MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

PALMER BRIDGE
RICHMOND ROAD (STATE ROUTE 197) OVER MAGOTTY MEADOW BROOK
LITCHFIELD, MAINE

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Kennebec County	Soils Report 2019-46
WIN 22246.00	Bridge No. 5141

State Project No. 22246.00

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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 Palmer Bridge, which carries Richmond Road (State Route 197) over Magotty Meadow Brook in Litchfield, Maine. This report presents the subsurface information obtained at the site during the subsurface investigation, foundation design recommendations, and geotechnical parameters for design of the new bridge structure.

Palmer Bridge was constructed in 1948 and is a 13-foot 9-inch diameter steel pipe culvert with a shallow concrete invert lining. The pipe is buried one foot below the road surface. The culvert has severe holes and corrosion at both ends and small holes throughout the middle. The concrete invert lining is covering severe corrosion along the bottom of the pile and is in poor condition. The bridge has a sufficiency rating of 51.1 , is structurally deficient and has minimal remaining service lift.

The Maine Department of Transportation (MaineDOT) Preliminary Design Report (PDR) dated September 5, 2019 recommends replacement of the steel pipe with a 17-foot span three-sided concrete frame on cast in place concrete strip footing constructed directly on bedrock. The frame will have a natural bottom of exposed bedrock The horizontal and vertical alignments will match the existing and the culvert skew will be adjusted to 6 degrees.

Richmond Road will be closed to traffic during construction of the replacement bridge, and traffic detoured.

2.0 GEOLOGIC SETTING

Palmer Bridge carries Richmond Road over Magotty Meadow Brook approximately 0.39 mile north of the Bowdoin town line, as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of Maine (2013) indicates the surficial soil units in the vicinity of the bridge are wetland deposits with contacts to the Presumpscot Formation and thin glacial drifts. Presumpscot Formation consists of glaciomarine silt, clay, and sand that washed out of the Lake Wisconsinan glacier when sea levels were higher than at present. Glacial drifts are characterized by layers of till overlying bedrock on slopes or depressions filled with Presumpscot Formation. Glacial till is a heterogeneous mixture of sand, silt, clay and stones.

The MGS Bedrock Geologic Map of the Bowdoinham Quadrangle, Maine, Open-File No. 10-20 (2010) cites the bedrock at the project site as a granite and pegmatite intrusion and the country bedrock as stratified schist and granofels of the Vassalboro Formation. The borings conducted at the site encountered as schist with granite-like, migmatite zones.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling two test borings and two auger probes to refusal. Borings BB-LMMB-101 and BB-LMMB-104 were drilled outside the south and north corners of the existing steel pipe, respectively; auger probes BB-LMMB-102 and BB-LMMB-103 were drilled outside the west and east corners, respectively. The boring and probe locations are shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile.

Test borings and auger probes were drilled on August 6 and 7, 2018 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.

Borings were performed by using a combination of solid stem auger, cased wash boring and rock coring techniques. Soil samples were typically obtained in 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 MaineDOT 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" prior to the test borings in April 2017. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.854 for both borings. 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 cores calculated. A MaineDOT geotechnical engineer logged the subsurface conditions encountered. The MaineDOT geotechnical engineer selected the boring locations and drilling methods, designated type and depth of sampling techniques, reviewed boring logs and identified field testing requirements. The borings were located in the field using taped measurements at the completion of the drilling program.

4.0 LABORATORY TESTING

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

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered generally consisted of Granular Fill underlain by 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 Granular Fill was encountered in the borings. The thickness of the fill unit encountered was approximately 14.6 to 17.4 feet at the boring locations. The Fill layer encountered generally consisted of:

- Brown, dry, sand, some gravel, trace silt;
- Brown and red brown, dry to wet, gravel, some sand, trace silt;
- Brown, dry, gravelly sand, little silt;
- Brown dry, sandy gravel, trace silt; and
- Cobbles.

SPT N-values in the layer ranged from 8 to 43 blows per foot (bpf), indicating the layer is loose to dense in consistency. Grain size analyses conducted on the Fill classified the soils as A-1-b under the AASHTO Soil Classification System and SW-SM under the Unified Soil Classification System (USCS). The natural water contents of the samples tested ranged from approximately 2 to 4 percent.

5.2 Bedrock

Bedrock was encountered and cored in borings BB-LMMB-101 and BB-LMMB-104. Auger probes BB-LMMB-102 and BB-LMMB-103 refused at what is assumed to be the bedrock surface.

Table 1 summarizes approximate depth to bedrock, corresponding approximate top of bedrock elevation, and RQD.

Boring/Auger Probe	Station	Offset (feet)	Approximate Depth to Bedrock (feet)	Approximate Elevation of Bedrock Surface (feet)	RQD (R1,R2) (%)
BB-LMMB-101	103+49.8	7.4 Rt	16.4	140.2	42, 92
BB-LMMB-102	103+52.7	5.5 Lt	17.4	139.7[1]	-
BB-LMMB-103	103+80.2	8.2 Rt	16.2	140.4[1]	-
BB-LMMB-104	103+81.7	5.2 Lt	15.6	141.4	37, 67

[1] Inferred bedrock surface based on auger refusal

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

The bedrock at the site is identified as white to black to deeply rust weathered, coarse grained quart-muscovite-biotite SCHIST with MIGMATIZE zones, moderately hard, moderately to severely weathered, joints/fractures are low angle to moderately dipping, at close to moderately close spacing, tight to open, with sandy infilling. The MIGMATITE zones are more competent, fresh, massive and very hard. Detailed bedrock descriptions and the RQD core run are provided on the boring logs in Appendix A – Boring Logs. Photographs of bedrock cores are provided in Appendix B – Bedrock Core Photographs.

An exposed intrusion of granite and pegmatite was observed at the site on the downstream side.

5.3 Groundwater

Groundwater was not observed in the boreholes. Water was introduced into the boreholes during drilling operations. Groundwater levels will fluctuate with changes in river water elevation, seasonally, with precipitation, runoff, and construction activities.

6.0 FOUNDATION ALTERNATIVES

Two culvert replacement options were evaluated during preliminary design:

- A precast concrete box culvert;
- A three-sided, natural bottomed, concrete arch/frame.

With bedrock being close to the roadway surface, installing a four-sided concrete box would be problematic and require a grade raise. A three-sided frame was determined to be better suited dimensionally to the shallow bedrock conditions. In addition, the three-sided frame allows a more natural bottom (bedrock) and better water flow. Therefore, the proposed alternative is a 17-foot span by 7-foot rise, three-sided concrete frame on cast-in-place concrete footings on bedrock.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

The project will require cast-in-place spread footings on bedrock to support the stem wasll for the precast concrete frame. Design recommendations in this Section are provided in accordance with AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017 (herein referred to as LRFD).

7.1 Precast Concrete Arch and Frame Design and Construction

Precast concrete arches and frames will typically be detailed on the contract plans with only basic layout and required hydraulic opening. The manufacturer selected by the Contractor is responsible for the design of the structure including determination of wall thickness, haunch thickness and reinforcement. Precast concrete arches and frames shall be designed in accordance with MaineDOT Standard Specification 534 - Precast Structural Concrete, MaineDOT Bridge Design Guide (BDG) Section 8 – Buried Structures and AASHTO LRFD.

The loading specified for the design of the frame shall be Modified HL-93 Strength I in which the HS-20 design truck wheel loads are increased by a factor of 1.25. The design should use Soil Type 4 as presented in the MaineDOT BDG Section 3.6 to design earth loads from the soil envelope. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf.

The soil envelope and backfill shall consist of Standard Specification 703.19 – Granular Borrow Material for Underwater Backfill with a maximum particle size of 4 inches. The granular borrow backfill should be placed in lifts of 6 to 8 inches thick, loose measure, and compacted to the manufacturer's specifications. In no case shall the backfill soil be compacted less than 92 percent of the AASHTO T-180 maximum dry density.

7.2 Concrete Headwalls

Concrete headwalls may be included in the buried structure design to retain riprap slopes and prevent riprap from dropping or eroding into the waterway. Nominal 1-foot by 1-foot concrete headwalls are recommended.

Headwalls that are fixed to the arch or frame should be designed using an at-rest earth pressure coefficient, K_o, of 0.47, assuming the walls are to be prevented from movement. 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 3 in Section 7.6 of this report.

7.3 Spread Footings on Bedrock

Bedrock was encountered at depths ranging from approximately 15.6 to 17.4 feet below the roadway surface at the proposed concrete frame location. Spread footings can be practically and economically constructed to bear on bedrock at this location, possibly without temporary soil support systems. The borings indicate that bedrock with an RQD of approximately 37 to 42 percent will be encountered at the bedrock surface at the boring location.

Based on the borings conducted at the anticipated locations for footings supporting a 17-foot span buried structure, the approximate bedrock surface is estimated to range from approximate El. 139 to 142 feet. The thickness of the frame stem wall footings and wingwall footings may be designed to vary in thickness to accommodate variations in the bedrock surface while maintaining a constant top of footing elevation.

7.4 Bearing Resistance of Spread Footings on Bedrock

Cast-in-place spread footings shall be proportioned to provide stability against bearing capacity failure. Application of permanent and transient loads shall be as specified in LRFD Article 11.5.5.

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.

The bearing resistance of cast-in-place spread footings constructed on bedrock shall be investigated at the service limit state using factored loads and a factored bearing resistance of 20 ksf. Resistance factors for the service limit state are taken as 1.0. A factored bearing resistance of 20 ksf shall also be used to control settlement when analyzing the footing for service limit state load and for preliminary footing sizing as allowed in LRFD C10.6.2.1.

Once the dimension of the cast-in-place spread footings is determined, the designer shall confirm that the factored bearing resistance at the strength limit state is greater than the applied factored vertical bearing pressure. The factored bearing resistance of the bedrock at the strength limit state has been calculated to be 11 ksf. This factored bearing resistance assumes a resistance factor, φ_b, for spread footings on bedrock of 0.45, based on bearing resistance evaluation using semi-empirical methods. However, the service limit state bearing resistance may govern the design. See Appendix D – Calculations for supporting calculations.

For footings on rock, the location of the resultant of the reaction forces shall be within the middle nine-tenths (9/10) of the base width.

In no instance shall the bearing stress exceed the nominal structural resistance of the structural concrete which may be taken as 0.3*f'c*. From a practical perspective, no footing shall be less than 3 feet wide regardless of the applied bearing pressure or bearing material.

7.5 Spread Footing Design - Frame Stem Wall Footing and Wingwalls

Spread footings and frame stem walls and wingwalls shall be designed for all relevant strength, service and extreme limit state load combinations specified in LRFD Articles 3.4.1 and 11.5.5 and 12.5. The design of spread footings at the strength limit state shall consider:

- bearing resistance,
- eccentricity,
- lateral sliding,
- reinforced-concrete structural design.

Wingwalls shall be designed as conventional retaining walls for all relevant strength and service limit state load combinations specified in LRFD Articles 3.4.1, 11.5.5 and 11.6.

For the scour protection of concrete frame spread footings and wingwall footings, project plans shall require construction of footings directly on bedrock surfaces cleaned of all soil and weathered, loose or potentially erodible rock. The strength and extreme event limit state designs consider foundation resistance after the design or check floods for scour. Buried structures and walls should be designed so that no movement of any part of the structure will occur as a result of scour. Extreme limit state design checks shall include those load combinations relating to certain hydraulic events and ice (if warranted by ice history or stream constriction by the buried structure). Resistance factors, φ, for the extreme event limit state shall be taken as 1.0, with the exception of bearing resistance of retaining walls for which a resistance factor of 0.80 shall be used.

For sliding analyses, a sliding resistance factor, φ_τ, of 0.80 shall be applied to the nominal sliding resistance of cast-in-place arch/frame spread footings constructed on bedrock.

Assuming that the rock subgrade will be prepared in-the-wet, some amount of sediment is expected to remain on the rock surface and the sliding computations for resistance of footings to lateral loads shall assume a maximum friction coefficient of 0.60 ($C^x \tan\phi_f$) at the bedrock-to-concrete interface. If the rock subgrade is prepared in-the-dry and cleaned with high pressure water and air prior to placing footing concrete, sliding computations for resistance to lateral loads may assume a maximum frictional coefficient of 0.70 ($C^x \tan\phi_f$) at level bedrock-to-concrete interfaces.

Anchorage of the footings to bedrock may be required 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 designer. If bedrock is observed to slope steeper than 4H:1V at the footing subgrade elevation, the bedrock should be benched to create level steps.

For spread footings cast directly on bedrock, the eccentricity of loading at the strength limit state, based on factored loads, 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.

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, and therefore a global stability evaluation may be waived.

7.6 Earth Pressures, Load Factors and Surcharge Forces

Calculation of earth pressures acting on frames and their footings should assume an at-rest earth pressure coefficient, K_o, of 0.47, assuming the frame footings are to be prevented from movement. Calculation of earth pressures mobilized to resisting outward thrust forces from the arches shall also assume an at-rest earth pressure coefficient, K_o, of 0.47. Based on LRFD Table 3.4.1-2 a resistance factor γ_{EH} of 0.90 is recommended for at-rest earth pressures mobilized to resist lateral outward thrust forces within the frame walls. For designing the stem wall or footing reinforcing steel for at-rest earth pressures resisting outward thrust forces, a maximum load factor, γ_{EH}, of 1.35 is recommended.

The designer may assume Soil Type 4 (MaineDOT BDG Section 3.6.1) for frame footing and wall backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 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. The live load surcharge on frame/arch stem walls may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from the Table 2 below:

Frame Stem Wall Height (feet)	h_{eq} (feet)
5	4.0
10	3.0
≥20	2.0

Table 2 - Equivalent Height of Soil for Estimating Live Load Surcharge

Wingwalls that are independent of the frame shall be designed as unrestrained meaning they are free to rotate at the top. Earth pressures shall be calculated using an active earth pressure coefficient, K_a, of 0.31, calculated using Rankine theory and assuming a level backslope. Wingwall sections with 2H:1V backslopes shall be designed using a Rankine active earth pressure coefficient, K_a, of 0.47. The recommended soil properties for Soil Type 4 to be used as backfill properties are: $\phi = 32°$ and $\gamma = 125$ pcf.

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 Table 3 below:

Retaining Wall Height (feet)	h_{eq} (feet)	
	Distance from wall pressure surface to edge of traffic = 0 feet	Distance from wall pressure surface to edge of traffic >=1 foot
5	5.0	2.0
10	3.5	2.0
>=20	2.0	2.0

Table 3 - Equivalent Height of Soil for Estimating Live Load Surcharge on Walls

Frame foundations 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 arches, frames 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 reduce the amount of fines and to minimize frost action behind the structure.

7.7 Settlement

No significant vertical or horizontal alignment changes are currently planned for the bridge replacement. We anticipate that all foundations will be constructed on bedrock. Therefore, we expect that any settlement of the foundations will be due to elastic compression of the bedrock and will be negligible.

The soil envelope and backfill for the precast frames and arches shall consist of Standard Specification 703.19 – Granular Borrow Material for Underwater Backfill with a maximum particle size of 4 inches. The granular borrow backfill should be placed in lifts of 6 to 8 inches thick loose measure and compacted to the manufacturer's specifications. To minimize post-construction settlement, the envelope and backfill soil shall be compacted to no less than 92 percent of the AASHTO T-180 maximum dry density.

7.8 Frost Protection

We anticipate that the structure footings will be founded directly on bedrock. For foundations on bedrock heave due to frost is not a design issue and no requirements for minimum depth of embedment are necessary.

Foundations placed on the native soils should be designed with an appropriate embedment for frost protection. According to BDG Figure 5-1, Maine Design Freezing Index Map, Litchfield has an air design freezing index of approximately 1500 F-degree days. An assumed water content of 10% was used for coarse-grained soils at the potential elevation of a footing. These components correlate to a frost depth of 6.8 feet.

We recommend that foundations constructed on soil be designed with an embedment of 6.8 feet for frost protection. See Appendix D – Calculations for supporting calculations.

7.9 Scour and Riprap

The buried structures and any wingwalls will be founded on spread footings founded on bedrock. For scour protection of the arch or frame 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. We anticipate that the remaining bedrock subgrade will be competent and is therefore not considered to be erodible or scourable. Therefore, no specific scour protection recommendations are needed for the foundations other than armoring with riprap.

Wingwalls shall be extended far enough from the structure to protect the structural portion of the soil envelope surrounding the precast concrete frame.

We recommend that sideslopes and footings supporting the structures 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 streambed consists of bedrock. The riprap slopes shall be constructed no steeper than a maximum 1.75H:1V extending from the edge of the roadway down to the existing ground surface.

7.10 Seismic Design Considerations

In conformance with LRFD Article 3.10.1, seismic analysis is not required for buried structures, except where they cross active faults. There are no known active faults in Maine; therefore seismic analysis is not required.

7.11 Construction Considerations

The precast concrete arches/frames shall be constructed in conformance with MaineDOT Bridge Design Guide (BDG) Section 8 and MaineDOT Standard Specification 534 – Precast Structural Concrete.

Construction of the arch or frame spread footings, headwalls and wingwalls will require soil and rock excavation and removal of the existing culvert. Cofferdams and temporary earth support systems may be required to permit construction of arch footings and wingwalls in the dry.

Construction activities should not be permitted to disturb the bedrock mass or create any open fissures. Irregularities in the existing bedrock surface or irregularities created during the excavation process should be backfilled with unreinforced, Class A concrete to the bearing elevation. Footings may be stepped for varying depths to bedrock along the centerline of the footing. The bottom of the footing elevation may vary based on the presence of fractured bedrock.

The subgrade for spread footings for arches, frames and retaining walls shall consist of sound bedrock. The nature, slope and degree of fracturing in the bedrock bearing surfaces 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 final bearing surface of bedrock shall be washed with high pressure water and air prior to concrete being placed for the arch or frame and wingwall footings

The slope of the bedrock subgrade for foundations shall be no steeper than 4H:1V or it shall be benched in level steps or excavated to be completely level. This criterion also applies for the bedrock subgrade for any wingwall footings. Anchoring, doweling or other means of improving sliding resistance may also be employed where the prepared bedrock surface is steeper than 4H:1V in any direction, at arch or wingwall footings.

Excavation of bedrock material may be done using conventional excavation methods, 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, as well as blast vibration monitoring at nearby residences and bridge structures in accordance with industry standards at the time of the blast.

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

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 foundation 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 all foundations are constructed in the dry.

Exposed soils may become saturated and water seepage may be encountered during construction. There may be localized sloughing and instability in some excavations and cut slopes. The contractor should control groundwater, surface water infiltration and soil erosion. Water should be controlled by pumping from sumps.

8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Palmer Bridge in Litchfield, 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 final design and specifications so that the earthwork and foundation recommendations and construction considerations in this report are properly interpreted and implemented in the design and specifications.

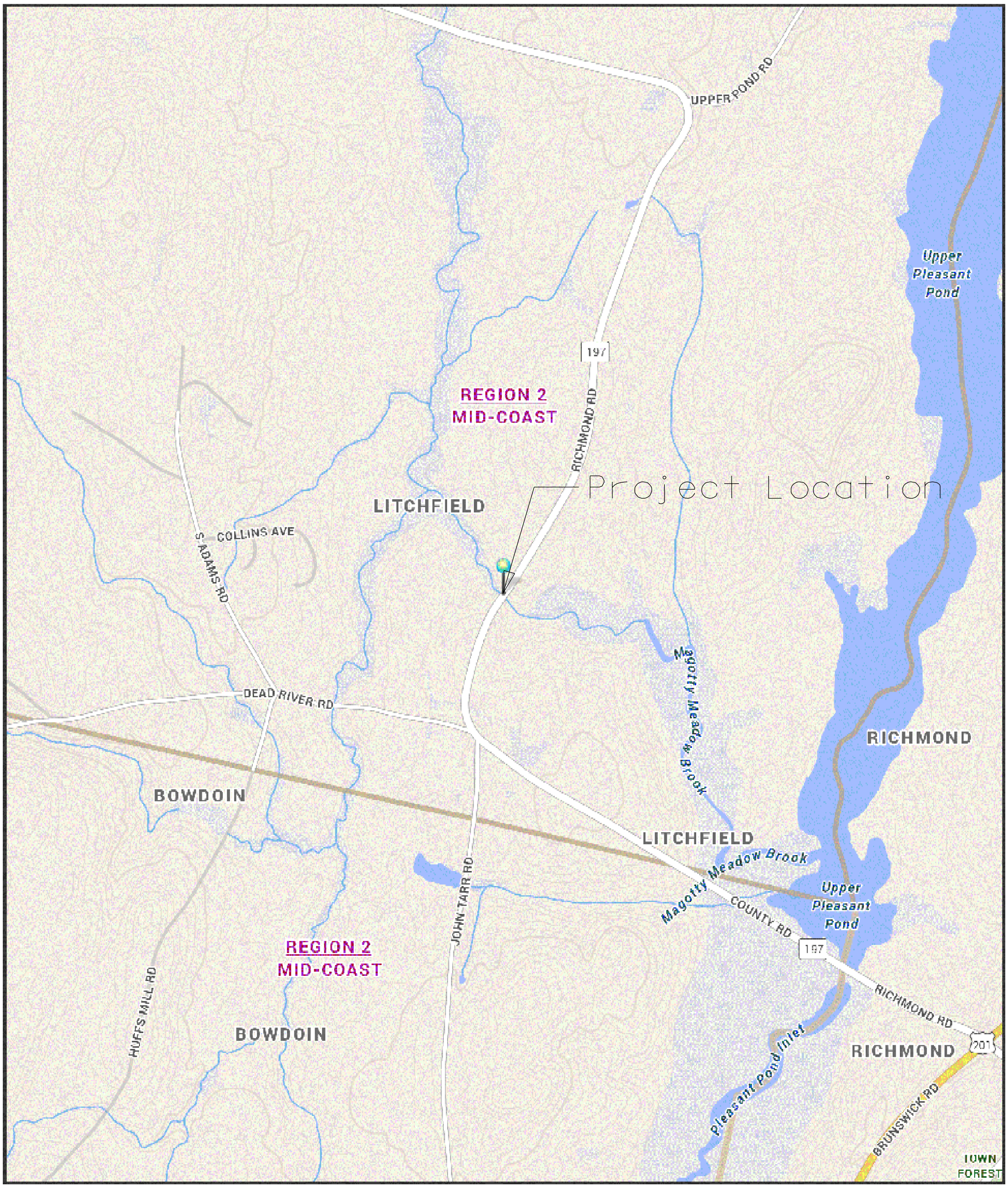
<u>Sheets</u>

Date:6/20/2019

Division: GEOTECH

Username: Terry.White

LITCHFIELD



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0.25 Miles

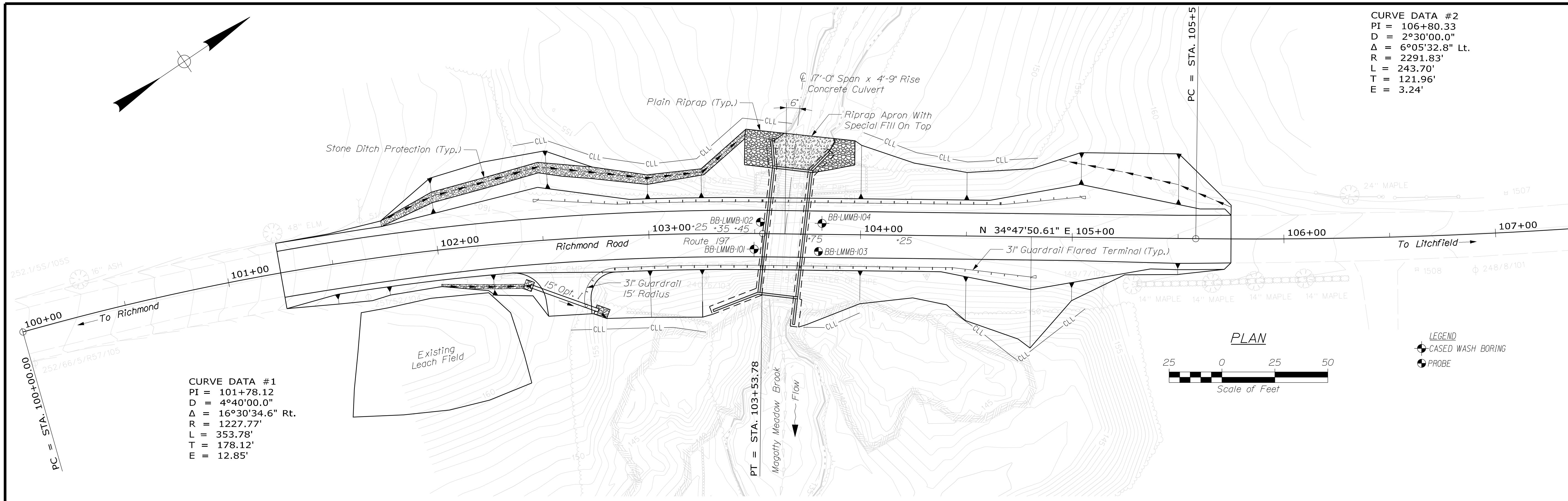
1 inch = 0.28 miles

Date: 6/20/2019
Time: 11:00:23 AM

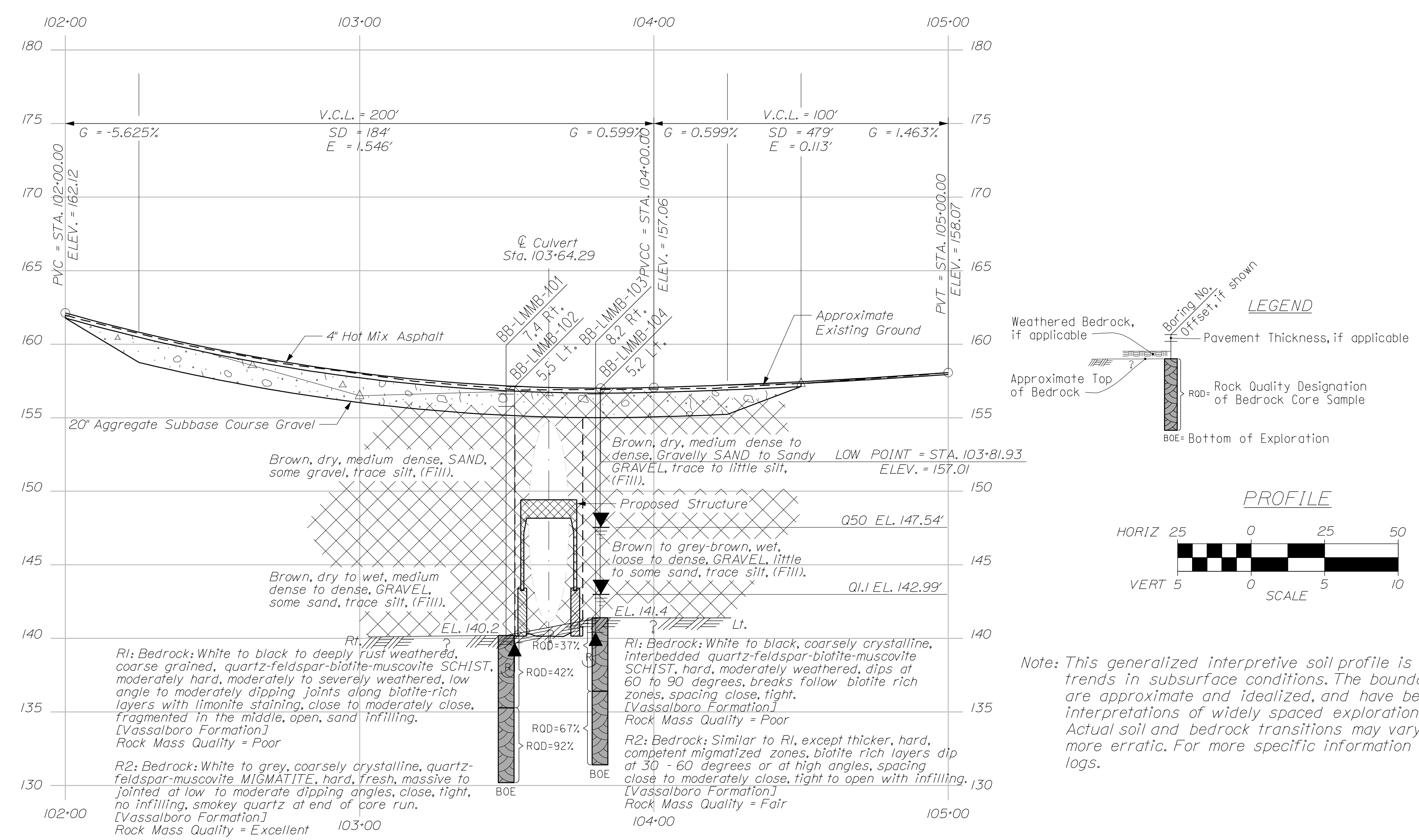
SHEET NUMBER	PALMER BRIDGE MAGOTTY MEADOW BROOK LITCHFIELD KENNEBEC COUNTY	STATE OF MAINE DEPARTMENT OF TRANSPORTATION
1		22246.00
OF 3	LOCATION MAP	WIN BRIDGE NO. 5141 22246.00 BRIDGE PLANS

Filename: ...\GEOTECH\MSTA\004_BLP&SP1.dgn Division: GEOTECH Username: Laura.Krusinski Date:11/5/2019

CURVE DATA #2
PI = 106+80.33
D = 2°30'00.0"
Δ = 6°05'32.8" Lt.
R = 2291.83'
L = 243.70'
T = 121.96'
E = 3.24'



CURVE DATA #1
PI = 101+78.12
D = 4°40'00.0"
Δ = 16°30'34.6" Rt.
R = 1227.77'
L = 353.78'
T = 178.12'
E = 12.85'



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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
22246.00

WIN 22246.00
BRIDGE NO. 5141
BRIDGE PLANS

	BY	DATE
PROJ. MANAGER JOEL KITTREDGE		
DESIGN-DETAILED	J.LEAVITT	
CHECKED-REVIEWED		
DESIGN2-DETAILED2	L.KRUSINSKI	T.WHITE	NOV 2019
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

PALMER BRIDGE
MAGOTTY MEADOW BROOK
LITCHFIELD KENNEBEC COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
2
OF 3

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

Project: Palmer Bridge #5141 carries State Route 197 over Magotty Meadow
Location: Litchfield, Maine

Boring No.: BB-LMMB-101
WIN: 22246.00

Driller:	MaineDOT	Elevation (ft.)	156.6	Auger ID/OD:	5" Solid Stem
Operator:	Daggett/Niles	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	A. Van Buskirk	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/6/2018; 08:30-11:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	103+49.8, 7.4 ft Rt.	Casing ID/OD:	NW-3"	Water Level*:	None Observed
Hammer Efficiency Factor: 0.928		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)
0							SSA	155.8
	1D	24/20	1.00 - 3.00	10/9/7/6	16	25		
5	2D	9.6/4	5.00 - 5.80	10/50(3.6")	---		25	
							27	
							32	
							27	
							35	
10	3D	24/2	10.00 - 12.00	4/5/6/5	11	17	8	
							24	
							23	
							29	
							35	
15	4D	8.4/7	15.00 - 15.70	12/50(2.4")	---		50	
	R1	59/57	16.40 - 21.32	RQD = 42%			a14 NQ-2	140.2
20	R2	61/61	21.40 - 26.48	RQD = 92%				135.2
25								130.2

Visual Description and Remarks:

10" HMA. (0.8)

Brown, dry, medium dense, SAND, some gravel, trace silt, (Fill). — Laboratory Testing Results/AASHTO and Unified Class: G#296594 A-1-b, SW-SM WC=3.8%

Brown, dry, GRAVEL, some sand, trace silt, coarse gravel wedged in tip of spoon, (Fill).

Brown, wet, medium dense, GRAVEL, some sand, trace silt, (Fill).

Red-brown, wet, GRAVEL, some sand, trace silt, (Fill).

a14 blows for 0.4 ft. (16.4)

Top of Bedrock at Elev. 140.2 ft.
R1: Bedrock: White to black to deeply rust weathered, coarse grained, quartz-feldspar-biotite-muscovite SCHIST, moderately hard, moderately to severely weathered, low angle to moderately dipping joints along biotite rich layers with limonite staining, close to moderately close, fragmented in the middle, open, sandy infilling.
[Vassalboro Formation]
Rock Mass Quality = Poor
R1: Core Times (min:sec)
16.4-17.4 ft (0:57)
17.4-18.4 ft (0:57)
18.4-19.4 ft (3:20)
19.4-20.4 ft (1:13)
20.4-21.4 ft (5:44)
100% Recovery (21.4)

R2: Bedrock: White to grey, coarsely crystalline, quartz-feldspar-muscovite MIGMATITE, hard, fresh, massive, three joints at low to moderate dipping angles, tight, no infilling, smokey quartz vein at end of core run.
[Vassalboro Formation]
Rock Mass Quality = Excellent
R2: Core Times (min:sec)
21.4-22.4 ft (3:57)
22.4-23.4 ft (4:21)
23.4-24.4 ft (4:10)
24.4-25.4 ft (3:20)
25.4-26.4 ft (3:27)
100% Recovery (26.4)

Bottom of Exploration at 26.4 feet below ground surface.

Remarks:

NW Casing crooked at 10.0 ft bgs., leaning away from rig.

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

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

Page 1 of 1 — Boring No.: BB-LMMB-101

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

Project: Palmer Bridge #5141 carries State Route 197 over Magotty Meadow
Location: Litchfield, Maine

Boring No.: BB-LMMB-102
WIN: 22246.00

Drilling Contractor:	MaineDOT	Elevation (ft.)	157.1	Auger ID/OD:	5" Dia.
Operator:	Daggett/Niles	Datum:	NAVD88	Sampler:	N/A
Logged By:	A. Van Buskirk	Rig Type:	CME 45C	Hammer Wt./Fall:	N/A
Date Start/Finish:	8/7/2018; 13:00-14:00	Drilling Method:	Solid Stem Auger	Core Barrel:	N/A
Boring Location:	103+52.7, 5.5 ft Lt.	Casing ID/OD:	N/A	Water Level*:	None Observed

Definitions:
D = Split Spoon Sample
S = Sample off Auger Flights
B = Bucket Sample off Auger Flights
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MV = Unsuccessful Field Vane Shear Test Attempt
V = Field Vane Shear Test, PP= Pocket Penetrometer
MU = Unsuccessful Thin Wall Tube Sample 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 1 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-value = Raw Field SPT N-value
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent ? = Similar or Equal too
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)
0						SSA	
17.4							139.70

Visual Description and Remarks:

No soil samples recovered or descriptions recorded.

Bottom of Exploration at 17.4 feet below ground surface. (17.4)
REFUSAL

Remarks:

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

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

Page 1 of 1 — Boring No.: BB-LMMB-102

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

Project: Palmer Bridge #5141 carries State Route 197 over Magotty Meadow
Location: Litchfield, Maine

Boring No.: BB-LMMB-103
WIN: 22246.00

Drilling Contractor:	MaineDOT	Elevation (ft.)	156.6	Auger ID/OD:	5" Dia.
Operator:	Daggett/Niles	Datum:	NAVD88	Sampler:	N/A
Logged By:	A. Van Buskirk	Rig Type:	CME 45C	Hammer Wt./Fall:	N/A
Date Start/Finish:	8/6/2018; 12:00-13:00	Drilling Method:	Solid Stem Auger	Core Barrel:	N/A
Boring Location:	103+80.2, 8.2 ft Rt.	Casing ID/OD:	N/A	Water Level*:	None Observed

Definitions:
D = Split Spoon Sample
S = Sample off Auger Flights
B = Bucket Sample off Auger Flights
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MV = Unsuccessful Field Vane Shear Test Attempt
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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 1 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-value = Raw Field SPT N-value
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent ? = Similar or Equal too
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)
0						SSA	
16.2							140.40

Visual Description and Remarks:

No soil samples recovered or descriptions recorded.

Bottom of Exploration at 16.2 feet below ground surface. (16.2)
REFUSAL

Remarks:

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

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

Page 1 of 1 — Boring No.: BB-LMMB-103

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

Project: Palmer Bridge #5141 carries State Route 197 over Magotty Meadow
Location: Litchfield, Maine

Boring No.: BB-LMMB-104
WIN: 22246.00

Driller:	MaineDOT	Elevation (ft.)	157.0	Auger ID/OD:	5" Solid Stem
Operator:	Daggett/Niles	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	A. Van Buskirk	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/7/2018; 08:30-11:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	103+81.7, 5.2 ft Lt.	Casing ID/OD:	NW-3"	Water Level*:	None Observed
Hammer Efficiency Factor: 0.928		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)
0							SSA	
	1D	24/14	1.00 - 3.00	9/16/12/11	28	43		
5	2D	24/4	5.00 - 7.00	5/5/6/9	11	17	14	
							23	
							16	
							21	
							27	
10							a1	
	3D	24/1	11.00 - 13.00	4/3/2/8	5	8	14	
							31	
							40	
							97	
15	4D R1	4.8/3 60/60	15.00 - 15.40 15.60 - 20.60	64(4.8") RQD = 37%	---		NQ-2	141.4
20	R2	60/59	20.60 - 25.60	RQD = 67%				
25								131.4

Visual Description and Remarks:

Brown, dry, dense, Gravelly SAND, little silt, (Fill). — Laboratory Testing Results/AASHTO and Unified Class: G#296595 A-1-b, SW-SM WC=1.8%

Brown, dry, medium dense, Sandy GRAVEL, trace silt, (Fill).

Roller Coned through cobble from 9.7-9.9 ft bgs.
aRoller Coned ahead to 11.0 ft bgs.

Brown, wet, loose, GRAVEL, some coarse sand, trace silt.

Grey brown, wet, GRAVEL, little sand, trace silt.
Roller Coned ahead to 15.6 ft bgs to top of bedrock. (15.6)

Top of Bedrock at Elev. 141.4 ft.
R1: Bedrock: White to black, coarsely crystalline, quartz-feldspar-biotite-muscovite SCHIST, hard, moderately weathered, dips at 60 to 90 degrees, breaks follow biotite rich zones, spacing close, tight.
[Vassalboro Formation]
Rock Mass Quality = Poor
R1: Core Times (min:sec)
15.6-16.6 ft (2:17)
16.6-17.6 ft (1:34)
17.6-18.6 ft (1:50)
18.6-19.6 ft (2:09)
16.9-20.6 ft (3:07)
100% Recovery

R2: Bedrock: Similar to R1, except thicker, hard, competent migmatized zones, biotite rich layers dip at 30 - 60 degrees or at high angles, spacing close to moderately close, open to tight, with infilling.
[Vassalboro Formation]
Rock Mass Quality = Fair
R2: Core Times (min:sec)
20.6-21.6 ft (2:39)
21.6-22.6 ft (1:58)
22.6-23.6 ft (3:20)
23.6-24.6 ft (3:00)
24.6-25.6 ft (2:20)
98% Recovery (25.6)

Bottom of Exploration at 25.6 feet below ground surface.

Remarks:

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

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

Page 1 of 1 — Boring No.: BB-LMMB-104

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
22246.00
BRIDGE NO. 5141
WIN 22246.00
BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	JOEL KITTREDGE		
DESIGN-DETAILED		J.LEAVITT	
CHECKED-REVIEWED			
DESIGN2-DETAILED2	L.KRUSINSKI	T.WHITE	NOV 2019
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE
P.E. NUMBER
DATE

PALMER BRIDGE
MAGOTTY MEADOW BROOK
LITCHFIELD KENNEBEC COUNTY

BORING LOGS

SHEET NUMBER 3 OF 3

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

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

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

MODIFIED BURMISTER SYSTEM

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations (in this order, if applicable):
Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Rock Type (granite, schist, sandstone, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
-dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.)
-spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet)
-tightness (tight, open, or healed)
-infilling (grain size, color, etc.)
Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock mass quality (very poor, poor, etc.)
ref: ASTM D6032 and AASHTO Standard Specification for Highway Bridges, 17th Ed. Table 4.4.8.1.2A
Recovery (inch/inch and percentage)
Rock Core Rate (X.X ft - Y.Y ft (min:sec))

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Palmer Bridge #5141 carries State Route 197 over Magotty Meadow Brook
Location: Litchfield, Maine

Boring No.: BB-LMMB-101
WIN: 22246.00

Drilling Contractor:	MaineDOT	**Elevation (ft.)**	156.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/6/2018; 08:30-11:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	103+49.8, 7.4 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed

Definitions: D = Spilt Spoon Sample
S = Sample off Auger Flights
B = Bucket Sample off Auger Flights
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MV = Unsuccessful Field Vane Shear Test Attempt
V = Field Vane Shear Test, PP= Pocket Penetrometer
MU = Unsuccessful Thin Wall Tube Sample 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
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$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-value = Raw Field SPT N-value
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent ≅ = Similar or Equal too
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA			10" HMA.	
							155.77		0.83	
	1D	24/20	1.00 - 3.00	10/9/7/6	16				Brown, dry, medium dense, SAND, some gravel, trace silt, (Fill).	G#296594 A-1-b, SW-SM WC=3.8%
5	2D	9.6/4	5.00 - 5.80	10/50(3.6")	---	25			Brown, dry, GRAVEL, some sand, trace silt, coarse gravel wedged in tip of spoon, (Fill).	
						27				
						32				
						27				
						35				
10	3D	24/2	10.00 - 12.00	4/5/6/5	11	8			Brown, wet, medium dense, GRAVEL, some sand, trace silt, (Fill).	
						24				
						23				
						29				
						35				
15	4D	8.4/7	15.00 - 15.70	12/50(2.4")	---	50			Red-brown, wet, GRAVEL, some sand, trace silt, (Fill).	
									a14 blows for 0.4 ft.	
	R1	59/57	16.40 - 21.32	RQD = 42%		a14 NQ-2	140.20		16.40 Top of Bedrock at Elev. 140.2 ft. R1: Bedrock: White to black to deeply rust weathered, coarse grained, quartz-feldspar-biotite-muscovite SCHIST, moderately hard, moderately to severely weathered, low angle to moderately dipping joints along biotite- rich layers with limonite staining, close to moderately close, fragmented in the middle, open, sandy infilling. [Vassalboro Formation] Rock Mass Quality = Poor R1: Core Times (min:sec) 16.4-17.4 ft (0:57) 17.4-18.4 ft (0:57) 18.4-19.4 ft (3:20) 19.4-20.4 ft (1:13) 20.4-21.4 ft (5:44) 100% Recovery	
20										
	R2	61/61	21.40 - 26.48	RQD = 92%			135.20		21.40 R2: Bedrock: White to grey, coarsely crystalline, quartz-feldspar-muscovite	
25										

Remarks:

NW Casing crooked at 10.0 ft bgs., leaning away from rig.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Palmer Bridge #5141 carries State Route 197 over Magotty Meadow Brook
Location: Litchfield, Maine

Boring No.: BB-LMMB-101
WIN: 22246.00

Drilling Contractor:	MaineDOT	**Elevation (ft.)**	156.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/6/2018; 08:30-11:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	103+49.8, 7.4 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed

Definitions: D = Spilt Spoon Sample
S = Sample off Auger Flights
B = Bucket Sample off Auger Flights
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MV = Unsuccessful Field Vane Shear Test Attempt
V = Field Vane Shear Test, PP= Pocket Penetrometer
MU = Unsuccessful Thin Wall Tube Sample 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 1 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-value = Raw Field SPT N-value
T_v = Pocket Torvane Shear Strength (psf)
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G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							130.20		MIGMATITE, hard, fresh, massive, three joints at low to moderate dipping angles, tight, no infilling, smokey quartz vein at end of core run. [Vassalboro Formation] Rock Mass Quality = Excellent R2: Core Times (min:sec) 21.4-22.4 ft (3:57) 22.4-23.4 ft (4:21) 23.4-24.4 ft (4:10) 24.4-25.4 ft (3:20) 25.4-26.4 ft (3:27) 100% Recovery — 26.40	
30									**Bottom of Exploration at 26.40 feet below ground surface.**	
35										
40										
45										
50										

Remarks:

NW Casing crooked at 10.0 ft bgs., leaning away from rig.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Palmer Bridge #5141 carries State Route 197 over Magotty Meadow Brook
Location: Litchfield, Maine

Boring No.: BB-LMMB-102
WIN: 22246.00

Drilling Contractor:	MaineDOT	**Elevation (ft.)**	157.1	**Auger ID/OD:**	5" Dia.
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	N/A
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	8/7/2018; 13:00-14:00	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	N/A
Boring Location:	103+52.7, 5.5 ft Lt.	**Casing ID/OD:**	N/A	**Water Level*:**	None Observed

Definitions: D = Spilt Spoon Sample
S = Sample off Auger Flights
B = Bucket Sample off Auger Flights
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MV = Unsuccessful Field Vane Shear Test Attempt
V = Field Vane Shear Test, PP= Pocket Penetrometer
MU = Unsuccessful Thin Wall Tube Sample 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 1 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-value = Raw Field SPT N-value
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent ≅ = Similar or Equal too
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA			No soil samples recovered or descriptions recorded.	
5										
10										
15										
							139.70		17.40	
									Bottom of Exploration at 17.40 feet below ground surface. REFUSAL	
20										
25										

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Palmer Bridge #5141 carries State Route 197 over Magotty Meadow Brook
Location: Litchfield, Maine

Boring No.: BB-LMMB-103
WIN: 22246.00

Drilling Contractor:	MaineDOT	**Elevation (ft.)**	156.6	**Auger ID/OD:**	5" Dia.
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	N/A
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	8/6/2018; 12:00-13:00	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	N/A
Boring Location:	103+80.2, 8.2 ft Rt.	**Casing ID/OD:**	N/A	**Water Level*:**	None Observed

Definitions: D = Spilt Spoon Sample
S = Sample off Auger Flights
B = Bucket Sample off Auger Flights
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MV = Unsuccessful Field Vane Shear Test Attempt
V = Field Vane Shear Test, PP= Pocket Penetrometer
MU = Unsuccessful Thin Wall Tube Sample 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 1 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-value = Raw Field SPT N-value
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent ≅ = Similar or Equal too
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA			No soil samples recovered or descriptions recorded.	
5										
10										
15										
							140.40		16.20	
									Bottom of Exploration at 16.20 feet below ground surface. REFUSAL	
20										
25										

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.

Boring No.: BB-LMMB-103

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Palmer Bridge #5141 carries State Route 197 over Magotty Meadow Brook
Location: Litchfield, Maine

Boring No.: BB-LMMB-104
WIN: 22246.00

Drilling Contractor:	MaineDOT	**Elevation (ft.)**	157.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/7/2018; 08:30-11:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	103+81.7, 5.2 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed

Definitions: D = Spilt Spoon Sample
S = Sample off Auger Flights
B = Bucket Sample off Auger Flights
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MV = Unsuccessful Field Vane Shear Test Attempt
V = Field Vane Shear Test, PP= Pocket Penetrometer
MU = Unsuccessful Thin Wall Tube Sample 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 1 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-value = Raw Field SPT N-value
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent ≅ = Similar or Equal too
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA			
	1D	24/14	1.00 - 3.00	9/16/12/11	28			Brown, dry, dense, Gravelly SAND, little silt, (Fill).	G#296595 A-1-b, SW-SM WC=1.8%
5	2D	24/4	5.00 - 7.00	5/5/6/9	11	14		Brown, dry, medium dense, Sandy GRAVEL, trace silt, (Fill).	
						23			
						16			
						21			
						27		Roller Coned through cobble from 9.7-9.9 ft bgs.	
10						a1		aRoller Coned ahead to 11.0 ft bgs.	
	3D	24/1	11.00 - 13.00	4/3/2/8	5	14		Brown, wet, loose, GRAVEL, some coarse sand, trace silt.	
						31			
						40			
						97			
15	4D	4.8/3	15.00 - 15.40	64(4.8")	---	NQ-2	141.40	Grey brown, wet, GRAVEL, little sand, trace silt. Roller Coned ahead to 15.6 ft bgs to top of bedrock.	
	R1	60/60	15.60 - 20.60	RQD = 37%				15.60	
								Top of Bedrock at Elev. 141.4 ft. R1: Bedrock: White to black, coarsely crystalline, quartz-feldspar-biotite-muscovite SCHIST, hard, moderately weathered, dips at 60 to 90 degrees, breaks follow biotite rich zones, spacing close, tight. [Vassalboro Formation] Rock Mass Quality = Poor R1: Core Times (min:sec) 15.6-16.6 ft (2:17) 16.6-17.6 ft (1:34) 17.6-18.6 ft (1:50) 18.6-19.6 ft (2:09) 16.9-20.6 ft (3:07) 100% Recovery	
20	R2	60/59	20.60 - 25.60	RQD = 67%				R2: Bedrock: Similar to R1, except thicker, hard, competent migmatized zones, biotite-rich layers dip at 30 - 60 degrees or at high angles, spacing close to moderately close, open to tight, with infilling. [Vassalboro Formation] Rock Mass Quality = Fair	
25									

Remarks:

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

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Palmer Bridge #5141 carries State Route 197 over Magotty Meadow Brook **Location:** Litchfield, Maine	**Boring No.:** BB-LMMB-104 **WIN:** 22246.00

Drilling Contractor:	MaineDOT	**Elevation (ft.)**	157.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/7/2018; 08:30-11:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	103+81.7, 5.2 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed

Definitions: D = Spilt Spoon Sample
S = Sample off Auger Flights
B = Bucket Sample off Auger Flights
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MV = Unsuccessful Field Vane Shear Test Attempt
V = Field Vane Shear Test, PP= Pocket Penetrometer
MU = Unsuccessful Thin Wall Tube Sample 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 1 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-value = Raw Field SPT N-value
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent ≅ = Similar or Equal too
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							131.40		R2: Core Times (min:sec) 20.6-21.6 ft (2:39) 21.6-22.6 ft (1:58) 22.6-23.6 ft (3:20) 23.6-24.6 ft (3:00) 24.6-25.6 ft (2:20) 98% Recovery	
									25.60 **Bottom of Exploration at 25.60 feet below ground surface.**	
30										
35										
40										
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.

Boring No.: BB-LMMB-104

Appendix B

Rock Core Photographs

Palmer Bridge #5141 over Magotty Meadow Brook
Litchfield, Maine
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Penetration (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-LMMR-101	R1	16.4-21.4	57	59	25	42%	SCHIST	1
BB-LMMR-101	R2	21.4-26.5	61	61	56	55%	MIGMATITE	2
BB-LMMR-104	R1	15.6-20.6	60	60	22	37%	SCHIST	3
BB-LMMR-104	R2	20.6-25.6	59	60	40	67%	SCHIST W/ MIGMATIZED ZONES	4

Full size photo of Core Box

Notes: 1. “Box row” indicates the section of the box where the core run is contained: 1 = top, 4 = bottom.

MaineDOT

Palmer Bridge #5141 over Magotty Meadow Brook
Litchfield, Maine
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Penetration (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-LMMR-101	R1 – left side	16.4-21.4	57	59	25	42%	SCHIST	1
BB-LMMR-101	R2 – left side	21.4-26.5	61	61	56	55%	MIGMATITE	2
BB-LMMR-104	R1 – left side	15.6-20.6	60	60	22	37%	SCHIST	3
BB-LMMR-104	R2 – left side	20.6-25.6	59	60	40	67%	SCHIST W/ MIGMATITE ZONES	4

Left Side of Core Box

Notes: 1. “Box row” indicates the section of the box where the core run is contained: 1 = top, 4 = bottom.

MaineDOT

Palmer Bridge #5141 over Magotty Meadow Brook
Litchfield, Maine
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Penetration (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-LMMR-101	R1 – right side	16.4-21.4	57	59	25	42%	SCHIST	1
BB-LMMR-101	R2 – right side	21.4-26.5	61	61	56	55%	MIGMATITE	2
BB-LMMR-104	R1 – right side	15.6-20.6	60	60	22	37%	SCHIST	3
BB-LMMR-104	R2 – right side	20.6-25.6	59	60	40	67%	SCHIST W/ MIGMATIZED ZONES	4

Right side of Core Box

Notes: 1. “Box row” indicates the section of the box where the core run is contained: 1 = top, 4 = bottom.

Appendix C

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Litchfield Work Number: 22246.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-LMMB-101, 1D	103+49.8	7.4 Rt.	1.0-3.0	296594	1	3.8			SW-SM	A-1-b	0
BB-LMMB-104, 1D	103+81.7	5.2 Lt	1.0-3.0	296595	1	1.8			SW-SM	A-1-b	0

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

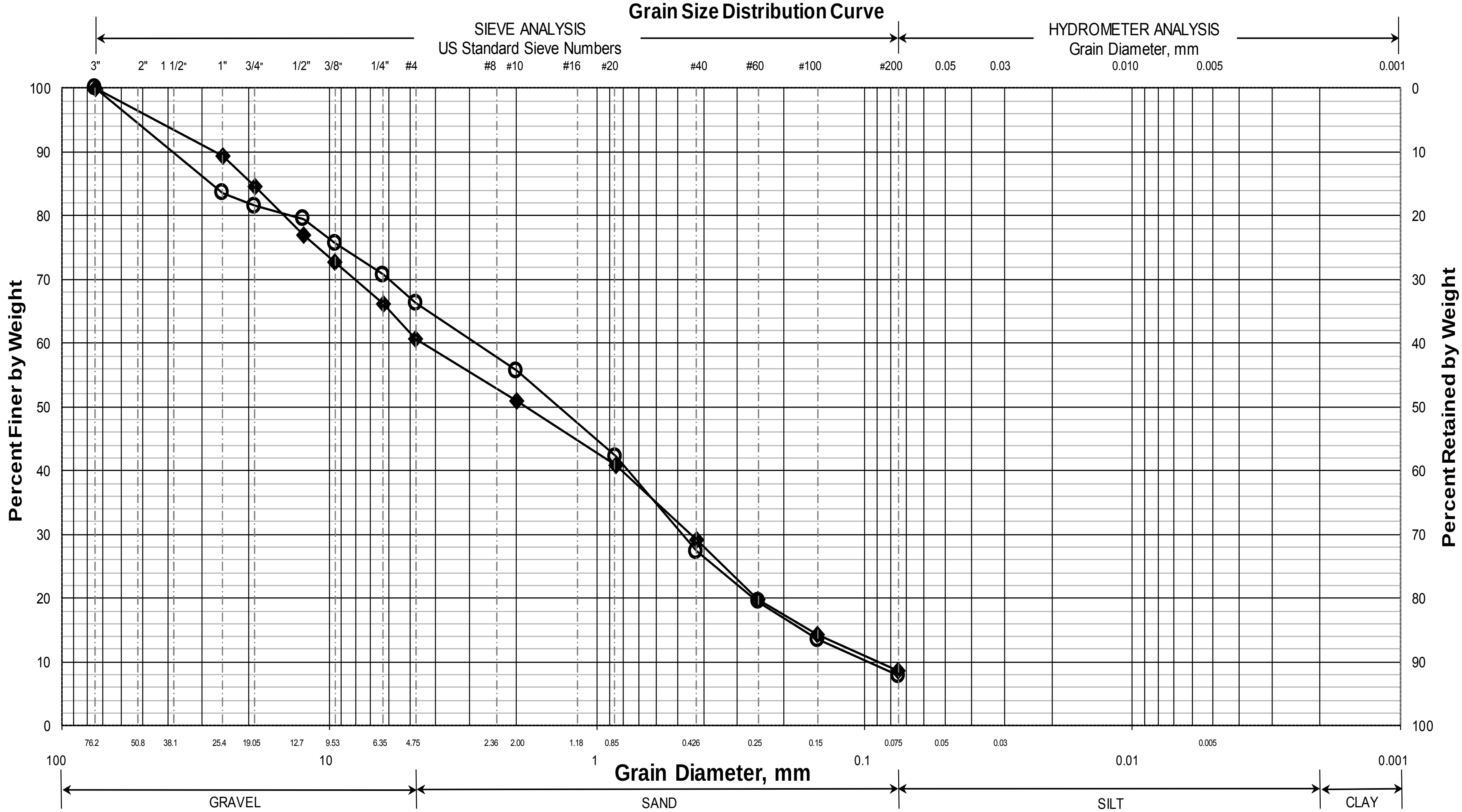
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

Maine Department of Transportation
Grain Size Distribution Curve



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-LMMB-101/1D	103+49.8	7.4 RT	1.0-3.0	SAND, some gravel, trace silt.	3.8			
◆	BB-LMMB-104/1D	103+81.7	5.2 LT	1.0-3.0	Gravelly SAND, trace silt.	1.8			
■									
●									
▲									
×									

WIN	
022246.00	
Town	
Litchfield	
Reported by/Date	
WHITE, TERRY A	8/28/2018

SHEET 1

Appendix D

Calculations

Earth Pressure

Soil Parameters:
Assume existing material removed and replaced with material with properties similar to Soil Type 4, MaineDOT BDG Section 3.6.1.

Unit weight	$\gamma := 125 \cdot pcf$
Internal friction angle	$\phi := 32 \cdot deg$
Cohesion	$c := 0 \cdot psf$

Frame walls and headwalls fixed to frame - At-Rest Earth Pressure - Jaky

Reference: Fang, Foundation Engineering Handbook 2nd ed. Pg. 224, Eq. 6.2
Formula for normally consolidated soils.

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

$$K_o = 0.47$$

Wingwalls free to rotate - 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:

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

$$K_{ar} = 0.31$$

For a sloped 2H:1V backfill

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

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

$$K_{ar\_slope} = 0.47$$

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

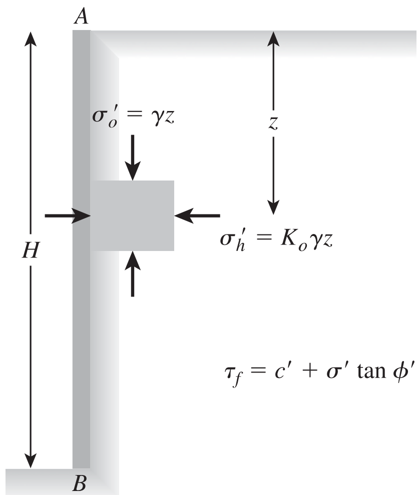


Figure 13.3
Earth pressure at rest

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

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

So,

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

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

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

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)

Figure 3-2 Calculating β with Broken Backfill Surface

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

3.6.5.2 Rankine Theory

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

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

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

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

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 $\beta > 0°$, the coefficient of active earth pressure (Rankine), K_a, may be taken as:

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

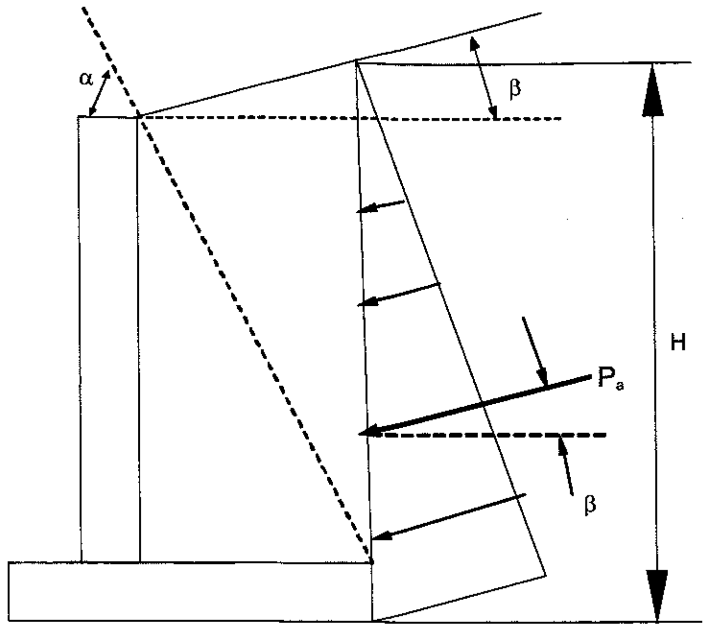


Figure 3-3 Rankine Theory

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

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

3.6.6 Coulomb Passive Lateral Earth Pressure Coefficient

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

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

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.

3.4 Construction Loads

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

3.5 Railroad Loads

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

3.6 Earth Loads

3.6.1 General

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

Table 3-3 Material Classification

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

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

Analysis
Calculation of nominal and factored bearing resistance of bedrock using Rock Mass Rating (RMR) based emperical 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. Wyllie, 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 boring BB-LMMB-101, R1:
White to black to deeply rust weathered, coarse grained, quartz-feldspar-biotite-muscovite SCHIST, moderately hard, moderately to severely weathered, low angle to moderately dipping joints along biotite layers with limonite staining, close to medium close, fragmented in the middle, open, sandy infilling along biotite rich joints.
RQD = 42%

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.

Coarse-grained igneous and metamorphic crystalline rock - Schist.
C_o = 1,400 - 21,000 psi
Choose 5,000 psi

$q_{uc1} := 5000\text{psi}$

$q_{uc1} = 720\cdot\text{ksf}$

B. Frame/Arch 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_{u1}= 720 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 = 42% (Poor) From LRFD Table 10.4.6.4-1, RQD 25% to 50%; Relative Rating = 8

3. Spacing of joints

Assume broken or highly weathered rock is removed. Breaks 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

Biotite rich break surfaces are open with infilling; Relative Rating = 12

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

Low angle to moderately dipping joints (5-55 degrees); Relative Rating = -7

ADJUSTED RMR

$$RMR := 4 + 8 + 10 + 12 + 7 - 7$$

$$RMR = 34$$

Determine Rock Type for LRFD Table 10.4.6.4.-4

Rock Type - E = Coarse grained polymినerallic igneous & metamorphic crystalline rocks.

Geomechanics Rock Mass Class Determined from Total Rating

From AASHTO LRFD Table 10.4.6.4-3, RMR = 34 is Class No. IV and described as Poor rock.

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

$RMR = 34$

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.

For a disturbed rock mass: m/m_i = exp ((RMR-100)/14)

s = exp ((RMR-100)/6)

m_i = m for intact rock

For Rock Type E for intact rock, m_i = 15 (Ref. # 4, Table 1):

$$m_i := 25$$

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

$$m = 0.224$$

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

$$s = 0.0000167$$

D. Nominal and Factored Bearing Resistance of Bedrock

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

$$C_{f1} := 1.0$$ 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

$$q_{n1} := C_{f1} \cdot \sqrt{s} \cdot q_{uc1} \cdot \left[1 + \sqrt{m \cdot \left(s^{\frac{-1}{2}}\right) + 1}\right]$$

$$q_{n1} = 25 \cdot ksf$$

Factored Bearing Resistances

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

$\phi_{bc} := 0.45$

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

$q_{r1} = 11 \cdot ksf$ 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.

$\phi_{ree} := 0.8$

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

$q_{r1} = 20 \cdot ksf$ 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.

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

$q_{n1} = 25 \cdot ksf$

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 Medium dense to dense Loose	12–20 8–14 4–12	14 10 6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense Medium dense to dense Loose	8–12 4–8 2–6	8 6 3
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense Medium dense to dense Loose	6–12 2–6 1–2	8 4 1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard Medium stiff to stiff Soft	4–8 2–6 1–2	6 3 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 $0.3 f'_c$.

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

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

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

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

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

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

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

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

4.4.8.2.3 Tolerable Movement

Refer to Article 4.4.7.2.3.

4.4.9 Overall Stability

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

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		• Very rough surfaces • Not continuous • No separation • Hard joint wall rock	• Slightly rough surfaces • Separation <0.05 in. • Hard joint wall rock	• Slightly rough surfaces • Separation <0.05 in. • Soft joint wall rock	• Slicken-sided surfaces or • Gouge <0.2 in. thick or • Joints open 0.05–0.2 in. • Continuous joints	• 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:

$$\tau = \left(\cot \phi_i' - \cos \phi_i'\right) m \frac{q_u}{8} \qquad (10.4.6.4\text{-}1)$$

in which:

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

$$h = 1 + \frac{16\left(m\sigma_n' + sq_u\right)}{(3m^2 q_u)}$$

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:

$$c_i = \tau - \sigma_n' \tan \phi_i' \qquad (C10.4.6.4\text{-}1)$$

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.

2012

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

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

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

Undisturbed or interlocking rock masses:

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

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

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) :

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

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.

Wyllie, Foundations on Rock, 2nd ed.

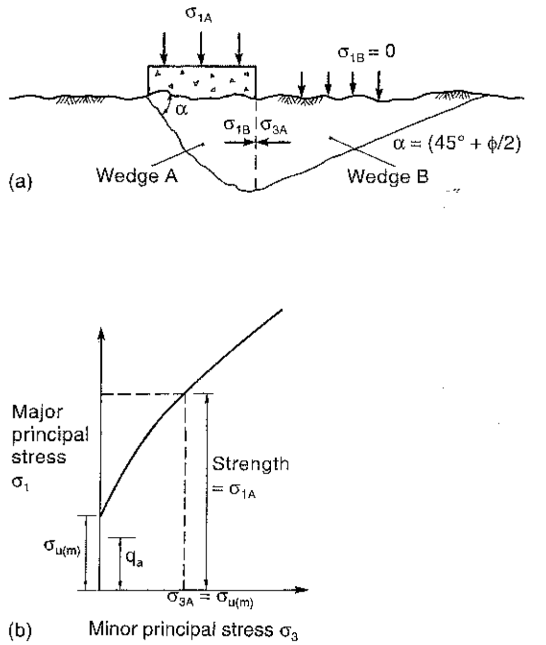


Figure 5.3 Analysis of bearing capacity of fractured rock: (a) active A and passive B wedges in foundation; and (b) curved rock mass strength envelope. Allowable bearing pressure = q_a, strength of bearing rock = σ_{1A}, factor of safety $FS = \sigma_{1A}/q_a$.

$$\sigma_1 = (m\sigma_{u(r)}(s\sigma^2_{u(r)})^{1/2} + s\sigma^2_{u(r)})^{1/2} + (s\sigma^2_{u(r)})^{1/2}$$
$$= s^{1/2}\sigma_{u(r)}[1 + (ms^{-1/2} + 1)^{1/2}] \qquad (5.3)$$

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}:

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

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

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

From Design Freezing Index Map: Litchfield, Maine
DFI = 1500 degree-days.
Case 1 - coarse grained granular fill soils W=10% .

Approximate DFI at project = 1500

$d := 82.1\text{in}$

$d = 6.8 \cdot \text{ft}$

Recommend 6.8 feet for frost protection of foundations constructed on soil

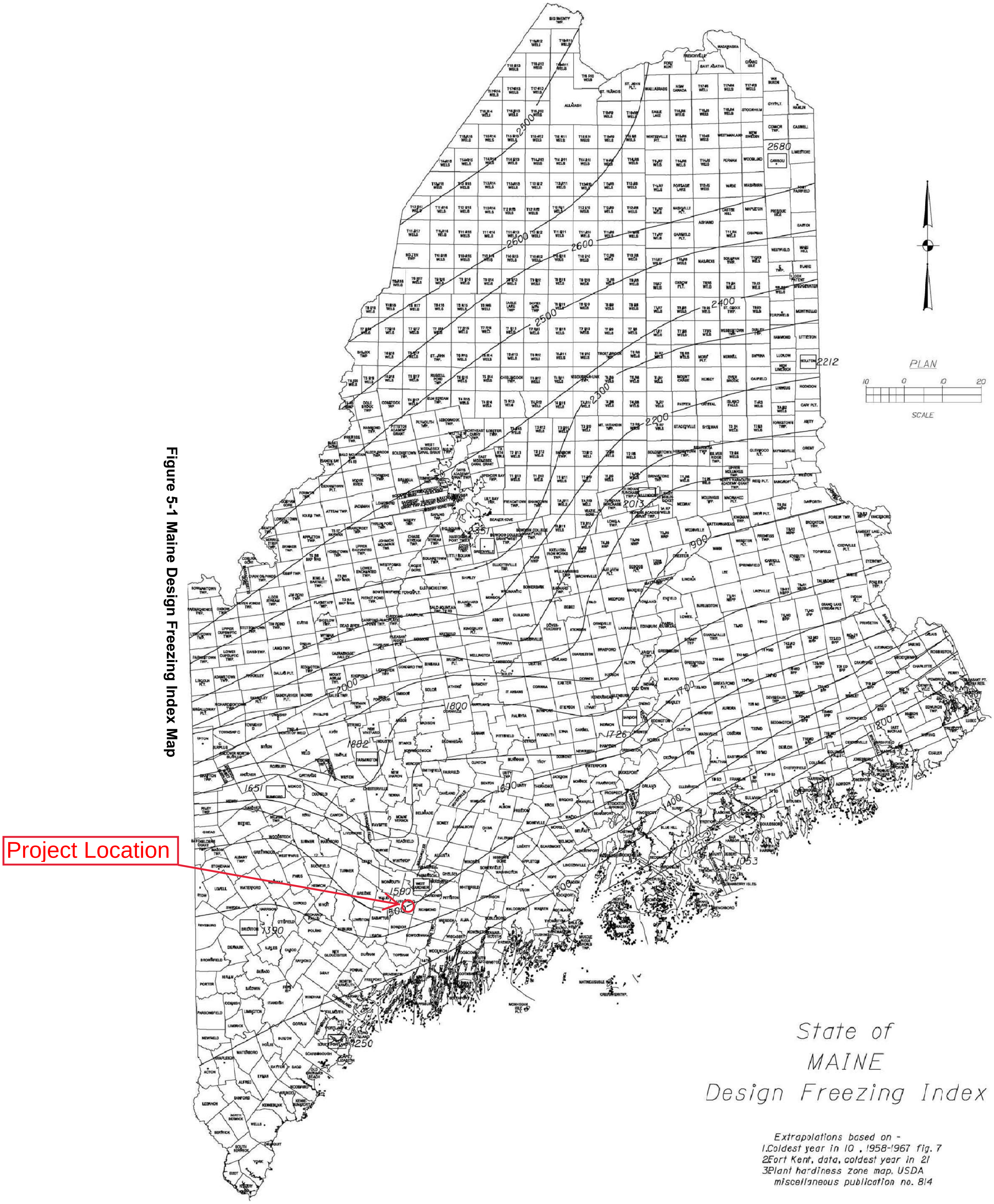


Figure 5-1 Maine Design Freezing Index Map

5.2 General

5.2.1 Frost

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

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

Table 5-1 Depth of Frost Penetration

Design Freezing Index	Frost Penetration (in)					
	Coarse Grained			Fine Grained		
	w=10%	w=20%	w=30%	w=10%	w=20%	w=30%
1000	66.3	55.0	47.5	47.1	40.7	36.9
1100	69.8	57.8	49.8	49.6	42.7	38.7
1200	73.1	60.4	52.0	51.9	44.7	40.5
1300	76.3	63.0	54.3	54.2	46.6	42.2
1400	79.2	65.5	56.4	56.3	48.5	43.9
1500	$\boxed{82.1}$	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

Frost penetration = 82.1"