

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

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

**EAST HASTINGS BRIDGE
ROUTE 212 OVER EAST HASTINGS BROOK
MERRILL, MAINE**

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Aroostook County
WIN 22252.00

Soils Report 2022-05
Bridge No. 3150

State Project No. 22252.00
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1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and provide geotechnical design recommendations for the replacement of East Hastings Bridge which carries Route 212 over East Hastings Brook in Merrill, Maine. This report presents the subsurface information obtained at the site during the subsurface investigations, geotechnical design parameters, and construction recommendations for the new bridge.

The existing East Hastings Bridge was constructed in 1935. The structure consists of a single 20-foot span, concrete deck slab with concrete abutments on spread footings founded on bedrock. According to the 2019 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the bridge is structurally deficient with concrete cracking and efflorescence in both the superstructure and substructure elements.

The proposed replacement structure is a precast prestressed solid slab beam superstructure on full height abutments founded on bedrock. The bridge will have a single 34-foot span and will include 8-foot-long identical flared wingwalls on all four corners, also founded on bedrock.

The new bridge will be located on a horizontal alignment that will approximately match the existing alignment. The vertical alignment will be raised approximately 9 inches. The combination of the road profile raise, and the shallower superstructure will increase freeboard and improve hydraulics at the project site. The existing bridge will be closed during construction and traffic detoured onto a single lane temporary bridge located on the downstream side.

2.0 GEOLOGIC SETTING

East Hastings Bridge carries Route 212 over East Hastings Brook as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of the Island Falls Quadrangle, Open-File No. 81-38 (1981), indicates the surficial soils in the vicinity of the bridge project consist of glacial till and swamp and tidal marsh deposits. Glacial till is a heterogeneous mixture of sand, silt, clay, and stones. Swamp and tidal marsh deposits consist of peat, silt, clay, and sand.

The MGS Bedrock Geology of Maine (1985) maps the bedrock in the vicinity of the project as interbedded Pelite and Sandstone of the Grand Pitch Formation. Geological Survey Professional Paper 527 published by the U.S. Department of the Interior (1967) maps the bedrock at the site as Slate and Quartzite of the Grand Pitch Formation. The bedrock cored in the test borings drilled at the site consisted of Metashale and Siltstone with quartz veins.

3.0 SUBSURFACE INVESTIGATION

Four test borings explored subsurface conditions at the site. Borings BB-MEHB-101 and BB-MEHB-201 were drilled behind the existing west abutment. Borings BB-MEHB-102 and BB-MEHB-202 were drilled behind the existing east abutment. All borings terminated in bedrock cores. The test boring locations are shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile.

The 100-series borings were drilled on June 27, 2018 and the 200-series borings were drilled October 22, 2018. All borings were drilled by the MaineDOT Drill Crew. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs.

The borings were performed by using a combination of solid stem auger, cased wash boring, and rock coring techniques. The borings were completed by backfilling and compacting the borehole with drill cuttings. 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 is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated per ASTM D4633 “Standard Test Method for Energy Measurement for Dynamic Penetrometers” in June 2019. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.928 to the raw field N-values. This 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 in the borings using an NQ-2” core barrel and the Rock Quality Designation (RQD) of the cores calculated. The MaineDOT geotechnical engineer selected the boring locations and drilling methods, designated type and depth of sampling techniques, and identified field-testing requirements, and reviewed the field logs for accuracy. A MaineDOT NETTCP Certified Subsurface Inspector logged the subsurface conditions encountered in the borings. The borings were located in the field using taped measurements at the completion of the drilling program and then located by MaineDOT Survey.

4.0 LABORATORY TESTING

A laboratory testing program was conducted on selected soil samples recovered from the test borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the project site. Laboratory testing consisted of three standard grain size analysis with natural water contents. The results of soil tests are included as Appendix C – Laboratory Test Results. Moisture content information and other soil test results are also shown on the boring logs provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings generally consisted of Fill, Glacial Till, and metamorphic Bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs. A generalized subsurface profile is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile. The following paragraphs summarize the subsurface conditions encountered:

5.1 Fill

A layer of Fill was encountered in the test borings. The thickness of the fill material encountered ranged from approximately 9 to 10 feet. The fill materials encountered were variable and consisted of:

- Brown, damp, GRAVEL, some sand, trace silt;
- Brown, damp, SAND, some gravel, little silt;
- Grey, wet, Gravelly SAND, some silt;
- Olive-brown, moist, fine to coarse SAND, some gravel, some silt;
- Cobbles; and
- Boulders.

Corrected SPT N-values in the fill layer ranged from 15 to 63 bpf, indicating the fill is medium dense to very dense in consistency. Two grain size analysis indicated the material is classified as A-1-a under the AASHTO Soil Classification System and GW-GM and SW-SM under the Unified Soil Classification System (USCS). The natural water content of the samples ranged from 4 to 6 percent.

All four borings encountered cobbles and/or boulders in the Fill layer.

5.2 Glacial Till

An approximate 1-foot thick deposit of Glacial Till was encountered in boring BB-MEHB-202 beneath the Fill. The deposit consisted of grey, SAND, little gravel, little silt.

Refusal was reached when performing an SPT on the Glacial Till layer indicating the material is Very Dense. One grain size analysis indicated the material is classified as A-1-b under the AASHTO Soil Classification System and SM under the USCS. The moisture content of the sample was approximately 12 percent.

5.3 Bedrock

Bedrock was encountered and cored in all borings. The following table summarizes approximate depth to bedrock, corresponding approximate top of bedrock elevation, and RQD.

Boring	Station	Offset (feet)	Approximate Depth to Bedrock (feet)	Approximate Elevation of Bedrock Surface (feet)	RQD (%), (R1, R2)
BB-MEHB-101	7+78.6	8.4 Rt	9.4	840.5	17, 47
BB-MEHB-102	8+18.8	8.7 Lt	9.5	840.2	42, 13
BB-MEHB-201	7+83.9	7.8 Lt	11.0	838.7	0,0
BB-MEHB-202	8+15.9	8.0 Rt	11.2	838.4	30, 60

The bedrock at the site is identified as interbedded, fine grained, black Metashale and grey Siltstone, hard, fresh, joints at low to irregular angles, moderately close to very close, rough, with white, quartz veins. The bedrock core of boring BB-MEHB-202 contained significant white, medium grained, hard, fresh, massive Quartz with black Metashale inclusions. The RQD's of the bedrock cores ranged from 0 to 60 percent corresponding to a rock quality of very poor to fair. Detailed bedrock descriptions are provided on the boring logs in Appendix A – Boring Logs and on Sheet 3 – Boring Logs. Rock core photographs are included in Appendix B – Rock Core Photographs.

5.4 Groundwater

Groundwater was measured at depths ranging from 6.0 to 7.5 feet below the roadway surface upon completion of the borings. Note that water was introduced into the boreholes during drilling operations and the measured levels may not represent stabilized groundwater elevations. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, river levels, and construction activities.

6.0 FOUNDATION ALTERNATIVES

The Preliminary Design Report (PDR) (MaineDOT, 2020) investigated three bridge replacement options. The alternatives were:

- Single span, precast Next D beam bridge on full height abutments with flared wingwalls;
- Single span, precast, solid slab beam bridge on full height abutments with flared wingwalls; and
- Buried structure on cast-in-place footings.

All options considered were founded on spread footings cast directly on bedrock. The buried structure option was eliminated due to shallow bedrock depth and bankfull width requirements. Of the two remaining alternatives, the precast solid slab beam bridge was deemed more appropriate due to weight savings and additional freeboard clearance.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

The following subsections provide geotechnical design parameters and foundation considerations for full height, cantilever-type abutments and wingwalls supported by cast-in-place concrete spread footings founded on bedrock. These design recommendations are in accordance with AASHTO LRFD Bridge Design Specifications 9th Edition, 2020.

7.1 Abutment and Wingwall Spread Footings

Abutment and wingwall spread footings shall be evaluated to meet AASHTO LRFD requirements for sliding, eccentricity, and bearing resistance. Abutments and walls shall be proportioned for all applicable load combinations specified in LRFD Articles 3.4.1 and 11.5.5 and shall be evaluated for all relevant strength, extreme and service limit states.

LRFD Figures C11.5.6-1 through C11.5.6-4 illustrate the typical load factors to produce the extreme, factored effect for bearing resistance, sliding and eccentricity evaluations.

7.1.1. Spread Footings on Bedrock

The proposed replacement substructures will consist of full height, cantilever type abutments and return wingwalls on spread footings on bedrock. The test borings conducted at the site encountered the bedrock surface at Elevations ranging from 838.4 to 840.5.

The borings indicate that the upper bedrock is highly fractured and has RQD's correlating to a rock quality that is Very Poor to Fair. The contractor should expect that bedrock excavation of fractured and loose rock will be required in some areas and footings may need to be thicker to accommodate deeper areas of the prepared bedrock subgrade.

The thickness of footings may be designed to vary in thickness to accommodate variations in the bedrock surface, while maintaining the top elevation of the footings and reinforcing steel layout as shown on the Plans.

7.1.2 Sliding

The spread footings and seals will be cast directly on bedrock. The table below presents the resistance factors, ϕ_r , for sliding analyses for spread footings on bedrock.

Resistance Factors for Sliding					
Substructure	Bearing Material	Condition	Limit State	Sliding Resistance Factor ϕ_r	LRFD Reference
Spread Footings	Bedrock or soil	Cast-in-place footings	Strength	0.80	Table 10.5.5.2.2-1
			Service	1.0	Article 10.5.5.1
			Extreme	1.0	Articles 10.5.5.3 and 11.5.8

The sliding resistance of footings to lateral loads shall be calculated using the maximum coefficient of friction provided in the table below for mass concrete cast on fractured bedrock (Row 1).

Maximum Coefficients of Friction for Sliding						
Substructure	Interface Materials	Limit State	Friction Angle, δ_{soil}	Coefficient of Friction, $\tan \delta$ ¹ (dim.)	C ²	LRFD Reference
Spread footings/seals cast on bedrock in-the-dry or underwater	Mass concrete on fractured rock	All	31°	0.60	na	Table C3.11.5.3-1
Spread footings/seals cast in-the-dry	Mass concrete on clean sound rock	All	35°	0.70	na	Table C3.11.5.3-1

¹ Use a $\tan \phi_f = \tan \delta$ in LRFD Equation 10.6.3.4-2 to calculate resistance to sliding

² LRFD Article 10.6.3.4 – C not applicable to concrete footings cast on bedrock

Passive resistance of soil or bedrock at the toe of spread footings should be neglected and shall not contribute to resisting forces.

7.1.3 Bearing Resistance and Eccentricity

Nominal and factored bearing resistances were calculated for spread footings on bedrock using a Rock Mass Rating (RMR)-based empirical correlation¹ in accordance with LRFD Article C10.4.6.4 and 10.6.2.6.2. RMR was evaluated in accordance with Table 10.4.6.4-1 of the 2012 LRFD code, whereas the current edition of LRFD does not include the RMR formulation. The table below summarizes the factored bearing resistances for spread footings.

Bearing Resistances					
Substructure	Assumed Bearing Material	Limit State	Resistance Factor, ϕ_b	Factored Bearing Resistance (ksf)	LRFD Reference
Spread Footings	Fractured Bedrock	Service	1.0	10.0	Article 10.5.5.1
		Strength	0.45	8.1	Table 10.5.5.2.2-1
		Extreme	0.8	14.5	Article 11.5.8

The recommended service limit state bearing resistance is lower than that recommended in LRFD Table C10.6.2.5.1-1 in consideration of the footing subgrade preparation underwater, the potential for some minor loosening of the fractured rock near the subgrade elevation, and to control settlement.

From a practical standpoint, footings shall have a minimum width of 3 feet wide regardless of the bearing resistance or bearing material.

Application of permanent and transient loads is specified in LRFD Article 11.5.6. The vertical stress distribution may be assumed to be triangular or trapezoidal over the effective base as shown in LRFD Figure 11.6.3.2-2 for footings on rock. In no instance shall bearing stress exceed the nominal structural resistance of the structural concrete which may be taken as $0.3f'_c$.

For spread footings, the eccentricity limits for the resultant force are presented below.

Eccentricity Limits			
Substructure	Assumed Bearing Material	Location of Resultant Forces	LRFD Reference
Abutment and wingwall spread footings	Bedrock	No further than 0.45 B from the centerline of the footing [correlates to within the middle nine-tenths (9/10) of the base width]	Articles 10.6.3.3 and 11.6.3.3

¹ "Foundations on Rock" by Duncan Wyllie.

7.1.4 Earth Pressure and Surcharge Forces

In designing for active pressure, a Rankine active earth pressure coefficient, K_a , is recommended. Active earth pressure coefficients were calculated assuming a level backfill ($\beta=0^\circ$) and a 2H:1V backfill. These coefficients are as follows:

- $K_a = 0.31$ (Active Earth Pressure – horizontal backslope)
- $K_a = 0.47$ (Active Earth Pressure – 27° (2H:1V backslope))

The designer may assume Soil Type 4 (MaineDOT Bridge Design Guide (BDG)) Section 3.6.1) for granular backfill soil properties. The backfill properties are as follows: $\phi = 32^\circ$, $\gamma = 125$ pcf.

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

Equivalent Height of Soil for Estimating Live Load Surcharge Of Abutments Perpendicular to Traffic	
Abutment Height (feet)	h_{eq} (feet)
5	4.0
10	3.0
≥ 20	2.0

The live load surcharge on wingwalls may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from LRFD Table 3.11.6.4-2.

Abutment and wingwall designs shall include a drainage system behind the wall stems to intercept any groundwater. Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage, of the MaineDOT BDG. Backfill within 10 feet of abutments and wingwalls shall conform to Granular Borrow for Underwater Backfill - MaineDOT Specification 709.19. This gradation specifies 7 percent or less of the material passing the No. 200 sieve. This material is specified in order to limit the amount of fines and to minimize frost action behind the structure and to eliminate the need to design for hydrostatic forces.

7.2 Considerations for Service and Extreme Limit States

For scour protection of abutment and wingwall spread footings or concrete seals, construct the spread footings or concrete seals directly on bedrock surfaces cleaned of all weathered, loose, and potentially erodible or scourable rock. With these precautions, strength and extreme limit state designs do not need to consider rock scour due to the design or check floods for scour.

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. We do not anticipate shear failure along adversely oriented joint surfaces in the rock mass below the foundation; therefore, a global stability evaluation may be waived.

7.3 Bedrock Removal and Bedrock Subgrade Preparation

The borings encountered up to approximately 12 feet of overburden at the proposed foundation excavations. It is feasible that spread footings can be constructed to bear on bedrock using a combination of braced soil support systems and sloped cuts. The borings indicate that the RQD of the bedrock surface at the proposed footing locations will range from approximately 0 to 42 percent. The Contractor should anticipate the need to clear the bearing area of possibly significant quantities of weathered, loose, highly fractured, and potentially erodible bedrock during construction.

The nature, slope, and degree of fracturing in the bedrock bearing surfaces will not be evident until the foundation excavations for the abutments and wingwalls are made. The bedrock surface shall be cleared of all loose fractured bedrock, loose decomposed bedrock, and soil. The final bearing surface shall be solid. Construction activities that create rock falls or open fissures is not permitted. The cleanliness and condition of the final bedrock surface shall be approved by the Resident prior to placement of the footing or fill concrete.

If bedrock is observed to slope steeper than 4H:1V at the subgrade elevation, the bedrock should be benched to create level steps or excavated to be completely level. The bedrock surface may be stepped along the centerline of bearing to create a workable bearing surface. Alternatively, anchoring or doweling the footings to bedrock may be utilized to resist sliding forces and improve stability. Dowels should be #9 reinforcing bars or larger and be embedded into the footings and bedrock by depths determined by the Engineer. Any irregularities in, or unsoundness of, the existing bedrock surface, or irregularities and fractures created during the excavation process, should be backfilled with unreinforced concrete to the bearing elevation.

The contractor should maintain the abutment and wingwall excavations so that the foundations can be constructed in-the-dry. Where foundations are constructed in-the-dry, the final bearing surface shall be washed with high pressure water and air prior to concrete being placed for the footing.

7.4 Settlement

An approximate 9-inch raise in the roadway grade is planned at the bridge site. The granular fill soils encountered beneath the bridge approaches are generally medium dense to dense in consistency. These coarse-grained materials undergo elastic, immediate compression in response to an increase in the vertical overburden pressure. As a result, any settlement is anticipated to be small and will occur relatively quickly during construction.

The backfill for abutments and wingwalls shall consist of Standard Specification 703.19 – Granular Borrow Material for Underwater Backfill. To minimize post-construction settlement, the Granular Borrow within 10 feet of bridge abutments should be placed and compacted according to Standard Specification 206.03 to reduce settlements.

All foundations will be constructed on bedrock. Based on our experience, for footings constructed on rock, and designed for the service limit state using presumptive bearing resistance values in accordance with LRFD Table C10.6.2.5.1-1, elastic settlements can generally be assumed to be less than ½-inch.

7.5 Frost Protection

Foundations placed on the fill or native soils should be designed with an appropriate embedment for frost protection. According to MaineDOT BDG Figure 5-1, Maine Design Freezing Index Map, Merrill has a design freezing index (DFI) of approximately 2300 F-degree days. A water content of 10% was used for coarse-grained soils. These components correlate to a frost depth of 8.5 feet.

It is recommended that foundations bearing on soil be designed with an embedment of approximately 8.5 feet for frost protection.

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

7.6 Scour and Riprap

For scour protection of the footings, construct the footings directly on bedrock surfaces cleaned of soil and all weathered, loose, highly fractured, and potentially erodible rock. All loose rock, highly fractured bedrock, or bedrock with gouge shall be removed by ripping. The remaining intact bedrock subgrade shall be competent.

Wingwalls shall extend far enough from the structure to protect the structural backfill from scour.

Where required, slopes shall be armored with a minimum 3-foot thick layer of riprap conforming to MaineDOT Standard Specification 703.26 – Plain and Hand Laid Riprap. The riprap shall be underlain by a Class 1 Erosion Control Geotextile and a 1-foot layer of bedding material

conforming to MaineDOT Standard Specification 703.19 Granular Borrow Material for Underwater Backfill. The toe of the riprap sections shall be constructed 1-foot below the streambed elevation unless the riprap toe intersects bedrock. When intersecting bedrock, the largest riprap should be placed at the toe to buttress the slope above. The riprap slopes shall be constructed no steeper than 1.75H:1V extending from the edge of the roadway down to the existing ground surface.

7.7 Seismic Design Considerations

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

Seismic Design Parameters	
Parameter	Design Value
Peak Ground Acceleration (PGA)	0.068g
Acceleration Coefficient (A_s)	0.08g
S_{DS} (Period = 0.2 sec)	0.19g
S_{D1} (Period = 1.0 sec)	0.09g
Site Class	C
Seismic Zone	1

In conformance with LRFD 4.7.4.2 seismic analysis is not required for single-span bridges regardless seismic zone. However, superstructure connections and minimum support length requirements shall be designed per LRFD Articles 3.10.9.2 and 4.7.4.4, respectively.

8.0 CONSTRUCTION CONSIDERATIONS

8.1 Excavation and Dewatering

The new abutments will be constructed in the approximate locations of the existing abutments. The contractor shall be responsible for excavating the existing substructures in their entirety.

Excavations for the foundations will extend approximately up to 15 feet below the existing grade. Internally braced cofferdams will be required to permit construction of the spread footings 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 excavations throughout the period of construction. Water encountered at the base of the foundation excavation should be removed by pumping from sumps located in the corners of the excavation outside of the foundation footprint. The contractor should maintain the excavation so that pier footing concrete is placed in-the-dry.

All sloped, open cuts shall be designed and constructed 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 and divert groundwater and surface water infiltration using temporary ditches, sumps, granular drainage blankets and stone ditch protection.

8.2 Subgrade Preparation

The subgrade for the footings 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 conventional excavations methods and mechanical means. Mechanical means include expansive agents, hydraulic hoe ram, hydraulic splitters or wedging and prying.

Blasting, when required, 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.

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 foundations.

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

9.0 CLOSURE

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

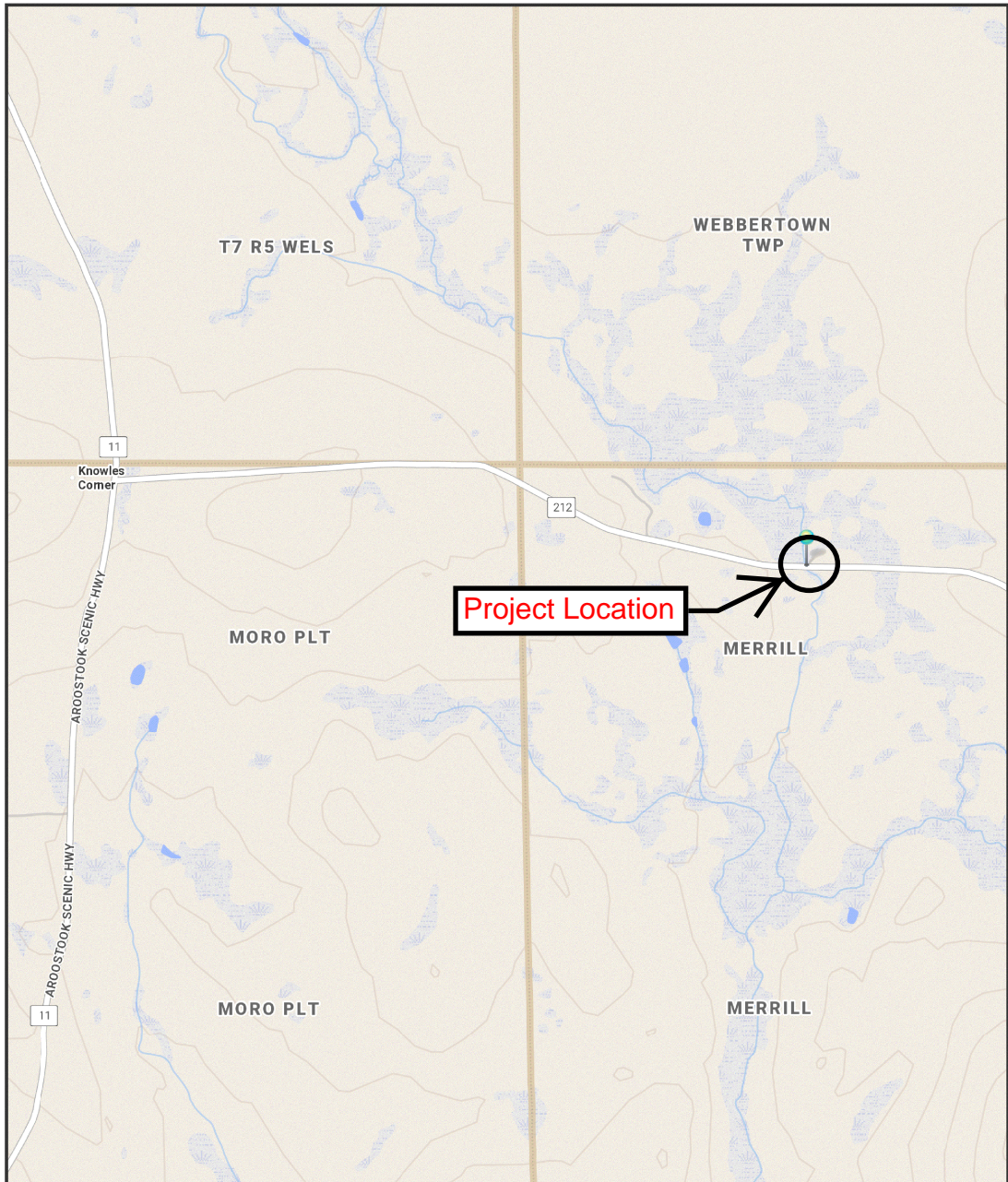
In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes 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 the geotechnical engineer be provided the opportunity for a review of the design and specifications so that the earthwork and foundation recommendations and construction considerations in the report are properly interpreted and implemented in the design and specifications.

Sheets

MERRILL, MAINE



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0.5 Miles
1 inch = 0.57 miles

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Time: 10:28:34 AM

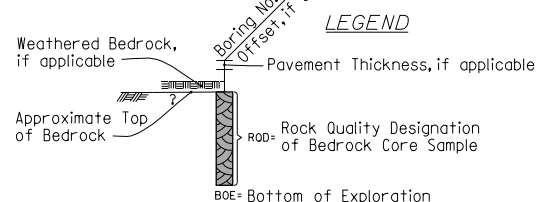
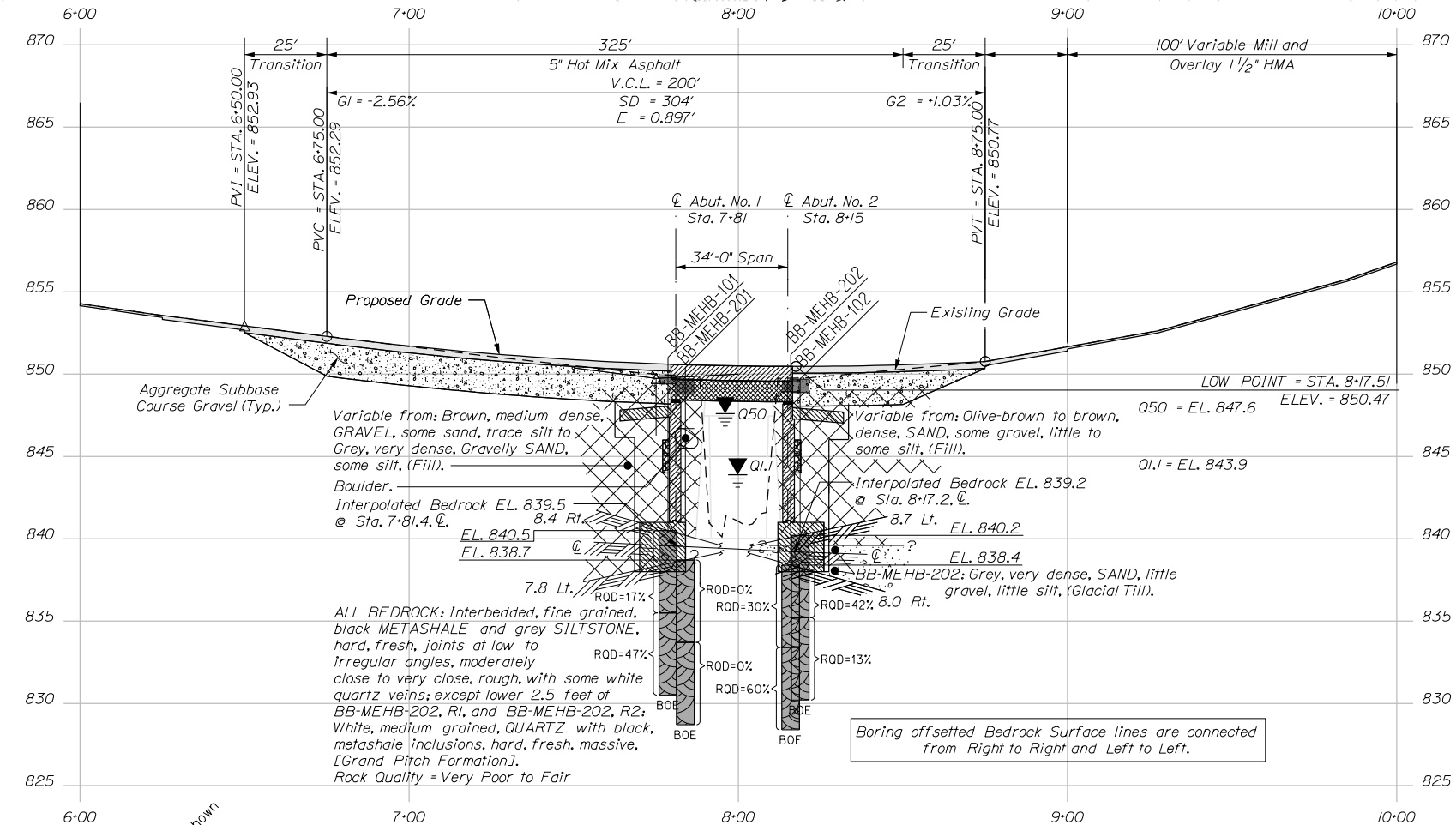
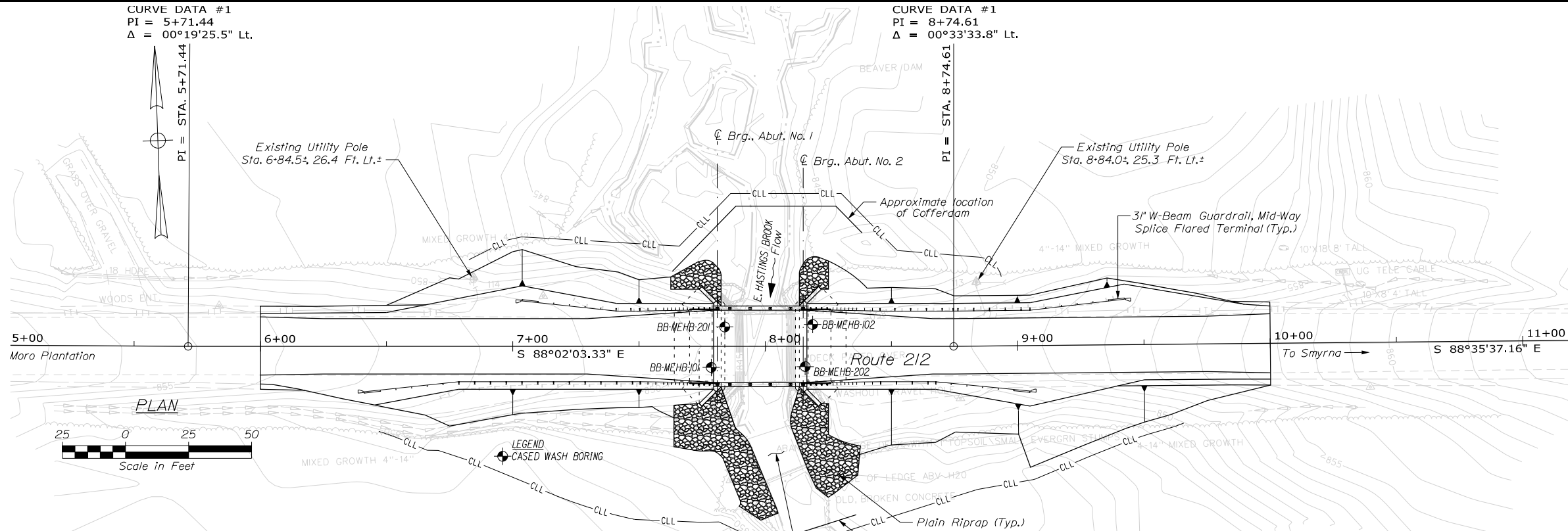
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		BRIDGE PLANS	

Date: 2/23/2022

Username: terry.white

Division: GEOTECH

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

STATE OF MAINE
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BRIDGE NO. 3150
BRIDGE PLANS

PROJ. MANAGER	DATE	BY	DATE
DESIGN-DETAILED		D. SHAW	
CHECKED-REVIEWED		T. WHITE	
DESIGN-DETAILED	JAN 2022		
DESIGN-DETAILED			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

EAST HASTINGS BRIDGE
EAST HASTINGS BROOK
MERRILL AROOSTOOK COUNTY
BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

2

OF 3

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.			<u>Descriptive Term</u> trace 0 - 10 little 11 - 20 some 21 - 35 adjective (e.g. Sandy, Clayey) 36 - 50															
		(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	SW	Well-graded sands, Gravelly sands, little or no fines			<u>TERMS DESCRIBING DENSITY/CONSISTENCY</u> Coarse-grained soils (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) Silty or Clayey gravels; and (3) Silty, Clayey or Gravelly sands. Density is rated according to standard penetration resistance (N-value). <table border="0"> <tr> <td><u>Density of Cohesionless Soils</u></td> <td><u>Standard Penetration Resistance N-Value (blows per foot)</u></td> </tr> <tr> <td>Very loose</td> <td>0 - 4</td> </tr> <tr> <td>Loose</td> <td>5 - 10</td> </tr> <tr> <td>Medium Dense</td> <td>11 - 30</td> </tr> <tr> <td>Dense</td> <td>31 - 50</td> </tr> <tr> <td>Very Dense</td> <td>> 50</td> </tr> </table>				<u>Density of Cohesionless Soils</u>	<u>Standard Penetration Resistance N-Value (blows per foot)</u>	Very loose	0 - 4	Loose	5 - 10	Medium Dense	11 - 30	Dense	31 - 50	Very Dense	> 50
		<u>Density of Cohesionless Soils</u>	<u>Standard Penetration Resistance N-Value (blows per foot)</u>																			
Very loose		0 - 4																				
Loose		5 - 10																				
Medium Dense	11 - 30																					
Dense	31 - 50																					
Very Dense	> 50																					
(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.			<u>Consistency of Cohesive soils</u> Very Soft WOH, WOR, WOP, <2 Soft 2 - 4 Medium Stiff 5 - 8 Stiff 9 - 15 Very Stiff 16 - 30 Hard >30 <u>Approximate Undrained Shear Strength (psf)</u> 0 - 250 250 - 500 500 - 1000 1000 - 2000 2000 - 4000 over 4000 <u>Field Guidelines</u> Fist easily penetrates Thumb easily penetrates Thumb penetrates with moderate effort Indented by thumb with great effort Indented by thumbnail Indented by thumbnail with difficulty																
		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):				Rock Quality Designation (RQD):																		
Color (Munsell color chart)				RQD (%) = $\frac{\text{sum of the lengths of intact pieces of core}^* > 4 \text{ inches}}{\text{length of core advance}}$																		
Moisture (dry, damp, moist, wet)				*Minimum NQ rock core (1.88 in. OD of core)																		
Density/Consistency (from above right hand side)				Rock Quality Based on RQD																		
Texture (fine, medium, coarse, etc.)				<u>Rock Quality</u> <u>RQD (%)</u>																		
Name (Sand, Silty Sand, Clay, etc., including portions - trace, little, etc.)				Very Poor ≤25																		
Gradation (well-graded, poorly-graded, uniform, etc.)				Poor 26 - 50																		
Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)				Fair 51 - 75																		
Structure (layering, fractures, cracks, etc.)				Good 76 - 90																		
Bonding (well, moderately, loosely, etc.,)				Excellent 91 - 100																		
Cementation (weak, moderate, or strong)				Desired Rock Observations (in this order, if applicable):																		
Geologic Origin (till, marine clay, alluvium, etc.)				Color (Munsell color chart)																		
Groundwater level				Texture (aphanitic, fine-grained, etc.)																		
				Rock Type (granite, schist, sandstone, etc.)																		
				Hardness (very hard, hard, mod. hard, etc.)																		
				Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)																		
				Geologic discontinuities/jointing:																		
				-dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.)																		
				-spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet)																		
				-tightness (tight, open, or healed)																		
				-infilling (grain size, color, etc.)																		
				Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)																		
				RQD and correlation to rock quality (very poor, poor, etc.)																		
				ref: ASTM D6032 and FHWA NHI-16-072 GEC 5 - Geotechnical Site Characterization, Table 4-12																		
				Recovery (inch/inch and percentage)																		
				Rock Core Rate (X.X ft - Y.Y ft (min:sec))																		
Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information				Sample Container Labeling Requirements:																		
				WIN		Blow Counts																
				Bridge Name / Town		Sample Recovery																
				Boring Number		Date																
				Sample Number		Personnel Initials																
				Sample Depth																		

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: East Hastings Bridge #3150 carries Route 212 over East Hastings Brook Location: Merrill, Maine	Boring No.: BB-MEHB-101 WIN: 22252.00
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Driller: MaineDOT	Elevation (ft.): 849.9	Auger ID/OD: 5" Solid Stem
Operator: Daggett/Niles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 6/27/2018; 07:30-11:30	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 7+78.6, 8.4 ft Rt.	Casing ID/OD: NW-3"	Water Level*: 7.5 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 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person	S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _u = 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					
0								SSA	849.4	6" HMA.		
										Cobble from 1.8-2.4 ft bgs.		
5	1D	24/9	5.00 - 7.00	6/6/4/7	10	15	27			Brown, damp, medium dense, GRAVEL, some sand, trace silt, (Fill).	G#296510 A-1-a, GW-GM WC=3.6%	
							19					
							23					
							32					
10	R1	60/58	9.40 - 14.40	RQD = 17%			a120		840.5	a120 blows for 0.4 ft.		
							NQ-2			Top of Bedrock at Elev. 840.5 ft.		
										R1: Bedrock: Interbedded, fine grained, black METASHALE and grey SILTSTONE, hard, fresh, contorted, breaks are irregular, rough, undulating, close to moderately close. [GRAND PITCH FORMATION] Rock Quality = Very Poor. R1: Core Times (min:sec) 9.4-10.4 ft (3:07) 10.4-11.4 ft (3:10) 11.4-12.4 ft (3:50) 12.4-13.4 ft (4:15) 13.4-14.4 ft (4:11) 97% Recovery		
15	R2	60/59	14.40 - 19.40	RQD = 47%						R2: Bedrock: Similar to R1 except breaks are primarily along convoluted foliation. Rock Quality = Poor. R2: No times taken. 98% Recovery		
20									830.5	Bottom of Exploration at 19.4 feet below ground surface.		
25												

Remarks:

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: East Hastings Bridge #3150 carries Route 212 over East Hastings Brook Location: Merrill, Maine	Boring No.: BB-MEHB-102 WIN: 22252.00
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Driller: MaineDOT	Elevation (ft.): 849.7	Auger ID/OD: 5" Solid Stem
Operator: Daggett/Niles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 6/27/2018; 11:30-14:00	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 8+18.8, 8.7 ft Lt.	Casing ID/OD: NW-3"	Water Level*: 6.0 ft bgs.

Hammer Efficiency Factor: 0.928	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	
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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 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	848.9	10" HMA.		
5	1D	24/16	5.00 - 7.00	5/13/14/18	27	42	23			Cobble from 3.6-4.0 ft bgs. Brown, damp, dense, SAND, some gravel, little silt, (Fill).	G#296511 A-1-a, SW-SM WC=5.8%	
10	R1	60/60	9.50 - 14.50	RQD = 42%			a75	NQ-2	840.2	a75 blows for 0.5 ft. Top of Bedrock at Elev. 840.2 ft. R1: Bedrock: Interbedded, fine grained, black METASHALE and grey SILTSTONE, hard, fresh, breaks are primarily low angle, rough, undulating, close to moderately close. Frequent white, healed calcite and quartz veins. [GRAND PITCH FORMATION] Rock Quality = Poor. R1: Core Times (min:sec) 9.5-10.5 ft (7:32) 10.5-11.5 ft (3:46) 11.5-12.5 ft (3:00) 12.5-13.5 ft (2:16) 13.5-14.5 ft (2:42) 100% Recovery		
15	R2	60/60	14.50 - 19.50	RQD = 13%						R2: Bedrock: Similar to R1 except close conjugate jointing. Rock Quality = very poor. R2: Core Times (min:sec) 14.5-15.5 ft (3:09) 15.5-16.5 ft (2:35) 16.5-17.5 ft (2:45) 17.5-18.5 ft (2:44) 18.5-19.5 ft (2:41) 100% Recovery		
20									830.2			
25												

Remarks:

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: East Hastings Bridge #3150 carries Route 212 over East Hastings Brook Location: Merrill, Maine	Boring No.: BB-MEHB-201 WIN: 22252.00
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Driller: MaineDOT	Elevation (ft.): 849.7	Auger ID/OD: 5" Solid Stem
Operator: Daggett/Niles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 10/22/2018; 09:30-12:00	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 7+83.9, 7.8 ft Lt.	Casing ID/OD: NW-3"	Water Level*: 7.1 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_u = 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 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	848.8	11" HMA.		
5	1D	24/6	5.00 - 7.00	17/26/15/31	41	63	21		848.7	Boulder from 3.0-4.2 ft bgs. Cobble from 4.5-4.8 ft bgs. Grey, wet, very dense, Gravelly SAND, some silt, (Fill).		
10									838.7	Top of Bedrock at Elev. 838.7 ft. R1: Bedrock: Interbedded, fine grained, black METASHALE and grey SILTSTONE, hard, fresh, contorted, breaks are irregular, rough, undulating, very close to close. [GRAND PITCH FORMATION] Rock Quality = Very Poor. R1: Core Times (min:sec) 11.0-12.0 ft (3:25) 12.0-13.0 ft (3:58) 13.0-14.0 ft (4:39) 14.0-15.0 ft (4:26) 15.0-16.0 ft (5:03) 92% Recovery		
15	R2	60/36	16.00 - 21.00	RQD = 0%					828.7	R2: Bedrock: Similar to R1. Rock Quality = Very Poor. R2: Core Times (min:sec) 16.0-17.0 ft (3:36) 17.0-18.0 ft (3:30) 18.0-19.0 ft (5:07) 19.0-20.0 ft (5:21) 20.0-21.0 ft (5:22) 60% Recovery		
20												
25												

Remarks:

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: East Hastings Bridge #3150 carries Route 212 over East Hastings Brook Location: Merrill, Maine	Boring No.: BB-MEHB-202 WIN: 22252.00
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Driller: MaineDOT	Elevation (ft.): 849.6	Auger ID/OD: 5" Solid Stem
Operator: Daggett/Niles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 10/22/2018; 12:00-15:30	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 8+15.9, 8.0 ft Rt.	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"/>	
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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 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	848.9	8" HMA.		
5	1D	24/16	5.00 - 7.00	4/11/11/8	22	34	10			Olive-brown, moist, dense, fine to coarse SAND, some gravel, some silt (Fill).		
10	2D		10.00 - 11.00	12/55(6")	---		85		839.6	Cobble from 9.0-9.4 ft bgs. Grey, very dense, SAND, little gravel, little silt, (Glacial Till).	G#336932 A-1-b, SM WC=12.1%	
	R1	60/58	11.20 - 16.20	RQD = 30%				NQ-2	838.4	Top of Bedrock at Elev. 838.4 ft. R1: Bedrock: Upper 2.5 feet is interbedded, fine grained, black METASHALE and grey SILTSTONE, hard, fresh, contorted, breaks are irregular, rough undulating, very close to close, then a 2.5 foot white QUARTZ vein. Rock Quality = Poor. R1: Core Times (min:sec) 11.2-12.2 ft (1:36) 12.2-13.2 ft (5:48) 13.2-14.2 ft (6:20) 14.2-15.2 ft (7:57) 15.2-16.2 ft (7:58) 97% Recovery		
15												
	R2	60/60	16.20 - 21.20	RQD = 60%						R2: Bedrock: White, medium grained, QUARTZ, with a few, black metashale inclusions, hard, massive, fresh. Rock Quality = Fair. R2: Core Times (min:sec) 16.2-17.2 ft (4:14) 17.2-18.2 ft (4:53) 18.2-19.2 ft (7:00) 19.2-20.2 ft (7:02) 20.2-21.2 ft (6:58) 100% Recovery		
20									828.4			
25												

Remarks:

Appendix B

Rock Core Photographs



MaineDOT
East Hastings Bridge #3150 Carries Route 212 Over East Hastings Brook
Merrill, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-MEHB-101	R1	9.40 – 14.40	60	58	10	17	METASHALE and SILTSTONE	1
BB-MEHB-101	R2	14.40 – 19.40	60	59	28	47	METASHALE and SILTSTONE	2
BB-MEHB-102	R1	9.50 – 14.50	60	60	25	42	METASHALE and SILTSTONE	3
BB-MEHB-102	R2	14.50 – 19.50	60	60	8	13	METASHALE and SILTSTONE	4



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



MaineDOT
East Hastings Bridge #3150 Carries Route 212 Over East Hastings Brook
Merrill, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-MEHB-201	R1	11.00-16.00	60	55	0	0	METASHALE and SILTSTONE	1
BB-MEHB-201	R2	16.00-21.00	60	36	0	0	METASHALE and SILTSTONE	2
BB-MEHB-202	R1	11.20-16.20	60	58	18	30	METASHALE and SILTSTONE/ QUARTZ	3
BB-MEHB-202	R2	16.20-21.20	60	60	36	60	QUARTZ	4

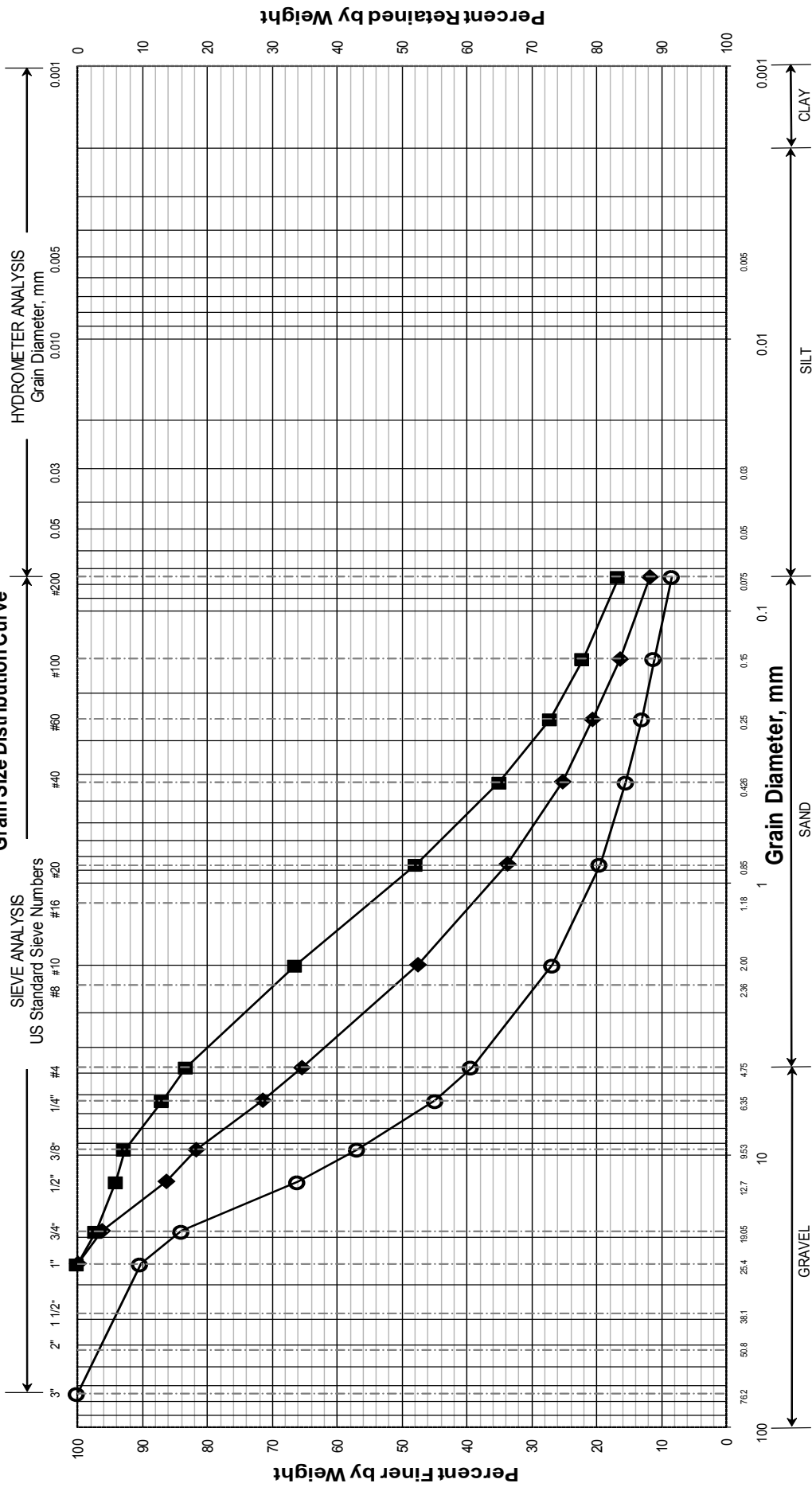


- Notes:** 1. "Box row" indicates the section of the box where the core run is contained: 1 = top, 4 = bottom.
2. Top of each core run is on the left and increases with depth to the right.

Appendix C

Laboratory Test Results

**Maine Department of Transportation
Grain Size Distribution Curve**



UNIFIED CLASSIFICATION

Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○ BB-MEHB-101/1D	7+78.6	8.4 RT	5.0-7.0	GRAVEL, some sand, trace silt.	3.6			
◆ BB-MEHB-102/1D	8+18.8	8.7 LT	5.0-7.0	SAND, some gravel, little silt.	5.8			
■ BB-MEHB-202/2D	7+83.9	7.8 LT	10.0-11.0	SAND, little gravel, little silt.	12.1			
●								
▲								
X								

WIN
022252.00
Town
Merrill
Reported by/Date
WHITE, TERRY A 12/10/2018

Appendix D

Calculations

Earth Pressure

Earth Pressure

Soil Parameters:

Existing backfill will be removed and replaced with granular borrow - Soil Type 4, MaineDOT BDG Section 3.6.1.

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

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

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

At Rest Earth Pressure

Reference: Das, Principles of Geotech Engr, 7th Edition, Eq. 13.5.
Formula for normally consolidated soils.

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

$$K_o = 0.47$$

Breastwall and Wingwalls - Active Earth Pressure - Rankine Theory

The earth pressure is applied to a plane extending vertically up from the heel of the wall base, and the weight of the soil on the inside of the vertical plane is considered as part of the wall weight. The failure sliding surface is not restricted by the top of the wall or back face of wall.

For cantilever walls with horizontal backslope:

$$K_{ar} := \tan\left(45 \cdot \text{deg} - \frac{\phi}{2}\right)^2$$

$$K_{ar} = 0.31$$

For flared wingwalls with sloped 2H:1V backfill

β = Angle of fill slope to the horizontal $\beta := 27 \cdot \text{deg}$

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

$$K_{ar_slope} = 0.47$$

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

For flared wingwalls with other backslope angles, see Page 2 of 2

Slope Behind Wall, β (deg)	K_a
0	0.307
1	0.307
2	0.308
3	0.308
4	0.309
5	0.311
6	0.312
7	0.314
8	0.316
9	0.318
10	0.321
11	0.324
12	0.328
13	0.331
14	0.336
15	0.341
16	0.346
17	0.352
18	0.358
19	0.366
20	0.374
21	0.383
22	0.393
23	0.405
24	0.418
25	0.434
26	0.451
27	0.473
28	0.498
29	0.531
30	0.574
31	0.639
32	0.848

TABLE 11-3
Rankine active earth pressure coefficients K_a using Eq. (11-7a)

β	$\phi = 26$	28	30	32	34	36	38	40	42
0	0.3905	0.3610	0.3333	0.3073	0.2827	0.2596	0.2379	0.2174	0.1982
5	0.3959	0.3656	0.3372	0.3105	0.2855	0.2620	0.2399	0.2192	0.1997
10	0.4134	0.3802	0.3495	0.3210	0.2944	0.2696	0.2464	0.2247	0.2044
15	0.4480	0.4086	0.3729	0.3405	0.3108	0.2834	0.2581	0.2346	0.2129
20	0.5152	0.4605	0.4142	0.3739	0.3381	0.3060	0.2769	0.2504	0.2262
25	0.6999	0.5727	0.4936	0.4336	0.3847	0.3431	0.3070	0.2750	0.2465
30	—	—	0.8660	0.5741	0.4776	0.4105	0.3582	0.3151	0.2784
35	—	—	—	—	—	0.5971	0.4677	0.3906	0.3340
40	—	—	—	—	—	—	—	0.7660	0.4668

Since $\cos \beta$ is a permanent entry it is convenient to include it with K_a of Eq. (11-7) or K_p of Eq. (11-8), giving, e.g.,

$$K_a = \cos \beta \frac{\cos \beta - \sqrt{\cos^2 \beta - \cos^2 \phi}}{\cos \beta + \sqrt{\cos^2 \beta - \cos^2 \phi}} \quad (11-7a)$$

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.

Figure 3-2 Calculating β with Broken Backfill Surface

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

3.6.5.2 Rankine Theory

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

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

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

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

where:

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

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

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

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

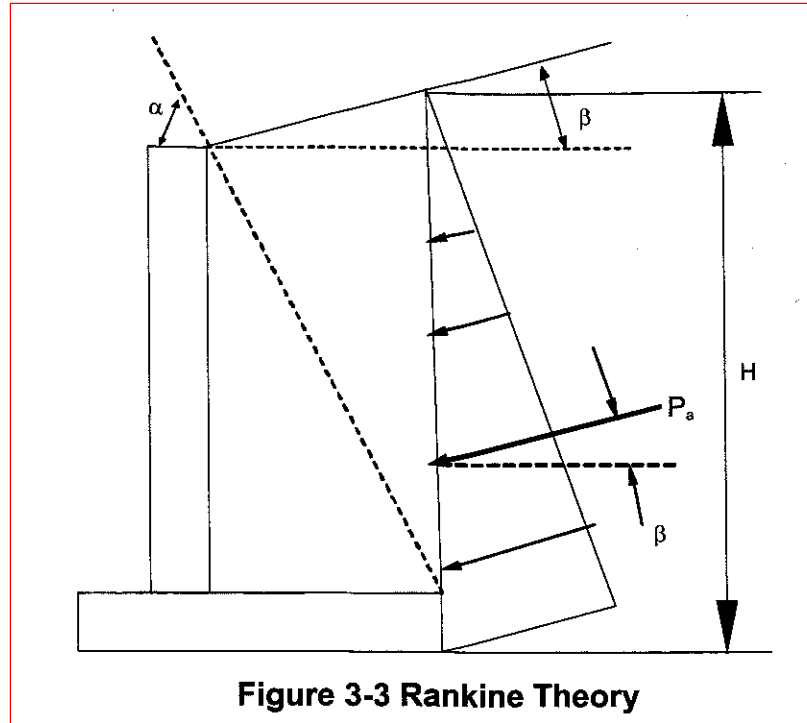


Figure 3-3 Rankine Theory

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

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

3.6.6 Coulomb Passive Lateral Earth Pressure Coefficient

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

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

where:

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

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

Bearing 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 Standard Specifications for Highway Bridges, 17th Ed. 2002
3. Wylie, Duncan C, "Foundations on Rock", Second Edition, 2009.
4. "The Hoek-Brown Failure Criterion - A 1988 Update", E. Hoek and E.T. Brown

A. Design Bedrock Properties

Model site bedrock based on the country rock encountered in borings:
Fine grained, METASHALE (SLATE) and SILTSTONE, hard, fresh.
RQD's of RI's range from 0 to 42%; Use RQD=20% for design purposes

Compressive Strength

Based on Table 4.4.8.1.2B Typical Range of Uniaxial Compressive Strength as a Function of Rock Category and Rock Type, Ref. 2.

Rock Category B - Siltstone and Slate (Metashale)

Siltstone $C_o = 1,400-17,000$ psi

Slate $C_o = 21,000-30,000$ psi

Use lower bound of the two rock types

$$q_{uc1} := \text{psi} \cdot \left[\frac{(17000 - 1400)}{2} + 1400 \right] \quad q_{uc1} = 9200 \cdot \text{psi} \quad q_{uc1} = 1325 \cdot \text{ksf}$$

$$q_{uc2} := \text{psi} \cdot \left[\frac{(30000 - 21000)}{2} + 21000 \right] \quad q_{uc2} = 25500 \cdot \text{psi}$$

$$q_{uc} := \left(q_{uc1} + \frac{q_{uc2} - q_{uc1}}{2} \right) \quad q_{uc} = 17350 \cdot \text{psi} \quad q_{uc} = 2498 \cdot \text{ksf}$$

B. Abutments and Wingwall 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

$$q_{uc1} = 1,325 \text{ ksf}$$

From LRFD Table 10.4.6.4-1 for Uniaxial compressive strength **Relative Rating = 7**

2. Drill Core Quality

RQD for design purposes (see above), 20%

From LRFD Table 10.4.6.4-1, RQD <25%; **Relative Rating = 3**

3. Spacing of joints

Rock core in BB-MEHB-201 has very close joints for the full core length. Rock core in BB-MEHB-101 has very close joints for the first 5 feet of core. Breaks are <2 in

From LRFD Table 10.4.6.4.-1; **Relative Rating = 5**

4. Condition of joints

Break surface are slightly rough; **Relative Rating = 15**

5. Groundwater conditions

Water under moderate pressure; **Relative Rating = 4**

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

For Foundations, Fair ; **Relative Rating = -7**

ADJUSTED RMR

$$\text{RMR} := 7 + 3 + 5 + 15 + 4 - 7$$

$$\text{RMR} = 27$$

Determine Rock Type for LRFD Table 10.4.6.4.-4

Rock Type - B

Geomechanics Rock Mass Class Determined from Total Rating

From AASHTO LRFD Table 10.4.6.4-3, RMR = 27 is Class No. IV and described as Poor Rock.

C. Rock Property Constants s and m (Ref. #1 and Ref. #4)

$$\text{RMR} = 27$$

Direct calculation of m and s is required, Reference 4 (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.

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

$$s = \exp((\text{RMR}-100)/6)$$

$$m_i = m \text{ for intact rock}$$

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

$$m_i := 10$$

$$m := m_i \cdot \exp\left(\frac{\text{RMR} - 100}{14}\right) \quad \text{Equation 18, Ref. 3}$$

$$m = 0.054$$

$$s := \exp\left(\frac{\text{RMR} - 100}{6}\right) \quad \text{Equation 19, Ref. 3}$$

$$s = 0.0000052$$

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 $C_{fl} = 1.0$ for $L/B > 6$

Nominal Bearing Resistance (Wyllie)

Reference #3: Wyllie "Foundations on Rock" Equation 5.4 Pg. 138

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

$$q_{nl} = 18.1 \cdot \text{ksf}$$

Factored Bearing Resistances

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

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

$$q_{r1} := q_{n1} \cdot \phi_{bc}$$

$$q_{r1} = 8.1 \cdot \text{ksf}$$

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.

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

$$q_{r1} := q_{n1} \cdot \phi_{rec}$$

$$q_{r1} = 14.5 \cdot \text{ksf}$$

Extreme Limit State

Factored Bearing Resistance - Service Limit State

Reference LRFD Table C10.6.2.5.1-1 Presumptive Bearing Resistance for Spread Footings at the Service Limit State

Type of Bearing Material - Weathered or broken bedrock of any type, except shale, Ordinary Range 16 - 24 ksf. Use 10 ksf in consideration of the footing subgraded being prepared under water and the potential for some loosening of the fractured rock near the subgrade elevation.

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

$$q_{r1} := 10 \cdot \text{ksf} \cdot \phi_{bc}$$

$$q_{r1} = 10 \cdot \text{ksf}$$

Service 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.

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

$$q_{n1} = 18.081 \cdot \text{ksf}$$

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

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

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

⁽²⁾Not including oil shale.

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

$$\alpha_E = 0.0231(RQD) - 1.32 \geq 0.15 \quad (4.4.8.2.2-4)$$

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

$$E_m = \alpha_E E_o \quad (4.4.8.2.2-3)$$

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

4.4.8.2.3 Tolerable Movement

Refer to Article 4.4.7.2.3.

4.4.9 Overall Stability

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

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.

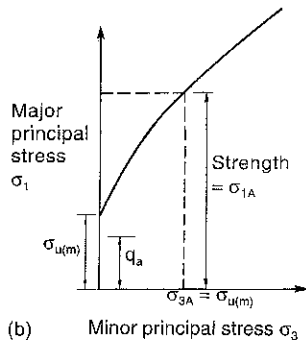
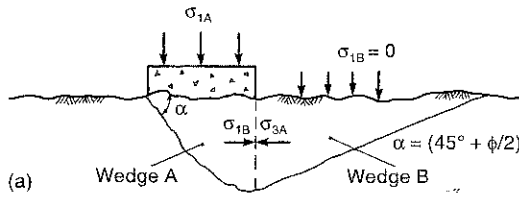


Figure 5.3 Analysis of bearing capacity of fractured rock: (a) active A and passive B wedges in foundation; and (b) curved rock mass strength envelope. Allowable bearing pressure = q_a , strength of bearing rock = σ_{1A} , factor of safety $FS = \sigma_{1A}/q_a$.

$$\begin{aligned} \sigma_1 &= (m\sigma_{u(r)}(s\sigma_{u(r)}^2)^{1/2} + s\sigma_{u(r)}^2)^{1/2} + (s\sigma_{u(r)}^2)^{1/2} \\ &= s^{1/2}\sigma_{u(r)}[1 + (ms^{-1/2} + 1)^{1/2}] \end{aligned} \quad (5.3)$$

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

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

From Design Freezing Index Map: Merrill, Maine

DFI = 2300 degree-days.

Case 1 - coarse grained soils W=10% (BB-MEHB-101, 1D; BB-MEHB-102, 1D).

For DFI = 2300

at w=10%

$d_2 := 102.3 \text{ in}$

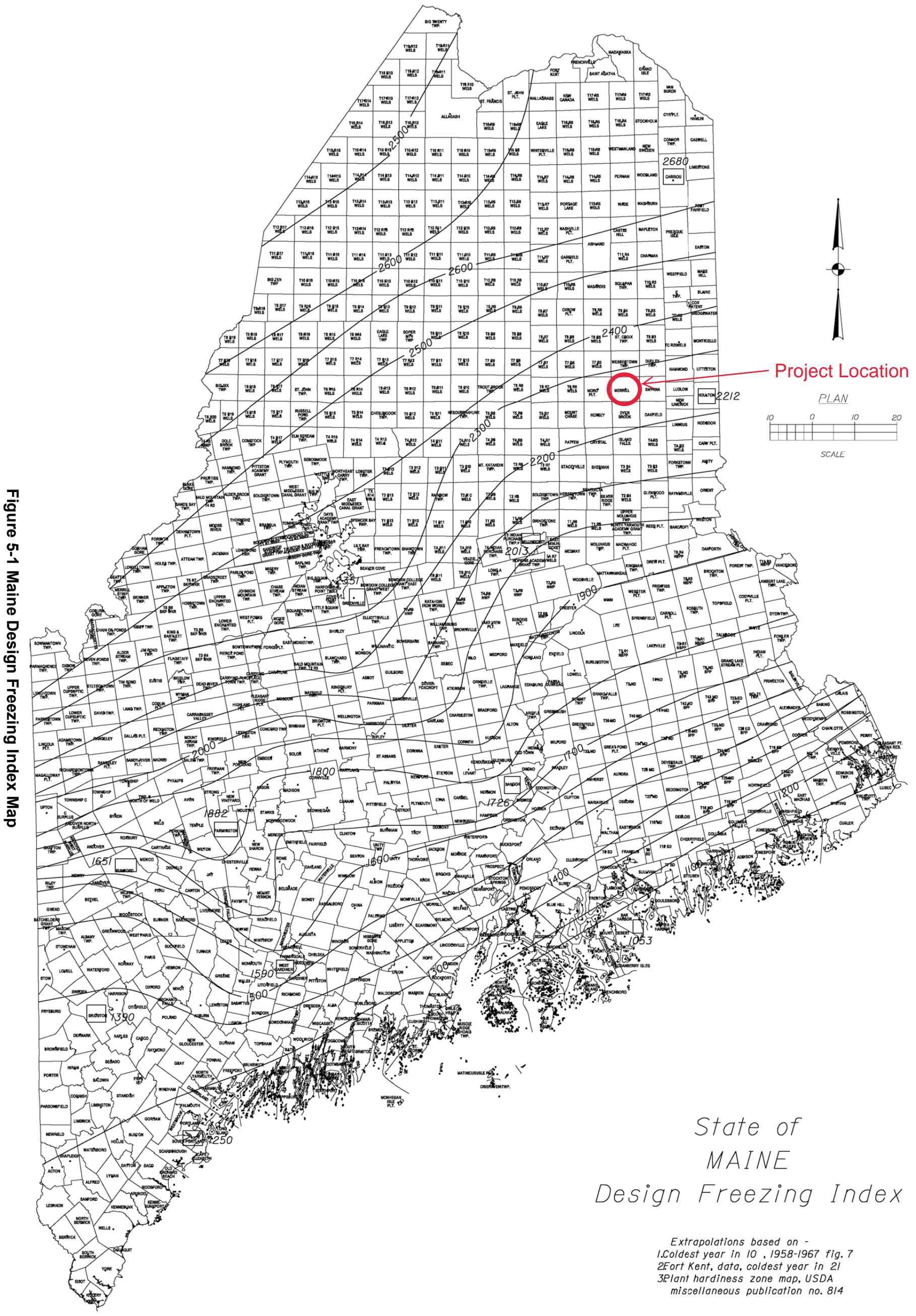


Figure 5-1 Maine Design Freezing Index Map

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*5.2.1 Frost*

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

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

Table 5-1 Depth of Frost Penetration

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

Seismic Parameters

BB-MEHB-101			
Depth	N ₆₀	di	di/N
5	15	9.4	0.63
9.4	100	90.6	0.91
SUM		100	1.53

di/di/N 65.25

BB-MEHB-102			
Depth	N ₆₀	di	di/N
5	42	9.5	0.23
9.5	100	90.5	0.91
SUM		100	1.13

di/di/N 88.40

BB-MEHB-201			
Depth	N ₆₀	di	di/N
5	63	11	0.17
11	100	89	0.89
SUM		100	1.06

di/di/N 93.93

BB-MEHB-202			
Depth	N ₆₀	di	di/N
5	34	10	0.29
10	50	1.2	0.02
11.2	100	88.8	0.89
SUM		100	1.21

di/di/N 82.91

SUM	Nav.	82.62
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Nav. > 50 bpf

Conclusion: Site Class C

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

Merrill, East Hastings Bridge #3150

WIN 22252.00

February 8, 2022

Abutment No. 1 and 2 Seismic Parameters

Conterminous 48 States

2007 AASHTO Bridge Design Guidelines

AASHTO Spectrum for 7% PE in 75 years

Latitude = 46.201940

Longitude = -068.285280

Site Class B

Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.068	PGA - Site Class B
0.2	0.160	Ss - Site Class B
1.0	0.050	S1 - Site Class B

Conterminous 48 States

2007 AASHTO Bridge Design Guidelines

Spectral Response Accelerations SDs and SD1

Latitude = 46.201940

Longitude = -068.285280

As = FpgaPGA, SDs = FaSs, and SD1 = FvS1

Site Class C - Fpga = 1.20, Fa = 1.20, Fv = 1.70

Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.082	As - Site Class C
0.2	0.192	SDs - Site Class C
1.0	0.085	SD1 - Site Class C

Merrill, East Hastings Bridge #3150

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February 8, 2022

