MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

SAWYER BRIDGE
SHAW HILL ROAD OVER FALLS BROOK
INDUSTRY, MAINE

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Franklin County
WIN 21695.00

Soils Report 2018-35
Bridge No. 5047

October 3, 2017
STP-2169(500)

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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 Sawyer Bridge, which carries Shaw Hill Road over Falls Brook in Industry, 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 Sawyer Bridge was constructed in 1925. The superstructure has a twenty-foot curb-to-curb width and consists of a two-span concrete slab. Each span is 13 feet long. The pier and abutments consist of mass concrete founded on unlevel, jagged, bedrock.

The 2017 Maine Department of Transportation (MaineDOT) Bridge Maintenance inspection report rates the substructure, deck, and the superstructure a 4 – poor. The existing bridge has a Federal Highway Administrations Sufficiency Rating of 36.3. The overall poor condition of the structure warranted the project scope of replacement.

Preliminary design identified two feasible replacement structure types. The structure types were a traditional bridge with full height abutments founded on bedrock and a buried arch with spread footings constructed on bedrock. A buried arch with spread footings on bedrock was identified as the preferred replacement structure type. Because precast concrete arches and composite arch systems are suitable replacement arch structures, this project will utilize a detail-build contracting method. With detail-build contracts, the replacement structure is designed, detailed, and constructed in accordance with a detail-build special provision that is provided in the contract documents.

2.0 GEOLOGIC SETTING

Sawyer Bridge carries Shaw Hill Road over Falls Brook in Industry, Maine approximately 200 feet south of State Route 148, as shown on Sheet 1 – Location Map.

The Maine Geologic Survey (MGS) Surficial Geology of the New Vineyard Quadrangle, Maine, Open-file No. 09-48 (2009) indicates the surficial soils in the vicinity of the bridge project consist of undifferentiated glacial outwash and glacial till. Undifferentiated glacial outwash is comprised of sand, silt, and gravel deposited by meltwater streams and may include postglacial stream terrace deposits that resemble outwash. Glacial till is comprised of a mixture of clay, silt, sand, pebbles, cobbles, and boulders.

The MGS Geologic Map of the Kingfield and Anson Quadrangles, MGS Map No. GM-7 (1979), indicates the bedrock in the vicinity of the bridge project as the Sangerville Formation. The principal part of the Sangerville Formation consists of weakly calcareous metagraywacke with grain sizes ranging from fine silt to coarse sand.

3.0 SUBSURFACE INVESTIGATION

Two test borings explored subsurface conditions at the site. Boring BB-IFB-101 was located south of the existing structure behind Abutment No. 1. Boring BB-IFB-102 was located north 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 September 13, 2017. Details and sampling methods used, field data obtained, and soil conditions encountered are presented on the boring logs in 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 dill 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 April 2017. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.854 to the raw field N-values. This hammer efficiency factor (0.854) 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 samples recovered from the test borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the site. Soil laboratory testing consisted of four standard grain size analyses with natural water content tests. Bedrock laboratory testing consisted of two unconfined compressive test with elastic moduli. The results of laboratory soil tests are included in Appendix B – Laboratory Test Results. Moisture content information and other soil and rock test results are 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 metamorphic bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs. A generalized subsurface profile is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile. The following paragraphs discuss the subsurface conditions encountered in detail:

5.1 Fill Soils

The fill unit encountered was approximately 11.4 to 12.5 feet thick at the test boring locations. The fill material generally consisted of:

- Brown, damp, sand, some gravel, some to trace silt;
- Reddish-brown, moist, gravelly sand, little silt;
- Grey, wet, medium dense, sandy gravel, little silt; and
- Cobbles.

Corrected SPT N-values in the fill soils ranged from 7 to 28 blows per foot (bpf), indicating the fill soils are 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 GM, SM, or SW-SM by the Unified Soil Classification System (USCS). The natural water content of the samples tested ranged from approximately 5 to 16 percent.

5.2 Bedrock

Bedrock was encountered and cored in BB-IFB-101 and BB-IFB-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-IFB-101	8+97.1	6.0 Rt.	12.8	498.7	R1	44
					R2	20
BB-IFB-102	9+45.5	5.3 Lt.	12.8	497.1	R1	0
					R2	63

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

The bedrock is identified as grey, fine-grained metasiltstone to metasandstone, moderately hard, fresh, joints are moderately dipping, close, open to healed (calcite), joint surfaces are rough with occasional iron staining. The RQD of the bedrock cored ranged from 0 to 63 percent correlating to a Rock Mass Quality of poor to fair.

Two laboratory unconfined compressive strength and secant modulus tests were conducted on bedrock core samples. The test results are included in Appendix B. The testing yielded unconfined compressive strengths ranging from 11.1 to 19.6 ksi and Young's modulus values ranging from 7,920 to 11,700 ksi.

5.3 Groundwater

The measured depth to groundwater in test boring BB-IFB-101 was approximately 9.5 feet bgs. The measurement was recorded after completion of the test boring. 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

Preliminary design identified two feasible replacement structure types. The structure types were a traditional bridge with full height abutments founded on spread footings constructed on bedrock and a buried arch founded on spread footings constructed on bedrock. A buried arch with spread footings on bedrock is the preferred replacement structure type due its lower substructure costs and quicker construction. Because precast concrete arches and composite arch systems are suitable replacement arch structures, this project will utilize a detail-build contracting method. With detail-build contracts, the replacement structure is designed, detailed, and constructed in accordance with a detail-build special provision that is provided in the contract documents.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

The project "detail-build" special provision requires cast-in-place spread footing on bedrock to support the buried arch structure and flared wingwalls. The design recommendations in this Section are provided in accordance with AASHTO LRFD Bridge Design Specifications, 8^{th} Edition with interim revisions through 2018 (LRFD).

7.1 Precast Concrete Arch Design and Construction

Buried precast concrete arches are typically detailed on the contract plans with only a basic layout and required hydraulic opening. The manufacturer selected by the Contractor is responsible for the design of the overall structure including the determination of wall thickness, haunch thickness, and reinforcement. Buried arches and frames shall be designed for all relevant strength and service limit states and load combinations specified in LRFD Article 3.4.1, LRFD Section 12, Special Provision 531 – Detail-Build Bridge Structure, and Standard Specification 534 – Precast Structural Concrete Arches, Box Culverts, Frames. The loading specified for the structure should be Modified HL-93 Strength I in which the HS-20 design truck wheel loads are increased by a factor of 1.25. The design should use Soil Type 4 as presented in the MaineDOT Bridge Design Guide (BDG) Section 3.6 to design earth loads from the soil envelope. The backfill properties are as follows: ϕ =32 degrees, γ = 125 pcf.

The precast concrete arch or frame shall be constructed in conformance with MaineDOT BDG Section 8 and Standard Specification 534.

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

7.2 Concrete Headwalls

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

Larger precast or cast-in-place concrete headwalls shall be designed as conventional retaining walls for all relevant strength and service limit state load combinations specified in LRFD Articles 3.4.1, 11.5.5 and 11.6. The headwalls shall be designed to resist all lateral earth loads, live loads, creep and temperature and shrinkage deformations of the concrete arch. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 2 below:

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

Table 2 – Equivalent Height of Soil for Estimating Live Load Surcharge on Headwalls

Headwalls that are fixed to the arch or frame to should be designed using an at-rest earth pressure coefficient, K_o, of 0.47, assuming the walls are to be prevented from movement. Headwalls that are independent of the arch or frame structure shall be designed as unrestrained meaning that they are free to rotate at the top in an active state of earth pressure using the Rankine active earth pressure coefficient, K_a, of 0.31 assuming a level backslope. The active earth pressure should be recalculated if the backslope conditions are different. The designer may assume Soil Type 4 (MaineDOT BDG Section 3.6.1) for headwall backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf.

Footings for headwalls which are not fixed to the arch or frame shall be constructed on footings cast directly on bedrock and shall meet the additional design requirements for spread footings on bedrock detailed in Sections 7.3 through 7.6 of this report.

7.3 Bedrock Removal and Bedrock Subgrade Preparation

The overburden encountered in the borings at the proposed footing locations is approximately 12.5 feet thick. It is feasible that spread footings can be practically and economically constructed to bear on bedrock at this location, possibly without temporary soil support systems. The boring indicates that bedrock with an RQD ranging from approximately 0 to 44 percent will be encountered at the bedrock surface at the footing locations. However, based upon roller cone behavior, approximately one foot of weathered bedrock is also likely present at the proposed Abutment No. 2 location. 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, fill concrete, or seal 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 arch 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.

7.4 Bearing Resistance of Spread Footings on Bedrock

Cast-in-place spread footings shall be proportioned to provide stability against bearing capacity failure. Application of permanent and transient loads shall be as specified in LRFD Article 11.5.6. 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.

For extreme limit state load combinations, a factored bearing resistance of 19 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 be 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.

7.5 Spread Footing Design for Arches and Wingwalls

Spread footings for arches 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. Spread footings for arches and wingwalls shall be designed to resist all lateral earth loads, vehicular loads, deadloads, and withstand temperature and shrinkage effects. The design of spread footings 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 footings to bedrock may be required to resist sliding forces and improve stability. Dowels should be #9 reinforcing bars or larger and embedded a nominal 12 inches into bedrock and into the footings by a depth determined by the designer. If bedrock is observed to slope steeper than 4H:1V at the footing subgrade elevation, the bedrock should be benched to create level steps 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 footing.

For the service limit state, a resistance factor, φ,of 1.0 shall be used to assess spread footing design for settlement, horizontal movement, bearing resistance, sliding, and eccentricity. The overall global stability of foundations are 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.

For scour protection of arch and wingwall spread footings, 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.

7.6 Earth Pressures and Surcharge Forces

Calculation of earth pressures acting on arches and their footings should assume an at-rest earth pressure coefficient, K_o, of 0.47, assuming the arch footings are prevented from movement. Calculation of earth pressures mobilized to resist outward thrust forces from the arches shall also assume an at-rest earth pressure coefficient, K_o, of 0.47.

The designer may assume Soil Type 4 (MaineDOT BDG Section 3.6.1) for arch footing and wall backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf.

Independent wingwalls shall be designed as unrestrained meaning that they are free to rotate at the top in an active state of earth pressure. Earth loads shall be calculated using an active earth pressure coefficient, K_a, of 0.31 assuming a level backfill. For a 2H:1V backslope, the designer shall use a Rankine active earth pressure coefficient, K_a, of 0.52. The active earth pressure coefficient will change if the backslope conditions are different. See Appendix C – Calculations for supporting documentation.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the MaineDOT BDG. Construction surcharge on arch stem wall footings and retaining walls may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) of 2.0 feet, per LRFD Table 3.11.6.4-2. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from the Table 3 below:

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

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

Arch foundations and wingwall designs shall include a drainage system behind the wall stems to intercept any groundwater. Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage, of the MaineDOT BDG.

Backfill within 10 feet of arches and wingwalls shall conform to Granular Borrow for Underwater Backfill - MaineDOT Specification 703.19. This gradation specifies 7 percent or less of the material passing the No. 200 sieve. Limiting the amount of fines is intended to minimize frost action and eliminate the need to design for hydrostatic forces by promoting drainage behind the structure.

7.7 Settlement

The approximately 11.4 to 12.5-foot thick fill unit is 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 structure and its foundations will be constructed on bedrock. Any settlement of foundations placed on properly prepared bedrock will be due to elastic compression of the bedrock and will be negligible.

7.8 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, Industry has a design freezing index (DFI) of approximately 1800 F-degree days. A water content of 20% was used for coarse-grained soils. These components correlate to a frost depth of 6.2 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Farmington, Maine has an air DFI from the Modberg database of approximately 2023 F-degree days. Farmington was selected because it lies near the same isoline as Industry and Industry is not available in the Modberg database. A water content of 20% was assumed. These components correlate to a frost depth of approximately 8.0 feet.

Based on the MaineDOT BDG methodology it is recommended that foundations bearing on coarse-grained soils be designed with an embedment of approximately 6.2 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.

7.8 Scour and Riprap

For scour protection of the footings, construct the concrete footings 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 Standard Specification Section 703.26 – Plain and Hand Laid Riprap. The riprap shall be underlain by a Class 1 erosion control geotextile and a 1-foot layer of bedding material conforming to MaineDOT Standard Specification 703.19 Granular Borrow Material for Underwater backfill. The toe of the riprap sections shall be constructed 1 foot below the streambed elevation unless the streambed consists of bedrock. 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 of 1.75H:1V extending from the edge of the roadway down to the existing ground surface.

7.9 Seismic Design Considerations

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

7.10 Construction Considerations

Construction of the arch spread footings, headwalls, and wingwalls will require soil and rock excavation and removal of the existing bridge abutments and pier. Cofferdams will be required to permit construction of footings in the dry.

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

The subgrade for spread footings for arches and retaining walls shall consist of sound bedrock. The nature, slope, and degree of fracturing in the bedrock bearing surfaces will not be evident until the foundation excavation is made. The bedrock subgrade surface shall be cleaned of all overburden soils and loose or dislodged bedrock fragments by mechanical means. Expansive agents, hydraulic hoe ram, hydraulic splitters, and wedging and prying are examples of tools and methods for removing bedrock by mechanical means. The final bedrock bearing surface shall be washed with high pressure water and air prior to concrete being placed for the arch and wingwall footings.

The bedrock subgrade for foundations shall slope less than 4H:1V. If the bedrock subgrade slopes greater than 4H:1V, the subgrade 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 at arch or wingwall footing locations.

The Contractor may excavate bedrock using conventional excavation methods, but excavation of the bedrock may require drilling and blasting techniques. Blasting should be conducted in accordance with Section 105.2.6 of the MaineDOT Standard Specifications. It is also recommended that the contractor conduct pre-and post-blast surveys. The Resident shall approve the final bedrock surface prior to placement of the footing concrete.

It is anticipated that there will be seepage of water from fractures and joints exposed in the bedrock surface. Surface water should be diverted from the foundation excavation throughout the period of construction. Water encountered at the base of the foundation excavation should be removed by using a sump pump located in the corner of the excavation outside of the foundation footprint. The contractor should maintain the excavation so that all foundations are constructed in the dry.

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

8.0 CLOSURE

This report has been prepared for use by the MaineDOT Bridge Program for the specific application to the proposed replacement of Sawyer Bridge in Industry, 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

Date:1/22/2018

Division: GEOTECH

Username: Terry.White

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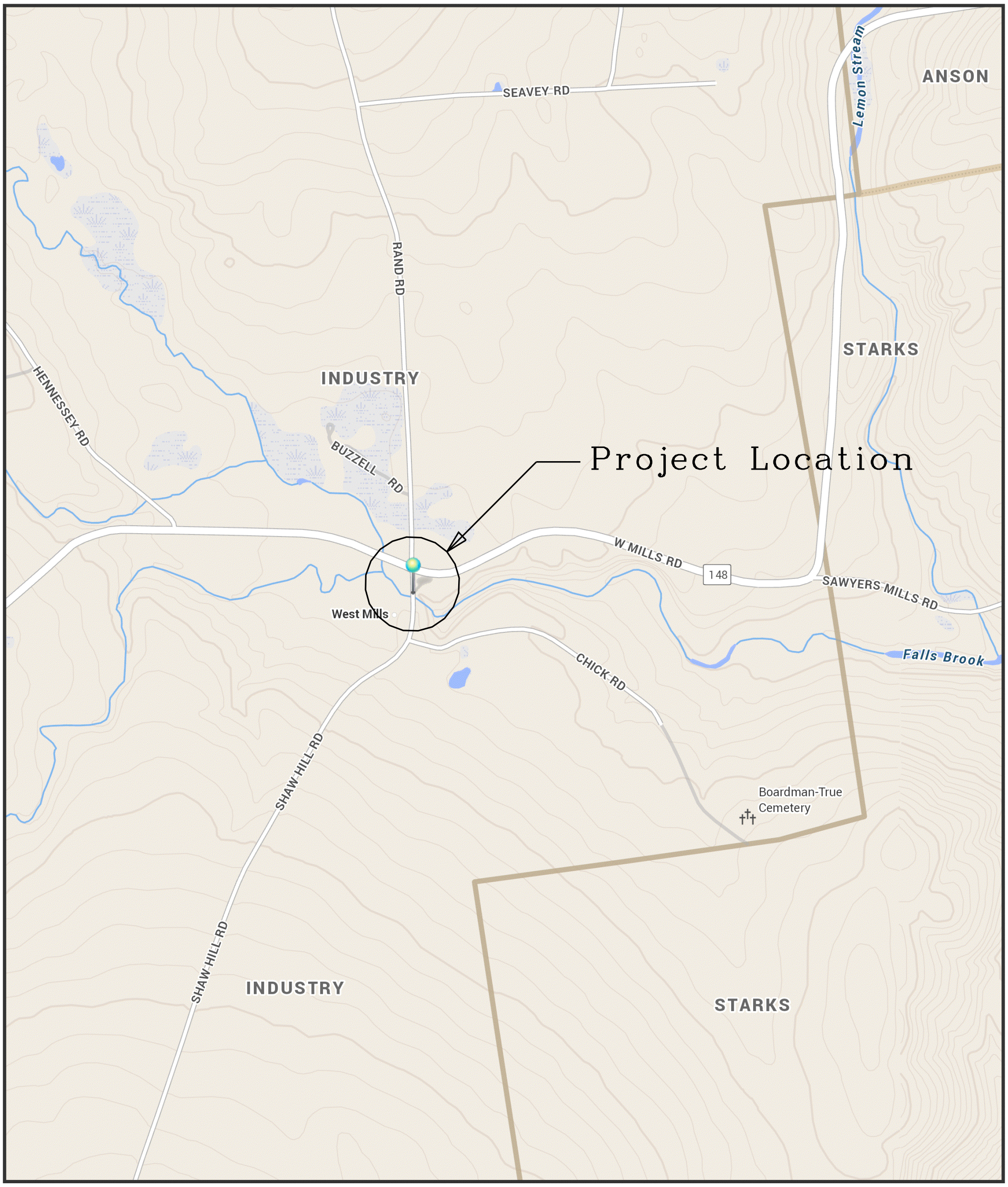
INDUSTRY, MAINE



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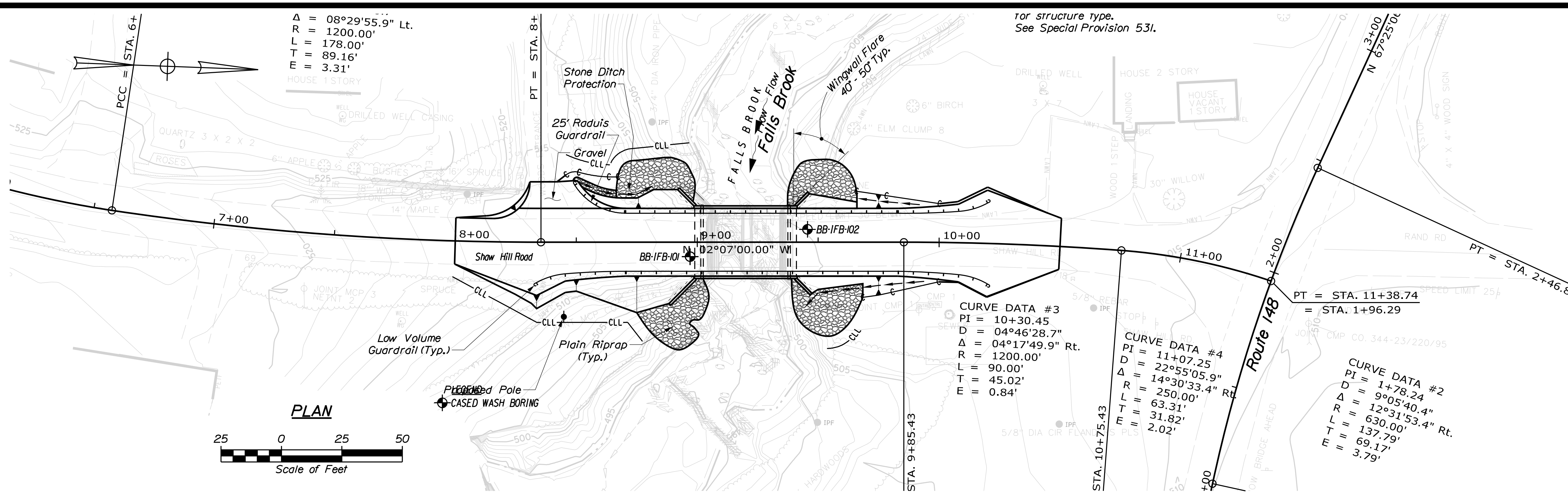
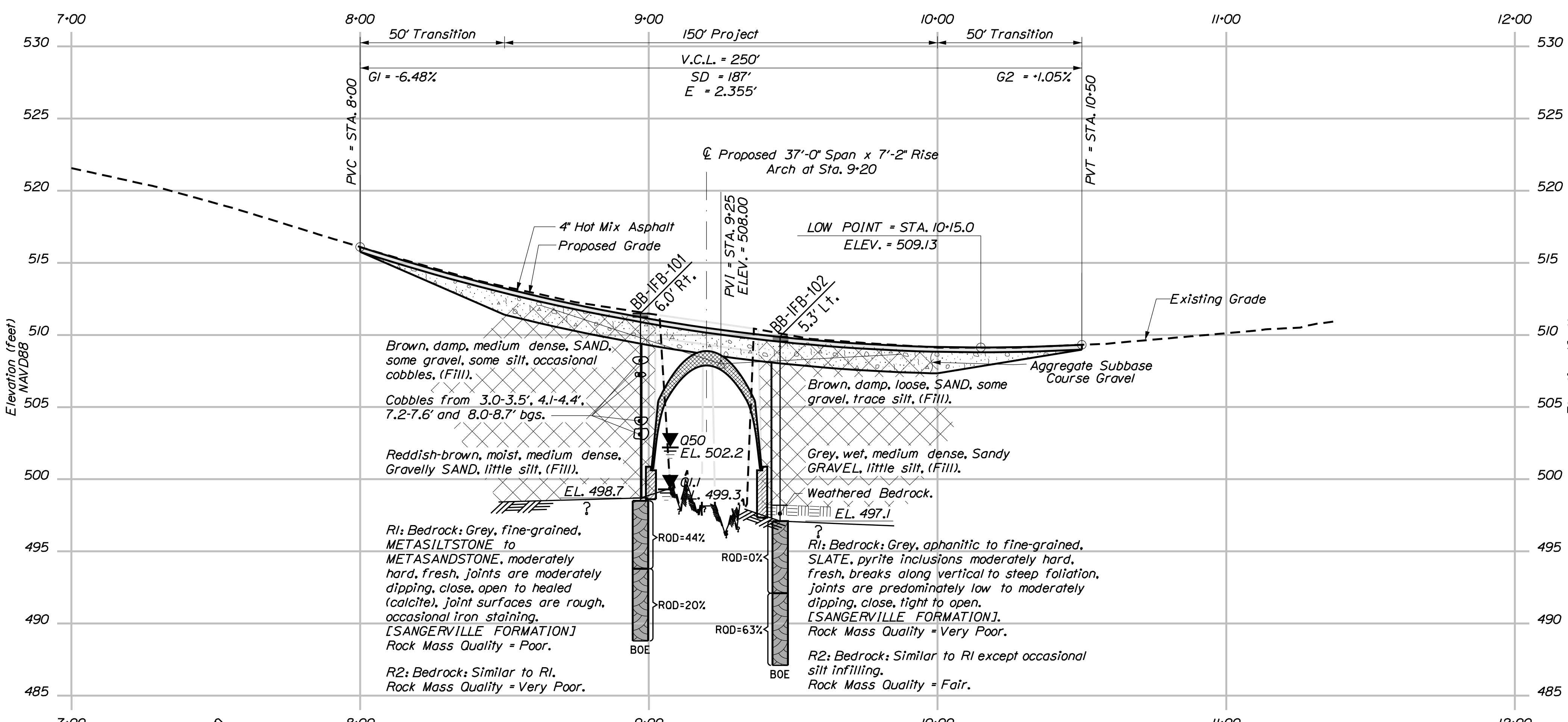
0.25 Miles
1 inch = 0.28 miles

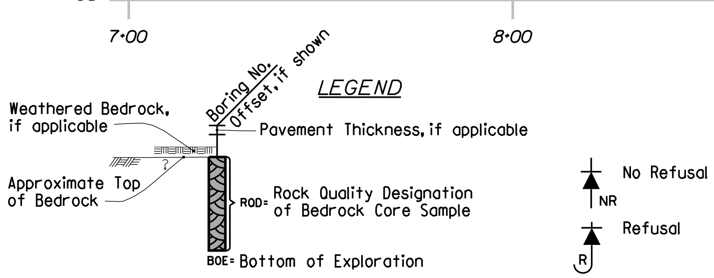
Date: 1/22/2018
Time: 6:57:00 AM

SHEET NUMBER	SAWYER BRIDGE	STATE OF MAINE DEPARTMENT OF TRANSPORTATION
1	FALLS BROOK	STP-2169(500)
	INDUSTRY FRANKLIN COUNTY	WIN
OF 3	LOCATION MAP	BRIDGE NO. 5047 21695.00 BRIDGE PLANS

Filename: ...\GEOTECH\MSTA\004_BLP&SP1.dgn Division: GEOTECH Username: Brandon.Sloven Date:10/24/2018







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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2169(500)

WIN 21695.00

BRIDGE NO. 5047 BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		
CHECKED-REVIEWED	B.SLAVEN	
DESIGN2-DETAILED2	T.WHITE	SEP 2017
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

SAWYER BRIDGE
FALLS BROOK
INDUSTRY FRANKLIN COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

2

OF 3

Filename: ...\MSTA\005_BORING LOGS1.dgn   Division: GEOTECH   Username: Brandon.Sloven   Date:10/24/2018

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Sawyer Bridge #5047 carries Shaw Hill Road over Falls Brook
Location: Industry, Maine

Boring No.: BB-IFB-101
WIN: 21695.00

Driller:	MaineDOT	Elevation (ft.)	511.5	Auger ID/OD:	5" Solid Stem
Operator:	Travis/James	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	9/13/2017; 11:00-14:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	8+97.1, 6.0 ft Rt.	Casing ID/OD:	NW-3"	Water Level*:	9.5 ft bgs.
Hammer Efficiency Factor: 0.854		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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	511.3	3" HMA. 0.3	
									Cobble from 3.0-3.5 ft bgs.	
									Cobble from 4.1-4.4 ft bgs.	
5	1D	24/14	5.00 - 7.00	6/10/7/7	17	24	65		Brown, damp, medium dense, SAND, some gravel, some silt, occasional cobbles, (Fill).	G#303028 A-1-b, SM WC=15.2%
							56			
							166		Roller Coned ahead from 7.0-10.0 ft bgs. Cobble from 7.2-7.6 ft bgs.	
							134		Cobble from 8.0-8.7 ft bgs.	
							17			
10	2D	24/15	10.00 - 12.00	8/8/12/28	20	28	16		Reddish-brown, moist, medium dense, Gravelly SAND, little silt, (Fill).	G#303029 A-1-b, SM WC=16.0%
							42			
							a157	498.7	a157 blows for 0.8 ft. 12.8	
	R1	56.4/55	13.00 - 17.70	RQD = 44%			NQ-2		Top of Bedrock at Elev. 498.7 ft. Roller Coned ahead to 13.0 ft bgs. R1: Bedrock: Grey, fine-grained, METASILTSTONE to METASANDSTONE, moderately hard, fresh, joints are moderately dipping, close, open to healed (calcite), joint surfaces are rough, occasional iron staining. [SANGERVILLE FORMATION] Rock Mass Quality = Poor. R1: Core Times (min:sec) 13.0-14.0 ft (1:48) 14.0-15.0 ft (2:50) 15.0-16.0 ft (1:33) 16.0-17.0 ft (1:50) 17.0-17.7 ft (3:00) Core Blocked 99% Recovery	13.2-13.5 q_p= 19.5 ksi
15										
	R2	60/60	17.70 - 22.70	RQD = 20%					R2: Bedrock: Similar R1. Rock Mass Quality = Very Poor. R2: Core Times (min:sec) 17.7-18.7 ft (1:18) 18.7-19.7 ft (1:17) 19.7-20.7 ft (1:06) 20.7-21.7 ft (1:24) 21.7-22.7 ft (1:31) 100% Recovery	
20								488.8	22.7 Bottom of Exploration at 22.7 feet below ground surface.	
25										

Remarks:

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

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

Page 1 of 1

Boring No.: BB-IFB-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Sawyer Bridge #5047 carries Shaw Hill Road over Falls Brook
Location: Industry, Maine

Boring No.: BB-IFB-102
WIN: 21695.00

Driller:	MaineDOT	Elevation (ft.)	509.9	Auger ID/OD:	5" Solid Stem
Operator:	Travis/James	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	9/13/2017; 08:00-11:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	9+45.5, 5.3 ft Lt.	Casing ID/OD:	NW-3"	Water Level*:	None Observed
Hammer Efficiency Factor: 0.854		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	509.6	4" HMA. 0.3	
5	1D	24/7	5.00 - 7.00	5/2/3/3	5	7	13		Brown, damp, loose, SAND, some gravel, trace silt, (Fill).	G#303030 A-1-b, SW-SM WC=4.8%
							9			
							8			
							12			
							35			
10	2D	24/13	10.00 - 12.00	10/11/8/11	19	27	27		Grey, wet, medium dense, Sandy GRAVEL, little silt, (Fill).	G#303031 A-1-a, GM WC=11.4%
							35	498.2	Weathered bedrock in tip of spoon. 11.7	
	R1	60/60	12.80 - 17.80	RQD = 0%			a150 NQ-2	497.1	Weathered Bedrock. a150 blows for 0.8 ft. 12.8 Top of Bedrock at Elev. 497.1 ft. R1: Bedrock: Grey, aphanitic to fine-grained, SLATE, pyrite inclusions moderately hard, fresh, breaks along vertical to steep foliation, joints are predominately low to moderately dipping, close, tight to open. [SANGERVILLE FORMATION]. Rock Mass Quality = Very Poor. R1: Core Times.(min:sec) 12.8-13.8 ft (1:50) 13.8-14.8 ft (2:28) 14.8-15.8 ft (1:45) 15.8-16.8 ft (1:42) 16.8-17.8 ft (2:10) 100% Recovery	
15										
	R2	60/60	17.80 - 22.80	RQD = 63%					R2: Bedrock: Similar to R1 except occasional silt infilling. Rock Mass Quality = Fair. R2: Core Times (min:sec) 17.8-18.8 ft (1:37) 18.8-19.8 ft (1:43) 19.8-20.8 ft (1:50) 20.8-21.8 ft (1:31) 21.8-22.8 ft (1:44) 100% Recovery	21.7-22.0 q_p= 11.1 ksi
20								487.1	22.8 Bottom of Exploration at 22.8 feet below ground surface.	
25										

Remarks:

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

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

Page 1 of 1

Boring No.: BB-IFB-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2169(500)

BRIDGE NO. 5047

WIN 21695.00

BRIDGE PLANS

PROJ. MANAGER	BY	DATE
DESIGN-DETAILED		
CHECKED-REVIEWED		
DESIGN2-DETAILED2	B.SLAVEN T.WHITE	AUG 2018
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

SAWYER BRIDGE
FALLS BROOK
INDUSTRY FRANKLIN COUNTY

BORING LOGS

SHEET NUMBER
3
OF 3

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

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

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

MODIFIED BURMISTER SYSTEM

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations (in this order, if applicable):
Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Rock Type (granite, schist, sandstone, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
- -dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.)
- -spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet)
- -tightness (tight, open, or healed)
- -infilling (grain size, color, etc.)

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock mass quality (very poor, poor, etc.)
ref: ASTM D6032 and AASHTO Standard Specification for Highway Bridges, 17th Ed. Table 4.4.8.1.2A
Recovery (inch/inch and percentage)
Rock Core Rate (X.X ft - Y.Y ft (min:sec))

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project:	Sawyer Bridge #5047 carries Shaw Hill Road over Falls Brook	**Boring No.:**	BB-IFB-101		
Location:	Industry, Maine	**WIN:**	21695.00		
Driller:	MaineDOT	**Elevation (ft.)**	511.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Travis/James	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/13/2017; 11:00-14:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	8+97.1, 6.0 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	9.5 ft bgs.
Hammer Efficiency Factor:	0.854	**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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	511.3		3" HMA. 0.3	
										Cobble from 3.0-3.5 ft bgs.	
										Cobble from 4.1-4.4 ft bgs.	
5	1D	24/14	5.00 - 7.00	6/10/7/7	17	24	65			Brown, damp, medium dense, SAND, some gravel, some silt, occasional cobbles, (Fill).	G#303028 A-1-b, SM WC=15.2%
							56				
							166			Roller Coned ahead from 7.0-10.0 ft bgs. Cobble from 7.2-7.6 ft bgs.	
							134			Cobble from 8.0-8.7 ft bgs.	
							17				
10	2D	24/15	10.00 - 12.00	8/8/12/28	20	28	16			Reddish-brown, moist, medium dense, Gravelly SAND, little silt, (Fill).	G#303029 A-1-b, SM WC=16.0%
							42				
							[a]157	498.7		[a]157 blows for 0.8 ft. 12.8	
	R1	56.4/55	13.00 - 17.70	RQD = 44%			NQ-2			Top of Bedrock at Elev. 498.7 ft. Roller Coned ahead to 13.0 ft bgs. R1: Bedrock: Grey, fine-grained, METASILTSTONE to METASANDSTONE, moderately hard, fresh, joints are moderately dipping, close, open to healed (calcite), joint surfaces are rough, occasional iron staining. [SANGERVILLE FORMATION] Rock Mass Quality = Poor. R1: Core Times (min:sec) 13.0-14.0 ft (1:48) 14.0-15.0 ft (2:50) 15.0-16.0 ft (1:33) 16.0-17.0 ft (1:50) 17.0-17.7 ft (3:00) Core Blocked 99% Recovery	13.2-13.5 q_p= 19.5 ksi
15											
	R2	60/60	17.70 - 22.70	RQD = 20%							
20										R2: Bedrock: Similar to R1. Rock Mass Quality = Very Poor. R2: Core Times (min:sec) 17.7-18.7 ft (1:18) 18.7-19.7 ft (1:17) 19.7-20.7 ft (1:06) 20.7-21.7 ft (1:24) 21.7-22.7 ft (1:31) 100% Recovery	
								488.8			
25										22.7	

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Sawyer Bridge #5047 carries Shaw Hill Road over Falls Brook
Location: Industry, Maine

Boring No.: BB-IFB-101
WIN: 21695.00

Driller:	MaineDOT	**Elevation (ft.)**	511.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Travis/James	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/13/2017; 11:00-14:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	8+97.1, 6.0 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	9.5 ft bgs.
Hammer Efficiency Factor:	0.854	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										**Bottom of Exploration at 22.7 feet below ground surface.**	
30											
35											
40											
45											
50											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Sawyer Bridge #5047 carries Shaw Hill Road over Falls Brook
Location: Industry, Maine

Boring No.: BB-IFB-102
WIN: 21695.00

Driller:	MaineDOT	**Elevation (ft.)**	509.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Travis/James	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/13/2017; 08:00-11:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	9+45.5, 5.3 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.854	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	509.6	4" HMA.	
									0.3	
5	1D	24/7	5.00 - 7.00	5/2/3/3	5	7	13		Brown, damp, loose, SAND, some gravel, trace silt, (Fill).	G#303030 A-1-b, SW-SM WC=4.8%
							9			
							8			
							12			
							35			
10	2D	24/13	10.00 - 12.00	10/11/8/11	19	27	27		Grey, wet, medium dense, Sandy GRAVEL, little silt, (Fill). Weathered bedrock in tip of spoon.	G#303031 A-1-a, GM WC=11.4%
							35	498.2	11.7	
	R1	60/60	12.80 - 17.80	RQD = 0%			a150 NQ-2		Weathered Bedrock. a150 blows for 0.8 ft.	
								497.1	12.8	
									Top of Bedrock at Elev. 497.1 ft. R1: Bedrock: Grey, aphanitic to fine-grained, SLATE, pyrite inclusions moderately hard, fresh, breaks along vertical to steep foliation, joints are predominately low to moderately dipping, close, tight to open. [SANGERVILLE FORMATION]. Rock Mass Quality = Very Poor. R1: Core Times.(min:sec) 12.8-13.8 ft (1:50) 13.8-14.8 ft (2:28) 14.8-15.8 ft (1:45) 15.8-16.8 ft (1:42) 16.8-17.8 ft (2:10) 100% Recovery	
15										
	R2	60/60	17.80 - 22.80	RQD = 63%						
20									R2: Bedrock: Similar to R1 except occasional silt infilling. Rock Mass Quality = Fair. R2: Core Times (min:sec) 17.8-18.8 ft (1:37) 18.8-19.8 ft (1:43) 19.8-20.8 ft (1:50) 20.8-21.8 ft (1:31) 21.8-22.8 ft (1:44) 100% Recovery	21.7-22.0 q_p= 11.1 ksi
								487.1	22.8	
25										

Remarks:

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

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

Boring No.: BB-IFB-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Sawyer Bridge #5047 carries Shaw Hill Road over Falls Brook	**Location:** Industry, Maine	**Boring No.:** BB-IFB-102	**WIN:** 21695.00

Driller:	MaineDOT	**Elevation (ft.)**	509.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Travis/James	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/13/2017; 08:00-11:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	9+45.5, 5.3 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.854	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										**Bottom of Exploration at 22.8 feet below ground surface.**	
30											
35											
40											
45											
50											

Remarks:

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

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

Boring No.: BB-IFB-102

Appendix B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Industry   Work Number: 21695.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-IFB-101, 1D	8+97	6.0 Rt.	5.0-7.0	303028	1	15.2			SM	A-1-b	II
BB-IFB-101, 2D	8+97	6.0 Rt.	10.0-12.0	303029	1	16.0			SM	A-1-b	II
BB-IFB-102, 1D	9+45.5	5.3 Lt.	5.0-7.0	303030	1	4.8			SW-SM	A-1-b	0
BB-IFB-102, 2D	9+45.5	5.3 Lt.	10.0-12.0	303031	1	11.4			GM	A-1-a	0

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

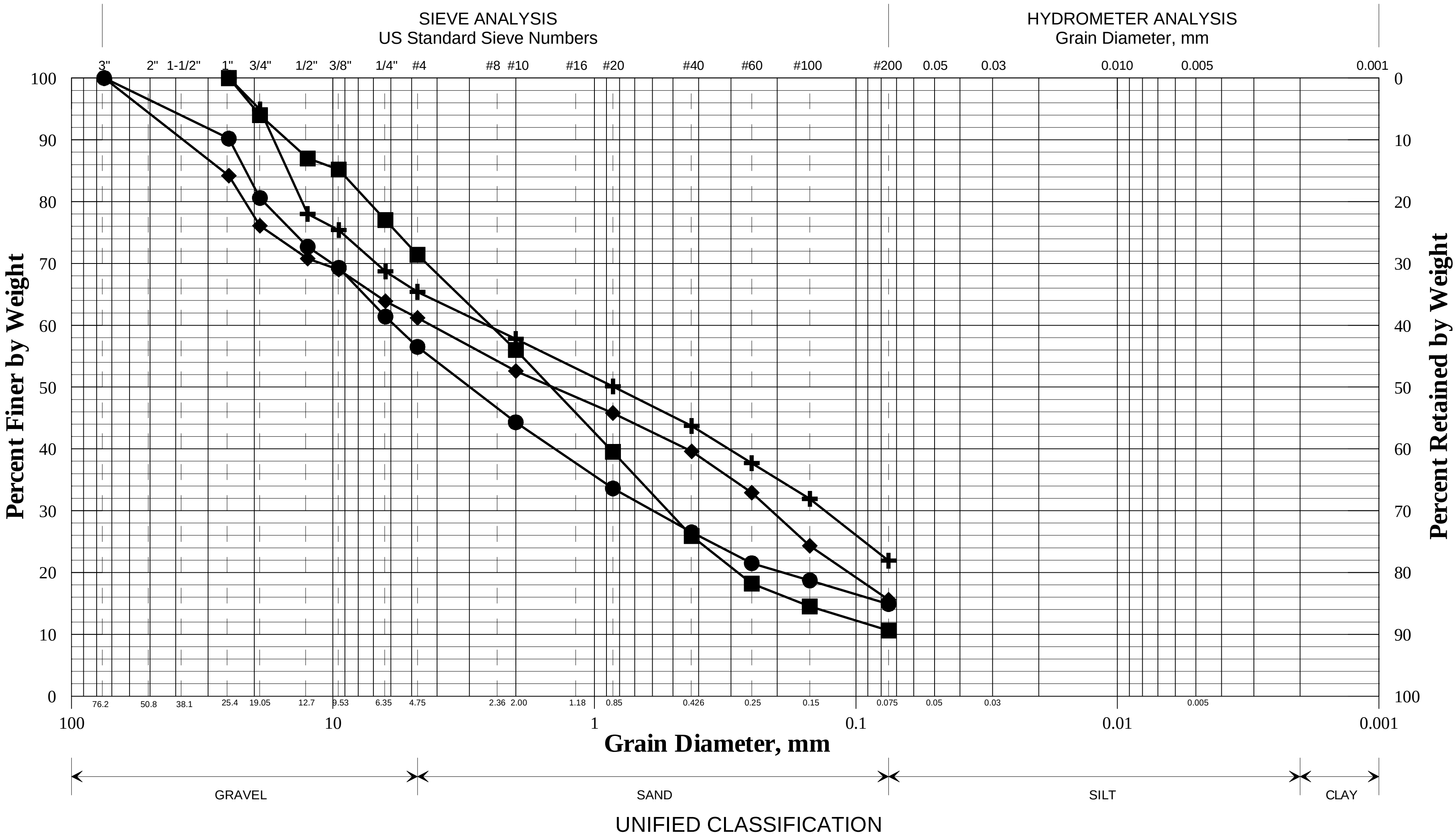
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-IFB-101/1D	8+97	6.0 RT	5.0-7.0	SAND, some gravel, some silt.	15.2			
◆	BB-IFB-101/2D	8+97	6.0 RT	10.0-12.0	Gravelly SAND, little silt.	16.0			
■	BB-IFB-102/1D	9+45.5	5.3 LT	5.0-7.0	SAND, some gravel, trace silt.	4.8			
●	BB-IFB-102/2D	9+45.5	5.3 LT	10.0-12.0	Sandy GRAVEL, little silt.	11.4			
▲									
✖									

WIN
021695.00
Town
Industry
Reported by/Date
WHITE, TERRY A 11/6/2017

SHEET 1

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Geotechnical Test Report **1/16/2018**

GTX-307457

Sawyer Br, Shaw Hill Rd over Falls Brook

Industry, ME

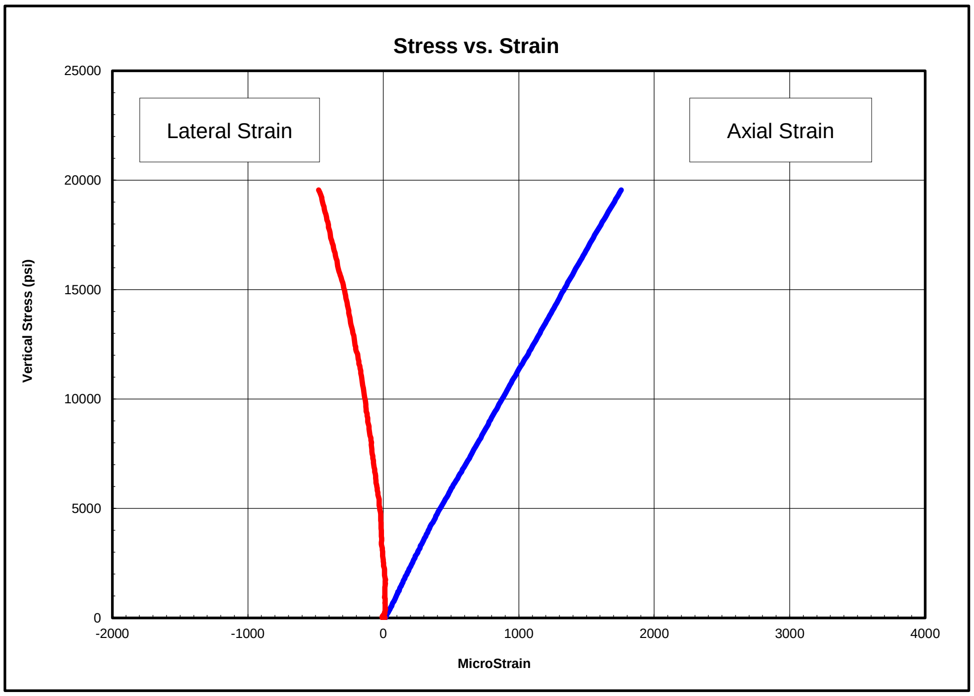
Client Project No.: 21695.00

Prepared for:

Maine Department of Transportation

Client:	Maine Department of Transportation
Project Name:	Sawyer, Br, Shaw Hill Rd over Falls Brook
Project Location:	Industry, ME
GTX #:	307457
Test Date:	1/5/2018
Tested By:	rlc
Checked By:	jsc
Boring ID:	BB-IFB-101
Sample ID:	R1
Depth, ft:	13.15-13.50
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

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



Peak Compressive Stress: 19,554 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
2000-7200	11,600,000	0.18
7200-12400	11,000,000	0.27
12400-17600	11,100,000	0.41

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:	Sawyer, Br, Shaw Hill Rd over Falls Brook	Tested By:	rlc
Project Location:	Industry, ME	Checked By:	jsc
GTX #:	307457		
Boring ID:	BB-IFB-101		
Sample ID:	R1		
Depth:	13.15-13.50 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.07	4.07	4.07
Specimen Diameter, in:	1.96	1.96	1.96
Specimen Mass, g:	543.26		
Bulk Density, lb/ft^3	168	**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.1	**Length to Diameter Ratio Tolerance Met?**	YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap ≤ 0.02 in.? NO

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? NO

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00030	-0.00030	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00020	-0.00020	-0.00030	-0.00040
Diameter 2, in (rotated 90°)	0.00000	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00010	0.00010	0.00020	0.00020	0.00020	0.00020	0.00020

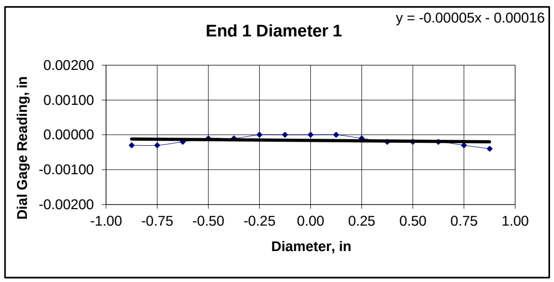
Difference between max and min readings, in:
0° = 0.00040 90° = 0.00030

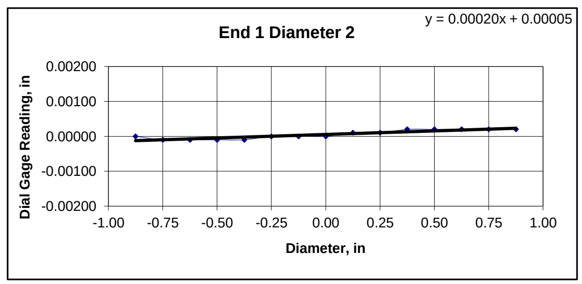
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00020	-0.00020	-0.00020	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00020	-0.00030	-0.00030
Diameter 2, in (rotated 90°)	-0.00020	-0.00020	-0.00020	-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° = 0.0003 90° = 0.0003

Maximum difference must be < 0.0020 in. Difference = ± 0.00020

Flatness Tolerance Met? YES





DIAMETER 1

End 1:

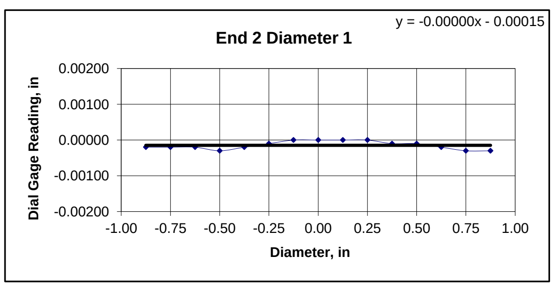
Slope of Best Fit Line	0.00005
Angle of Best Fit Line:	0.00286

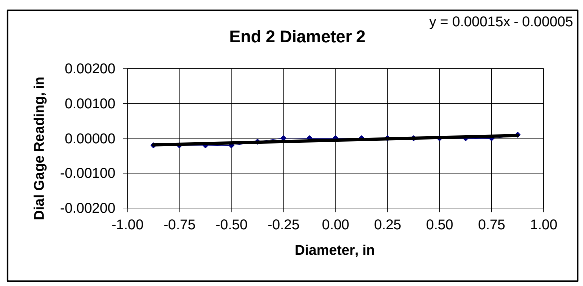
End 2:

Slope of Best Fit Line	0.00000
Angle of Best Fit Line:	0.00000

Maximum Angular Difference: 0.00286

Parallelism Tolerance Met? YES
Spherically Seated





DIAMETER 2

End 1:

Slope of Best Fit Line	0.00020
Angle of Best Fit Line:	0.01146

End 2:

Slope of Best Fit Line	0.00015
Angle of Best Fit Line:	0.00859

Maximum Angular Difference: 0.00286

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?
Diameter 1, in	0.00040	1.960	0.00020	0.012	YES
Diameter 2, in (rotated 90°)	0.00030	1.960	0.00015	0.009	YES
END 2					
Diameter 1, in	0.00030	1.960	0.00015	0.009	YES
Diameter 2, in (rotated 90°)	0.00030	1.960	0.00015	0.009	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? YES

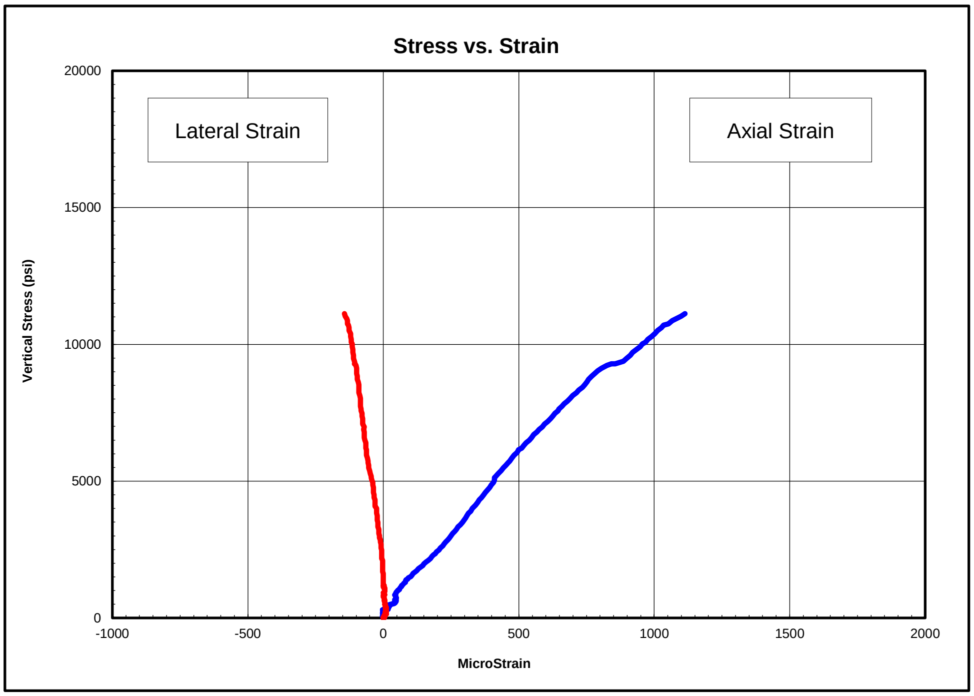
Client:	Maine Department of Transportation
Project Name:	Sawyer, Br, Shaw Hill Rd over Falls Brook
Project Location:	Industry, ME
GTX #:	307457
Test Date:	1/5/2018
Tested By:	rlc
Checked By:	jsc
Boring ID:	BB-IFB-101
Sample ID:	R1
Depth, ft:	13.15-13.50

After cutting and grinding

After break

Client:	Maine Department of Transportation
Project Name:	Sawyer, Br, Shaw Hill Rd over Falls Brook
Project Location:	Industry, ME
GTX #:	307457
Test Date:	1/5/2018
Tested By:	rlc
Checked By:	jsc
Boring ID:	BB-IFB-102
Sample ID:	R2
Depth, ft:	21.68-22.03
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

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



Peak Compressive Stress: 11,122 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1100-4100	10,500,000	0.11
4100-7000	11,700,000	0.19
7000-10000	7,920,000	0.11

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.

GeoTesting EXPRESS

Client:	Maine Department of Transportation	Test Date:	1/3/2018
Project Name:	Sawyer, Br, Shaw Hill Rd over Falls Brook	Tested By:	rlc
Project Location:	Industry, ME	Checked By:	jsc
GTX #:	307457		
Boring ID:	BB-IFB-102		
Sample ID:	R2		
Depth:	21.68-22.03 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average		
Specimen Length, in:	4.00	4.00	4.00		
Specimen Diameter, in:	1.97	1.97	1.97		
Specimen Mass, g:	559.62				
Bulk Density, lb/ft^3	174			**Minimum Diameter Tolerence Met?**	**YES**
Length to Diameter Ratio:	2.0			**Length to Diameter Ratio Tolerance Met?**	**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap $\leq$ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? **YES**

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00020	0.00010	0.00000	0.00000	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	0.00000	0.00010
Diameter 2, in (rotated 90°)	0.00060	0.00000	0.00000	0.00000	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00010	0.00000	-0.00010	-0.00020	-0.00010	0.00000

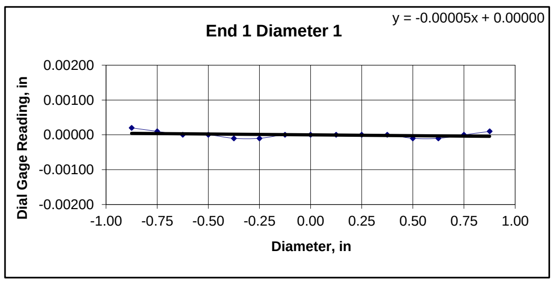
Difference between max and min readings, in:
0° = 0.00030 90° = 0.00080

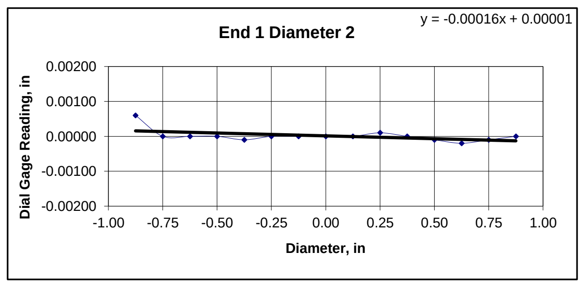
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00030	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00030
Diameter 2, in (rotated 90°)	-0.00020	-0.00020	-0.00020	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	0.00000	0.00010	0.00020	0.00010

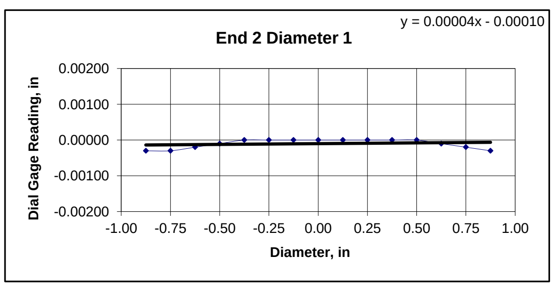
Difference between max and min readings, in:
0° = 0.0003 90° = 0.0005

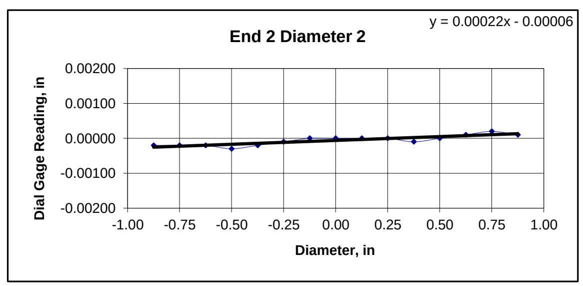
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00040

Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:		
	Slope of Best Fit Line	0.00005
	Angle of Best Fit Line:	0.00286
End 2:		
	Slope of Best Fit Line	0.00004
	Angle of Best Fit Line:	0.00229
Maximum Angular Difference:		0.00057

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

End 1:		
	Slope of Best Fit Line	0.00016
	Angle of Best Fit Line:	0.00917
End 2:		
	Slope of Best Fit Line	0.00022
	Angle of Best Fit Line:	0.01261
Maximum Angular Difference:		0.00344

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?
Diameter 1, in	0.00030	1.970	0.00015	0.009	YES
Diameter 2, in (rotated 90°)	0.00080	1.970	0.00041	0.023	YES
END 2					
Diameter 1, in	0.00030	1.970	0.00015	0.009	YES
Diameter 2, in (rotated 90°)	0.00050	1.970	0.00025	0.015	YES

Maximum angle of departure must be $\leq$ 0.25°

Perpendicularity Tolerance Met? **YES**

Appendix C

Calculations

Earth Pressure

Backfill engineering strength parameters

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

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

Resisting earth pressures and walls restrained from movement - At-Rest Earth Pressure

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

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

$$\boxed{K_o = 0.47}$$

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

Unrestrained walls level backfill - Active Earth Pressure

For walls with tops unrestrained from movement and level backfill use active earth pressure.

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.

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

$$\boxed{K_{ar} = 0.31}$$

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

Flared Unrestrained Wingwalls - Active Earth Pressure

For walls with tops unrestrained from movement and sloping backfill, assume the back face of the wall does not interfere with the development of the failure surface and use Rankine theory.

Recalculate active earth pressure for 2H:1V sloping backfill

β = Angle of fill slope to the horizontal

$$\beta := \mathrm{atan}\left(\frac{1}{2}\right) = 27\,\mathrm{deg}$$

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

Bowles, Foundation Analysis and Design 5th ed. p. 601 Eq. 11-7

$$K_{ar\_slope} = 0.52$$

Pa is oriented at an angle of β to the vertical plane

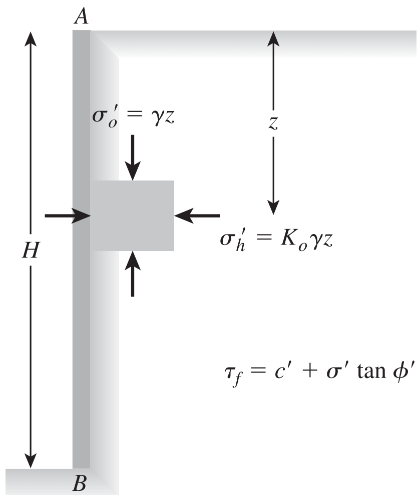


Figure 13.3
Earth pressure at rest

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

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

So,

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

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

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

where ϕ' = drained friction angle.

While designing a wall that may be subjected to lateral earth pressure at rest, one must take care in evaluating the value of K_o. Sherif, Fang, and Sherif (1984), on the basis of their laboratory tests, showed that Jaky's equation for K_o [Eq. (13.5)] gives good results when the backfill is loose sand. However, for a dense, compacted sand backfill, Eq. (13.5) may grossly underestimate the lateral earth pressure at rest. This underestimation results because of the process of compaction of backfill. For this reason, they recommended the design relationship

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

where γ_d = actual compacted dry unit weight of the sand behind the wall
$\gamma_{d(\min)}$ = dry unit weight of the sand in the loosest state (Chapter 3)

Substituting the preceding values into Eq. (13.16), we get

$$\sigma'_a = \gamma z \tan^2\left(45 - \frac{\phi'}{2}\right) - 2c' \tan\left(45 - \frac{\phi'}{2}\right) \tag{13.17}$$

The variation of σ'_a with depth is shown in Figure 13.7c. For cohesionless soils, $c' = 0$ and

$$\sigma'_a = \sigma'_o \tan^2\left(45 - \frac{\phi'}{2}\right) \tag{13.18}$$

The ratio of σ'_a to σ'_o is called the *coefficient of Rankine's active earth pressure* and is given by

$$K_a = \frac{\sigma'_a}{\sigma'_o} = \tan^2\left(45 - \frac{\phi'}{2}\right) \tag{13.19}$$

Again, from Figure 13.7b we can see that the failure planes in the soil make $\pm(45 + \phi'/2)$-degree angles with the direction of the major principal plane—that is, the horizontal. These are called potential *slip planes* and are shown in Figure 13.7d.

It is important to realize that a similar equation for σ_a could be derived based on the total stress shear strength parameters—that is, $\tau_f = c + \sigma \tan \phi$. For this case,

$$\sigma_a = \gamma z \tan^2\left(45 - \frac{\phi}{2}\right) - 2c \tan\left(45 - \frac{\phi}{2}\right) \tag{13.20}$$

13.5 Theory of Rankine's Passive Pressure

Rankine's passive state can be explained with the aid of Figure 13.8. *AB* is a frictionless wall that extends to an infinite depth (Figure 13.8a). The initial stress condition on a soil element is represented by the Mohr's circle *a* in Figure 13.8b. If the wall gradually is *pushed into the soil mass,* the effective principal stress σ'_h will increase. Ultimately, the wall will reach a situation where the stress condition for the soil element can be expressed by the Mohr's circle *b*. At this time, failure of the soil will occur. This situation is referred to as *Rankine's passive state.* The lateral earth pressure σ'_p, which is the major principal stress, is called *Rankine's passive earth pressure.* From Figure 13.8b, it can be shown that

$$\begin{aligned}\sigma'_p &= \sigma'_o \tan^2\left(45 + \frac{\phi'}{2}\right) + 2c' \tan\left(45 + \frac{\phi'}{2}\right)\\ &= \gamma z \tan^2\left(45 + \frac{\phi'}{2}\right) + 2c' \tan\left(45 + \frac{\phi'}{2}\right)\end{aligned} \tag{13.21}$$

The water also contributes lateral pressure and has $K_a = K_p = 1$ since $\phi_w = 0°$. Thus,

$$p_w = \gamma_w z_w = 9.807(3.5) = \mathbf{34.3}\text{ kPa}$$

These pressure values are plotted on Fig. E11-2*b* so the several pressure areas can be numerically integrated to obtain the total wall force. By using triangles and rectangles as shown, the total wall force is the sum from the several areas and the forces act through the centroids of the areas as shown so that we can easily sum moments about the base to obtain

$$R\bar{y} = \sum P_i y_i$$

$$P_1 = 30.7(3.5) = 107.5\text{ kN} \qquad y_i = 3.5 + \frac{3.5}{2} = 5.25\text{ m}$$

$$P_2 = 17.7\left(\frac{3.5}{2}\right) = 31.0\text{ kN} \qquad y_2 = 3.5 + \frac{3.5}{3} = 4.67\text{ m}$$

$$P_3 = 52.5(3.5) = 183.8\text{ kN} \qquad y_3 = \frac{3.5}{2} = 1.75\text{ m}$$

Include water with P_4 since both areas are triangles:

$$P_4 = (34.3 + 11.0)\left(\frac{3.5}{2}\right) = 79.3\text{ kN} \qquad y_4 = \frac{3.5}{3} = 1.17$$

$$R = \sum P_i = 107.5 + 31.0 + 183.8 + 79.3 = \mathbf{401.6}\text{ kN}$$

Now sum the moments for $\bar{y}$:

$$401.6\bar{y} = 107.5(5.25) + 31.0(4.67) + 183.8(1.75) + 79.3(1.17)$$

$$\bar{y} = \frac{1123.6}{401.6} = \mathbf{2.80}\text{ m} \qquad \text{(above wall base)}$$

////

11-5 RANKINE EARTH PRESSURES

Rankine (ca. 1857) considered soil in a state of plastic equilibrium and used essentially the same assumptions as Coulomb, except that *he assumed no wall friction or soil cohesion.* The Rankine case is illustrated in Fig. 11-7 with a Mohr's construction for the general case shown in Fig. 11-8. From Fig. 11-8 we can develop the Rankine active and passive pressure cases by making substitution of the equation for r (shown on the figure) into the equations for EF (and FG) (also shown on the figure). Then substitution into the expression for K'_a (with OB canceling and using $\sin^2\beta = 1 - \cos^2\beta$) gives the pressure ratio acting parallel to backfill slope β as

$$\boxed{K'_a = \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} \qquad (11\text{-}7)}$$

We note that the *horizontal component* of active earth pressure is obtained as

$$\sigma_{a,\text{hor}} = \sigma_a \cos\beta \qquad (= OE\cos\beta = OA \text{ of Fig. 11-8}b)$$

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. Wyllie, Duncan C, "Foundations on Rock", Second Edition, 2009.
4. "The Hoek-Brown Failure Criterion - A 1988 Update", E. Hoek and E.T. Brown

A. Design Bedrock Properties

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-IFB-101;R2 recovered between 21.68 - 22.03 feet below roadway surface.
q_{uc} = 1100 psi.

Use q_u = 11,000 psi or 1584 ksf

$q_{uc} := 11000 \cdot psi$

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

$q_{u1=}$ 11,000 psi or 1584 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 to rough, open; Relative Rating = 6

5. Groundwater conditions

General condition is water under moderate pressure; Relative Rating = 4

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

Breaks along bedding are steep, joints are low to moderate dipping use Relative Rating = -7

ADJUSTED RMR

$$RMR := 7 + 8 + 10 + 6 + 4 - 7$$

$$RMR = 28$$

Determine Rock Type for LRFD Table 10.4.6.4.-4

Rock Type - B = Lithified Argrillaceous 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 = 28 is Class No. IV and described as Poor rock.

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

$RMR = 28$

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

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

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

m_i = m for intact rock

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

$m_i := 10$

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

$m = 0.058$

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

$s = 0.0000061$

D. Nominal and Factored Bearing Resistance of Bedrock

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

$C_{f1} := 1.0$ Conservative selection of C_{fl} = 1.0 for L/B>6

Nominal Bearing Resistance (Wyllie)

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

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

$q_{n1} = 23 \cdot ksf$

Factored Bearing Resistances

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

$\phi_{bc} := 0.45$

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

$q_{r1} = 11 \cdot ksf$ Strength Limit State

Recommendation: Use 11 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.

$\phi_{ree} := 0.8$

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

$q_{r1} = 19 \cdot ksf$ Extreme Limit State

Recommendation: Use 19 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.

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

$$q_{n1} = 23 \cdot ksf$$

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

$\phi_r := 1.0$ $q_{1nominal} := 20 \cdot ksf$

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

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

$q_{factored} = 20 \cdot ksf$ Service Limit State

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

B. 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

2012

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

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

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

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

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

in which:

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

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

where:

τ = the shear strength of the rock mass (ksf)

ϕ'_i = the instantaneous friction angle of the rock mass (degrees)

q_u = average unconfined compressive strength of rock core (ksf)

σ'_n = effective normal stress (ksf)

m, s = constants from Table 10.4.6.4-4 (dim)

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

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

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

2012

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

Rock Type

A = Carbonate rocks with well developed crystal cleavage—*dolomite, limestone and marble*
B = Lithified argrillaceous rocks—*mudstone, siltstone, shale and slate (normal to cleavage)*
C = Arenaceous rocks with strong crystals and poorly developed crystal cleavage—*sandstone and quartzite*
D = Fine grained polymineralic igneous crystalline rocks—*andesite, dolerite, diabase and rhyolite*
E = Coarse grained polymineralic igneous & metamorphic crystalline rocks—*amphibolite, gabbro gneiss, granite, norite, quartz-diorite*

Rock Quality	Constants	A	B	C	D	E
INTACT ROCK SAMPLES Laboratory size specimens free from discontinuities. CSIR rating: *RMR* = 100	*m* *s*	7.00 1.00	10.00 1.00	15.00 1.00	17.00 1.00	25.00 1.00
VERY GOOD QUALITY ROCK MASS Tightly interlocking undisturbed rock with unweathered joints at 3–10 ft CSIR rating: *RMR* = 85	*m* *s*	2.40 0.082	3.43 0.082	5.14 0.082	5.82 0.082	8.567 0.082
GOOD QUALITY ROCK MASS Fresh to slightly weathered rock, slightly disturbed with joints at 3–10 ft CSIR rating: *RMR* = 65	*m* *s*	0.575 0.00293	0.821 0.00293	1.231 0.00293	1.395 0.00293	2.052 0.00293
FAIR QUALITY ROCK MASS Several sets of moderately weathered joints spaced at 1–3 ft CSIR rating: *RMR* = 44	*m* *s*	0.128 0.00009	0.183 0.00009	0.275 0.00009	0.311 0.00009	0.458 0.00009
POOR QUALITY ROCK MASS Numerous weathered joints at 2 to 12 in.; some gouge. Clean compacted waste rock. CSIR rating: *RMR* = 23	*m* *s*	0.029 3×10^{-6}	0.041 3×10^{-6}	0.061 3×10^{-6}	0.069 3×10^{-6}	0.102 3×10^{-6}
VERY POOR QUALITY ROCK MASS Numerous heavily weathered joints spaced <2 in. with gouge. Waste rock with fines. CSIR rating: *RMR* = 3	*m* *s*	0.007 1×10^{-7}	0.010 1×10^{-7}	0.015 1×10^{-7}	0.017 1×10^{-7}	0.025 1×10^{-7}

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

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

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

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

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

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

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

Disturbed rock masses :

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

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

Undisturbed or interlocking rock masses:

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

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

where

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

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

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

Limitations on using failure criterion

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

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

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

Wylie, Foundations on Rock, 2nd ed.

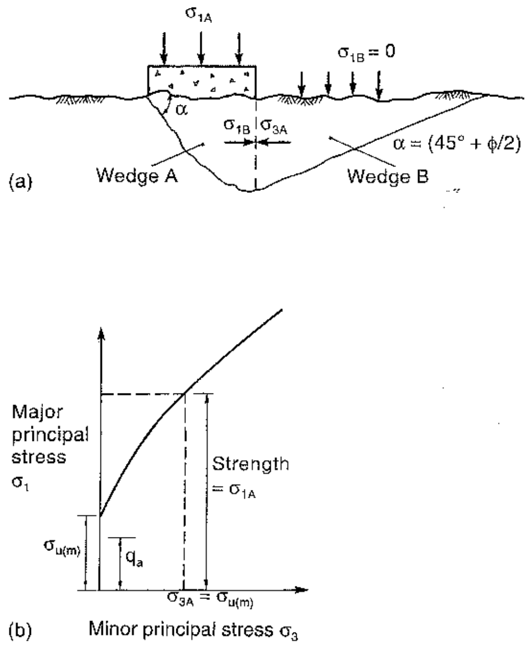


Figure 5.3 Analysis of bearing capacity of fractured rock: (a) active A and passive B wedges in foundation; and (b) curved rock mass strength envelope. Allowable bearing pressure = q_a, strength of bearing rock = σ_{1A}, factor of safety $FS = \sigma_{1A}/q_a$.

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

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

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

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

The factor C_{f1} is applied to the calculated allowable bearing pressure to account for the shape of the foundation and has the values given in Table 5.4 (Sowers, 1970).

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

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

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

5.2.3 Recessed footings

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

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

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

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 = 1800 $d := 74.5in$

Depth of Frost Penetration $d = 75 \cdot in$ $d = 6.2 \cdot ft$

Method 2 - ModBerg Software

Examine foundations placed on coarse grained fill soils

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

--- ModBerg Results ---

Project Location: Farmington, Maine
Air Design Freezing Index = 2023 F-days
N-Factor = 0.80
Surface Design Freezing Index = 1618 F-days
Mean Annual Temperature = 41.2 deg F
Design Length of Freezing Season = 145 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	95.4	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 = 7.95 ft = 95.4 in.

Recommendation: 6.2 feet for design of foundations constructed on coarse grained soils

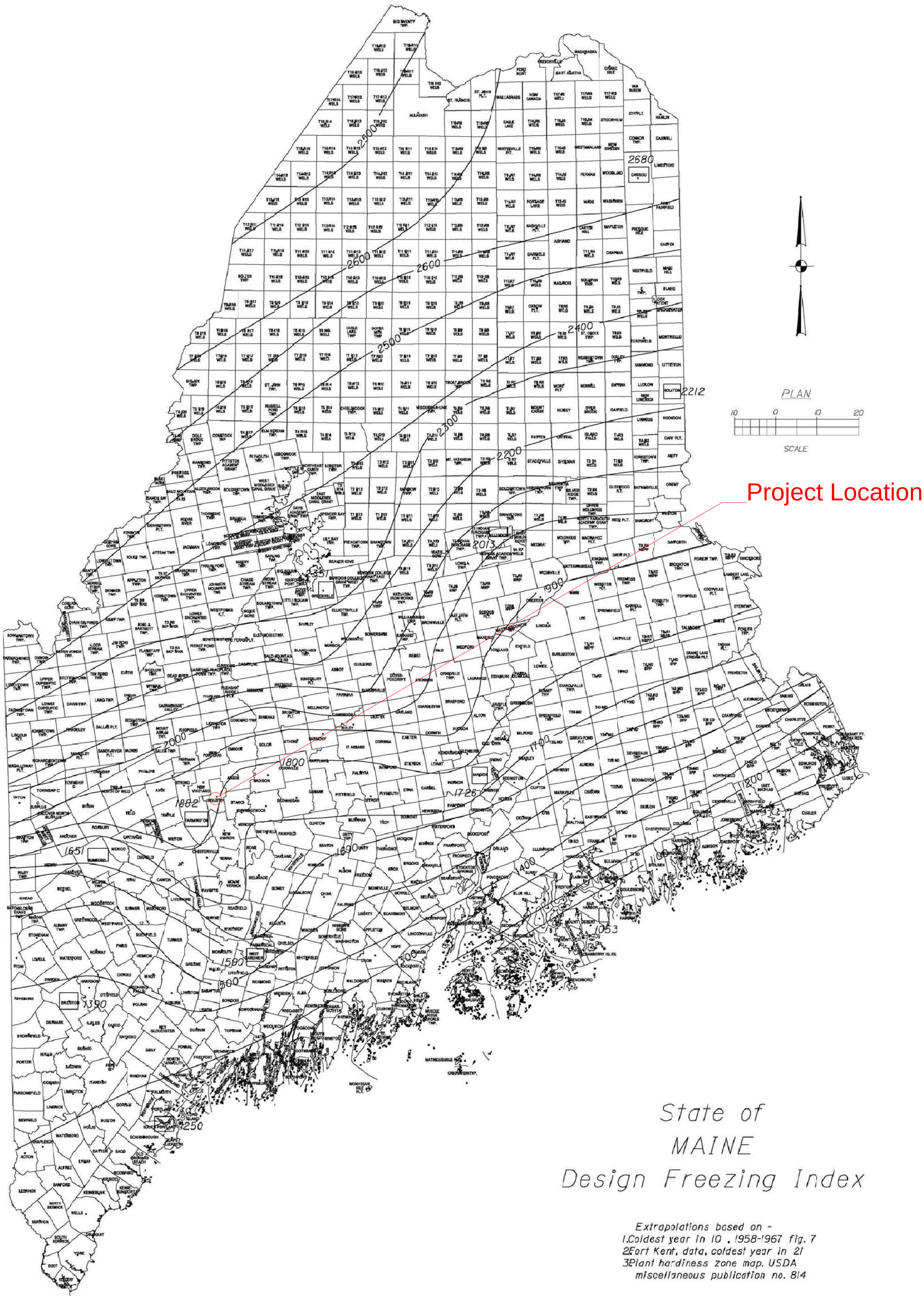


Figure 5-1 Maine Design Freezing Index Map

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