

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

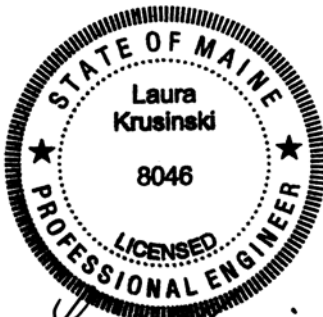
GEOTECHNICAL DESIGN REPORT

For the Replacement of:

**HODGDON MILLS BRIDGE
HODGDON MILLS ROAD OVER SOUTH BRANCH MEDUXNEKEAG RIVER
HODGDON, MAINE**

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Soils Report 2018-32
Bridge No. 3103

September 7, 2018
STP-2263(800)

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1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and geotechnical design recommendations for the replacement of Hodgdon Mills Bridge, which carries Hodgdon Mills Road over the South Branch Meduxnekeag River in Hodgdon, Maine. This report presents the subsurface information obtained at the site during the subsurface investigation, foundation recommendations, and geotechnical design parameters for design of the new bridge substructures.

The existing single span Hodgdon Mills Bridge was constructed in 1933. The structure has a span length of 26 feet and is comprised of concrete tee beams supported by concrete abutments on spread footings founded on bedrock. According to the 1933 construction plans, the existing Hodgdon Mills Bridge Abutment No. 2 incorporates an old stone abutment into the existing abutment backwall. The Hodgdon Mills Dam is approximately 150 feet upstream of the structure. Exposed bedrock outcrops are visible between the dam and bridge as well as in the river channel downstream of the bridge.

The 2017 Maine Department of Transportation (MaineDOT) Bridge Maintenance inspection report assigns the substructure a condition rating of 4 – poor, the deck a 4 – poor, and the superstructure a 4 – poor. The overall poor condition of the structure warrants bridge the replacement. The existing bridge has a Federal Highway Administrations Sufficiency Rating of 42.9.

The replacement Hodgdon Mills Bridge will have a span of 57.5 feet and a 28-foot curb-to-curb width. The replacement superstructure is three precast concrete NEXT F beams supported by full height cantilever-type abutments with spread footings founded on concrete subfootings constructed on bedrock. The horizontal alignment and vertical profile of the proposed structure will closely match the existing. A temporary bridge located upstream will maintain one lane of alternating traffic during construction of the new Hodgdon Mills Bridge.

2.0 GEOLOGIC SETTING

Hodgdon Mills Bridge carries Hodgdon Mills Road over the South Branch Meduxnekeag River in Hodgdon, Maine approximately 1 mile west of the junction with Route 1, as shown on Sheet 1 – Location Map.

The Maine Geologic Survey (MGS) Surficial Geology of the Houlton Quadrangle, Maine, Open-file No. 81-9 (1981) indicates the surficial soils in the vicinity of the bridge project consist of glacial till with nearby contacts to glacial-lake beach and glacial-stream deposits. MGS conducted reconnaissance field mapping of the surficial geology of the Bridgewater, Houlton, Howe Brook, and Smyrna Mills Quadrangles in 1981. The findings are published in Open-file No. 81-7 and indicate that a probable glacial lake or possible marine deposit exists at the project location.

The United States Geological Survey (USGS) Geologic Map of the Houlton Quadrangle, USGS Map No. GQ-920 (1971), indicates the bedrock in the vicinity of the bridge project as the Smyrna Mills Formation. The Smyrna Mills Formation is a predominantly calcareous and noncalcareous, grey-green and green metasiltstone, quartzite, quartz greywacke, slate, and phyllite.

3.0 SUBSURFACE INVESTIGATION

Two test borings explored subsurface conditions at the site. Test boring BB-HMR-101 was located west of the existing structure behind Abutment No. 1. Test boring BB-HMR-102 was located east of the existing structure behind Abutment No. 2. The test boring locations are shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile.

The MaineDOT Drill Crew drilled the borings on May 11, 2016. Details and sampling methods used, field data obtained, and soil conditions encountered are presented in the boring logs Appendix A – Boring Logs and on Sheet 3 – Boring Logs.

The borings were drilled using solid stem auger, cased wash boring, and rock coring techniques. Soil samples were typically obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The 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” in October 2014. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.908 to the raw field N-values. This hammer efficiency factor (0.908) and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring log.

Bedrock was cored in both test borings using an NQ-2” bore barrel and the Rock Quality Designation (RQD) of the bedrock core was calculated. A subsurface inspector, certified by the Northeast Transportation Technician Certification Program (NETTCP), 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 and bedrock samples recovered from the test borings to assist in classification, evaluation of engineering properties, and geologic assessment of the site. Soil laboratory testing consisted of two standard grain size analyses with natural water content. Bedrock laboratory testing consisted of one unconfined compressive test with elastic moduli. The results of laboratory tests are included in Appendix B – Laboratory Test Results. Rock compressive strength, soil moisture content information,

and other soil test results are also shown on the boring logs provided in Appendix A – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings consisted of fill soils 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 discuss the subsurface conditions encountered in detail:

5.1 Fill Soils

The fill unit encountered was approximately 17.5 feet thick at the test boring locations. The fill material generally consisted of:

- Brown, damp, loose, silty sand, some gravel;
- Brown, moist to wet, very loose to loose, gravelly sand, some to little silt;
- Brown, damp, medium dense, sand, some silt, little gravel; and
- Occasional cobbles and boulders.

Corrected SPT N-values in the fill soils ranged from 3 to 30 blows per foot (bpf), indicating the fill soils are very loose to medium dense in consistency. Grain size analyses conducted on samples of the fill unit indicate the soil is classified as A-1-a and A-1-b by the AASHTO Soil Classification System and SM or SW-SM by the Unified Soil Classification System (USCS). The natural water contents of the samples tested ranged from approximately 9 to 37 percent.

5.2 Bedrock

Bedrock was encountered and cored in borings BB-HMR-101 and BB-HMR-102. Table 1 summarizes the approximate top of bedrock elevation at the boring locations and RQD's.

Boring No.	Station	Offset (feet)	Approximate Depth to Bedrock, bgs (feet)	Approximate Top of Bedrock Elevation (feet)	RQD (%)	
BB-HMR-101	12+80.1	9.1 Lt.	17.8	404.0	R1	33
					R2	73
BB-HMR-102	13+24.1	4.7 Rt.	17.6	405.2	R2	0
					R3	20
					R4	0

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

The bedrock recovered at the site is identified as grey-green metasiltstone, moderately hard, fresh to high hydrothermal weathering, irregular breaks, vertical to moderately dipping joints, healed to open, with frequent iron staining. The RQD of the bedrock cored ranged from 0 to 73 percent indicating a Rock Mass Quality of very poor to fair.

One laboratory unconfined compressive strength and secant modulus test was conducted on a bedrock core sample. The test results are included in Appendix B. The testing yielded an unconfined compressive strength of 9.5 ksi and Young's modulus values ranging from 14.9 ksi to 15.6 ksi.

5.3 Groundwater

No groundwater elevations were recorded during drilling of the test borings. Note that water was introduced into the borehole during drilling operations. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, river levels, and construction activities.

6.0 FOUNDATION ALTERNATIVES

The Preliminary Design Report (PDR) dated May 1, 2018 investigated integral abutments on driven H-piles, integral abutments on rock-socketed H-piles, and full height cantilevered abutments on spread footings founded on bedrock.

Integral abutments supported with driven H-piles relies on sufficient overburden to develop fixity, or at least a pinned condition at the pile tip. The subsurface conditions indicate the depth from the bottom of an integral abutment to the top of bedrock is as little as 5 feet. For this reason, conventional driven pile foundation alternatives were dismissed. Construction of rock sockets to provide enough effective pile length to control stresses and provide fixity with a grout-filled bottom, would require significant pre-excavation to remove cobbles and boulders; therefore, this pile alternative was eliminated from consideration. Based on the presence of shallow bedrock across the site, the most effective foundation type is full height cantilever-type abutments on spread footings founded directly on bedrock or on concrete seals constructed on bedrock.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

The following subsections discuss the foundation considerations and recommendations for semi-integral abutments supported on cast-in-place concrete spread footings on concrete seals or subfootings founded on bedrock. The design recommendations in this Section are provided in accordance with AASHTO LRFD Bridge Design Specifications, 8th Edition, with 2018 interim revisions (LRFD).

7.1 Full Height Cantilever Abutments and Wingwalls on Spread Footing Design

Abutments and wingwalls shall be designed for all relevant strength, service, and extreme limit state load combinations specified in AASHTO LRFD Articles 3.4.1 and 11.5.5.

Abutments and wingwalls shall be designed to resist all lateral earth loads, vehicular loads, and deadloads, and withstand temperature and shrinkage effects. The design of spread footings for abutments and wingwalls at the strength limit state shall consider;

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

For sliding analyses, a sliding resistance factor, ϕ_{τ} , of 0.80 shall be applied to the nominal sliding resistance of cast-in-place concrete spread footings or concrete seals constructed on bedrock, assuming the rock subgrade will be prepared in-the-wet. If the rock subgrade is prepared in-the-dry and cleaned with high pressure water and air prior to placing seal concrete, a sliding resistance factor, ϕ_{τ} , of 0.90 may be assumed.

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 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 at level bedrock-to-concrete interfaces.

Anchorage of the footing to the seal is required by the MaineDOT Bridge Design Guide (BDG) Section 5.2.2. The dowels should be drilled and grouted into the seal concrete after dewatering and prior to placing footing concrete. Anchorage of the seal concrete 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 seals and bedrock by depths determined by the designer. A minimum embedment of 2 feet into bedrock is recommended. If bedrock is observed to slope steeper than 4H:1V at the subgrade elevation, the bedrock should be benched to create level steps, or excavated to be completely level.

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 base width.

Extreme limit state design checks for abutments and wingwalls shall include bearing resistance, eccentricity, failure by sliding, and structural failure with respect to extreme event load conditions relating to certain hydraulic events, ice (if warranted by ice history or stream constriction by the abutments) and seismic forces. Resistance factors, ϕ , for the extreme limit state shall be taken as 1.0 with the exception of bearing resistance for which a resistance factor of 0.8 shall be used.

For scour protection of abutment and wingwall spread footings or concrete seals, construct the spread footings or concrete seals directly on bedrock surfaces cleaned of all weathered,

loose, and potentially erodible or scourable rock. With these precautions, strength and extreme limit state designs do not need to consider rock scour due to the design or check floods for scour.

For the service limit state, a resistance factor, ϕ , of 1.0 shall be used to assess spread footing design for settlement, horizontal movement, bearing resistance, sliding, and eccentricity. The overall global stability of foundations is typically investigated at the Service I Load Combination and a resistance factor, ϕ , of 0.65. Shear failure along adversely oriented joint surfaces in the rock mass below the foundation is not anticipated; therefore, a global stability evaluation may be waived.

7.2 Earth Pressures and Surcharge Forces

Calculation of earth pressures acting on semi-integral cantilever-type abutments and wingwalls shall assume active earth pressure over the abutment height using a Rankine theory active earth pressure coefficient, K_a , of 0.31. Additionally, earth pressures acting on semi-integral abutments should include a uniform pressure distribution due to the height of soil behind the superstructure backwall. If the abutments are prevented from movement, the designer should use an at-rest earth pressure coefficient, K_o , of 0.47.

Calculation of earth pressures acting on the superstructure backwall design shall assume full passive earth pressure using a Coulomb passive earth pressure coefficient, K_p , of 6.73 assuming a level backfill. Developing full passive pressure assumes that the ratio of lateral backwall movement to backwall height (y/H) exceeds 0.005. If the calculated displacements are significantly less than that required to develop full passive pressure the designer may consider using the Rankine passive earth pressure coefficient of 3.25. Earth pressure coefficients will need to be recalculated if there is a sloping backfill surface.

A load factor for passive earth pressure is not specified in LRFD. A maximum load factor, γ_{EH} , of 1.50 is recommended to calculate factored passive earth pressures for superstructure backwall reinforcing steel design.

The designer may assume Soil Type 4 (MaineDOT BDG Section 3.6.1) for abutment and wall backfill material soil properties. The backfill properties are as follows; $\phi = 32$ degrees, $\gamma = 125$ pcf.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required on the abutments per Section 3.6.8 of the MaineDOT BDG. The live load surcharge on abutments may be estimated as a uniform horizontal earth pressure due to an equivalent height (h_{eq}) of soil taken from Table 2 below:

Abutment 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

The live load surcharge on return wingwalls with a distance from the wall backface to edge of traffic that is greater than 1 foot may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil of 2.0 feet, per LRFD Table 3.11.6.4-2.

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

7.3 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 11.5.6. The bearing vertical 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 footing 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 at the strength limit state has been calculated to be 12 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. See Appendix C – Calculations for supporting documentation.

Extreme limit state load combinations shall use a factored bearing resistance of 20 ksf. This assumes a bearing resistance factor of 0.8 for gravity and semi-gravity walls in accordance with LRFD C11.5.8. See Appendix C – Calculations for supporting documentation.

In no instance shall the bearing stress exceed the nominal structural resistance of the structural concrete which may be taken as $0.3f'_c$. No footing shall be less than 2 feet wide regardless of the applied bearing pressure or bearing material.

7.4 Bedrock Removal and Bedrock Subgrade Preparation

The overburden encountered in the borings at the proposed footing locations is approximately 17.5 feet thick. It is feasible that spread footings or concrete subfootings can be practically and economically constructed to bear on bedrock at this location. The borings indicate that bedrock with an RQD ranging from approximately 0 to 33 percent will be encountered at the bedrock surface at the footing locations. The Contractor should anticipate the need to clear the bearing area of all weathered, loose, highly fractured and potentially erodible bedrock encountered during construction.

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

If bedrock is observed to slope steeper than 4H:1V at the subgrade elevation, the bedrock should be benched to create level steps or excavated to be completely level. The bedrock surface may be stepped along the centerline of bearing to create a workable bearing surface.

The bottom of footing elevation may vary based on the presence of fractured or weathered bedrock and the variability of the bedrock surface.

The contractor should maintain the abutment and wingwall excavations so that the foundations can be constructed in-the-dry. Where foundations are constructed in-the-dry, the final bearing surface shall be washed with high pressure water and air prior to concrete being placed for the footing. For spread footings are constructed in-the-dry, any irregularities in the existing bedrock surface or irregularities created during the excavation process should be backfilled with unreinforced concrete to the bearing elevation. In-the-dry excavation of highly sloped and loose fractured bedrock material may be done using conventional excavation methods.

For concrete subfootings that are constructed underwater, concrete shall be placed on bedrock cleaned of weathered rock, loose fractured bedrock, boulders, and soil. Cofferdam excavation inspection and reporting shall be in accordance with Standard Specifications Section 511 – Cofferdams. Section 511 requires the contractor to submit a written procedure to the Resident detailing their proposed procedures for sediment/overburden removal, subgrade cleaning, and cofferdam inspection. At a minimum, the inspection shall include bedrock elevation and sediment measurements, taken at a minimum of five locations along

each of the cofferdam perimeter walls and on a grid with twenty evenly spaced internal locations. Bedrock subgrade elevation, sediment, and hardness measurements may be conducted manually with a steel probe consisting of a reinforcing bar or a 24-inch reinforcing bar suspended on a non-stretch tape.

7.5 Settlement

The approximately 17.5-foot-thick fill unit is very loose to medium dense in consistency. The coarse-grained unit undergoes elastic, immediate, compression in response to an increase of vertical overburden pressure. The proposed vertical alignment is nearly the same as the existing vertical alignment and no increase in overburden pressure is anticipated. No approach roadway settlement is anticipated.

The bridge and its foundations will be constructed on bedrock. Any settlement of the bridge foundations placed on a bedrock subgrade that is properly cleaned will be due to elastic compression of the bedrock and will be negligible.

7.6 Frost Protection

Foundations constructed on bedrock require no minimum depth of embedment for frost protection. Foundations placed on the native soils should be designed with an appropriate embedment for frost protection. According to MaineDOT BDG Figure 5-1, Maine Design Freezing Index Map, Hodgdon has a design freezing index (DFI) of approximately 2200 F-degree days. A water content of 20% was used for coarse-grained soils. These components correlate to a frost depth of 6.9 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Houlton, Maine has a DFI from the Modberg database of approximately 2189 F-degree days. A water content of 20% was used. These components correlate to a frost depth of approximately 8.3 feet.

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

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

7.7 Scour and Riprap

For scour protection of the footings, construct the concrete subfootings directly on bedrock surfaces cleaned of soil and all weathered, loose, highly fractured, and potentially erodible rock. The remaining intact bedrock subgrade shall be competent.

Sideslopes and footings supporting the structure should be armored with riprap conforming to MaineDOT Specifications Section 703.26 for Plain and Hand Laid Riprap and Section 703.28 for Heavy 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. When riprap intersects bedrock, the largest riprap should be placed at the toe to buttress the slope above. 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.

7.8 Seismic Design Considerations

The United States Geological Survey Seismic Design CD (Version 2.1) provided with the LRFD Manual, and LRFD Articles 3.10.3.1 and 3.10.6 were used to develop parameters for seismic design. Based on site coordinates, the software provided the recommended AASHTO Response Spectra for a 7 percent probability of exceedance in 75 years. The seismic design parameters are summarized in Table 3:

Parameter	Design Value
Peak Ground Acceleration (PGA)	0.07g
Acceleration Coefficient (A_s)	0.08g
S_{DS} (Period = 0.2 sec)	0.18g
S_{D1} (Period = 1.0 sec)	0.08g
Site Class	C
Seismic Zone	1

Table 3 – Seismic Design Parameters

In conformance with LRFD Articles 4.7.4, seismic analysis is not required for bridges in Seismic Zone 1 or single-span bridges regardless of seismic zone. However, superstructure connections and minimum support length requirements shall be designed per LRFD Articles 3.10.9.2 and 4.7.4.4, respectively.

See Appendix C – Calculations for supporting documentation.

7.9 Construction Considerations

Construction of the abutments, footings, subfootings, and wingwalls will require soil and rock excavation. Temporary earth support systems may be required to permit construction of footings and wingwalls.

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 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 concrete subfootings 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, degraded bedrock, and loose or 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 abutment and wingwall footings.

The bedrock subgrade shall slope no steeper than 4H:1V or the bedrock shall be benched in level steps or excavated to be completely level. 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.

Excavation of bedrock material may be done using conventional excavation methods. The final bedrock surface shall be approved by the Resident prior to placement of the footing concrete.

For concrete subfootings that are constructed underwater, concrete shall be placed on bedrock cleaned of weathered rock, loose fractured bedrock, boulders, and soil. Cofferdam excavation inspection and reporting shall be in accordance with Standard Specifications Section 511 – Cofferdams. Section 511 requires the contractor to submit a written procedure to the Resident detailing their proposed procedures for sediment/overburden removal, subgrade cleaning, and cofferdam inspection. At a minimum, the inspection shall include bedrock elevation and sediment measurements, taken at a minimum of five locations along each of the cofferdam perimeter walls and on a grid with twenty evenly spaced internal locations. Bedrock subgrade elevation, sediment, and hardness measurements may be conducted manually with a steel probe consisting of a reinforcing bar or a 24-inch reinforcing bar suspended on a non-stretch tape.

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. 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 use by the MaineDOT Bridge Program for the specific application to the proposed replacement of Hodgdon Mills Bridge in Hodgdon, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied.

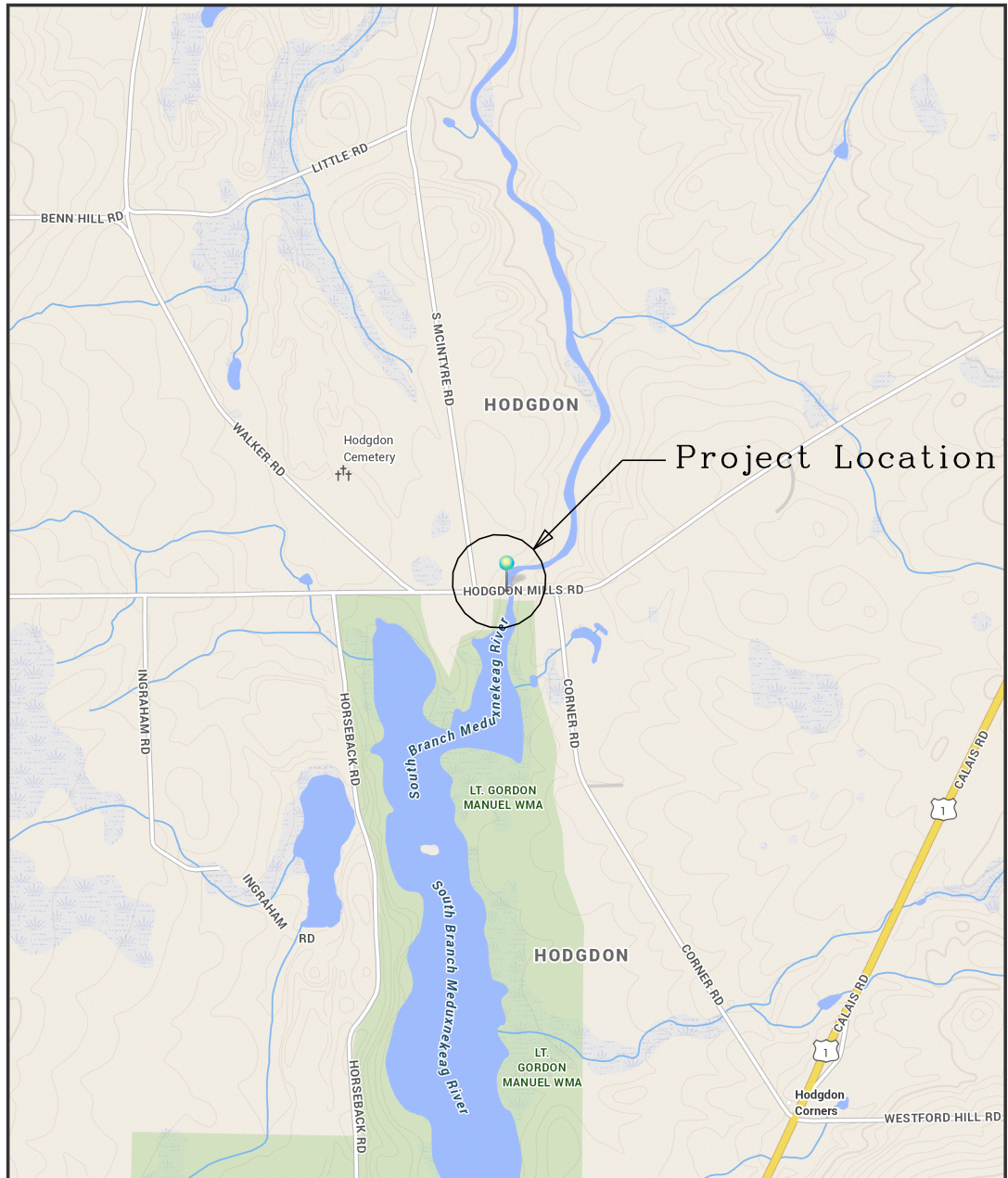
In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes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

Sheets



HODGDON, MAINE

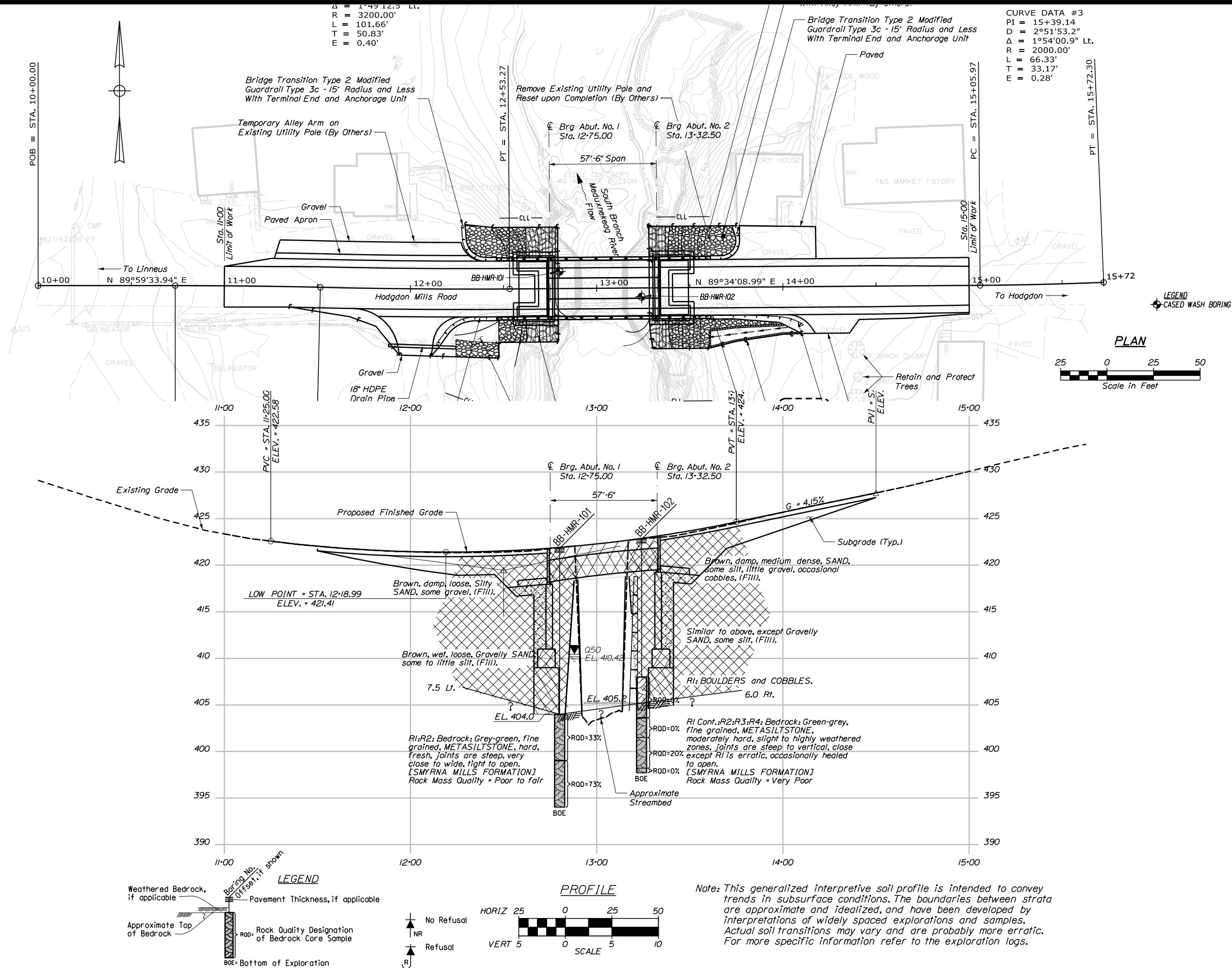


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0.25 Miles
1 inch = 0.28 miles

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SHEET NUMBER 1 OF 3	HODGDON MILLS BRIDGE SOUTH BRANCH MEDUXNEKEAG RIVER HODGDON AROOSTOOK COUNTY	STATE OF MAINE DEPARTMENT OF TRANSPORTATION	
	LOCATION MAP	STP-2263(800)	
		WIN 022638.00 BRIDGE NO. 3103 BRIDGE PLANS	



PROJ. MANAGER	DATE	BY	DATE	SIGNATURE
M. WIGHT		D. SHAW	SEP 2018	
CHECKED-REVIEWED		T. WHITE		
DESIGN DETAILING				
REVISIONS 1				
REVISIONS 2				
REVISIONS 3				
REVISIONS 4				
FIELD CHANGES				

HODGDON MILLS BRIDGE
SOUTH BRANCH MEDUXNEKEAG RIVER
HODGDON AROOSTOOK COUNTY
BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

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

Project: Hodgdon Mills Bridge carries
Hodgdon Mills Rd over S. Branch
Location: Hodgdon, Maine

Boring No.: BB-HMR-101
WIN: 22638.00

Driller: MainDOT
Operator: Daggett/Austin
Logged By: B. Wilder
Date Start/Finish: 5/11/2016: 11:30-15:00
Boring Location: 12+80.1, 9.1 ft Lt.

Elevation (ft.): 421.8
Datum: NAVD88
Rig Type: CME 45C
Drilling Method: Cased Wash Boring
Casing ID/OD: NW-3"

Auger ID/OD: 5" Solid Stem
Sampler: Standard Split Spoon
Hammer Wt./Fall: 140#/30"
Core Barrel: NQ-2"
Water Level#: None Observed

Hammer Efficiency Factor: 0.908
Definitions
R = Rock Core Sample
S = Split Spoon Sample
MS = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test
MW = Unsuccessful Field Vane Shear Test Attempt

SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WRC = Weight of 140lb. Hammer
N = Pocket Penetrometer
WRC = Weight of Rods or Casing
WOP = Weight of One Person

Automatic ☒ Hydraulic ☐ Rope & Cathead ☐
Su = Peak/Retained Field Vane Undrained Shear Strength (psf)
Su(1g) = Lab Vane Undrained Shear Strength (psf)
qu = Unconfined Compressive Strength (ksf)
N = Uncorrected Raw Field SPT N-value
Ncor = Hammer Efficiency Factor x Rig Specific Annual Calibration Value
Nsp = SPT Uncorrected Corrected for Hammer Efficiency
Nsp = Hammer Efficiency Factor/60%Uncorrected

Tv = Pocket Torque 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										Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
Depth (ft.)	Sample No.	Pen./Rec. (in)	Sample Depth (ft.)	Blows 1/6 in. Shear Strength (psf) or RQD (%)	N-uncorrected	Nsp	Casing	Blows					
									SSA	421.4		5" HMA.	0.4
5	10	24/8	5.00 - 7.00	3/2/3/4	5	8	6					Brown, damp, loose, Silty SAND, some gravel, (Fill).	
10												NW Casing sunk to 12.5 ft bgs while washing ahead.	
	20	24/4	12.50 - 14.50	1/1/1/1	2	3	WOC/4					Brown, wet, very loose, Gravely SAND, some silt.	
15	30	24/10	15.00 - 17.00	2/2/2/8	4	6	4					Brown, wet, loose, Gravely SAND, little silt.	GW270701 A-1-b, SW-SM WC=36.6%
	R1	60/60	17.80 - 22.80	RQD = 33%								080 blows for 0.8 ft.	
20										404.0		Top of Bedrock at Elev. 404.0 ft. R1: Bedrock: Grey-green, fine grained, METASILTSTONE, moderately hard, fresh, irregular breaks, joints are undulating to rough, steep to moderately dipping, very close to wide, tight to open. [SMYRNA MILLS FORMATION] Rock Mass Quality = Poor. No Core Times taken. 100% Recovery	17.6
25	R2	60/60	22.80 - 27.80	RQD = 73%								R2: Bedrock: Similar to above except joints are very close to moderately close. No Core Times taken. Rock Mass Quality = Fair. 100% Recovery	22.9'-23.4' Sp= 9.5 ksf
30													
35													
40													
45													
50													

Remarks:

Stratification lines represent approximate boundaries between soil type 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-HMR-101

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

Project: Hodgdon Mills Bridge carries
Hodgdon Mills Rd over S. Branch
Location: Hodgdon, Maine

Boring No.: BB-HMR-102
WIN: 22638.00

Driller: MainDOT
Operator: Daggett/Austin
Logged By: B. Wilder
Date Start/Finish: 5/11/2016: 08:00-11:00
Boring Location: 13+24.1, 4.7 ft Rt.

Elevation (ft.): 422.8
Datum: NAVD88
Rig Type: CME 45C
Drilling Method: Cased Wash Boring
Casing ID/OD: NW-3"

Auger ID/OD: 5" Solid Stem
Sampler: Standard Split Spoon
Hammer Wt./Fall: 140#/30"
Core Barrel: NQ-2"
Water Level#: None Observed

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Sample Information										Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
Depth (ft.)	Sample No.	Pen./Rec. (in)	Sample Depth (ft.)	Blows 1/6 in. Shear Strength (psf) or RQD (%)	N-uncorrected	Nsp	Casing	Blows					
									SSA	422.4		5" HMA.	0.4
5												Brown, damp, medium dense, SAND, some silt, little gravel, occasional cobble, (Fill).	GW270702 A-1-b, SW WC=8.5%
	10	24/20	2.00 - 4.00	6/12/8/10	20	30							
10	20	24/8	5.00 - 7.00	11/13/4/6	17	26						Brown, moist, medium dense, Gravely SAND, some silt.	
15													
	30	24/4	10.00 - 12.00	9/9/1/3	16	24	3					Similar to above.	
20													
	R1	52.8/45	14.80 - 19.20	RQD = 0%								0168 blows for 0.8 ft. R1: Boulder and COBBLES. R1: Core Times (min:sec) 14.8-15.8 ft (2:12) 15.8-16.8 ft (1:04) 16.8-17.8 ft (1:16)	
25													
	R2	25.2/ 25.2	19.20 - 21.30	RQD = 0%								Top of Bedrock at Elev. 405.2 ft. R1: Bedrock: Grey-green, fine grained, METASILTSTONE, moderately hard, highly weathered zones, joints are steep, erodic, open, frequent iron staining. [SMYRNA MILLS FORMATION] Rock Mass Quality = Very Poor. R1: Core Times Cont. 17.8-18.8 ft (2:30) 18.8-19.2 ft (2:40) Core Blocked 85% Recovery	17.6
	R3	39.6/ 39.6	21.30 - 24.60	RQD = 20%								R2: Bedrock: Grey-green, fine grained, METASILTSTONE, moderately hard, slight hydrothermal weathering, joints are steep to vertical, close, occasionally healed to open. Rock Mass Quality = Very Poor. R2: Core Times (min:sec) 19.2-20.2 ft (1:57) 20.2-21.2 ft (2:34) 21.2-21.3 ft (2:00) Core Blocked 100% Recovery	
30													
35													
40													
45													
50													

Remarks:

Stratification lines represent approximate boundaries between soil type transitions may be gradual.
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Page 1 of 1
Boring No.: BB-HMR-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
STP-2263(800)

BRIDGE NO. 3103
WIN
022638.00
BRIDGE PLANS

HODGDON MILLS BRIDGE
SOUTH BRANCH MEDUXNEKEAG RIVER
HODGDON
AROOSTOOK COUNTY

REVISIONS
1
2
3
4
FIELD CHANGES

DESIGN-DETAILED
CHECKED-REVIEWED
DESIGNS-DETAILED
DESIGNS-DETAILED
DESIGNS-DETAILED

BY
M. WIGHT
D. SHAW
T. WHITE
B. SLAVEN

DATE
OCT 2018

SIGNATURE
P.E. NUMBER
DATE

SHEET NUMBER
3
OF 3

BORING LOGS

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM					MODIFIED BURMISTER SYSTEM			
MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES				
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.	Descriptive Term		Portion of Total (%)	
		(little or no fines)	GP	Poorly-graded gravels, gravel sand mixtures, little or no fines.	trace	0 - 10		
					little	11 - 20		
					some	21 - 35		
					adjective (e.g. sandy, clayey)	36 - 50		
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.	TERMS DESCRIBING DENSITY/CONSISTENCY			
			GC	Clayey gravels, gravel-sand-clay mixtures.				
		CLEAN SANDS	SW	Well-graded sands, gravelly sands, little or no fines	<u>Coarse-grained soils</u> (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).			
		(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	<u>Density of Cohesionless Soils</u>		
	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.	Standard Penetration Resistance		N-Value (blows per foot)	
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	Very loose	0 - 4		
					Loose	5 - 10		
				Medium Dense	11 - 30			
				Dense	31 - 50			
	SILTS AND CLAYS (liquid limit greater than 50)	OL	Organic silts and organic silty clays of low plasticity.	Very Dense	> 50			
		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	<u>Fine-grained soils</u> (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.				
		CH	Inorganic clays of high plasticity, fat clays.					
			OH	Organic clays of medium to high plasticity, organic silts.	<u>Approximate Undrained Shear Strength (psf)</u>			
HIGHLY ORGANIC SOILS	Pt	Peat and other highly organic soils.	<u>Consistency of Cohesive soils</u>		<u>SPT N-Value (blows per foot)</u>		<u>Field Guidelines</u>	
				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
<u>Desired Soil Observations (in this order, if applicable):</u> 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					<u>Rock Quality Designation (RQD):</u> RQD (%) = $\frac{\text{sum of the lengths of intact pieces of core} * > 4 \text{ inches}}{\text{length of core advance}}$ *Minimum NQ rock core (1.88 in. OD of core)			
					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		
<u>Desired Rock Observations (in this order, if applicable):</u> 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))					<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth			
Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information								

Maine Department of Transportation <u>Soil/Rock Exploration Log</u> <u>US CUSTOMARY UNITS</u>				Project: Hodgdon Mills Bridge carries Hodgdon Mills Rd over S. Branch Meduxnekeag Location: Hodgdon, Maine				Boring No.: BB-HMR-101 WIN: 22638.00																							
Driller: MaineDOT				Elevation (ft.): 421.8				Auger ID/OD: 5" Solid Stem																							
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Boring Location: 12+80.1, 9.1 ft Lt.				Casing ID/OD: NW-3"				Water Level*: None Observed																							
Hammer Efficiency Factor: 0.908				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																							
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rowspan="12"></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>30</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>35</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>40</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>45</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Hodgdon Mills Bridge carries Hodgdon Mills Rd over S. Branch Meduxnekeag Location: Hodgdon, Maine				Boring No.: BB-HMR-102 WIN: 22638.00			
Driller: MaineDOT				Elevation (ft.): 422.8				Auger ID/OD: 5" Solid Stem			
Operator: Daggett/Austin				Datum: NAVD88				Sampler: Standard Split Spoon			
Logged By: B. Wilder				Rig Type: CME 45C				Hammer Wt./Fall: 140#/30"			
Date Start/Finish: 5/11/2016; 08:00-11:00				Drilling Method: Cased Wash Boring				Core Barrel: NQ-2"			
Boring Location: 13+24.1, 4.7 ft Rt.				Casing ID/OD: NW-3"				Water Level*: None Observed			
Hammer Efficiency Factor: 0.908				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐							
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test											
Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
0							SSA	422.4		5" HMA.	0.4
	1D	24/20	2.00 - 4.00	6/12/8/10	20	30				Brown, damp, medium dense, SAND, some silt, little gravel, occasional cobble, (Fill).	G#270702 A-1-b, SM WC=8.5%
5	2D	24/8	5.00 - 7.00	11/13/4/6	17	26				Brown, moist, medium dense, Gravelly SAND, some silt.	
10	3D	24/4	10.00 - 12.00	9/9/7/3	16	24	3			Similar to above.	
							15				
							29				
15	R1	52.8/45	14.80 - 19.20	RQD = 0%			a168 NQ-2			a168 blows for 0.8 ft. R1:BOULDER and COBBLES. R1:Core Times (min:sec) 14.8-15.8 ft (2:12) 15.8-16.8 ft (1:04) 16.8-17.8 ft (1:16)	
								405.2		Top of Bedrock at Elev. 405.2 ft. R1:Bedrock: Grey-green, fine grained, METASILTSTONE, moderately hard, highly weathered zones, joints are steep, erratic, open, frequent iron staining. [SMYRNA MILLS FORMATION] Rock Mass Quality = Very Poor. R1:Core Times Cont. 17.8-18.8 ft (2:30) 18.8-19.2 ft (2:40) Core Blocked 85% Recovery	
20	R2	25.2/25.2	19.20 - 21.30	RQD = 0%						R2:Bedrock: Grey-green, fine grained, METASILTSTONE, moderately hard, slight hydrothermal weathering, joints are steep to vertical, close, occasionally healed to open.	
											
	R3	39.6/39.6	21.30 - 24.60	RQD = 20%							
											
25	R4	6/6	24.60 - 25.10	RQD = 0%							
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 2 Boring No.: BB-HMR-102	

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Hodgdon Mills Bridge carries Hodgdon Mills Rd over S. Branch Meduxnekeag Location: Hodgdon, Maine				Boring No.: BB-HMR-102 WIN: 22638.00			
Driller: MaineDOT			Elevation (ft.) 422.8			Auger ID/OD: 5" Solid Stem					
Operator: Daggett/Austin			Datum: NAVD88			Sampler: Standard Split Spoon					
Logged By: B. Wilder			Rig Type: CME 45C			Hammer Wt./Fall: 140#/30"					
Date Start/Finish: 5/11/2016; 08:00-11:00			Drilling Method: Cased Wash Boring			Core Barrel: NQ-2"					
Boring Location: 13+24.1, 4.7 ft Rt.			Casing ID/OD: NW-3"			Water Level*: None Observed					
Hammer Efficiency Factor: 0.908			Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐								
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140 lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S_{u(lab)} = Lab Vane Undrained Shear Strength (psf) q_u = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T_v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test											
Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
25								397.7		Rock Mass Quality = Very Poor. R2:Core Times (min:sec) 19.2-20.2 ft (1:57) 20.2-21.2 ft (2:34) 21.2-21.3 ft (2:00) Core Blocked 100% Recovery R3:Bedrock: Similar to R2. Rock Mass Quality = Very Poor. R3:Core Times (min:sec) 21.3-22.3 ft (1:40) 22.3-23.3 ft (2:52) 23.3-24.3 ft (2:44) 24.3-24.6 ft (2:00) Core Blocked 100% Recovery R4:Bedrock: Similar to R2. Rock Mass Quality = Very Poor. R4:Core Times (min:sec) 24.6-25.1 ft (3:00) Core Blocked 100% Recovery	
30											
35											
40											
45											
50											
Remarks:											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 2 of 2	
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										Boring No.: BB-HMR-102	

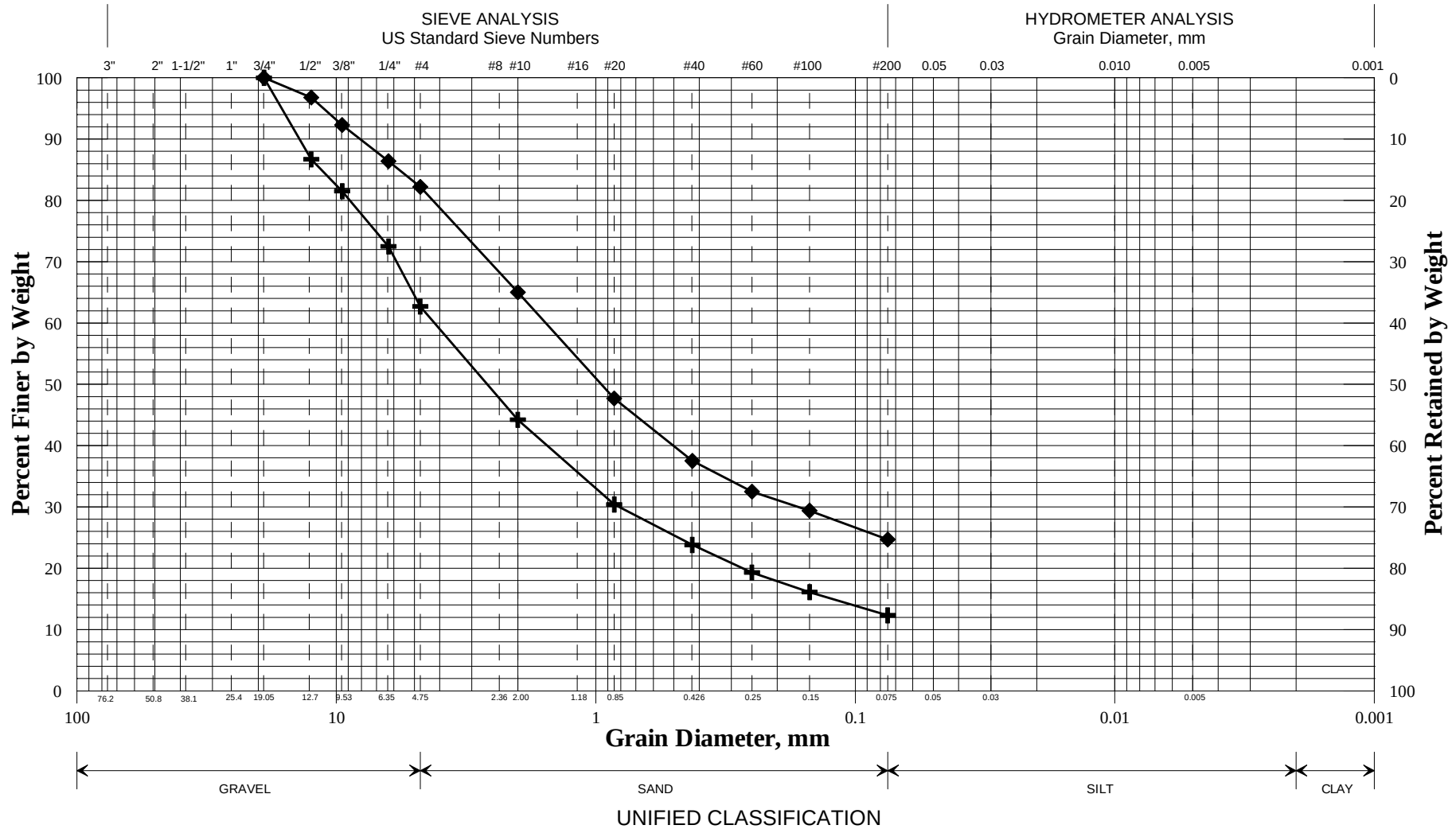
Appendix B

Laboratory Test Results

Work Number: 22638.00

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-HMR-101/3D	12+80.1	9.1 LT	15.0-17.0	Gravelly SAND, little silt.	36.6			
◆	BB-HMR-102/1D	13+24.1	4.7 RT	2.0-4.0	SAND, some silt, little gravel.	8.5			
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WIN
022638.00
Town
Hodgdon
Reported by/Date
WHITE, TERRY A 5/31/2016



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1/16/2018

GTX-307459

**Hodgdon Mills Br, Hodgdon Mills Rd
over S. Br. Meduxnekeag**

Hodgdon, ME

Client Project No.: 22638.00

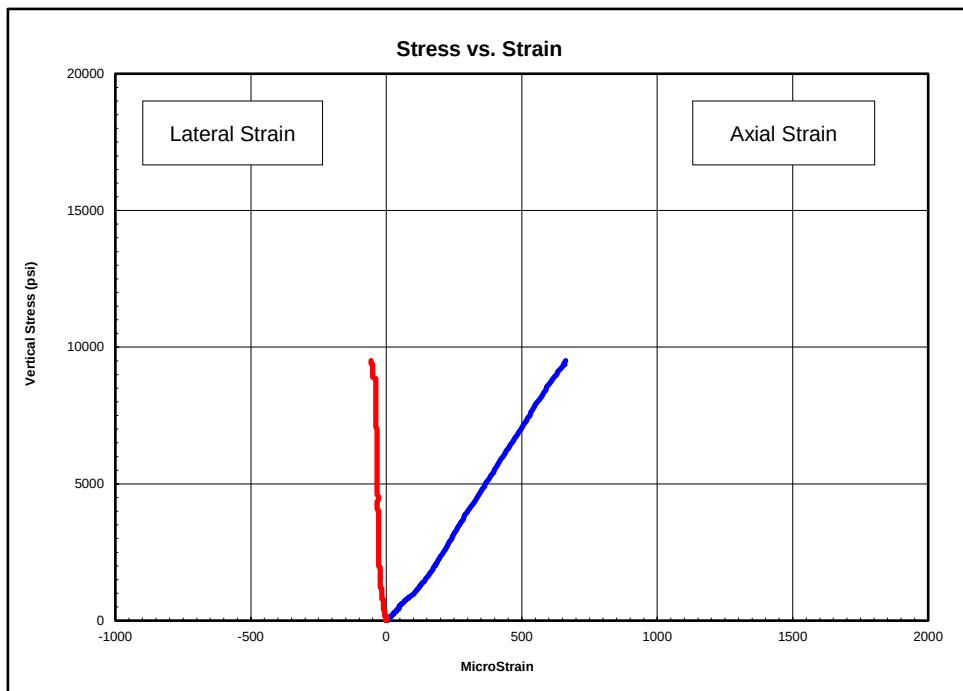
Prepared for:

Maine Department of Transportation



Client:	Maine Department of Transportation
Project Name:	Hodgdon Mills Br, Hodgdon Mills Rd over S. Br. Meduxnekeag
Project Location:	Hodgdon, ME
GTX #:	307459
Test Date:	1/5/2018
Tested By:	rlc
Checked By:	jsc
Boring ID:	BB-HMR-101
Sample ID:	R2
Depth, ft:	23-23.35
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure Specimen area calculated from chipped end

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



Peak Compressive Stress: 9,508 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1000-3500	14,900,000	0.10
3500-6000	15,400,000	0.08
6000-8600	15,600,000	0.07

Notes:

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

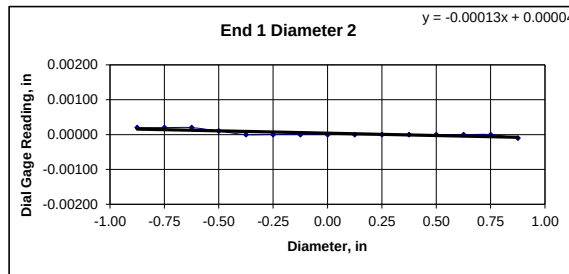
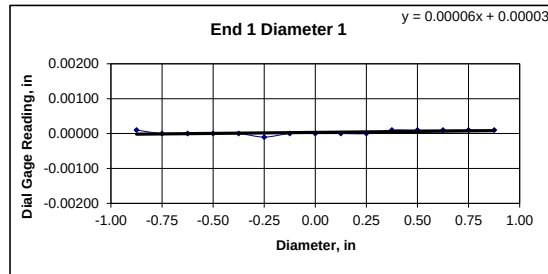


Client:	Maine Department of Transportation	Test Date:	1/3/2018
Project Name:	Hodgdon Mills Br, Hodgdon Mills Rd over S. Br.	Tested By:	rlc
Project Location:	Meduxnekeag Hodgdon, ME	Checked By:	jsc
GTX #:	307459		
Boring ID:	BB-HMR-101		
Sample ID:	R2		
Depth:	23-23.35 ft		
Visual Description:	See photographs		

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

BULK DENSITY				DEVIATION FROM STRAIGHTNESS (Procedure S1)	
	1	2	Average	Maximum gap between side of core and reference surface plate: Is the maximum gap $\leq$ 0.02 in.? NO	
Specimen Length, in:	4.07	4.07	4.07	Maximum difference must be < 0.020 in.	
Specimen Diameter, in:	1.97	1.97	1.97	Straightness Tolerance Met? NO	
Specimen Mass, g:	560.46				
Bulk Density, lb/ft ³ :	172				
Length to Diameter Ratio:	2.1				
		Minimum Diameter Tolerance Met?	YES		
		Length to Diameter Ratio Tolerance Met?	YES		

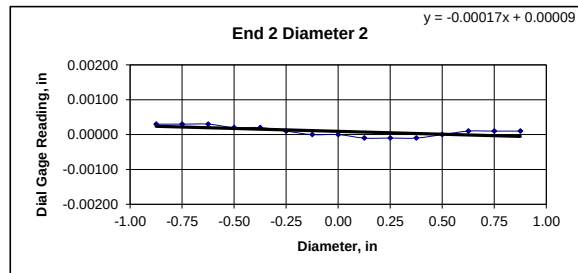
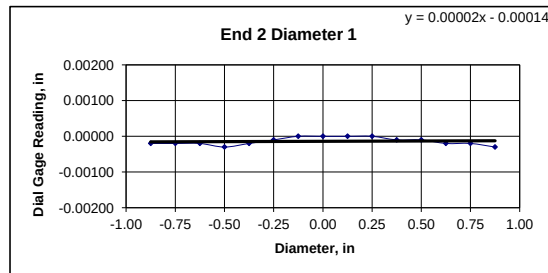
END FLATNESS AND PARALLELISM (Procedure FP1)														
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750
Diameter 1, in	0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010
Diameter 2, in (rotated 90°)	0.00020	0.00020	0.00020	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010
Difference between max and min readings, in:														
0° = 0.00020 90° = 0.00030														
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750
Diameter 1, in	-0.00020	-0.00020	-0.00020	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00020	-0.00030
Diameter 2, in (rotated 90°)	0.00030	0.00030	0.00030	0.00020	0.00020	0.00010	0.00000	0.00000	-0.00010	-0.00010	-0.00010	0.00000	0.00010	0.00010
Difference between max and min readings, in:														
0° = 0.0003 90° = 0.0004														
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00020														
Flatness Tolerance Met? YES														



DIAMETER 1

End 1:		
Slope of Best Fit Line	0.00006	
Angle of Best Fit Line:	0.00344	
End 2:		
Slope of Best Fit Line	0.00002	
Angle of Best Fit Line:	0.00115	
Maximum Angular Difference:	0.00229	

Parallelism Tolerance Met? YES
Spherically Seated



DIAMETER 2

End 1:		
Slope of Best Fit Line	0.00013	
Angle of Best Fit Line:	0.00745	
End 2:		
Slope of Best Fit Line	0.00017	
Angle of Best Fit Line:	0.00974	
Maximum Angular Difference:	0.00229	

Parallelism Tolerance Met? YES
Spherically Seated

PERPENDICULARITY (Procedure P1)						(Calculated from End Flatness and Parallelism measurements above)	
END 1		Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?	Maximum angle of departure must be $\leq$ 0.25°
Diameter 1, in		0.00020	1.970	0.00010	0.006	YES	
Diameter 2, in (rotated 90°)		0.00030	1.970	0.00015	0.009	YES	Perpendicularity Tolerance Met? YES
END 2							
Diameter 1, in		0.00030	1.970	0.00015	0.009	YES	
Diameter 2, in (rotated 90°)		0.00040	1.970	0.00020	0.012	YES	

Client:	Maine Department of Transportation
Project Name:	Hodgdon Mills Br, Hodgdon Mills Rd over S. Br.
Project Location:	Meduxnekeag Hodgdon, ME
GTX #:	307459
Test Date:	1/5/2018
Tested By:	rlc
Checked By:	jsc
Boring ID:	BB-HMR-101
Sample ID:	R2
Depth, ft:	23-23.35



After cutting and grinding



After break

Appendix C

Calculations

Earth Pressure

Backfill engineering strength parameters

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

Unit weight	$\gamma_1 := 125 \cdot \text{pcf}$
Internal friction angle	$\phi' := 32 \cdot \text{deg}$
Cohesion	$c_1 := 0 \cdot \text{psf}$

Semi-Integral Abutment Backwall - Passive Earth Pressure - Coulomb Theory

α = Angle of fill slope to the horizontal	$\alpha := 0 \cdot \text{deg}$
ϕ_1 = Angle of internal friction	$\phi' = 32 \cdot \text{deg}$
β = Angle of back face of wall to the horizontal	$\beta := 90 \cdot \text{deg}$

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

For formed concrete against clean sand, silty sand-gravel mixture use $\delta = 17 - 22$, per LRFD Table 3.11.5.3-1

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

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

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

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

$$K_{p_coulomb} = 6.73$$

Semi-Integral Abutment Backwall - Passive Earth Pressure - Rankine Theory

Use Rankine only if the ratio of wall height to wall movement is significantly less than .005 and the fill slope is horizontal to the top of the wall. Bowles does not recommend use of Rankine method for K_p when $\alpha > 0$ and its use in this situation may be unconservative due Rankine Theory failing to account for wall friction.

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

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

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

$$K_{p_rank} = 3.25$$

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

At-Rest Earth Pressure

For walls with tops restrained from movement and for surcharge loading, use at-rest earth pressure.

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

$$K_o = 0.47$$

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

Abutment Breastwall and Wingwalls Active Earth Pressure Coefficient

If δ is taken as 0 and the slope of the backslope is horizontal, there is no difference in the active earth pressure coefficient when using either Rankine or Coulomb.

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

$$K_{ar} = 0.31$$

Das, Principles of
Geotechnical Engineering
7th Ed. p 434 Eq. 13.19

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

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

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

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

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

where

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

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

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

7.13

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

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

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

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

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

Again, at $z = 3$ m,

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

Hence,

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

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

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

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

7.11

Rankine Passive Earth Pressure: Vertical Backface and Inclined Backfill

Granular Soil

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

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

and the passive force is

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

where

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

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

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

Again, at $z = 3$ m,

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

Hence,

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

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

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

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

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

and the passive force is

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

where

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

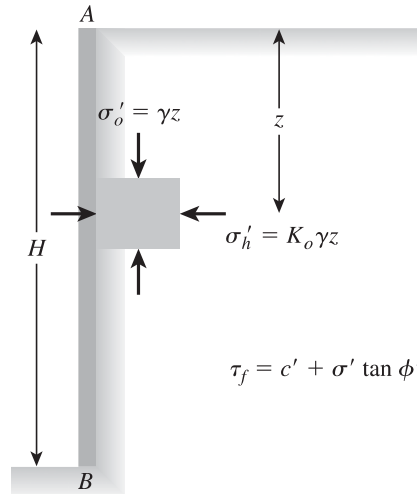


Figure 13.3
Earth pressure at rest

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

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

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

So,

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

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

$$K_o = 1 - \sin \phi' \quad (13.5)$$

where $\phi' =$ 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

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

where $\gamma_d =$ 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)

Bearing Resistance

Analysis

Calculation of nominal and factored bearing resistance on rock for Strength and Extreme Limit State Analysis.

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, with 2018 Interims
2. AASHTO Standard Specifications for Highway Bridges, 17th Ed. 2002
3. Wylie, Duncan C, "Foundations on Rock", Second Edition, 2009.
4. "The Hoek-Brown Failure Criterion - A 1988 Update", E. Hoek and E.T. Brown

A. Design Bedrock Properties

Green-grey, fine grained, METASILTSTONE, moderately hard, fresh to slight geothermal weathering, occasional breaks along steep relic bedding are tight, joints are low angle to steep, very close to close, open, occasional iron staining. **RQD = 33%. Rock Mass Quality = Poor.**

Compressive Strength

UCT conducted on sample BB-HMR-101;R2 recovered between 22.9 - 23.4 feet below roadway surface.

$q_{uc} = 9500$ psi.

Use $q_u = 9,500$ psi or 1365 ksf

$q_{uc} := 9500 \cdot \text{psi}$

B. 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

$q_{u1} = 9,500$ psi or 1365 ksf

From LRFD Table 10.4.6.4-1 for Uniaxial compressive strength = 1365 ksf: **Relative Rating = 7**

2. Drill Core Quality

Assume very poor rock is cleaned away.

Bedrock RQD = 33% (Poor) From LRFD Table 10.4.6.4-1, RQD between 25 and 50%; **Relative Rating = 8**

3. Spacing of joints

Assume broken or highly weathered rock is removed. Breaks of intact bedrock are close (2 in - 12 in).

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

4. Condition of joints

Break surfaces are slightly slick, open; **Relative Rating = 6**

5. Groundwater conditions

General conditions moist; **Relative Rating = 7**

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

Breaks along bedding are steep, joints low angle to steep and cross cut use **Relative Rating = -7**

ADJUSTED RMR

$$\text{RMR} := 7 + 8 + 10 + 6 + 7 - 7$$

$$\text{RMR} = 31$$

Determine Rock Type for LRFD Table 10.4.6.4-4

Rock Type - B = Lithified Argillaceous Rocks
mudstone, siltstone, shale, and slate (normal to cleavage)

Geomechanics Rock Mass Class Determined from Total Rating

From AASHTO LRFD Table 10.4.6.4-3, RMR = 31 is Class No. IV and described as Poor rock.

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

$$\text{RMR} = 31$$

Direct calculation of m and s is required, Reference 4 (Hoek and Brown, 1988), Equations 18 and 19 and Table 1. Assume isotropic behavior caused by the number and inconsistency of closely spaced discontinuity sets where none is significantly weaker than the other.

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

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

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

For Rock Type B, for intact rock, RMR=100, $m_i = 10$ (Ref. # 4, Table 1) and $s = 1$

$$m_i := 10$$

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

$$m = 0.072$$

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

$$s = 0.0000101$$

D. Nominal and Factored Bearing Resistance of Bedrock

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

$$C_{fl} := 1.0$$

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

Nominal Bearing Resistance (Wyllie)

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

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

$$q_{n1} = 26 \cdot \text{ksf}$$

Factored Bearing Resistances

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

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

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

$$q_{r1} = 12 \cdot \text{ksf}$$

Strength Limit State

Recommendation: Use 12 ksf for Factored Bearing Resistance at the Strength Limit State

Factored Bearing Resistances

Use a bearing resistance factor of 0.80 LRFD 11.5.8 consistent with the design objective of no collapse.

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

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

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

Extreme Limit State

Recommendation: Use 20 ksf for Factored Bearing Resistance at the Extreme Limit State

Verify Nominal Bearing Resistance per Carter and Kulhawy (1988)

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

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

$$q_{n1} = 26 \cdot \text{ksf}$$

Analysis

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

Method 1

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

Method 2

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

Abutment No. 1 and Abutment No. 2

Bearing Material:	Weathered or broken bedrock of any kind, except shale.
Consistency in Place:	Medium hard rock
Allowable Bearing Pressure	Range: 16-24 ksf
AASHTO Recommended Value	20 ksf

Resistance Factor for Service Limit State

$$\phi_r := 1.0$$

$$q_{1\text{nominal}} := 20 \cdot \text{ksf}$$

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

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

$$q_{\text{factored}} = 20 \cdot \text{ksf}$$

Service Limit State

Recommendation: Use 20 ksf for Factored Bearing Resistance at the Service Limit State

2012

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:

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

in which:

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

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

where:

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

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

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

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

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	Rock Type				
		A = Carbonate rocks with well developed crystal cleavage— <i>dolomite, limestone and marble</i> B = Lithified argillaceous rocks— <i>mudstone, siltstone, shale and slate (normal to cleavage)</i> C = Arenaceous rocks with strong crystals and poorly developed crystal cleavage— <i>sandstone and quartzite</i> D = Fine grained polyminerallic igneous crystalline rocks— <i>andesite, dolerite, diabase and rhyolite</i> E = Coarse grained polyminerallic igneous & metamorphic crystalline rocks— <i>amphibolite, gabbro gneiss, granite, norite, quartz-diorite</i>				
		A	B	C	D	E
INTACT ROCK SAMPLES Laboratory size specimens free from discontinuities. CSIR rating: <i>RM</i> <i>R</i> = 100	<i>m</i> <i>s</i>	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: <i>RM</i> <i>R</i> = 85	<i>m</i> <i>s</i>	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: <i>RM</i> <i>R</i> = 65	<i>m</i> <i>s</i>	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: <i>RM</i> <i>R</i> = 44	<i>m</i> <i>s</i>	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: <i>RM</i> <i>R</i> = 23	<i>m</i> <i>s</i>	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: <i>RM</i> <i>R</i> = 3	<i>m</i> <i>s</i>	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 :

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

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

Undisturbed or interlocking rock masses:

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

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

where

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

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

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

Limitations on using failure criterion

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

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

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

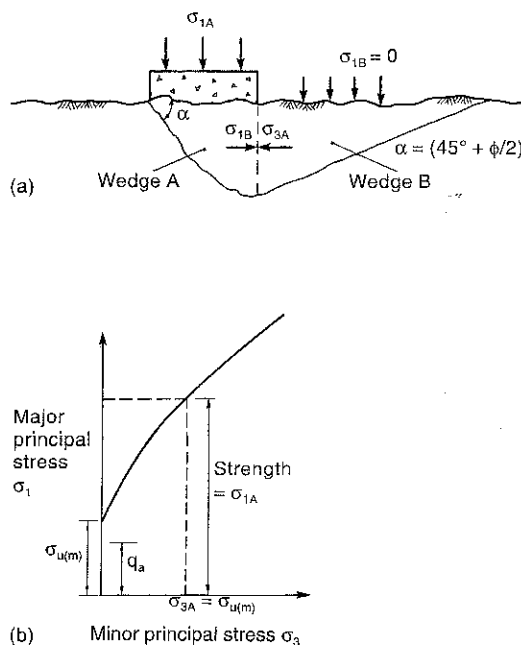


Figure 5.3 Analysis of bearing capacity of fractured rock: (a) active A and passive B wedges in foundation; and (b) curved rock mass strength envelope. Allowable bearing pressure = q_a , strength of bearing rock = σ_{1A} , factor of safety $FS = \sigma_{1A}/q_a$.

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

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

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

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

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

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

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

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

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

5.2.3 Recessed footings

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

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

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

Frost Depth

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

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

DFI = 1850 degree-days.

Case 1 - coarse grained granular fill soils W=20% (assumed).

For DFI = 2200

$d := 82.6\text{in}$

Depth of Frost Penetration

$d = 83\cdot\text{in}$

$d = 6.9\cdot\text{ft}$

Method 2 - ModBerg Software

Examine foundations placed on coarse grained fill soils

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

--- ModBerg Results ---

Project Location: Houlton 5 N, Maine

Air Design Freezing Index = 2189 F-days

N-Factor = 0.80

Surface Design Freezing Index = 1751 F-days

Mean Annual Temperature = 40.8 deg F

Design Length of Freezing Season = 148 days

Layer

#:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	99.4	20.0	125.0	34	46	3.8	1.9	3,600

t = Layer thickness, in inches.

w% = Moisture content, in percentage of dry density.

d = Dry density, in lbs/cubic ft.

Cf = Heat Capacity of frozen phase, in BTU/(cubic ft degree F).

Cu = Heat Capacity of thawed phase, in BTU/(cubic ft degree F).

Kf = Thermal conductivity in frozen phase, in BTU/(ft hr degree).

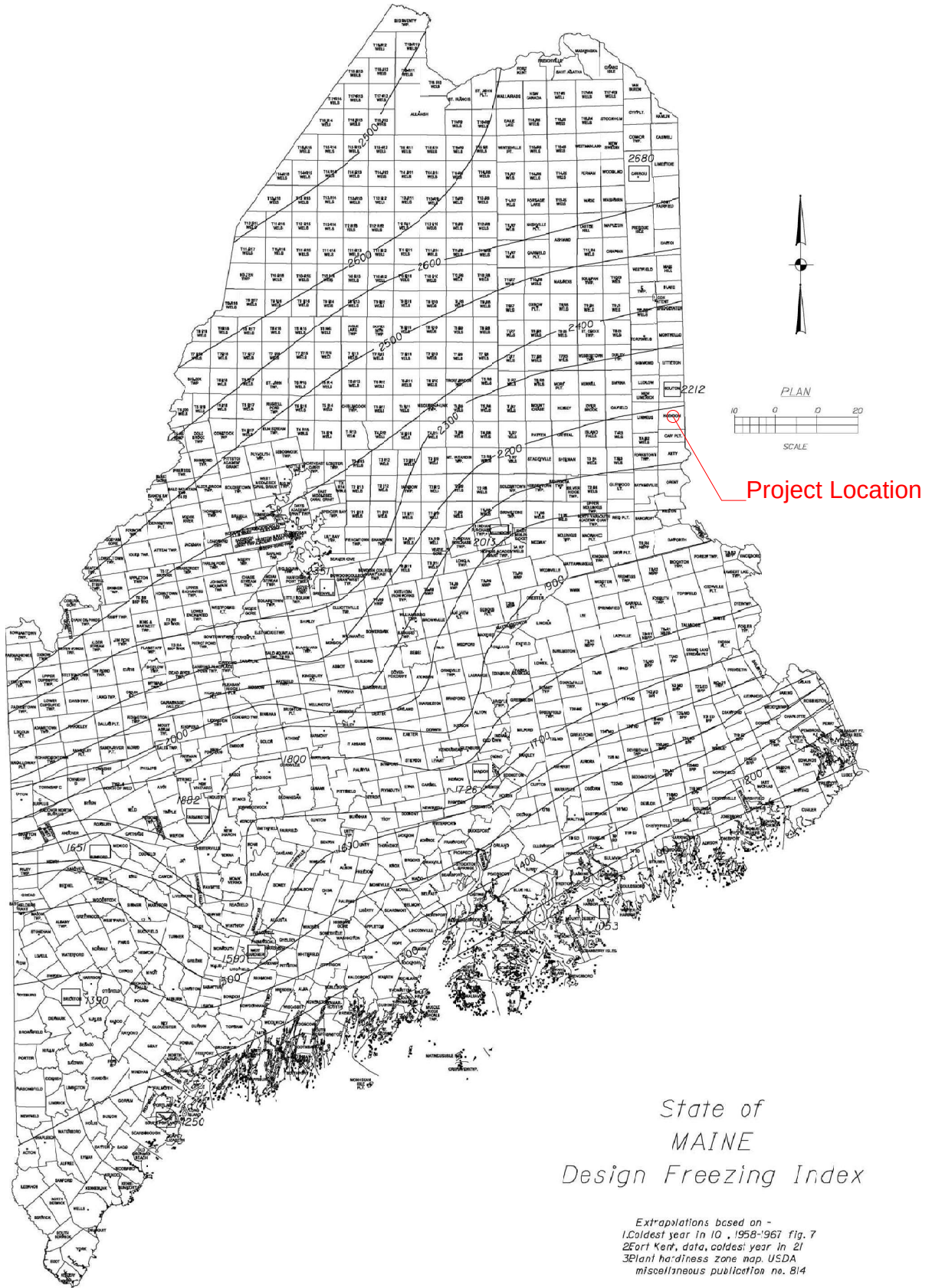
Ku = Thermal conductivity in thawed phase, in BTU/(ft hr degree).

L = Latent heat of fusion, in BTU / cubic f

Total Depth of Frost Penetration = 8.28 ft = 99.4 in.

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

Figure 5-1 Maine Design Freezing Index Map



5.2 General

MaineDOT Bridge Design Guide

5.2.1 Frost

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

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

Table 5-1 Depth of Frost Penetration

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

Seismic Design

Site Classification per LRFD Table C3.10.3.1-1

BB-HMR-101 - Method B				
Depth	101 N ₆₀		di	di/N
6	8		6	0.75
12.5	3		6.5	2.17
16	6		3.5	0.58
17.8	100	Bedrock	1.8	0.02
100	100		82.2	0.82
SUM			100	4.34

di/di/N 23.04
Site Class D

BB-HMR-102 - Method B				
Depth	N ₆₀		di	di/N
3	12		3	0.25
6	14		3	0.21
11	100		5	0.05
14.8	100		3.8	0.04
17.8	100		3	0.03
100	100	Bedrock	82.2	0.82
SUM			100	1.40

di/di/N 71.21
Site Class C

Sum N_{av.} 47.13

15< Nav. <50 blows/ft

Conclusion: Site Class C

Hodgdon, Hodgdon Mills Bridge
22638.00

Seismic Parameters

B. Slaven
Aug 2018
Check by: LK 8/2018

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 46.053986
Longitude = -067.868523

Site Class B

Data are based on a 0.05 deg grid spacing.

	Period (sec)	Sa (g)
0.0	0.067	PGA - Site Class B
0.2	0.152	Ss - Site Class B
1.0	0.047	S1 - Site Class B

2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1

Latitude = 46.053986

Longitude = -067.868523

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

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

Data are based on a 0.05 deg grid spacing.

	Period (sec)	Sa (g)
0.0	0.080	As - Site Class C
0.2	0.183	SDs - Site Class C
1.0	0.080	SD1 - Site Class C

Table 3.4.2.3-2—Values of F_v as a Function of Site Class and Mapped 1-sec Period Spectral Acceleration Coefficient

Site Class	Mapped Spectral Response Acceleration Coefficient at 1-sec Periods				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See AASHTO Article 3.4.3				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = C and $S_1 = 0.047$ g, $F_v = 1.700$

Equation (3.4.1-1):

$$A_S = F_{PGA} \text{ PGA} = 1.200 \times 0.067 = 0.080 \text{ g}$$

Equation (3.4.1-2):

$$S_{DS} = F_a S_S = 1.200 \times 0.152 = 0.183 \text{ g}$$

Equation (3.4.1-3):

$$S_{D1} = F_v S_1 = 1.700 \times 0.047 = 0.080 \text{ g}$$

Figure 3.4.1-1: Design Response Spectrum

