped open cut methods in accordance with MaineDOT and OSHA requirements. Excavations will expose soils that may become saturated and water seepage may occur during construction. There may be localized sloughing and instability in some excavations and cut slopes. The contractor should control groundwater, surface water infiltration, and soil erosion. Water should be controlled by pumping from sumps.

8.2 Pier Footing Excavation and Subgrade Preparation

Excavation for the pier foundation will extend approximately 8 feet below the existing grade. An internally braced cofferdam will be required to permit construction of the spread footing in-the-dry. It is anticipated that there will be seepage of water from fractures and joints exposed in the bedrock surface. Surface water should be diverted from the pier excavation throughout the period of construction. Water encountered at the base of the foundation excavation should be removed by using a sump pump located in the corner of the excavation outside of the foundation footprint. The contractor should maintain the excavation so that pier footing concrete is placed in-the-dry.

Sliding parameters provided in this report assume that the pier footing concrete is cast in-the-dry.

The subgrade for the pier footing shall consist of sound bedrock. The nature, slope and degree of fracturing in the bedrock bearing surface will not be evident until the foundation excavation is made. The bedrock subgrade surface shall be cleaned of all overburden soils and loose, dislodged bedrock fragments by mechanical means. Mechanical means include expansive agents, hydraulic hoe ram, hydraulic splitters or wedging and prying.

The slope of the bedrock subgrade shall be no steeper than 4H:1V or it shall be benched to create level steps or excavated to be completely level. Anchoring or doweling the footing to bedrock may be utilized to improve sliding resistance where the prepared bedrock surface is steeper than 4H:1V. Dowels should be #9 reinforcing bars or larger and be embedded into the footings and bedrock by depths determined by the engineer.

The final bearing surface of bedrock shall be washed with high pressure water and air prior to concrete being placed for the pier foundation.

The final bedrock surface shall be approved by the Resident prior to placement of the footing concrete.

The borings indicate that the site bedrock is hard with RQD's that classify the rock quality as Good to Excellent. Any necessary excavation of bedrock material may be done using

conventional excavation methods and mechanical means but may require drilling and blasting techniques. Blasting should be conducted in accordance with Section 105.2.7 of the MaineDOT Standard Specifications. It is also recommended that the contractor conduct pre- and post-blast surveys, at the nearby residences, and bridge structures in accordance with industry standards at the time of the blast. Construction activities should not be permitted to disturb the bedrock mass or create any open fissures.

8.3 Pile Installation

Abutment No. 1 (West Abutment)

The Abutment No. 1 piles will be installed in pre-drilled holes extending from the abutment/pile cap subgrade to the bottom of the proposed bedrock sockets:

- Bedrock sockets should be drilled through the overburden using temporary casing that is seated in the top of rock prior to drilling the sockets.
- For preliminary design, a 5-foot deep bedrock socket can be assumed. The bedrock socket shall be backfilled with grout 1-foot below and 2 feet above the pile base plate.
- Rock sockets should be at least 26-inches in diameter and cleaned of all loose material using an airlift or vacuum truck. The socket based should be inspected using a camera and sounding rod or weighted tape, prior to placing the pile.
- Piles should be equipped with centralizers, base plates and tremie tubes prior to installation. A pile template or frame should be used to positively establish the pile top location.
- The grout tremie tube should extend at least 3-inches below the base plate and consist of black steel. The upper portion of the tube may be steel or PVC tubing.
- The piles should be suspended from the top of the casing so that the base plate is approximately 1-foot above the base of the rock socket, then tremie grouted up to 2 feet above the plate with minimum 6,000 psi grout. The grout should be allowed to set for at least 48 hours before backfilling the temporary casing with stone and removing the casing.
- Stone consisting of 703.22 Underdrain Backfill Material Type C or pea stone should be used to backfill the rock socket and temporary casing.
- Tight clearance in the tremie pile and a congested end plate detail preclude the use of a crushed ledge aggregate, which compacts or “knits” together and is less workable. The grout mix aggregate should be a rounded gravel pea stone passing the 3/8-inch sieve. The grout mix should contain a non-shrink expansive additive, equivalent to CTS Komponent.

Abutment No. 2 (East Abutment)

Abutment No. 2 piles require cast steel driving pile points conforming to MaineDOT Standard Specification 711.10 to protect pile tips and improve penetration into bedrock.

The contract plans shall require the contractor to perform a wave equation analysis of the proposed pile driving hammer system. Driving stresses predicted by the wave equation analysis shall be less than 45 ksi. 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).

The first pile driven should be dynamically tested, with signal matching, to confirm the nominal pile resistance and verify the stopping criteria developed by the contractor in the wave equation analysis. A minimum 24-hour restrike test will be required to verify time-dependent loss of pile resistance does not occur. If a loss in pile resistance does occur, the driving criteria shall be adjusted. Additional dynamic tests may be required as part of the pile field quality control program should pile behavior vary radically between adjacent piles, should a pile “refuse” on a boulder or cobble above bedrock, should the pile tip be not firmly embedded in bedrock, or if piles “walk” out of position.

8.4 Abutment Sections

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

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).

9.0 CLOSURE

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

In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the

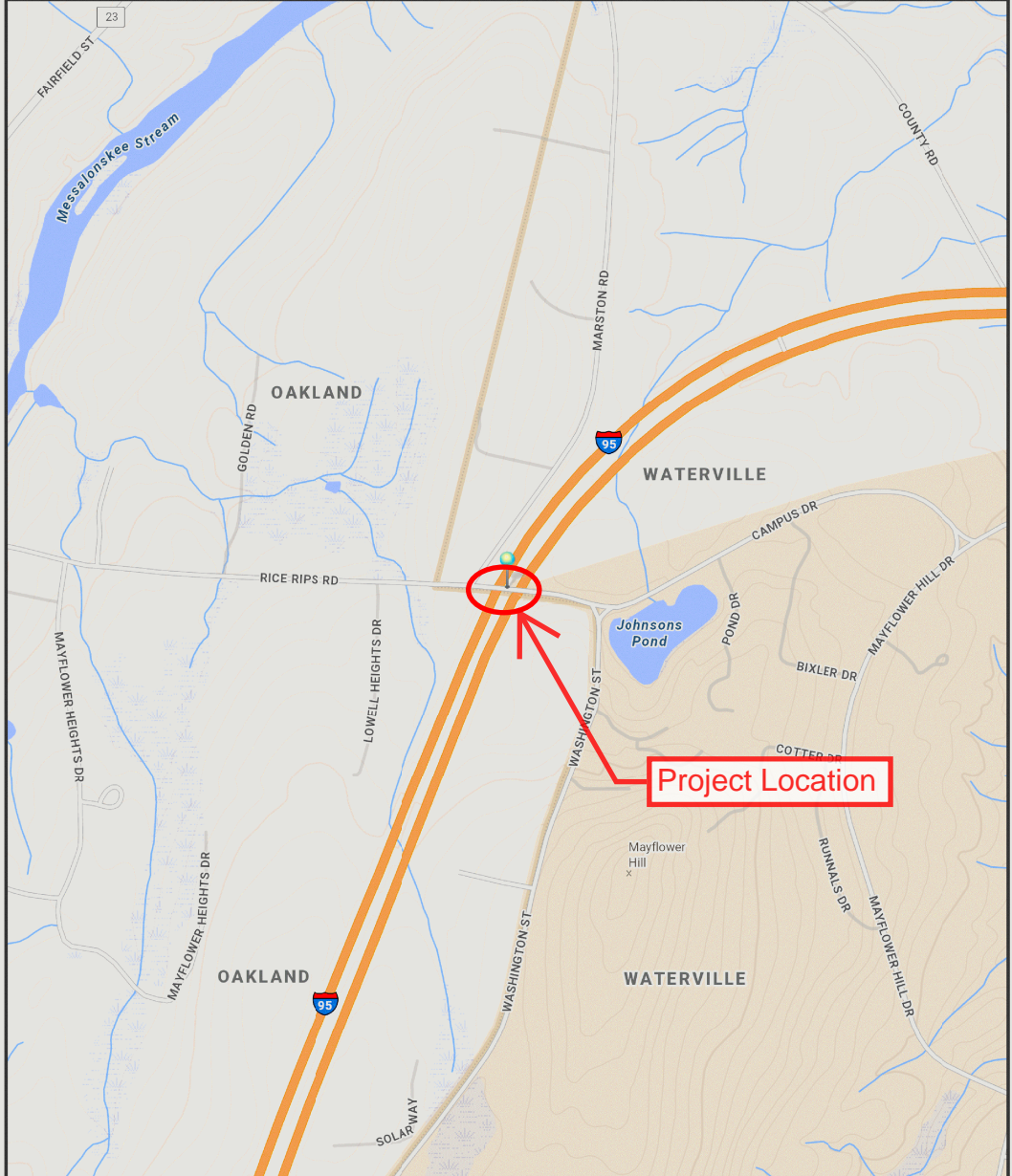
recommendations as appropriate to reflect the changes in design. These analyses and recommendations are based in part upon limited subsurface investigations at discrete exploratory locations completed at the site. If variations from the conditions encountered during the investigation appear evident during construction, it may also become necessary to re-evaluate the recommendations made in this report.

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

Sheets



WATERVILLE, MAINE



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0.2 Miles
1 inch = 0.23 miles

Date: 9/14/2020
Time: 2:17:52 PM

SHEET NUMBER

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OF 5

ARMSTRONG ROAD BRIDGE
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LOCATION MAP

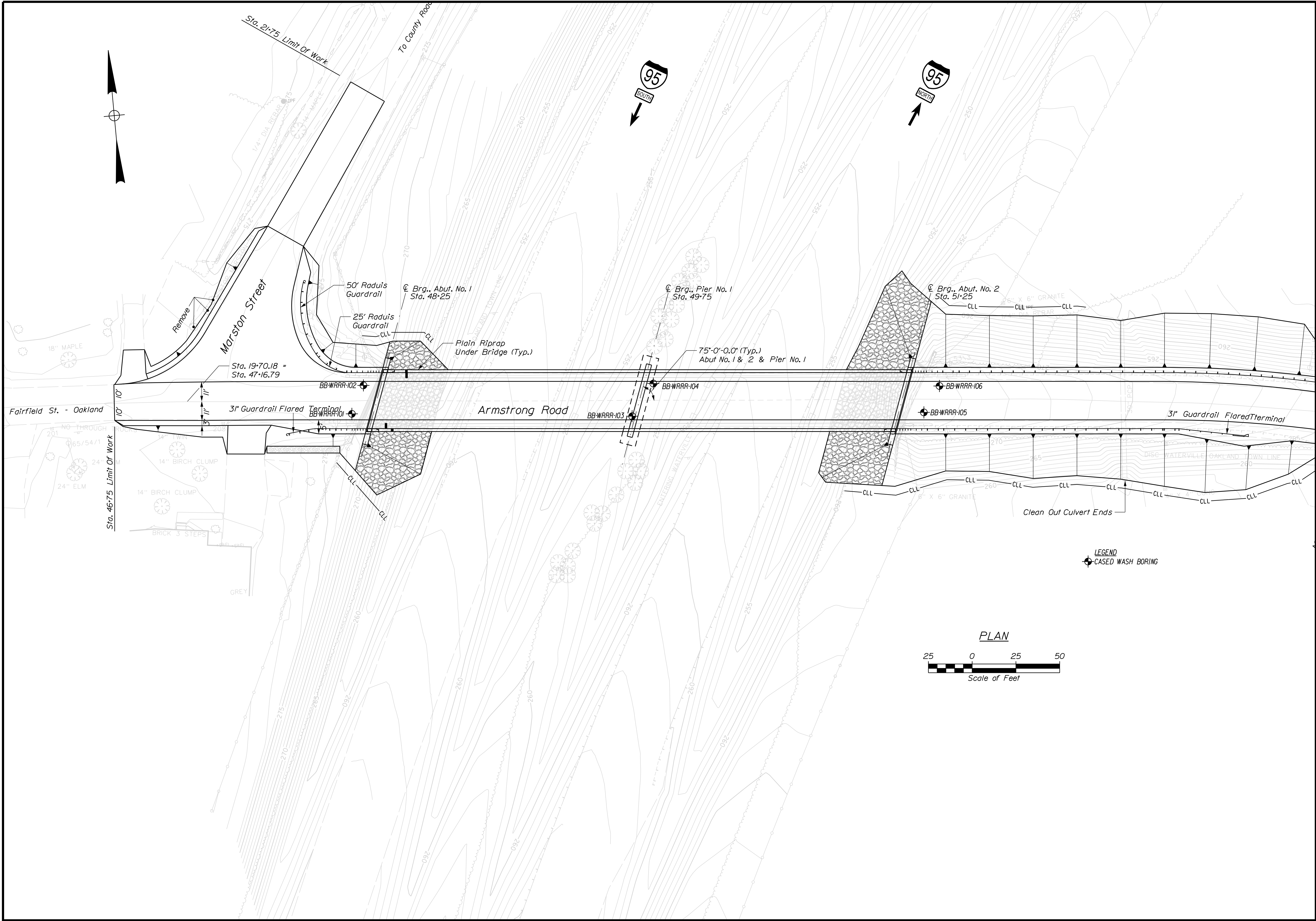
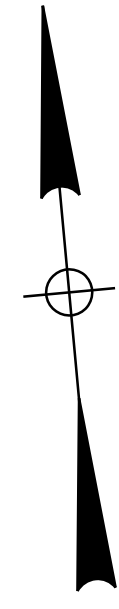
STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

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WIN

23559.00

BRIDGE PLANS



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
2350500
WIN
23505.00
BRIDGE NO. 6815
BRIDGE PLANS

PROJ. MANAGER	D. Eaton	DATE	SIGNATURE
DESIGN/DETAILED	Kendra N.	Mar 2019	
CHECKED/REVIEWED	L. KRUSINSKI	SEP 2020	
DESIGN/DETAILED	T. WHITE		
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

ARMSTRONG ROAD BRIDGE
OVER INTERSTATE - 95
WATERVILLE
KENNEBEC COUNTY
BORING LOCATION PLAN

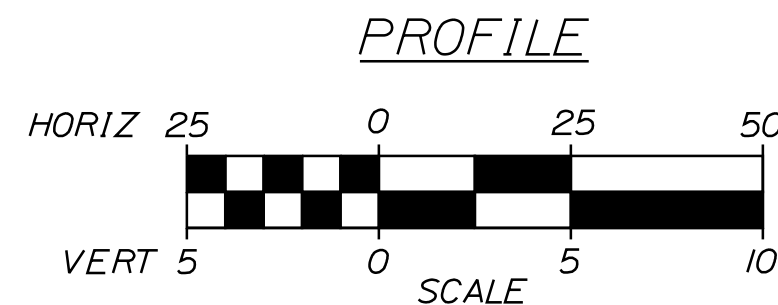
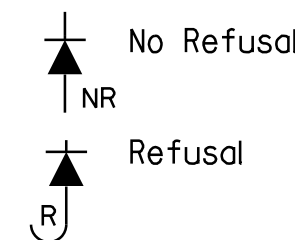
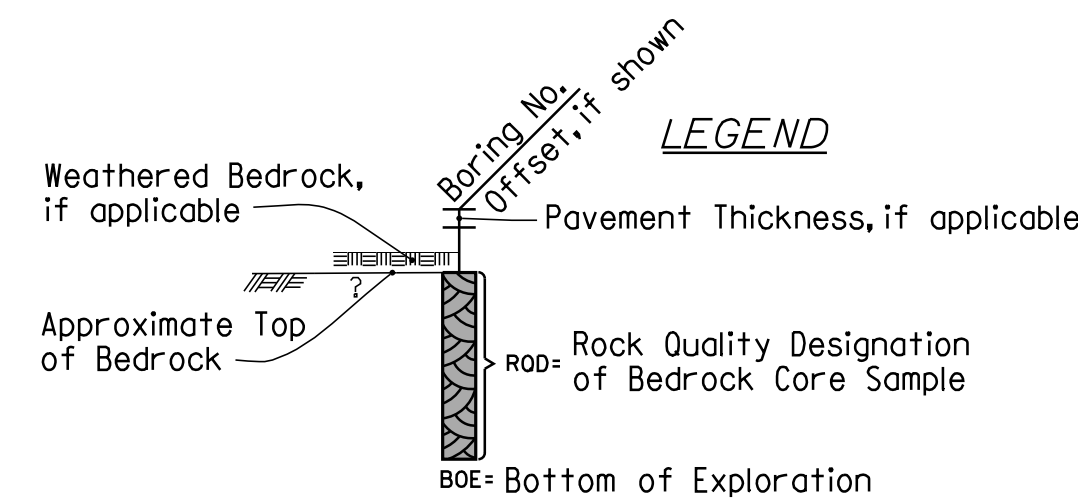
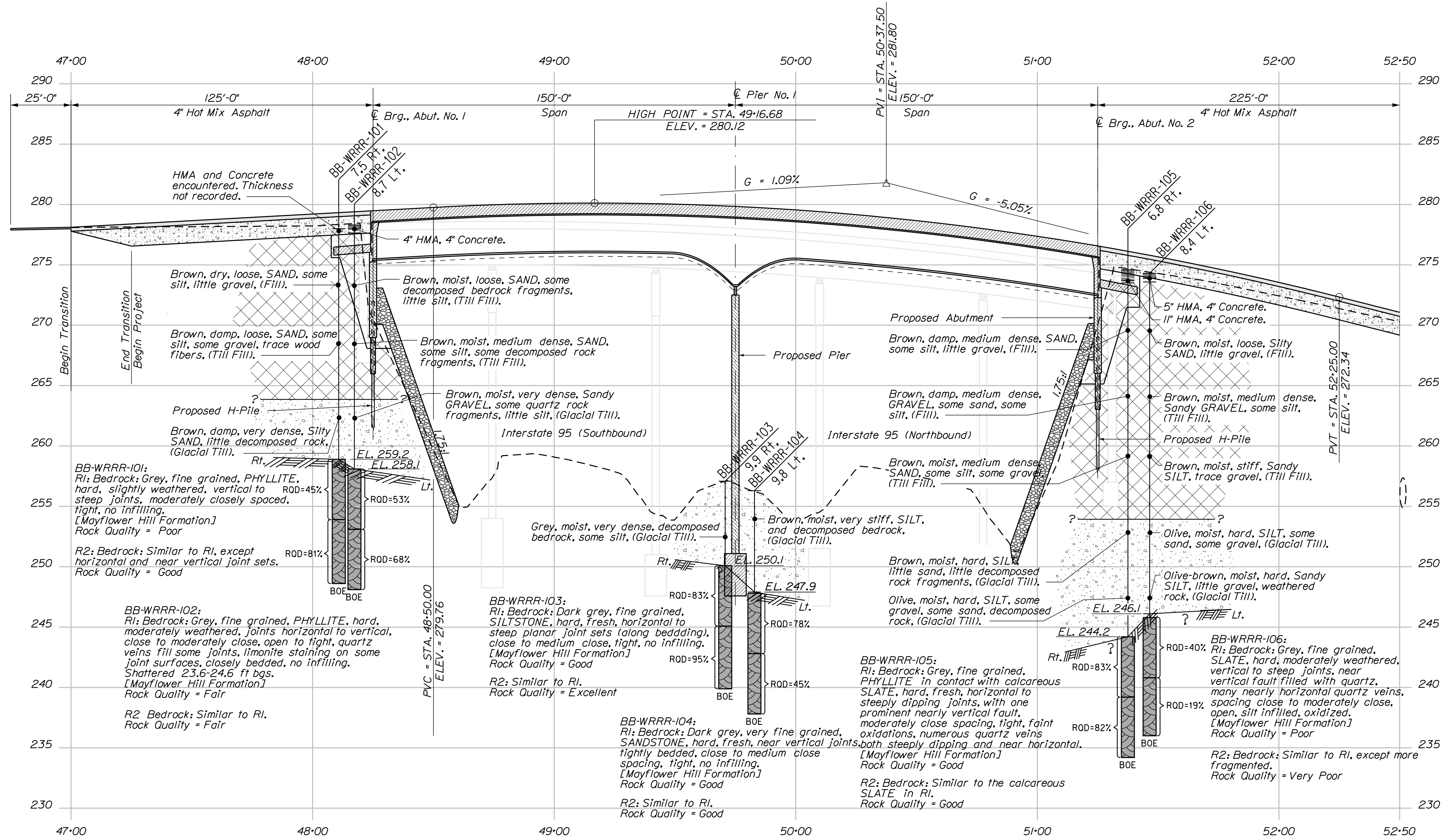
SHEET NUMBER
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OF 5

Username: James.Moran

Date: 9/24/2020

Division: GEOTECH

Filename: ... \00\GEOTECH\MSTA007_ISP1.dgn



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.

DESIGN	DATE	BY	PROJ. MGR.
DESIGN-REVIEWED	Mar 2019	Kendra N.	D. Eaton
DESIGN-REVIEWED	SEP 2020	T. WHITE	
DESIGN-REVIEWED			
DESIGN-REVIEWED			

REVISIONS	DATE	DESCRIPTION
1		
2		
3		
4		

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM				MODIFIED BURMISTER 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	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.													
		(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):				Desired Rock 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				Rock Quality Designation (RQD): RQD (%) = $\frac{\text{sum of the lengths of intact pieces of core}^* > 4 \text{ inches}}{\text{length of core advance}}$ *Minimum NQ rock core (1.88 in. OD of core)													
Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information				Rock Quality Based on RQD <table border="1"> <thead> <tr> <th>Rock Quality</th> <th>RQD (%)</th> </tr> </thead> <tbody> <tr> <td>Very Poor</td> <td>≤25</td> </tr> <tr> <td>Poor</td> <td>26 - 50</td> </tr> <tr> <td>Fair</td> <td>51 - 75</td> </tr> <tr> <td>Good</td> <td>76 - 90</td> </tr> <tr> <td>Excellent</td> <td>91 - 100</td> </tr> </tbody> </table>		Rock Quality	RQD (%)	Very Poor	≤25	Poor	26 - 50	Fair	51 - 75	Good	76 - 90	Excellent	91 - 100
Rock Quality	RQD (%)																
Very Poor	≤25																
Poor	26 - 50																
Fair	51 - 75																
Good	76 - 90																
Excellent	91 - 100																
				Sample Container Labeling Requirements: WIN Bridge Name / Town Boring Number Sample Number Sample Depth Blow Counts Sample Recovery Date Personnel Initials													

Driller: MaineDOT	Elevation (ft.): 278.0	Auger ID/OD: 5" Solid Stem
Operator: Daggett/Wilder/Niles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: A. Van Buskirk	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 4/9/2019; 08:45-14:15	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 48+10.8, 7.5 ft Rt.	Casing ID/OD: NW-3"	Water Level*: 9.0 ft bgs.
Hammer Efficiency Factor: 0.928	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	

Definitions: R = Rock Core Sample S_U = Peak/Remolded Field Vane Undrained Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger S_{U(lab)} = Lab Vane Undrained Shear Strength (psf) WC = Water Content, percent
 MD = Unsuccessful Split Spoon Sample Attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw Field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample Attempt WOH = Weight of 140lb. Hammer Hammer Efficiency Factor = Rig Specific Annual Calibration Value PI = Plasticity Index
 V = Field Vane Shear Test, PP = Pocket Penetrometer WOR/C = Weight of Rods or Casing N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency G = Grain Size Analysis
 MV = Unsuccessful Field Vane Shear Test Attempt WO1P = Weight of One Person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected 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					
0											HMA and Concrete encountered. Thickness not recorded.	
5	1D	24/19	5.00 - 7.00	4/3/3/4	6	9					Brown, dry, loose, SAND, some silt, little gravel, (Fill).	
10	2D	24/15	10.00 - 12.00	3/2/2/4	4	6					Brown, damp, loose, SAND, some silt, some gravel, trace wood fibers, (Till Fill).	G#337218 A-2-4, SM WC=13.87%
15	3D	16.8/15	15.00 - 16.40	11/18/68(4.8")	---			94	264.0		Brown, damp, very dense, Silty SAND, little decomposed rock, (Glacial Till).	
								294			a119 blows for 0.1 ft. NW Casing REFUSAL at 17.1 ft bgs. Roller Coned ahead to 19.0 ft bgs.	
20	MD R1	.08/0 60/60	19.00 - 19.01 19.10 - 24.10	30(1") RQD = 45%	---			NQ-2	258.9		Top of Bedrock at Elev. 259.2 ft. R1: Bedrock: Grey, fine grained, PHYLLITE, hard, slightly weathered, vertical to steep joints, moderately closely spaced, tight, no infilling. [Mayflower Hill Formation] Rock Quality = Poor R1: Core Times (min:sec) 19.1-19.6 ft (not recorded) 19.6-20.1 ft (1:43) 20.1-21.1 ft (3:09) 21.1-22.1 ft (3:09) 22.1-23.1 ft (4:37) core blocked at 22.0 ft bgs.	GTX 20.0-20.4 q _p = 10577 psi
25	R2	63.6/63.6	24.10 - 29.40	RQD = 81%								

Remarks:

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Armstrong Road Bridge #5815 over Interstate 95 Location: Waterville, Maine	Boring No.: BB-WRRR-101 WIN: 23505.00
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Driller: MaineDOT	Elevation (ft.): 278.0	Auger ID/OD: 5" Solid Stem
Operator: Daggett/Wilder/Niles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: A. Van Buskirk	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 4/9/2019; 08:45-14:15	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 48+10.8, 7.5 ft Rt.	Casing ID/OD: NW-3"	Water Level*: 9.0 ft bgs.

Hammer Efficiency Factor: 0.928	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	
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 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										23.1-24.1 ft (4:10) 100% Recovery R2: Bedrock: Grey, fine grained, PHYLLITE, fresh, horizontal and near vertical joint sets, moderately close, few quartz veins, tight. [Mayflower Hill Formation] Rock Quality = Good R2: Core Times (min:sec) 24.1-25.1 ft (3:17) 25.1-26.1 ft (3:10) 26.1-27.1 ft (2:52) 27.1-28.1 ft (3:22) core blocked at 28.0 ft. 28.1-29.4 ft (5:51) 100% Recovery 29.4 Bottom of Exploration at 29.4 feet below ground surface.		
30												
35												
40												
45												
50												


Remarks:

Driller: MaineDOT	Elevation (ft.): 278.3	Auger ID/OD: 5" Solid Stem
Operator: Daggett/Wilder/Niles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: A. Van Buskirk	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 4/5/2019; 07:45-11:30	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 48+17.3, 8.7 ft Lt.	Casing ID/OD: NW-3"	Water Level*: See note (1).
Hammer Efficiency Factor: 0.928	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	

Definitions: R = Rock Core Sample S_U = Peak/Remolded Field Vane Undrained Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger S_{U(lab)} = Lab Vane Undrained Shear Strength (psf) WC = Water Content, percent
 MD = Unsuccessful Split Spoon Sample Attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone WOH = Weight of 140lb. Hammer N-uncorrected = Raw Field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample Attempt PP = Pocket Penetrometer Hammer Efficiency Factor = Rig Specific Annual Calibration Value PI = Plasticity Index
 V = Field Vane Shear Test, PP = Pocket Penetrometer WOR/C = Weight of Rods or Casing N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency G = Grain Size Analysis
 MV = Unsuccessful Field Vane Shear Test Attempt WO1P = Weight of One Person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected 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					
0								SSA	278.0 277.6	4" HMA. 4" CONCRETE.		
5	1D	24/14	5.00 - 7.00	3/3/3/3	6	9				Brown, moist, loose, SAND, some decomposed bedrock fragments, little silt, (Till Fill).		
10	2D	24/19	10.00 - 12.00	5/5/5/6	10	15				Brown, moist, medium dense, SAND, some silt, some decomposed rock fragments, (Till Fill).	G#337219 A-2-4, SM WC=11.0%	
15	3D	24/22	15.00 - 17.00	31/34/51/57	85	131	81		263.3	Brown, moist, very dense, Sandy GRAVEL, some quartz rock fragments, little silt, (Glacial Till). Roller Coned ahead from 17.0-20.0 ft bgs.		
20	4D R1	1.2/0 60/60	20.00 - 20.10 20.20 - 25.20	51(1.2") RQD = 53%	---		NQ-2		258.1	Roller Coned easily from 19.7-20.0 ft bgs. Grey, bedrock fragments in tip of spoon. Roller Coned ahead from 20.0-20.2 ft bgs. Top of Bedrock at Elev. 258.1 ft. R1: Bedrock: Grey, fine grained, PHYLLITE, hard, moderately weathered, joints horizontal to vertical, close to moderately close, open to tight, quartz veins fill some joints, limonite staining on some joint surfaces, closely bedded, no infilling. Shattered 23.6-24.6 ft bgs. [Mayflower Hill Formation] Rock Quality = Fair		
25												

Remarks:
 After withdrawal of casing, water was observed to fill the borehole to 1 ft bgs.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Armstrong Road Bridge #5815 over Interstate 95 Location: Waterville, Maine		Boring No.: <u>BB-WRRR-102</u>									
				WIN: <u>23505.00</u>											
Driller: MaineDOT		Elevation (ft.): 278.3		Auger ID/OD: 5" Solid Stem											
Operator: Daggett/Wilder/Niles		Datum: NAVD88		Sampler: Standard Split Spoon											
Logged By: A. Van Buskirk		Rig Type: CME 45C		Hammer Wt./Fall: 140#/30"											
Date Start/Finish: 4/5/2019; 07:45-11:30		Drilling Method: Cased Wash Boring		Core Barrel: NQ-2"											
Boring Location: 48+17.3, 8.7 ft Lt.		Casing ID/OD: NW-3"		Water Level*: See note (1).											
Hammer Efficiency Factor: 0.928				Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>											
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 Information								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	Elevation (ft.)							
25	R2	60/60	25.20 - 30.20	RQD = 68%				248.1		<p>R1: Core Times (min:sec) 20.2-21.2 ft (4:54) 21.2-22.2 ft (2:23) 22.2-23.2 ft (2:13) 23.2-24.2 ft (2:12) 24.2-25.2 ft (2:34) 100% Recovery, 9" of R1 recovered in R2.</p> <p>R2: Bedrock: Grey, fine grained, PHYLLITE, hard, slightly weathered, horizontal to steep joints, close to medium close, open to tight, quartz veins fill some joints with one near vertical fault (unhealed), limonite staining on some joints, some oxidized surfaces, some silt infilling. [Mayflower Hill Formation] Rock Quality = Fair</p> <p>R2: Core Times (min:sec) 25.2-26.2 ft (2:18) 26.2-26.8 ft (2:48) core blocked 26.8-27.2 ft (0:58) 27.2-28.2 ft (3:07) 28.2-29.2 ft (3:21) 29.2-30.2 ft (3:48) 100% Recovery</p> <p style="text-align: right;">30.2</p> <p style="text-align: center;">Bottom of Exploration at 30.2 feet below ground surface.</p>					
30															
35															
40															
45															
50															
Remarks:															
After withdrawal of casing, water was observed to fill the borehole to 1 ft bgs.															
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 2 of 2					
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										Boring No.: BB-WRRR-102					

Driller: MaineDOT	Elevation (ft.): 274.6	Auger ID/OD: 5" Solid Stem
Operator: Daggett/Wilder/Niles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: A. Van Buskirk	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 4/12/2019; 08:00-12:00	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 51+37.5, 6.8 ft Rt.	Casing ID/OD: NW-3"	Water Level*: 15.0 ft bgs.

Hammer Efficiency Factor: 0.928 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger $S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf) WC = Water Content, percent
 MD = Unsuccessful Split Spoon Sample Attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
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 MV = Unsuccessful Field Vane Shear Test Attempt WO1P = Weight of One Person N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected 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					
0									273.7	11" HMA.		
									273.4	4" CONCRETE.		
5	1D	24/12	5.00 - 7.00	4/7/6/6	13	20				Brown, damp, medium dense, SAND, some silt, little gravel, (Fill).		
10	2D	24/15	10.00 - 12.00	7/5/7/11	12	19				Brown, damp, medium dense, GRAVEL, some sand, some silt, (Fill).	G#337220 A-1-b, GM WC=7.7%	
15	3D	24/16	15.00 - 17.00	3/4/6/6	10	15	18			Brown, moist, medium dense, SAND, some silt, some gravel, (Till Fill).	G#337221 A-2-4, SM WC=14.7%	
20	4D	24/17	20.00 - 22.00	14/22/20/23	42	65	28		254.6	Brown, moist, hard, SILT, little sand, little decomposed rock fragments, (Glacial Till).		
25							108					

Remarks:

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Armstrong Road Bridge #5815 over Interstate 95 Location: Waterville, Maine				Boring No.: BB-WRRR-105			
Driller: MaineDOT				Elevation (ft.): 274.6		Auger ID/OD: 5" Solid Stem					
Operator: Daggett/Wilder/Niles				Datum: NAVD88		Sampler: Standard Split Spoon					
Logged By: A. Van Buskirk				Rig Type: CME 45C		Hammer Wt./Fall: 140#/30"					
Date Start/Finish: 4/12/2019; 08:00-12:00				Drilling Method: Cased Wash Boring		Core Barrel: NQ-2"					
Boring Location: 51+37.5, 6.8 ft Rt.				Casing ID/OD: NW-3"		Water Level*: 15.0 ft bgs.					
Hammer Efficiency Factor: 0.928				Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>							
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 Information								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	Elevation (ft.)			
25	5D	24/20	25.00 - 27.00	20/16/25/36	41	63	RC			Grey-brown, moist, hard SILT and weathered rock, (Glacial Till).	
30	6D R1	4.8/4 60/60	30.00 - 30.40 30.40 - 35.40	53(4.8") RQD = 83%	---		NQ-2	244.2		Roller Coned ahead to 30.0 ft bgs, Drove NW Casing to 30.4 ft bgs. Olive, moist, hard, SILT, some gravel, some sand, decomposed rock, (Glacial Till). Top of Bedrock at Elev. 244.2 ft. R1: Bedrock: Grey, fine grained, PHYLLITE in contact with calcareous SLATE, hard, fresh, horizontal to steeply dipping joints, with one prominent nearly vertical fault, moderately close spacing, tight, faint oxidations, numerous quartz veins both steeply dipping and near horizontal. [Mayflower Hill Formation] Rock Quality = Good R1: Core Times (min:sec) 30.4-31.4 ft (2:54) 31.4-32.4 ft (2:01) 32.4-33.4 ft (2:09) 33.4-34.4 ft (2:22) 34.4-35.4 ft (2:31) 100% Recovery R2: Bedrock: Similar to the calcareous SLATE in R1. Rock Quality = Good R2: Core Times (min:sec) 35.4-36.4 ft (2:24) 36.4-37.4 ft (2:19) 37.4-38.4 ft (2:00) 38.4-39.4 ft (2:31) 39.4-40.4 ft (2:27) 100% Recovery	GTX 30.4-30.8 q _p =7047 psi
35	R2	60/60	35.40 - 40.40	RQD = 82%							
40								234.2			
45											
50											
Remarks: Stratification lines represent approximate boundaries between soil types; transitions may be gradual. * Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.											
										Page 2 of 2	
										Boring No.: BB-WRRR-105	

Driller: MaineDOT	Elevation (ft.): 274.3	Auger ID/OD: 5" Solid Stem
Operator: Daggett/Wilder/Niles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: A. Van Buskirk	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 4/4/2019; 08:45-13:30	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 51+46.5, 8.4 ft Lt.	Casing ID/OD: NW-3"	Water Level*: None Observed
Hammer Efficiency Factor: 0.928	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	

Definitions: R = Rock Core Sample S_U = Peak/Remolded Field Vane Undrained Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger S_{U(lab)} = Lab Vane Undrained Shear Strength (psf) WC = Water Content, percent
 MD = Unsuccessful Split Spoon Sample Attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
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 V = Field Vane Shear Test, PP = Pocket Penetrometer WOR/C = Weight of Rods or Casing N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency G = Grain Size Analysis
 MV = Unsuccessful Field Vane Shear Test Attempt WO1P = Weight of One Person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected 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					
0									273.9			
									273.6			
5	1D	24/14	5.00 - 7.00	3/3/3/6	6	9					Brown, moist, loose, Silty SAND, little gravel, (Fill).	
10	2D	24/15	10.00 - 12.00	3/6/7/7	13	20	16				Brown, moist, medium dense, Sandy GRAVEL, some silt, (Till Fill).	G#337222 A-1-b, GM WC=11.3%
15	3D	24/13	15.00 - 17.00	6/4/6/8	10	15	19				Brown, moist, stiff, Sandy SILT, trace gravel, (Till Fill).	
20	4D	24/16	20.00 - 22.00	15/21/21/24	42	65	48		255.8		Olive, moist, hard, SILT, some sand, some gravel, (Glacial Till).	G#337223 A-4, SM WC=12.5%
25												

Remarks:

Appendix B

Rock Core Photographs



MaineDOT

Armstrong Road Bridge #5815 Carries Armstrong Road Over Interstate 95

Waterville, ME

Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-WRRR-101	R1	19.1–24.1	60	100%	27	45%	PHYLLITE	1
BB-WRRR-101	R2	24.1–29.4	64	100%	52	81%	PHYLLITE	2 and 3





Armstrong Road Bridge #5815 Carries Armstrong Road Over Interstate 95
Waterville, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-WRRR-102	R1	20.2-25.2	60	100%	32	53%	PHYLLITE	Box 1, Row 4; Box 2, Row 1
BB-WRRR-102	R2	25.2-30.2	60	60	41	68	PHYLLITE	Box 2, Row 1 and 2





MaineDOT

Armstrong Road Bridge #5815 Carries Armstrong Road Over Interstate 95

Waterville, ME

Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-WRRR-103	R1	7.0-12.0	60	60	50	83%	Siltstone	3
BB-WRRR-103	R2	12.0-17.2	62	62	59	95%	Siltstone	4





MaineDOT

Armstrong Road Bridge #5815 Carries Armstrong Road Over Interstate 95

Waterville, ME

Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-WRRR-104	R1	8.5-13.5	60	60	47	78%	Sandstone	1
BB-WRRR-104	R2	13.5-18.5	60	60	54	90%	Sandstone	2





MaineDOT

Armstrong Road Bridge #5815 Carries Armstrong Road Over Interstate 95

Waterville, ME

Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-WRRR-105	R1	30.4-35.4	60	60	50	83%	Phyllite and Slate	Row 1
BB-WRRR-105	R2	35.4-40.4	60	60	49	82%	Phyllite and Slate	Row 2 and 3





MaineDOT
Armstrong Road Bridge #5815 Carries Armstrong Road Over Interstate 95
Waterville, ME
Rock Core Photographs

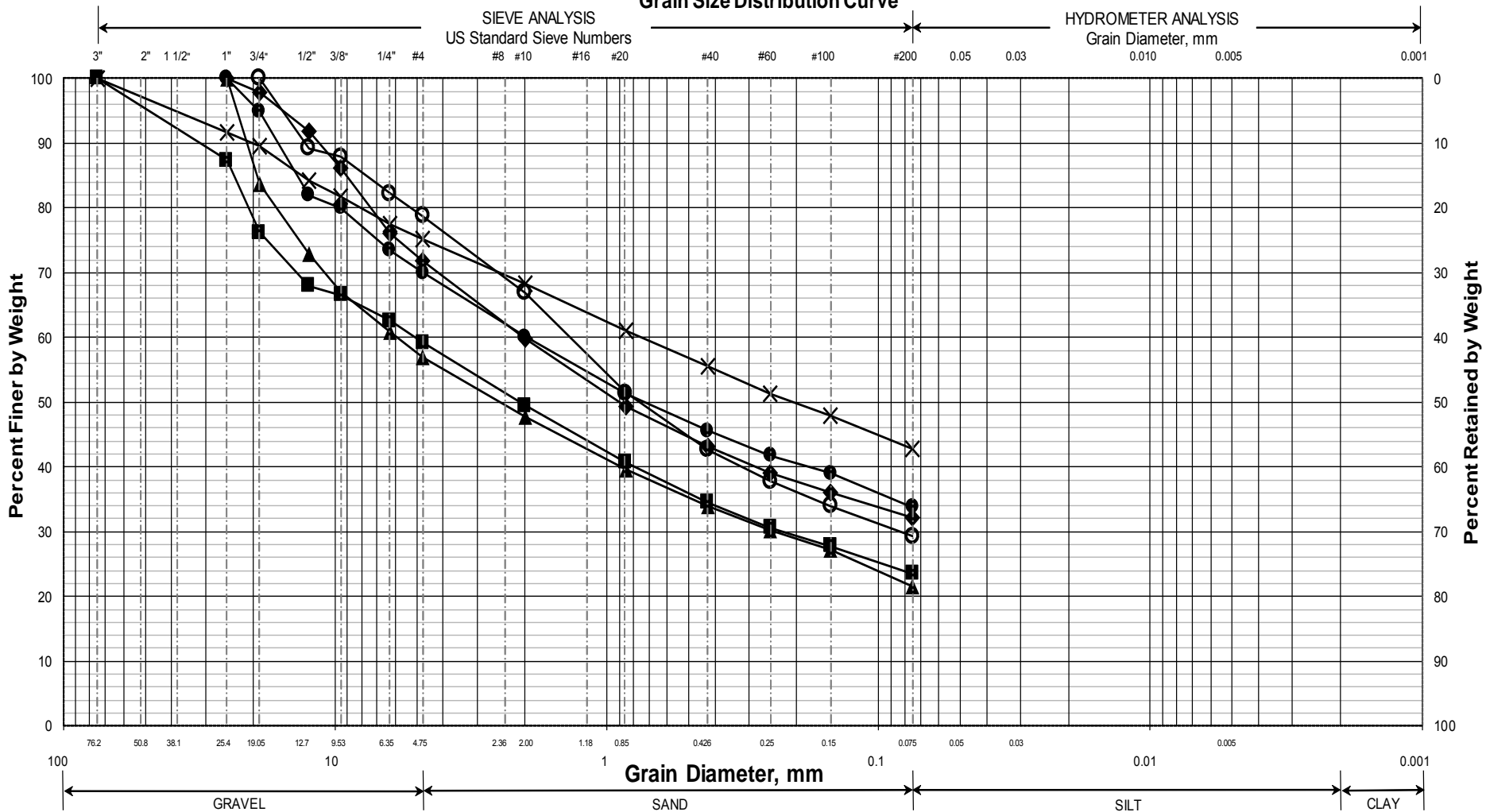
Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-WRRR-106	R1	28.5-33.5	60	60	24	40%	Slate	Row 1 and 2
BB-WRRR-106	R2	33.5-38.5	58	58	11	19%	Slate	Row 2 and 3



Appendix C

Laboratory Test Results

Maine Department of Transportation Grain Size Distribution Curve



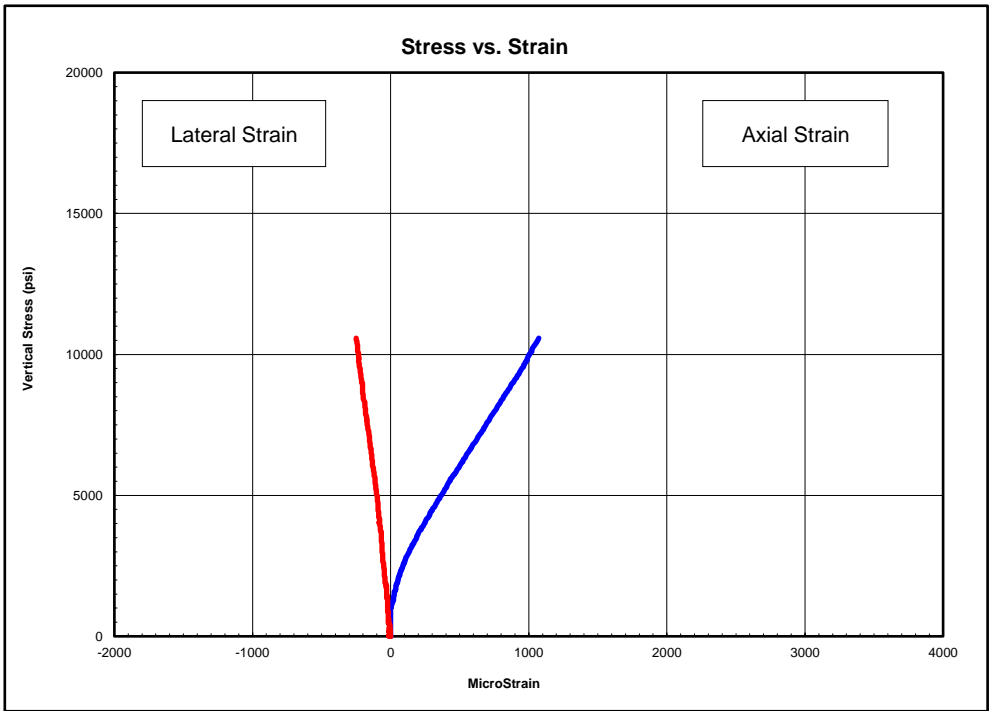
	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-WRRR-101/2D	48+10.8	7.5 RT	10.0-12.0	SAND, some silt, some gravel.	13.8			
◆	BB-WRRR-102/2D	48+17.3	8.7 LT	10.0-12.0	SAND, some silt, some gravel.	11			
■	BB-WRRR-105/2D	51+37.5	6.8 RT	10.0-12.0	GRAVEL, some sand, some silt.	7.7			
●	BB-WRRR-105/3D	51+37.5	6.8 RT	15.0-17.0	SAND, some silt, some gravel.	14.7			
▲	BB-WRRR-106/2D	51+46.5	8.4 LT	10.0-12.0	Sandy GRAVEL, some silt.	11.3			
×	BB-WRRR-106/4D	51+46.5	8.4 LT	20.0-22.0	SILT, some sand, some gravel.	12.5			

WIN
023505.00
Town
Waterville
Reported by/Date
WHITE, TERRY A 4/30/2019



Client:	Maine Department of Transportation
Project Name:	Armstrong Road Bridge
Project Location:	Waterville, ME
GTX #:	310323
Test Date:	7/30/2019
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-WRRR-101
Sample ID:	R1
Depth, ft:	20.00-20.38
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D



Peak Compressive Stress: 10,577 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1100-3900	12,600,000	0.26
3900-6700	7,900,000	0.20
6700-9500	7,650,000	0.22

Notes: Test specimen tested at the approximate as-received moisture content and at standard laboratory temperature. The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes. Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed. Calculations assume samples are isotropic, which is not necessarily the case.

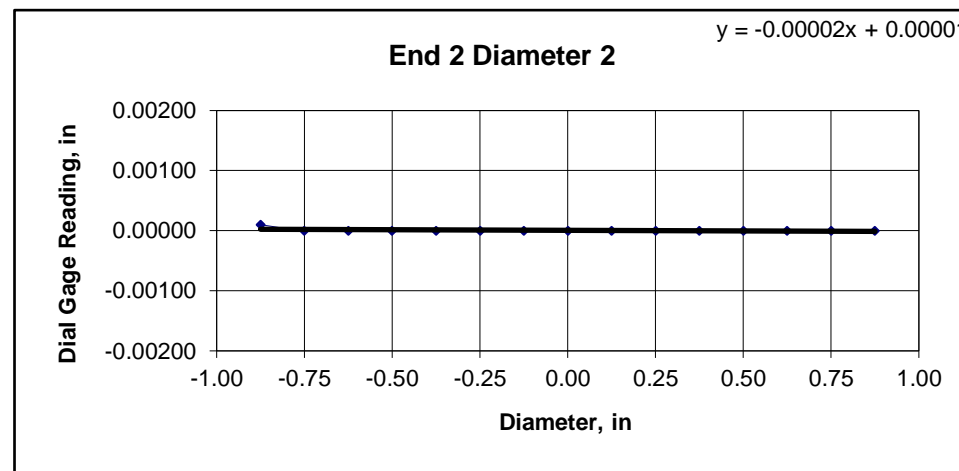
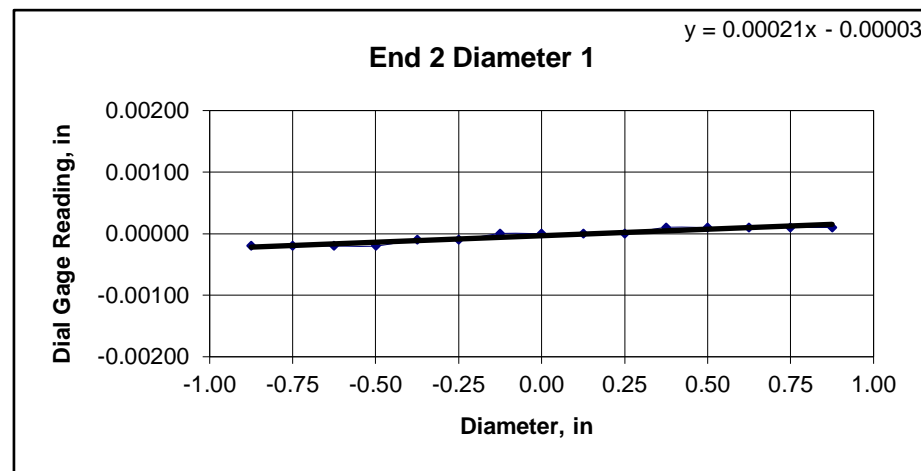
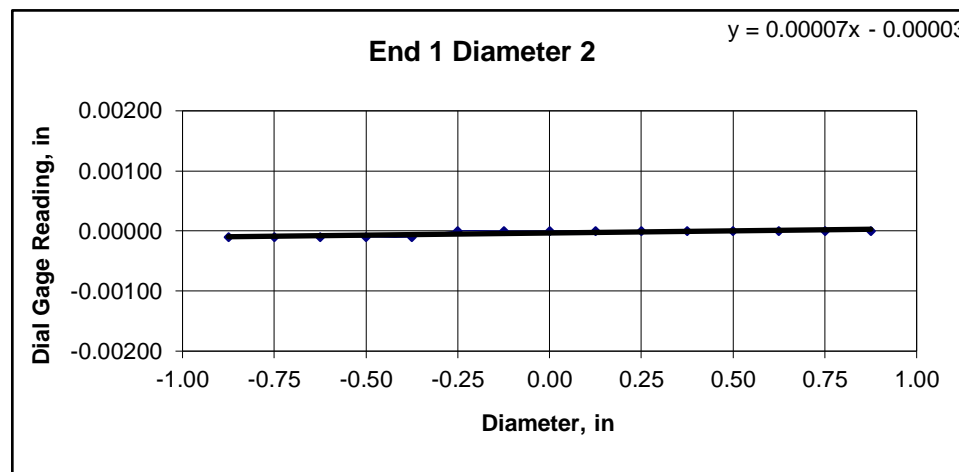
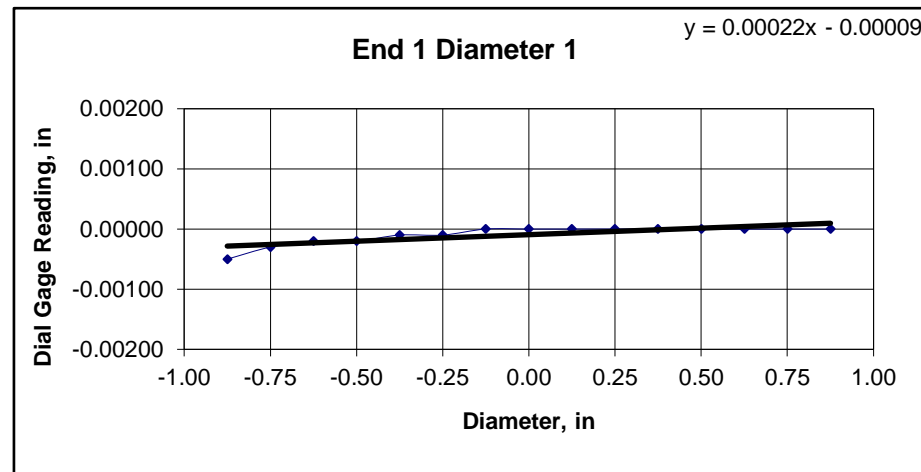


Client:	Maine Department of Transportation	Test Date:	7/25/2019
Project Name:	Armstrong Road Bridge	Tested By:	cmh
Project Location:	Waterville, ME	Checked By:	jsc
GTX #:	310323		
Boring ID:	BB-WRRR-101		
Sample ID:	R1		
Depth:	20.00-20.38 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY				DEVIATION FROM STRAIGHTNESS (Procedure S1)			
	1	2	Average	Maximum gap between side of core and reference surface plate:			
Specimen Length, in:	4.45	4.44	4.45	Is the maximum gap \leq 0.02 in.? YES			
Specimen Diameter, in:	1.98	1.98	1.98	Maximum difference must be $<$ 0.020 in.			
Specimen Mass, g:	589.94			Straightness Tolerance Met? YES			
Bulk Density, lb/ft ³ :	164						
Length to Diameter Ratio:	2.2						
		Minimum Diameter Tolerance Met? YES					
		Length to Diameter Ratio Tolerance Met? YES					

END FLATNESS AND PARALLELISM (Procedure FP1)															
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00050	-0.00030	-0.00020	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
	Difference between max and min readings, in:														
	0° = 0.00050							90° = 0.00010							
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00020	-0.00020	-0.00020	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010	0.00010
Diameter 2, in (rotated 90°)	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
	Difference between max and min readings, in:														
	0° = 0.0003							90° = 0.0001							
	Maximum difference must be $<$ 0.0020 in. Difference = \pm 0.00025														
	Flatness Tolerance Met? YES														



DIAMETER 1	
End 1:	Slope of Best Fit Line: 0.00022 Angle of Best Fit Line: 0.01244
End 2:	Slope of Best Fit Line: 0.00021 Angle of Best Fit Line: 0.01211
Maximum Angular Difference:	0.00033
Parallelism Tolerance Met? YES	Spherically Seated

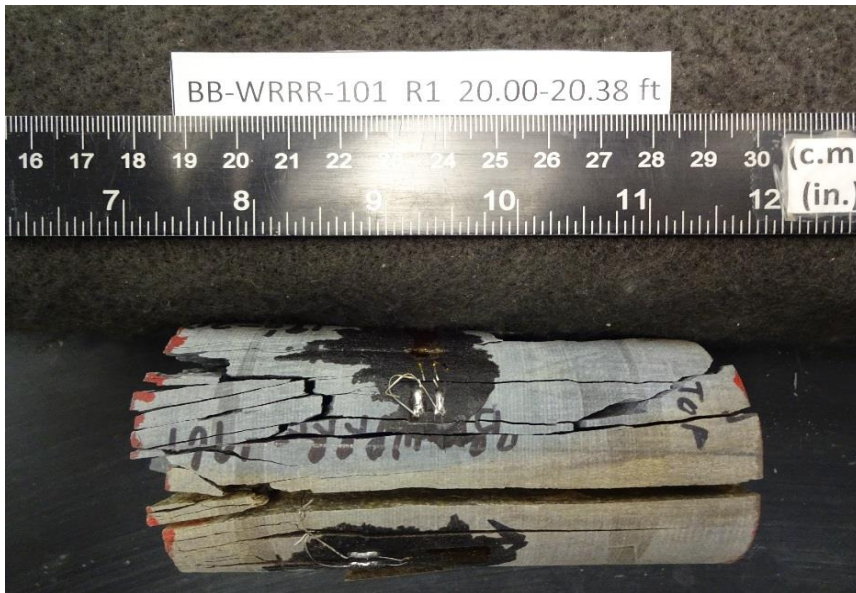
DIAMETER 2	
End 1:	Slope of Best Fit Line: 0.00007 Angle of Best Fit Line: 0.00409
End 2:	Slope of Best Fit Line: 0.00002 Angle of Best Fit Line: 0.00115
Maximum Angular Difference:	0.00295
Parallelism Tolerance Met? YES	Spherically Seated

PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)						Maximum angle of departure must be \leq 0.25°	
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?		
Diameter 1, in	0.00050	1.980	0.00025	0.014	YES		
Diameter 2, in (rotated 90°)	0.00010	1.980	0.00005	0.003	YES	Perpendicularity Tolerance Met? YES	
END 2							
Diameter 1, in	0.00030	1.980	0.00015	0.009	YES		
Diameter 2, in (rotated 90°)	0.00010	1.980	0.00005	0.003	YES		

Client:	Maine Department of Transportation
Project Name:	Armstrong Road Bridge
Project Location:	Waterville, ME
GTX #:	310323
Test Date:	7/26/2019
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-WRRR-101
Sample ID:	R1
Depth, ft:	20.00-20.38



After cutting and grinding

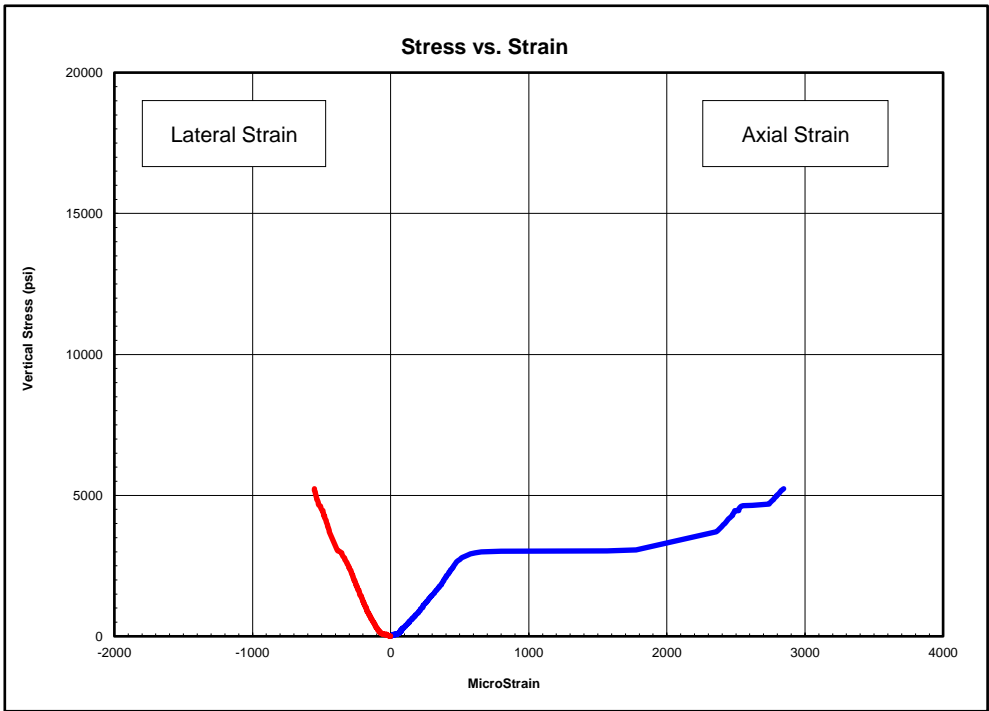


After break



Client:	Maine Department of Transportation
Project Name:	Armstrong Road Bridge
Project Location:	Waterville, ME
GTX #:	310323
Test Date:	7/30/2019
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-WRRR-104
Sample ID:	R1
Depth, ft:	9.29-9.67
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D



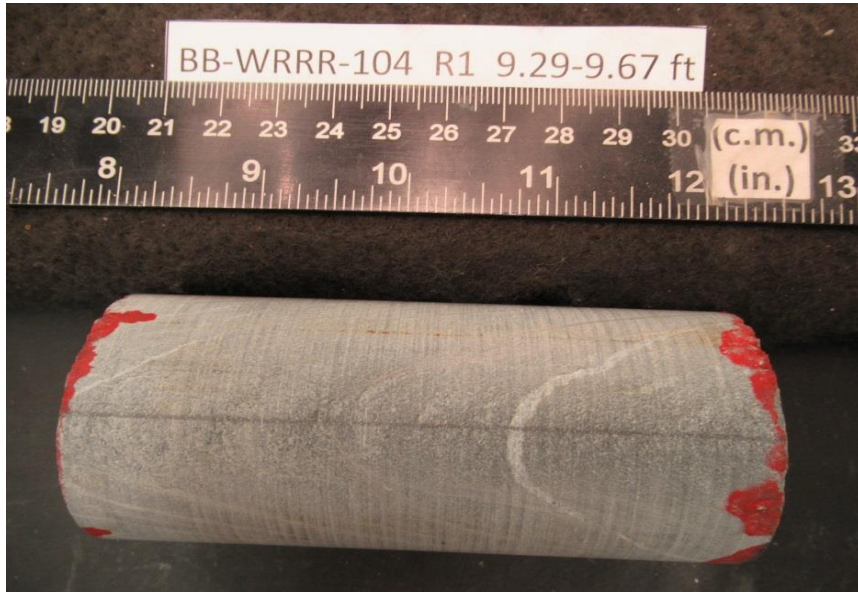
Peak Compressive Stress: 5,236 psi

The axial strain gauges picked up an initial failure within the specimen and then continued reading until total failure occurred. Young's Modulus and Poisson's Ratio could not be determined within the second and third stress range.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
500-1900	5,810,000	---
1900-3300	---	---
3300-4700	---	---

Notes: Test specimen tested at the approximate as-received moisture content and at standard laboratory temperature. The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes. Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed. Calculations assume samples are isotropic, which is not necessarily the case.

Client:	Maine Department of Transportation
Project Name:	Armstrong Road Bridge
Project Location:	Waterville, ME
GTX #:	310323
Test Date:	7/31/2019
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-WRRR-104
Sample ID:	R1
Depth, ft:	9.29-9.67



After cutting and grinding

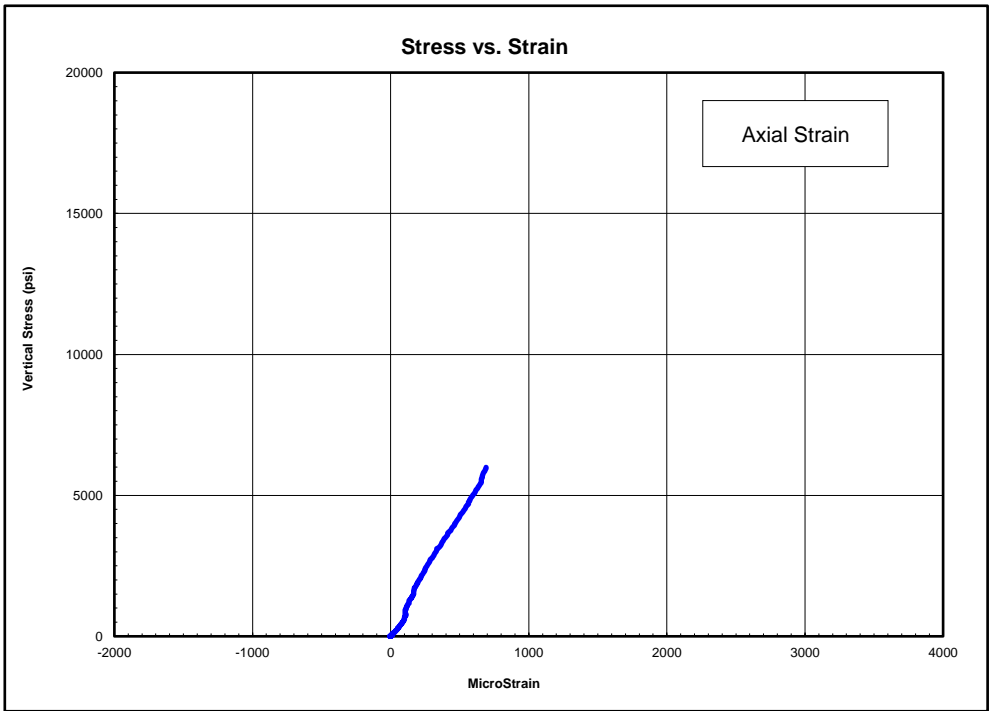


After break



Client:	Maine Department of Transportation
Project Name:	Armstrong Road Bridge
Project Location:	Waterville, ME
GTX #:	310323
Test Date:	7/30/2019
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-WRRR-105
Sample ID:	R1
Depth, ft:	30.43-30.81
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D



Peak Compressive Stress: 7,047 psi

Both lateral strain gauges failed to record meaningful data. Poisson's Ratio could not be determined.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
700-2600	10,700,000	---
2600-4500	7,290,000	---
4500-6300	10,700,000	---

Notes: Test specimen tested at the approximate as-received moisture content and at standard laboratory temperature. The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes. Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed. Calculations assume samples are isotropic, which is not necessarily the case.

Client:	Maine Department of Transportation
Project Name:	Armstrong Road Bridge
Project Location:	Waterville, ME
GTX #:	310323
Test Date:	7/31/2019
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-WRRR-105
Sample ID:	R1
Depth, ft:	30.43-30.81



After cutting and grinding



After break

Appendix D

Calculations

Driven H-Pile Resistance

Design of H-piles

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

Bedrock Properties

Rock Type: Phyllite

$\phi = 27-34$ (AASHTO LRFD Table C.10.4.6.4-1);

Phyllite $C_o = 3,500 - 35,000$ psi (AASHTO Standard Specifications for Bridges 17th Edition, Table 4.4.8.1.2B)

For Design Purposes, use bedrock data from BB-WRRR-105: RQD = 83% and an Unconfined Compressive Strength of 7,047 psi based on lab testing of bedrock core. BB-WRRR-105 R1.

Pile Properties

Use the following piles: 14x89, 14x117

$$A_s := \begin{pmatrix} 26.1 \\ 34.4 \end{pmatrix} \cdot \text{in}^2$$

$$d := \begin{pmatrix} 13.8 \\ 14.2 \end{pmatrix} \cdot \text{in}$$

$$b := \begin{pmatrix} 14.7 \\ 14.9 \end{pmatrix} \cdot \text{in}$$

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

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

Note: All matrices set up in this order

14x89

14x117

$r_s =$ radius of gyration

$$r_s := \begin{pmatrix} 3.53 \\ 3.59 \end{pmatrix} \cdot \text{in}$$

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

Pile yield strength

$$F_y := 50 \cdot \text{ksi}$$

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

Use LRFD Equation 6.9.2.1-1 $Pr = \phi P_n$

Nominal Axial Structural Resistance

Determine equivalent yield resistance $P_o = QF_y A_s$ (LRFD 6.9.4.1.1)

$Q := 1.0$ LRFD Article 6.9.4.2

$$P_o := Q \cdot F_y \cdot A_s \quad \boxed{P_o = \begin{pmatrix} 1305 \\ 1720 \end{pmatrix} \cdot \text{kip}}$$

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 $K_{\text{eff}} := 2.1$

LRFD Table C4.6.2.5-1
(assume plastic hinge develops, K selected per Vtrans Design Procedure)

$l =$ "unbraced" length $l_{\text{unbraced_top}} := 4 \cdot \text{ft}$

LRFD eq. 6.9.4.1.2-1

$$P_e := \left[\frac{\pi^2 \cdot E}{\left(\frac{K_{\text{eff}} \cdot l_{\text{unbraced_top}}}{r_s} \right)^2} \cdot A_s \right] \quad P_e = \begin{pmatrix} 9162 \\ 12489 \end{pmatrix} \cdot \text{kip}$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 7.02 \\ 7.261 \end{pmatrix} \quad \text{If } P_e/P_o > \text{ or } = 0.44, \text{ then:} \quad P_n := \begin{pmatrix} P_o \\ 0.658 \cdot P_e \end{pmatrix} \cdot P_o \quad \text{LRFD Eq. 6.9.4.1.1-1}$$

then:

$$\boxed{P_n = \begin{pmatrix} 1229 \\ 1624 \end{pmatrix} \cdot \text{kip}}$$

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

$$P_r = \begin{pmatrix} 861 \\ 1137 \end{pmatrix} \cdot \text{kip}$$

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

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

K = effective length factor

$$K_{\text{eff}} := 1.0$$

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

$$l_{\text{unbraced_mid}} := 13 \cdot \text{ft}$$

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

LRFD eq. 6.9.4.1.2-1

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

$$P_e = \begin{pmatrix} 3825 \\ 5214 \end{pmatrix} \cdot \text{kip}$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 2.931 \\ 3.032 \end{pmatrix}$$

If $P_e/P_o > \text{or} = 0.44$, then:

$$P_n := \begin{pmatrix} P_o \\ 0.658 \cdot P_e \cdot P_o \end{pmatrix}$$

LRFD Eq. 6.9.4.1.1-1

then:

this applies to all pile sizes

$$P_n = \begin{pmatrix} 1131 \\ 1498 \end{pmatrix} \cdot \text{kip}$$

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

$$P_r = \left(\begin{array}{c} 792 \\ 1049 \end{array} \right) \cdot \text{kip}$$

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

LRFD eq. 6.9.4.1.2-1

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

$$P_e = \left(\begin{array}{c} 2 \times 10^8 \\ 2 \times 10^8 \end{array} \right) \cdot \text{kip}$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \left(\begin{array}{c} 1.172 \times 10^5 \\ 1.213 \times 10^5 \end{array} \right) \quad \text{If } P_e/P_o > \text{ or } = 0.44, \text{ then:}$$

$$P_n := \left(\begin{array}{c} P_o \\ 0.658 \cdot P_e \cdot P_o \end{array} \right)$$

LRFD Eq. 6.9.4.1.1-1

then:

this applies to all pile sizes

$$P_n = \left(\begin{array}{c} 1305 \\ 1720 \end{array} \right) \cdot \text{kip}$$

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 (P_r) per LRFD 6.9.2.1-1 is

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

Factored structural compressive resistance, P_r

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

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:

$$P_n := \begin{pmatrix} 1131 \\ 1498 \end{pmatrix} \text{kip}$$

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

$$\phi_c := 0.5$$

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

$$P_r = \begin{pmatrix} 566 \\ 749 \end{pmatrix} \cdot \text{kip}$$

14x89
14x117

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

$$\phi_c := 1.0$$

$$P_{r_cc} := \phi_c \cdot P_n$$

$$P_{r_cc} = \begin{pmatrix} 1131 \\ 1498 \end{pmatrix} \cdot \text{kip} \quad \begin{matrix} 14 \times 89 \\ 14 \times 117 \end{matrix}$$

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, Drivability 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 12 blows per inch (bpi). Piles may start to "walk" due to sloping rock with driving criteria greater than 12 bpi.

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

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

$$\phi_{dyn} := 0.65$$

Reference LRFD Table 10.5.5.2.3-1 - for Strength Limit State

$$\phi := 1.0$$

For Extreme and Service Limit States

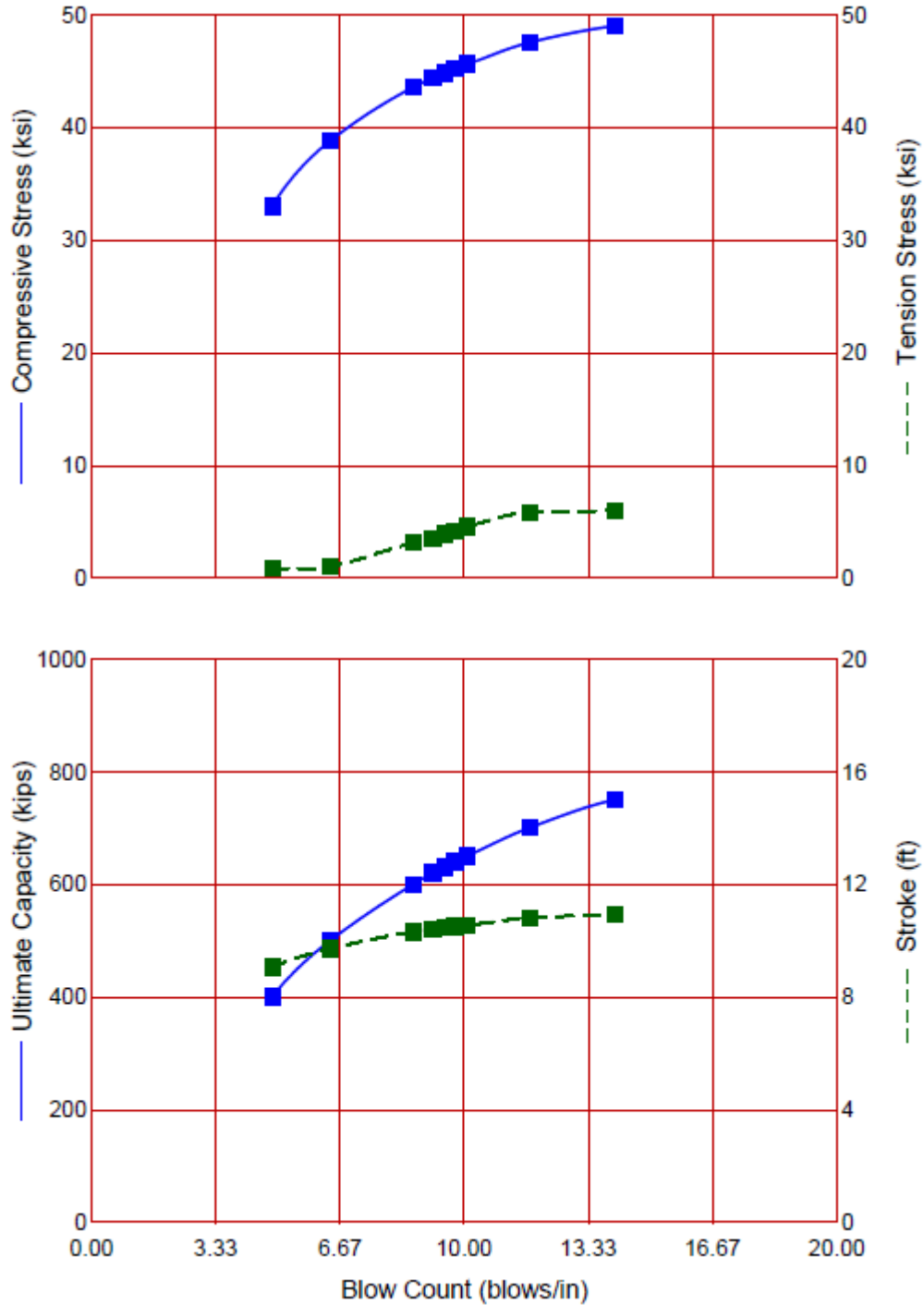
GRLWeap Soil and Pile Model Assumptions

Based on proposed bottom of footing of elevation 266 at Abutment 2, estimated pile lengths will be approx. 20 ft. Assume contractor drives pile lengths of 25 ft (extra length accommodates for attachment of dynamic testing equipment, embedment into abutment, variation in bedrock surface).

Use constant shaft resistances so that GRLWeap will assign approx. 100 kips 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 14 x 89

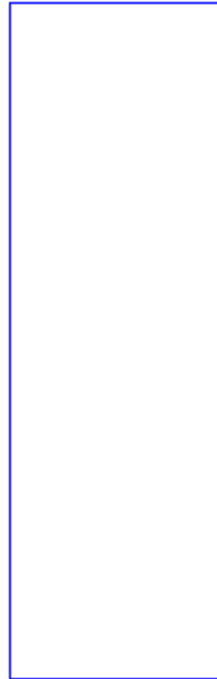
The 14x89 pile can be driven to the resistances below with a D 19-42 hammer at fuel setting 1 (100% of Max) and 1.9 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:



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	25.00 ft
Pile Penetration	20.00 ft
Pile Top Area	26.10 in ²

Pile Model



Skin Friction Distribution



Res. Shaft = 100.0 kips
(Constant Res. Shaft)

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	32.97	0.85	4.9	9.06	17.03
500.0	38.77	1.04	6.4	9.72	17.85
600.0	43.54	3.15	8.6	10.32	18.76
620.0	44.35	3.56	9.2	10.42	19.00
630.0	44.81	3.89	9.5	10.47	19.08
640.0	45.23	4.20	9.8	10.51	19.20
650.0	45.57	4.58	10.1	10.55	19.28
700.0	47.50	5.84	11.8	10.81	19.82
750.0	48.97	6.00	14.1	10.93	20.05

Limit stress to 45 ksi

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

Strength Limit State

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

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

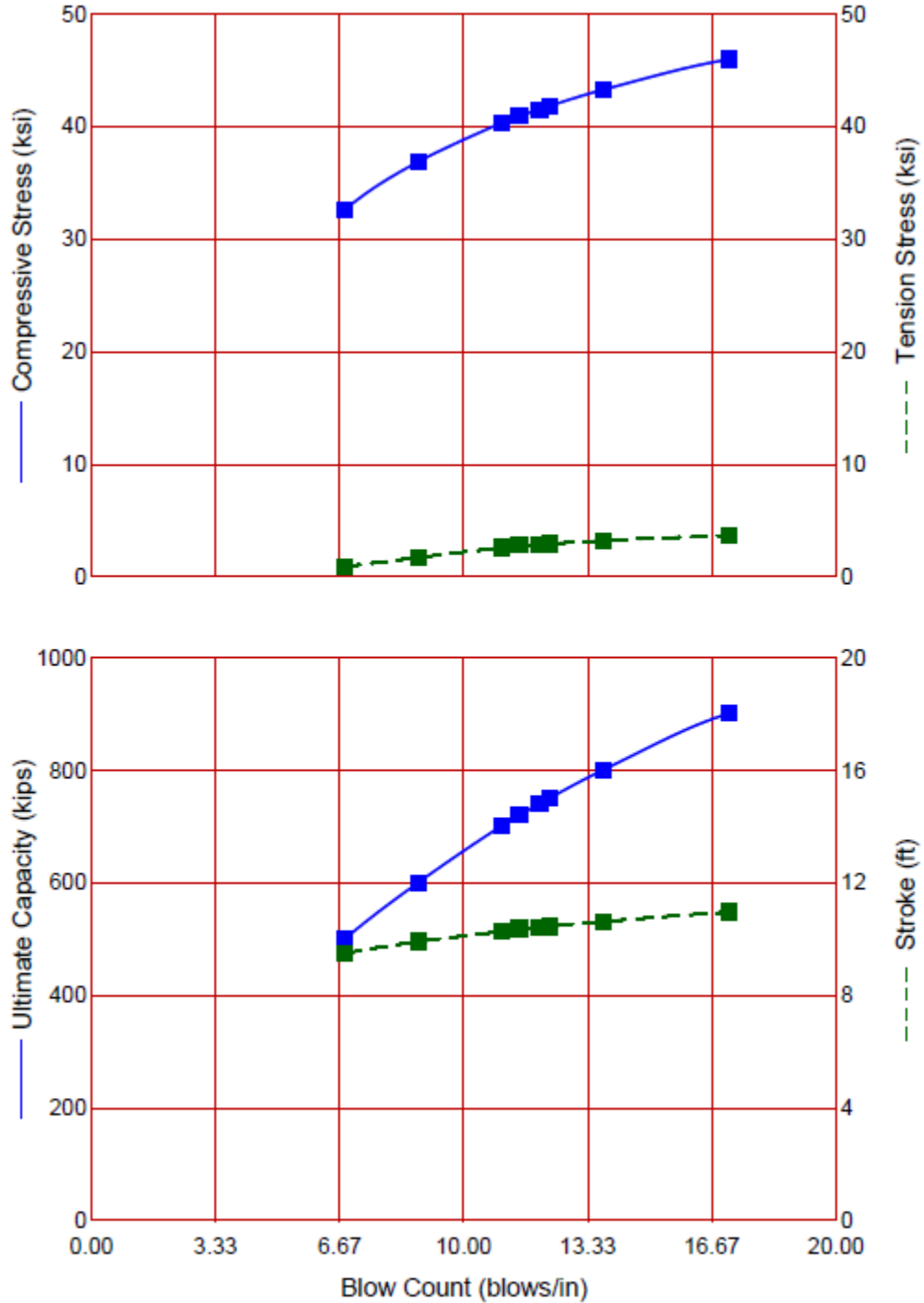
Extreme and
Service Limit States

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

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

Pile Size is 14 x 117

The 14x117 pile can be driven to the resistances below with a D 19-42 hammer at fuel setting 1 (100% of max) and 1.9 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:



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	25.00 ft
Pile Penetration	20.00 ft
Pile Top Area	34.40 in ²

Pile Model



Skin Friction Distribution



Res. Shaft = 100.0 kips
(Constant Res. Shaft)

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
500.0	32.58	0.90	6.8	9.49	17.32
600.0	36.83	1.73	8.8	9.91	18.20
700.0	40.27	2.60	11.0	10.27	18.94
720.0	40.96	2.80	11.5	10.36	19.12
740.0	41.46	2.85	12.0	10.41	19.23
750.0	41.78	2.94	12.3	10.45	19.31
800.0	43.23	3.22	13.8	10.62	19.67
900.0	45.91	3.68	17.1	10.95	20.43

Limit to 12 bpi

$$R_{ndr} := 740 \cdot \text{kip}$$

Strength Limit State

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

$$R_{fdr} = 481 \cdot \text{kip}$$

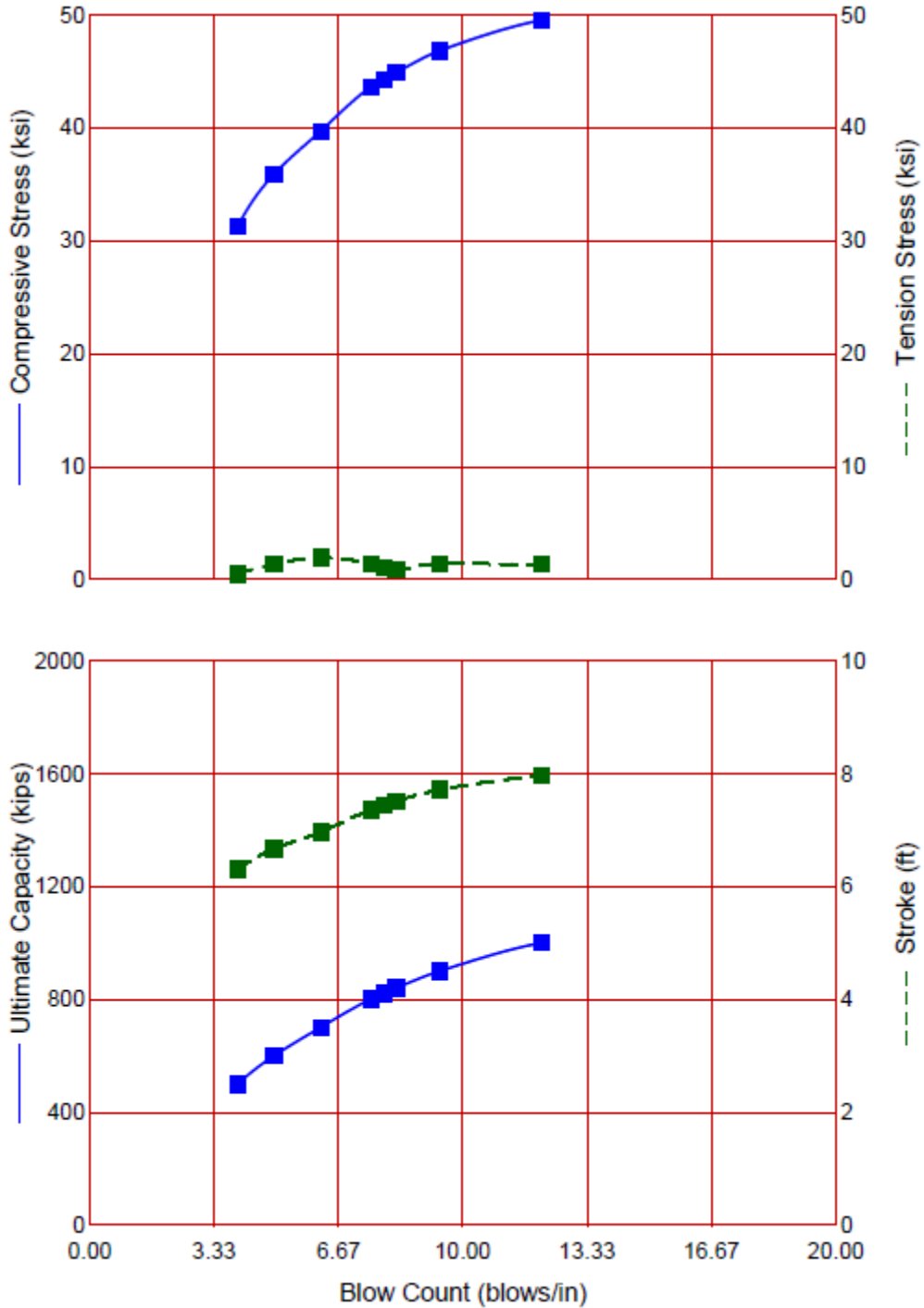
Extreme and Service Limit States

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

$$R_{dr} = 740 \cdot \text{kip}$$

Pile Size is 14 x 117

The 14x117 pile can be driven to the resistances below with a D 36-32 at fuel setting 4 hammer and 2.7 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below:



DELMAG D 36-32	
Ram Weight	7.93 kips
Efficiency	0.800
Pressure	1095 (73%) psi
Helmet Weight	2.70 kips
Hammer Cushion	109975 kips/in
COR of H.C.	0.800
Skin Quake	0.100 in
Toe Quake	0.040 in
Skin Damping	0.050 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	25.00 ft
Pile Penetration	20.00 ft
Pile Top Area	34.40 in ²

Pile Model



Skin Friction Distribution



Res. Shaft = 100.0 kips
(Constant Res. Shaft)

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
500.0	31.26	0.47	4.0	6.30	22.30
600.0	35.83	1.36	4.9	6.66	23.08
700.0	39.63	1.94	6.2	6.96	23.59
800.0	43.54	1.41	7.5	7.35	24.72
820.0	44.22	1.00	7.9	7.43	24.84
840.0	44.85	0.89	8.2	7.51	25.05
900.0	46.77	1.42	9.4	7.71	25.53
1000.0	49.49	1.38	12.1	7.96	26.03

Limit stress to 45 ksi

$$R_{ndr} := 840 \cdot \text{kip}$$

Strength Limit State

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

$$R_{fdr} = 546 \cdot \text{kip}$$

Extreme and Service
Limit States

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

$$R_{dr} = 840 \cdot \text{kip}$$

Rock Socketed H-Pile Resistance

Design of Rock Socketed H-piles

Reference:

1. AASHTO LRFD Bridge Design Specifications, 8th Ed. 2017
2. AASHTO Standard Specifications for Highway Bridges, 17th Ed. 2002

Assumptions:

1. For Design Purposes, use bedrock data from three UCS lab tests of bedrock core.
2. Compressive strength of grout 4,000 psi.
3. Piles are 14x89 or 14x117
4. End plate is at least 10-inch by 10-inch

Design Unconfined Compressive Strength of Bedrock

$$\sigma_{ur} := \frac{10577 \cdot \text{psi} + 5236 \cdot \text{psi} + 7047 \cdot \text{psi}}{3}$$

$$\sigma_{ur} = 7620 \cdot \text{psi}$$

$$\sigma_{ur} = 1097 \cdot \text{ksf}$$

1. Nominal and Factoral Bearing Resistance of Bedrock

$$\sigma_{ur} = 7620 \cdot \text{psi}$$

Compressive Strength of Intact Rock

$$q_p := 2.5 \sigma_{ur} = 2743.2 \cdot \text{ksf}$$

Nominal bearing resistance of bedrock
LRFD Article 10.8.3.5.4C-1

$$\phi_{stat} := 0.5$$

LRFD Article 10.5.5.2.4-1

$$q_r := q_p \cdot \phi_{stat} = 1372 \cdot \text{ksf}$$

Factored unit tip resistance

$$A_{\text{square}} := 10 \text{in} \cdot 10 \text{in} = 100 \cdot \text{in}^2$$

Area of Toe Plate

$$A_p := A_{\text{square}} - \pi \cdot 1 \text{in}^2 = 97 \cdot \text{in}^2$$

Area of Toe Plate less 2" hole for tremie pipe

$$R_{\text{bedrock}} := q_r \cdot A_p = 923 \cdot \text{kip}$$

Factored End Bearing Resistance of Pile

2. Nominal and Factoral Bearing Resistance of End Plate on Grout

12-inch by 12-inch End Plate

$f_c := 4000\text{psi}$	Compressive Strength of Grout
$\sigma_{ur} := (f_c)$	
$q_p := 2.5\sigma_{ur} = 1440\cdot\text{ksf}$	Nominal bearing resistance of bedrock LRFD Article 10.8.3.5.4C-1
$\phi_{stat} := 0.5$	LRFD Article 10.5.5.2.4-1
$q_r := q_p \cdot \phi_{stat} = 720\cdot\text{ksf}$	Factored unit tip resistance
$A_{\text{square}} := 12\text{in} \cdot 12\text{in} = 144\cdot\text{in}^2$	Area of Toe Plate
$A_p := A_{\text{square}} - \pi \cdot 1\text{in}^2 = 141\cdot\text{in}^2$	Area of Toe Plate less 2" hole for tremie pipe
$R_{\text{bedrock}} := q_r \cdot A_p = 704\cdot\text{kip}$	Factored End Bearing Resistance of Pile

10 inch by 10-inch End Plate

$f_c := 4000\text{psi}$	Compressive Strength of Grout
$\sigma_{ur} := (f_c)$	
$q_p := 2.5\sigma_{ur} = 1440\cdot\text{ksf}$	Nominal bearing resistance of bedrock LRFD Article 10.8.3.5.4C-1
$\phi_{stat} := 0.5$	LRFD Article 10.5.5.2.4-1
$q_r := q_p \cdot \phi_{stat} = 720\cdot\text{ksf}$	Factored unit tip resistance
$A_{\text{square}} := 10\text{in} \cdot 10\text{in} = 100\cdot\text{in}^2$	Area of Toe Plate
$A_p := A_{\text{square}} - \pi \cdot 1\text{in}^2 = 97\cdot\text{in}^2$	Area of Toe Plate less 2" hole for tremie pipe
$R_{\text{bedrock}} := q_r \cdot A_p = 484\cdot\text{kip}$	Factored End Bearing Resistance of Pile

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 269.0 at Abutment 1, Elevation 259 at Abutment 2.
- 2) MaineDOT Bridge Design Guide (BDG) Soil Type 4 will be used for integral abutment backfill.
- 3) Abutment 1 piles are rock-socketed H-Piles, Abutment 2 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.

Abutment 1

Soil Layer No. 1 Granular Borrow (New Construction) El. 278.0 - 269.0

Internal Angle of Friction

$$\phi_2 := 32$$

BDG Soil Type 4, BDG Table 3-3

Dry Unit Weight

$$\gamma_{1dry} := 125 \text{ pcf}$$

BDG Soil Type 4, BDG Table 3-3

Constant soil modulus with depth, k: Dense sand above water table for static loading = 225 pci

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Soil Layer No. 2 (Submerged, medium dense Sand) El. 269.0 - 264.0

Design $N_{60} = 15$ bpf

Sample
BB-WRRR-102;2D

Internal Angle of Friction

Peck, Hanson, Thornburn, 1974

$$\phi_2 := 32$$

Dry Unit Weight

Medium dense Sandy Gravel

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

$$\gamma_{1dry} := 122 \text{ pcf}$$

Natural water content at saturated state:

$$w_{1sat} := .11$$

sample BB-WRRR-102; 2D

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

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

Saturated Unit Weight

$$\gamma_{1\text{saturated}} = 135 \cdot \text{pcf}$$

Effective Unit Weight

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

$$\gamma'_1 := \gamma_{1\text{saturated}} - \gamma_w$$

$$\gamma'_1 = 73 \cdot \text{pcf}$$

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

Representative constant giving the variation of soil modulus with depth, k:
Medium sand below water table for static loading = 60 pci

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Soil Layer No. 3 (Submerged, Glacial Till) El. 264.0 - 258.9

Design $N_{60} = 131$ bpf

Sample BB-WRRR-102;3D

Internal Angle of Friction

$$\phi_3 := 43$$

Peck, Hansen, Thornburn, 1974

Dry Unit Weight

Dry, Dense Glacial Till = 134 pcf

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

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

Saturated Unit Weight

Natural water content at saturated state:

$$w_{3\text{sat}} := .1387$$

Sample BB-WRRR-101;2D

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

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

$$\gamma_{3\text{saturated}} = 153 \cdot \text{pcf}$$

Soil Effective Unit Weight

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

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

$$\gamma'_3 = 90 \cdot \text{pcf}$$

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

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Abutment 2

Soil Layer No. 1 (New Compacted Granular Borrow) El. 274.3 - 266

Internal Angle of Friction

Soil Type 4, BDG Table 3-3

$$\phi_4 := 32$$

Dry Unit Weight

$$\gamma_{4dry} := 125 \text{pcf}$$

Soil Type 4, BDG Table 3-3

Representative constant giving the variation of soil modulus with depth, k: Dense sand above water table for static loading = 225 pci
--

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 p. 96
Soil Layer No. 2 (Stiff Silt) El. 266 - 261.5Design $N_{60} = 20$ bpf

Internal Angle of Friction

$$\phi_5 := 33$$

Peck, Hansen, Thornburn, 1974

Dry Unit Weight

$$\gamma_{5dry} := 112 \text{pcf}$$

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

Saturated Unit Weight

Natural water content at saturated state:

$$w_{5sat} := .113$$

sample BB-RRR-106;2D

$$\gamma_{5saturated} := \gamma_{5dry} \cdot (1 + w_{5sat})$$

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

$$\gamma_{5saturated} = 125 \cdot \text{pcf}$$

Soil Effective Unit Weight

$$\gamma'_5 := \gamma_{5saturated} - \gamma_w$$

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

$\gamma'_5 = 62 \cdot \text{pcf}$

Representative constant giving the variation of soil modulus with depth, k: Stiff silt below water table for static loading = 60 pci

Soil Layer No. 3 (Stiff Silt) El. 261.5 - 255.8Design $N_{60} = 15$ bpf

Sample BB-WRRR-106;3D

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Internal Angle of Friction

Peck, Hansen, Thornburn, 1974

 $\phi_6 := 31$

Dry Unit Weight

Dense angular silty sand = 121 pcf

Das, Principles of Geotechnical Eng. 7th Ed. p. 59:
Table 3.2 - dry unit weight $\gamma_{6dry} := 121$ pcf

Natural water content at saturated state:

 $w_{6sat} := .113$

sample BB-WRRR-106;3D

 $\gamma_{6saturated} := \gamma_{6dry} \cdot (1 + w_{6sat})$ Das, Principles of Geotechnical Eng. 7th Ed. p. 59:
Table 3.1 Unit Weight Relationships

Saturated Unit Weight

 $\gamma_{6saturated} = 135$ pcf $\gamma'_6 := \gamma_{6saturated} - \gamma_w$ $\gamma'_6 = 72$ pcf

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

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Soil Layer No. 4 (Hard Silt) El. 256 - 246Design $N_{60} = 65$ bpf

Sample BB-RRR-106;4D

Internal Angle of Friction

Peck, Hansen, and Thornburn, 1974

$$\phi_7 := 43$$

Dry Unit Weight

Glacial Till = 134 pcf

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

$$\gamma_{7\text{dry}} := 134 \text{ pcf}$$

Natural water content at saturated state:

$$w_{7\text{sat}} := .125$$

sample BB-WRRR-105;4D

Saturated Unit Weight

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

$$\gamma_{7\text{saturated}} := \gamma_{7\text{dry}} \cdot (1 + w_{7\text{sat}})$$

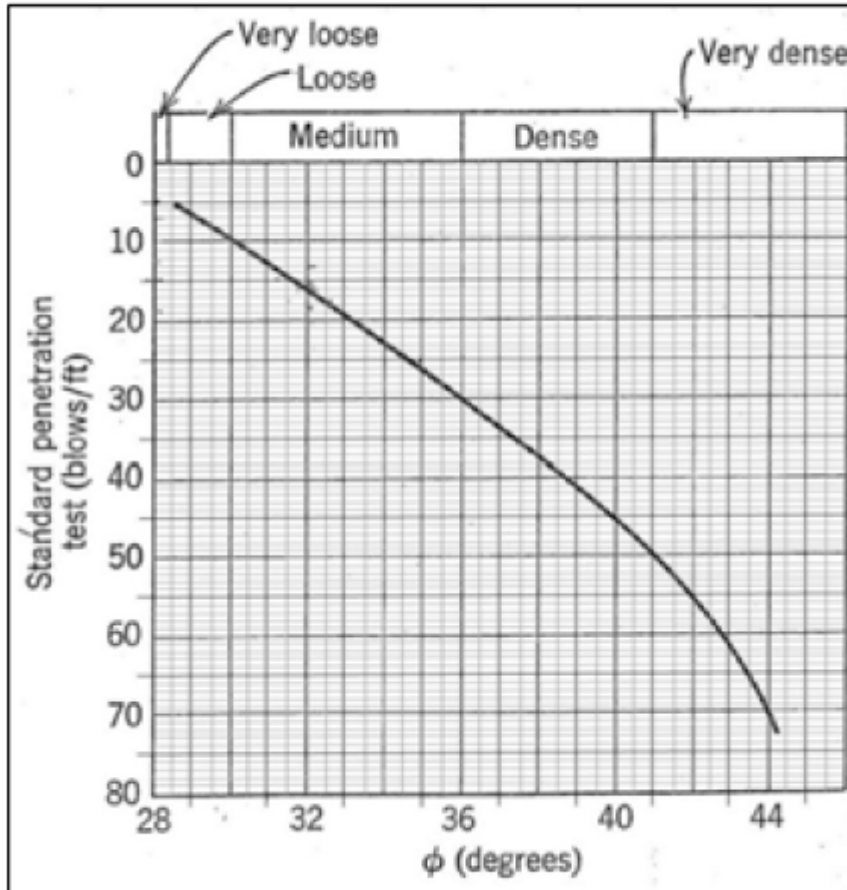
$$\gamma_{7\text{saturated}} = 151 \cdot \text{pcf}$$

$$\gamma'_7 := \gamma_{7\text{saturated}} - \gamma_w$$

$$\gamma'_7 = 88 \cdot \text{pcf}$$

Representative constant giving the variation of soil modulus with depth, k:
Medium sand above water table for static loading = 90 pciTechnical Manual
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(Peck, Hanson, and Thornburn, 1974)



Driller: MaineDOT	Elevation (ft.): 278.0	Auger ID/OD: 5" Solid Stem
Operator: Daggett/Wilder/Niles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: A. Van Buskirk	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 4/9/2019; 08:45-14:15	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 48+10.8, 7.5 ft Rt.	Casing ID/OD: NW-3"	Water Level*: 9.0 ft bgs.
Hammer Efficiency Factor: 0.928	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	

Definitions: R = Rock Core Sample S_U = Peak/Remolded Field Vane Undrained Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger S_{U(lab)} = Lab Vane Undrained Shear Strength (psf) WC = Water Content, percent
 MD = Unsuccessful Split Spoon Sample Attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw Field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample Attempt WOH = Weight of 140lb. Hammer Hammer Efficiency Factor = Rig Specific Annual Calibration Value PI = Plasticity Index
 V = Field Vane Shear Test, PP = Pocket Penetrometer WOR/C = Weight of Rods or Casing N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency G = Grain Size Analysis
 MV = Unsuccessful Field Vane Shear Test Attempt WO1P = Weight of One Person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected 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					
0										HMA and Concrete encountered. Thickness not recorded.		
5	1D	24/19	5.00 - 7.00	4/3/3/4	6	9				Brown, dry, loose, SAND, some silt, little gravel, (Fill). Layer 1 (El. 278.0-269.0)		
10	2D	24/15	10.00 - 12.00	3/2/2/4	4	6				Brown, damp, loose, SAND, some silt, some gravel, trace wood fibers, (Till Fill). Groundwater	G#337218 A-2-4, SM WC=13.87%	
15	3D	16.8/15	15.00 - 16.40	11/18/68(4.8")	---	94		264.0		Brown, damp, very dense, Silty SAND, little decomposed rock, (Glacial Till). Layer 2 (El. 269.0-264.0)		
20	MD R1	.08/0 60/60	19.00 - 19.01 19.10 - 24.10	30(1") RQD = 45%	---	NQ-2		258.9		Brown, damp, very dense, Silty SAND, little decomposed rock, (Glacial Till). a119 blows for 0.1 ft. NW Casing REFUSAL at 17.1 ft bgs. Roller Coned ahead to 19.0 ft bgs. Layer 3 (El. 264.0-258.9)	GTX 20.0-20.4 q _p = 10577 psi	
25	R2	63.6/63.6	24.10 - 29.40	RQD = 81%						Top of Bedrock at Elev. 259.2 ft. R1: Bedrock: Grey, fine grained, PHYLLITE, hard, slightly weathered, vertical to steep joints, moderately closely spaced, tight, no infilling. [Mayflower Hill Formation] Rock Quality = Poor R1: Core Times (min:sec) 19.1-19.6 ft (not recorded) 19.6-20.1 ft (1:43) 20.1-21.1 ft (3:09) 21.1-22.1 ft (3:09) 22.1-23.1 ft (4:37) core blocked at 22.0 ft bgs. Layer 4 (El. 258.9-)		

Remarks:

Driller: MaineDOT	Elevation (ft.): 274.3	Auger ID/OD: 5" Solid Stem
Operator: Daggett/Wilder/Niles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: A. Van Buskirk	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 4/4/2019; 08:45-13:30	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 51+46.5, 8.4 ft Lt.	Casing ID/OD: NW-3"	Water Level*: None Observed
Hammer Efficiency Factor: 0.928	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	

Definitions: R = Rock Core Sample S_U = Peak/Remolded Field Vane Undrained Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger S_{U(lab)} = Lab Vane Undrained Shear Strength (psf) WC = Water Content, percent
 MD = Unsuccessful Split Spoon Sample Attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw Field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample Attempt WOH = Weight of 140lb. Hammer Hammer Efficiency Factor = Rig Specific Annual Calibration Value PI = Plasticity Index
 V = Field Vane Shear Test, PP = Pocket Penetrometer WOR/C = Weight of Rods or Casing N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency G = Grain Size Analysis
 MV = Unsuccessful Field Vane Shear Test Attempt WO1P = Weight of One Person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected 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					
0								SSA	273.9 273.6	5" HMA. 4" CONCRETE.		
5	1D	24/14	5.00 - 7.00	3/3/3/6	6	9				Brown, moist, loose, Silty SAND, little gravel, (Fill).		
10	2D	24/15	10.00 - 12.00	3/6/7/7	13	20	16			Brown, moist, medium dense, Sandy GRAVEL, some silt, (Till Fill).	G#337222 A-1-b, GM WC=11.3%	
15	3D	24/13	15.00 - 17.00	6/4/6/8	10	15	19			Brown, moist, stiff, Sandy SILT, trace gravel, (Till Fill).		
20	4D	24/16	20.00 - 22.00	15/21/21/24	42	65	48		255.8	Olive, moist, hard, SILT, some sand, some gravel, (Glacial Till).	G#337223 A-4, SM WC=12.5%	
25							76					

Remarks:

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Armstrong Road Bridge #5815 over Interstate 95 Location: Waterville, Maine	Boring No.: BB-WRRR-106 WIN: 23505.00
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Driller: MaineDOT	Elevation (ft.): 274.3	Auger ID/OD: 5" Solid Stem
Operator: Daggett/Wilder/Niles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: A. Van Buskirk	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 4/4/2019; 08:45-13:30	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 51+46.5, 8.4 ft Lt.	Casing ID/OD: NW-3"	Water Level*: None Observed

Hammer Efficiency Factor: 0.928	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	
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 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	5D	24/19	25.00 - 27.00	16/23//25/27	48	74	RC			Olive-brown, moist, hard, Sandy SILT, little gravel, weathered rock, (Glacial Till).		
	R1	60/60	28.50 - 33.50	RQD = 40%			NQ-2	246.1		Roller Coned ahead to 28.5 ft bgs.		
										Top of Bedrock at Elev. 246.1 ft.		
30										R1: Bedrock: Grey, fine grained, SLATE, hard, moderately weathered, vertical to steep joints, near vertical fault filled with quartz, many nearly horizontal quartz veins, spacing close to moderately close, open, silt infilled, oxidized. [Mayflower Hill Formation] Rock Quality = Poor		
	R2	57.6/57.6	33.50 - 38.30	RQD = 19%						R1: Core Times (min:sec) 28.5-29.5 ft (3:36) 29.5-30.5 ft (2:58) 30.5-31.5 ft (2:42) 31.5-32.5 ft (2:19) 32.5-33.5 ft (3:28) 100% Recovery R2: Bedrock: Similar to R1, except more fragmented. Rock Quality = Very Poor		
35										R2: Core Times (min:sec) 33.5-34.5 ft (2:43) 34.5-35.5 ft (2:56) 35.5-35.8 ft (1:49) core blocked 35.8-36.5 ft (1:42) 36.5-37.5 ft (2:56) 37.5-38.3 ft (3:22) 100% Recovery		
40								236.0		Bottom of Exploration at 38.3 feet below ground surface.		
45												
50												

Remarks:

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	$\frac{(1 + w)G_s\gamma_w}{1 + e}$	γ, w	$\frac{\gamma}{1 + w}$	G_s, e	$\frac{(G_s + e)\gamma_w}{1 + e}$
S, G_s, e	$\frac{(G_s + Se)\gamma_w}{1 + e}$	G_s, e	$\frac{G_s\gamma_w}{1 + e}$	G_s, n	$[(1 - n)G_s + n]\gamma_w$
w, G_s, S	$\frac{(1 + w)G_s\gamma_w}{1 + \frac{wG_s}{S}}$	G_s, n	$G_s\gamma_w(1 - n)$	G_s, w_{sat}	$\left(\frac{1 + w_{sat}}{1 + w_{sat}G_s}\right)G_s\gamma_w$
w, G_s, n	$G_s\gamma_w(1 - n)(1 + w)$	G_s, w, S	$\frac{G_s\gamma_w}{1 + \left(\frac{wG_s}{S}\right)}$	e, w_{sat}	$\left(\frac{e}{w_{sat}}\right)\left(\frac{1 + w_{sat}}{1 + e}\right)\gamma_w$
S, G_s, n	$G_s\gamma_w(1 - n) + nS\gamma_w$	e, w, S	$\frac{eS\gamma_w}{(1 + e)w}$	n, w_{sat}	$n\left(\frac{1 + w_{sat}}{w_{sat}}\right)\gamma_w$
		γ_{sat}, e	$\gamma_{sat} - \frac{e\gamma_w}{1 + e}$	γ_d, e	$\gamma_d + \left(\frac{e}{1 + e}\right)\gamma_w$
		γ_{sat}, n	$\gamma_{sat} - n\gamma_w$	γ_d, n	$\gamma_d + n\gamma_w$
		γ_{sat}, G_s	$\frac{(\gamma_{sat} - \gamma_w)G_s}{(G_s - 1)}$	γ_d, S	$\left(1 - \frac{1}{G_s}\right)\gamma_d + \gamma_w$
				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

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, $\tan \delta$, 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

$$m = \frac{P_u - P_m}{y_u - y_m} \dots\dots\dots (3-62)$$

- b. Compute the power of the parabolic section using

$$n = \frac{P_m}{m y_m} \dots\dots\dots (3-63)$$

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

$$\bar{C} = \frac{P_m}{y_m^{1/n}} \dots\dots\dots (3-64)$$

8. Compute the y value defining point k using

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

Compute the p value defining point k using

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

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

ϕ_1 = Angle of internal friction

$\phi' = 32 \cdot \text{deg}$

β = Angle of back face of wall to the horizontal

$\beta := 90 \cdot \text{deg}$

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

For formed concrete IAB abutment against clean sand, silty sand-gravel mixture use $\delta = 17 - 22$, per

LRFD Table 3.11.5.3-1

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

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

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

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

$K_{p_coulomb} = 6.02$

Integral Abutment and Wingwall - Passive Earth Pressure - Rankine Theory

Use Rankine only if the ratio of wall height to wall movement is 0.005 or less and the fill slope is horizontal to the top of the wall. Bowles does not recommend use of Rankine method for K_p when $\alpha > 0$.

α = Angle of fill slope to the horizontal

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

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

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

$K_{p_rank} = 3.25$

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

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

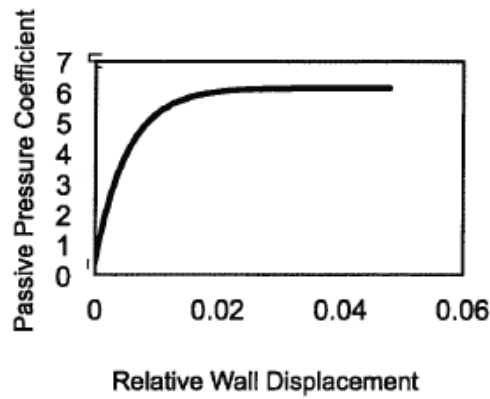


Figure 3.10.8-1: Plot of Passive Pressure Coefficient, K, vs. Relative Wall Displacement, δ_T/H .

Based on an estimated Relative Wall Displacement of 0.02:

$$K := 0.43 + 5.7 \cdot (1 - \exp(-3.42))$$

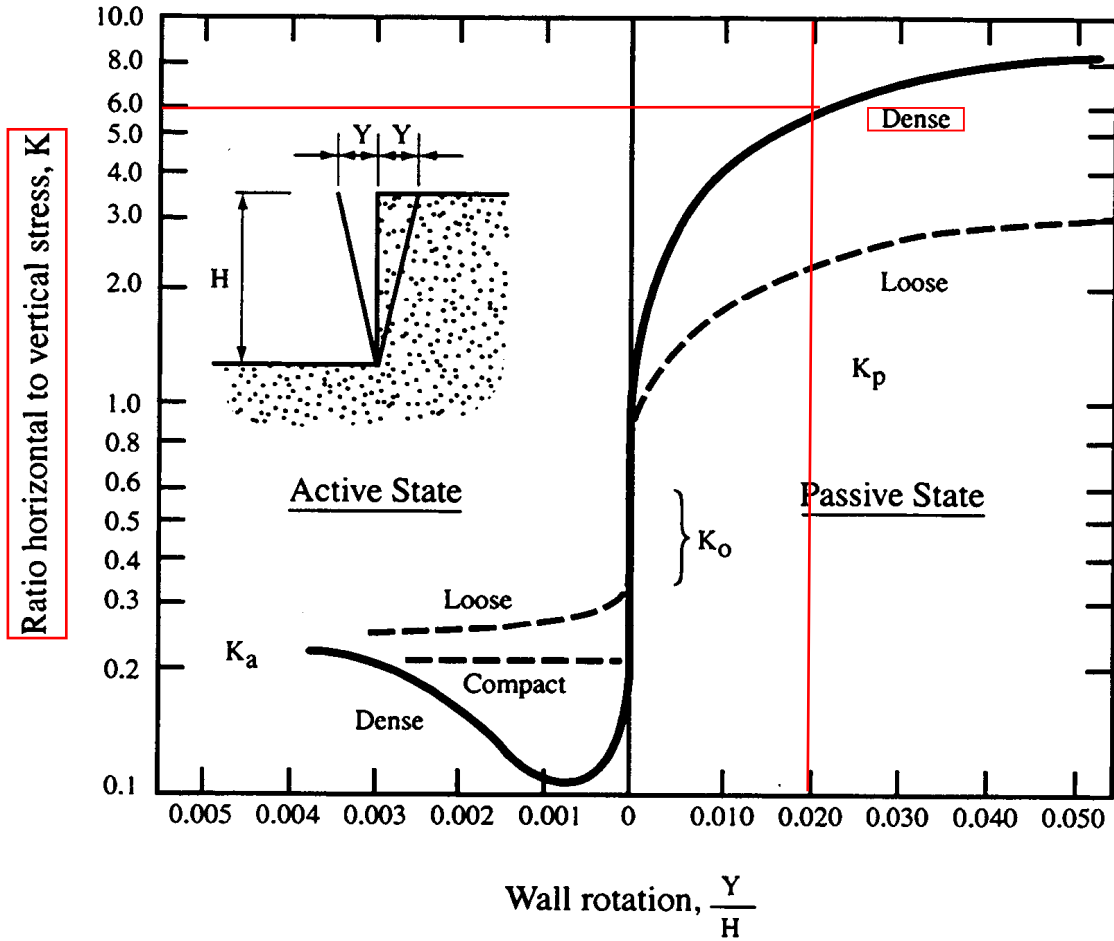
$$K = 5.94$$

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

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

$$K_o = 0.47$$

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



Magnitude of Wall Rotation to Reach Failure

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

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

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

3.11.5.4—Passive Lateral Earth Pressure Coefficient, k_p

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

For cohesive soils, passive pressures may be estimated by:

C3.11.5.4

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

Table 7.9 (Continued)

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

7.12 Coulomb's Passive Earth Pressure

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

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

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

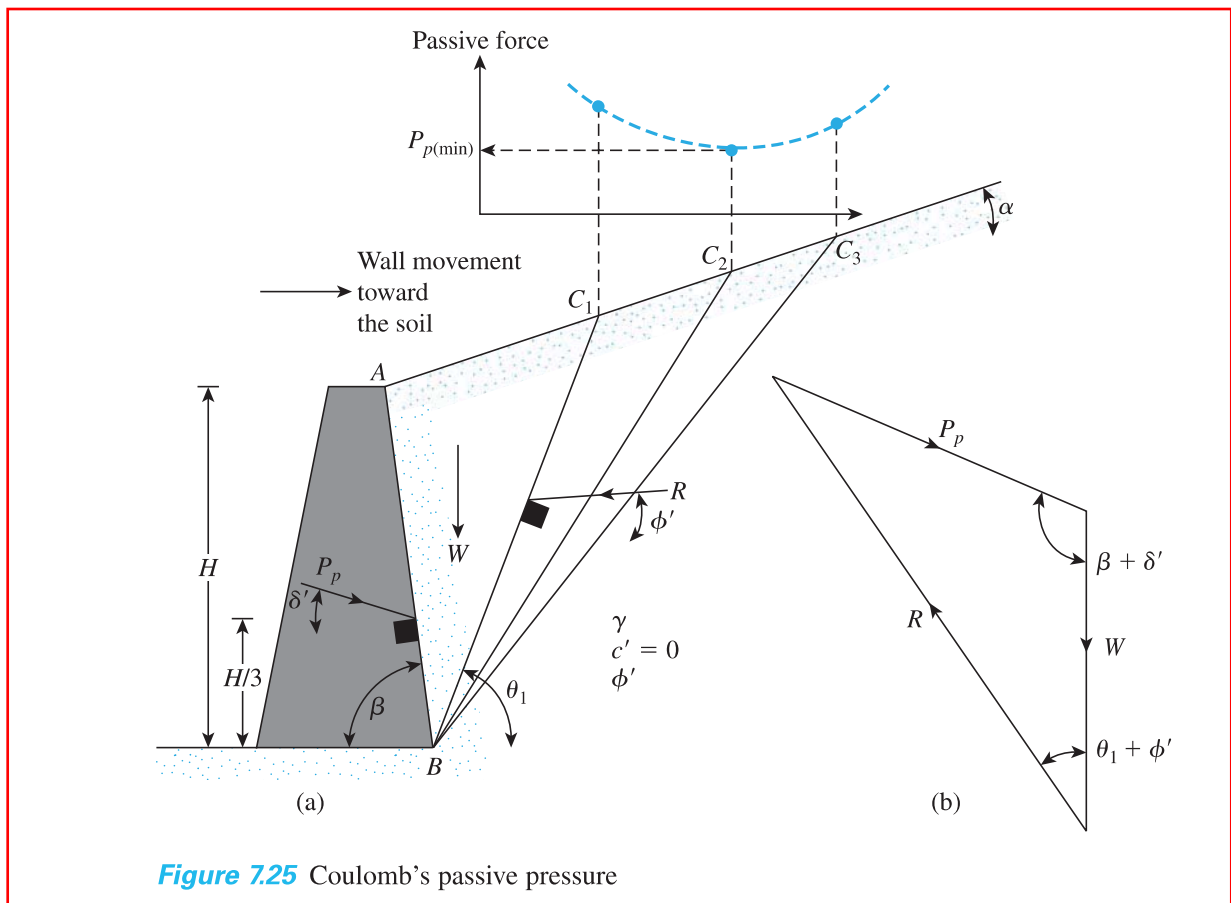


Figure 7.25 Coulomb's passive pressure

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

ϕ' (deg)	δ' (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 $ABC_1, ABC_2, ABC_3, \dots$, can be constructed, and the corresponding values of P_p can be determined. The top part of Figure 7.25a shows the nature of variation of the P_p values for different wedges. The *minimum value of P_p* in this diagram is *Coulomb's passive force*, mathematically expressed as

$$P_p = \frac{1}{2} \gamma H^2 K_p \quad (7.70)$$

where

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

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

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

7.13

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

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

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

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

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

Again, at $z = 3$ m,

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

Hence,

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

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, $u = (1)(\gamma_w) = 9.81 \text{ kN/m}^2$.

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
		$P_p \approx 223.3 \text{ kN/m}$

7.11

Rankine Passive Earth Pressure: Vertical Backface and Inclined Backfill

Granular Soil

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

$$\sigma'_p = \gamma z K_p \quad (7.65)$$

and the passive force is

$$P_p = \frac{1}{2} \gamma H^2 K_p \quad (7.66)$$

where

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

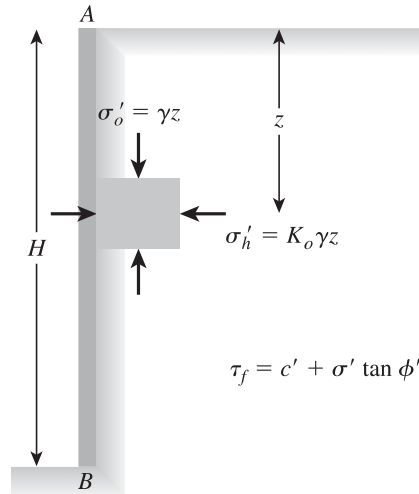


Figure 13.3 Earth pressure at rest

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

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

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)

Pier Footing Resistance

Analysis

Calculation of nominal and factored bearing resistance of bedrock using Rock Mass Rating (RMR) based empirical correlation

Method

Use data from boring and calculate the nominal bearing resistance as follows:

1. Estimation of Rock Mass Rating
2. Determine rock property constants s and m
3. Calculate nominal bearing resistance of bedrock, q_n , using RMR method in Wylie "Foundations on Rock"/AASHTO (2012) LRFD 10.4.6.4 - Rock Mass Strength

References

1. AASHTO LRFD Bridge Design Specifications, 8th Ed, 2017, (C10.4.6.4 and 10.6.2.6.2)
2. AASHTO LRFD Bridge Design Specifications, 2012, RMR, Table 10.4.6.4-1
3. AASHTO Standard Specifications for Highway Bridges, 17th Ed. 2002
4. Wylie, Duncan C, "Foundations on Rock", Second Edition, 2009.
5. "The Hoek-Brown Failure Criterion - A 1988 Update", E. Hoek and E.T. Brown

A. Design Bedrock Properties

Mayflower Hill Formation is generally characterized by thick beds of Wacke (Sandstone) and thinner beds of Phyllite.

Model bedrock at pier based on rock encountered in boring BB-WRRR-104, R1:

Dark grey, very fine grained, SANDSTONE, hard, fresh, near vertical joints, tightly bedded, close to medium close spacing, tight, no infilling, RQD = 78%,

and boring BB-WRRR-103, R1: Dark grey, fine grained, SILTSTONE, hard, fresh, horizontal to steep joint sets, close to medium close, tight, no infilling, RQD - 83%.

Compressive Strength

Lithified arcillaceous rock - Siltstone. (ROCK TYE B) and Arenaceous Rock - Sandstone (ROCK TYPE C)

From laboratory testing: 5,236 psi

$$q_{uc1} := 5236 \text{ psi}$$

$$q_{uc1} = 754 \cdot ksf$$

B. Pier Spread Footings

Determination of Rock Mass Rating (RMR) from LRFD (2012) Table 10.4.6.4-1 Geomechanics Classification of Rock Mass

Use RMR to supplement engineering judgment on rock competency according to LRFD 10.6.3.2.1. RMR is determined from the sum of five relative ratings listed in LRFD (2012) Table 10.4.6.4-1

1. Strength of intact rock

$q_{u1} = 754$ ksf

From LRFD Table 10.4.6.4-1 for Uniaxial compressive strength = 520-1080 ksf **Relative Rating = 4**

2. Drill Core Quality

Bedrock RQD = 83% (good) From LRFD Table 10.4.6.4-1, RQD 75% to 90%; **Relative Rating = 17**

3. Spacing of joints

Joints of intact bedrock are close to moderately close (2 in. - 3 ft).

From LRFD Table 10.4.6.4-1 Spacing of joints 2 in. - 1 ft; **Relative Rating = 10**

4. Condition of joints

Break surfaces are tight with no infilling; **Relative Rating = 20**

5. Groundwater conditions

<400 gal/hr, most only; **Relative Rating = 7**

6. From LRFD Table 10.4.6.4-2 Geomechanics Rating Adjustment for Joint Orientations

Near vertical (85-90 degrees); **Relative Rating = -7**

ADJUSTED RMR

$RMR := 4 + 17 + 10 + 20 + 7 - 7$

RMR = 51

Determine Rock Type for LRFD Table 10.4.6.4-4

Rock Type - B = Arenaceous rocks.

Geomechanics Rock Mass Class Determined from Total Rating

From AASHTO LRFD 2012 Table 10.4.6.4-3, RMR = 51 is Class No. III and described as Fair rock.

C. Rock Property Constants s and m (Ref. #1 and Ref. #5)

$$\text{RMR} = 51$$

Direct calculation of m and s is required, Reference 5 (Hoek and Brown, 1988), Equations 18 and 19 and Table 1. Assume isotropic behavior caused by the number and inconsistency of closely spaced discontinuity sets where none is significantly weaker than the other.

For a disturbed rock mass: $m/m_i = \exp((\text{RMR}-100)/14)$

$$m_i = m \text{ for intact rock} \quad s = \exp((\text{RMR}-100)/6)$$

For Rock Type B for intact rock, $m_i = 10$ (Ref. # 4, Table 1):

$$m_i := 10$$

$$m := m_i \cdot \exp\left(\frac{\text{RMR} - 100}{14}\right) \quad \text{Equation 18, Ref. 5}$$

$$m = 0.302$$

$$s := \exp\left(\frac{\text{RMR} - 100}{6}\right) \quad \text{Equation 19, Ref. 5}$$

$$s = 0.000284$$

D. Nominal and Factored Bearing Resistance of Bedrock

Correction Factor for Foundation Shape, from Wyllie Table 5.4 Pg. 138 (Ref. #2)

$$C_{fl} := 1.0$$

Conservative selection of $C_{fl} = 1.00$ for $L/B > 6$

Nominal Bearing Resistance (Wyllie)

Reference #4: Wyllie "Foundations on Rock" Equation 5.4 Pg. 138

$$q_{n1} := C_{fl} \cdot \sqrt{s} \cdot q_{uc1} \cdot \left[1 + \sqrt{m \cdot \left(\frac{-1}{s} \right) + 1} \right]$$

$$q_{n1} = 68 \cdot \text{ksf}$$

Factored Bearing Resistances

Use a bearing resistance factor of 0.45 for Footings on Rock per LRFD Table 10.5.5.2.2-1

$$\phi_{bc} := 0.45$$

$$q_{rl} := q_{nl} \cdot \phi_{bc}$$

$$q_{rl} = 31 \cdot \text{ksf}$$

Strength Limit State

Recommendation: Use 31 ksf for Pier Factored Bearing Resistance at the Strength Limit State

Factored Bearing Resistance

Use a bearing resistance factor of 0.80 LRFD 11.5.8 consistent with the design objective of no collapse of any substructure (consistency between piers and abutments).

$$\phi_{rec} := 0.8$$

$$q_{rl} := q_{nl} \cdot \phi_{rec}$$

$$q_{rl} = 54 \cdot \text{ksf}$$

Extreme Limit State

Recommendation: 54 ksf for Pier Factored Bearing Resistance at the Extreme Limit State

Verify Nominal Bearing Resistance per Carter and Kulhawy (1988)

Reference : NCHRP, Report 651, LRFD Design and Construction of Shallow Foundations for Highway Bridge Structures, pg 40, Eq. 82b, and referred to in LRFD C.10.6.3.2.2. Same equation.

$$q_{nl} := q_{uc1} \cdot \left[\sqrt{s} + \sqrt{m \cdot (\sqrt{s}) + s} \right]$$

$$q_{nl} = 68 \cdot \text{ksf}$$

Analysis

Calculation of nominal and factored bearing resistance on bedrock for Service Limit State Analysis

Method 1

Per AASHTO LRFD 10.6.2.4.4 - Settlement of Footings on Rock, "For footings bearing on fair to very good rock according to Geomechanics Classification system (i.e. RMR), as defined in Article 10.4.6.4, and designed in accordance with the provisions of this Section, **elastic settlement may generally be assumed to be less than 0.5 inch.**"

Method 2

LRFD Table C10.6.2.6.1-1, Presumptive Bearing Resistance for Spread Footings at the Service Limit State, based on *NavFac DM 7.2, May 1983, Foundations and Earth Structures*, Table 1, 7.2-142, "Presumptive Values of Allowable Bearing Pressures for Spread Foundations".

Bearing Material: Sedimentary rock: shales, siltstone, sandstone.	
Consistency in Place:	Hard sound rock
Allowable Bearing Pressure	Range: 30-50 ksf
AASHTO Recommended Value	40 ksf
MaineDOT Recommended Value	$q_{1nominal} := 30 \cdot \text{ksf}$

Resistance Factor for Service Limit State

$$\phi_r := 1.0$$

Per LRFD Article C10.6.2.6.1, when using presumptive bearing resistance values for the factored bearing resistance for Service Limit State Analyses, settlement is typically limited to 1 inch

$$q_{\text{factored}} := \phi_r \cdot q_{1nominal}$$

$$q_{\text{factored}} = 30 \cdot \text{ksf} \quad \text{Service Limit State}$$

Recommendation: Use 30 ksf at Pier Factored Bearing Resistance at the Service Limit State

Table 10.4.6.4-1—Geomechanics Classification of Rock Masses

Parameter		Ranges of Values							
1	Strength of intact rock material	Point load strength index	>175 ksf	85–175 ksf	45–85 ksf	20–45 ksf	For this low range, uniaxial compressive test is preferred		
		Uniaxial compressive strength	>4320 ksf	2160–4320 ksf	1080–2160 ksf	520–1080 ksf	215–520 ksf	70–215 ksf	20–70 ksf
	Relative Rating	15	12	7	4	2	1	0	
2	Drill core quality RQD	90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%			
	Relative Rating	20	17	13	8	3			
3	Spacing of joints	>10 ft	3–10 ft	1–3 ft	2 in.–1 ft	<2 in.			
	Relative Rating	30	25	20	10	5			
4	Condition of joints	<ul style="list-style-type: none"> • Very rough surfaces • Not continuous • No separation • Hard joint wall rock 	<ul style="list-style-type: none"> • Slightly rough surfaces • Separation <0.05 in. • Hard joint wall rock 	<ul style="list-style-type: none"> • Slightly rough surfaces • Separation <0.05 in. • Soft joint wall rock 	<ul style="list-style-type: none"> • Slicken-sided surfaces or • Gouge <0.2 in. thick or • Joints open 0.05–0.2 in. • Continuous joints 	<ul style="list-style-type: none"> • Soft gouge >0.2 in. thick or • Joints open >0.2 in. • Continuous joints 			
	Relative Rating	25	20	12	6	0			
5	Groundwater conditions (use one of the three evaluation criteria as appropriate to the method of exploration)	Inflow per 30 ft tunnel length	None	<400 gal./hr.	400–2000 gal./hr.	>2000 gal./hr.			
		Ratio = joint water pressure/major principal stress	0	0.0–0.2	0.2–0.5	>0.5			
	General Conditions	Completely Dry	Moist only (interstitial water)	Water under moderate pressure	Severe water problems				
	Relative Rating	10	7	4	0				

Table 10.4.6.4-2—Geomechanics Rating Adjustment for Joint Orientations

Strike and Dip Orientations of Joints		Very Favorable	Favorable	Fair	Unfavorable	Very Unfavorable
Ratings	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

Table 10.4.6.4-3—Geomechanics Rock Mass Classes Determined from Total Ratings

RMR Rating	100-81	80-61	60-41	40-21	<20
Class No.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

The shear strength of fractured rock masses should be evaluated using the Hoek and Brown criteria, in which the shear strength is represented as a curved envelope that is a function of the uniaxial compressive strength of the intact rock, q_u , and two dimensionless constants m and s . The values of m and s as defined in Table 10.4.6.4-4 should be used.

The shear strength of the rock mass should be determined as:

$$\tau = (\cot \phi'_i - \cos \phi'_i) m \frac{q_u}{8} \quad (10.4.6.4-1)$$

in which:

$$\phi'_i = \tan^{-1} \left\{ 4h \cos^2 \left[30 + 0.33 \sin^{-1} \left(\frac{-3}{h^2} \right) \right] - 1 \right\}^{-1/2}$$

$$h = 1 + \frac{16(m\sigma'_n + sq_u)}{(3m^2q_u)}$$

where:

- τ = the shear strength of the rock mass (ksf)
- ϕ'_i = the instantaneous friction angle of the rock mass (degrees)
- q_u = average unconfined compressive strength of rock core (ksf)
- σ'_n = effective normal stress (ksf)
- m, s = constants from Table 10.4.6.4-4 (dim)

This method was developed by Hoek (1983) and Hoek and Brown (1988, 1997). Note that the instantaneous cohesion at a discrete value of normal stress can be taken as:

$$c_i = \tau - \sigma'_n \tan \phi'_i \quad (C10.4.6.4-1)$$

The instantaneous cohesion and instantaneous friction angle define a conventional linear Mohr envelope at the normal stress under consideration. For normal stresses significantly different than that used to compute the instantaneous values, the resulting shear strength will be unconservative. If there is considerable variation in the effective normal stress in the zone of concern, consideration should be given to subdividing the zone into areas where the normal stress is relative constant and assigning separate strength parameters to each zone. Alternatively, the methods of Hoek (1983) may be used to compute average values for the range of normal stresses expected.

Table 10.4.6.4-4—Approximate Relationship between Rock-Mass Quality and Material Constants Used in Defining Nonlinear Strength (Hoek and Brown, 1988)

Rock Quality	Constants	Rock Type				
		A = Carbonate rocks with well developed crystal cleavage— <i>dolomite, limestone and marble</i> B = Lithified argillaceous rocks— <i>mudstone, siltstone, shale and slate (normal to cleavage)</i> C = Arenaceous rocks with strong crystals and poorly developed crystal cleavage— <i>sandstone and quartzite</i> D = Fine grained polyminerallic igneous crystalline rocks— <i>andesite, dolerite, diabase and rhyolite</i> E = Coarse grained polyminerallic igneous & metamorphic crystalline rocks— <i>amphibolite, gabbro gneiss, granite, norite, quartz-diorite</i>				
		A	B	C	D	E
INTACT ROCK SAMPLES Laboratory size specimens free from discontinuities. CSIR rating: <i>RMR</i> = 100	<i>m</i>	7.00	10.00	15.00	17.00	25.00
	<i>s</i>	1.00	1.00	1.00	1.00	1.00
VERY GOOD QUALITY ROCK MASS Tightly interlocking undisturbed rock with unweathered joints at 3–10 ft CSIR rating: <i>RMR</i> = 85	<i>m</i>	2.40	3.43	5.14	5.82	8.567
	<i>s</i>	0.082	0.082	0.082	0.082	0.082
GOOD QUALITY ROCK MASS Fresh to slightly weathered rock, slightly disturbed with joints at 3–10 ft CSIR rating: <i>RMR</i> = 65	<i>m</i>	0.575	0.821	1.231	1.395	2.052
	<i>s</i>	0.00293	0.00293	0.00293	0.00293	0.00293
FAIR QUALITY ROCK MASS Several sets of moderately weathered joints spaced at 1–3 ft CSIR rating: <i>RMR</i> = 44	<i>m</i>	0.128	0.183	0.275	0.311	0.458
	<i>s</i>	0.00009	0.00009	0.00009	0.00009	0.00009
POOR QUALITY ROCK MASS Numerous weathered joints at 2 to 12 in.; some gouge. Clean compacted waste rock. CSIR rating: <i>RMR</i> = 23	<i>m</i>	0.029	0.041	0.061	0.069	0.102
	<i>s</i>	3×10^{-6}	3×10^{-6}	3×10^{-6}	3×10^{-6}	3×10^{-6}
VERY POOR QUALITY ROCK MASS Numerous heavily weathered joints spaced <2 in. with gouge. Waste rock with fines. CSIR rating: <i>RMR</i> = 3	<i>m</i>	0.007	0.010	0.015	0.017	0.025
	<i>s</i>	1×10^{-7}	1×10^{-7}	1×10^{-7}	1×10^{-7}	1×10^{-7}

Where it is necessary to evaluate the strength of a single discontinuity or set of discontinuities, the strength along the discontinuity should be determined as follows:

- For smooth discontinuities, the shear strength is represented by a friction angle of the parent rock material. To evaluate the friction angle of this type of discontinuity surface for design, direct shear tests on samples should be performed. Samples should be formed in the laboratory by cutting samples of intact core.
- For rough discontinuities the nonlinear criterion of Barton (1976) should be applied.

The range of typical friction angles provided in Table C10.4.6.4-1 may be used in evaluating measured values of friction angles for smooth joints.

order to permit construction of the models. Consequently, our ability to predict the strength of jointed rock masses on the basis of direct tests or of model studies is severely limited.

In searching for a solution to this problem in order to provide a basis for the design of underground excavations in rock, Hoek and Brown (1980a) felt that some attempt had to be made to link the constants m and s of their criterion to measurements or observations which could be carried out by any competent geologist in the field. Recognizing that the characteristics of the rock mass which control its strength and deformation behaviour are similar to the characteristics which had been adopted by Bieniawski (1974) and by Barton, Lien and Lunde (1974) for their rock mass classifications, Hoek and Brown (1980a) proposed that these rock mass classifications could be used for estimating the material constants m and s .

Because of the lack of suitable methods for estimating the strength of rock masses, the first table relating rock mass classifications to material properties published by Hoek and Brown (1980a) was widely accepted by the geotechnical community and has been used on a large number of projects. Experience gained from these applications showed that the estimated rock mass strengths were reasonable when used for slope stability studies in which the rock mass is usually disturbed and loosened by relaxation due to excavation of the slope. However, the estimated rock mass strengths generally appeared to be too low in applications involving underground excavations where the confining stresses do not permit the same degree of loosening as would occur in a slope.

In order to incorporate the lessons learned from practical applications, Brown and Hoek (1988) proposed a revised set of relationships between the rock mass rating (RMR) from Bieniawski's (1974) rock mass classification and the constants m and s . Following Priest and Brown (1983), the relationships were presented in the form of the following equations:

Disturbed rock masses :

$$\frac{m}{m_i} = \exp\left(\frac{\text{RMR} - 100}{14}\right) \quad (18)$$

$$s = \exp\left(\frac{\text{RMR} - 100}{6}\right) \quad (19)$$

Undisturbed or interlocking rock masses:

$$\frac{m}{m_i} = \exp\left(\frac{\text{RMR} - 100}{28}\right) \quad (20)$$

$$s = \exp\left(\frac{\text{RMR} - 100}{9}\right) \quad (21)$$

where

m and s are the rock mass constants and m_i is the value of m for the *intact* rock.

Equations 18 to 21 have been used to construct Table 1 which shows the approximate relationship between rock mass quality and the Hoek-Brown material constants. Note that the value of the Tunnelling Quality Index Q from the NGI rock mass classification by Barton, Lien and Lunde (1974) has been calculated from the relationship proposed by Bieniawski (1976) :

$$\text{RMR} = 9 \text{Log}_e Q + 44 \quad (22)$$

Limitations on using failure criterion

Figure 1 illustrates a jointed rock mass in to which a tunnel has been mined. The circles adjacent to the right hand wall of the tunnel enclose different rock mass volumes and the comments on the right hand side of the drawing indicate situations to which the Hoek-Brown failure criterion can be applied.

When the volume of rock under consideration is small enough that it does not contain any structural discontinuities, equation 1 can be applied, using the m and s values for *intact* rock. This condition would apply to small scale specimens which has been extracted for laboratory testing or to the analysis of concentrated forces such as those which may be exerted by an individual pick on a tunnel boring machine cutter.

When the volume of rock being considered is such that only a few structural discontinuities are contained in this volume, the Hoek-Brown criterion should not be used. The behaviour of this rock is likely to be highly anisotropic and the Hoek-Brown failure criterion, which is only applicable to isotropic rock, will give erroneous results.

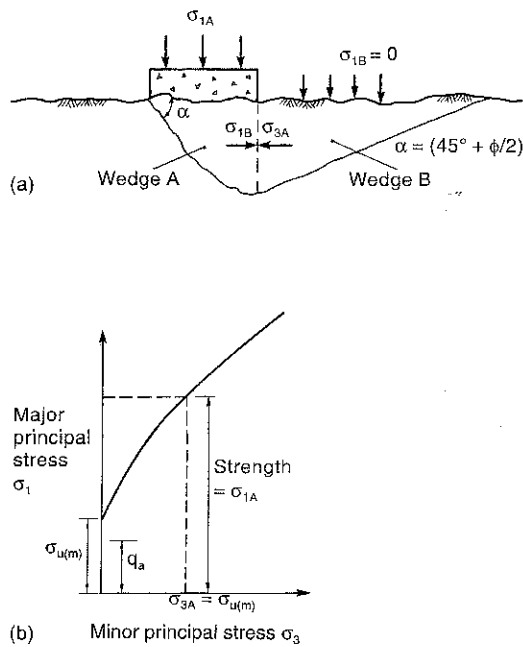


Figure 5.3 Analysis of bearing capacity of fractured rock: (a) active A and passive B wedges in foundation; and (b) curved rock mass strength envelope. Allowable bearing pressure = q_a , strength of bearing rock = σ_{1A} , factor of safety $FS = \sigma_{1A}/q_a$.

$$\sigma_1 = (m\sigma_{u(r)}(s\sigma_{u(r)}^2)^{1/2} + s\sigma_{u(r)}^2)^{1/2} + (s\sigma_{u(r)}^2)^{1/2}$$

$$= s^{1/2}\sigma_{u(r)}[1 + (ms^{-1/2} + 1)^{1/2}] \quad (5.3)$$

The plot in Fig. 5.3(b) shows the relationship between the strength σ_{1A} and the confining stresses provided by the surrounding rock σ_{3A} . This illustrates that a very significant increase in the bearing capacity is produced by a small increase in the confining pressure.

The allowable bearing pressure q_a is related to the rock mass strength by the factor of safety FS and the correction factor C_{f1} :

$$q_a = \frac{C_{f1}s^{1/2}\sigma_{u(r)}[1 + (ms^{-1/2} + 1)^{1/2}]}{FS} \quad (5.4)$$

The factor C_{f1} is applied to the calculated allowable bearing pressure to account for the shape of

the foundation and has the values given in Table 5.4 (Sowers, 1970).

A more comprehensive procedure for calculating the ultimate bearing capacity of fractured rock is described by Serrano and Olalla (1994) in which the rock mass strength is defined by the Hoek and Brown strength criteria as above. The method of analysis can accommodate recessed footings, inclined loads and foundations located on sloping ground surfaces.

For most loading conditions on sound rock the factor of safety will be in the range 2–3 for which there is little risk of settlement. A factor of safety of 3 is used for the dead load plus the maximum live load. If part of the live load is temporary such as wind and earthquake, then a factor of safety of 2 can be used (US Department of the Navy, 1982).

In the equations to calculate the allowable bearing capacity for a fractured rock mass with the strength defined by curved strength envelopes, it is important to distinguish between the compressive strength of the intact rock and that of the rock mass. The intact rock strength $\sigma_{u(r)}$ is determined from laboratory tests on rock cores, while for fractured rock the strength is defined by equation 5.1 with the degree of fracturing of the rock mass being accounted for by the constants m and s .

5.2.3 Recessed footings

In the case of a footing which is recessed into the rock surface, it is necessary to modify equation 5.4 to account for the increase in the stress σ_{1s} as a result of the confining stress q_s applied at the ground surface. That is, the minor principal stress

Table 5.4 Correction factors for foundation shapes (L = length, B = width)

Foundation shape	C_{f1}	C_{f2}
Strip ($L/B > 6$)	1.0	1.0
Rectangular		
$L/B = 2$	1.12	0.9
$L/B = 5$	1.05	0.95
Square	1.25	0.85
Circular	1.2	0.7

Table C10.6.2.6.1-1—Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	120–200	160
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	60–80	70
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	30–50	40
Weathered or broken bedrock of any kind, except highly argillaceous rock (shale)	Medium hard rock	16–24	20
Compaction shale or other highly argillaceous rock in sound condition	Medium hard rock	16–24	20
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very dense	16–24	20
Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)	Very dense	12–20	14
	Medium dense to dense	8–14	10
	Loose	4–12	6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense	8–12	8
	Medium dense to dense	4–8	6
	Loose	2–6	3
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense	6–10	6
	Medium dense to dense	4–8	5
	Loose	2–4	3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense	6–10	6
	Medium dense to dense	4–8	5
	Loose	2–4	3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense	6–12	8
	Medium dense to dense	2–6	4
	Loose	1–2	1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard	4–8	6
	Medium stiff to stiff	2–6	3
	Soft	1–2	1

10.6.2.6.2—Semiempirical Procedures for Bearing Resistance

Bearing resistance on rock shall be determined using empirical correlation to the Geomechanic Rock Mass Rating System, RMR. Local experience should be considered in the use of these semi-empirical procedures.

If the recommended value of presumptive bearing resistance exceeds either the unconfined compressive strength of the rock or the nominal resistance of the concrete, the presumptive bearing resistance shall be taken as the lesser of the unconfined compressive strength of the rock or the nominal resistance of the concrete. The nominal resistance of concrete shall be taken as $0.3f'_c$.

Frost Depth

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

From Design Freezing Index Map: **Waterville, Maine**

DFI = 1600 degree-days.

Case 1 - coarse grained granular fill soils W=15% (assumed).

For DFI = 1600

at w=20% $d_1 := 70.2\text{in}$

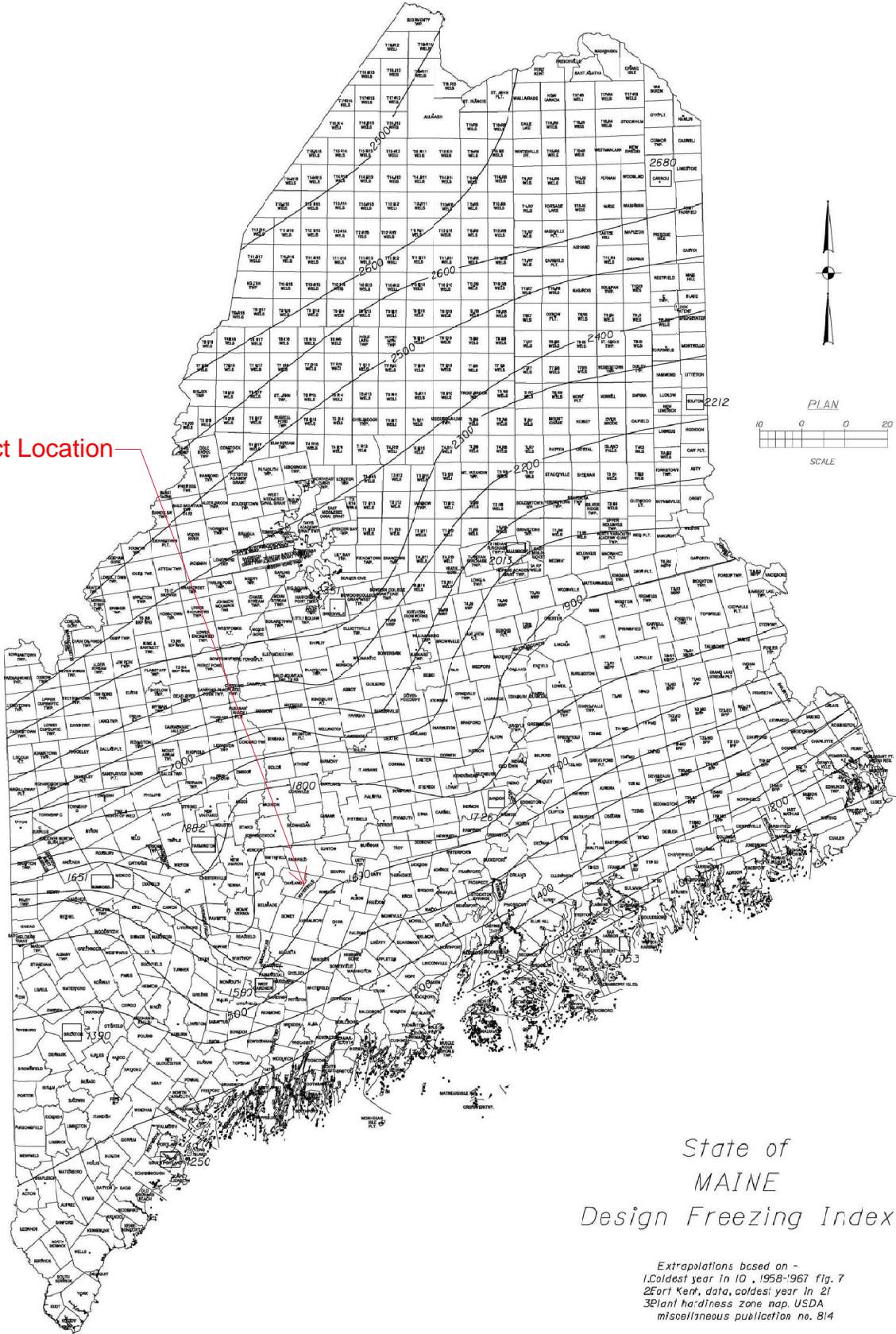
at w=10% $d_2 := 84.8\text{in}$

Depth of Frost Penetration

$$d := \frac{d_2 + d_1}{2} \quad d = 77.5\text{in} \quad d = 6.5\text{ft}$$

Project Location

Figure 5-1 Maine Design Freezing Index Map



State of
MAINE
Design Freezing Index

Extrapolations based on -
1) Coldest year in 10, 1958-1967 fig. 7
2) Fort Kent, data, coldest year in 21
3) Plant hardiness zone map, USDA
miscellaneous publication no. 814

5.2 General

MaineDOT Bridge Design Guide

5.2.1 Frost

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

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

Table 5-1 Depth of Frost Penetration

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

Seismic Design

BB-WRRR-101			
Depth	N ₆₀	di	di/N
5	9	7	0.78
10	6	7	1.17
15	100	5	0.05
19.1	100	81	0.81
SUM		100	2.80

di/di/N 35.66

BB-WRRR-105			
Depth	N ₆₀	di	di/N
5	20	7	0.35
10	19	8	0.42
15	15	5	0.33
20	65	10	0.15
25	100	70	0.70
SUM		100	1.96

di/di/N 51.07

BB-WRRR-102			
Depth	N ₆₀	di	di/N
5	9	7	0.78
10	15	8	0.53
15	100	5	0.05
20.2	100	80	0.80
SUM		100	2.16

di/di/N 46.27

BB-WRRR-106			
Depth	N ₆₀	di	di/N
5	9	7	0.78
10	20	8	0.40
15	15	4.5	0.30
20	65	6.5	0.10
25	74	3	0.04
30	100	71	0.71
SUM		100	2.33

di/di/N 42.95

SUM	Nav.	43.99
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15 < N_{av.} < 50 bpf

Conclusion: Site Class D

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

Waterville Armstrong Road Bridge #5815

WIN 23505.00

September 24, 2020

Abutment No. 1 and 2 Seismic Parameters

Conterminous 48 States

2007 AASHTO Bridge Design Guidelines

AASHTO Spectrum for 7% PE in 75 years

Latitude = 44.566148

Longitude = -069.669273

Site Class B

Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.077	PGA - Site Class B
0.2	0.160	Ss - Site Class B
1.0	0.046	S1 - Site Class B

Conterminous 48 States

2007 AASHTO Bridge Design Guidelines

Spectral Response Accelerations SDs and SD1

Latitude = 44.566148

Longitude = -069.669273

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	0.257	SDs - Site Class D
1.0	0.111	SD1 - Site Class D

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