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GEOTECHNICAL DESIGN REPORT
LISBON CENTER BRIDGE NO. 5007 OVER THE SABATTUS RIVER

MAINE DOT WIN 18970.00
LISBON, MAINE

Prepared for:
Maine Department of Transportation
Augusta, Maine

March 2018
09.0025944.00

Prepared by:
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VIA EMAIL

March 27, 2018
File No. 09.0025944.00

Ms. Laura Krusinski, P.E.
Maine Department of Transportation
16 State House Station
Augusta, Maine 04333-0016

Re: Geotechnical Design Report
Lisbon Center Bridge No. 5007
MaineDOT WIN 18970.00
Lisbon, Maine

Dear Laura:

We are pleased to provide this Final Geotechnical Design Report for MaineDOT Bridge No. 5007 over the Sabattus River in Lisbon, Maine. Our work was completed in accordance with Assignment Letters No. 8 and 16 (dated June 2, 2017 and March 9, 2018) associated with the Bridge Program Multi-PIN Project Contract Number 20150608000000000793 between MaineDOT and GZA dated July 22, 2015, which incorporates GZA's Proposal No. 09.P000037.18, dated June 1, 2017, and the attached Limitations included in Appendix A of this report.

It has been a pleasure serving the Maine Department of Transportation team on this project. If you have any questions regarding the report, or if we can provide further assistance, please do not hesitate to contact the undersigned.

Very truly yours,

GZA GEOENVIRONMENTAL, INC.

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Project Engineer

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APPENDICES

1.0 INTRODUCTION

GZA GeoEnvironmental, Inc. (GZA) has prepared this geotechnical design report for the proposed replacement of Maine Department of Transportation (MaineDOT) Lisbon Center Bridge No. 5007 which carries Mill Street over the Sabattus River in Lisbon, Maine. Our services were provided in accordance with Assignment Letter No. 8 and No. 16 (dated June 2, 2017 and March 9, 2018) associated with the Bridge Program Multi-PIN Project Contract Number 20150608000000000793 between MaineDOT and GZA dated July 22, 2015, which incorporates GZA's Proposal No. 09.P000037.18, dated June 1, 2017. This report is subject to the *Limitations* included in **Appendix A**.

GZA previously presented two preliminary design memoranda for the project, dated June 26, 2017 and July 25, 2017. The evaluations and recommendations presented in this report supersede our previous preliminary memoranda.

1.1 BACKGROUND

The existing Lisbon Center Bridge No. 5007 carries Mill Street over the Sabattus River in Lisbon, at the location shown on **Figure 1**. The existing bridge was constructed in 1926 and consists of a 58-foot-long, two-span bridge with a concrete T-Beam superstructure. The bridge is supported on a concrete center pier and concrete abutments supported by spread footings bearing on bedrock.

The replacement bridge is planned to be constructed on the existing alignment. The replacement bridge alternative selected in the Preliminary Design Report (PDR) is an 85-foot-long, single-span bridge consisting of NEBT precast concrete beams and composite concrete deck superstructure with semi-integral abutments bearing on tremie seals and bedrock. The proposed bridge layout is shown on **Figure 2**. A road closure and traffic detour will be implemented during bridge construction.

An existing dam, the Farnsworth Mill Dam, is located approximately 60 feet downstream from the bridge. The dam is approximately 10 feet high and 140 feet long, and it has a 10-foot-wide section that was previously breached to reduce the difference in water levels directly upstream and downstream of the dam. We understand the dam may be completely removed in the future.

1.2 OBJECTIVES AND SCOPE OF SERVICES

The objectives of our work were to evaluate subsurface conditions and provide final design geotechnical engineering recommendations and construction considerations for bridge replacement. To meet these objectives, GZA completed the following Scope of Services:

- Reviewed boring logs including rock core sample review to evaluate subsurface conditions;
- Conducted a site visit to observe the exposed conditions and understand how the site conditions could affect design and constructability;
- Reviewed a laboratory testing program conducted by MaineDOT to evaluate engineering properties of the site soils and bedrock;

- Conducted geotechnical engineering analyses for refinement of subsurface profiles and engineering properties; evaluation of bedrock properties relative to stability and foundation support; settlement of proposed approach fills; and bearing resistance of the abutment footings bearing on rock;
- Developed geotechnical engineering recommendations including foundation design recommendations for footings on rock, lateral earth pressures and seismic design parameters;
- Prepared two preliminary geotechnical design memoranda summarizing our findings and preliminary design recommendations; and
- Prepared this final report summarizing our findings and design recommendations.

2.0 SUBSURFACE EXPLORATIONS

2.1 SUBSURFACE EXPLORATIONS

On July 19 and 20, 2016, MaineDOT drilled and logged two test borings (BB-LSR-101 and -102). One boring was drilled behind each abutment at the locations shown on the Boring Location Plan (prepared by MaineDOT – reviewed by GZA), **Figure 2**. The borings were drilled using a track-mounted drill rig to depths of approximately 21 to 28 feet below ground surface (bgs) and were terminated approximately 9 to 10 feet into bedrock. The as-drilled boring locations and elevations were included on the logs provided to GZA by MaineDOT.

The borings were drilled using 3-inch casing and drive-and-wash drilling techniques. Standard penetration testing (SPT) and split-spoon sampling were performed at 5-foot typical intervals in the overburden using a 24-inch-long, 1-3/8-inch inside-diameter sampler, driven with an automatic hammer with a rated hammer efficiency factor of 0.94. Bedrock cores were obtained using NQ2 wire-line coring equipment in each test boring.

MaineDOT developed draft logs of the borings and provided them to GZA. GZA reviewed and edited the logs to reflect engineering review of draft logs, laboratory soil test results and bedrock classifications. The final logs are presented in **Appendix B**.

2.2 REVIEW OF ROCK CORE

GZA requested access to the rock core samples in order to make an independent assessment of the rock type and characteristics. After receiving approval from the MaineDOT Geotechnical Group, a GZA engineer visited MaineDOT's laboratory in Bangor, reviewed the available rock core specimens, and prepared an independent description of core samples from borings BB-LSR-101 and BB-LSR-102. The GZA observations are provided on the logs in **Appendix B**. GZA also took wet and dry photographs of the rock core specimens, which are presented in **Appendix D**.

3.0 LABORATORY TESTING

MaineDOT completed a laboratory testing program consisting of water content and gradation analysis/AASHTO Classification/Frost Classification assessments on three soil samples, hydrometer analysis on one of the gradation samples, and an Atterberg limits test on one soil sample. MaineDOT retained GeoTesting Express of Acton, Massachusetts to complete two (2) unconfined compressive strength / secant modulus tests on bedrock core samples.

Results of the testing are included in **Appendix C**.

4.0 SUBSURFACE CONDITIONS

4.1 SURFICIAL AND BEDROCK GEOLOGY

Based on available surficial geology mapping[1], the surficial unit along the river at the bridge alignment consists of stream (river) alluvium, described as fine sand and silt with some gravel. Marine nearshore deposits are mapped adjacent to the river alluvium near the bridge alignment, which are described as gravel, sand and mud deposited in nearshore or shallow marine environments.

Bedrock in the vicinity of the site is mapped[2] as the Vassalboro formation. The Vassalboro formation is characterized as occasionally massive, medium-gray quartz-plagioclase-biotite granofels and medium gray calc-silicate granofels.

4.2 SUBSURFACE PROFILE

Two soil units were encountered and interpreted by MaineDOT in the test borings overlying bedrock: Fill and River Alluvium Deposit. Approximately 4 to 5 inches of asphalt pavement was encountered in the borings. The thicknesses and generalized descriptions of the soil units are presented in the following table, in descending order from existing ground surface. Detailed descriptions of the materials encountered at specific locations are provided in the boring logs in **Appendix B**. The subsurface conditions are also shown in relation to the bridge alignment on an interpretive subsurface profile presented in **Figure 2**.

[1] Lisbon Falls North Quadrangle, Open-File No. 03-14 2003, Maine Geological Survey
[2] Bedrock Geology of the Lewiston Quadrangle, Maine, Open File No. 83-4, 1983, Maine Geological Survey

GENERALIZED SUBSURFACE CONDITIONS		
Soil Unit	**Approximate Encountered Thickness (ft)**	**Generalized Description**
Fill	11 to 13	Upper Fill (6 to 9 ft): Brown to yellow-brown, moist to wet, loose to medium dense SAND, trace to some Gravel, trace to some Silt, with brick fragments (USCS: SP, SP-SM) Lower Fill (4 to 5 ft): Grey, wet, loose Silty fine SAND to medium stiff SILT, trace Gravel, with timber fragments (USCS: SM, ML). • MaineDOT Frost Classification = 0-III • Encountered in both approach borings
River Alluvium Deposit	5	Grey, wet, medium dense, Sandy GRAVEL, trace Silt (USCS: GW-GM). • Encountered in BB-LSR-101 only
Top of Bedrock Elevation	Encountered Top of Rock: Approximate El. 137.1 (BB-LSR-101) to El. 144.2 (BB-LSR-102)	

GZA did not observe the soil samples from borings. We relied on classifications made by MaineDOT combined with laboratory test results for our description of the soil.

4.2.1 Bedrock

Bedrock was cored in each test boring and was primarily described as hard, fresh to slightly weathered, white to black, medium to coarse grained, GNEISS. Joints are primarily low angle, rough to smooth, planar to undulating, very close to close, fresh to discolored and tight to moderately wide. The deeper core runs at each location were observed to have an increased presence of quartz. Based on GZA's review of the rock core specimens on June 15, 2017, the quartz-rich materials are interpreted to consist of pink, white and black, hard, coarse-grained PEGMATITE intrusions. The encountered Pegmatite intrusions were less than 2 feet thick. The Rock Quality Designation (RQD) in the core runs ranged from 27 to 87 percent.

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

SUMMARY OF BEDROCK STRENGTH TEST RESULTS							
Boring	**Approximate Depth Below Existing Ground (ft bgs)**	**Approximate Depth below Top of Rock (ft bgs)**	**Approximate Elevation @ Top of Sample (ft NAVD 88)**	**Unconfined Compressive Strength (psi)**	**Secant Modulus @ 50% of Failure Stress (ksi)**	**Unit Weight (pcf)**	**Rock Type**
BB-LSR-101	19.8-20.4	1.0	136.1	9,593	4,260	169	GNEISS
BB-LSR-102	12.1-12.5	0.8	143.4	9,900	3,710	171	GNEISS

Wet and dry photographs of the core boxes are presented in **Appendix D**.

4.2.2 Groundwater

Groundwater levels were not recorded on the boring logs. The soil samples were described as wet at a depth of approximately 5 feet; therefore, it is possible that groundwater is as high as El. 151. Considering the presence of the river adjacent to the approaches, groundwater levels in the approaches are anticipated to be at or above the river level. Fluctuations in groundwater levels will occur due to variations in season, precipitation, river level and other factors.

5.0 ENGINEERING EVALUATIONS

5.1 GENERAL

GZA has conducted geotechnical engineering evaluations in accordance with 2014 AASHTO LRFD Bridge Design Specifications, 7th Edition, with Interims (herein known as AASHTO) and the MaineDOT Bridge Design Guide, 2014 Edition (MaineDOT BDG). Supporting calculations developed by GZA for the project are attached in **Appendix E** of this report.

5.2 APPROACH EMBANKMENTS

The existing approaches are approximately 15 and 12 feet high at Abutments 1 and 2, measured from the existing roadway to the riverbed. At Abutment 1 and its wingwalls, the new substructures will be located on the landside of the proposed abutments. Therefore, maximum fill height will be approximately equal to the raised roadway profile; approximately 2.5 feet. At Abutment 2, the wingwalls extend outside of the current approach embankment footprint. Therefore, maximum fill heights will be approximately 3 feet and 10 feet at the proposed left and right wingwalls, respectively.

We anticipate that the fill will be constructed over loose to medium dense sand to medium stiff silt. In our experience, total settlement will likely be on the order of ½ inch or less where the new fill height is about 3 feet or less. Where the new fill height is 10 feet, the estimated settlement is approximately 1 inch. Settlement will occur rapidly during embankment construction. Therefore, we do not anticipate that significant (greater than ½-inch) post-construction settlement will be observed in the paved approaches. Based on the proposed geometry and anticipated subsurface conditions, it is our opinion that the potential for global instability of the proposed embankments is low.

5.3 SEISMIC DESIGN CONSIERATIONS

Seismic site class was evaluated in accordance with the 2014 AASHTO LRFD, along with consideration of the *2011 AASHTO Guide Specifications for LRFD Bridge Design* (Seismic Guide Specification).

The planned foundation type consists of spread footings bearing on bedrock. Commentary Article C3.4.2.2 of the Seismic Guide Specification indicates that the ground motion at the abutment controls transfer of earthquake motion to relatively short bridges, and the site classification should be determined at the base of the approach fill. For this project, the approach fills are generally bearing directly on or near bedrock, and the foundations will bear directly on sound bedrock. Considering the high strength of the rock and our experience with similar rock types in Maine and New Hampshire, we anticipate that the shear wave velocity of rock that will support the bridge exceeds 5,000 feet per second. Therefore, the bridge should be assigned to Site Class A.

5.4 EVALUATION OF FOUNDATIONS

5.4.1 Foundation Type Assessment

Foundation types considered for the proposed bridge included integral abutments supported by driven piles, socketed H-piles, or spun piles; and semi-integral abutments[3] supported on spread footings with tremie seals. Driven piles are not feasible due to the shallow bedrock depth, and rock-socketed H-piles would likely be more expensive than spun piles. Therefore, the two foundation types considered in greater detail included spun piles supporting integral abutments and spread footings supporting semi-integral abutments.

We understand a conventional spread footing/tremie seal on rock is more economical for Abutment 2, where rock is shallower, and the cost is comparable between spun piles and a cofferdam-tremie seal-spread footing arrangement at Abutment 1.

There is a potential for the downstream dam to be removed in the future, which could alter the river flow and potentially initiate scour of soil near the abutments. A footing bearing on rock is not susceptible to performance concerns if scour occurs, but there is a potential for reduction or loss of soil support for spun piles. Therefore, spread footings with tremie seals bearing on competent bedrock are the preferred alternative for abutment support.

5.4.2 Load and Resistance Factors

AASHTO LRFD load factors should be applied to horizontal earth pressure (EH), vertical earth pressure (EV), earth surcharge (ES), live load surcharge (LS) loads, and components and attachments (DC) loads using the load factors for permanent loads (γ_p) provided in LRFD Tables 3.4.1-2 for strength and extreme limit state foundation design. For service limit state, a load factor of 1.0 should be applied to these loads.

Recommended LRFD resistance factors for strength limit state design of the bedrock-bearing foundations were derived from LRFD Table 10.5.5.2.2-1, and are presented in the following table.

RESISTANCE FACTORS – STRENGTH		
Foundation Resistance Type	**Method/Condition**	**Resistance Factor (ϕ_τ)**
Bearing	Footing on Rock	0.45
Sliding	Tremie Concrete on Rock[1]	0.8

[1] Sliding resistance factor for concrete on rock or concrete is taken as equal to footing on sand.

Resistance factors for service and extreme limit state design should be taken as 1.0.

5.5 SPREAD FOOTINGS BEARING ON ROCK

Nominal and factored bearing resistances were calculated for footings bearing on rock using the Rock Mass Rating (RMR)-based empirical correlation presented in "Foundations on Rock," by Duncan Wyllie.

[3] The superstructure was planned to consist of semi-integral abutments at the time of foundation type selection, but has since been changed to slab over backwall. Foundation type considerations are generally the same between these two superstructure types.

RMR was evaluated in accordance with Table 10.4.6.4-1 of the 2012 AASHTO LRFD Bridge Design Specifications, 6th Edition (AASHTO). The current (7th Edition) of the AASHTO Design Specifications does not include the Rock Mass Rating (RMR) formulation included in the previous version (6th Edition). However, Articles C10.4.6.4 and 10.6.2.6.2 of the 7th Edition refer to RMR-based design procedures for footings on rock, so the 6th Edition methodology was followed.

GZA used bedrock data obtained in test borings BB-LSR-101 and BB-LSR-102 to develop foundation design parameters at the abutments. The bedrock properties used in the bearing resistance evaluation are presented below:

DESIGN BEDROCK PROPERTIES FOR BEARING RESISTANCE EVALUATION					
Rock Type	**RQD (percent)**	**Unconfined Compressive Strength (ksi)**	**Rock Mass Rating (RMR)**	**m**	**s**
Gneiss	29 to 44	9.0	42	0.4	0.000065

Based on the available data and the stated methodology, the calculated nominal bearing resistance is 85 kips per square foot (ksf). This provides a factored bearing resistance of 38 ksf for the strength loading condition.

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

5.5.1 Lateral Earth Pressure

Thermal expansion of the bridge will cause the superstructure backwall (end diaphragm) to move toward the backfill, which will result in earth pressures ranging from at-rest to passive earth pressure. Therefore, the end diaphragms should be designed for passive earth pressure. The semi-integral abutments and wingwalls will be free to rotate and therefore should be designed for active earth pressure. The material properties will be controlled by the backfill material, which is anticipated to consist of BDG Type 4 soil. Soil properties for Type 4 soil are provided in **Section 6.2** of this report.

5.5.2 Frost Penetration

Fill soils are anticipated to be present at the abutments, as imported backfill. Based on the MaineDOT BDG, Section 5.2.1, the Freezing Index for the site is 1,400, and with low to moderate moisture content (±15%) soils, the estimated depth of frost penetration is 6 feet. However, because the foundations consist of footings on sound bedrock, there is no requirement for footing embedment.

6.0 RECOMMENDATIONS

6.1 EMBANKMENT DESIGN CONSIDERATIONS

Embankment side slopes should generally be designed with MaineDOT typical slope angles of 2H:1V or less, and should be provided with loam and seed for permanent erosion protection.

A maximum slope inclination of 1.75H:1V may be used for riprap-protected slopes. Riprap should be a minimum of 3 feet thick for plain riprap and 4 feet thick for heavy riprap and should be underlain by a minimum 12-inch thick protective aggregate cushion and non-woven Erosion Control Geotextile in accordance with MaineDOT Standard Details 610(02) and/or 610(03).

6.2 SEISMIC DESIGN

The United States Geological Survey online Design Maps Tool was used to develop parameters for bridge design. Based on the site coordinates, the software provided the recommended AASHTO Response Spectra (Site Class A) for a 7 percent probability of exceedance in 75 years. These results are summarized for the site as follows:

SITE CLASS A SEISMIC DESIGN PARAMETERS	
Parameter	**Design Value**
Fpga	0.8
Fa	0.8
Fv	0.8
As (Period = 0.0 sec)	0.067 g
SDs (Period = 0.2 sec)	0.136 g
SD1 (Period = 1.0 sec)	0.036 g

Per AASHTO Article 4.7.4.2, single span bridges need not be analyzed for seismic loads, but the minimum requirements for superstructure connections and support lengths as specified in AASHTO Articles 4.7.4.4 and 3.10.9 apply.

6.3 SPREAD FOOTING DESIGN

- The new abutments may be supported on spread footing foundations bearing on sound, intact bedrock free of all loose soil and rock material. Footings designed to bear on intact bedrock should be designed for a nominal bearing resistance, q_n, of 85 ksf. At the strength limit state, footings should be designed for a maximum factored bearing resistance of 38 ksf. A bearing resistance of 38 ksf should be used for service limit state design.
- Spread footings founded on bedrock should be checked for eccentricity with LRFD Section 10.6.3.3. Eccentricity of the footing reaction at the strength limit state should be limited such that the resultant reaction on the base of the footing is no further than 0.45 B from the centerline of the footing, where B is the principal dimension of the footing perpendicular to the axis of rotation.
- For foundations bearing on bedrock, we recommend that sliding resistance be assessed using a nominal sliding resistance coefficient (tan δ) equal to 0.7 for cast-in-place concrete on sound rock. Therefore, the nominal sliding resistance between footings and bedrock subgrades is equal to the vertical force multiplied by 0.7. The factored sliding resistance coefficient is 0.56 for the Strength Limit State.
- Anchoring, doweling, benching or other means of improving sliding resistance are recommended at locations where the prepared bedrock surface is steeper than 4H:1V in any direction.

6.4 ABUTMENT AND WINGWALL DESIGN

- Backfill in a zone between new abutments and a 1.5H:1V plane extending up and back from the bottom of the abutment to the pavement subgrade should consist of MaineDOT 703.19 Granular Borrow for Underwater Backfill, BDG Type 4 soil. Recommended soil properties for Type 4 soils to be used as backfill are as follows:
  - Internal Friction Angle of Soil = 32°
  - Soil Total Unit Weight = 125 pcf
  - Coefficient of Passive Earth Pressure, K_p (use for design of end diaphragms):
    - For a ratio of lateral movement to backwall height (y/H) equal to or exceeding 0.005, use Coloumb theory coefficient, K_p= 6.73;
    - For a ratio of y/H significantly less than 0.005, use Rankine theory coefficient, K_p= 3.25;
  - Coefficient of Active Earth Pressure, K_a=0.31 (use for design of abutments and wingwalls):
- Live load surcharge should be applied as a uniform lateral surcharge pressure using the equivalent fill height (Heq) values developed in accordance with AASHTO Article 3.11.6.4 based on the abutment/wingwall height and distance from the wall back face to the edge of traffic.
- Foundation drainage should be provided in accordance with Section 5.4.1.9 of the BDG.
  - We recommend the use of French drains on the uphill side of abutments and wing walls to prevent buildup of differential hydrostatic pressure. Foundation drains should be sloped to drain by gravity and should daylight through weep holes in the abutments.
  - Due to the lack of vertical space in the proposed backwall configuration between the top of footing and bottom of backwall, the standard 2-foot square French drain under the backwall at Abutment 2 will need to be relocated. The drain should sit directly behind the backwall, over the proposed footing. The drain will tie into the weep holes by filling the space between the top of the footing, the bottom of the backwall and the French drain with crushed stone. A sketch showing the recommended detail is presented in **Appendix E**.

7.0 CONSTRUCTION CONSIDERATIONS

This section describes geotechnical-related issues that have the potential to impact design and cost considerations for bridge construction.

7.1 EXCAVATION AND DEWATERING

Excavations for abutment foundations will extend up to about 20 and 14 feet below existing grade at Abutments 1 and 2, respectively, to expose bedrock, corresponding to depths of approximately 14 and 7 feet below the elevation of wet soil in the borings (El. 151), and approximately 12 and 5 feet below the Q1.1 river elevation (El. 149.28). These excavation depths would typically require a four-sided, internally-braced sheet pile cofferdam for excavation support. The new abutments are partially located on the land side of the existing abutments. It may be feasible to utilize portions of the existing abutments to reduce the sheet pile support requirements if justified by design calculations.

The contractor should be responsible for design of all temporary cofferdam structures. Design should be completed by a professional engineer registered in the State of Maine. If existing foundations will be relied on for support, shop drawings describing the proposed construction sequence and design calculations should show the suitability of the foundations to serve the intended use.

Dewatering considerations will be related to the river level and groundwater level at the time of construction. If river levels are several feet above bedrock elevations, we anticipate that dewatering will be impractical, and the foundations will be constructed on tremie seals placed in the wet. Due to the greater depth to bedrock, this condition is judged by GZA to be more likely for Abutment 1 than for Abutment 2. It may be feasible to dewater cofferdam(s) by pumping from sumps placed within the excavations in low water conditions. This condition is judged by GZA to be more likely for Abutment 2 than for Abutment 1, since the depth to bedrock is less there. The contractor should be responsible for controlling groundwater, surface runoff, infiltration and water from all other sources by methods. Discharge of pumped groundwater and river water should comply with all local, State, and federal regulations.

7.2 SUBGRADE PREPARATION

If necessary, bedrock bearing surface preparation may be conducted in the wet. We anticipate water depths up to 14 feet, and that the bedrock surface will be variable in terms of elevation, slope and localized weathering. A combination of standard excavation equipment, hydraulic hoe-ramming equipment, and/or air lifting may be needed to remove the overburden and fractured/weathered rock. The existing foundations, all soil, and loose, decomposed, highly weathered and fractured bedrock should be removed from the footing bearing surface prior to placement of tremie seals. The prepared bearing surfaces should be checked by depth probing in conjunction with visual means such as diver and/or remotely operated vehicle (ROV) video inspection. A Special Provision should be prepared to define the project-specific requirements for subgrade preparation and quality assurance/quality control.

The Geotechnical Engineer and Designer should be provided cross-sections showing the prepared rock surface geometry prior to placement of concrete to evaluate whether benching, doweling, or subfooting reinforcement are needed for that foundation location. If the exposed bedrock surface is steeper than 4 horizontal to 1 vertical (4H:1V), then anchoring, doweling, benching or other means should be employed to improving sliding resistance.

7.3 REUSE OF ON-SITE MATERIALS

Based on the test boring results, both of the fill samples tested had greater than 20 percent passing the No. 200 sieve, indicating the fill may not meet MaineDOT specifications for Granular Borrow and/or Granular Borrow for Underwater Backfill and is unsuitable for use as structural backfill. The material is considered suitable for use as Common Borrow.

If the contractor wishes to reuse excavated material as embankment fill or in other areas, we recommend that the proposed material be stockpiled and tested for grain size distribution. Stockpiled materials meeting the appropriate MaineDOT specifications may be reused on the project.

FIGURES

SOURCE : THIS MAP CONTAINS THE ESRI ARCGIS ONLINE USA TOPOGRAPHIC MAP SERVICE, PUBLISHED DECEMBER 12, 2009 BY ESRI ARCIMS SERVICES AND UPDATED AS NEEDED. THIS SERVICE USES UNIFORM NATIONALLY RECOGNIZED DATUM AND CARTOGRAPHY STANDARDS AND A VARIETY OF AVAILABLE SOURCES FROM SEVERAL DATA PROVIDERS. THIS MAP ALSO CONTAINS THE ESRI ARCGIS ONLINE USA COUNTIES WHICH PROVIDES DETAILED BOUNDARIES THAT ARE CONSISTENT WITH THE TRACT, BLOCK GROUP, AND STATE DATA SETS AND ARE EFFECTIVE AT REGIONAL AND STATE LEVELS.

0 1,000 2,000 4,000 6,000

SCALE IN FEET

Data Supplied by :

PROJ. MGR.: BMC
DESIGNED BY: BMC
REVIEWED BY: ARB
OPERATOR: BMC

DATE: 06-26-2017

LOCUS PLAN

LISBON CENTER BRIDGE NO. 5007, MAINEDOT WIN 18970.00
LISBON, ME

JOB NO.
09.0025944.00

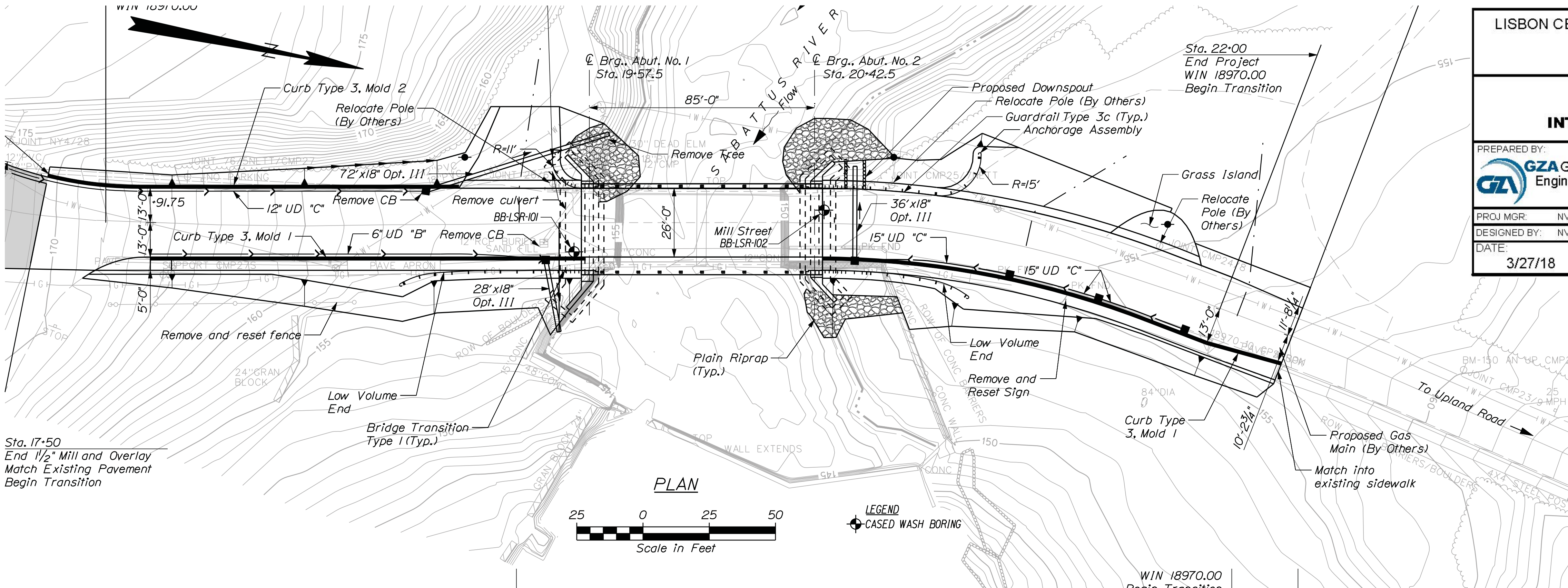
FIGURE NO.
1

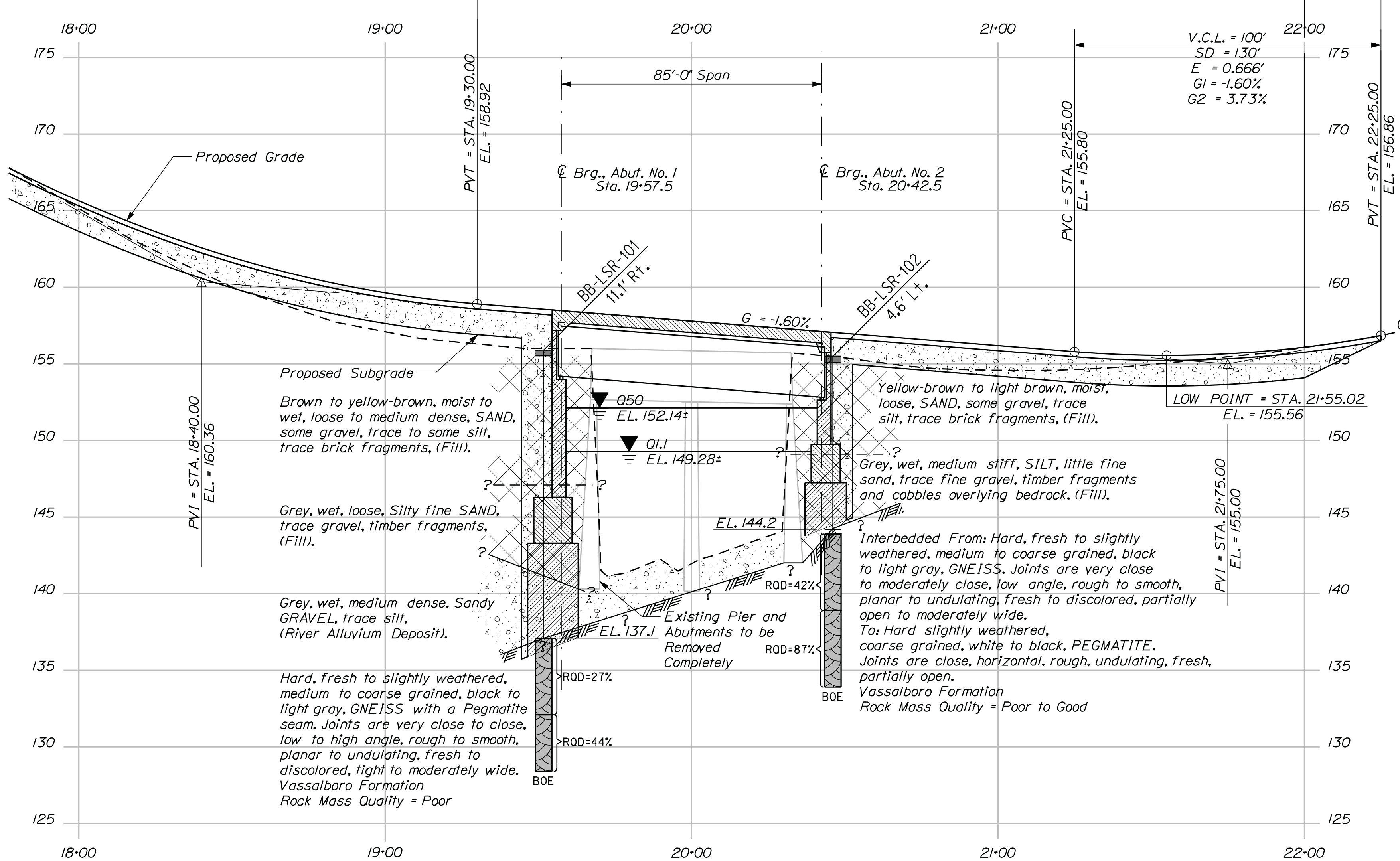
LISBON CENTER BRIDGE NO. 5007 OVER SABATTUS RIVER
MAINEDOT WIN 018970.00
LISBON, ME

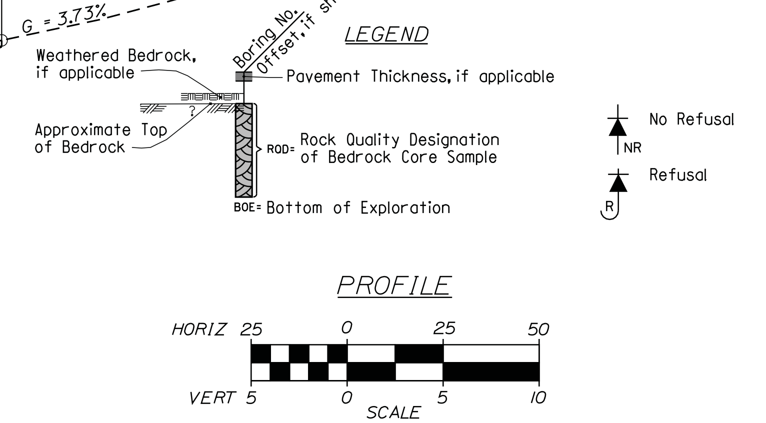
BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

PREPARED BY: GZA GeoEnvironmental, Inc. Engineers and Scientists www.gza.com	PREPARED FOR: MAINE DEPARTMENT OF TRANSPORTATION	
PROJ MGR: NVW	REVIEWED BY: ARB	CHECKED BY: CLS
DESIGNED BY: NVW	DRAWN BY: TW	SCALE:
DATE: 3/27/18	PROJECT NO. 09.0025944.00	REVISION NO. 0

FIG 2
SHEET NO. 1 OF 1







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 transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
018970.00
WIN 18970.00
BRIDGE PLANS
BRIDGE NO. 5007

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

SIGNATURE
P.E. NUMBER
DATE

LISBON CENTER BRIDGE
SABATTUS RIVER
LISBON ANDROSCOGGIN COUNTY
BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
2
OF 3

Filename: ...\GEOTECH\MSTA\006_BLP&ISP1.dgn Division: GEOTECH Username: Terry.White Date:3/23/2018

APPENDIX A – LIMITATIONS

GEOTECHNICAL LIMITATIONS

Use of Report

1. GZA GeoEnvironmental, Inc. (GZA) prepared this report on behalf of, and for the exclusive use of our Client for the stated purpose(s) and location(s) identified in the Proposal for Services and/or Report. Use of this report, in whole or in part, at other locations, or for other purposes, may lead to inappropriate conclusions; and we do not accept any responsibility for the consequences of such use(s). Further, reliance by any party not expressly identified in the contract documents, for any use, without our prior written permission, shall be at that party's sole risk, and without any liability to GZA.

Standard of Care

2. GZA's findings and conclusions are based on the work conducted as part of the Scope of Services set forth in Proposal for Services and/or Report, and reflect our professional judgment. These findings and conclusions must be considered not as scientific or engineering certainties, but rather as our professional opinions concerning the limited data gathered during the course of our work. If conditions other than those described in this report are found at the subject location(s), or the design has been altered in any way, GZA shall be so notified and afforded the opportunity to revise the report, as appropriate, to reflect the unanticipated changed conditions.

3. GZA's services were performed using the degree of skill and care ordinarily exercised by qualified professionals performing the same type of services, at the same time, under similar conditions, at the same or a similar property. No warranty, expressed or implied, is made.

4. In conducting our work, GZA relied upon certain information made available by public agencies, Client and/or others. GZA did not attempt to independently verify the accuracy or completeness of that information. Inconsistencies in this information which we have noted, if any, are discussed in the Report.

Subsurface Conditions

5. The generalized soil profile(s) provided in our Report are based on widely-spaced subsurface explorations and are intended only to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and were based on our assessment of subsurface conditions. The composition of strata, and the transitions between strata, may be more variable and more complex than indicated. For more specific information on soil conditions at a specific location refer to the exploration logs. The nature and extent of variations between these explorations may not become evident until further exploration or construction. If variations or other latent conditions then become evident, it will be necessary to reevaluate the conclusions and recommendations of this report.

6. In preparing this report, GZA relied on certain information provided by the Client, state and local officials, and other parties referenced therein which were made available to GZA at the time of our evaluation. GZA did not attempt to independently verify the accuracy or completeness of all information reviewed or received during the course of this evaluation.

7. Water level readings have been made in test holes (as described in this Report) and monitoring wells at the specified times and under the stated conditions. These data have been reviewed and interpretations have been made in this Report. Fluctuations in the level of the groundwater however occur due to temporal or spatial variations in areal recharge rates, soil heterogeneities, the presence of subsurface utilities, and/or natural or artificially induced perturbations. The water table encountered in the course of the work may differ from that indicated in the Report.

8. GZA's services did not include an assessment of the presence of oil or hazardous materials at the property. Consequently, we did not consider the potential impacts (if any) that contaminants in soil or groundwater may have on construction activities, or the use of structures on the property.

9. Recommendations for foundation drainage, waterproofing, and moisture control address the conventional geotechnical engineering aspects of seepage control. These recommendations may not preclude an environment that allows the infestation of mold or other biological pollutants.

Compliance with Codes and Regulations

10. We used reasonable care in identifying and interpreting applicable codes and regulations. These codes and regulations are subject to various, and possibly contradictory, interpretations. Compliance with codes and regulations by other parties is beyond our control.

Cost Estimates

11. Unless otherwise stated, our cost estimates are only for comparative and general planning purposes. These estimates may involve approximate quantity evaluations. Note that these quantity estimates are not intended to be sufficiently accurate to develop construction bids, or to predict the actual cost of work addressed in this Report. Further, since we have no control over either when the work will take place or the labor and material costs required to plan and execute the anticipated work, our cost estimates were made by relying on our experience, the experience of others, and other sources of readily available information. Actual costs may vary over time and could be significantly more, or less, than stated in the Report.

Additional Services

12. GZA recommends that we be retained to provide services during any future: site observations, design, implementation activities, construction and/or property development/redevelopment. This will allow us the opportunity to: i) observe conditions and compliance with our design concepts and opinions; ii) allow for changes in the event that conditions are other than anticipated; iii) provide modifications to our design; and iv) assess the consequences of changes in technologies and/or regulations.

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APPENDIX B – 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

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))

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Lisbon Center Bridge (No. 5007) carries Mill Street over the Sabattus River
Location: Lisbon, Maine

Boring No.: BB-LSR-101

PIN: 18970.00

Driller:	MaineDOT	**Elevation (ft.)**	155.9	**Auger ID/OD:**	5-inch-diameter
Operator:	T. Daggett/A. Burpee	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140 pounds/30
Date Start/Finish:	7/19/2016; 08:30-12:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ2 (2-inch-
Boring Location:	Sta 19+51.7, 11.1 ft Rt.	**Casing ID/OD:**	NW (3/3.5 inches)	**Water Level*:**	Not
Hammer Efficiency Factor:	0.943	**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 = Insitu Vane Shear Test
MV = Unsuccessful Insitu 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 = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane 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.
0							SSA	155.5		4.5-inch-thick layer of pavement. — 0.4	
	1D	24/14	1.0 - 3.0	8/9/7/5	16	25				Brown, moist, medium dense, SAND, some gravel, trace silt, (Fill).	
5	2D	24/15	5.0 - 7.0	2/2/3/4	5	8	23			Yellow-brown, wet, loose, SAND, some silt, trace brick fragments, (Fill).	
							13				
							8				
							8	146.9		— 9.0	
							12				
10	3D	24/18	10.0 - 12.0	3/2/2/2	4	6	8			Gray, wet, loose, silty fine SAND, trace gravel, timber fragments, (Fill).	G#270901 A-4, SM WC=57.0%
							10				
							11				
							20	142.4		— 13.5	
							16				
15	4D	24/12	15.0 - 17.0	6/13/14/19	27	42	38			Gray, wet, medium dense, Sandy GRAVEL, trace silt, (River Alluvium Deposit).	G#270902 A-1-a, GW-GM WC=14.0%
							47				
							100			a65 blows for 9.6 inches.	
	R1	60/43	18.8 - 23.8	RQD = 27%			a65 NQ2	137.1		— 18.8 Top of Bedrock at Elevation 137.1 feet. R1: Hard, fresh to slightly weathered, black and light gray, medium to coarse grained, GNEISS. Joints are horizontal to low angle, very close to close, smooth to rough, undulating to planar, fresh to discolored, open. Biotite on joint surfaces. Vassalboro Formation Rock Mass Quality = Poor R1: Core Times (min:sec): 18.8-19.8 feet (3:04), 19.8-20.8 feet (3:11), 20.8-21.8 feet (2:34), 21.8-22.8 feet (2:55), 22.8-23.8 feet (3:47)	UCT q_p = 9, 593 psi
20											
	R2	45/45	23.8 - 27.6	RQD = 44%						R2: 23.8'-25.6' Hard, fresh, black to light gray, medium to coarse grained, GNEISS. Joints are low angle to high	
25											

Remarks:

-bgs = below existing ground surface
Bedrock descriptions by GZA based on observation of core boxes.

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: Lisbon Center Bridge (No. 5007) carries Mill Street over the Sabattus River
Location: Lisbon, Maine

Boring No.: BB-LSR-101
PIN: 18970.00

Driller:	MaineDOT	**Elevation (ft.)**	155.9	**Auger ID/OD:**	5-inch-diameter
Operator:	T. Daggett/A. Burpee	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140 pounds/30
Date Start/Finish:	7/19/2016; 08:30-12:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ2 (2-inch-
Boring Location:	Sta 19+51.7, 11.1 ft Rt.	**Casing ID/OD:**	NW (3/3.5 inches)	**Water Level*:**	Not
Hammer Efficiency Factor:	0.943	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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 = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane 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										angle, rough to smooth, planar, undulating, very close to close, fresh, tight to moderately wide. Biotite on joint surfaces. Vassalboro Formation	
								130.3		25.6	
								130.1		R2: 25.6'-25.8': Hard, fresh, pink/white/black, coarse grained, PEGMATITE. Feldspar (white), biotite, quartz, discolored (rusty). Vassalboro Formation Rock Mass Quality = Poor	
										25.8	
								128.4		R2: 25.8'-27.5': Hard, black/light gray, medium to coarse grained, GNEISS. Joints are low angle, very close to close, rough to smooth, planar to undulating, tight to moderately wide. One high angle joint. Vassalboro Formation R2: Core Times (min:sec): 23.8-24.8 feet (2:42), 24.8-25.8 feet (2:19), 25.9-26.8 feet (2:32), 26.8-27.5 feet (3:00) Core Blocked	
										27.5	
										Bottom of Exploration at 27.50 feet below ground surface.	
30											
35											
40											
45											
50											

Remarks:

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Bedrock descriptions by GZA based on observation of core boxes.

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: Lisbon Center Bridge (No. 5007) carries Mill Street over the Sabattus River
Location: Lisbon, Maine

Boring No.: BB-LSR-102
PIN: 18970.00

Driller:	MaineDOT	**Elevation (ft.)**	155.5	**Auger ID/OD:**	5-inch-diameter
Operator:	T. Daggett/A. Burpee	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140 pounds/30
Date Start/Finish:	7/20/2016; 07:00-11:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ2 (2-inch-
Boring Location:	Sta 20+46.1, 4.6 ft Lt.	**Casing ID/OD:**	NW (3/3.5 inches)	**Water Level*:**	Not
Hammer Efficiency Factor:	0.943	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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.
0							SSA	155.1		5-inch-thick layer of pavement. 0.4 Yellow-brown, moist, SAND, some gravel, trace silt, trace brick fragments (Fill).	
5	1D/A	24/19	5.0 - 7.0	3/3/3/3	6	9	6			1D (5-6 feet): Light brown, moist, loose, SAND, trace silt, trace brick fragments, (Fill).	
							12	149.5		6.0 1D(A) (6-7 feet): Gray, wet, medium stiff, SILT, little sand, trace gravel (Fill).	G#270903 A-4, ML WC=25.3% Non-Plastic
							20				
							64				
							19				
10	2D	9.6/2	10.0 - 10.8	2/50(3.6")	---		17				
	R1	60/55	11.6 - 16.6	RQD = 42%			a82 NQ2	144.2		Timber fragments and cobbles. a82 blows for 3.6 inches. RC to 11.6 feet bgs. 11.3	UCT q_p = 9,900 psi
								142.1 141.8		Top of Bedrock at Elevation 144.2 feet. R1: 11.6'-13.4': Hard, fresh to slightly weathered, black to light gray, medium to coarse grained, GNEISS. Joints are low angle, rough, undulating, very close to close, fresh to discolored, partially open to moderately wide. Biotite on joint surfaces. Vassalboro Formation Rock Mass Quality = Poor Core Times (min:sec): 11.6-12.6 feet (2: 38), 12.6-13.6 feet (2:38), 13.6-14.6 feet (2:51), 14.6-15.6 feet (2:20), 15.6-16.6 feet (3:00) 13.4	
15											
	R2	60/60	16.6 - 21.6	RQD = 87%				138.4		R1: 13.4'-13.7': Hard, discolored, white, coarse to very coarse grained, PEGMATITE. No joints. Vassalboro Formation 13.7	
								137.2			
20								135.6		R1: 13.7'-16.6': Hard, fresh to slightly weathered, black to light gray, medium to coarse grained, GNEISS. Joints are low angle, rough, undulating, very close to close, fresh to discolored, partially open to moderately wide. Biotite on joint surfaces. Vassalboro Formation	
								133.9		R2: 16.6'-17.1': Hard, fresh, dark gray, medium grained, GNEISS. Joints are low angle, rough, planar, very close to close, fresh to discolored, partially open. Vassalboro Formation 17.1	
25											

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Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

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Location: Lisbon, Maine

Boring No.: BB-LSR-102

PIN: 18970.00

Driller:	MaineDOT	**Elevation (ft.)**	155.5	**Auger ID/OD:**	5-inch-diameter
Operator:	T. Daggett/A. Burpee	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140 pounds/30
Date Start/Finish:	7/20/2016; 07:00-11:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ2 (2-inch-
Boring Location:	Sta 20+46.1, 4.6 ft Lt.	**Casing ID/OD:**	NW (3/3.5 inches)	**Water Level*:**	Not
Hammer Efficiency Factor:	0.943	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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										R2: 17.1'-18.3': Hard, slightly weathered, light gray and black, coarse grained, PEGMATITE. No joints. Vassalboro Formation	
										18.3	
										R2: 18.3'-19.9': Hard, slightly weathered, light gray and black banded, medium grained, GNEISS. Joints are low angle, smooth, planar, moderately close, fresh to discolored, open. Biotite on joint surfaces. Vassalboro Formation	
										19.9	
30										R2: 19.9'-21.6': Fresh to slightly weathered, light gray, coarse grained, P EGMATITE. Joints are horizontal, rough, undulating, close, fresh, partially open. Vassalboro Formation Rock Mass Quality = Good R2: Core Times (min:sec): 16.6-17.6 feet (2: 15), 17.6-18.6 feet (3:27), 18.6-19.6 feet (3:50), 19.6-20.6 feet (3:43), 20.6-21.6 feet (4:16)	
										21.6	
										Bottom of Exploration at 21.60 feet below ground surface.	
35											
40											
45											
50											

Remarks:

-bgs = below existing ground surface
Bedrock descriptions by GZA based on observation of core boxes.

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.

APPENDIX C – LABORATORY TEST RESULTS

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Lisbon   Work Number: 18970.00

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-LSR-101, 3D	19+51.7	11.1 Rt.	10.0-12.0	270901	1	57.0			SM	A-4	III
BB-LSR-101, 4D	19+51.7	11.1 Rt.	15.0-17.0	270902	1	14.0			GW-GM	A-1-a	0
BB-LSR-102, 1D/A	20+46.1	4.6 Lt.	6.0-7.0	270903	1	25.3	-N	P-	CL	A-4	IV

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.

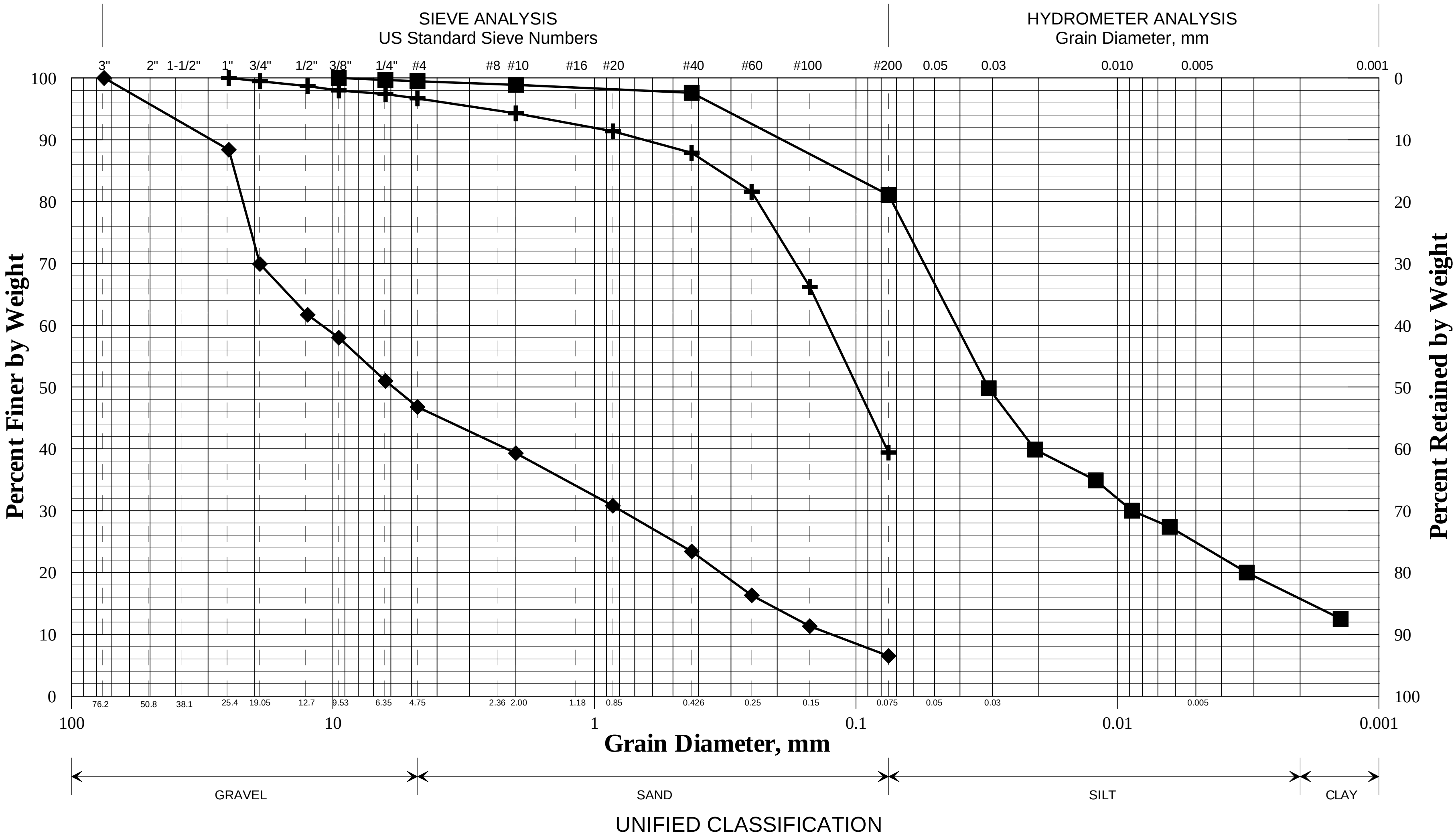
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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-LSR-101/3D	19+51.7	11.1 RT	10.0-12.0	Sandy SILT, trace gravel.	57.0			
◆	BB-LSR-101/4D	19+51.7	11.1 RT	15.0-17.0	Sandy GRAVEL, trace silt.	14.0			
■	BB-LSR-102/1D(A)	20+46.1	4.6 LT	6.0-7.0	SILT, little sand, little clay, trace gravel.	25.3			NP
●									
▲									
✖									

WIN
018970.00
Town
Lisbon
Reported by/Date
WHITE, TERRY A 8/16/2016

SHEET 1

MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
270901	**BB-LSR-101/3D**	**GEOTECHNICAL (DISTURBED)**	**7/19/2016**	**8/5/2016**

Sample Type: **GEOTECHNICAL** Location: Station: **19+51.7** Offset, ft: **11.1 RT** Dbfg, ft: **10.0-12.0**

WIN/Town **018970.00 - LISBON** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	**100.0**
¾ in. [19.0 mm]	**99.5**
½ in. [12.5 mm]	**98.7**
⅜ in. [9.5 mm]	**98.0**
¼ in. [6.3 mm]	**97.4**
No. 4 [4.75 mm]	**96.7**
No. 10 [2.00 mm]	**94.3**
No. 20 [0.850 mm]	**91.4**
No. 40 [0.425 mm]	**87.9**
No. 60 [0.250 mm]	**81.6**
No. 100 [0.150 mm]	**66.2**
No. 200 [0.075 mm]	**39.4**

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**57.0**

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **8/10/2016**

Paper Copy: Lab File; Project File; Geotech File

MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
270902	**BB-LSR-101/4D**	**GEOTECHNICAL (DISTURBED)**	**7/19/2016**	**8/5/2016**

Sample Type: **GEOTECHNICAL** Location: Station: **19+51.7** Offset, ft: **11.1 RT** Dbfg, ft: **15.0-17.0**

WIN/Town **018970.00 - LISBON** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	**100.0**
1 in. [25.0 mm]	**88.4**
¾ in. [19.0 mm]	**69.9**
½ in. [12.5 mm]	**61.7**
⅜ in. [9.5 mm]	**58.0**
¼ in. [6.3 mm]	**51.0**
No. 4 [4.75 mm]	**46.8**
No. 10 [2.00 mm]	**39.3**
No. 20 [0.850 mm]	**30.8**
No. 40 [0.425 mm]	**23.4**
No. 60 [0.250 mm]	**16.3**
No. 100 [0.150 mm]	**11.3**
No. 200 [0.075 mm]	**6.5**

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**14.0**

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **8/10/2016**

Paper Copy: Lab File; Project File; Geotech File

MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
270903	BB-LSR-102/1D(A)	GEOTECHNICAL (DISTURBED)	7/20/2016	8/5/2016

Sample Type: **GEOTECHNICAL** Location: Station: **20+46.1** Offset, ft: **4.6 LT** Dbfg, ft: **6.0-7.0**

WIN/Town **018970.00 - LISBON** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	100.0
¼ in. [6.3 mm]	99.7
No. 4 [4.75 mm]	99.5
No. 10 [2.00 mm]	98.9
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	97.6
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	81.1
[0.0310 mm]	49.8
[0.0206 mm]	39.9
[0.0121 mm]	34.9
[0.0088 mm]	30.0
[0.0063 mm]	27.4
[0.0032 mm]	20.0
[0.0014 mm]	12.5

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	NP
Specific Gravity, Corrected to 20°C (T 100)	2.61
Loss on Ignition, % (T 267)	
Water Content (T 265), %	25.3

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **8/10/2016**

Paper Copy: Lab File; Project File; Geotech File

GeoTesting
EXPRESS
Technologies to manage risk for infrastructure

Boston
Atlanta
Chicago
Los Angeles
New York
www.geotesting.com

Transmittal

TO:
Nathan Sherwood
Maine DOT
16 State House Sta.
Augusta, ME 04333

DATE: 1/25/2017	GTX NO: 305861
RE: Lisbon Center Bridge	

COPIES	DATE	DESCRIPTION
	1/25/2017	**January 2017 Laboratory Test Report**

REMARKS:

SIGNED:
Jonathan Campbell, Assistant Laboratory Manager

CC:

APPROVED BY:
Mark Dobday, P.G., Laboratory Manager

GeoTesting
EXPRESS

Technologies to manage risk
for infrastructure

Boston
Atlanta
Chicago
Los Angeles
New York
www.geotesting.com

January 25, 2017

Nathan Sherwood
Maine DOT
16 State House Sta.
Augusta, ME 04333

RE: Lisbon Center Bridge, Lisbon, ME (GTX-305861)

Dear Nathan Sherwood:

Enclosed are the test results you requested for the above referenced project. GeoTesting Express, Inc. (GTX) received two samples from you on 1/10/2017. These samples were labeled as follows:

Boring Number	Sample Number	Depth
BB-LSR-101	R1	19.8-20.4 ft
BB-LSR-102	R1	12.1-12.5 ft

GTX performed the following tests on these samples:

2 ASTM D7012 Method D- Elastic Moduli of Rock in Uniaxial Compression

A copy of your test request is attached.

The results presented in this report apply only to the items tested. This report shall not be reproduced except in full, without written approval from GeoTesting Express. The remainder of these samples will be retained for a period of sixty (60) days and will then be discarded unless otherwise notified by you. Please call me if you have any questions or require additional information. Thank you for allowing GeoTesting Express the opportunity of providing you with testing services. We look forward to working with you again in the future.

Respectfully yours,

Jonathan Campbell
Assistant Laboratory Manager

GeoTesting
EXPRESS

Technologies to manage risk
for infrastructure

Boston
Atlanta
Chicago
Los Angeles
New York
www.geotesting.com

Geotechnical Test Report **1/25/2017**

GTX-305861

Lisbon Center Bridge

Lisbon, ME

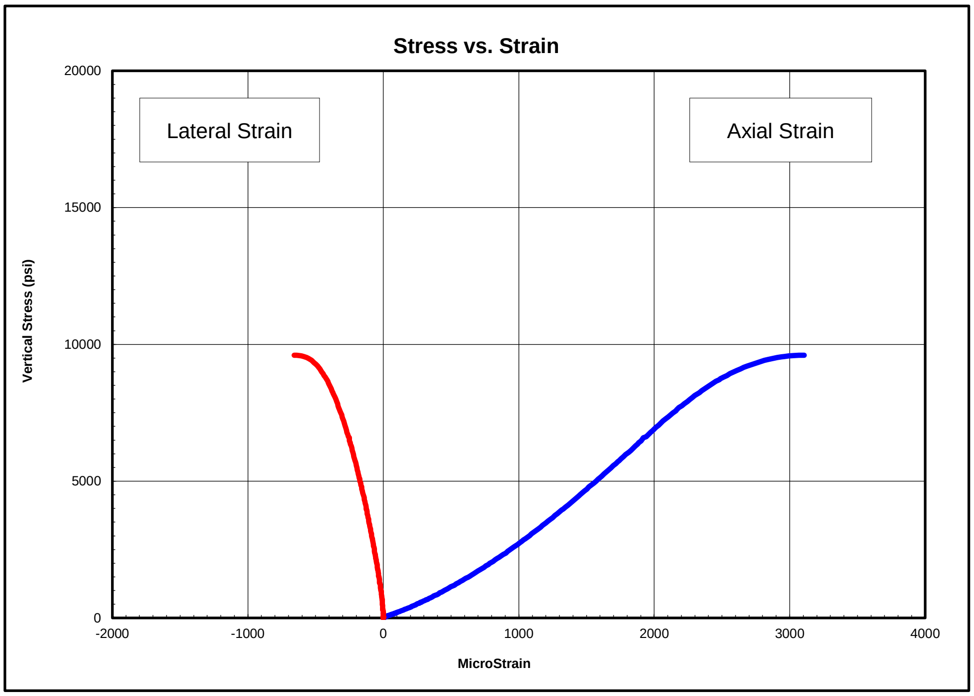
Client Project No.: 18970.00

Prepared for:

Maine DOT

Client:	Maine DOT
Project Name:	Lisbon Center Bridge
Project Location:	Lisbon, ME
GTX #:	305861
Test Date:	1/17/2017
Tested By:	daa/rlc
Checked By:	jsc
Boring ID:	BB-LSR-101
Sample ID:	R1
Depth, ft:	19.8-20.4
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

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



Peak Compressive Stress: 9,593 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1000-3500	3,290,000	0.12
3500-6100	4,260,000	0.20
6100-8600	4,080,000	0.29

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 DOT	Test Date:	1/12/2017
Project Name:	Lisbon Center Bridge	Tested By:	daa/rlc
Project Location:	Lisbon, ME	Checked By:	jsc
GTX #:	305861		
Boring ID:	BB-LSR-101		
Sample ID:	R1		
Depth:	19.8-20.4 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:	4.47	4.47	4.47
Specimen Diameter, in:	1.95	1.95	1.95
Specimen Mass, g:	593.54		
Bulk Density, lb/ft^3	169	**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.3	**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.? 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	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00010	0.00020	0.00010	0.00010	0.00010
Diameter 2, in (rotated 90^o)	-0.00010	-0.00010	-0.00010	-0.00010	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00000

Difference between max and min readings, in:

0° = 0.00030 90° = 0.00030

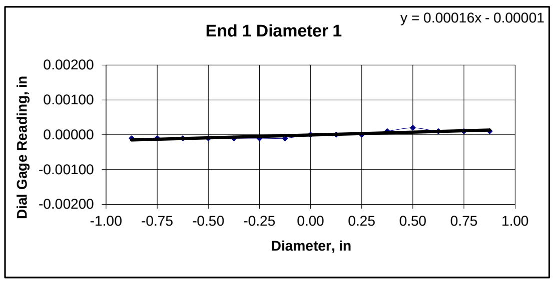
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00020	-0.00020	-0.00010	-0.00010	-0.00010	-0.00020	-0.00020	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	0.00000
Diameter 2, in (rotated 90^o)	-0.00020	-0.00020	-0.00020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

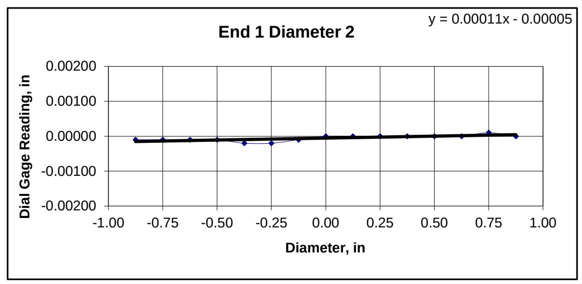
Difference between max and min readings, in:

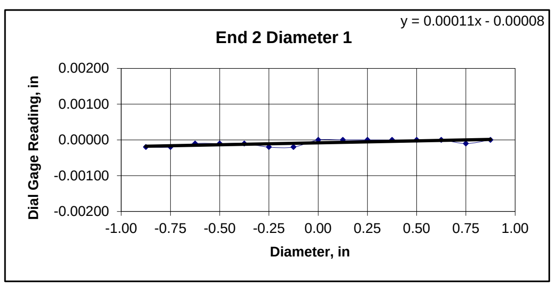
0° = 0.0002 90° = 0.0002

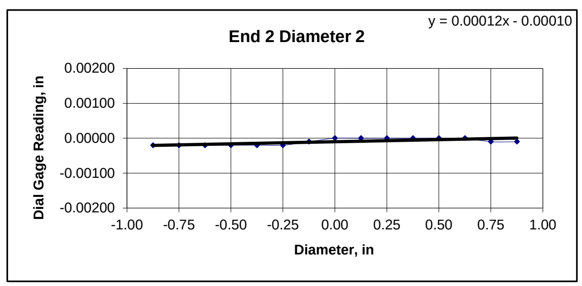
Maximum difference must be < 0.0020 in. Difference = ± 0.00015

Flatness Tolerance Met? YES









DIAMETER 1

End 1:
- Slope of Best Fit Line 0.00016
- Angle of Best Fit Line: 0.00917

End 2:
- Slope of Best Fit Line 0.00011
- Angle of Best Fit Line: 0.00630

Maximum Angular Difference: 0.00286

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:
- Slope of Best Fit Line 0.00011
- Angle of Best Fit Line: 0.00630

End 2:
- Slope of Best Fit Line 0.00012
- Angle of Best Fit Line: 0.00688

Maximum Angular Difference: 0.00057

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	1.950	0.00015	0.009	YES
Diameter 2, in (rotated 90^o)	0.00030	1.950	0.00015	0.009	YES
END 2					
Diameter 1, in	0.00020	1.950	0.00010	0.006	YES
Diameter 2, in (rotated 90^o)	0.00020	1.950	0.00010	0.006	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? YES

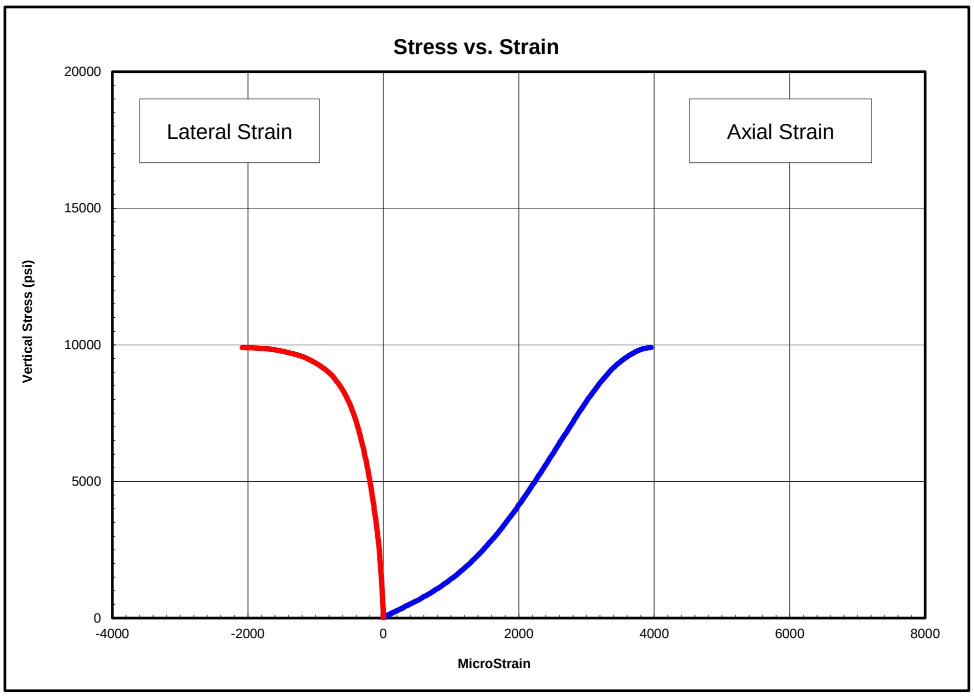
Client:	Maine DOT
Project Name:	Lisbon Center Bridge
Project Location:	Lisbon, ME
GTX #:	305861
Test Date:	1/17/2017
Tested By:	daa/rlc
Checked By:	jsc
Boring ID:	BB-LSR-101
Sample ID:	R1
Depth, ft:	19.8-20.4

After cutting and grinding

After break

Client:	Maine DOT
Project Name:	Lisbon Center Bridge
Project Location:	Lisbon, ME
GTX #:	305861
Test Date:	1/17/2017
Tested By:	daa/rlc
Checked By:	jsc
Boring ID:	BB-LSR-102
Sample ID:	R1
Depth, ft:	12.1-12.5
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

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



Peak Compressive Stress: 9,900 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1000-3600	2,370,000	0.08
3600-6300	3,710,000	0.26
6300-8900	3,650,000	---

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

GeoTesting EXPRESS

Client:	Maine DOT	Test Date:	1/12/2017
Project Name:	Lisbon Center Bridge	Tested By:	daa/rlc
Project Location:	Lisbon, ME	Checked By:	jsc
GTX #:	305861		
Boring ID:	BB-LSR-102		
Sample ID:	R1		
Depth:	12.1-12.5 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:	4.60	4.60	4.60		
Specimen Diameter, in:	1.95	1.96	1.96		
Specimen Mass, g:	622.65				
Bulk Density, lb/ft^3	171			**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.4			**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.? 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	0.00010	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010	0.00000	0.00000	-0.00010	-0.00010	-0.00020	-0.00020	-0.00010	-0.00020
Diameter 2, in (rotated 90°)	-0.00020	-0.00020	-0.00020	-0.00020	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	-0.00020	-0.00030	-0.00030	-0.00030

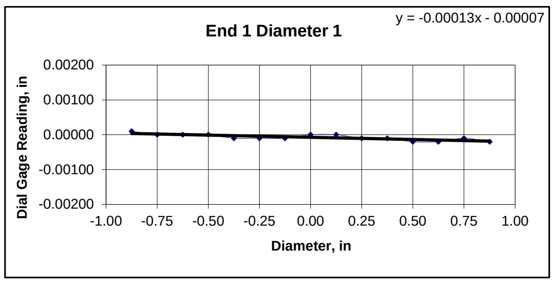
Difference between max and min readings, in:
0° = 0.00030 90° = 0.00030

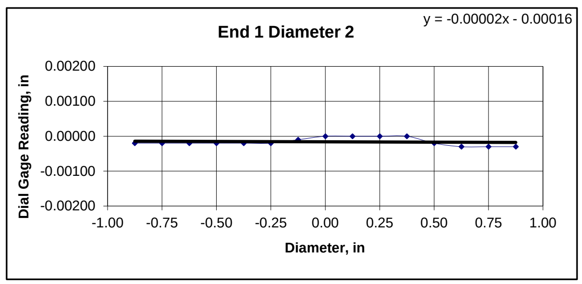
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00020	-0.00020
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	-0.00010	-0.00010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

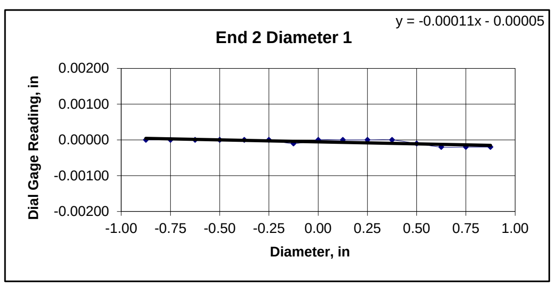
Difference between max and min readings, in:
0° = 0.0002 90° = 0.0002

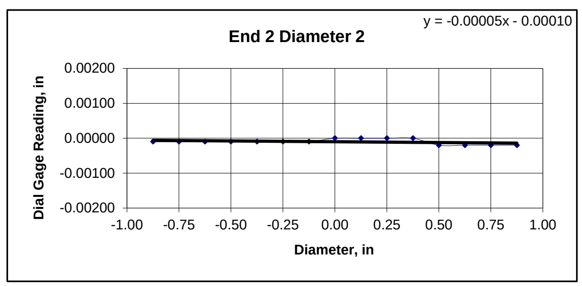
Maximum difference must be < 0.0020 in. Difference = ± 0.00015

Flatness Tolerance Met? YES









DIAMETER 1

End 1:
Slope of Best Fit Line -0.00013
Angle of Best Fit Line: -0.00745

End 2:
Slope of Best Fit Line -0.00011
Angle of Best Fit Line: -0.00630

Maximum Angular Difference: 0.00115

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line -0.00002
Angle of Best Fit Line: -0.00115

End 2:
Slope of Best Fit Line -0.00005
Angle of Best Fit Line: -0.00286

Maximum Angular Difference: 0.00172

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	1.955	0.00015	0.009	YES
Diameter 2, in (rotated 90°)	0.00030	1.955	0.00015	0.009	YES
END 2					
Diameter 1, in	0.00020	1.955	0.00010	0.006	YES
Diameter 2, in (rotated 90°)	0.00020	1.955	0.00010	0.006	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? YES

Client:	Maine DOT
Project Name:	Lisbon Center Bridge
Project Location:	Lisbon, ME
GTX #:	305861
Test Date:	1/17/2017
Tested By:	daa/rlc
Checked By:	jsc
Boring ID:	BB-LSR-102
Sample ID:	R1
Depth, ft:	12.1-12.5

After cutting and grinding

After break

GeoTesting EXPRESS

ROCK CHAIN OF CUSTODY & TEST REQUEST

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125 Nagog Park
Acton, MA 01720
800 434 1062 Toll Free
978 635 0266 Fax

2358 Perimeter Park Drive, Suite 320
Atlanta, GA 30341
770 645 6575 Tel
770 645 6570 Fax

www.geotesting.com

CLIENT		INVOICE (complete if different from Client)	
Company: Maine Department of Transportation - Bridge Program		Company:	
Address: 16 State House Station		Address:	
City, State, Zip: Augusta, Maine, 04333		City, State, Zip:	
Contact: Nate Sherwood	Phone: 207-624-3452	Contact:	Phone:
E-mail: Nathan.A.Sherwood@maine.gov	Cell: 207-240-5872	E-mail:	Cell:

PROJECT		
Project Name: Lisbon Center Bridge	Client Project #: 18970.00	Purchase Order#:
Project Location: Lisbon, Maine	GTX Sales Order #:	Requested Turnaround: Standard time
On-site Contact: Nate Sherwood	E-mail: same as contact	Phone: same as contact

ROCK Core Run #	Sample ID	Depth	CERCHAR Abrasivity (ASTM D 7625) * 55HRC/40HRC	Direct Shear (ASTM D5607)*	Direct Tensile Strength (ASTM D 2936)	Elastic Moduli in Triaxial Compression (ASTM D 7012B)	Elastic Moduli in Uniaxial Compression (ASTM D 7012D)	Unit Weight (ISRM)	Petrographic Analysis (ISRM)	Point Load Index (ASTM D 5731)* Diametral, Axial, Lump/Block	Punch Penetration (Handewith)	Slake Durability (ASTM D 4644)	Splitting (Brazilian) Tensile Strength (ASTM D 3967)	Schmidt Hammer (ASTM D 5873)	Total Hardness (Schmidt Hammer and Taber Abrasion)	Triaxial Compression (ASTM D 7012A)	Unconfined Compression (ASTM D 7012C)	Other:____	Other:____
R1	BB-LSR-101	19.8' to 20.4'					x												
R1	BB-LSR-102	12.1' to 12.5'					x												

*Specify Test Conditions (Undisturbed or Remolded, Density and Moisture, Test Normal Loads, Test Confining Stresses, etc.):

AUTHORIZE BY SIGNING AND DATING:

SIGNATURE: Nathan A. Sherwood   PRINT NAME: NATHAN SHERWOOD   DATE: ~~11/14/16~~ 1/9/17

For GTX Use Only
Incoming Sample Inspection Performed ☐
Adverse conditions:

Relinquished By: Nate Sherwood	DATE: 01/09/17 TIME: 11:00 am	Received By: [signature]	DATE: 1/10/2017 TIME: 11:30AM
Relinquished By:	DATE: TIME:	Received By:	DATE: TIME:

WARRANTY and LIABILITY

GeoTesting Express (GTX) warrants that all tests it performs are run in general accordance with the specified test procedures and accepted industry practice. GTX will correct or repeat any test that does not comply with this warranty. GTX has no specific knowledge as to conditioning, origin, sampling procedure or intended use of the material.

GTX may report engineering parameters that require us to interpret the test data. Such parameters are determined using accepted engineering procedures. However, GTX does not warrant that these parameters accurately reflect the true engineering properties of the *in situ* material. Responsibility for interpretation and use of the test data and these parameters for engineering and/or construction purposes rests solely with the user and not with GTX or any of its employees.

GTX's liability will be limited to correcting or repeating a test which fails our warranty. GTX's liability for damages to the Purchaser of testing services for any cause whatsoever shall be limited to the amount GTX received for the testing services. GTX will not be liable for any damages, or for any lost benefits or other consequential damages resulting from the use of these test results, even if GTX has been advised of the possibility of such damages. GTX will not be responsible for any liability of the Purchaser to any third party.

Commonly Used Symbols

A	pore pressure parameter for $\Delta\sigma_1 - \Delta\sigma_3$
B	pore pressure parameter for $\Delta\sigma_3$
CAI	CERCHAR Abrasiveness Index
CIU	isotropically consolidated undrained triaxial shear test
CR	compression ratio for one dimensional consolidation
CSR	cyclic stress ratio
C_c	coefficient of curvature, $(D_{30})^2$ / (D_{10} x D_{60})
C_u	coefficient of uniformity, D_{60}/D_{10}
C_c	compression index for one dimensional consolidation
C_α	coefficient of secondary compression
c_v	coefficient of consolidation
c	cohesion intercept for total stresses
c'	cohesion intercept for effective stresses
D	diameter of specimen
D	damping ratio
D_{10}	diameter at which 10% of soil is finer
D_{15}	diameter at which 15% of soil is finer
D_{30}	diameter at which 30% of soil is finer
D_{50}	diameter at which 50% of soil is finer
D_{60}	diameter at which 60% of soil is finer
D_{85}	diameter at which 85% of soil is finer
d_{50}	displacement for 50% consolidation
d_{90}	displacement for 90% consolidation
d_{100}	displacement for 100% consolidation
E	Young's modulus
e	void ratio
e_c	void ratio after consolidation
e_o	initial void ratio
G	shear modulus
G_s	specific gravity of soil particles
H	height of specimen
H_R	Rebound Hardness number
i	gradient
I_S	Uncorrected point load strength
$I_{S(50)}$	Size corrected point load strength index
H_A	Modified Taber Abrasion
H_T	Total hardness
K_o	lateral stress ratio for one dimensional strain
k	permeability
LI	Liquidity Index
m_v	coefficient of volume change
n	porosity
PI	plasticity index
P_c	preconsolidation pressure
p	$(\sigma_1 + \sigma_3) / 2$, $(\sigma_v + \sigma_h) / 2$
p'	$(\sigma'_1 + \sigma'_3) / 2$, $(\sigma'_v + \sigma'_h) / 2$
p'_c	p' at consolidation
Q	quantity of flow
q	$(\sigma_1 - \sigma_3) / 2$
q_f	q at failure
q_o, q_i	initial q
q_c	q at consolidation
S_r	Post cyclic undrained shear strength
T	temperature
t	time
U, UC	unconfined compression test
UU, Q	unconsolidated undrained triaxial test
u_a	pore gas pressure
u_e	excess pore water pressure
u, u_w	pore water pressure
V	total volume
V_g	volume of gas
V_s	volume of solids
V_s	shear wave velocity
V_v	volume of voids
V_w	volume of water
V_o	initial volume
v	velocity
W	total weight
W_s	weight of solids
W_w	weight of water
w	water content
w_c	water content at consolidation
w_f	final water content
w_l	liquid limit
w_n	natural water content
w_p	plastic limit
w_s	shrinkage limit
w_o, w_i	initial water content
α	slope of q_f versus p_f
α'	slope of q_f versus p_f'
γ_t	total unit weight
γ_d	dry unit weight
γ_s	unit weight of solids
γ_w	unit weight of water
ε	strain
ε_{vol}	volume strain
ε_h, ε_v	horizontal strain, vertical strain
μ	Poisson's ratio, also viscosity
σ	normal stress
σ'	effective normal stress
σ_c, σ'_c	consolidation stress in isotropic stress system
σ_h, σ'_h	horizontal normal stress
σ_v, σ'_v	vertical normal stress
σ'_{vc}	Effective vertical consolidation stress
σ_1	major principal stress
σ_2	intermediate principal stress
σ_3	minor principal stress
τ	shear stress
φ	friction angle based on total stresses
φ'	friction angle based on effective stresses
φ'_r	residual friction angle
φ_{ult}	φ for ultimate strength

APPENDIX D – ROCK CORE PHOTOGRAPHS

MaineDOT Lisbon Center
Bridge #5007 Carries Mill Street over the Sabattus River
Lisbon, ME
Rock Core Photographs

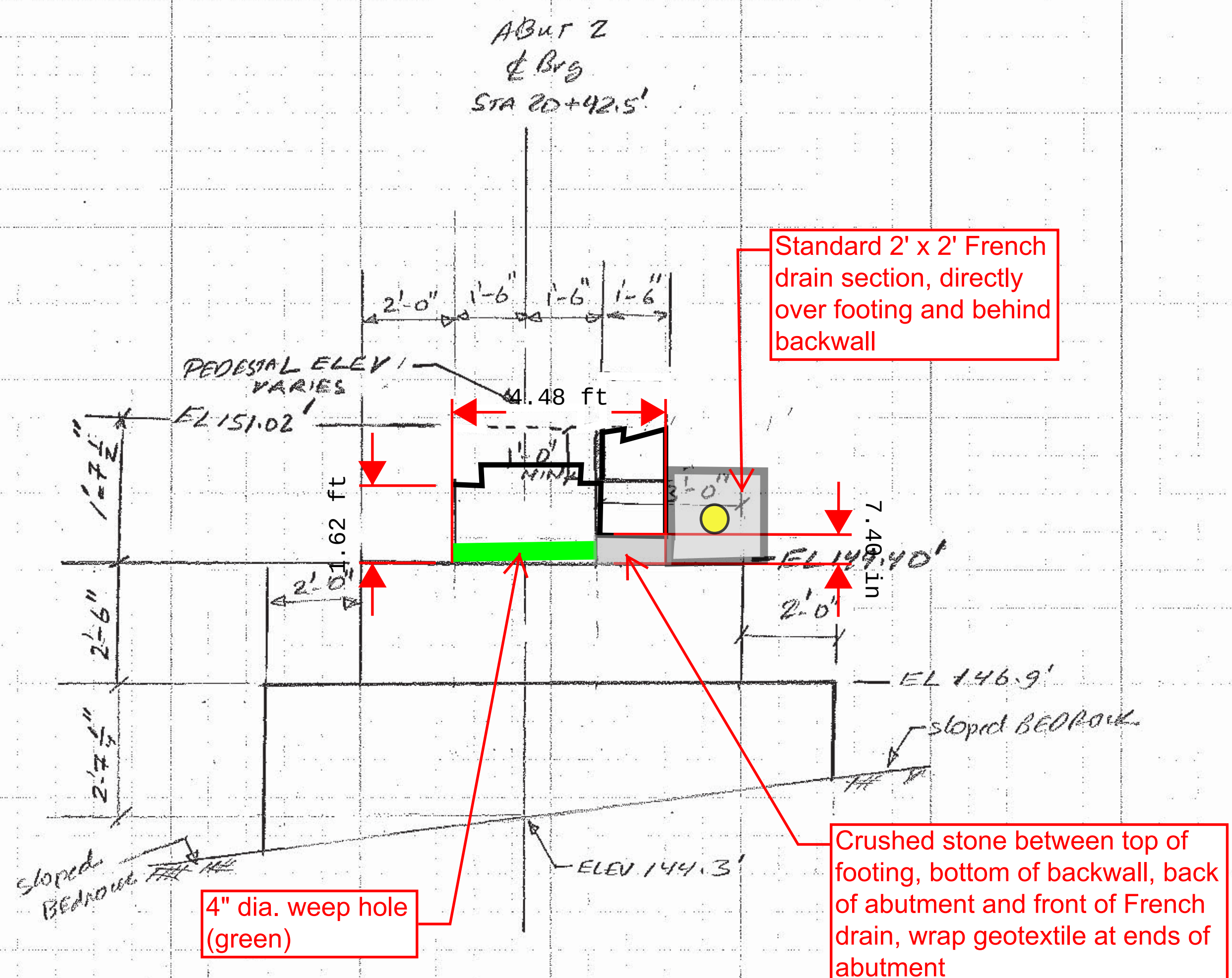
Boring No.	Run	Depth (ft)			Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-LSR-101	R1	18.8	-	23.8	43	72%	16	27%	GNEISS	1
BB-LSR-101	R2	23.8	-	27.5	45	100%	20	44%	GNEISS, PEGMATITE	2
BB-LSR-102	R1	11.6	-	16.6	55	92%	25	42%	GNEISS, PEGMATITE	3
BB-LSR-102	R2	16.6	-	21.6	60	100%	52	87%	GNEISS, PEGMATITE	4

Notes:
1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 4=Bottom.
2. Top photo is dry, bottom photo is wet.
3. Transition between core runs within a row are marked by wood separators.

APPENDIX E – CALCULATIONS

PRELIM. BY ______ DATE ______ PROJ. NO. ______ FILE NO. ______ OF ______
FINAL CHK. BY ______ DATE ______ LOCATION ______ SHEET NO. ______ OF ______
ITEM NO. ______ SUBJECT ______

Recommendations for Drainage at Abutment 2
Lisbon Center Bridge. WIN 18970.00



ABUT 2
SECTION A2-A2 @ ℄ of Const
SCALE 1/4" = 1'-0"

GZA
GeoEnvironmental, Inc
477 Congress Street
Suite 700
Portland, Maine 04101
207-879-9190
Fax 207-879-0099

Engineers and Scientists

JOB: 09.0025944.00 Lisbon Ctr Bridge
SUBJECT: Approach Settlement, Janbu Method
SHEET: 1 OF 6
CALCULATED BY N. Williams 12/1/17
CHECKED BY A. Blaisdell 1/3/18

Objective

Calculate amount of settlement of in-situ soil due to placement of fill using the Janbu Tangent Modulus Method and SPT data.

References

1. MaineDOT Bridge Design Guide (BDG)
2. NAVFAC Design Manual 7.01, pg. 7.1-22 - Table 6 (see Sheet 3 attached).
3. Janbu Tangent Modulus Method (Janbu 1963, 1967) taken from Engineering Manual for Shallow Foundations, NCHRP Project 24-4, pg 116-120 (see Sheets 4 through 6 attached).

Soil Properties and Geotechnical Inputs

$N_{60} := 9$ — N_{60} value from SPT of in-situ soil at the location of maximum fill height.

$\gamma_w := 62.4 \cdot pcf$ — Unit weight of water.

Soil properties taken from Table 6 of NAVFAC, based on medium stiff SILT from SPT information

$\gamma_{SILT} := 110pcf$ — Wet Unit weight of silt

$n := 45$ — n = porosity based on Table 6 from NAVFAC

Assume groundwater table at the top of the in-situ soil (Approx. El. 147 From ISP, and described as "wet" in BB-LSR-102)

Embankment Dimensions

$H_{fill} := 10 \cdot ft$ — Max fill height occurs to the right of abutment 2

$\gamma_{FILL} := 125 \cdot pcf$ — Total unit weight of soil based on Granular Borrow for Underwater Backfill - BDG Type 4 soil

Calculation

$$S := \Delta z \cdot \frac{\Delta\sigma_v}{M_t}$$

Settlement equation based on Janbu's Tangent Modulus

S = settlement
M.t = Tangent value of constrained modulus of the soil (stress units -- the same units as Δσ.v')
Δσ.v = Increase in effective stress within the sublayer due to the load (stress units -- the same units as M.t).
Δz = sublayer thickness

$$M_t := m \cdot p_a \cdot \left(\frac{\sigma_{va}}{p_a}\right)^{.5}$$

Tangent Value of Constrained Modulus equation, based on the modulus number (table 5.5) and the average vertical stress

m = dimensionless modulus number
σ_{va}' = average vertical stress = 1/2*(σ_{vo}' + σ_{vf}')
p_a = atmospheric pressure

Layer Thickness and Effective Stress

Based on boring BB-LSR-102 at abutment 2, a 5' thick layer of medium stiff SILT is used in the settlement analysis

$\Delta z := 5.0 \cdot ft$ — Sublayer thickness

$d := \frac{\Delta z}{2}$ — d = depth of the midpoint of the layer

$\sigma_{vo} := d \cdot (\gamma_{SILT} - \gamma_w) = 119 \cdot psf$ — Initial vertical effective stress - at midpoint of the layer

$\Delta\sigma_v := H_{fill} \cdot \gamma_{FILL} = 1250\, psf$ — Change in vertical effective stress due to the fill

$\sigma_{vf} := \Delta\sigma_v + \sigma_{vo}$ — Final vertical effective stress

$\sigma_{va} := .5 \cdot (\sigma_{vo} + \sigma_{vf})$ — Average vertical effective stress at the midpoint of the layer

$p_a := 2116 \cdot psf$ — Atmospheric Pressure

Settlement

$m := 60$ — Modulus number from Table 5.5, and assuming the soil normally consoidated

$M_t := m \cdot p_a \cdot \left(\frac{\sigma_{va}}{p_a}\right)^{.5} = 75283\, psf$ — Tangent value of constrained modulus of the layer

$$S := \Delta z \cdot \frac{\Delta\sigma_v}{M_t}$$

$\boxed{S = 1.0 \cdot in}$ — Estimated settlement under the new fill is 1 inch. Settlement is expected to occur rapidly in the silt.

TABLE 6
Typical Values of Soil Index Properties

	Particle Size and Gradation				Voids[1]					Unit Weight[2] (lb./cu.ft.)						
	Approximate Size Range (mm)		Approx. D_{10} (mm)	Approx. Range Uniform Coefficient C_u	Void Ratio			Porosity (%)		Dry Weight			Wet Weight		Submerged Weight	
	D_{max}	D_{min}			e_{max} loose	e_{cr}	e_{min} dense	n_{max} loose	n_{min} dense	Min loose	100% Mod. AASHO	Max dense	Min loose	Max dense	Min loose	Max dense
GRANULAR MATERIALS																
Uniform Materials																
a. Equal spheres (theoretical values)	-	-	-	1.0	0.92	-	0.35	47.6	26	-	-	-	-	-	-	-
b. Standard Ottawa SAND	0.84	0.59	0.67	1.1	0.80	0.75	0.50	44	33	92	-	110	93	131	57	69
c. Clean, uniform SAND (fine or medium)	-	-	-	1.2 to 2.0	1.0	0.80	0.40	50	29	83	115	118	84	136	52	73
d. Uniform, inorganic SILT	0.05	0.005	0.012	1.2 to 2.0	1.1	-	0.40	52	29	80	-	118	81	136	51	73
Well-graded Materials																
a. Silty SAND	2.0	0.005	0.02	5 to 10	0.90	-	0.30	47	23	87	122	127	88	142	54	79
b. Clean, fine to coarse SAND	2.0	0.05	0.09	4 to 6	0.95	0.70	0.20	49	17	85	132	138	86	148	53	86
c. Micaceous SAND	-	-	-	-	1.2	-	0.40	55	29	76	-	120	77	138	48	76
d. Silty SAND & GRAVEL	100	0.005	0.02	15 to 300	0.85	-	0.14	46	12	89	-	146[3]	90	155[3]	56	92
MIXED SOILS																
Sandy or Silty CLAY	2.0	0.001	0.003	10 to 30	1.8	-	0.25	64	20	60	130	135	100	147	38	85
Skip-graded Silty CLAY with stones or rk fgmts	250	0.001	-	-	1.0	-	0.20	50	17	84	-	140	115	151	53	89
Well-graded GRAVEL, SAND, SILT & CLAY mixture	250	0.001	0.002	25 to 1000	0.70	-	0.13	41	11	100	140	148[4]	125	156[4]	62	94
CLAY SOILS																
CLAY (30%-50% clay sizes)	0.05	0.5μ	0.001	-	2.4	-	0.50	71	33	50	105	112	94	133	31	71
Colloidal CLAY (-0.002 mm: 50%)	0.01	10Å	-	-	12	-	0.60	92	37	13	90	106	71	128	8	66
ORGANIC SOILS																
Organic SILT	-	-	-	-	3.0	-	0.55	75	35	40	-	110	87	131	25	69
Organic CLAY (30% - 50% clay sizes)	-	-	-	-	4.4	-	0.70	81	41	30		100	81	125	18	62

5.4 SETTLEMENTS OF FOOTINGS ON SANDS, SILTS AND CLAYS BY JANBU'S TANGENT MODULUS METHOD

Janbu (1963, 1967) developed a unified approach for estimating settlements of footings on soil using tangent modulus values to characterize soil compressibility.

In Janbu's method the soil beneath the footing is divided into a number of sublayers, each characterized by a value of constrained tangent modulus (M_t). The settlement of the footing is estimated by summing the reductions in thickness of each of the sublayers beneath the footing, as shown by the expression:

$$\rho = \Sigma \left[\frac{\Delta\sigma_v'}{M_t} \cdot \Delta Z\right] \qquad (5.4.1)$$

where ρ = settlement (any length units)

$\Delta\sigma_v'$ = increase in effective stress within the sublayer due to the load on the footing (stress units -- the same units as M_t). This increase in stress due to the footing load can be estimated using elastic stress distribution theory.

M_t = tangent value of constrained modulus of the soil (stress units -- the same units as $\Delta\sigma_v'$).

ΔZ = sublayer thickness (length units -- the same units as ρ).

Values of constrained tangent modulus vary with the type of soil, its density, whether it is normally consolidated or overconsolidated, and the stresses acting on it before and after the load is applied to the footing.

Values of M_t can be measured using conventional laboratory consolidation tests (oedometer, or one-dimensional compression tests). In the case of natural soils the tests should be performed on good quality undisturbed specimens. In the case of compacted fills, the specimens should be prepared at the same dry density and compaction water content as the fill in the field.

The pressures used in the laboratory tests should cover the range from the initial pressure on the sublayer (before the load is applied) to the final pressure on the sublayer (after the load is applied). As shown in Fig. 5.8, M_t is determined by dividing the stress increment in the sublayer ($\Delta\sigma_v' = \sigma_{vf}' - \sigma_{vo}'$) by the corresponding strain, $\Delta\epsilon_v$.

When laboratory tests are not available for evaluation of M_t, values can be estimated using the information shown in Table 5.5, together with the following equations:

For sands and silts, values of M_t can be estimated using the expression:

$$M_t = m\, p_a \left(\frac{\sigma_{va}'}{p_a}\right)^{0.5} \qquad (5.4.2)$$

where m = dimensionless modulus number shown in Table 5.5

σ_{va}' = average vertical stress = 1/2 ($\sigma_{vo}' + \sigma_{vf}'$) expressed in the same pressure units as M_t and p_a.

p_a = atmospheric pressure, expressed in the same pressure units as M_t and σ_{va}'.

Table 5.5 Values of Modulus Number (m) for Sands, Silts and Clays (after Janbu, 1985)

Sand	Relative Density (Dr)	$\sigma_{vo}' = P_p$	Value of m $\sigma_{vo}' < P_p < \sigma_{vf}'$	$\sigma_{vf}' < P_p$
	30% (Loose)	80 - 160	120 - 300	240 - 500
	50% (Medium)	120 - 240	200 - 400	350 - 700
	70% (Dense)	200 - 400	300 - 700	600 - 1200
Silt	**Porisity (N)**	**$\sigma_{vo}' = P_p$**	**Values of m $\sigma_{vo}' < P_p < \sigma_{vf}'$**	**$\sigma_{vf}' < P_p$**
	50%	25 - 50	40 - 200	120 - 240
	40%	60 - 120	80 - 400	300 - 600
	30%	100 - 200	150 - 700	500 - 1000
Clay	**In Situ Water Content**	**$\sigma_{vo}' = P_p$**	**Value of m $\sigma_{vo}' < P_p < \sigma_{vf}'$**	**$\sigma_{vf}' < P_p$**
	70%	6 - 12	10 - 80	60 - 120
	50%	9 - 18	15 - 120	90 - 180
	30%	15 - 35	25 - 200	150 - 350

Assume normally consolidated.
For n=45%, say m=60

P_p = preconsolidation pressure = highest pressure to which the soil has been subjected in the past.

Objective

Assess nominal and factored bearing resistance of a foundation on rock based on support in GNEISS from borings BB-LSR-101, and -102.

Methodology

Use data from test borings and evaluate the nominal bearing resistance as follows:

1. Bedrock Properties From Test Borings
2. Calculation of Rock Mass Rating
3. Determine Rock Property Constants s and m
4. Calculate Nominal Bearing Resistance of Bedrock q_n

References

1. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 6th edition, 2012. (AASHTO LRFD).

   Note: AASHTO 7th Edition is now in effect, but the coefficients used in the bedrock bearing evaluations are understood to be correlated relative to the older Hoek and Brown 1988 methodology. Therefore, RMR is used for the evaluation per LRFD 6th Edition rather than GSI per LRFD 7th Edition.

2. Wyllie, Duncan C., "Foundations on Rock", Second edition, 1992.

1. Rock Properties

Bedrock properties were obtained from rock core specimens and logs completed for the Lisbon Center Bridge #5007 Project in Lisbon, ME. This calculation is based on the data from borings BB-LSR-101, and -102.

Bedrock Quality

Boring	Run	Length of Core Run (ft)	Rec (%)	RQD %	Joint Spacing Desc.	Corr. Spacing (in)	Aperture Desc.	Corr. Aperture (in)	Joint Weathering
BB-LSR-101	R1	5.0	72%	30%	Close	8	Partially Open to open	0.01-0.1	Sl. Weathered
BB-LSR-101	R2	5.0	100%	44%	Close to Moderate	2.5-24	Open	0.02-0.1	Sl. Weathered
BB-LSR-102	R1	3.5	92%	42%	Close to Moderate	2.5-24	Open	0.02-0.1	Sl. Weathered
BB-LSR-102	R2	3.3	100%	85%	Close to Moderate	2.5-24	Open	0.02-0.1	Sl. Weathered
			Avg RQD	50%					
			St. Dev RQD	21%					

RQD between 29% and 44% representative of rock for BB-LSR-101, and -102 (mean-1 std deviation to max in upper 5 ft)

Bedrock Strength

Boring	Run	Depth of Sample (ft)	Depth of Sample into Rock (ft)	Elev Top of Sample (ft)	UCS (psi)	Modulus (ksi)	Unit Wt (pcf)	Rock Type
BB-LSR-101	R1	19.8	1.0	136.1	9,593	3,876	169	GNEISS
BB-LSR-102	R1	12.1	0.8	143.8	9,900	3,243	171	GNEISS

Select design unconfined compressive strength of 9,000 psi.

2. Calculation of Rock Mass Rating (RMR)

From AASHTO LRFD 6th Ed. Table 10.4.6.4-1, determine the RMR.

Parameter 1- Uniaxial Compressive Strength

$\sigma_{u.r} := 9.0\text{ksi} = 1296\cdot\text{ksf}$

Minimum unconfined compressive strength of rock at BB-LSR-101 is 9,593 psi, 9,000 psi was selected for design.

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_1 := 7$ for $\sigma_{u.r}$= 1080 - 2160 ksf

Parameter 2- Drill Core Quality

Representative RQD from table above: 29-44%; choose 25-50%

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_2 := 8$

Parameter 3- Spacing of Joints

From Boring Logs, generally close to moderately spaced = 2.5 in to 2 feet, Typical spacing was 3 in. to 8 in.

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating

$RR_3 := 10$

Parameter 4- Condition of Joints

From boring logs, moderately hard to hard joint walls and appeared smooth on surface, with typical open joint separation between 0.01 to 0.1 inches., and described slightly weathered.

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_4 := 20$

Parameter 5- Ground Water Conditions

Hydrostatic Conditions- Water under moderate pressure considering bottom of tremie seal may be well below static water level

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_5 := 4$

Parameter 6-Adjustment for joint orientation

The joint sets are generally low angle and generally smooth and open. Orientation of low angle joints is unlikely to be unfavorable considering that footing will not be near a steep exposed rock face. Therefore the joint orientation is considered Fair.

From AASHTO LRFD Table 10.4.6.4-2

Relative Rating $RR_6 := -7$

Total RMR Rating

$$RMR := RR_1 + RR_2 + RR_3 + RR_4 + RR_5 + RR_6$$

$$RMR = 42$$

From AASHTO LRFD Table 10.4.6.4-3 RMR= 41 to 60 is indicative of Fair Rock Quality

3. Determine Rock Property Constants s and m

Use AASHTO LRFD 6th Ed. Table 10.4.6.4-4 to develop empirircal rock property constants

Gneiss is categorized as rock type E, Coarse grained polyminerallic metamorphic, RMR=42, using s and m values interpolated from the logarithmic trend of plotted values from AASHTO Table 10.4.6.4-4 (plots on sheet 8).

$m := 0.400$

$s := 0.000065$

4. Calculate Nominal and Factored Bearing Resistance of Bedrock q_n and q_R

From Wyllie "Foundations on Rock"

Eq. 5.4 Pg.138

$$q_n := C_{f1} \cdot \sqrt{s} \cdot \sigma_{u.r} \cdot \left[1 + \sqrt{m \cdot \left(s^{-\frac{1}{2}}\right) + 1}\right]$$

Where

$C_{f1} := 1.0$ From Wyllie Table 5.4 Pg. 138 Correction factor for foundation shape for rectangular foundation:

$s = 0.000065$

$m = 0.4$

$\sigma_{u.r} = 9 \cdot ksi$

For L/B>6, use factor C_{fl}=1.0,
For L/B=1, use factor C_{fl}=1.12, therefore,
For conservatism, assume long strip, lowest C_{fl}.

Nominal Bearing Resistance

$$q_n := C_{f1} \cdot \sqrt{s} \cdot \sigma_{u.r} \cdot \left[1 + \sqrt{m \cdot \left(s^{-\frac{1}{2}}\right) + 1}\right]$$

$q_n = 84.8 \cdot ksf$ Say 85ksf

Factored Bearing Resistance

Bearing Resistance Factor is specified in Table 10.5.5.2.2-1

$\phi_b := 0.45$ Footing on rock

$q_R := \phi_b \cdot q_n$

$q_R = 38.2 \cdot ksf$ Say 38 ksf

Reference:I:\Mathcad\units.xmcd

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

	Parameter					
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

	Parameter					
5	Ground water conditions (use one of the three evaluation criteria as appropriate to the method of exploration)	Inflow per 30 ft. tunnel length	None	<400 gal./hr.	400–2000 gal./hr.	>2000 gal./hr.
		Ratio = joint water pressure/ major principal stress	0	0.0–0.2	0.2–0.5	>0.5
		General Conditions	Completely Dry	Moist only (interstitial water)	Water under moderate pressure	Severe water problems
	Relative Rating		10	7	4	0

Table 10.4.6.4-2 Geomechanics Rating Adjustment for Joint Orientations.

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

Table 10.4.6.4-3 Geomechanics Rock Mass Classes Determined From Total Ratings.

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

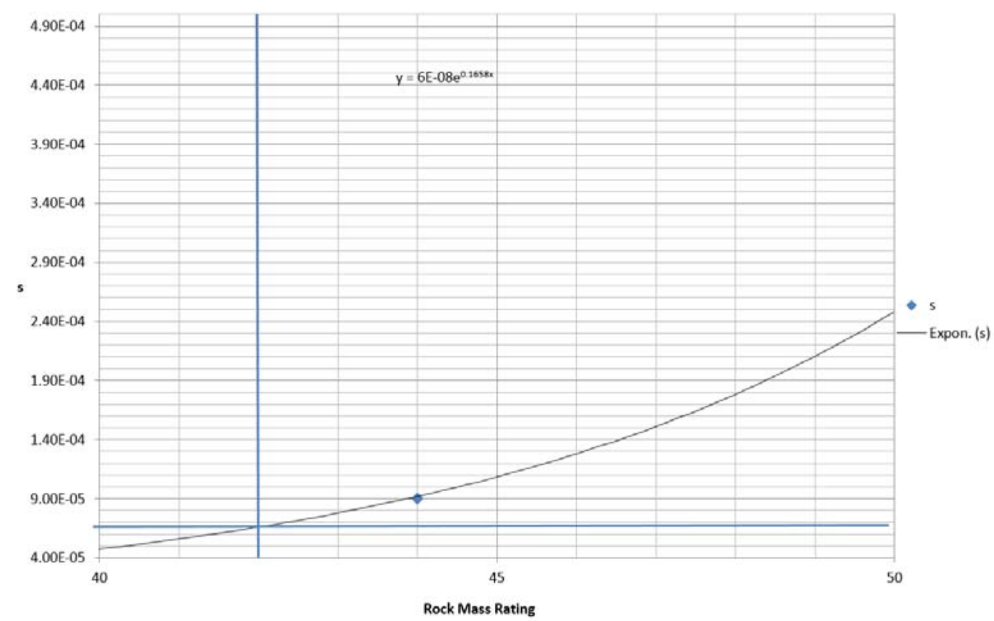
Table 10.4.6.4-4 Approximate relationship between rock-mass quality and material constants used in defining nonlinear strength (*Hoek and Brown, 1988*)

Rock Type

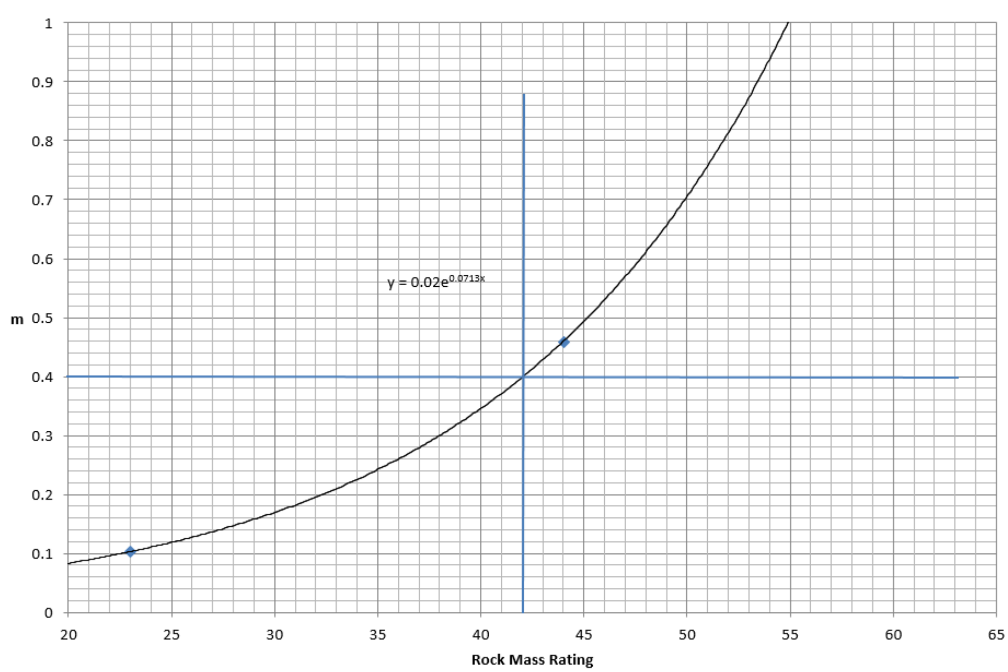
A = Carbonate rocks with well developed crystal cleavage—*dolomite, limestone and marble*
B = Lithified argrillaceous rocks—*mudstone, siltstone, shale and slate (normal to cleavage)*
C = Arenaceous rocks with strong crystals and poorly developed crystal cleavage—*sandstone and quartzite*
D = Fine grained polyminerallic igneous crystalline rocks—*andesite, dolerite, diabase and rhyolite*
E = Coarse grained polyminerallic 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}

s for Rock Type E



m for Rock Type E



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Engineers and Scientists

JOB: 09.0025944.00 Lisbon Center
SUBJECT: Lateral Earth Pressures
SHEET: 1 OF 2
CALCULATED BY A. Blaisdell 01/22/18
CHECKED BY C.Snow 01/22/18

Subject:

Evaluate lateral earth pressure coefficients

References:

1. MaineDOT Bridge Design Guide, Chapter 3
2. AASHTO LRFD Bridge Design Specifications, 7th Edition (2014, with 2015 and 2016 Interims)

Input Parameters:

$\beta := 0\text{deg}$ Angle of backfill to the horizontal

$\theta := 90\text{deg}$ Angle of backface of wall to the horizontal

$\phi := 32\text{deg}$ Effective angle of internal friction *(Granular borrow, Soil Type 4, BDG Table 3-3)*

$\delta_f := 19.5\text{deg}$ Average value, precast concrete against clean sand/silty sand-gravel mixture *(AASHTO LRFD Table 3.11.5.3-1)*

Earth Pressure Coefficients:

Thermal expansion of the bridge will cause the superstructure backwall (end diaphragm) to move towards the backfill, which will result in earth pressures ranging from at-rest to passive earth pressure. Therefore, the end diaphragms should be designed for passive earth pressure. The semi-integral abutments and wingwalls will be free to rotate and therefore should be designed for active earth pressure.

Passive Earth Pressure (End Diaphragms)

Per BDG Section 5.4.2.11, developing full passive pressure requires that ratio of lateral abutment movement (y) to abutment height (Hb) exceeds 0.005. The structural engineer should use the Coloumb Kp coefficient for y/Hb > 0.005 and may use Rankine Kp for y/Hb < 0.005.

Coloumb Passive Earth Pressure Coefficient

$$K_{pc} := \frac{(\sin(\theta - \phi))^2}{\left[(\sin(\theta))^2 \cdot \sin(\theta + \delta_f) \cdot \left[1 - \sqrt{\frac{(\sin(\phi + \delta_f) \cdot \sin(\phi + \beta))}{(\sin(\theta + \delta_f) \cdot \sin(\theta + \beta))}}\right]^2\right]}$$

$\boxed{K_{pc} = 6.73}$

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Rankine Passive Earth Pressure Coefficient

$$K_{pr} := \frac{1 + \sin(\phi)}{1 - \sin(\phi)}$$

$$\boxed{K_{pr} = 3.25}$$

Active Earth Pressure (Abutments and Wingwalls)

Rankine Active Earth Pressure Coefficient

$$K_{ar} := \cos(\beta) \cdot \frac{\left[\cos(\beta) - \sqrt{(\cos(\beta))^2 - (\cos(\phi))^2}\right]}{\left[\cos(\beta) + \sqrt{(\cos(\beta))^2 - (\cos(\phi))^2}\right]}$$

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