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GEOTECHNICAL DESIGN REPORT
MATTAWAMKEAG BRIDGE NO. 2522
ROUTE 2 OVER MATTAWAMKEAG RIVER
MAINE DOT WIN 23234.01
MATTAWAMKEAG, MAINE

July 2020
09.0025975.01

Prepared for:
HNTB Corporation
Westbrook, Maine

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VIA EMAIL

July 10, 2020
File No. 09.0025975.01

Ms. Lori Driscoll, P.E.
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Re: Geotechnical Design Report
Replacement of Mattawamkeag Bridge No. 2522
Route 2 over Mattawamkeag River
MaineDOT WIN 23234.01
Mattawamkeag, Maine

Dear Lori:

We are pleased to provide this Geotechnical Design Report, which includes geotechnical design recommendations for the replacement of Mattawamkeag Bridge No. 2522, Route 2 over Mattawamkeag River in Mattawamkeag, Maine. Our work was completed in accordance with GZA GeoEnvironmental, Inc.'s Project Contract for Task Order No. 001.01 which incorporates our July 31, 2019 proposal, HNTB File No. 75774-DS-001-001-E008, dated August 16, 2019, our Master/Task Order Agreement dated August 16, 2019, and the attached Limitations contained in **Appendix A** of this report.

It has been a pleasure serving HNTB on this phase of the project, and we look forward to our continued work with you through project completion. If you have any questions regarding the report, please do not hesitate to contact the undersigned.

Very truly yours,

GZA GEOENVIRONMENTAL, INC.

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Attachment: Geotechnical Design Report



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1.0 INTRODUCTION

This report presents the results of the geotechnical evaluation by GZA GeoEnvironmental, Inc. (GZA) for the subject project. Our services were completed in accordance with GZA's Project Contract for Task Order No. 001.01 which incorporates our July 31, 2019 proposal, HNTB File No. 75774-DS-001-001-E008, dated August 16, 2019, our Master/Task Order Agreement dated August 16, 2019, and the attached Limitations contained in **Appendix A** of this report.

GZA is providing geotechnical engineering services as a Subconsultant to HNTB, who is under contract with Maine Department of Transportation (MaineDOT) for design of the proposed bridge replacement.

1.1 BACKGROUND

The project includes replacement of the Mattawamkeag Bridge No. 2522 carrying Route 2 over Mattawamkeag River in Mattawamkeag, Maine, the location of which is shown on **Figure 1**. Our understanding of the existing bridge is based on drawings for the Mattawamkeag Bridge over Mattawamkeag River, dated February 28, 1962. The Mattawamkeag Bridge was built in 1928 and widened in 1962. It is a scour-critical, approximately 355-foot-long, seven-span, concrete T-beam bridge with a concrete deck on footings embedded about 4 feet below the lowest riverbed elevation at each footing.

We understand plans are to construct a replacement bridge consisting of an approximately 387-foot-long, 45-foot-wide, three-span bridge, supported by stub abutments and two river piers. The bridge will be constructed on the existing alignment. We understand a temporary work trestle is planned upstream (east) of the existing bridge and a temporary bridge is planned to the east of the work trestle. The proposed south abutment (Abutment 1) is planned to be approximately 30 feet south of the existing abutment, and the proposed north abutment (Abutment 2) is planned to be less than 10 feet north of the existing abutment.

The Route 2 approach roadway elevation is proposed to be within about 1 foot of existing grades. Relatively minor filling, up to about 8 feet high, will be needed outside of the roadway limits for the proposed 10-foot embankment widening on the east side of the bridge approaches.

A 3-foot-thick riprap layer is planned to be placed over the river banks in front of each abutment, extending down the riverbed level along the south river bank and to the location of the existing pier along the north river bank.

1.2 OBJECTIVES AND SCOPE OF SERVICES

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

- Conducted a site visit to observe surficial conditions and reviewed mapped surficial and bedrock geology of the site;



- Reviewed subsurface data collected by GZA during the preliminary subsurface exploration program, which consisted of three test borings;
- Coordinated and observed a supplemental subsurface exploration program, consisting of four test borings, to further evaluate subsurface conditions;
- Conducted a laboratory testing program to evaluate engineering and index properties of the site soils and bedrock;
- Conducted geotechnical engineering analyses for soil and bedrock properties; stability and settlement of approach embankments; frost susceptibility and drainage of approach embankments; AASHTO LRFD load and resistance factors associated with geotechnical design elements; nominal resistance of pile foundations; pile drivability; lateral pile evaluations; spread footing bearing and lateral resistance; and seismic design considerations;
- Developed geotechnical engineering recommendations including foundation design recommendations for driven piles, spread footings bearing on rock, lateral earth pressures and seismic design parameters; recommended construction considerations; and
- Prepared this report summarizing our findings and design recommendations.

2.0 SUBSURFACE EXPLORATIONS

GZA completed a preliminary design exploration program in 2018 consisting of three test borings and a final design exploration program in 2019 consisting of four test borings. The as-drilled boring locations were surveyed by MaineDOT and provided to GZA, and are shown on **Figure 2**. Elevations referenced in this report are in feet and refer to North American Vertical Datum of 1988.

New England Boring Contractors of Hermon, Maine provided drilling services and coordinated utility clearance. Borings were drilled using 3-, 4- and 5-inch casing, and drive- or spin-and-wash drilling techniques, as noted on the boring logs. Standard penetration testing (SPT) and split spoon sampling were performed continuously or at standard 5-foot intervals using a 24-inch-long, 1-3/8-inch inside diameter sampler. Bedrock cores were obtained using NX or NQ2 wire-line coring equipment in each test boring. Photographic logs of the recovered rock core specimens are included in **Appendix C**. GZA personnel monitored the drilling work and prepared logs of each boring that are included in **Appendix B**. Additional details of each program are described below.

2.1 PRELIMINARY BORINGS

Borings BB-MMR-101 through BB-MMR-103 were drilled between May 22 and May 24, 2018. The test borings were completed using a Mobile B-53 drill rig. Test borings were drilled to depths of 30 to 63 feet below ground surface (bgs). Borings were terminated approximately 10 to 20 feet into bedrock.

Standard Penetration Testing (SPT) was conducted using automatic hammer NEBC No. 2, which had a rated hammer energy transfer ratio of 0.677 at the time of drilling. The borings were generally backfilled with ¾-inch crushed stone and/or soil cuttings and topped with asphalt cold patch in roadway areas.



2.2 FINAL DESIGN PHASE BORINGS

Borings BB-MMR-201 through BB-MMR-204 were drilled between September 25 and October 1, 2019. Two borings were drilled through the existing bridge in the river to the left and right of each proposed pier. The borings were drilled using a truck-mounted Mobile B-53 drill rig.

The borings were drilled to depths of approximately 20 to 22 feet bgs and terminated approximately 10 to 13 feet into bedrock. SPTs were conducted using automatic hammer NEBC B-24, which had a rated hammer efficiency factor of 0.937 at the time of drilling.

3.0 LABORATORY TESTING

GZA retained Thielsch Engineering of Cranston, Rhode Island to complete a laboratory testing program to assess the gradation and index properties of the soil and bedrock. The testing program included:

Soil

- 12 gradation analysis / MaineDOT Frost Classification / Unified Soil Classification System (USCS) assessments;
- Seven moisture content tests; and
- One Atterberg limits analysis.

Rock

- Five unconfined compression/ secant modulus tests.

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

4.0 SUBSURFACE CONDITIONS

4.1 SURFICIAL AND BEDROCK GEOLOGY

Based on available surficial geologic mapping¹, the surficial unit in the vicinity of the site consists of coarse-grained facies of glaciomarine deposits, described as sand, gravel and minor amounts of silt.

Based on available bedrock geologic mapping², the site is mapped near the contact of two bedrock units. The site is mapped within and along the southeast boundary of the Madrid Formation (Dsm), which is a metasedimentary formation consisting of quartzite and calc-silicate rock. An undifferentiated unit of

¹ Thompson, W.B. and Borns, Jr., H.W., 1985. Surficial Geologic Map of Maine: Maine Geological Survey, map, scale 1:500,000.

² Osberg, P.H., Hussey, A.M., and Boone, G.M., 1985, Bedrock geologic map of Maine: Maine Geological Survey, Dept. of Conservation, scale 1:500,000.



metamorphic and sedimentary rock (Spu) is mapped immediately southeast of the site, in which the major constituents are identified as slate and sandstone, and the minor constituents are identified as graywacke and conglomerate.

4.2 SUBSURFACE PROFILE

Three soil units were encountered beneath asphalt pavement or topsoil and above bedrock at the site: Fill, River Bottom Deposit, and Glaciomarine Deposit. The approximate thicknesses and generalized descriptions of the subsurface 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**. An interpretive subsurface profile based on the test boring results is presented as **Figure 3**, Interpretive Subsurface Profile.

Soil Unit	Approximate Encountered Thickness (ft)	Generalized Description
Fill	12 to 13	Grey/brown, loose to very dense, fine to coarse SAND, with varying amounts of Gravel, little to trace Silt. Probable cobbles, boulders and granite blocks are present throughout, specifically observed in BB-MMR-101 at 3.8-4.5 ft and 11-13.5 ft and in BB-MMR-103 at 9-13.5 ft. USCS: SP-SM, SM; MaineDOT Frost Classification = II <i>Encountered in borings BB-MMR-101 and BB-MMR-103 (behind existing abutments).</i>
River Bottom Deposit	2 to 3	Varying <u>from</u> Grey to brown, loose to very dense, fine to coarse GRAVEL, some medium to coarse Sand, trace Silt, <u>to</u> loose to medium dense, fine to coarse SAND, little Gravel, little to trace Silt, with wood fragments. USCS: GW, SW, SP-SM, MaineDOT Frost Classification = 0 <i>Encountered in all river borings.</i>
Glaciomarine Deposit	7 to 40	Varying <u>from</u> Grey to brown-grey, dense to very dense, Gravelly fine to coarse SAND <u>to</u> Sandy GRAVEL, some to trace Silt, with possible cobbles and boulders. Significant cobble/boulder presence at bottom of stratum in BB-MMR-103 at 48-53'. USCS: GW-GM, SP-SM, GW, SM; MaineDOT Frost Classification = 0 to III <i>Encountered in all borings.</i> <i>In boring BB-MMR-103 only: 5-foot layer of grey, soft to medium stiff Clayey SILT, between Fill and Glaciomarine Sand/Gravel (13.5 to 18.5 feet bgs).</i> USCS: ML, CL; Maine DOT Frost Classification = II to IV
Encountered Top of Bedrock Elevation	South Abutment (BB-MMR-101): Approx. El. 175 (34.4 feet bgs) Pier 1 (BB-MMR-201 & -202) Approx. El. 175 to 176 (8 to 10 feet bgs) Pier 2 (BB-MMR-203 & -204) Approx. El. 172 to 173 (10 to 11 feet bgs) North Abutment (BB-MMR-103): Approx. El. 156 (53 feet bgs)	

4.2.1 Bedrock

Bedrock was cored in each test boring and was identified as Slate and Schist. Slate was encountered in borings BB-MMR-101, -102, and -201 through -204 and was generally described as hard to very hard, fresh to moderately weathered, aphanitic to medium grained, and grey, with calcite stringers. The joints are extremely close to wide, vertical to low angle, fresh to decomposed, planar to undulating, smooth



to rough, and tight to moderately wide, with occasional silt infilling. A 3-foot zone of pitted slate was encountered in one boring, BB-MMR-204, between 2.4 and 5.4 feet below top of rock. The Rock Quality Designation (RQD) in the Slate ranged from 0 to 100 percent. RQD values in the upper approximately 7 feet of Slate ranged from 0 to 89 percent (weighted average of 43 percent), corresponding to a Rock Mass Quality of Very Poor to good, and the RQD increased to between 33 and 100 percent (weighted average of 67 percent) between 7 and 22 feet into the rock, corresponding to a Rock Mass Quality of Poor to Excellent.

Schist was encountered in the bottom of boring BB-MMR-102 and in BB-MMR-103 and was generally described as moderately hard to hard, fresh to moderately weathered, aphanitic to fine grained, and grey to grey-white. The joints are extremely close to moderately spaced, low to high angle, planar to undulating, smooth to rough, decomposed to fresh, and moderately wide to open, with silt infilling in the upper 3 feet of rock. The RQD in the Schist ranged from 0 to 100 percent (weighted average of 66 percent), corresponding to a Rock Mass Quality of Very Poor to Excellent.

Five laboratory unconfined compressive strength / secant modulus tests were conducted on bedrock core samples of Slate and Schist. The test results are included in **Appendix C**. The testing yielded:

- Slate (three tests): Unconfined compressive strengths ranging from 9.9 to 25.3 kips per square inch (ksi) and Young's modulus values ranging from 3,680 to 6,230 ksi;
- Pitted Slate sample (one test): Unconfined compressive strength of 2.0 kips per square inch (ksi) and Young's modulus value of 2,700 ksi; and
- Schist (one test): Unconfined compressive strength of 13.1 ksi and Young's modulus value of 2,870 ksi.

4.2.2 Groundwater

Groundwater depth was measured in boring BB-MMR-101 at approximately 10.4 feet bgs, corresponding to El. 198.6. Groundwater levels in the borings were measured during or immediately after drilling and may have been affected by drilling procedures, which included introduction of water for drilling purposes.

The groundwater observations were made at the times and under the conditions stated in the boring logs. Fluctuations in groundwater level occur due to variations in season, precipitation, river level and construction activities in the area. Consequently, water levels during construction are likely to vary from those encountered at the time the observations were made.

5.0 ENGINEERING EVALUATIONS

5.1 GENERAL

GZA conducted geotechnical engineering evaluations in accordance with *2017 AASHTO LRFD Bridge Design Specifications, 8th Edition* (herein designated as AASHTO) and the *MaineDOT Bridge Design Guide, 2003 Edition*, with updates through 2018 (MaineDOT BDG). The sections that follow describe the



evaluations and the geotechnical basis for each element. Supporting calculations are included in **Appendix F**.

5.2 APPROACH EMBANKMENTS

The proposed bridge approaches will be constructed with maximum side slope angles of approximately 2 horizontal to 1 vertical (2H:1V), or flatter. The proposed embankments will be within about a foot of existing grades with the proposed roadway. We estimate that up to 8 feet of fill will be placed over the existing embankment slope at the northeast bridge approach to facilitate the proposed sidewalk being added along the east side of the bridge and road.

We anticipate that the embankments will be constructed over primarily dense to very dense granular Glaciomarine Deposit supporting soil. Due to the typical strength and low compressibility, embankment settlement and global stability are not considered a concern for this supporting soil. Boring BB-MMR-103 encountered approximately 5 feet of soft to medium stiff silt and clay soil directly beneath the existing embankment fill. Due to its limited thickness, this layer is anticipated to have negligible effect on embankment settlement or global stability of the widened embankment.

5.3 SCOUR CONSIDERATIONS

Available laboratory data for the soil at and near riverbed elevations were evaluated to estimate D50 values for use by others for scour depth evaluation, as summarized in the table below.

Boring & Sample ID	Depth (feet)	Elevation (ft NAVD 88)	Water Content (%)	D ₅₀ (mm)	Classification	
					USCS	AASHTO
BB-MMR-101, 4D	15-17	193.9-191.9	--	3.7	SP-SM	A-1-a
BB-MMR-101, 6D	25-27	183.9-181.9	--	0.5	SM	A-2-4
BB-MMR-102, 1D	0-2	184.7-182.7	15.4	12	GW	A-1-a
BB-MMR-102, 2D	5-7	179.7-177.7	7.8	1.7	GM	A-1-b

5.4 EVALUATION OF FOUNDATION TYPES

5.4.1 Abutment Foundations

We understand that the proposed stub abutment configuration would place the bottom of a footing or pile cap approximately 17 feet below the pavement elevation, corresponding to approximately El. 192. The test borings indicate that dense to very dense Glaciomarine sand and/or gravel would be exposed at or within a few feet of this elevation (potentially below some Fill or Glaciomarine clay/silt). Bedrock was encountered at approximately El. 175 and El. 156 at borings BB-MMR-101 and -103, respectively, which corresponds to depths of 17 and 35 feet to rock at Abutments 1 and 2, respectively, from the bottom of the stub abutment foundation.

Feasible foundation types from a geotechnical support standpoint include spread footings bearing on dense to very dense granular soil or driven piles. However, based on the estimated scour depth, spread footings on soil are considered susceptible to undermining by scour. The preferred foundation type for



the abutments are H-piles driven to refusal on or near bedrock since they may be designed to accommodate the anticipated scour.

5.4.2 Pier Foundations

The interpreted top of rock profile shown on **Figure 3** indicates a relatively level top of rock surface within the footprint areas of Piers 1 and 2. Based on the available top of bedrock elevations, we estimate the top of rock beneath the southern and northern piers would be approximately 10 feet and 13 feet below mudline, respectively.

We anticipate that spread footings constructed inside cofferdams with tremie seals on bedrock are the most cost-effective alternative for piers with the anticipated bedrock depths. Therefore, we conclude spread footings on tremie seals bearing on bedrock are the preferred foundation type for the piers.

5.5 LOAD AND RESISTANCE FACTORS

AASHTO LRFD load factors should be applied to horizontal earth pressure (EH), vertical earth pressure (EV), earth surcharge (ES), and live load surcharge (LS) loads, using the load factors for permanent loads (γ_p) provided in LRFD Table 3.4.1-2 for strength limit state foundation design.

The recommended LRFD resistance factors for strength limit state design of foundations were derived from LRFD Tables 10.5.5.2.2-1, 10.5.5.2.3-1 and 10.5.5.2.4-1 and are presented in the following table.

GEOTECHNICAL RESISTANCE FACTORS – STRENGTH LIMIT STATE			
Foundation Resistance Type	Method/Condition	Resistance Factor (ϕ)	AASHTO Reference
Bearing	Footing on Rock	0.45	10.5.5.2.2-1
Sliding	Footing on Rock	0.80	10.5.5.2.2-1
Axial Pile Resistance	Driven Pile, Installed with Dynamic Testing	0.65	10.5.5.2.3-1

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

Structural resistance of the piles should be checked at the strength limit state considering a resistance factor $\phi_c=0.50$, per AASHTO LRFD Article 10.7.3.2.3 for hard driving conditions. Per LRFD Article 6.5.4.2, the axial resistance factor $\phi_c=0.7$ and the flexural resistance factor $\phi_f=1.0$ should be applied when evaluating the combined axial and flexural resistance of the pile in the interaction equation (AASHTO LRFD Eq. 6.9.2.2-1).

5.6 PILE DESIGN CONSIDERATIONS

5.6.1 Pile Type and Loading

We understand steel HP14x89 H-section piles, including plumb piles and battered piles, are planned to support each abutment. HNTB indicated that the pile design is controlled by the lateral loading on the battered pile, which requires that a 600 kips nominal resistance be developed and results in excess vertical load capacity of the pile. This results in a required maximum factored geotechnical resistance



of 390 kips, calculated by multiplying the nominal pile load by a geotechnical resistance factor of 0.65, per LRFD Table 10.5.5.2.3-1, for piles installed using dynamic pile testing with signal-matching analysis to assess nominal geotechnical pile resistance.

5.6.2 Pile Design Considerations

Considering the relatively thin overburden thickness at the abutments, we anticipate that piles will be driven to or near the bedrock surface. The skin friction resistance was evaluated for use as an input in drivability analyses using the computer analytical software *APile* by Ensoft. Considering the variability in top of rock between the two abutments, GZA evaluated two possible conditions to characterize the range of anticipated pile behavior. The pile lengths used in the analysis were 17 and 35 feet for Abutment 1 and Abutment 2, respectively. To achieve a nominal resistance of 600 kips, we estimate the side friction to represent approximately 10 and 15 percent of the nominal resistance for Abutments 1 and 2, respectively. The calculated side resistance was used as an input in wave equation analyses.

Since the piles will gain support largely in end bearing, there is no reduction for group interaction in axial compression. Axial tensile geotechnical (uplift) resistance was not evaluated because the structural loads provided by HNTB do not include uplift loading on the piles. By utilizing steel H-piles for support of the abutments, total and differential settlement will be limited to elastic compression of the piles and should be less than ½ inch.

5.6.3 Preliminary Wave Equation Analysis

GZA completed preliminary wave equation analyses to assess the drivability of an HP 14x89 pile with a nominal geotechnical resistance of 600 kips at the abutments. Analyses were completed using a Delmag D30 diesel hammer with a ram weight of 6,600 pounds and a maximum rated energy of 82,625 foot-pounds (ft-lbs).

To assess possible variations in transfer of energy to the pile tip, the 17-foot-long pile was assumed to encounter very hard driving conditions (toe quake of 0.04) and the 35-foot-long pile was assumed to encounter moderately hard driving conditions (toe quake of 0.07). The results are summarized below.

SUMMARY OF WEAP ANALYSES					
Pile Analysis and Type	Embedded Pile Length	Driving System¹	Required Nominal Geotechnical Resistance (kips)	Max Driving Stress (ksi)	Final Penetration Resistance (blows per inch)
Abutment 1 HP 14x89	17 feet	Delmag D 30 (Fuel setting 3, 80% of maximum pressure)	600	43	8
Abutment 2 HP 14x89	35 feet	Delmag D 30 (Fuel setting 2, 90% of maximum pressure)	600	43	9

Since the driving stresses do not exceed the limiting driving stress of 45 ksi for ASTM A572 steel (50 ksi yield stress), and the calculated penetration resistance is within the MaineDOT preferred range of 6 to 15 blows per inch, the analyzed hammer system is judged acceptable to install the piles to the required



nominal resistance noted. Results of the preliminary wave equation analyses are provided in **Appendix E**.

5.7 SPREAD FOOTING DESIGN CONSIDERATIONS

5.7.1 Footing Bearing Resistance

Nominal and factored bearing resistances were calculated for bedrock-bearing footings using the Rock Mass Rating- (RMR-) based empirical correlation presented in “Foundations on Rock,” by Duncan Wyllie. 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 (8th) Edition of the AASHTO Design Specifications does not include the RMR formulation included in the previous version (6th Edition). However, Articles C10.4.6.4 and 10.6.2.6.2 of the 8th 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 to develop foundation design parameters at the abutment locations. The unconfined compressive strength from the laboratory tests varied from 2.0 to 25.3 ksi. To select a representative strength for use in design, GZA considered an average of the lowest two values as suitably conservative to represent the overall rock mass that will support the footings. This results in a representative unconfined compressive strength of 5.9 ksi. It is noted that 3 of 5 tests had measured strengths of 13.1 to 25.3 ksi, indicating the typical rock mass strength is likely higher than the representative value. The remaining test result was 2.0 ksi and was judged to be a local anomaly encountered over only 3 feet of core in only one of the four borings. 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
Slate/Schist	64	5.9	49	0.26	0.0002

Based on these parameters, the calculated nominal bearing resistance is 66 kips per square foot (ksf), resulting in a factored bearing resistance of 30 ksf for the strength limit state. Supporting calculations are provided in **Appendix E**.

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 anticipated to experience ½ inch or less of elastic settlement.

5.8 SEISMIC DESIGN CONSIDERATIONS

The subsurface profile for seismic design includes the approach fills (including backfill behind abutments), River Bottom Deposit, and Glaciomarine Deposits overlying bedrock. Seismic site class was determined in general accordance with LRFD Table C3.10.3.1, considering the average SPT N-values in granular soils which encompasses most of the soil encountered in the borings. The average SPT N-value is generally between 15 and 50 blows per foot in the abutment borings, which is the range defining Site Class D. Therefore, the bridge is assigned to Site Class D.



The available subsurface data indicates that the natural materials encountered at the site are sufficiently cohesive or dense that the potential for liquefaction is low.

5.9 LATERAL EARTH PRESSURE

Article 3.6.4 of the BDG states that abutments with a height of 5 feet or more should be assumed to experience sufficient horizontal movement of the top of the wall to develop active conditions due to structural deformation of the stem and rotation of the foundation. For the proposed approximately 14-foot abutment height (measured above the pile cap), this would correspond to about ¼ inch of lateral movement of the top of the abutment. The structural engineer should confirm that this amount of deflection is anticipated.

Based on Article 3.6.4 of the BDG, we conclude active earth pressure is appropriate for design of the abutments. Based on Figure C3.11.5.3-1 of LRFD, the abutment is considered to be a short-heeled wall. Therefore, Coulomb theory should be used to calculate active earth pressures. 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 6.3** of this report.

5.10 FROST PROTECTION

Fill soils are anticipated to be present at the abutments and embankments, either as existing fill or imported backfill. Based on the MaineDOT BDG, Section 5.2.1, the Freezing Index for the site is 1,910, and with low-moisture-content (<10 percent) soils, the estimated depth of frost penetration is approximately 7.8 feet.

We understand that pile cap depths were limited to approximately 4 feet below the bottom of the riprap layer to limit the abutment height and pile loads, corresponding to pile cap bearing elevations of El. 192.5 and El. 194.5 for Abutment 1 and Abutment 2, respectively. To limit the potential for frost to exert expansive pressure against the bottom of the pile cap, we recommend the use of ¾-inch crushed stone as non-frost-susceptible material beneath the pile caps, extending to at least 7.8 feet below the nearest ground exposed to freezing. This is considered to occur at the base of any surficial riprap layer.

6.0 RECOMMENDATIONS

6.1 EMBANKMENT DESIGN CONSIDERATIONS

Typical embankment side slopes should be designed with MaineDOT-typical slope angles of 2H:1V or flatter, except in front of abutments where the typical 1.75H:1V slope angle will be used with a riprap surface treatment. Earth slopes should be provided with loam and seed for permanent erosion protection. Riprap should also be provided for scour protection where the embankment side slope angle will be 2H:1V and the slopes will be near or below typical water levels in the Mattawamkeag River.

6.2 SEISMIC DESIGN

The peak ground acceleration coefficient, short- and long-period spectral acceleration coefficients were interpolated from the AASHTO design guide maps (3.10.2.1-1 through -21 as appropriate). Based on the



site coordinates, the software provided the recommended AASHTO Response Spectra (Site Class D) for a 7 percent probability of exceedance in 75 years. These results are summarized for the site as follows:

SITE CLASS D SEISMIC DESIGN PARAMETERS	
Parameter	Design Value
F _{pga}	1.6
F _a	1.6
F _v	2.4
A _s (Period = 0.0 sec)	0.106 g
S _{Ds} (Period = 0.2 sec)	0.248 g
S _{D1} (Period = 1.0 sec)	0.113 g

6.3 ABUTMENT AND WINGWALL DESIGN

- Backfill between new abutments and wingwalls and a 1.5H:1V plane extending up from the bottom of the abutment to the pavement subgrade should consist of MaineDOT 703.19 Granular Borrow for Underwater Backfill, MaineDOT BDG Type 4 soil. Recommended soil properties for Type 4 soils are as follows:
 - Internal Friction Angle of Soil = 32°
 - Soil Total Unit Weight = 125 pcf
 - Coulomb Active Earth Pressure, $K_a = 0.28$ (use for design of abutment and wingwalls)
- Live load surcharge should be applied as a uniform lateral surcharge pressure using the equivalent fill height (H_{eq}) values developed in accordance with LRFD Section 3.11.6.4, based on the abutment/wingwall height and distance from the wall backface to the edge of traffic. A minimum H_{eq} of 2 feet is recommended.
- Foundation drainage should be provided in accordance with Section 5.4.1.9 of the MaineDOT BDG. We recommend the use of French drains on the uphill side of abutments and wing walls to prevent buildup of differential hydrostatic pressure. The drains should be sloped to drain by gravity and should outlet through a series of 4-inch-diameter weep holes, spaced approximately 10 feet center-to-center.

6.4 RECOMMENDATIONS FOR FOUNDATIONS

6.4.1 Pile and Pile Cap Design

- The proposed abutments may be supported on HP14x89 ASTM A572, Grade 50 steel (50 ksi yield stress) H-piles driven to the required nominal resistance, anticipated to be developed primarily end-bearing on the bedrock surface.
- To limit driving damage, the steel H-piles should be fitted with cast steel driving tips in accordance with MaineDOT Standard Specification Section 501.10 – Pile Tips.
- Pile installation should be controlled using wave equation analysis and field logging of the pile installation with final penetration resistance based on dynamic pile testing with signal matching analysis.



- The piles should be driven to a nominal resistance of 600 kips to provide the required nominal resistance for battered piles provided by HNTB. This results in excess axial load capacity based on the maximum factored axial load of 299 kips.
- Preliminary wave equation analyses indicate that the piles can be driven to the required nominal resistance using a diesel hammer with a rated energy of about 82,625 ft-lbs for the anticipated 17- to 35-foot-long, ASTM A572 Grade 50 HP14x89 piles without exceeding the allowable driving stress of 45 ksi (0.9F_y for 50 ksi steel), and with a final penetration resistance of 8 to 9 blows per inch, which is within the MaineDOT range of 6 to 15 blows per inch.
- The pile tip elevations used in the drawings should correspond to the bedrock elevations encountered in the borings (approximately El. 175 at Abutment 1, and approximately El. 156 at Abutment 2). A provision should be included in the drawings to account for variability in the top of rock surface and the potential for piles to penetrate several feet into the bedrock.
- We recommend that the first battered pile be dynamically tested at each abutment at the end of initial drive to assess driving stress and establish the penetration resistance criteria to achieve the required nominal resistance for the production piles. The plans should also require a 24-hour restrike test on each test pile, to assess potential relaxation.
- Piles should be spliced in accordance with MaineDOT Section 501.047.
- Approach slabs should be constructed at each abutment to smooth the transition from the approach embankment to the bridge.
- Pile caps should have a minimum of 7.8 feet of equivalent embedment, measured below the bottom of the riprap layer, to limit frost-induced heave forces against the bottom of the cap. This equivalent embedment can be developed through a combination of 4 feet of pile cap embedment and at least 3.8 feet of ¾-inch crushed stone (MaineDOT Item 703.13). The crushed stone should be encapsulated in Separation Geotextile (MaineDOT Item 722.04) to prevent migration of fines into the stone.

6.4.2 Spread Footing Design

- The proposed piers should be supported on spread footing foundations over tremie seals bearing on sound, intact bedrock. Footings designed to bear on intact bedrock should be designed using a nominal bearing resistance, q_n , of 66 ksf. At the strength limit state, footings should be designed for a maximum factored bearing resistance of 30 ksf. A bearing resistance of 30 ksf should be used for service limit state design.
- Spread footings founded on bedrock should be checked for eccentricity with AASHTO Article 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 footing width perpendicular to the axis of rotation.
- The recommended base resistance against sliding is based on NAVFAC DM7.02-63, Table 1, which indicates the sliding resistance coefficient ($\tan \delta$) is equal to 0.7 for cast-in-place concrete on sound rock. The nominal sliding resistance between footings and bedrock subgrades is equal to the vertical force multiplied by 0.7. Including the recommended resistance factor of 0.8 for sliding, the recommended factored sliding resistance coefficient is 0.56 for Strength Limit State.



- The bedrock surface should be cleaned of loose soil or rock at the time of concrete placement for tremie seals. Bearing surface preparation should be in accordance with **Section 7.3**.
- We understand the footings will be constructed over tremie seals bearing on bedrock. For tremie seal evaluation, the top of bedrock elevation in the borings was found to be within the ranges in the table that follows.

ESTIMATED BEDROCK LEVELS FOR TREMIE SEAL DESIGN	
Foundation Element	Estimated Range in Bedrock Elevation (feet, NAVD 88)
Pier 1	El. 175 to 176
Pier 2	El. 172 to 173

It is important to note that the top of intact rock cannot be known for the entire foundation area prior to construction. We expect that intact rock may be encountered above and below the anticipated levels. Some construction-phase engineering should be anticipated to address the potential variability of the encountered conditions.

- In accordance with current MaineDOT geotechnical drawing standards, we recommend that the Pier Geometry plan sheets show bedrock elevations only at the boring locations, rather than providing an interpreted bedrock surface. The intent of this recommendation is to limit the potential for claims associated with unanticipated conditions.
- If the bedrock level extends above the design bottom of tremie seal elevation, the footing may be raised and vertical reinforcement shortened in the wall subject to review and approval of the Designer to limit overexcavation of bedrock.
- If the exposed bedrock surface after cleaning is below the design footing bearing level, the tremie seal may be thickened to the proposed footing bearing level.
- Concrete used for cofferdam seals, subfootings and footings should consist of Class S Concrete in accordance with MaineDOT Standard Specification Section 502.05.
- Anchoring, doweling, benching or other means of improving sliding resistance are recommended if the prepared bedrock surface is steeper than 4H:1V in any direction.
- Although not anticipated to be needed, rock dowels may be used to supplement the sliding resistance for the footing if sloping rock is encountered. The dowels should be grouted a minimum of 2 feet into intact bedrock and embedded at least 1.5 feet into concrete. The unconfined compressive strength of the bedrock should be assumed to be 2 ksi for design of rock dowels, assuming potential weaker rock may be present near the bearing level.
- Dowels should be grouted with a cementitious grout on the MaineDOT Qualified Products List of Grout Materials for Keyways and Anchoring (pre-qualified for anchoring). Epoxy grout should not be used.
- Since the footings will be founded on bedrock, there is no minimum embedment required for frost protection per BDG Article 5.2.1.



7.0 CONSTRUCTION CONSIDERATIONS

This section provides guidance regarding quality control during pile installation, excavation, dewatering, and foundation subgrade preparation and protection. These items are discussed in the paragraphs that follow.

7.1 PILE INSTALLATION CONTROL

We recommend that the H-pile installation be controlled using wave equation analysis of the contractor's proposed driving system, field logging of the pile installation, and determination of final penetration resistance based on dynamic pile testing with signal matching analysis. As previously noted, the piles should be driven to a nominal resistance calculated by dividing the maximum factored pile load by a resistance factor of 0.65, per AASHTO Table 10.5.5.2.3-1.

AASHTO Table 10.5.5.2.3-1 requires that at least one load test with signal matching be performed per substructure to use a resistance factor of 0.65. We recommend that one Dynamic Load Test (PDA test) with Signal Matching be completed at each abutment on a battered pile, including at least one pile restrike at each abutment.

7.2 EXCAVATION, TEMPORARY LATERAL SUPPORT AND DEWATERING

Excavations for abutment foundations are anticipated to extend approximately 20 feet below existing pavement grades. It is our understanding that Route 2 traffic or construction traffic will be adjacent to the new abutment construction area during construction of the new bridge. Therefore, we anticipate it may be necessary to install temporary sheet pile excavation support to allow excavation adjacent to vehicular traffic areas. In areas where sufficient space is available and water conditions permit, the excavation slopes may consist of sloped, open cuts. In all cases, temporary excavations should comply with OSHA excavation safety requirements.

We anticipate that the inflow of groundwater or surface water to abutment excavations can be handled by open pumping from sumps installed at the bottoms of excavations if cofferdams are installed. The contractor should be responsible for controlling groundwater, surface runoff, infiltration, and water from all other sources by methods which preserve the undisturbed condition of the subgrade and permit foundation construction in-the-dry. Discharge of pumped groundwater and river water should comply with all local, State, and federal regulations.

We anticipate that temporary sheet pile cofferdams with tremie seals will be necessary for the construction of the piers. Given the lack of potential toe-in, we anticipate that the system will most likely require internal bracing until the tremie seal is placed.

7.3 SUBGRADE PREPARATION

We anticipate that bedrock bearing surface preparation will be conducted in the wet. Water depths ranging from about 20 to 24 feet above the bedrock bearing level are anticipated at the Q1.1 water level (El. 194.73). 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. 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 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 soundings and 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. Based on the test borings we do not anticipate steeply sloping bedrock conditions at the abutments. However, if the exposed bedrock surface is steeper than 4H:1V, then anchoring, doweling, benching or other means should be employed to improve sliding resistance.

We recommend that the completed seals be cored to assess the quality of the tremie seal concrete and the condition of the contact between the tremie seal and bedrock. The concrete and bedrock coring activities should be observed by the Geotechnical Engineer to assess changes in conditions based on drill action and coring penetration rate. The Geotechnical Engineer, Designer and Resident should review and evaluate the core results. If the observed conditions indicate inadequate concrete quality and/or unsuitable material at the concrete/rock interface, additional coring will be recommended, and remedial seal repairs may be required.



06/08/2020

GEOTECHNICAL DESIGN REPORT
MATTAWAMKEAG RIVER BRIDGE NO. 2522 - MATTAWAMKEAG
09.0025975.01

TABLES



TABLE 1 - SUMMARY OF SUBSURFACE EXPLORATIONS
Mattawamkeag Bridge #2522, Route 2 Over Mattawamkeag River
Mattawamkeag, ME
WIN 23234.00

	BB-MMR-101	BB-MMR-102	BB-MMR-103	BB-MMR-201	BB-MMR-202	BB-MMR-203	BB-MMR-204
Station, Offset							
Nearest Substructure	Abutment 1	--	Abutment 2	Pier 1	Pier 1	Pier 2	Pier 2
<u>Depth (ft) to:</u>							
Fill	1.5	NE	0.8	NE	NE	NE	NE
River Bottom Deposit	NE	0	NE	0	0	0	0
Glaciomarine Deposit (Silt/Clay)	NE	NE	13.5	NE	NE	NE	NE
Glaciomarine Deposit (Sand/Gravel)	13.5	2.5	18.5	1.8	3.0	2.0	2.5
Bedrock	34.4	10.2	53	8.3	9.8	10.0	11.2
Groundwater	10.4	River	ND	River	River	River	River
Bottom of Exploration	44.4	30.5	63	21.7	21.0	20.0	21.7
<u>Thickness (ft) of:</u>							
Fill	12.0	NE	12.7	NE	NE	NE	NE
River Bottom Deposit	NE	2.5	NE	1.8	3.0	2.0	2.5
Glaciomarine Deposit (Silt/Clay)	NE	NE	5	NE	NE	NE	NE
Glaciomarine Deposit (Sand/Gravel)	20.9	7.7	34.5	6.5	6.8	8.0	8.7
<u>Approximate Elevations (ft):</u>							
Ground Surface ²	208.9	184.7	209.2	184.4	184.9	182.8	184.2
Top of Fill	207.4	NE	208.4	NE	NE	NE	NE
Top of River Bottom Deposit	NE	184.7	NE	184.4	184.9	182.8	184.2
Top of Glaciomarine Deposit (Silt/Clay)	NE	NE	195.7	NE	NE	NE	NE
Top of Glaciomarine Deposit (Sand/Gravel)	195.4	182.2	190.7	182.6	181.9	180.8	181.7
Top of Bedrock	174.5	174.5	156.2	176.1	175.1	172.8	173
Groundwater	198.5	River	ND	River	River	River	River
Bottom of Exploration	164.5	154.2	146.2	162.7	163.9	162.8	162.5

NE - Not Encountered
 ND - Water Level Not Determined
 River - Boring completed in river

Notes:

1. Ground surface elevations at the were surveyed by MaineDOT and provided to GZA by HNTB.
2. Stratum depths, thickness and elevations are rounded to the nearest 0.1 foot as interpreted on the boring logs, but this does not represent the precision of the data.
3. See Appendix B for boring logs.



TABLE 2 - SUMMARY OF BEDROCK DATA
Mattawamkeag Bridge #2522, Route 2 Over Mattawamkeag River
Mattawamkeag, ME
WIN 23234.01

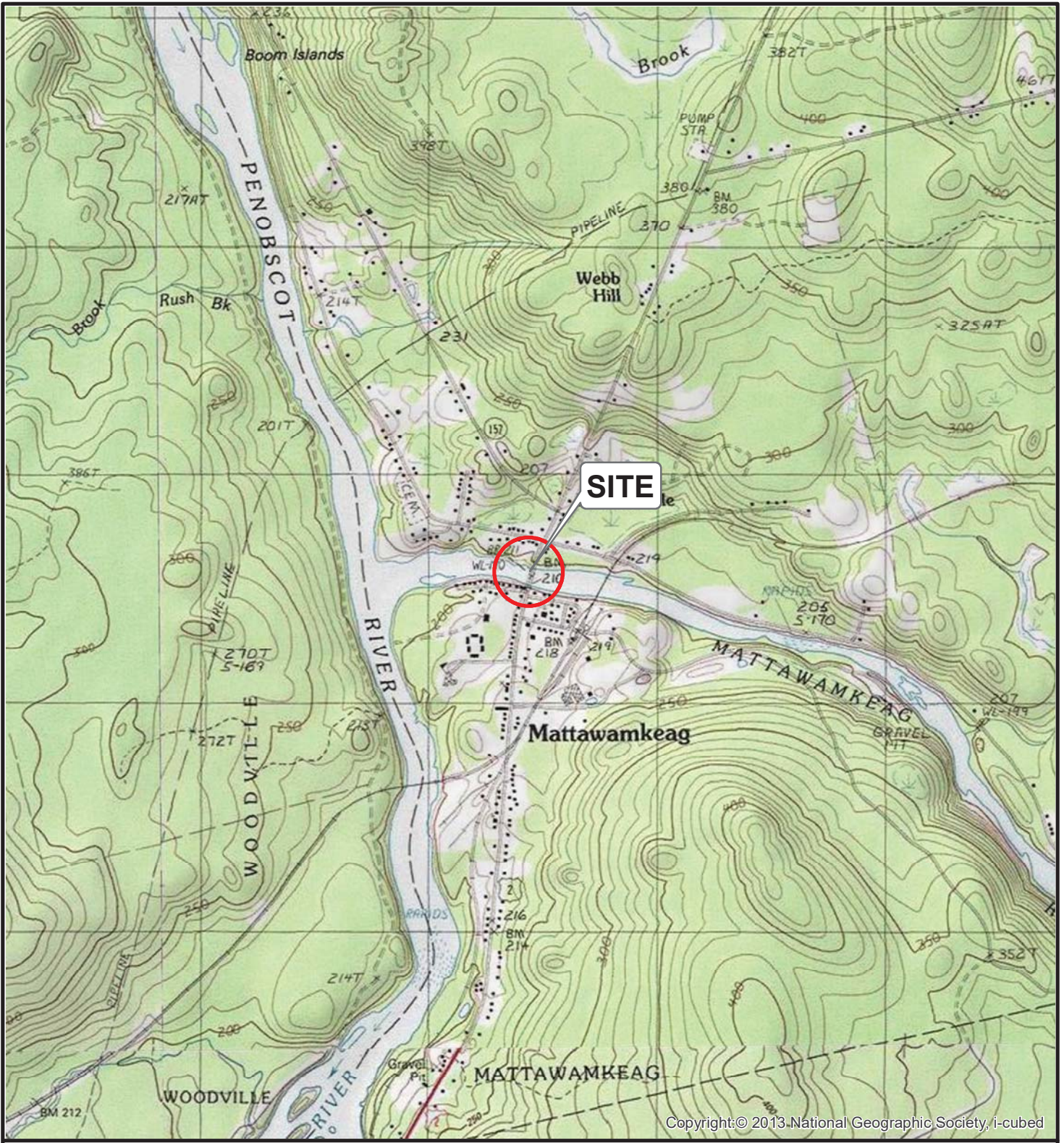
Boring	Run	GS Elevation	Depth of Core Run below GS (ft)			Depth to Rock (ft)	Depth Below Top of Rock (ft)			Length of Core Run (in)	Rec (in)	Rec (%)	RQD (in)	RQD %	Joint Spacing (in)	Joint Aperture (in)	Elev. (ft)		LAB					Rock Type	
			Top		Bottom		Top		Bottom								Top	Bottom	Top	Bottom	Depth of Sample (ft)	Depth of Sample into Rock (ft)	Elev Top of Sample (ft)		UCS (psi)
BB-MMR-101	R1	208.9	34.4	-	37.2	34.4	0.0	-	2.8	33	28	85%	8	24%	<0.75-8	0.1-0.4	174.5	171.7	34.4	0	174.5	23,157	4,020	169.2	SLATE
BB-MMR-101	R2	208.9	37.2	-	40.3	34.4	2.8	-	5.9	37	34	92%	11	30%	<0.75-24	0.02-0.4	171.7	168.6							SLATE
BB-MMR-101	R3	208.9	40.3	-	41.8	34.4	5.9	-	7.4	18	17	94%	0	0%	0.75	0.02-0.1	168.6	167.1							SLATE
BB-MMR-101	R4	208.9	41.8	-	44.4	34.4	7.4	-	10.0	31	31	100%	21	69%	<0.75-24	0.02-0.1	167.1	164.5							SLATE
BB-MMR-102	R1	184.7	8.9	-	11.4	10.2	-1.3	-	1.2	30	17	57%	0	0%	N/A	N/A-N/A	175.8	173.3							SLATE
BB-MMR-102	R2	184.7	11.4	-	12.2	10.2	1.2	-	2.0	9	9	100%	0	0%	<0.75	0.1-0.4	173.3	172.5							SLATE
BB-MMR-102	R3	184.7	12.2	-	15.8	10.2	2.0	-	5.6	43	43	100%	18	42%	0.75-8	0.004-0.1	172.5	168.9	13.2	3	171.5	25,336	6,230	164.9	SLATE
BB-MMR-102	R4	184.7	15.8	-	17.0	10.2	5.6	-	6.8	14	14	100%	0	0%	<0.75-0.75	0.004-0.1	168.9	167.7							SLATE
BB-MMR-102	R5	184.7	17.0	-	19.8	10.2	6.8	-	9.6	34	34	100%	24	71%	8-24	0.02-0.1	167.7	164.9							SLATE
BB-MMR-102	R6	184.7	19.8	-	21.3	10.2	9.6	-	11.1	18	18	100%	6	33%	<0.75-8	0.02-0.1	164.9	163.4							SLATE
BB-MMR-102	R7	184.7	21.3	-	25.5	10.2	11.1	-	15.3	50	50	100%	47	94%	<0.75-24	0.02-0.1	163.4	159.2							SLATE
BB-MMR-102	R8	184.7	25.5	-	30.5	10.2	15.3	-	20.3	60	60	100%	54	90%	24	0.1-0.4	159.2	154.2							SLATE / SCHIST
BB-MMR-103	R2	209.2	53.0	-	53.5	53.0	0.0	-	0.5	6	4	67%	0	0%	N/A	N/A-N/A	156.2	155.7							SCHIST
BB-MMR-103	R3	209.2	53.5	-	56.5	53.0	0.5	-	3.5	36	30	83%	9	25%	<0.75-8	0.1-0.4	155.7	152.7							SCHIST
BB-MMR-103	R4	209.2	56.5	-	61.5	53.0	3.5	-	8.5	60	60	100%	60	100%	24	0.1-0.1	152.7	147.7							SCHIST
BB-MMR-103	R5	209.2	61.5	-	63.0	53.0	8.5	-	10.0	18	10	56%	10	55%	24	0.1-0.1	147.7	146.2	61.8	8.8	147.4	13,092	2,870	170.2	SCHIST
BB-MMR-201	R1	184.4	8.4	-	10.1	8.3	0.1	-	1.8	20	20	100%	6	30%	0.75-24	0.02-0.1	176.0	174.3							SLATE
BB-MMR-201	R2	184.4	10.1	-	11.9	8.3	1.8	-	3.6	22	20	91%	14	64%	0.75-24	0.02-0.1	174.3	172.5							SLATE
BB-MMR-201	R3	184.4	11.9	-	16.7	8.3	3.6	-	8.4	58	47	81%	27	47%	0.75-24	0.02-0.1	172.5	167.7	12.5	4.2	171.9	9,861	3,680	167.5	SLATE
BB-MMR-201	R4	184.4	16.7	-	21.7	8.3	8.4	-	13.4	60	60	100%	46	77%	0.75-24	0.01-0.02	167.7	162.7							SLATE
BB-MMR-202	R1	184.9	10.0	-	12.1	9.8	0.2	-	2.3	25	21	84%	0	0%	<0.75-8	0.1-0.4	174.9	172.8							SLATE
BB-MMR-202	R2	184.9	12.1	-	16.7	9.8	2.3	-	6.9	55	55	100%	43	78%	0.75-80	0.02-0.4	172.8	168.2							SLATE
BB-MMR-202	R3	184.9	16.7	-	21.0	9.8	6.9	-	11.2	52	50	96%	30	58%	0.75-24	0.02-0.1	168.2	163.9							SLATE
BB-MMR-203	R2	182.8	10.0	-	11.5	10.0	0.0	-	1.5	18	18	100%	0	0%	<0.75-0.75	0.02-0.1	172.8	171.3							SLATE
BB-MMR-203	R3	182.8	11.5	-	16.5	10.0	1.5	-	6.5	60	60	100%	53	89%	8-24	0.01-0.1	171.3	166.3							SLATE
BB-MMR-203	R4	182.8	16.5	-	18.0	10.0	6.5	-	8.0	18	18	100%	18	100%	No Joints		166.3	164.8							SLATE
BB-MMR-203	R5	182.8	18.0	-	20.0	10.0	8.0	-	10.0	24	15	63%	11	46%	1 Joint		164.8	162.8							SLATE
BB-MMR-204	R2	184.2	10.9	-	11.5	11.2	-0.3	-	0.3	7	6	86%	0	0%	<0.75	#N/A	173.3	172.7							SLATE
BB-MMR-204	R3	184.2	11.5	-	13.6	11.2	0.3	-	2.4	25	17	68%	0	0%	<0.75-8	0.01-0.02	172.7	170.6							SLATE
BB-MMR-204	R4	184.2	13.6	-	16.9	11.2	2.4	-	5.7	40	35	88%	26	65%	8-24	0.01-0.1	170.6	167.3	15	3.8	169.2	1,976	2,700	166.2	SLATE
BB-MMR-204	R5	184.2	16.9	-	21.7	11.2	5.7	-	10.5	57.0	55	96%	13	23%	<0.75-24	0.01-0.1	167.3	162.5							SLATE



06/08/2020

GEOTECHNICAL DESIGN REPORT
MATTAWAMKEAG RIVER BRIDGE NO. 2522 - MATTAWAMKEAG
09.0025975.01

FIGURES



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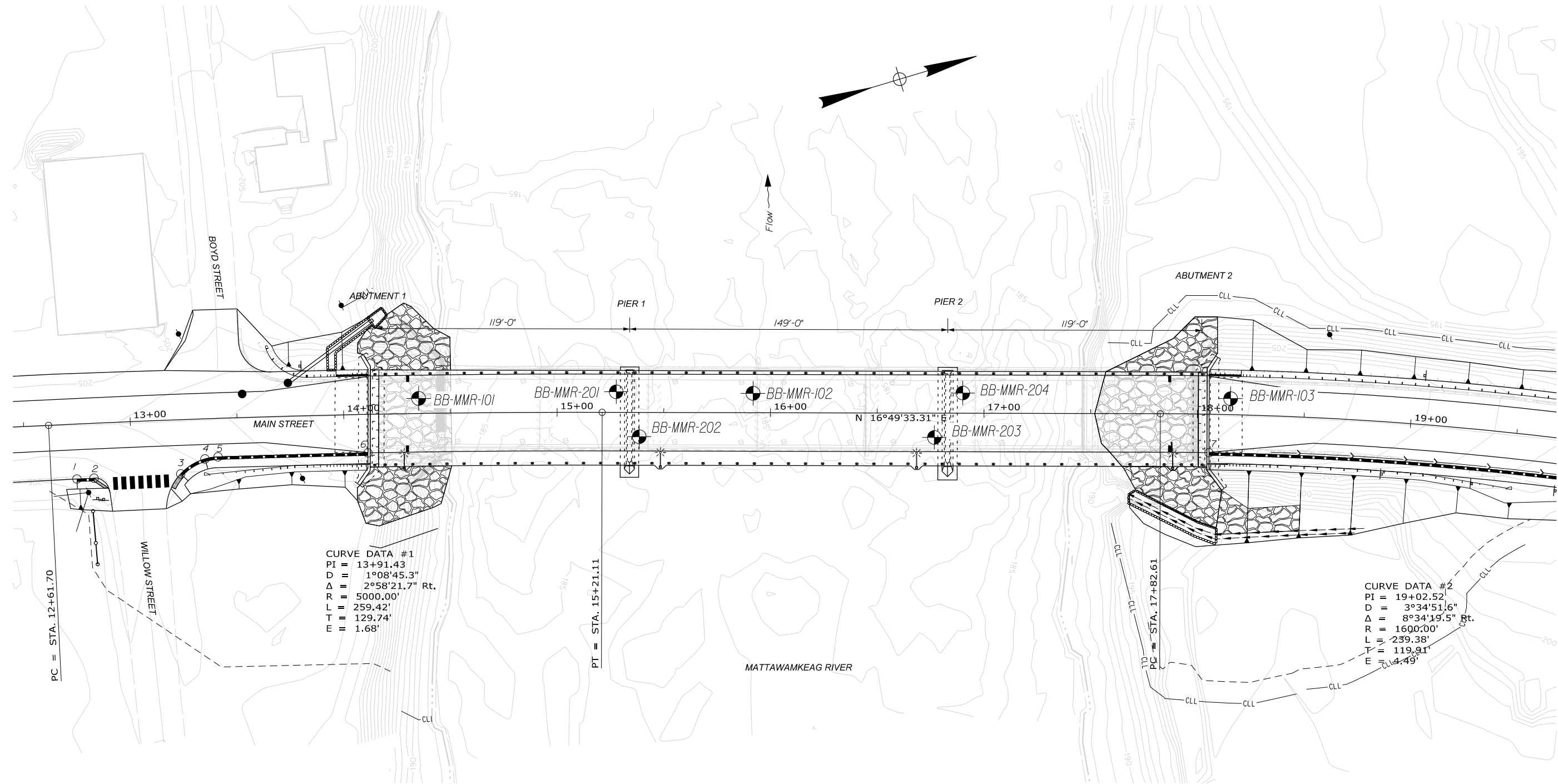
SOURCE : THIS MAP CONTAINS THE ESRI ARCGIS ONLINE USA TOPOGRAPHIC MAP SERVICE, PUBLISHED DECEMBER 12, 2009 BY ESRI ARCSIMS SERVICES AND UPDATED AS NEEDED. THIS SERVICE USES UNIFORM NATIONALLY RECOGNIZED DATUM AND CARTOGRAPHY STANDARDS AND A VARIETY OF AVAILABLE SOURCES FROM SEVERAL DATA PROVIDERS. THIS 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.



PROJ. MGR.: BMC
 DESIGNED BY: BMC
 REVIEWED BY: ARB
 OPERATOR: BMC
 DATE: 06-2-2020

LOCUS PLAN
 MAINEDOT MATTAWAMKEAG BRIDGE
 # 2522 WIN 23234.01

JOB NO.
 09.0025975.01
 FIGURE NO.
 1



CURVE DATA #1
 PI = 13+91.43
 D = 1°08'45.3"
 Δ = 2°58'21.7" Rt.
 R = 5000.00'
 L = 259.42'
 T = 129.74'
 E = 1.68'

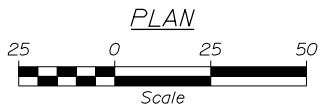
CURVE DATA #2
 PI = 19+02.52
 D = 3°34'51.6"
 Δ = 8°34'19.5" Rt.
 R = 1600.00'
 L = 239.38'
 T = 119.91'
 E = 11.44'

NOTES

- 1) Base map developed from electronic files (Alignment.dgn, Bridge.dgn, Highway.dgn, Contours.dgn, Plan.dgn, Points.dgn, and Topo.dgn) provided by HNTB on April 22, 2019, May 6, 2019, and May 1, 2020.
- 2) The as-drilled locations of the borings were surveyed by a MaineDOT survey crew and provided to GZA in an electronic file (Borings.dgn).

BORING LOCATION PLAN LEGEND

- BB-MMR-103 Indicates borings performed by New England Boring Contractors of Hermon, Maine between May 22, and 24, 2018 and observed by GZA personnel.
- BB-MMR-204 Indicates borings performed by New England Boring Contractors of Hermon, Maine between September 25, and October 1, 2019 and observed by GZA personnel.

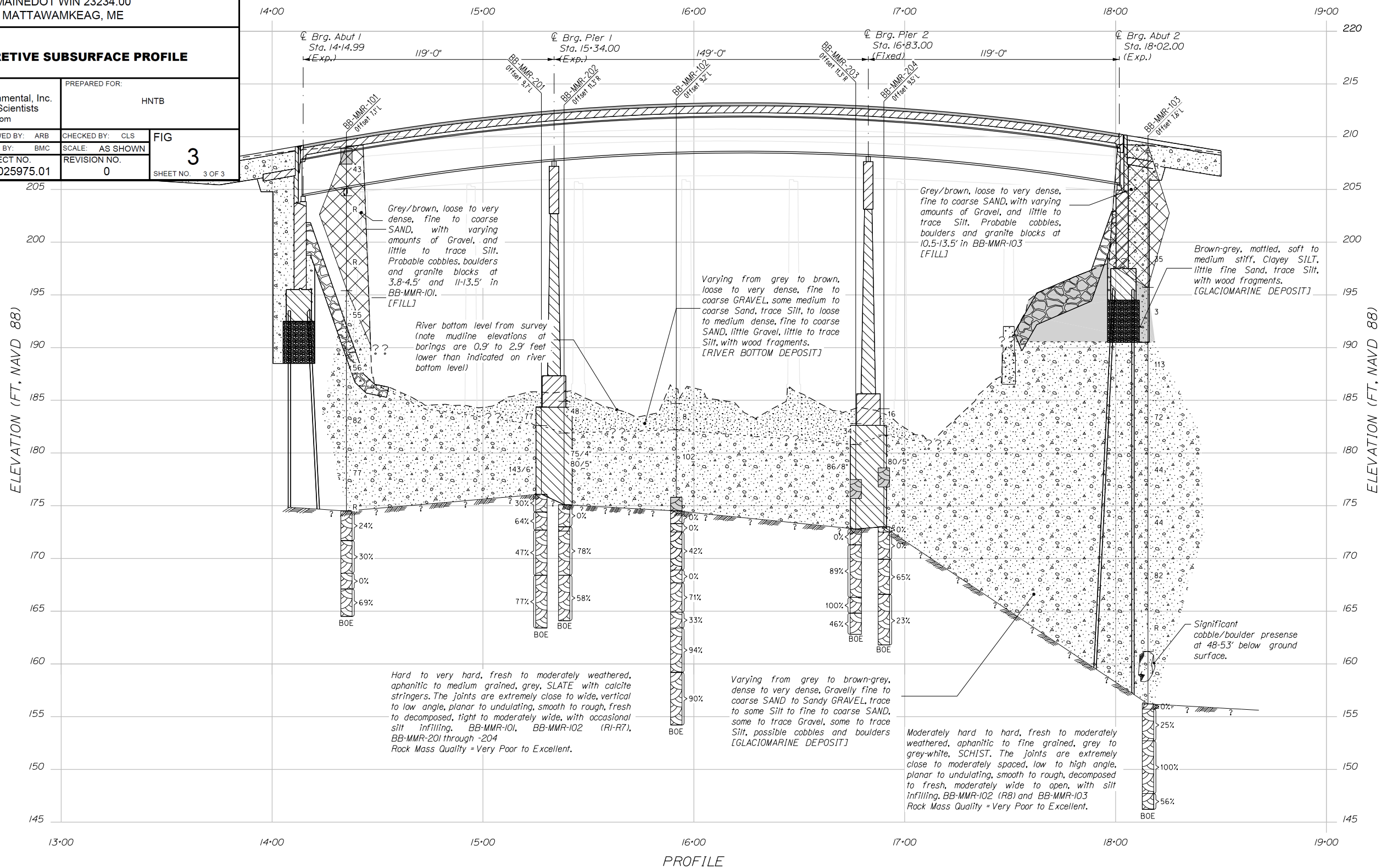


MATTAWAMKEAG BRIDGE NO. 2522 MAINEDOT WIN 23234.00 MATTAWAMKEAG, ME			
BORING LOCATION PLAN			
PREPARED BY: GZA GeoEnvironmental, Inc. Engineers and Scientists www.gza.com		PREPARED FOR: HNTB	
PROJ MGR: BMC DESIGNED BY: BMC DATE: 6/2/2020	REVIEWED BY: ARB DRAWN BY: BMC PROJECT NO. 09.0025975.01	CHECKED BY: CLS SCALE: AS SHOWN REVISION NO. 0	FIG 2 SHEET NO. 2 OF 3

STATE OF MAINE DEPARTMENT OF TRANSPORTATION	WIN 23234.01 Bridge NO. 2522																				
BORING LOCATION PLAN																					
MATTAWAMKEAG BRIDGE NO. 2522 ROUTE 2 OVER MATTAWAMKEAG RIVER MATTAWAMKEAG, ME PENOBSCOT COUNTY	SHEET NUMBER <div style="font-size: 2em; text-align: center;">8</div> OF 69																				
<table border="1" style="width: 100%; border-collapse: collapse;"> <thead> <tr> <th>DATE</th> <th>SIGNATURE</th> <th>P.E. NUMBER</th> <th>DATE</th> </tr> </thead> <tbody> <tr> <td> </td> <td> </td> <td> </td> <td> </td> </tr> <tr> <td> </td> <td> </td> <td> </td> <td> </td> </tr> <tr> <td> </td> <td> </td> <td> </td> <td> </td> </tr> <tr> <td> </td> <td> </td> <td> </td> <td> </td> </tr> </tbody> </table>	DATE	SIGNATURE	P.E. NUMBER	DATE																	BRIDGE PLANS
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INTERPRETIVE SUBSURFACE PROFILE

PREPARED BY: GZA GeoEnvironmental, Inc. Engineers and Scientists www.gza.com		PREPARED FOR: HNTB	
PROJ MGR: BMC	REVIEWED BY: ARB	CHECKED BY: CLS	FIG
DESIGNED BY: BMC	DRAWN BY: BMC	SCALE: AS SHOWN	3
DATE: 6/2/2020	PROJECT NO. 09.0025975.01	REVISION NO. 0	SHEET NO. 3 OF 3



NOTES

- 1) Base map developed from electronic files provided by HNTB on May 1, 2020 (File included Profile.dgn)
- 2) The as-drilled boring locations were surveyed by a MaineDOT survey crew and provided to GZA in an electronic file (Borings.dgn).
- 3) BB-MMR-100 and BB-MMR-200 series bridge borings were performed by New England Boring Contractors and observed by GZA personnel between May 22 and 24, 2018 and September 25 to October 1, 2019 respectively.
- 4) This generalized interpretive soil profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil and rock transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

DATE	SIGNATURE	P.E. NUMBER	DATE

SHEET NUMBER

Division: Bridge
 Filename: ... \ISP\Mattawamkeag_ISP.dgn
 Date: 7/11/2020
 Username: BMC



06/08/2020

GEOTECHNICAL DESIGN REPORT
MATTAWAMKEAG RIVER BRIDGE NO. 2522 - MATTAWAMKEAG
09.0025975.01

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.

P:\09 Jobs\0025900s\09.0025975.00 - HNTB - Mattawamkeag Bridge\09.0025975.01 - Final Design\Report\FINAL 25975.01 Mattawamkeag Bridge GDR 07102020.docx



06/08/2020

GEOTECHNICAL DESIGN REPORT
MATTAWAMKEAG RIVER BRIDGE NO. 2522 - MATTAWAMKEAG
09.0025975.01

APPENDIX B – TEST BORING LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM				MODIFIED BURMISTER SYSTEM																																																					
MAJOR DIVISIONS		GROUP SYMBOLS	TYPICAL NAMES	Descriptive Term	Portion of Total (%)																																																				
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS	GW Well-graded gravels, gravel-sand mixtures, little or no fines. GP Poorly-graded gravels, gravel sand mixtures, little or no fines.	trace little some adjective (e.g. sandy, clayey)	0 - 10 11 - 20 21 - 35 36 - 50																																																				
		GRAVEL WITH FINES (Appreciable amount of fines)	GM Silty gravels, gravel-sand-silt mixtures. GC Clayey gravels, gravel-sand-clay mixtures.	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). <table border="1"> <thead> <tr> <th>Density of Cohesionless Soils</th> <th>Standard Penetration Resistance N-Value (blows per foot)</th> </tr> </thead> <tbody> <tr><td>Very loose</td><td>0 - 4</td></tr> <tr><td>Loose</td><td>5 - 10</td></tr> <tr><td>Medium Dense</td><td>11 - 30</td></tr> <tr><td>Dense</td><td>31 - 50</td></tr> <tr><td>Very Dense</td><td>> 50</td></tr> </tbody> </table> 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. <table border="1"> <thead> <tr> <th>Consistency of Cohesive soils</th> <th>SPT N-Value (blows per foot)</th> <th>Approximate Undrained Shear Strength (psf)</th> <th>Field Guidelines</th> </tr> </thead> <tbody> <tr><td>Very Soft</td><td>WOH, WOR, WOP, <2</td><td>0 - 250</td><td>Fist easily penetrates</td></tr> <tr><td>Soft</td><td>2 - 4</td><td>250 - 500</td><td>Thumb easily penetrates</td></tr> <tr><td>Medium Stiff</td><td>5 - 8</td><td>500 - 1000</td><td>Thumb penetrates with moderate effort</td></tr> <tr><td>Stiff</td><td>9 - 15</td><td>1000 - 2000</td><td>Indented by thumb with great effort</td></tr> <tr><td>Very Stiff</td><td>16 - 30</td><td>2000 - 4000</td><td>Indented by thumbnail</td></tr> <tr><td>Hard</td><td>>30</td><td>over 4000</td><td>Indented by thumbnail with difficulty</td></tr> </tbody> </table> Rock Quality Designation (RQD): RQD (%) = $\frac{\text{sum of the lengths of intact pieces of core} * > 4 \text{ inches}}{\text{length of core advance}}$ *Minimum NQ rock core (1.88 in. OD of core) Correlation of RQD to Rock Mass Quality <table border="1"> <thead> <tr> <th>Rock Mass Quality</th> <th>RQD (%)</th> </tr> </thead> <tbody> <tr><td>Very Poor</td><td>≤25</td></tr> <tr><td>Poor</td><td>26 - 50</td></tr> <tr><td>Fair</td><td>51 - 75</td></tr> <tr><td>Good</td><td>76 - 90</td></tr> <tr><td>Excellent</td><td>91 - 100</td></tr> </tbody> </table> 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))			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	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 Mass Quality	RQD (%)	Very Poor	≤25	Poor	26 - 50	Fair	51 - 75	Good	76 - 90	Excellent
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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.	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																																																						
		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.	Sample Container Labeling Requirements: WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth																																																						
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.																																																							

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Mattawamkeag Bridge #2522
Route 2 Over Mattawamkeag River
Location: Mattawamkeag, Maine

Boring No.: BB-MMR-101
WIN: 23234.00

Driller: New England Boring Contractors	Elevation (ft.): 208.9	Auger ID/OD: SSA
Operator: Brad Enos	Datum: NAVD 88	Sampler: Standard Splitspoon
Logged By: E. Friede	Rig Type: Mobile B-53	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 5-22-18 / 5-22-18	Drilling Method: Spin & Wash	Core Barrel: NQ
Boring Location: N614662.0, E2173871.8	Casing ID/OD: 5"/5.5", 4"/4.5", 3"/3.5"	Water Level*: 10.4'
Hammer Efficiency Factor: 0.677	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	

Definitions:
 D = Split Spoon Sample
 MD = Unsuccessful Split Spoon Sample Attempt
 U = Thin Wall Tube Sample
 MU = Unsuccessful Thin Wall Tube Sample Attempt
 V = Field Vane Shear Test, PP = Pocket Penetrometer
 MV = Unsuccessful Field Vane Shear Test Attempt
 R = Rock Core Sample
 SSA = Solid Stem Auger
 HSA = Hollow Stem Auger
 RC = Roller Cone
 WOH = Weight of 140lb. Hammer
 WOR/C = Weight of Rods or Casing
 WO1P = Weight of One Person
 S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
 S_u(lab) = Lab Vane Undrained Shear Strength (psf)
 q_p = Unconfined Compressive Strength (ksf)
 N-uncorrected = Raw Field SPT N-value
 Hammer Efficiency Factor = Rig Specific Annual Calibration Value
 N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency
 N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected
 T_v = Pocket Torvane Shear Strength (psf)
 WC = Water Content, percent
 LL = Liquid Limit
 PL = Plastic Limit
 PI = Plasticity Index
 G = Grain Size Analysis
 C = Consolidation Test

Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
0								208.3		0'-0.6': Asphalt	
	1D	22/16	1.5 - 3.3	12-13-25-60/4"	38	43		207.4		0.6'-1.5': Reinforced Concrete (possible approach slab)	
										Grey, dense, dry, Gravelly fine to coarse SAND, little silt, (Fill).	G#1 A-1-b, SP-SM WC=3.3%
								205.1			
								204.4		3.8'-4.5': Granite blocks; core sample retrieved at 4.5'.	
5	2D	1/1	6.0 - 6.1	50/1"	R					Recovery is a granite block fragment.	
										Advance roller cone to 10.0'. Observed heavy grinding, with intermittent resistance.	
10	3D	16/10	10.0 - 11.3	112-40-100/4"	R					Grey and brown, moist, very dense, fine to coarse SAND, little gravel, trace silt, (Fill).	
										Heavy grinding with roller cone consistent with granite blocks from 11.0'-13.5'.	
								195.4			
15	4D	24/9	15.0 - 17.0	13-25-24-12	49	55				Grey-brown, moist, very dense, Gravelly fine to coarse SAND, trace silt, trace organic material (decomposed wood), (Glaciomarine Deposit).	G#2 A-1-a, SP-SM
20	5D	24/22	20.0 - 22.0	23-22-28-35	50	56				Top 5": Grey-brown, moist, very dense, Gravelly medium to coarse SAND, trace silt, trace organic material (decomposed wood), (Glaciomarine Deposit). Bottom 17": Orange-brown-mottled grey, moist, very dense, fine SAND, some gravel, little silt, (Glaciomarine Deposit).	
25											

Remarks:

- Automatic Hammer NEBC #2, Energy Transfer Ratio = 0.677.
- SSA to refusal at 3.8'.
- Lost water during drilling from 3.5'-4.5' and 6.0'-19'.

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.

Driller: New England Boring Contractors	Elevation (ft.): 208.9	Auger ID/OD: SSA
Operator: Brad Enos	Datum: NAVD 88	Sampler: Standard Splitspoon
Logged By: E. Friede	Rig Type: Mobile B-53	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 5-22-18 / 5-22-18	Drilling Method: Spin & Wash	Core Barrel: NQ
Boring Location: N614662.0, E2173871.8	Casing ID/OD: 5"/5.5", 4"/4.5", 3"/3.5"	Water Level*: 10.4'

Hammer Efficiency Factor: 0.677 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger S_{u(lab)} = Lab Vane Undrained Shear Strength (ksf) WC = Water Content, percent
 MD = Unsuccessful Split Spoon Sample Attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw Field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample Attempt WOH = Weight of 140 lb. Hammer Hammer Efficiency Factor = Rig Specific Annual Calibration Value PI = Plasticity Index
 V = Field Vane Shear Test, PP = Pocket Penetrometer WOR/C = Weight of Rods or Casing N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency G = Grain Size Analysis
 MV = Unsuccessful Field Vane Shear Test Attempt WO1P = Weight of One Person N₆₀ = (Hammer Efficiency Factor(60%)*N-uncorrected) C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25	6D	24/22	25.0 - 27.0	28-36-37-37	73	82				Grey, moist, very dense, fine to coarse SAND, some silt, some gravel, (Glaciomarine Deposit).	G#3 A-2-4(0) SM	
30	7D	20/10	30.0 - 31.7	20-22-46-50/2"	68	77				Grey, moist, very dense, fine to coarse SAND, some gravel, some silt, (Glaciomarine Deposit). Brown, angular gravel pieces in spoon.	G#4 A-1-b SM	
35	S8 R1	0/0 33/28	34.4 - 34.4 34.4 - 37.2	50/0" RQD = 24%	R				174.5	Split spoon refusal at 34.4', apparent top of bedrock. Set up to core. R1: Very hard, fresh, aphanitic to medium grained, grey, SLATE with calcite stringers. Joints are extremely close to close, vertical to high angle, undulating, rough, discolored (rusty orange) to disintegrated, moderately wide, grey/orange silt infilling. Rock Mass Quality = Very Poor Recovery = 85% Rock Core Times (min:sec): 34.4-35.4' (4:05), 35.4-36.4' (3:43), 36.4-37.2' (3:07) R2: Very hard, fresh, aphanitic to medium grained, grey, SLATE with calcite stringers. Recovery is gravel in upper 3" of recovery. Joints are extremely close at top to moderately spaced at bottom, vertical to low angle, stepped and undulating, rough, fresh to discolored (orange), open to moderately wide, trace grey silt infilling. Rock Mass Quality = Poor Recovery = 92% Rock Core Times (min:sec): 37.2-37.4' (1:12), 37.4-38.4' (5:23), 38.4-39.4' (5:11), 39.4-40.3' (2:50) R3: Moderately hard to very hard, slightly weathered, aphanitic to fine grained, grey, SLATE with calcite stringers. Joints are very close, high angle, undulating, smooth to rough, decomposed to fresh, open, brown silt infilling. Weathered zone from 40.3'-41.3', joint walls are soft, friable with apparent foliation and healed fractures. Rock Mass Quality = Very Poor Recovery = 94% Rock Core Times (min:sec): 40.4-40.4' (1:47), 40.4-41.4' (3:08), 41.4-41.8' (1:30) R4: Very hard, fresh to moderately weathered, aphanitic to medium grained, grey, SLATE with calcite stringers. Joints are moderately spaced to extremely close, low to high angle, undulating, rough, discolored, open, no infilling. 41.8'-43.3': Apparent healed fractures with calcite infilling. 43.3'-44.4': Weathered zone, joint walls are soft, friable.	q _p =3335 ksf	
40	R2	37/34	37.2 - 40.3	RQD = 30%								
45	R3	18/17	40.3 - 41.8	RQD = 0%					164.5			
50	R4	31/31	41.8 - 44.4	RQD = 69%								

Remarks:

- Automatic Hammer NEBC #2, Energy Transfer Ratio = 0.677.
- SSA to refusal at 3.8'.
- Lost water during drilling from 3.5'-4.5' and 6.0'-19'.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

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Route 2 Over Mattawamkeag River
Location: Mattawamkeag, Maine

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 G = Grain Size Analysis
 C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
50											Rock Mass Quality = Fair Recovery = 100% Rock Core Times (min:sec): 41.8-42.4' (4:09), 42.4-43.4' (5:48) 44.4 Bottom of Exploration at 44.4 feet below ground surface.	
55												
60												
65												
70												
75												

Remarks:

- Automatic Hammer NEBC #2, Energy Transfer Ratio = 0.677.
- SSA to refusal at 3.8'.
- Lost water during drilling from 3.5'-4.5' and 6.0'-19'.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Mattawamkeag Bridge #2522
Route 2 Over Mattawamkeag River
Location: Mattawamkeag, Maine

Boring No.: BB-MMR-102

WIN: 23234.00

Driller: New England Boring Contractors	Elevation (ft.): 184.7	Auger ID/OD: --
Operator: Brad Enos	Datum: NAVD 88	Sampler: Standard Splitspoon
Logged By: E. Friede	Rig Type: Mobile B-53	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 5-24-18 / 5-24-18	Drilling Method: Drive/Spin & Wash	Core Barrel: NQ
Boring Location: N614812.8, E2173914.5	Casing ID/OD: 4"/4.5", 3"/3.5"	Water Level*: River

Hammer Efficiency Factor: 0.677 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger S_{u(lab)} = Lab Vane Undrained Shear Strength (psf) WC = Water Content, percent
 MD = Unsuccessful Split Spoon Sample Attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw Field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample Attempt WOH = Weight of 140 lb. Hammer Hammer Efficiency Factor = Rig Specific Annual Calibration Value PI = Plasticity Index
 V = Field Vane Shear Test, PP = Pocket Penetrometer WOR/C = Weight of Rods or Casing N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency G = Grain Size Analysis
 MV = Unsuccessful Field Vane Shear Test Attempt WO1P = Weight of One Person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25	R8	60/60	25.5 - 30.5	RQD = 90%					154.2	high angle, planar, smooth, discolored (rusty), open. Rock Mass Quality = Fair Recovery = 100% Rock Core Times (min:sec): 17.0-17.8' (2:28), 17.8-18.8' (2:10), 18.8-19.8' (2:17) R6: Hard, fresh, aphanitic to fine grained, grey, SLATE with white calcite stringers. Joints are extremely close to close, low to high angle, fresh to discolored, open. Rock Mass Quality = Poor Recovery = 100% Rock Core Times (min:sec): 19.8-20.8' (2:40), 20.8-21.3' (3:00) R7: Hard, fresh, aphanitic to fine grained, grey, SLATE with white calcite stringers. Joints are moderately spaced to extremely close, low to high angle, discolored to fresh, planar to stepped, smooth to rough, open. Apparent shear zone at 22.6'. Contact (stylolitic) at 24.6', more apparent foliation below, pyrite present. Rock Mass Quality = Excellent Recovery = 100% Rock Core Times (min:sec): 21.3-21.8' (1:54), 21.8-22.8' (1:57), 22.8-23.8' (2:28), 23.8-24.8' (1:53) R8: 25.5'-30.5': Hard, fresh, fine to medium grained, grey, SLATE and SCHIST with calcite stringers. Joints are moderately spaced, high angle, fresh, moderately wide, with medium grained mica (Schist). Rock Mass Quality = Good Recovery = 100% Rock Core Times (min:sec): 25.5-26.5' (1:98), 26.5-27.5' (1:52), 27.5-28.5' (2:10), 29.5-29.5' (1:42), 29.5-30.5' (1:50)		
30												
35												
40												
45												
50												

Remarks:

- Automatic Hammer NEBC #2, Energy Transfer Ratio = 0.677.
- Boring was drilled through 0.9' thick concrete bridge deck. Mudline was 25.5' below top of deck. Boring depths are referenced to the mudline. Mudline elevation calculated using the surveyed bridge deck elevation and measured distance to mudline.

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Mattawamkeag Bridge #2522
Route 2 Over Mattawamkeag River
Location: Mattawamkeag, Maine

Boring No.: BB-MMR-103
WIN: 23234.00

Driller: New England Boring Contractors	Elevation (ft.): 209.2	Auger ID/OD: SSA
Operator: Brad Enos	Datum: NAVD 88	Sampler: Standard Splitspoon
Logged By: E. Friede	Rig Type: Mobile B-53	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 5-23-18 / 5-23-18	Drilling Method: Drive & Wash	Core Barrel: NQ
Boring Location: N615026.4, E2173981.1	Casing ID/OD: 4"/4.5", 3"/3.5"	Water Level*:
Hammer Efficiency Factor: 0.677	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	

Definitions:
 D = Split Spoon Sample
 MD = Unsuccessful Split Spoon Sample Attempt
 U = Thin Wall Tube Sample
 MU = Unsuccessful Thin Wall Tube Sample Attempt
 V = Field Vane Shear Test, PP = Pocket Penetrometer
 MV = Unsuccessful Field Vane Shear Test Attempt
 R = Rock Core Sample
 SSA = Solid Stem Auger
 HSA = Hollow Stem Auger
 RC = Roller Cone
 WOH = Weight of 140lb. Hammer
 WOR/C = Weight of Rods or Casing
 WO1P = Weight of One Person
 S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
 S_u(lab) = Lab Vane Undrained Shear Strength (psf)
 q_p = Unconfined Compressive Strength (ksf)
 N-uncorrected = Raw Field SPT N-value
 Hammer Efficiency Factor = Rig Specific Annual Calibration Value
 N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency
 N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected
 T_v = Pocket Torvane Shear Strength (psf)
 WC = Water Content, percent
 LL = Liquid Limit
 PL = Plastic Limit
 PI = Plasticity Index
 G = Grain Size Analysis
 C = Consolidation Test

Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
0	1D	15/11	0.8 - 2.1	19-18-50/3"	R		SSA	208.4	0'-0.8': Asphalt		
									0.8'-1.0': Grey-brown, moist, dense to very dense, fine to coarse SAND, some fine to coarse gravel, little silt, (Fill).	G#7 A-1-b SP-SM WC=2.9%	
5	2D	24/12	5.0 - 7.0	4-3-3-4	6	7			Brown, moist, loose, fine to coarse SAND, little silt, little gravel, (Fill).	G#8 A-1-b SM WC=6.4%	
10	3D	24/13	10.0 - 12.0	13-15-16-4	31	35	CASED		Increased roller cone resistance and lost water return 9.0'-10.0', possible boulders/granite blocks. Top 5": Brown, moist, fine to coarse SAND, little gravel, trace silt, (Fill). Bottom 6": Grey, moist, GRAVEL, some fine to coarse sand, trace silt, (Fill). Brown, moist, fine to medium clayey sand with gravel in tip of spoon. Increased roller cone resistance 10.5'-13.5', possible boulders/granite blocks.		
15	4D	24/12	15.0 - 17.0	2-1-2-3	3	3		195.7	Brown-grey mottled, moist, soft to medium stiff, Clayey SILT, little fine sand, with black nodules, (Glaciomarine Deposit).	G#9 A-6(10) ML LL=36 PL=25 PI=11 WC=31.6%	
20	5D	24/13	20.0 - 22.0	27-68-32-15	100	113		190.7	Top 13": Brown, moist, very dense, Sandy fine to coarse GRAVEL, trace silt, (Glaciomarine Deposit). Bottom 2": Grey, moist, hard, Silty CLAY, little fine to medium sand, trace gravel, (Glaciomarine Deposit).	G#10 A-1-a GW-GM	
25								185.7			

Remarks:

- Automatic Hammer NEBC #2, Energy Transfer Ratio = 0.677.
- Drilling pressure was reduced due to bent casing. Core times may not represent typical drilling effort.
- 18" of rock cored in R4 was recovered in R5.

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: Mattawamkeag Bridge #2522
Route 2 Over Mattawamkeag River
Location: Mattawamkeag, Maine

Boring No.: BB-MMR-103

WIN: 23234.00

Driller: New England Boring Contractors	Elevation (ft.): 209.2	Auger ID/OD: SSA
Operator: Brad Enos	Datum: NAVD 88	Sampler: Standard Splitspoon
Logged By: E. Friede	Rig Type: Mobile B-53	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 5-23-18 / 5-23-18	Drilling Method: Drive & Wash	Core Barrel: NQ
Boring Location: N615026.4, E2173981.1	Casing ID/OD: 4"/4.5", 3"/3.5"	Water Level*:

Hammer Efficiency Factor: 0.677 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger S_{u(lab)} = Lab Vane Undrained Shear Strength (psf) WC = Water Content, percent
 MD = Unsuccessful Split Spoon Sample Attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw Field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample Attempt WOH = Weight of 140 lb. Hammer Hammer Efficiency Factor = Rig Specific Annual Calibration Value PI = Plasticity Index
 V = Field Vane Shear Test, PP = Pocket Penetrometer WOR/C = Weight of Rods or Casing N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency G = Grain Size Analysis
 MV = Unsuccessful Field Vane Shear Test Attempt WO1P = Weight of One Person N₆₀ = (Hammer Efficiency Factor(60%)*N-uncorrected) C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in. Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
50	11D	5/5	50.9 - 51.3	50/5"					158.3	R1: Apparent cobbles, boulders or fractured rock, advance split spoon.		
										Grey, wet, very dense, GRAVEL, little fine to coarse sand, trace silt, granite rock fragments in gravel, (Glaciomarine Deposit). Advance 3" casing to 51.2'; roller cone heavy grinding from 51.0'-51.6'. Advance 3" casing to 53.0'.		
	R2	6/4	53.0 - 53.5	RQD = 0%					156.2	R2: Moderately hard, slightly weathered, aphanitic to fine grained, grey, foliated SCHIST with quartz stringers. Fractured into rock fragments.		
	R3	36/30	53.5 - 56.5	RQD = 25%						Rock Mass Quality = Very Poor Recovery = 67%		
55										Rock Core Times (min:sec): 53.0-53.6' (7:28)		
	R4	60/60	56.5 - 61.5	RQD = 100%						R3: Hard, slightly to moderately weathered, aphanitic to fine grained, grey-white, foliated SCHIST with Pegmatite intrusion. Joints are extremely close to close, low to high angle, planar to undulating, rough to smooth, discolored to decomposed, moderately wide, with grey Silt infilling. Pegmatite begins at 54.5' and is coarse grained, with highly fractured, weathered zones.		
										Rock Mass Quality = Very Poor Recovery = 83%		
60										Rock Core Times (min:sec): 53.5-54.0' (4:30), 54.0-55.0' (6:38), 55.0-55.5' (6:22)		
	R5	18/10	61.5 - 63.0	RQD = 56%					146.2	R4: Hard, fresh, aphanitic, grey with white bands, foliated SCHIST. Joints are moderately spaced, low to high angle, planar to stepped, rough to smooth, fresh, moderately wide to open. Foliation is high angle.	q _p =1885 ksf	
										Rock Mass Quality = Excellent Recovery = 100%		
										Rock Core Times (min:sec): 56.6-57.0' (3:48), 57.0-58.0' (4:02), 58.0-59.0' (3:56), 59.0-60.0' (4:15), 60.0-61.0' (4:00), 61.0-61.5' (3:28)		
65										R5: Hard, fresh, aphanitic, grey with white bands, foliated SCHIST. Joints are moderately spaced, low to high angle, planar to stepped, rough to smooth, fresh, moderately wide to open. Foliation is high angle.		
										Rock Mass Quality = Fair Recovery = 56%		
										Rock Core Times (min:sec): 61.5-62.0' (2:20), 62.0-63.0' (3:15)		
										Bottom of Exploration at 63.0 feet below ground surface.		
70												
75												

Remarks:

- Automatic Hammer NEBC #2, Energy Transfer Ratio = 0.677.
- Drilling pressure was reduced due to bent casing. Core times may not represent typical drilling effort.
- 18" of rock cored in R4 was recovered in R5.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Mattawamkeag Bridge #2522
Route 2 Over Mattawamkeag River
Location: Mattawamkeag, Maine

Boring No.: BB-MMR-203

WIN: 23234.00

Driller: New England Boring Contractors	Elevation (ft.): 182.8	Auger ID/OD: --
Operator: Tom Shaeffer	Datum: NAVD 88	Sampler: Standard Splitspoon
Logged By: B. Woodman	Rig Type: Truck Mobile B-53	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 9/26/19-9/27/19	Drilling Method: Drive/Spin/Wash	Core Barrel: NX
Boring Location: N614888.2, E2173958.7	Casing ID/OD: 4"/3"	Water Level*: River
Hammer Efficiency Factor: .937	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	

Definitions:
 D = Split Spoon Sample
 MD = Unsuccessful Split Spoon Sample Attempt
 U = Thin Wall Tube Sample
 MU = Unsuccessful Thin Wall Tube Sample Attempt
 V = Field Vane Shear Test, PP = Pocket Penetrometer
 MV = Unsuccessful Field Vane Shear Test Attempt

R = Rock Core Sample
 SSA = Solid Stem Auger
 HSA = Hollow Stem Auger
 RC = Roller Cone
 WOH = Weight of 140lb. Hammer
 WOR/C = Weight of Rods or Casing
 WO1P = Weight of One Person

S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
 S_{u(lab)} = Lab Vane Undrained Shear Strength (psf)
 q_p = Unconfined Compressive Strength (ksf)
 N-uncorrected = Raw Field SPT N-value
 Hammer Efficiency Factor = Rig Specific Annual Calibration Value
 N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency
 N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected

T_v = Pocket Torvane Shear Strength (psf)
 WC = Water Content, percent
 LL = Liquid Limit
 PL = Plastic Limit
 PI = Plasticity Index
 G = Grain Size Analysis
 C = Consolidation Test

Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.	
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0	1D	24/7	0.0 - 2.0	7-7-15-13	22	34	1	180.8		Brown, wet, medium dense, fine to coarse SAND, little fine gravel, little silt, (River Bottom Deposit).		
5	2D	17/10	5.3 - 6.7	30-31-75/3"	--		RC			Grey, wet, very dense, fine to coarse SAND, some silt, little gravel, (Glaciomarine Deposit).		
	R1	20/20	7.0 - 8.7	--			NX				Roller cone resistance at 6.9'. Advance roller cone to 7.0' and set up to core. Cored from 7.0'-8.7' before breaking through. Probable Boulder from 6.9'-8.7'.	
							RC					
10	R2	18/18	10.0 - 11.5	RQD = 0%			NX	172.8		Roller cone from 8.7'-9.5', intermittent resistance indicates probable cobbles.		
	R3	60/60	11.5 - 16.5	RQD = 89%							Increased resistance at 10.0'. Probable top of rock. Set up to core. R2: Hard, fresh to slightly discolored, aphanitic to fine grained, grey, SLATE, with calcite stringers and 1/2" think veins. Joints are extremely close to very close, high angle, undulating, rough, discolored, open. Rock Mass Quality = Very Poor Recovery = 100% Rock Core Times (min:sec): 10.0-11.0' (2:05), 11.0-11.5' (1:12)	
											R3: Hard, fresh, aphanitic to fine grained, SLATE, with calcite stringers. Joints are close to moderately close, high angle, undulating, rough, partially open to open. Rock Mass Quality = Good	
	R4	18/18	16.5 - 18.0	RQD = 100%							R4: Hard, fresh, aphanitic to fine grained, grey, SLATE, with calcite stringers. No joints. Rock Mass Quality = Excellent Recovery = 100%	
	R5	24/15	18.0 - 20.0	RQD = 46%							R5: Hard, fresh, aphanitic to fine grained, grey, SLATE, with one calcite stringer. One joint, high angle, undulating, rough, fresh to discolored. Rock Mass Quality = Poor Recovery = 63%	
20								162.8		Rock Core Times (min:sec): 16.5-17.5' (1:17), 17.5-18.0' (0:51)		
										R5: Hard, fresh, aphanitic to fine grained, grey, SLATE, with one calcite stringer. One joint, high angle, undulating, rough, fresh to discolored. Rock Mass Quality = Poor Recovery = 63%		
										Rock Core Times (min:sec): 18.0-19.0' (1:33), 19.0-20.0' (1:40)		
										Bottom of Exploration at 20.0 feet below ground surface.		
25												

Remarks:

- Automatic hammer NEBC #B-24 Energy Transfer Ratio = 0.937
- Cored through bridge deck using a 6" diameter concrete corer. Mudline measured 27.6' below top of bridge deck.

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.



06/08/2020

GEOTECHNICAL DESIGN REPORT
MATTAWAMKEAG RIVER BRIDGE NO. 2522 - MATTAWAMKEAG
09.0025975.01

APPENDIX C – LABORATORY TEST RESULTS



195 Frances Avenue
 Cranston RI, 02910
 Phone: (401)-467-6454
 Fax: (401)-467-2398
<http://www.thielsch.com>
Let's Build a Solid Foundation

Client Information:
 GZA GeoEnvironmental, Inc
 Portland, ME
 PM: Theodore Baire
 Assigned By: EDF
 Collected By: EDF

Project Information:
Mattawamkeag Bridge #2522
Mattawamkeag, ME
 GZA Project Number: 09.0025975.00
 Summary Page: 1 of 2
 Report Date: 06.15.18

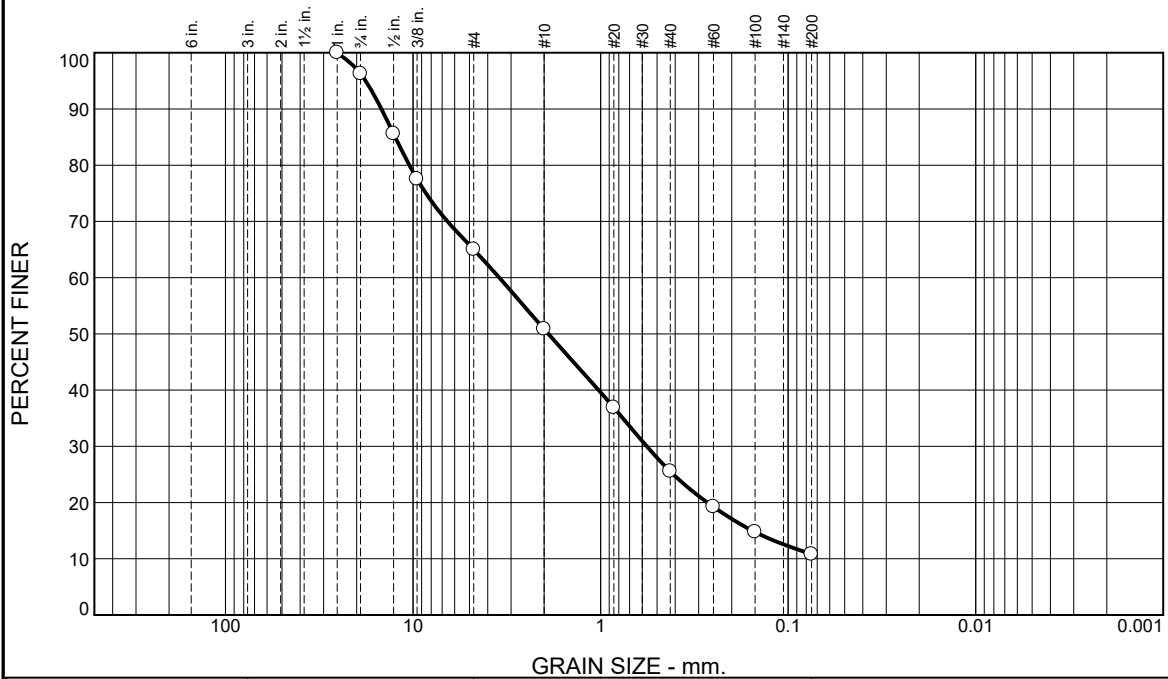
LABORATORY TESTING DATA SHEET

Boring ID	Sample No.	Depth (ft)	Laboratory No.	Identification Tests								Proctor / Direct Shear Tests						Laboratory Log and Soil Description		
				Water Content %	LL %	PL %	Gravel %	Sand %	Fines %	Org. %	G _s	Dry unit wt. pcf	γ_d MAX (pcf) W _{opt} (%)	γ_d MAX (pcf) W _{opt} (%) (Corr.)	Strength (psi) @ 250 psf	Strength (psi) @ 500 psf	Strength (psi) @ 750 psf		Strength (psi) @ 1000 psf	EST. Shear Angle
BB-MMR-101	1D	1.5-3.5	S-1	3.32			35.0	54.2	10.8											Brown Gravelly fine to coarse SAND, little Silt
BB-MMR-101	4D	15-17	S-2				46.1	47.3	6.6											Light Brown Gravelly fine to coarse SAND, trace Silt
BB-MMR-101	6D	25-27	S-3				22.9	48.3	28.8											Grey f-c SAND, some Silt, some f-c Gravel
BB-MMR-101	7D	30-31.7	S-4				27.3	47.3	25.4											Grey f-c SAND, some fine Gravel, some Silt
BB-MMR-102	1D	0-2	S-5	15.4			73.6	25.9	0.5											Dark Brown f-c GRAVEL, some m-c Sand, trace Silt
BB-MMR-102	2D	5-7	S-6	7.8			41.6	36.1	22.3											Grey Sandy fine to coarse GRAVEL, some Silt
BB-MMR-103	1D	0.8-2.1	S-7	2.9			31.6	57.6	10.8											Brown f-c SAND, some f-c Gravel, little Silt
BB-MMR-103	2D	5-7	S-8	6.4			12.6	71.8	15.6											Brown f-c SAND, little Silt, little fine Gravel
BB-MMR-103	4D	15-17	S-9	31.6	36	25	0.0	15.4	84.6											Light Brown CLAY & SILT, little fine Sand
BB-MMR-103	5D	20-22	S-10				51.8	41.7	6.5											Brown Sandy fine to coarse GRAVEL, trace Silt
BB-MMR-103	7D	30-32	S-11	6.9			12.7	35.4	51.9											Grey Sandy SILT & CLAY, little fine Gravel
BB-MMR-103	8D	35-37	S-12				22.3	75.2	2.5											Grey f-c SAND, some fine Gravel, trace Silt

Reviewed By SAW

Date Reviewed 06.15.2018

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	3.7	31.3	14.2	25.3	14.7	10.8	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	96.3		
0.5"	85.6		
0.375"	77.5		
#4	65.0		
#10	50.8		
#20	36.9		
#40	25.5		
#60	19.2		
#100	14.8		
#200	10.8		

Material Description

Brown Gravelly fine to coarse SAND, little Silt

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= SP-SM AASHTO (M 145)= A-1-b

Coefficients

D₉₀= 14.7487 D₈₅= 12.4545 D₆₀= 3.4696
D₅₀= 1.8991 D₃₀= 0.5671 D₁₅= 0.1551
D₁₀= C_u= C_c=

Remarks

Date Received: 06.04.18 Date Tested: 06.13.18

Tested By: MN / JS

Checked By: Rebecca Roth

Title: Laboratory Manager

* (no specification provided)

Source of Sample: BB-MMR-101
Sample Number: 1D

Depth: 1.5-3.5'

Date Sampled:

Thielsch Engineering Inc.

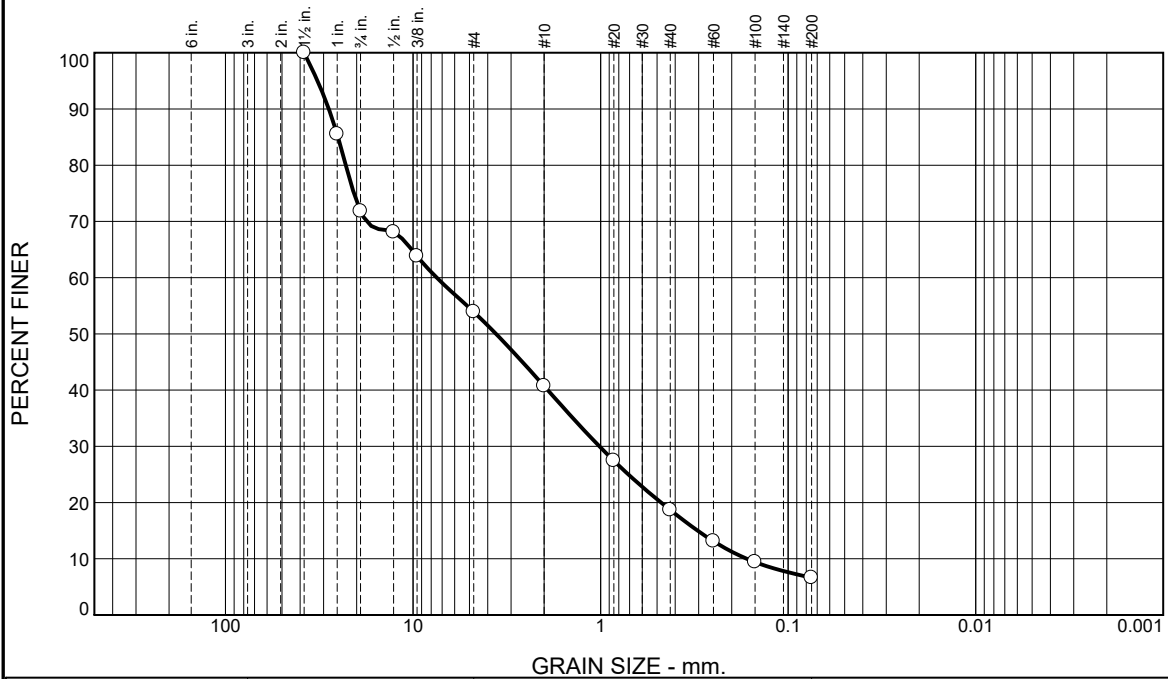
Cranston, RI

Client: GZA GeoEnvironmental
Project: Mattawamkeag Bridge #2522
Mattawamkeag, ME

Project No: 09.0025975.00

Figure S-1

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	28.2	17.9	13.2	22.0	12.1	6.6	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1.5"	100.0		
1"	85.5		
0.75"	71.8		
0.5"	68.1		
0.375"	63.8		
#4	53.9		
#10	40.7		
#20	27.4		
#40	18.7		
#60	13.1		
#100	9.4		
#200	6.6		

* (no specification provided)

Material Description

Light Brown Gravelly fine to coarse SAND, trace Silt

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= SP-SM AASHTO (M 145)= A-1-a

Coefficients

D ₉₀ = 28.1094	D ₈₅ = 25.1333	D ₆₀ = 7.4725
D ₅₀ = 3.6096	D ₃₀ = 1.0153	D ₁₅ = 0.3037
D ₁₀ = 0.1660	C _u = 45.02	C _c = 0.83

Remarks

Date Received: 06.04.18 Date Tested: 06.13.18

Tested By: MN / JS

Checked By: Rebecca Roth

Title: Laboratory Manager

Source of Sample: BB-MMR-101
Sample Number: 4D

Depth: 15-17'

Date Sampled:

Thielsch Engineering Inc.

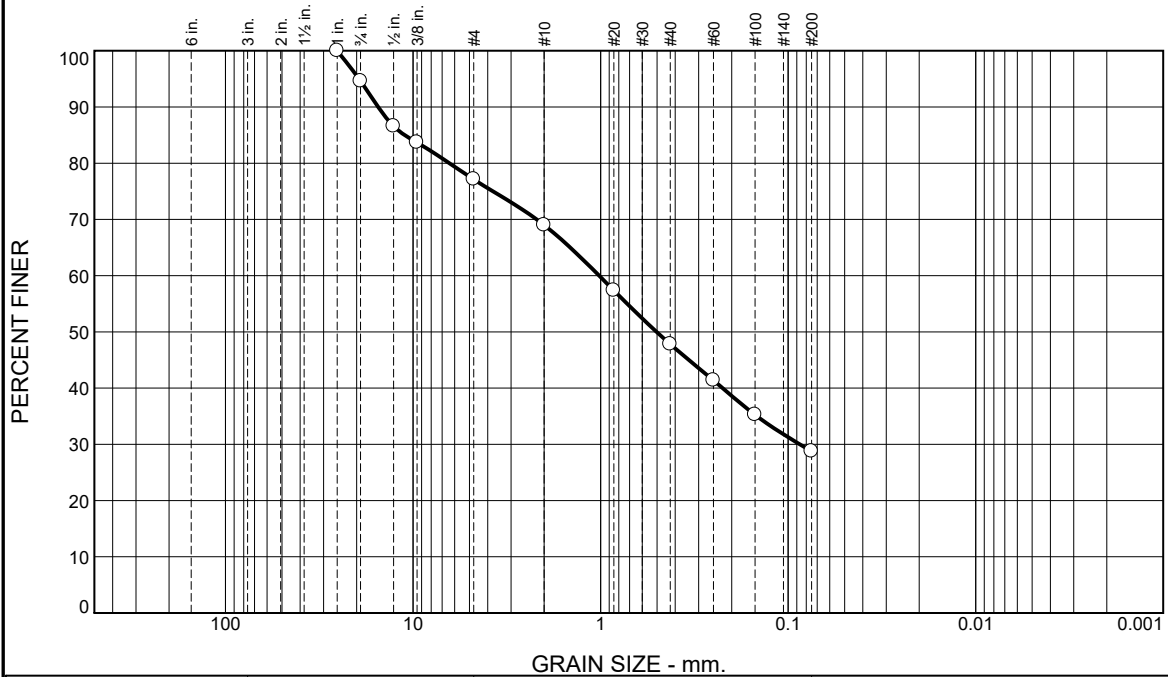
Cranston, RI

Client: GZA GeoEnvironmental
Project: Mattawamkeag Bridge #2522
Mattawamkeag, ME

Project No: 09.0025975.00

Figure S-2

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	5.4	17.5	8.1	21.2	19.0	28.8	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	94.6		
0.5"	86.6		
0.375"	83.7		
#4	77.1		
#10	69.0		
#20	57.4		
#40	47.8		
#60	41.4		
#100	35.3		
#200	28.8		

Material Description

Grey f-c SAND, some Silt, some f-c Gravel

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= SM AASHTO (M 145)= A-2-4(0)

Coefficients

D₉₀= 15.3465 D₈₅= 11.1053 D₆₀= 1.0184
D₅₀= 0.5028 D₃₀= 0.0866 D₁₅=
D₁₀= C_u= C_c=

Remarks

Date Received: 06.04.18 Date Tested: 06.13.18

Tested By: MN / JS

Checked By: Rebecca Roth

Title: Laboratory Manager

* (no specification provided)

Source of Sample: BB-MMR-101
Sample Number: 6D

Depth: 25-27'

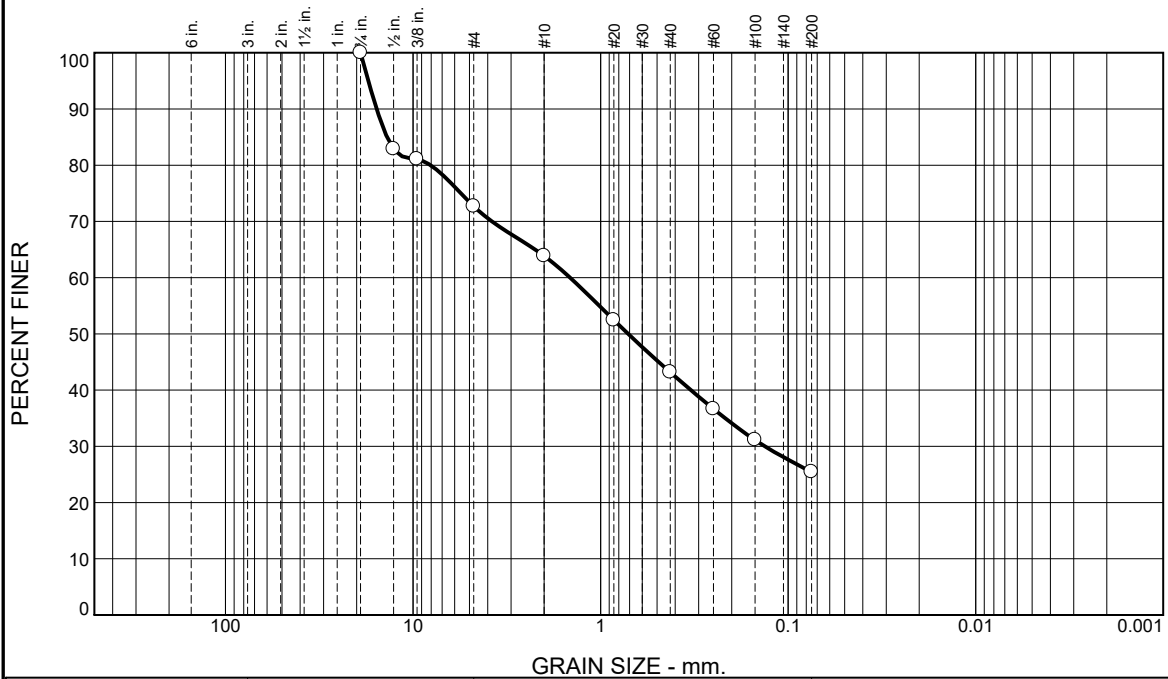
Date Sampled:

Thielsch Engineering Inc.
Cranston, RI

Client: GZA GeoEnvironmental
Project: Mattawamkeag Bridge #2522
Mattawamkeag, ME
Project No: 09.0025975.00

Figure S-3

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	27.3	8.8	20.7	17.8	25.4	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.75"	100.0		
0.5"	82.9		
0.375"	81.1		
#4	72.7		
#10	63.9		
#20	52.4		
#40	43.2		
#60	36.7		
#100	31.1		
#200	25.4		

Material Description

Grey f-c SAND, some fine Gravel, some Silt

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= SM AASHTO (M 145)= A-1-b

Coefficients

D₉₀= 15.6408 D₈₅= 13.7671 D₆₀= 1.4541
 D₅₀= 0.7129 D₃₀= 0.1329 D₁₅=
 D₁₀= C_u= C_c=

Remarks

Date Received: 06.05.18 Date Tested: 06.13.18

Tested By: MN / JS

Checked By: Rebecca Roth

Title: Laboratory Manager

* (no specification provided)

Source of Sample: BB-MMR-101
 Sample Number: 7D

Depth: 30-31.7'

Date Sampled:

Thielsch Engineering Inc.

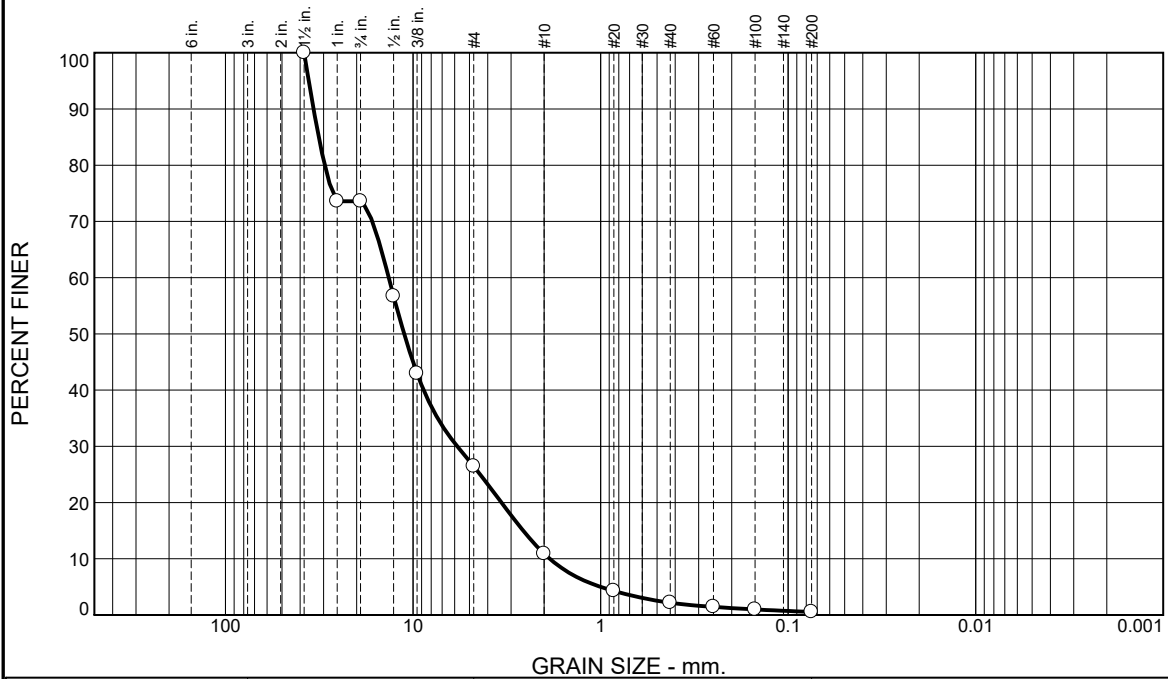
Cranston, RI

Client: GZA GeoEnvironmental
 Project: Mattawamkeag Bridge #2522
 Mattawamkeag, ME

Project No: 09.0025975.00

Figure S-4

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	26.4	47.2	15.5	8.7	1.7	0.5	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1.5"	100.0		
1"	73.6		
0.75"	73.6		
0.5"	56.7		
0.375"	43.0		
#4	26.4		
#10	10.9		
#20	4.3		
#40	2.2		
#60	1.4		
#100	1.0		
#200	0.5		

* (no specification provided)

Material Description

Dark Brown f-c GRAVEL, some m-c SAND, trace Silt

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= GW AASHTO (M 145)= A-1-a

Coefficients

D₉₀= 33.9145 D₈₅= 31.8235 D₆₀= 13.4785
D₅₀= 11.1591 D₃₀= 5.8163 D₁₅= 2.5862
D₁₀= 1.8663 C_u= 7.22 C_c= 1.34

Remarks

Date Received: 06.05.18 Date Tested: 06.13.18

Tested By: MN / JS

Checked By: Rebecca Roth

Title: Laboratory Manager

Source of Sample: BB-MMR-102
Sample Number: 1D

Depth: 0-2'

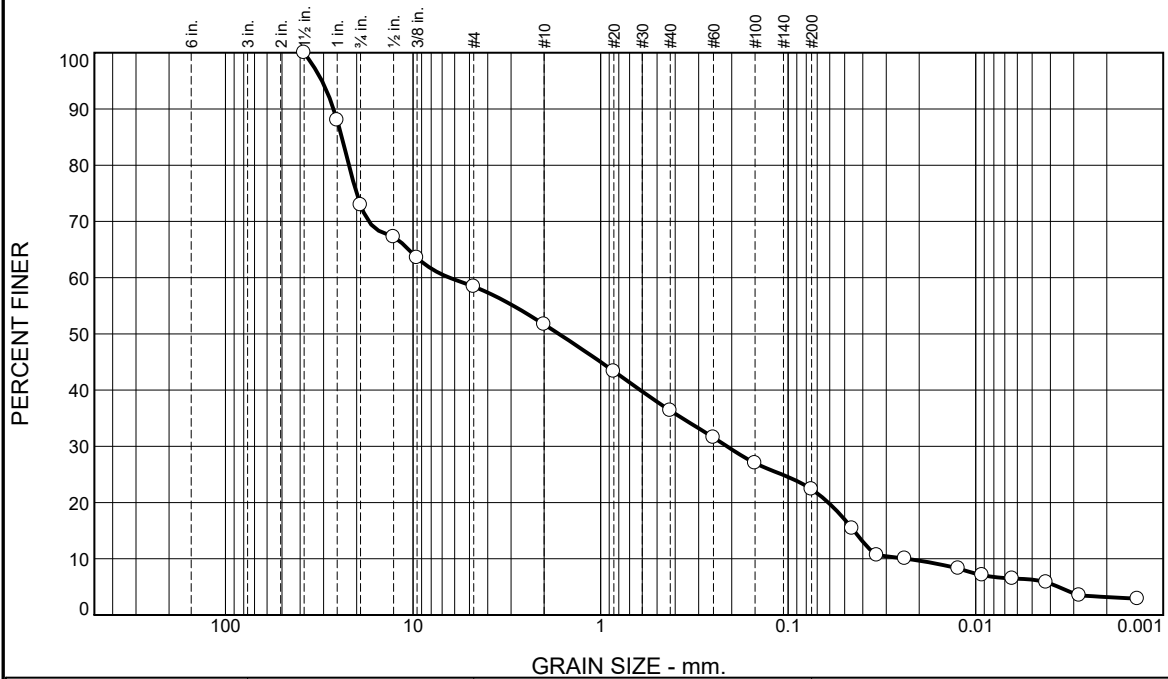
Date Sampled:

Thielsch Engineering Inc.
Cranston, RI

Client: GZA GeoEnvironmental
Project: Mattawamkeag Bridge #2522
Mattawamkeag, ME
Project No: 09.0025975.00

Figure S-5

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	27.1	14.5	6.7	15.3	14.1	19.1	3.2

Test Results (D7928 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1.5"	100.0		
1"	88.0		
0.75"	72.9		
0.5"	67.2		
0.375"	63.5		
#4	58.4		
#10	51.7		
#20	43.4		
#40	36.4		
#60	31.6		
#100	27.0		
#200	22.3		
0.0456 mm.	15.4		
0.0336 mm.	10.6		
0.0239 mm.	10.0		
0.0124 mm.	8.3		
0.0092 mm.	7.1		
0.0064 mm.	6.5		
0.0042 mm.	5.8		
0.0028 mm.	3.5		
0.0014 mm.	2.9		

* (no specification provided)

Material Description

Grey Sandy fine to coarse GRAVEL, some Silt

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= GM AASHTO (M 145)= A-1-b

Coefficients

D₉₀= 26.5169 D₈₅= 23.9801 D₆₀= 6.3873
D₅₀= 1.6692 D₃₀= 0.2119 D₁₅= 0.0445
D₁₀= 0.0235 C_u= 271.85 C_c= 0.30

Remarks

Date Received: 06.04.18 Date Tested: 06.15.18

Tested By: MN

Checked By: _____

Title: _____

Source of Sample: BB-MMR-102
Sample Number: 2D

Depth: 5-7'

Date Sampled:

Thielsch Engineering Inc.

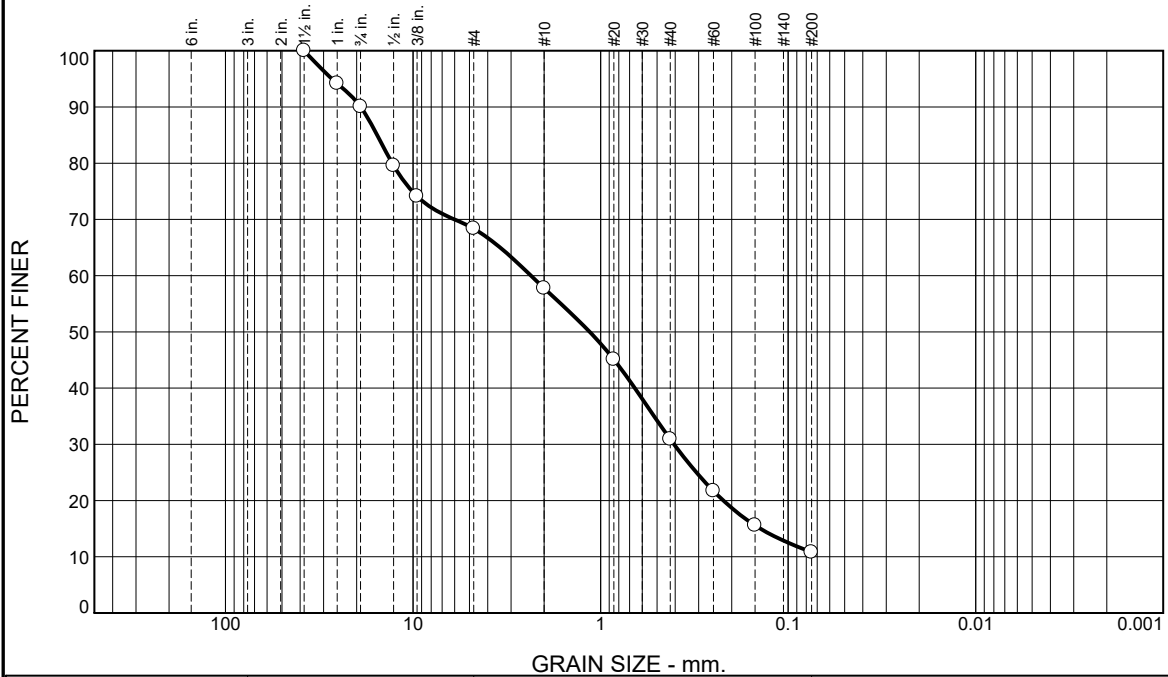
Cranston, RI

Client: GZA GeoEnvironmental
Project: Mattawamkeag Bridge #2522
Mattawamkeag, ME

Project No: 09.0025975.00

Figure S-6

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	10.0	21.6	10.6	26.9	20.1	10.8	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1.5"	100.0		
1"	94.2		
0.75"	90.0		
0.5"	79.6		
0.375"	74.1		
#4	68.4		
#10	57.8		
#20	45.1		
#40	30.9		
#60	21.7		
#100	15.6		
#200	10.8		

* (no specification provided)

Material Description

Brown f-c SAND, some f-c Gravel, little Silt

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= SP-SM AASHTO (M 145)= A-1-b

Coefficients

D₉₀= 19.0065 D₈₅= 15.5337 D₆₀= 2.3508
D₅₀= 1.1430 D₃₀= 0.4059 D₁₅= 0.1406
D₁₀= C_u= C_c=

Remarks

Date Received: 06.05.18 Date Tested: 06.13.18

Tested By: MN / JS

Checked By: Rebecca Roth

Title: Laboratory Manager

Source of Sample: BB-MMR-103
Sample Number: 1D

Depth: 0.8-2.1'

Date Sampled:

Thielsch Engineering Inc.

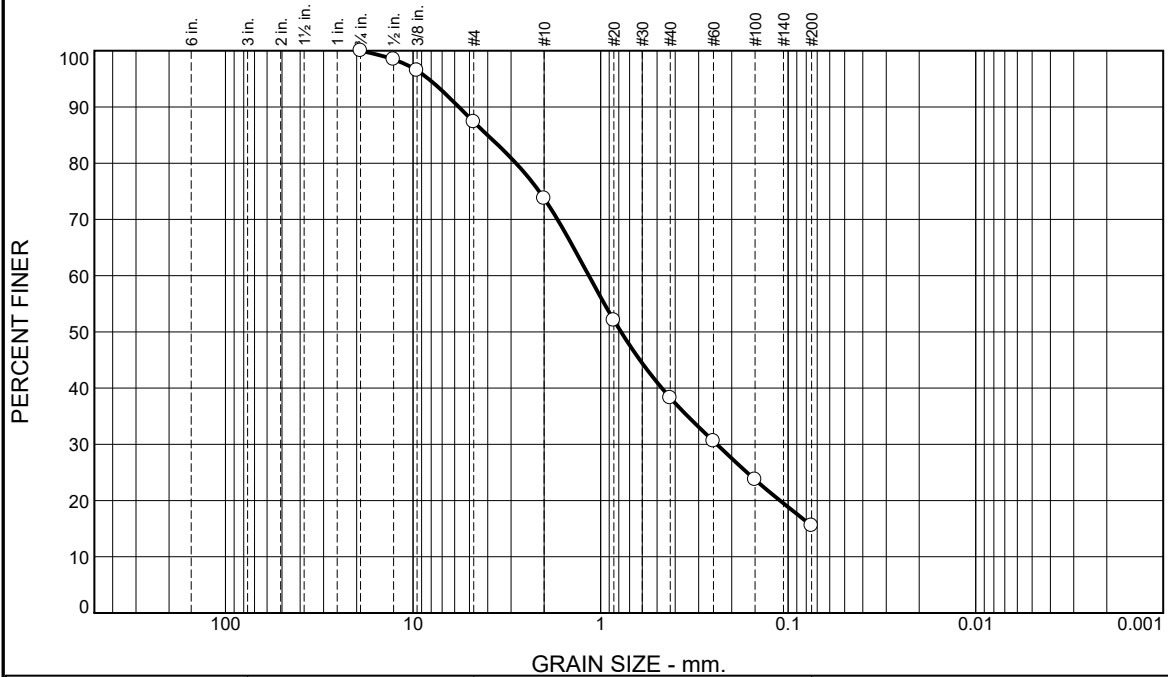
Cranston, RI

Client: GZA GeoEnvironmental
Project: Mattawamkeag Bridge #2522
Mattawamkeag, ME

Project No: 09.0025975.00

Figure S-7

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	12.6	13.7	35.4	22.7	15.6	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.75"	100.0		
0.5"	98.5		
0.375"	96.5		
#4	87.4		
#10	73.7		
#20	52.1		
#40	38.3		
#60	30.6		
#100	23.7		
#200	15.6		

Material Description

Brown f-c SAND, little Silt, little fine Gravel

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= SM AASHTO (M 145)= A-1-b

Coefficients

D₉₀= 5.7012 D₈₅= 4.0046 D₆₀= 1.1551
D₅₀= 0.7787 D₃₀= 0.2400 D₁₅=
D₁₀= C_u= C_c=

Remarks

Date Received: 06.05.18 Date Tested: 06.13.18

Tested By: MN / JS

Checked By: Rebecca Roth

Title: Laboratory Manager

* (no specification provided)

Source of Sample: BB-MMR-103
Sample Number: 2D

Depth: 5-7'

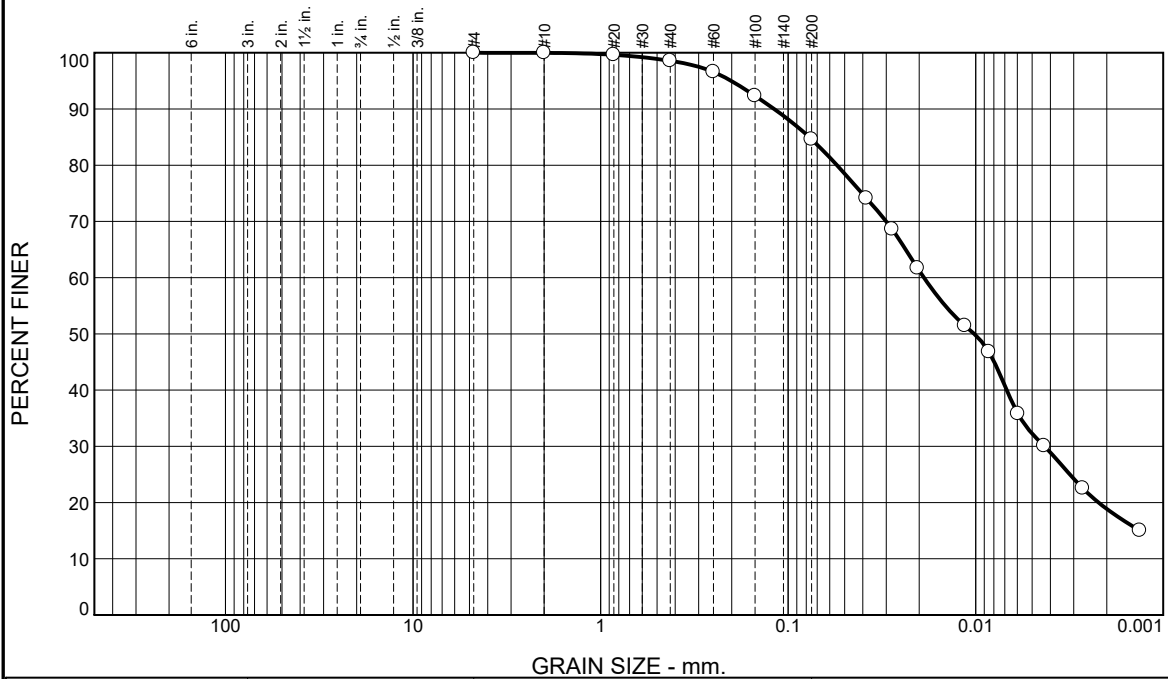
Date Sampled:

Thielsch Engineering Inc.
Cranston, RI

Client: GZA GeoEnvironmental
Project: Mattawamkeag Bridge #2522
Mattawamkeag, ME
Project No: 09.0025975.00

Figure S-8

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	1.4	14.0	65.7	18.9

Test Results (D7928 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	100.0		
#10	100.0		
#20	99.6		
#40	98.6		
#60	96.6		
#100	92.3		
#200	84.6		
0.0383 mm.	74.1		
0.0279 mm.	68.6		
0.0204 mm.	61.7		
0.0114 mm.	51.4		
0.0085 mm.	46.8		
0.0060 mm.	35.8		
0.0043 mm.	30.1		
0.0027 mm.	22.5		
0.0013 mm.	15.0		

* (no specification provided)

Material Description

Light Brown CLAY & SILT, little fine Sand

Atterberg Limits (ASTM D 4318)

PL= 25 LL= 36 PI= 11

Classification

USCS (D 2487)= ML AASHTO (M 145)= A-6(10)

Coefficients

D₉₀= 0.1186 D₈₅= 0.0772 D₆₀= 0.0189
D₅₀= 0.0102 D₃₀= 0.0043 D₁₅=
D₁₀= C_u= C_c=

Remarks

Date Received: 06.05.18 Date Tested: 06.15.18

Tested By: MN

Checked By: Rebecca Roth

Title: Laboratory Manager

Source of Sample: BB-MMR-103
Sample Number: 4D

Depth: 15-17'

Date Sampled:

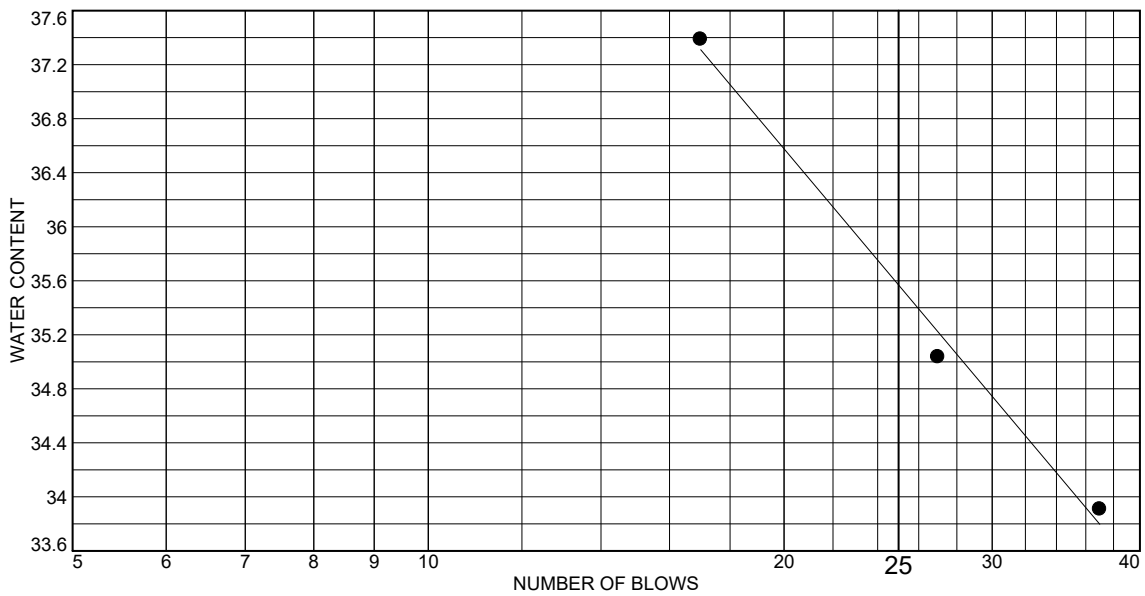
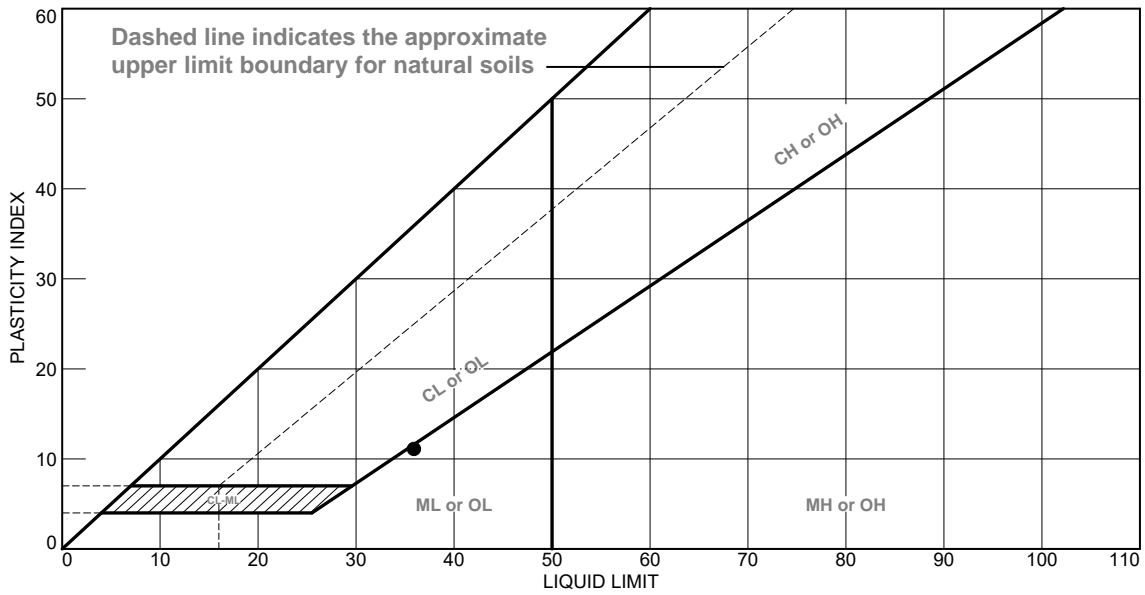
Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Mattawamkeag Bridge #2522
Mattawamkeag, ME
Project No: 09.0025975.00

Figure S-9

LIQUID AND PLASTIC LIMITS TEST REPORT



MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
● Light Brown CLAY & SILT, little fine Sand	36	25	11	98.6	84.6	ML

Project No. 09.0025975.00 **Client:** GZA GeoEnvironmental
Project: Mattawamkeag Bridge #2522
 Mattawamkeag, ME
Source of Sample: BB-MMR-103 **Depth:** 15-17'
Sample Number: 4D

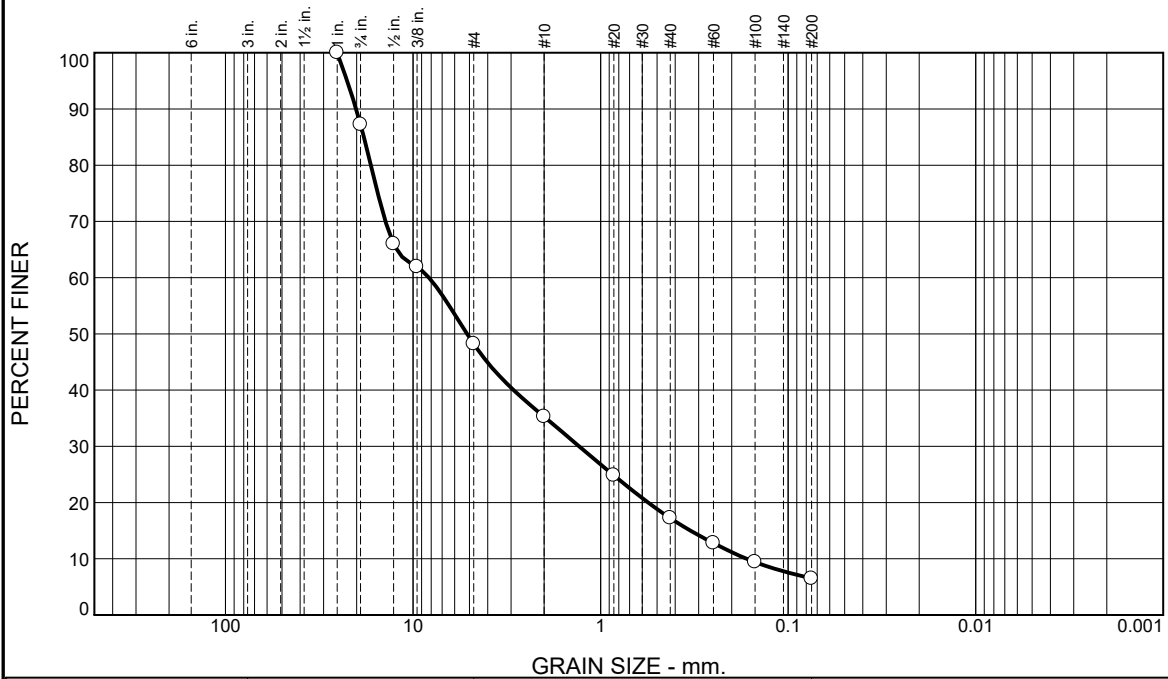
Remarks:

Thielsch Engineering Inc.
Cranston, RI

Figure L-9

Tested By: MN **Checked By:** RR

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	12.8	39.0	12.9	18.1	10.7	6.5	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	87.2		
0.5"	66.0		
0.375"	61.9		
#4	48.2		
#10	35.3		
#20	24.8		
#40	17.2		
#60	12.8		
#100	9.4		
#200	6.5		

Material Description

Brown Sandy fine to coarse GRAVEL, trace Silt

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= GW-GM AASHTO (M 145)= A-1-a

Coefficients

D₉₀= 20.1137 D₈₅= 18.2952 D₆₀= 8.2259
D₅₀= 5.1549 D₃₀= 1.3003 D₁₅= 0.3312
D₁₀= 0.1664 C_u= 49.44 C_c= 1.24

Remarks

Date Received: 06.05.18 Date Tested: 06.13.18

Tested By: MN / JS

Checked By: Rebecca Roth

Title: Laboratory Manager

* (no specification provided)

Source of Sample: BB-MMR-103
Sample Number: 5D

Depth: 20-22'

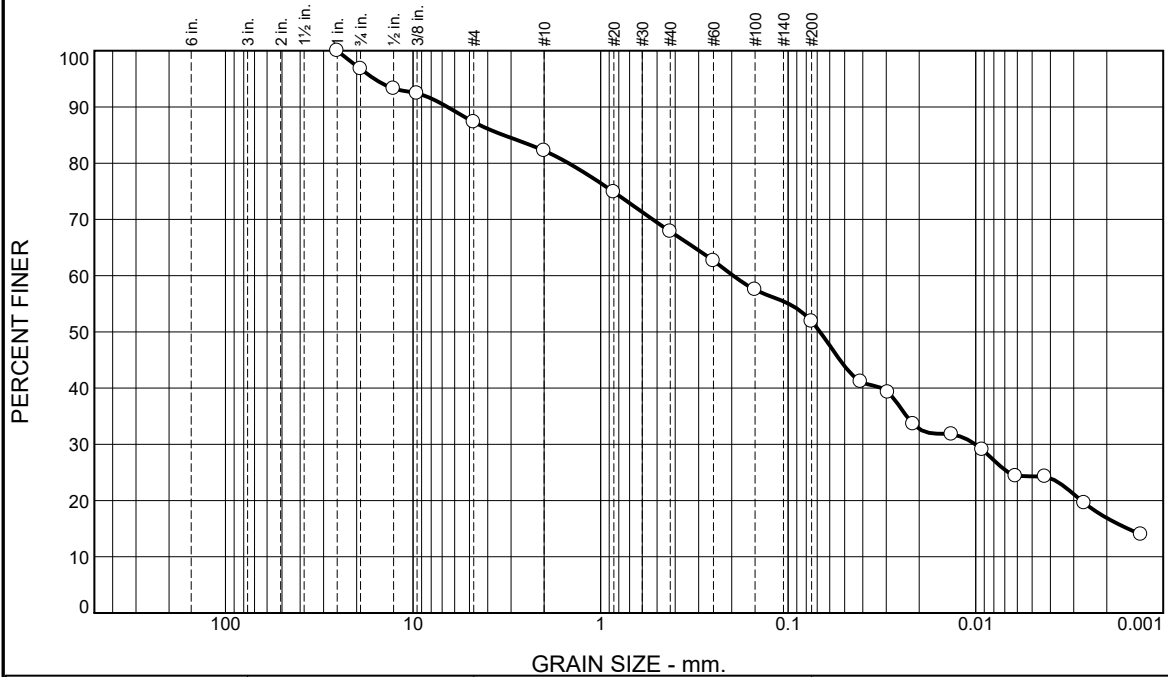
Date Sampled:

Thielsch Engineering Inc.
Cranston, RI

Client: GZA GeoEnvironmental
Project: Mattawamkeag Bridge #2522
Mattawamkeag, ME
Project No: 09.0025975.00

Figure S-10

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	3.2	9.5	5.1	14.3	16.0	35.0	16.9

Test Results (D7928 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	96.8		
0.5"	93.3		
0.375"	92.4		
#4	87.3		
#10	82.2		
#20	74.9		
#40	67.9		
#60	62.6		
#100	57.5		
#200	51.9		
0.0410 mm.	41.2		
0.0294 mm.	39.3		
0.0216 mm.	33.6		
0.0135 mm.	31.8		
0.0092 mm.	29.0		
0.0061 mm.	24.4		
0.0043 mm.	24.2		
0.0026 mm.	19.6		
0.0013 mm.	14.0		

* (no specification provided)

Material Description

Grey Sandy SILT & CLAY, little fine Gravel

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= ML AASHTO (M 145)= A-4(0)

Coefficients

D₉₀= 6.5467 D₈₅= 3.2919 D₆₀= 0.1962
D₅₀= 0.0675 D₃₀= 0.0100 D₁₅= 0.0015
D₁₀= C_u= C_c=

Remarks

Sample visually classified as plastic. Sample rolled to 1/8".

Date Received: 06.04.18 Date Tested: 06.15.18

Tested By: MN

Checked By: Steven Accetta

Title: Laboratory Manager

Source of Sample: BB-MMR-103
Sample Number: 7D

Depth: 30-32'

Date Sampled:

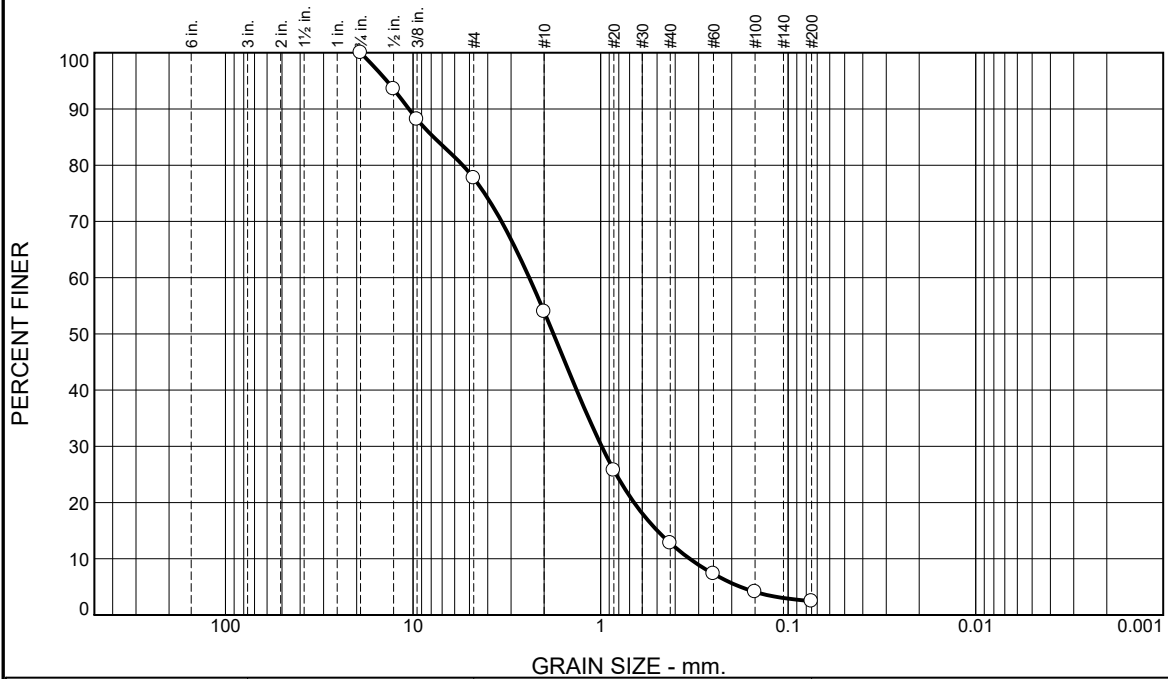
Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Mattawamkeag Bridge #2522
Mattawamkeag, ME
Project No: 09.0025975.00

Figure S-11

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	22.3	23.8	41.1	10.3	2.5	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.75"	100.0		
0.5"	93.6		
0.375"	88.2		
#4	77.7		
#10	53.9		
#20	25.8		
#40	12.8		
#60	7.3		
#100	4.1		
#200	2.5		

* (no specification provided)

Material Description

Grey f-c SAND, some fine Gravel, trace Silt

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= SW AASHTO (M 145)= A-1-b

Coefficients

D₉₀= 10.5362 D₈₅= 7.7674 D₆₀= 2.4018
D₅₀= 1.7846 D₃₀= 0.9875 D₁₅= 0.4982
D₁₀= 0.3343 C_u= 7.18 C_c= 1.21

Remarks

Date Received: 06.05.18 Date Tested: 06.13.18

Tested By: MN / JS

Checked By: Rebecca Roth

Title: Laboratory Manager

Source of Sample: BB-MMR-103
Sample Number: 8D

Depth: 35-37'

Date Sampled:

Thielsch Engineering Inc.
Cranston, RI

Client: GZA GeoEnvironmental
Project: Mattawamkeag Bridge #2522
Mattawamkeag, ME
Project No: 09.0025975.00

Figure S-12



195 Frances Avenue
 Cranston RI, 02910
 Phone: (401)-467-6454
 Fax: (401)-467-2398
<http://www.thielsch.com>
Let's Build a Solid Foundation

Client Information:
 GZA GeoEnvironmental, Inc
 Portland, ME
 PM: Theodore Baire
 Assigned By: EDF
 Collected By: EDF

Project Information:
Mattawamkeag Bridge #2522, WIN 23234.00
Mattawamkeag, ME
 GZA Project Number: 09.0025975.00
 Summary Page: 1 of 1
 Report Date: 06.19.18

LABORATORY TESTING DATA SHEET

Boring No.	Sample No.	Depth (ft)	Laboratory No.	Specimen Data						Compressive Strength Tests								Rock Formation or Description or Remarks	
				Mohs Hardness	Diameter (in)	Length (in)	(1) Unit Weight (PCF)	(2) Wet Density (PCF)	Bulk G _s	(3) Other Tests	(4) Strength PSI	(5) Strain %	(6) E sec PSI EE+06	(7) Poisson's Ratio	σ _t PSI	I _{S50} PSI	(8) s _c PSI		
BB-MMR-101	R1	34.4-35.3	S-13		1.996	4.631	169.2					23157	0.435	4.02	0.09				SLATE; fresh break
Note: Minor break occurred around 1.6k psi.																			
BB-MMR-102	R3	13.2-13.9	S-14		1.990	4.780	164.9					25336	0.315	6.23	0.11				SLATE; fresh break
Note: Minor break occurred around 17k psi. Break did not affect Poisson's Ratio or Secant modulus since it occurred after 50% of Total Failure Stress.																			
(1) Volume Determined By Measuring Dimensions				Notes	(3) PLD=Point Load (diametrical),						Notes	(5) Strain at Peak Deviator Stress							
(2) Determined by Measuring Dimensions and Weight of Saturated Sample					PLA= Point Load (Axial) ST= Splitting Tensile							(6) Represents Secant Modulus at 50% of Total Failure Stress							
					U= Unconfined Compressive Strength							(7) Represents Secant Poisson's Ratio at 50% of Total Failure Stress							
					(4) Taken at Peak Deviator Stress							(8) Estimated UCS from Table 1 of ASTM D5731 for NX cores (I _s x 24)							

Reviewed By SKW

Date Reviewed 06.20.18



195 Frances Avenue
 Cranston RI, 02910
 Phone: (401)-467-6454
 Fax: (401)-467-2398
<http://www.thielsch.com>
Let's Build a Solid Foundation

Client Information:
 GZA GeoEnvironmental, Inc
 Portland, ME
 PM: Theodore Baire
 Assigned By: EDF
 Collected By: EDF

Project Information:
Mattawamkeag Bridge #2522, WIN 23234.00
Mattawamkeag, ME
 GZA Project Number: 09.0025975.00
 Summary Page: 1 of 1
 Report Date: 07.11.2018

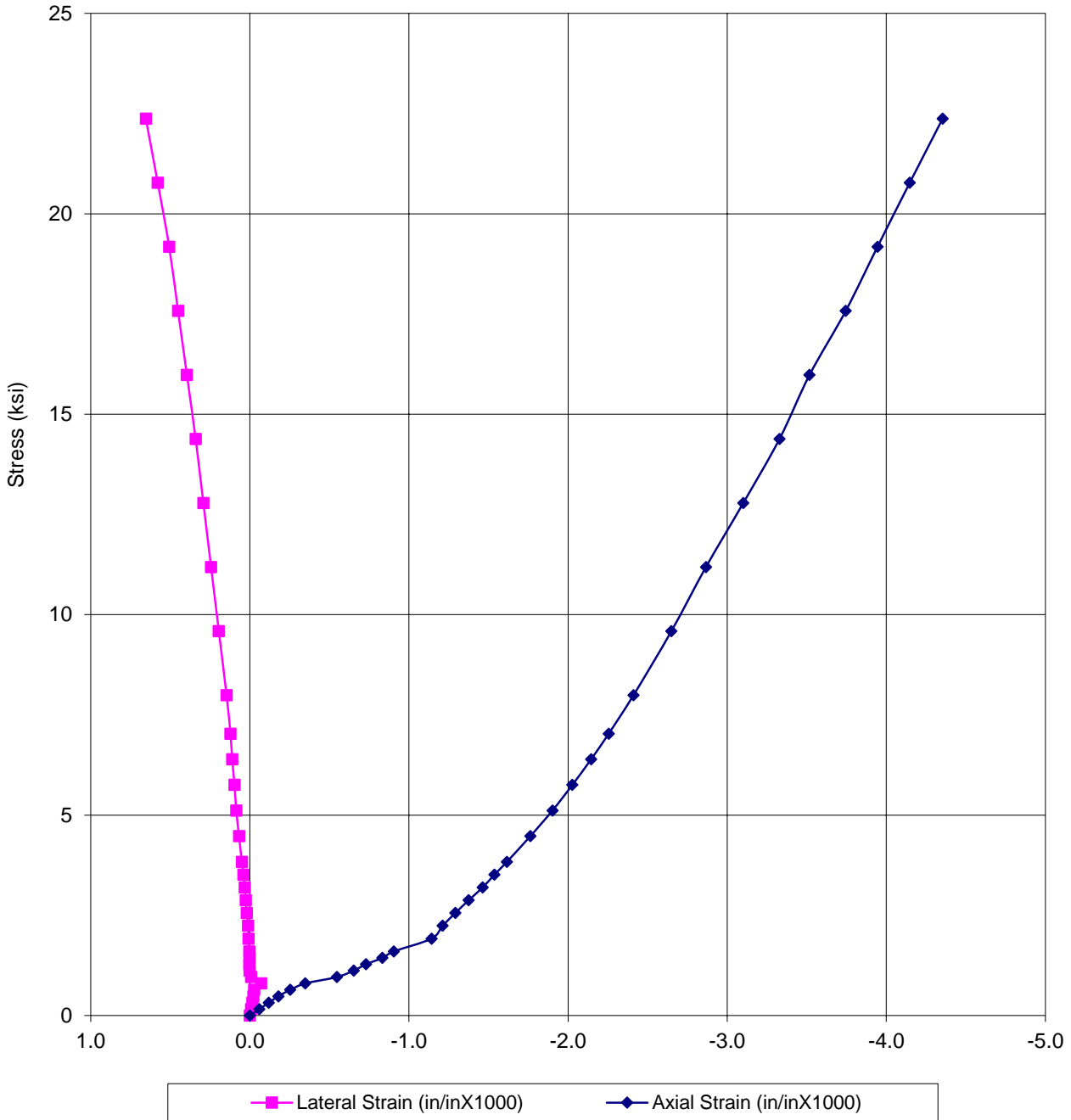
LABORATORY TESTING DATA SHEET

Boring No.	Sample No.	Depth (ft)	Laboratory No.	Specimen Data						Compressive Strength Tests								Rock Formation or Description or Remarks		
				Mohs Hardness	Diameter (in)	Length (in)	(1) Unit Weight (PCF)	(2) Wet Density (PCF)	Bulk G _s	(3) Other Tests	(4) Strength PSI	(5) Strain %	(6) E sec PSI EE+06	(7) Poisson's Ratio	σ _t PSI	I _{S50} PSI	(8) s _c PSI			
BB-MMR-103	R5	17-17.8	S-16		1.990	4.565	170.2					13092	0.349	2.87	0.52				SCHIST; fresh break	
<p>Note: Minor breaks occurred around 5.7 kpsi and 12.8 kpsi. The minor break that occurred slightly after 5.7 kpsi did affect Poisson's Ratio so Poisson's Ratio was taken at roughly 44%. Sample was test in air dried conditions.</p>																				
(1) Volume Determined By Measuring Dimensions				Notes	(3) PLD=Point Load (diametrical),						Notes	(5) Strain at Peak Deviator Stress								
(2) Determined by Measuring Dimensions and Weight of Saturated Sample					PLA= Point Load (Axial) ST= Splitting Tensile							(6) Represents Secant Modulus at 44% of Total Failure Stress								
					U= Unconfined Compressive Strength							(7) Represents Secant Poisson's Ratio at 44% of Total Failure Stress								
					(4) Taken at Peak Deviator Stress							(8) Estimated UCS from Table 1 of ASTM D5731 for NX cores (I _s x 24)								

Reviewed By SKW

Date Reviewed 07.12.2018

**Mattawamkeag Bridge #2522
Mattawamkeag, ME**



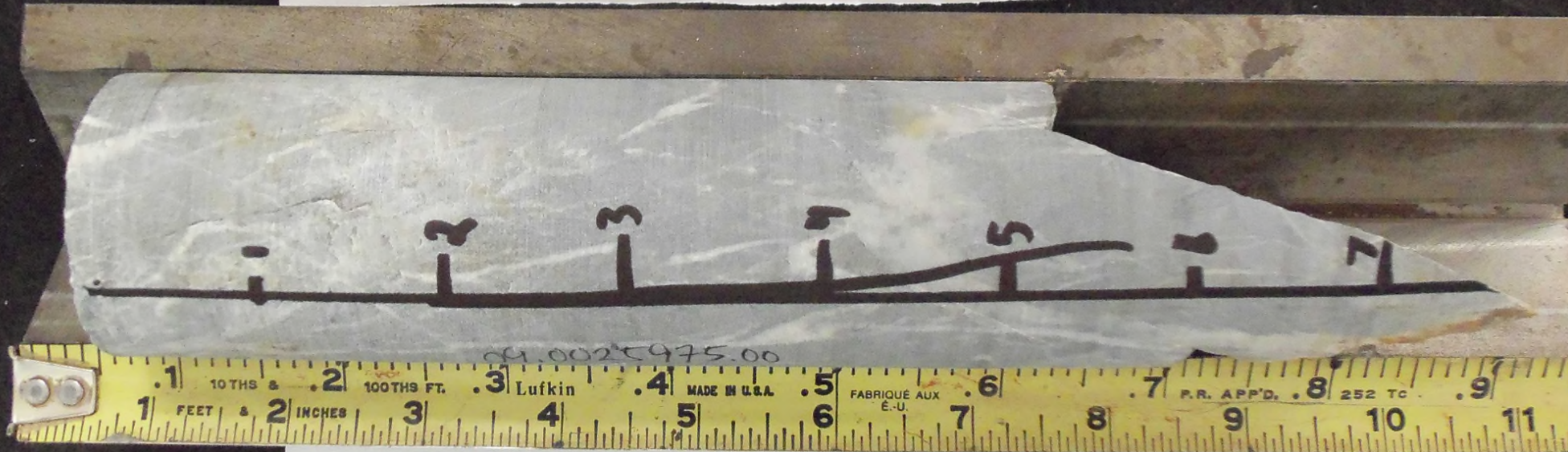
Rock Unconfined Compression Testing - ASTM D7012

Boring No. BB-MMR-101
 Sample No. R-1
 Depth: 34.4-35.3

File No. 09.0025975.00
 Date: 06.13.18
 Test No. U-13



Mattawamkeag Bridge
Mattawamkeag, ME
09.0025975.00



Boring No. Sample No. Depth

BB-MMR-101

R1

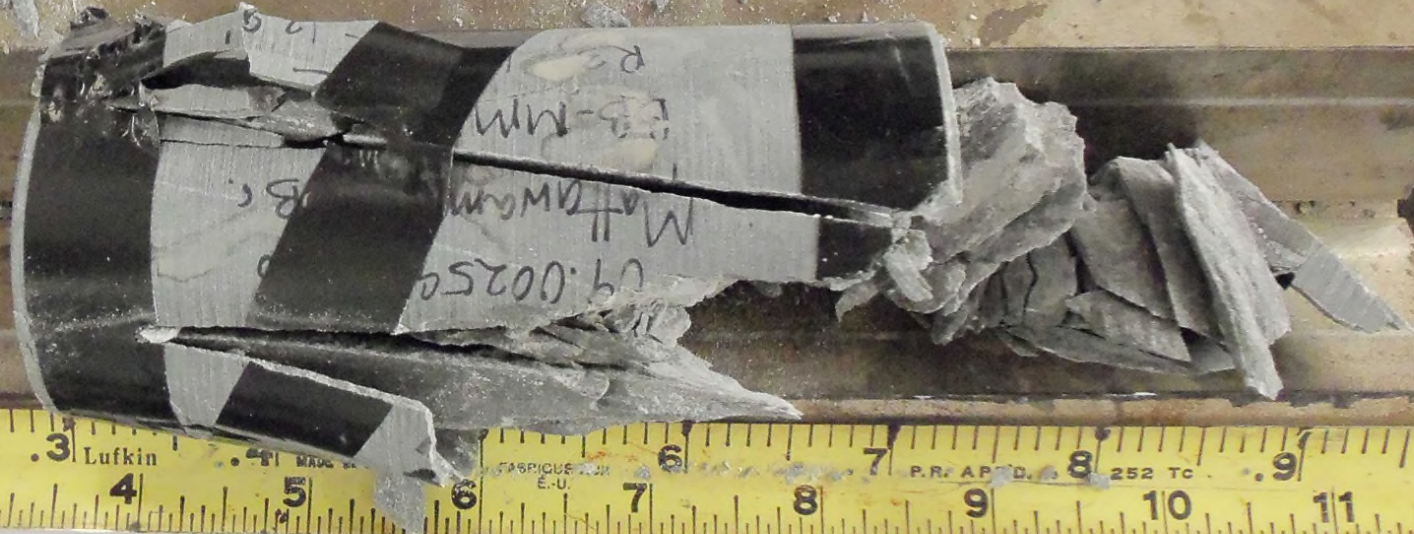
34.4-35.3'

Mattawamkeag Bridge
Mattawamkeag, ME
09.0025975.00



Boring No.	Sample No.	Depth
<u>BB-MMR-101</u>	<u>R1</u>	<u>34.4-35.3'</u>

Mattawamkeag Bridge
Mattawamkeag, ME
09.0025975.00



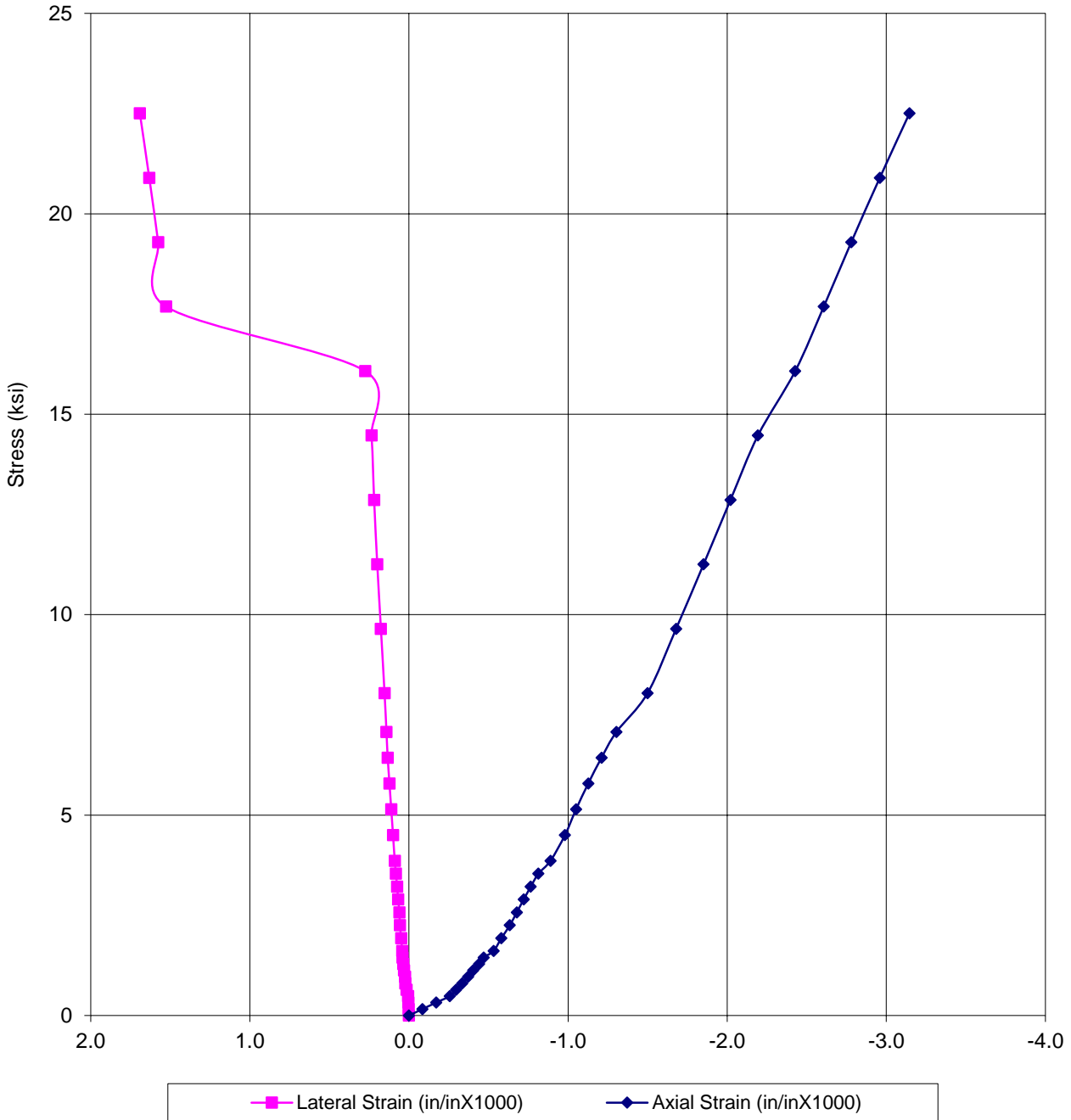
Boring No.	Sample No.	Depth
<u>BB-MMR-101</u>	<u>R1</u>	<u>34.4-35.3'</u>

Mattawamkeag Bridge
Mattawamkeag, ME
09.0025975.00



Boring No.	Sample No.	Depth
<u>BB-MMR-101</u>	<u>R1</u>	<u>34.4-35.3'</u>

**Mattawamkeag Bridge #2522
Mattawamkeag, ME**



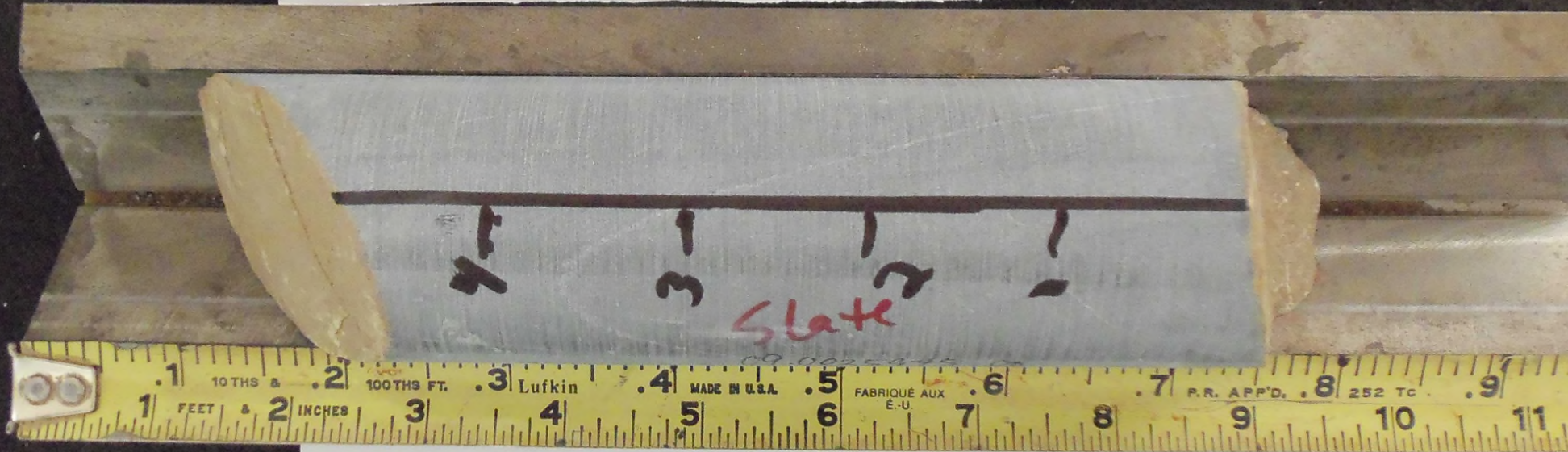
Rock Unconfined Compression Testing - ASTM D7012

Boring No. BB-MMR-102
 Sample No. R-3
 Depth: 13.2-13.9

File No. 09.0025975.00
 Date: 06.13.18
 Test No. U-14



Mattawamkeag Bridge
Mattawamkeag, ME
09.0025975.00



Boring No.

Sample No.

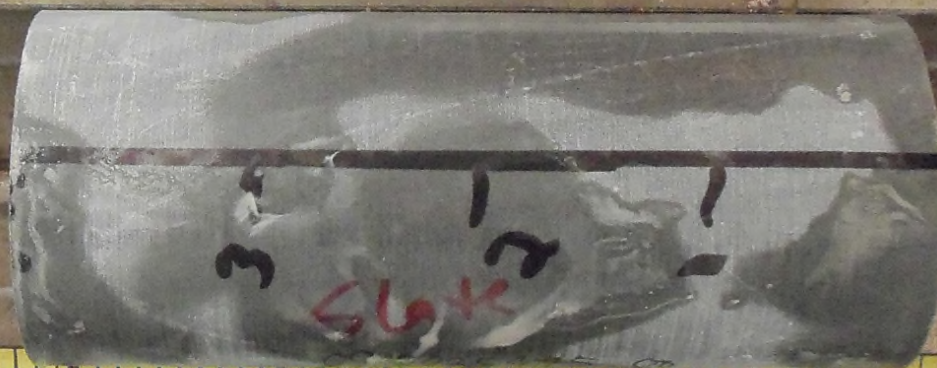
Depth

BB-MMR-102

R3

132-13.9'

Mattawamkeag Bridge
Mattawamkeag, ME
09.0025975.00



Boring No.

Sample No.

Depth

BB-MMR-102

R3

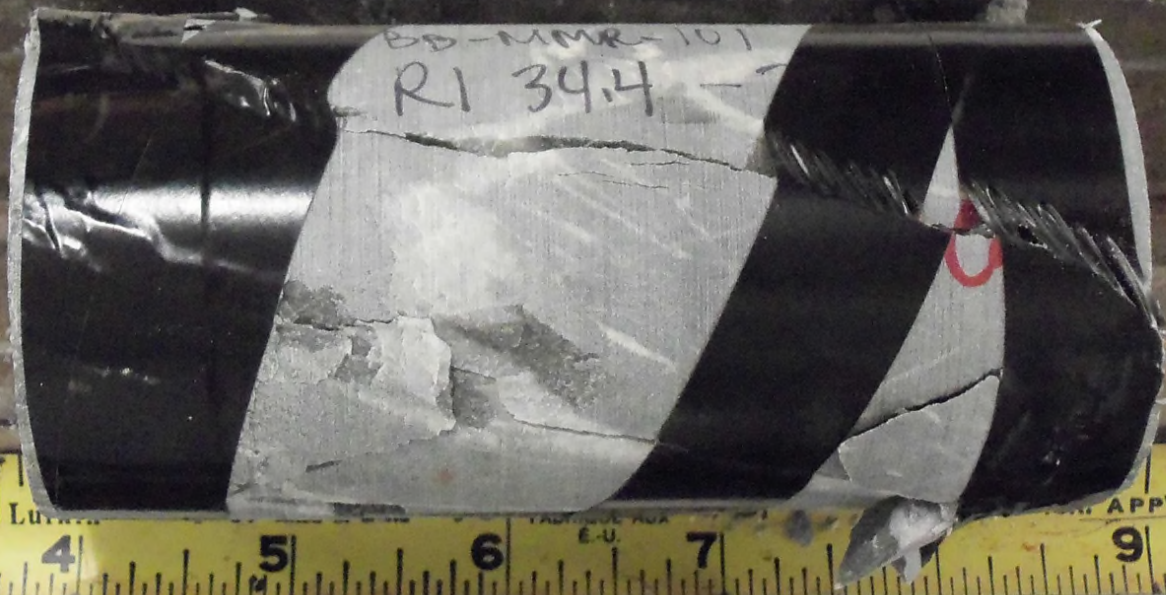
132-13.9'

Mattawamkeag Bridge
Mattawamkeag, ME
09.0025975.00



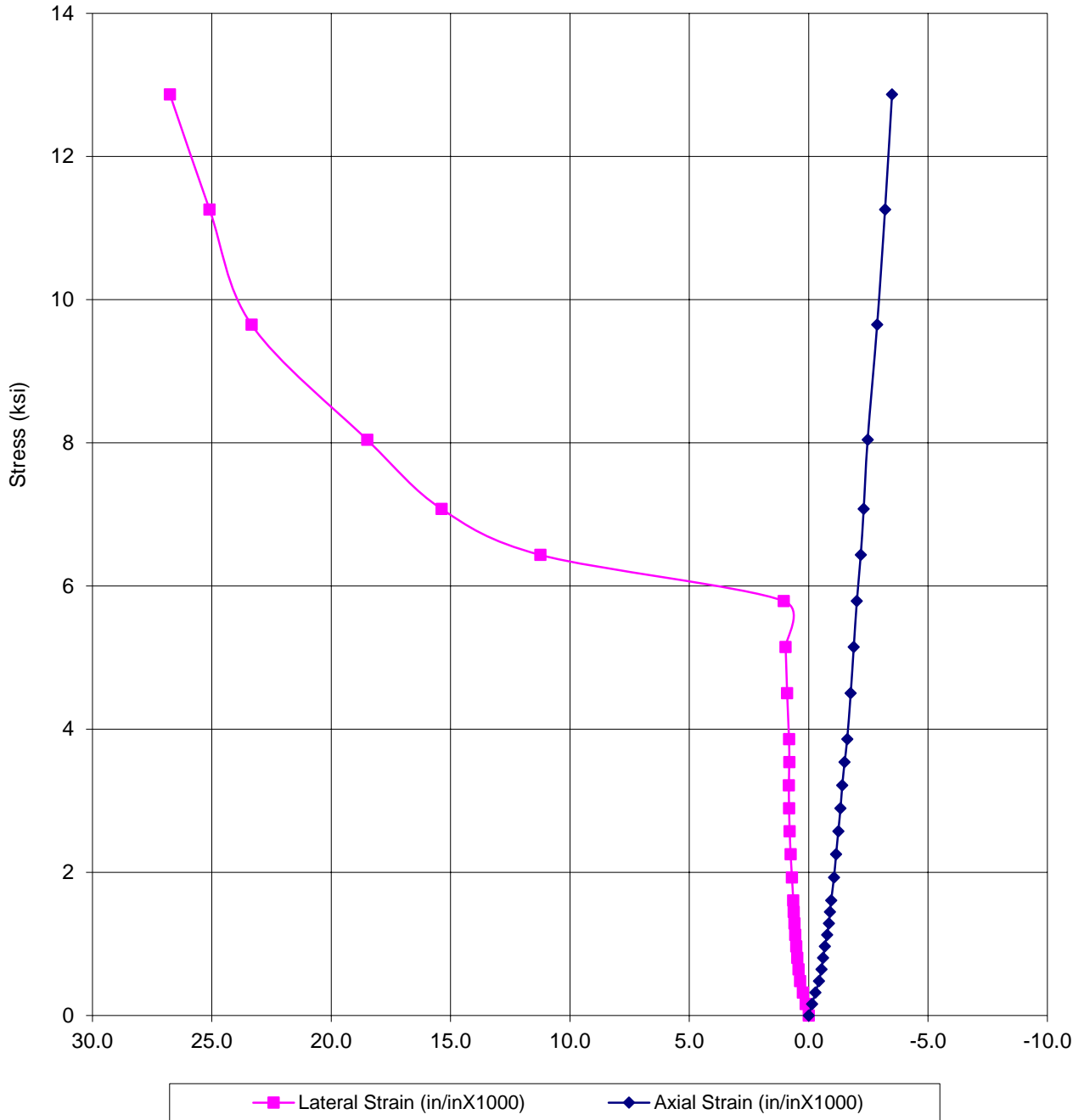
Boring No.	Sample No.	Depth
<u>BB-MMR-102</u>	<u>R3</u>	<u>132-13.9'</u>

Mattawamkeag Bridge
Mattawamkeag, ME
09.0025975.00



Boring No.	Sample No.	Depth
<u>BB-MMR-102</u>	<u>R3</u>	<u>132-13.9'</u>

Mattawamkeag Bridge #2522
Mattawamkeag, ME



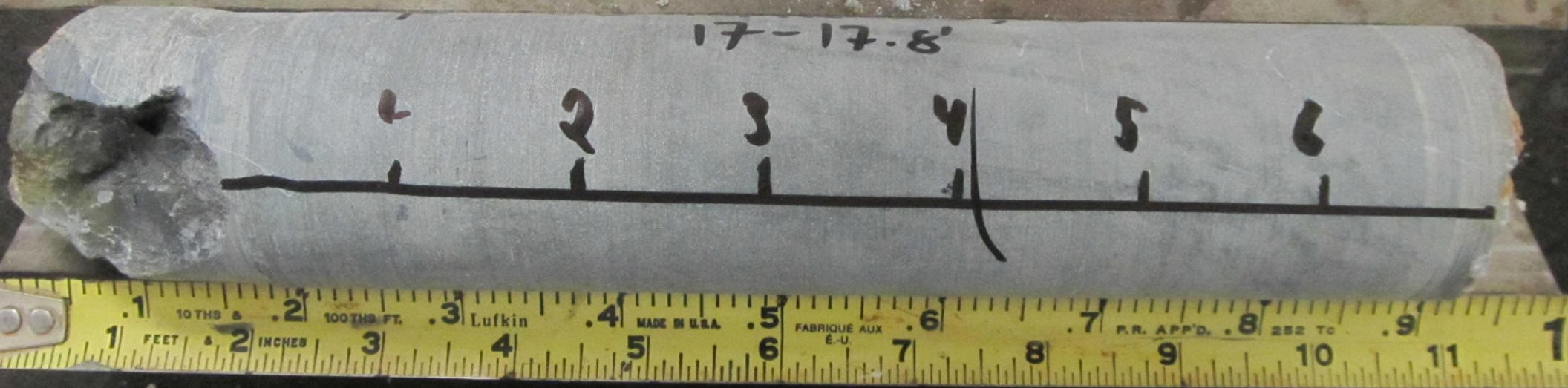
Rock Unconfined Compression Testing - ASTM D7012

Boring No. BB-MMR-103
Sample No. R5
Depth: 17-17.8

File No. 09.0025975.00
Date: 07.11.2018
Test No. U-16

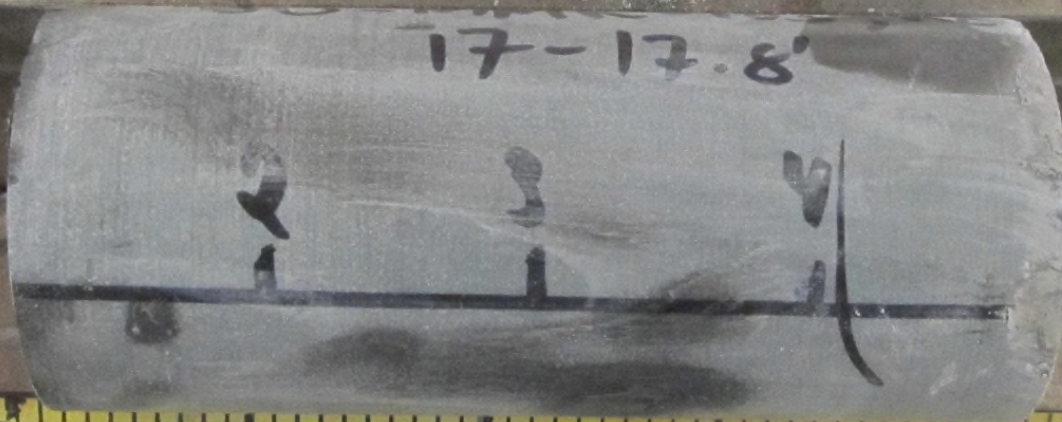


Mattawamkeag Bridge #2522, WIN 23234.00
Mattawamkeag, ME
09.0025975.00



Boring No.	Sample No.	Depth
<u>BB-MMR-103</u>	<u>R5</u>	<u>17-17.8'</u>

Mattawamkeag Bridge #2522, WIN 23234.00
Mattawamkeag, ME
09.0025975.00



Boring No.

Sample No.

Depth

BB-MMR-103

R5

17-17.8'

Mattawamkeag Bridge
Mattawamkeag, ME
09.0025975.00



Boring No.	Sample No.	Depth
<u>BB-MMR-103</u>	<u>R5</u>	<u>17-17.8'</u>

Mattawamkeag Bridge
Mattawamkeag, ME
09.0025975.00



Boring No.	Sample No.	Depth
<u>BB-MMR-103</u>	<u>R5</u>	<u>17-17.8'</u>



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 Fax: (401)-467-2398
thielsch.com
Let's Build a Solid Foundation

Client Information:
 GZA Geoenvironmental
 Portland, ME
 PM: Blaine Cardali
 Assigned By: Blaine Cardali
 Collected By: Blaine Cardali

Project Information:
Mattawamkeag Bridge #2552, WIN 23234.00
Mattawamkeag, ME
 GZA Project Number: 09.0025975.01
 Summary Page: 1 of 1
 Report Date: 11.08.19

LABORATORY TESTING DATA SHEET, Report No.: 7419-L-124

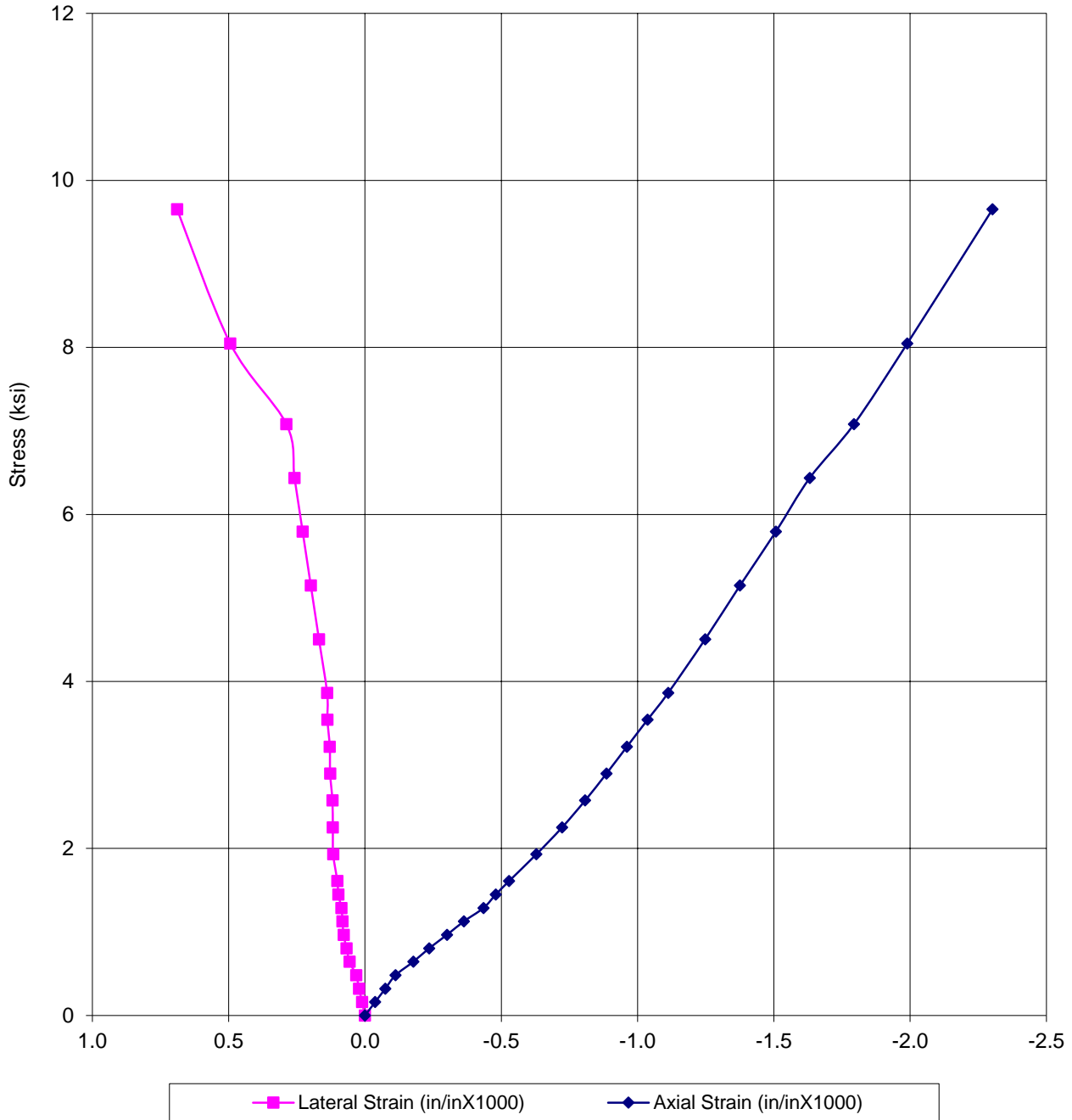
Boring No.	Sample No.	Depth (ft)	Laboratory No.	Specimen Data					Compressive Strength Tests								Rock Formation or Description or Remarks				
				Mohs Hardness	Diameter (in)	Length (in)	(1) Unit Weight (PCF)	(2) Wet Density (PCF)	Bulk G _s	(3) Other Tests	(4) Strength PSI	(5) Strain %	(6) E sec PSI EE+06	(7) Poisson's Ratio	σ _t PSI	I _{s50} PSI		(8) s _c PSI			
BB-MMR-201	R3	12.5-13.9	19-S-2420		1.989	4.775	167.5					9861	0.230	3.68	0.14				Slate with Quartz veins		
Broke along existing fracture and quartz vein.																					
BB-MMR-204	R4	15-16.6	19-S-2421		2.002	4.566	166.2					1976	0.078	2.7	0.68				Slate with Quartz veins		
Broke along foliation.																					
(1) Volume Determined By Measuring Dimensions				Notes	(3) PLD=Point Load (diametrical),								Notes	(5) Strain at Peak Deviator Stress							
(2) Determined by Measuring Dimensions and Weight of Saturated Sample					PLA= Point Load (Axial) ST= Splitting Tensile									(6) Represents Secant Modulus at 50% of Total Failure Stress							
					U= Unconfined Compressive Strength									(7) Represents Secant Poisson's Ratio at 50% of Total Failure Stress							
					(4) Taken at Peak Deviator Stress									(8) Estimated UCS from Table 1 of ASTM D5731 for NX cores (Is x 24)							

Date Received: 11.06.19

Reviewed By: 

Date Reviewed: 11.08.19

**Mattawamkeag Bridge
Mattawamkeag, ME**



Rock Unconfined Compression Testing - ASTM D7012

Boring No. BB-MMR-201
 Sample No. R3
 Depth: 12.5-13.9'

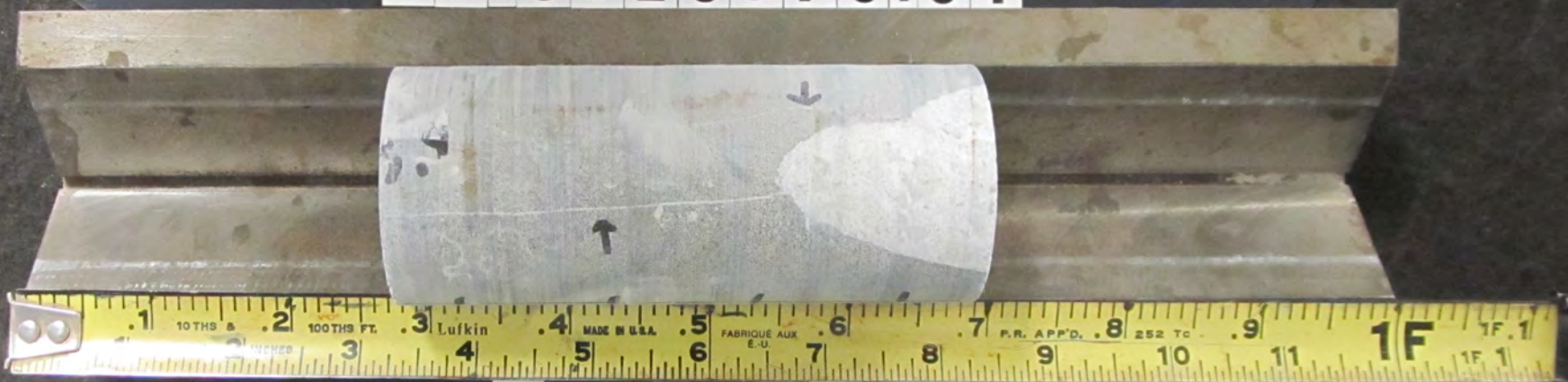
File No. 09.0025975.01
 Date: 11.07.19
 Test No. 19-S-2420



MATTAWAMKEAG BRIDGE

2522, WIN 23234.00

09.0025975.01



B-MMR-202

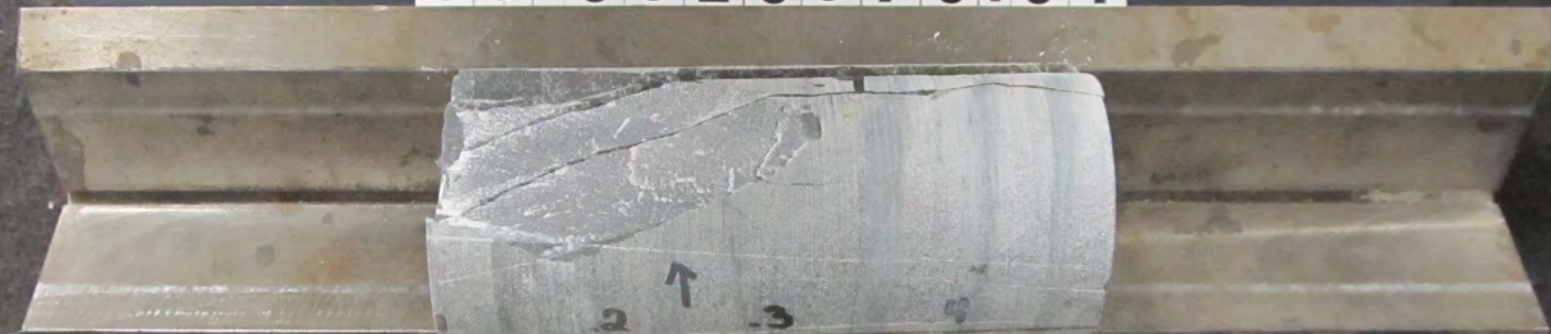
R3

12.5 = 13.9'

MATTAWAMKEAG BRIDGE

2522, WIN 23234.00

09.0025975.01



B-MMR-202

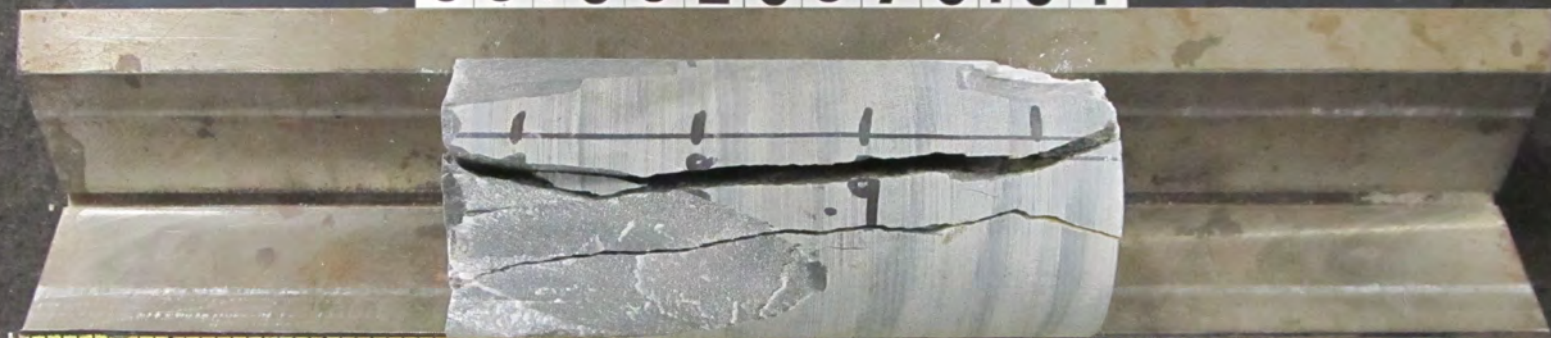
R3

12.5 = 13.9,

MATTAWAMKEAG BRIDGE

2522, WIN 23234.00

09.0025975.01



B-MMR-202

R3

12.5 - 13.9,

MATTAWAMKEAG BRIDGE

2522, WIN 23234.00

09.0025975.01

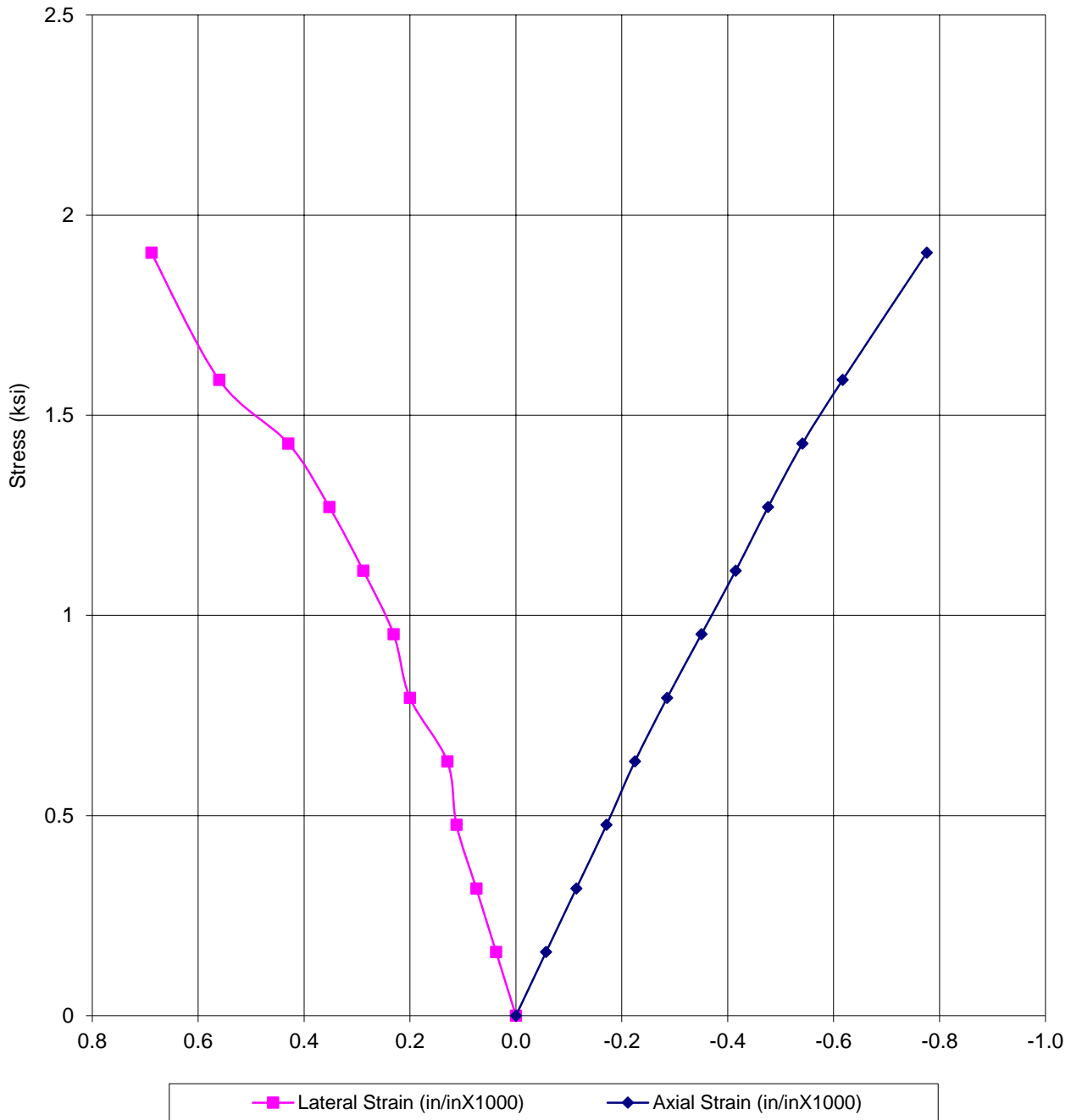


B-MMR-202

R3

12.5-13.9,

**Mattawamkeag Bridge
Mattawamkeag, ME**



Rock Unconfined Compression Testing - ASTM D7012

Boring No. BB-MMR-204
 Sample No. R4
 Depth: 15-16.6'

File No. 09.0025975.01
 Date: 11.07.19
 Test No. 19-S-2421



MATTAWAMKEAG BRIDGE

2522, WIN 23234.00

09.0025975.01



B-MMR-204

R4

15 = 16.6'

MATTAWAMKEAG BRIDGE

2522, WIN 23234.00

09.0025975.01



B-MMR-204

R4

15 - 16.6'

MATTAWAMKEAG BRIDGE

2522, WIN 23234.00

09.0025975.01



B-MMR-204

R4

15-16.6'

MATTAWAMKEAG BRIDGE

2522, WIN 23234.00

09.0025975.01



B-MMR-204

R4

15 = 16.6'



06/08/2020

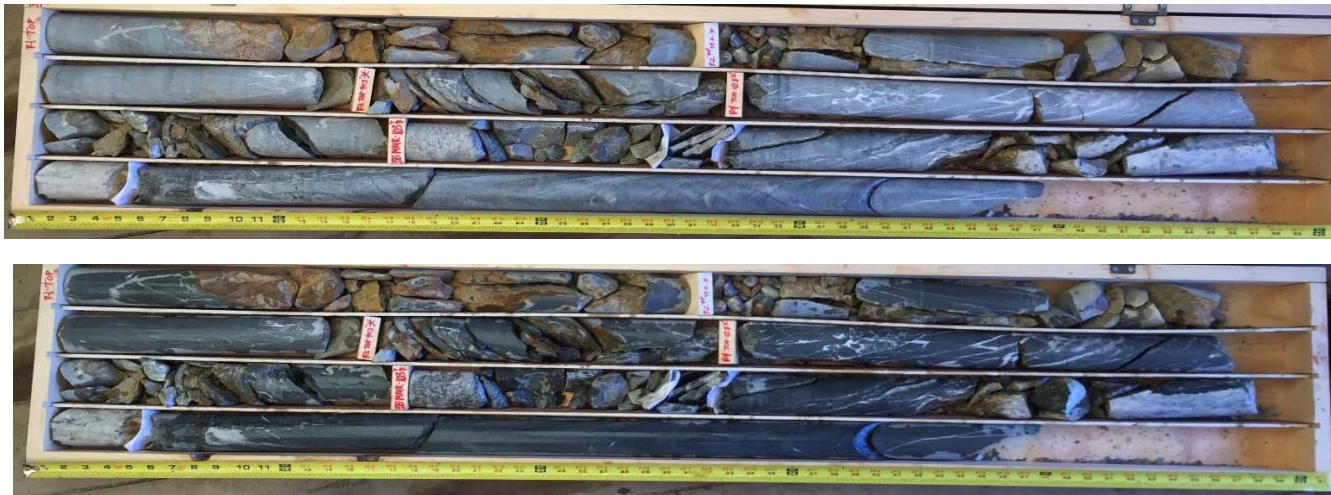
GEOTECHNICAL DESIGN REPORT
MATTAWAMKEAG RIVER BRIDGE NO. 2522 - MATTAWAMKEAG
09.0025975.01

APPENDIX D – BEDROCK CORE PHOTOGRAPHS



MaineDOT Mattawamkeag Bridge #2522
US Route 2 over Mattawamkeag River
Mattawamkeag, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-MMR-101	R1	34.4 - 37.2	28	85	8	24	SLATE	1
BB-MMR-101	R2	37.2 - 40.3	34	92	11	30	SLATE	1,2
BB-MMR-101	R3	40.3 - 41.8	17	94	0	0	SLATE	2
BB-MMR-101	R4	41.8 - 44.4	31	100	21	69	SLATE	2,3
BB-MMR-103	R1	49.7 - 50.9	9	64	0	0	GRANITE BLOCK/SCHIST	3
BB-MMR-103	R2	53.0 - 53.5	4	67	0	0	SCHIST	3
BB-MMR-103	R3	53.5 - 56.5	30	83	9	25	SCHIST	3,4
BB-MMR-103	*R4	56.5 - 61.5	60	100	60	100	SCHIST	4

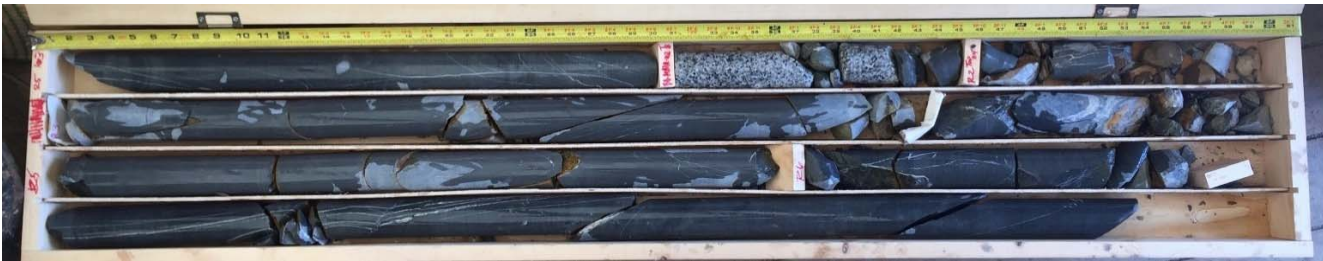


- Notes:**
1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 4=Bottom.
 2. Top photo is dry, bottom photo is wet.
 3. Transition between core runs within a row are marked by wood or paper separators.
- *Bottom 18 inches of R4 were recovered in R5, see next page



MaineDOT Mattawamkeag Bridge #2522
US Route 2 over Mattawamkeag River
Mattawamkeag, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-MMR-103	R5	61.5 - 63	10	56	10	56	SCHIST	1
BB-MMR-102	R1	8.9 - 11.4	17	57	NA	NA	GRANITE BOULDER/SLATE	1
BB-MMR-102	R2	11.4 - 12.2	9	100	0	0	SLATE	1
BB-MMR-102	R3	12.2 - 15.8	43	100	18	42	SLATE	2
BB-MMR-102	R4	15.8 - 17	14	100	0	0	SLATE	2
BB-MMR-102	R5	17 - 19.8	34	100	24	71	SLATE	3
BB-MMR-102	R6	19.8 - 21.3	18	100	6	33	SLATE	3
BB-MMR-102	R7	21.3 - 25.5	50	100	48	94	SLATE	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 or paper separators.



MaineDOT Mattawamkeag Bridge #2522
US Route 2 over Mattawamkeag River
Mattawamkeag, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-MMR-103	R8	25.5 - 30.5	60	100	54	90	SCHIST, SLATE	1



- 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.



MaineDOT Mattawamkeag Bridge #2522
US Route 2 over Mattawamkeag River
Mattawamkeag, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-MMR-201	R1	8.4 - 10.1	20	100	6	30	SLATE	1
BB-MMR-201	R2	10.1 - 11.9	20	91	14	64	SLATE	1
BB-MMR-201	R3	11.9 - 16.7	47	81	27	47	SLATE	1-2
BB-MMR-201	R4	16.7 - 21.7	60	100	46	77	SLATE	3-4



- 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. Photo is wet.
 2. Transition between core runs within a row are marked by wood or paper separators.



MaineDOT Mattawamkeag Bridge #2522
US Route 2 over Mattawamkeag River
Mattawamkeag, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-MMR-202	R1	10.0 - 12.1	21	84	0	0	SLATE	1
BB-MMR-202	R2	12.1 - 16.7	55	100	43	78	SLATE	1-2
BB-MMR-202	R3	16.7 - 21.0	50	96	30	58	SLATE	2-3
BB-MMR-203	R1	7.0 - 8.7	20	100	-	-	BOULDER	3
BB-MMR-203	R2	10.0 - 11.5	18	100	0	0	SLATE	3
BB-MMR-203	R3	11.5 - 16.5	60	100	53	89	SLATE	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, middle photo is wet, bottom photo is end of R3 dry
 3. Transition between core runs within a row are marked by wood or paper separators.



MaineDOT Mattawamkeag Bridge #2522
US Route 2 over Mattawamkeag River
Mattawamkeag, ME
 Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-MMR-203	R4	16.5 - 18	18	100	18	100	SLATE	1
BB-MMR-203	R5	18 - 20	15	63	11	46	SLATE	2
BB-MMR-204	R1	7.3 - 9.1	19	86	-	-	BOULDER	2
BB-MMR-204	R2	10.9 - 11.5	6	86	0	0	SLATE	2
BB-MMR-204	R3	11.5 - 13.6	17	68	0	0	SLATE	2-3
BB-MMR-204	R4	13.6 - 16.9	35	88	26	65	SLATE	3
BB-MMR-204	R5	16.9 - 21.7	55	96	13	23	SLATE	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 or paper separators.



06/08/2020

GEOTECHNICAL DESIGN REPORT
MATTAWAMKEAG RIVER BRIDGE NO. 2522 - MATTAWAMKEAG
09.0025975.01

APPENDIX E – CALCULATIONS

Seismic Site Class Calculation



Seismic Site Class Calculation Summary

Project: Mattawamkeag Bridge over Mattawamkeag River **Project No.:** 09.0025975.01
Location: Mattawamkeag, ME
Evaluated By/Date: BMC **Date** 5/11/2020
Checked By/Date: ARB **Date** 5/11/2020

Objective:

Determine seismic site class by performing calculations in accordance with the MaineDOT Bridge Manual 2003 Edition with updates in 2014, which references the AASHTO LRFD Seismic Bridge Design Specifications, 8th Edition.

Subsurface Data: Borings BB-MMR-101 through -103 were drilled by New England Boring Contractors, of Hermon, Maine between May 22 and 24, 2018. Borings BB-MMR-201 through -204 were drilled by New England Boring Contractors, of Hermon, Maine between September 25 and October 1, 2019. Borings BB-MMR-101 and -103 are used for this calculation which is representative of soil conditions at the abutments.

Assumptions: Soil borings extended to depths between 30 and 60 feet below the roadway level and bedrock was encountered in the soil borings.

Approach: 1) Evaluate if the procedure in AASHTO LRFD Seismic Section 3.10.2.1 for classifying a site is appropriate for the site. Sites with highly variable subsurface conditions or very large sites may require multiple site class determinations or a site-specific seismic response analysis. Furthermore, classifying a site based on the 100 feet of soil and rock beneath the ground surface may be inappropriate if deep deposits of weak soils are present below 100 feet, or if foundation structures are supported on firm soil or rock below soft soils which can be justified as having little effect on the structure's seismic response.

2) Evaluate if soil properties are known in sufficient detail to determine site class. If data is not known in sufficient detail, AASHTO permits the use of Site Class D, unless conditions for Site Class E or Site Class F are likely to be present.

3) Check for the four categories of Site Class F requiring site-specific evaluation:
- Soils vulnerable to potential failure (liquefiable soils, sensitive clays, weakly cemented soils)
- Peats or highly organic clays greater than 10 feet in thickness
- Thick layers (greater than 25 feet) of highly plastic clay (PI > 75)
- Very thick soft/medium stiff clays (greater than 125 feet)

4) Check for existence of greater than 10 feet of soft clay (where $s_u < 500$ psf, $w > 40\%$, and $PI > 20$). If these conditions are met, classify as Site Class E.

5) Categorize the site using one of the following three methods in AASHTO C3.10.3.1-1:
- \bar{v}_s (Method A) - \bar{N} (Method B) - \bar{N}_{ch} and \bar{s}_u (Method C)

If shear wave velocity data are available, they should be used to classify the site. The N and s_u methods should only be used if shear wave velocity data is not available, as the correlation between site amplification and these geotechnical parameters is more uncertain (and therefore more conservative) than the correlation with \bar{v}_s .

Results: Calculations of the Seismic Site Class based on Method B as described in section 3.10.3.1 of the LRFD Seismic Bridge Design Specifications are attached. Calculations results are summarized in the table below.

Boring ID	BB-MMR-101	BB-MMR-103	Average
N-Value	57	17	37

Conclusions: Based on the procedure outlined in section 3.10.3.1 and table 3.10.3.1-1 of the LRFD Seismic Bridge Design Specifications, we recommend that Site Class "D" be used for design.

Mattawamkeag Bridge over Mattawamkeag River
 Mattawamkeag, ME

Calculated By: BMC Date: 5/11/2020
 Checked By: ARB Date: 5/11/2020

INPUT

Exploration ID: BB-MMR-101

Ground Surface Elevation: 208.9 ft

Depth of Boring: 44.4 ft

Depth to Bedrock: 34.4 ft

EQUATIONS

$$\bar{N}_{ch} = \frac{\sum_{i=1}^m d_i}{\sum_{i=1}^m \frac{d_i}{N_i}}$$

where: m = number of layers

d_i = the thickness of all layers between 0 and 100 feet.

d_c = the thickness of any clay layers between 0 and 100 feet.

N_i = the Standard Penetration Resistance (ASTM D 1586) of cohesionless soil layers not to exceed 100 blows/ft, corrected for hammer energy for calibrated auto hammers (i.e., N_{60}).

Note: d_i calculated assuming breaks between sub-layers occur at the midpoint between SPT sample intervals (unless noted otherwise)

CALCULATION

$$\bar{N} = 57.4$$

Soil Strata	SPT Interval Depth		SPT Elevation (mid-interval)	SPT N-value	d_i	d_i / N_i	Comment
	Top, ft	Bottom, ft					
Fill	1.5	3.3	206.5	43	4.7	0.11	
	6.0	6.1	202.9	50	3.4	0.07	
	10.0	11.3	198.3	50	5.1	0.10	
River sediments	15.0	17.0	192.9	55	5.4	0.10	
	20.0	22.0	187.9	56	5.0	0.09	
	25.0	27.0	182.9	82	5.0	0.06	
	30.0	31.7	178.1	77	6.5	0.08	
Top of Rock	35.0						

35.00

Mattawamkeag Bridge over Mattawamkeag River
 Mattawamkeag, ME

Calculated By: BMC Date: 5/11/2020
 Checked By: ARB Date: 5/11/2020

INPUT

Exploration ID: BB-MMR-103 Ground Surface Elevation: 209.2 ft Depth of Boring: 63.0 ft
 Depth to Bedrock: 53.0 ft

EQUATIONS

$$\bar{N}_{ch} = \frac{\sum_{i=1}^m d_i}{\sum_{i=1}^m \frac{d_i}{N_i}}$$

where: m = number of layers
 d_i = the thickness of all layers between 0 and 100 feet.
 d_c = the thickness of any clay layers between 0 and 100 feet.
 N_i = the Standard Penetration Resistance (ASTM D 1586) of cohesionless soil layers not to exceed 100 blows/ft, corrected for hammer energy for calibrated auto hammers (i.e., N₆₀).
 Note: d_i calculated assuming breaks between sub-layers occur at the midpoint between SPT sample intervals (unless noted otherwise)

CALCULATION

$$\bar{N} = 17.2$$

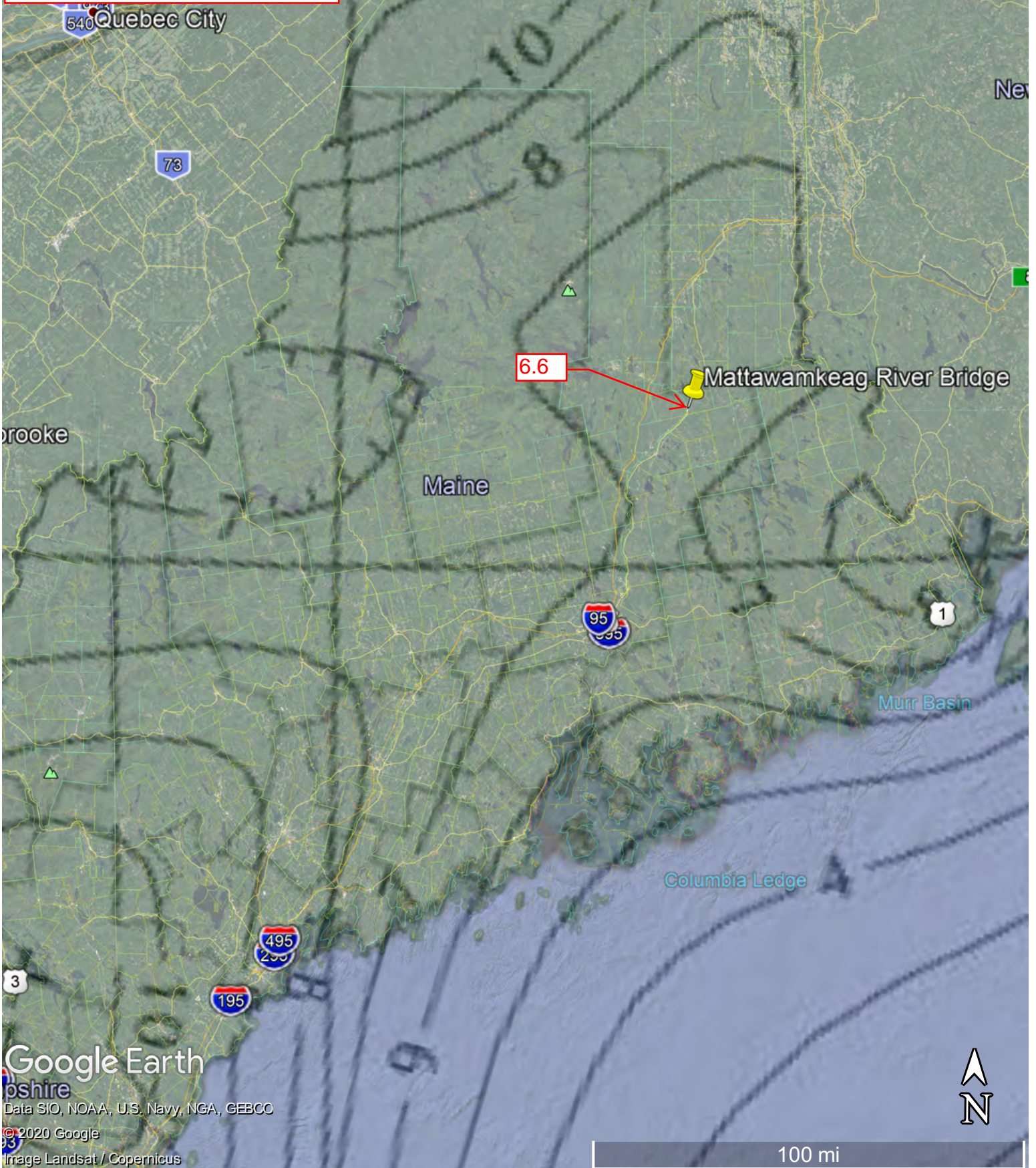
Soil Strata	SPT Interval Depth		SPT Elevation (mid-interval)	SPT N-value	d _i	d _i / N _i	Comment
	Top, ft	Bottom, ft					
Fill	0.8	2.1	207.8	100	3.6	0.04	
	5.0	7.0	203.2	7	5.0	0.71	
	10.0	12.0	198.2	35	5.0	0.14	
River sediments	15.0	17.0	193.2	3	5.0	1.67	
	20.0	22.0	188.2	113	5.0	0.04	
	25.0	27.0	183.2	72	5.0	0.07	
	30.0	32.0	178.2	44	5.0	0.11	
	35.0	37.0	173.2	44	5.0	0.11	
	40.0	42.0	168.2	82	5.0	0.06	
	45.0	45.8	163.8	100	4.9	0.05	
	50.9	51.3	158.1	100	4.0	0.04	
Top of Rock	53.0						

52.35

Seismic Design Parameters

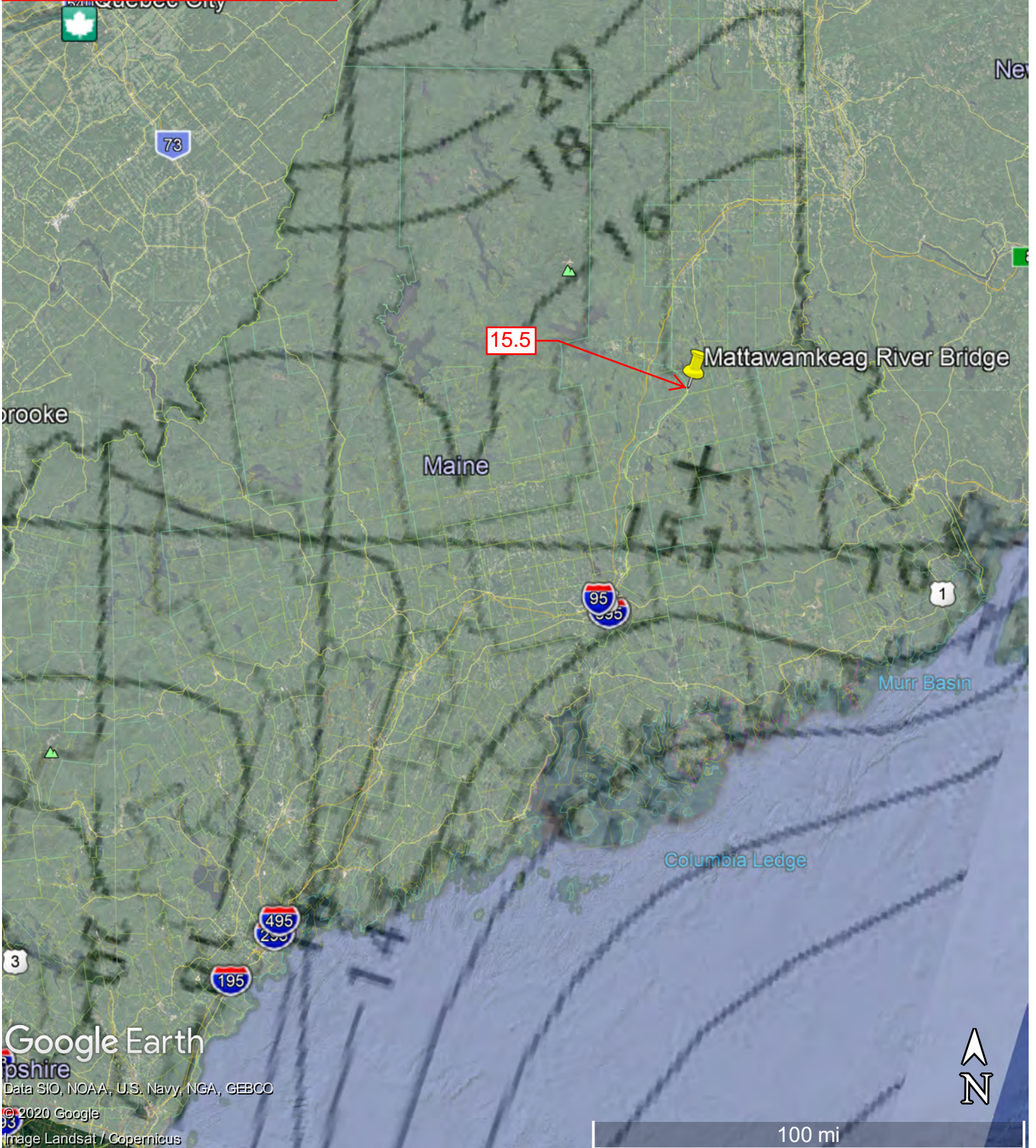


Horizontal Peak Ground
Acceleration Coefficient (PGA)



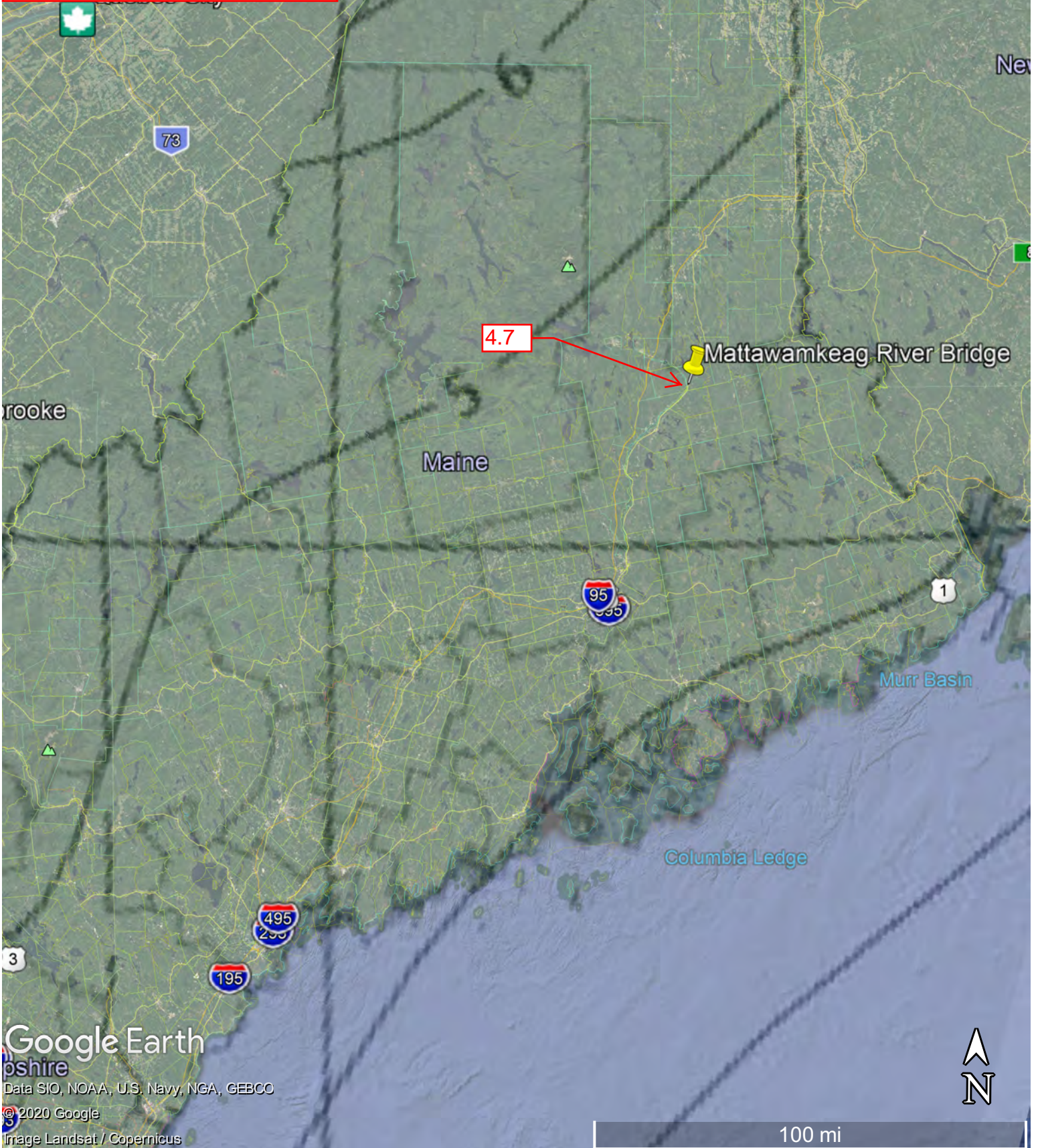


Horizontal Response Spectral
Acceleration Coefficient for period
of 0.2 s (S_s)





Horizontal Response Spectral
Acceleration Coefficient for period
of 1.0 s (S_1)



Google Earth

100 mi



Mattawamkeag Seismic Interpolation for Coefficients		
Seismic Parameter	Interpolated Value from Maps¹	Design Parameter
Horizontal Peak ground Acceleration Coefficient	6.6	$PGA = .066$
Horizontal Response Spectral Acceleration Coefficient for Period of 0.2s	15.5	$S_s = 0.155$
Horizontal Response Spectral Acceleration Coefficient for Period of 1.0s	4.7	$S_1 = .047$

Notes: 1. AASHTO Figures 3.10.2.1-1,-2, and -3 were overlaid within the Google Earth software. Coefficients were interpolated between lines on these figures as presented in pages 1 through 3 of this calculation.

For Class D, values of F_{PGA} and $F_a = 1.6$, and $F_v = 2.4$

Therefore:

$$A_s = F_{PGA} \times PGA = 1.6 \times 0.066 = 0.106 \text{ g}$$

$$S_{DS} = F_a \times S_s = 1.6 \times 0.155 = 0.248 \text{ g}$$

$$S_{D1} = F_v \times S_1 = 2.4 \times 0.047 = 0.113 \text{ g}$$

Summary:

SITE CLASS D SEISMIC DESIGN PARAMETERS	
Parameter	Design Value
Fpga	1.6
Fa	1.6
Fv	2.4
As (Period = 0.0 sec)	0.106 g
SDs (Period = 0.2 sec)	0.248 g
SD1 (Period = 1.0 sec)	0.113 g

Lateral Earth Pressures



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JOB: 09.0025975.01 Mattawamkeag
Bridge
 SUBJECT: Lateral Earth Pressures
 SHEET: 1 OF 1
 CALCULATED BY B. Cardali 6/3/20
 CHECKED BY A. Blaisdell 6/3/20

Subject: Evaluate lateral earth pressure coefficients for Abutments

- References:**
1. MaineDOT Bridge Design Guide, Chapter 3
 2. AASHTO LRFD Bridge Design Specifications, 8th Edition (2017)
 3. U.S. Army Corps of Engineers Engineer Manual 1110-2-2502, Retaining and Flood Walls

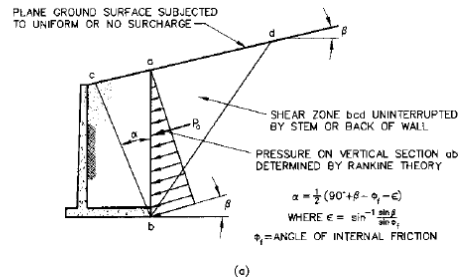
Input Parameters:

- $\phi := 32\text{deg}$ 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*)
- $\beta := 0\text{deg}$ Angle of backfill to the horizontal
- $\theta := 90\text{-deg}$ Angle of back face of wall to the horizontal

Article 3.6.4 of the BDG states that abutments with a height of 5 feet or more should be assumed to experience sufficient horizontal movement of the top of the wall to develop active conditions due to structural deformation of the stem and rotation of the foundation.

$$\alpha := \frac{(90\text{-deg} + \beta - \phi)}{2} = 29\text{ deg}$$

Based on Figure C3.11.5.3-1 of LRFD, the abutment is considered to be a short-heeled wall. See page two of this calculation for Abutment section. Therefore, Coulomb theory should be used to calculate active earth pressures.



Coulomb Active Earth Pressure Coefficient (Short-Heeled Wall)

$$\Gamma := \left[1 + \sqrt{\frac{\sin(\phi + \delta_f) \cdot \sin(\phi - \beta)}{\sin(\theta - \delta_f) \cdot \sin(\theta + \beta)}} \right]^2 = 2.77$$

$$K_{ac} := \frac{(\sin(\theta + \phi))^2}{\Gamma \cdot [(\sin(\theta))^2 \cdot \sin(\theta - \delta_f)]}$$

$K_{ac} = 0.28$

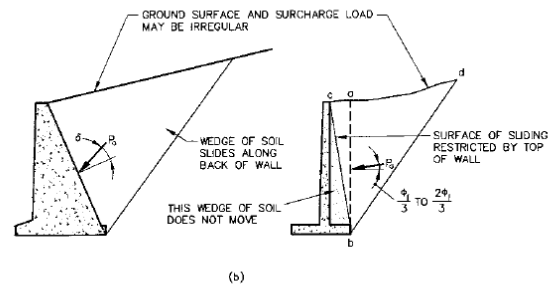


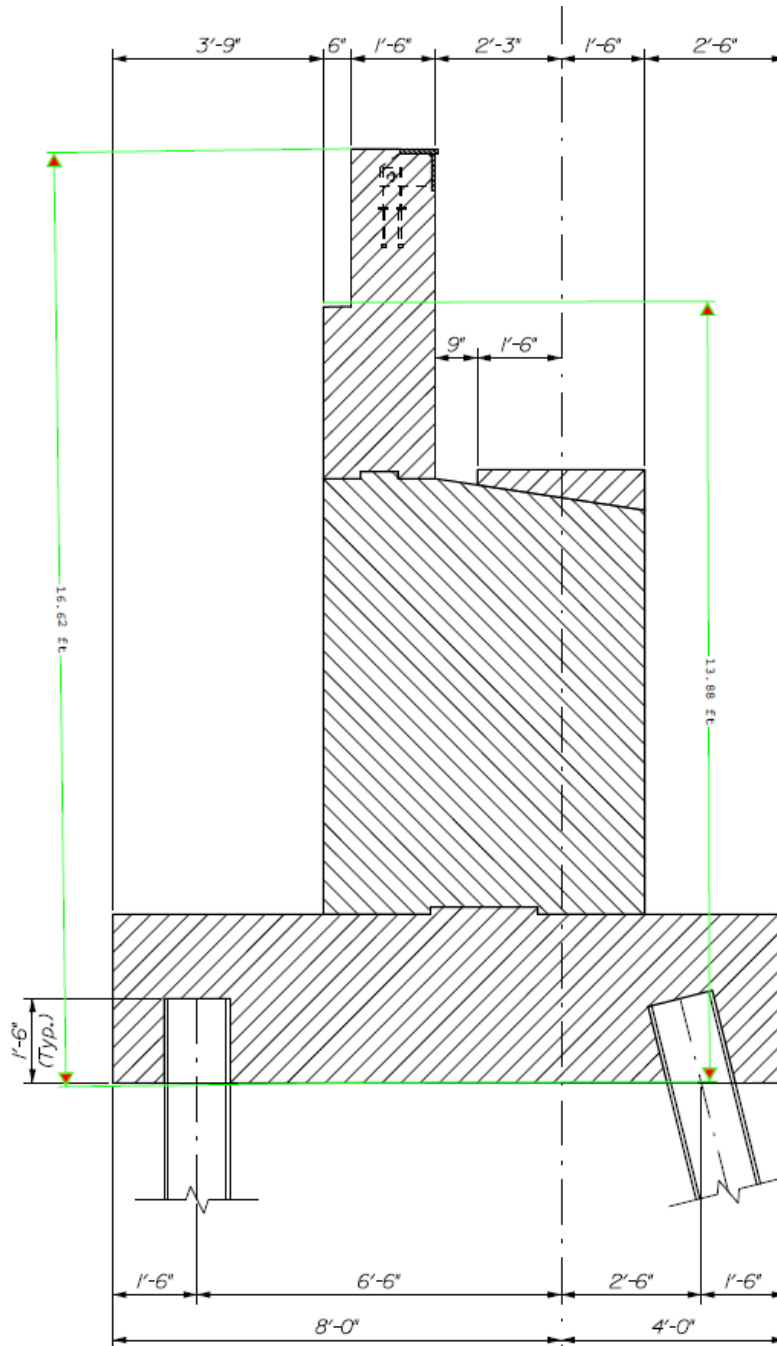
Figure C3.11.5.3-1—Application of (a) Rankine and (b) Coulomb Earth Pressure Theories in Retaining Wall Design



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Bridge
SUBJECT: Lateral Earth Pressures
SHEET: 2 OF 1
CALCULATED BY B. Cardali 6/3/20
CHECKED BY A. Blaisdell 6/3/20



Drivability



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JOB: 09.0025975.01 Mattawamkeag Bridge
 SUBJECT: Axial Pile Resistance
 SHEET: 1 OF 20
 CALCULATED BY M. Johnescu, 6/30/20
 REVIEWED BY A. Blaisdell, 7/9/20

Objective

Evaluate the axial geotechnical resistance of the abutment piles for the Mattawamkeag Bridge Replacement in Mattawamkeag, ME. Evaluations were conducted to assess a suitable driving system to install piles to the required geotechnical nominal resistance of 600 kips for abutment piles.

Methodology

Evaluate proposed pile section for governing factored axial compression resistance as follows.

1. Nominal Compressive Resistance
2. Factored Structural Compressive Resistance - Strength Limit State
3. Factored Structural Compressive Resistance - Extreme/Service Limit State
4. Geotechnical Resistance (Static Analysis)
5. Geotechnical Resistance (Drivability Analysis)
6. Factored Geotechnical Resistance - Strength Limit State
7. Factored Geotechnical Resistance - Extreme/Service Limit State

References

1. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 8th edition. (AASHTO LRFD)

Soil Properties

Consider Mattawamkeag Bridge Interpretive Subsurface Profile (see Figure 3), subsurface layering and properties relative to pile design are presented in the Apile outputs on pages 9-20.

Structural Properties

HP14x89, ASTM A572, Gr. 50

Yield Strength of Steel	$F_y := 50\text{ksi}$
Area of section	$A_s := 26.1\text{in}^2$
Young's Modulus of Steel	$E_s := 30000\cdot\text{ksi}$
Radius of gyration (weak axis)	$r_x := 5.88\text{in}$



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JOB: 09.0025975.01 Mattawamkeag Bridge
 SUBJECT: Axial Pile Resistance
 SHEET: 2 OF 20
 CALCULATED BY M. Johnescu, 6/30/20
 REVIEWED BY A. Blaisdell, 7/9/20

1. Nominal Structural Compressive Resistance P_n

Nominal Compressive Resistance: $P_n := 0.66^{\lambda} \cdot F_y \cdot A_s$ AASHTO Eq. 6.9.5.1-1

Determine normalized column slenderness factor λ

$$\lambda := \left(\frac{K \cdot l}{r_s \cdot \pi} \right)^2 \cdot \frac{F_y}{E} \quad \text{AASHTO Eq. 6.9.4.1-3} \quad \text{pg. 6-74}$$

$\lambda := 0$ Where the pile is fully embedded, AASHTO 10.7.3.13.1.

Giving: $P_n := 0.66^{\lambda} \cdot F_y \cdot A_s$ $P_n = 1305 \cdot \text{kip}$

2. Factored Structural Compressive Resistance - Strength Limit State:

Factor for piles in compression under hard driving conditions:

From Article 6.5.4.2 $\phi_c := 0.5$

Factored Compressive Resistance for Strength Limit State:

$$P_T := \phi_c \cdot P_n \quad \text{AASHTO Eq. 6.9.2.1-1} \quad \text{pg. 6-71}$$

$$P_T = 653 \cdot \text{kip}$$

Note: This is the maximum structural resistance assuming no unbraced length, actual structural resistance should be checked by the structural engineer.

3. Factored Structural Compressive Resistance - Service/Extreme Limit State:

Resistance Factors for Extreme Limit States:

From Article 10.5.5.1 and 10.5.5.3 $\phi := 1$

Factored Compressive Resistance for Service/Extreme Limit State:

$$P_{\max} := \phi \cdot P_n \quad \text{AASHTO Eq. 6.9.2.1-1} \quad \text{pg. 6-71}$$

$$P_T = 1305 \cdot \text{kip}$$

4. Geotechnical Axial Resistance - Static Analysis

AASHTO Article 10.7.3.2.3 states that the nominal resistance of piles driven to point bearing on hard rock is controlled by the structural limit state or potential for driving damage to occur during hard driving.

Required nominal resistance of 600 kips for design of the 14-pile configuration based on the maximum compression load on the battered piles, provided by the structural engineer and a resistance factor of 0.65.



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JOB: 09.0025975.01 Mattawamkeag Bridge
 SUBJECT: Axial Pile Resistance
 SHEET: 3 OF 20
 CALCULATED BY M. Johnescu, 6/30/20
 REVIEWED BY A. Blaisdell, 7/9/20

Based on geotechnical resistance calculations, the required nominal capacity is achieved with both a 17-foot and 35-foot pile for Abutment 1 and 2, respectively

The estimated % skin friction resistance is about 10% and 15% for Abutments 1 and 2, respectively, at the required nominal pile resistance, based on the estimated friction resistance.

5. Geotechnical Axial Resistance - Drivability Analysis

$$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y \quad \text{AASHTO Eq. 10.7.8.1}$$

$$f_y := 50 \text{ ksi} \quad \text{yield Strength of steel}$$

$$\phi_{da} := 1.0 \quad \text{AASHTO Table 10.5.5.2.3-1 Refers to Article 6.5.4.2, Pg. 6-28}$$

$$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y \quad \sigma_{dr} = 45 \cdot \text{ksi} \quad \text{Driving Stress in pile cannot exceed 45 ksi}$$

Abutment 1 - Drive pile through 17 feet of soil to rock with toe quake representative of hard driving conditions (0.04 in). Model pile length as 19 feet (2 foot stickup at end of drive).

Abutment 2 - Drive pile through 35 feet of soil to rock with toe quake representative of hard driving conditions (0.07 in). Model pile length as 37 feet (2 foot stickup at end of drive).

Drive piles with a Delmag D30 open-ended diesel hammer with a rated energy of 82625 ft-lb (fuel setting 3, 2 below maximum and fuel setting 2, 1 below maximum for Abutment 1 and 2, respectively). **The proposed hammer is sized to achieve the required nominal pile resistance for each of the potential driving scenarios; not the maximum driveability resistance for the pile section and profile.**

GRLWEAP Output is attached on Sheets 5 through 8.

Abutment 1: Required nominal geotechnical resistance, pile driving stress=43 ksi, final penetration resistance=8 bpi.

Abutment 2: Required nominal geotechnical resistance, pile driving stress=43 ksi, final penetration resistance=9 bpi.

6. Factored Drivability Resistance - Strength Limit State:

Strength Limit State Factored Drivability Resistance:

PDA, WEAP and CAPWAP used to establishing driving criteria $\phi_{dyn} := 0.65$ AASHTO Table 10.5.5.2.3-1

$$\text{Abutment 1: } R_{ndr1_factored} := R_{ndr1} \cdot \phi_{dyn}$$

$$R_{ndr1_factored} = 390 \cdot \text{kip}$$



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JOB: 09.0025975.01 Mattawamkeag Bridge
 SUBJECT: Axial Pile Resistance
 SHEET: 4 OF 20
 CALCULATED BY M. Johnescu, 6/30/20
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Abutment 2: $R_{\text{ndr2_factored}} := R_{\text{ndr2}} \cdot \phi_{\text{dyn}}$

$$R_{\text{ndr2_factored}} = 390 \cdot \text{kip}$$

7. Factored Drivability Resistance - Service/Extreme Limit States:

Service and Extreme Limit State Factored Drivability Resistance:

Resistance Factors for Extreme Limit States: $\phi_{\text{serv_ext}} := 1$

From Article 10.5.5.1 and 10.5.5.3

$$R_{\text{ndr1_serv_ext}} := R_{\text{ndr1}} \cdot \phi_{\text{serv_ext}}$$

$$R_{\text{ndr1_serv_ext}} = 600 \cdot \text{kip}$$

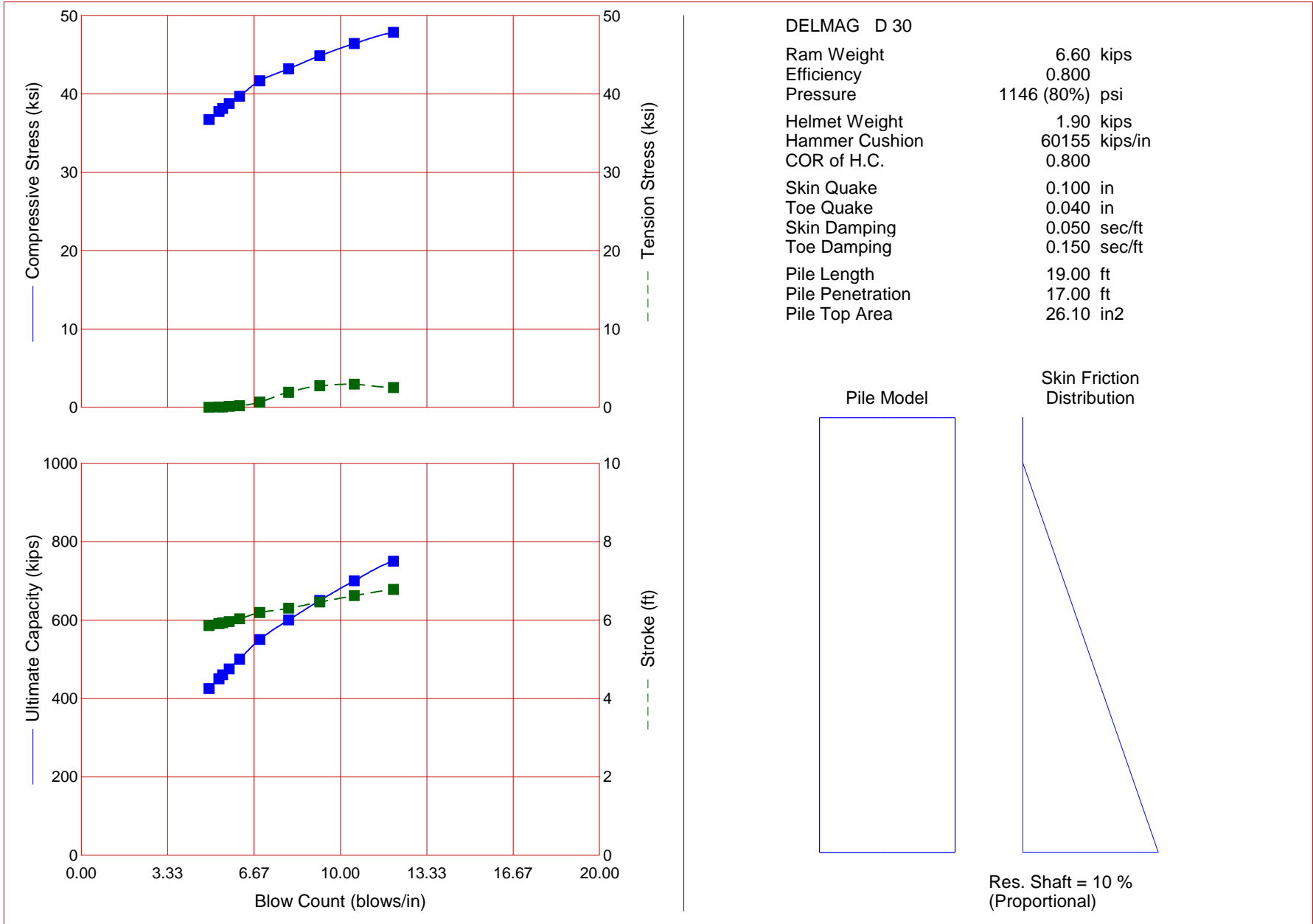
Summary of Results - Axial Loading:

ASTM A572, HP 14x89	Structural Resistance (kips)	Geotechnical Static Resistance (kips)	Required Geotechnical Resistance (kips)	Governing Resistance (kips)
Abutment 1 & 2: Strength Limit State, Design	653	n/a	390	390
Abutment 1 & 2: Service/Extreme Limit State, Design	1305	n/a	600	600

Since the driving stresses do not exceed the limiting driving stress of 45 ksi for ASTM A572 steel (50 ksi yield stress), and the calculation penetration resistance is for piles are within the preferred range of 6 to 15 blows per inch, the analyzed hammer system is judged to be acceptable to initial piles to the required nominal geotechnical resistance of 600 kips.

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Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
425.0	36.71	0.01	4.9	5.86	16.27
450.0	37.74	0.05	5.3	5.91	16.17
460.0	38.14	0.04	5.5	5.93	16.21
475.0	38.76	0.14	5.7	5.96	16.17
500.0	39.70	0.20	6.1	6.03	16.19
550.0	41.66	0.66	6.9	6.19	16.45
600.0	43.20	1.93	8.0	6.30	16.39
650.0	44.87	2.77	9.2	6.46	16.64
700.0	46.42	2.96	10.5	6.62	16.97
750.0	47.85	2.52	12.0	6.78	17.30

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Abutment 1, 17' Pile

=====

APILE for Windows, Version 2018.8.5

Serial Number : 653550831

A Program for Analyzing the Axial Capacity
and Short-term Settlement of Driven Piles
under Axial Loading.

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=====

This program is licensed to :

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Path to file locations : P:\09 Jobs\0025900s\09.0025975.00 - HNTB -
Mattawamkeag Bridge\09.0025975.01 - Final Design\Work\Calcs\Apile\AB1 (17' pile)\
Name of input data file : AB1-Vert.ap8d
Name of output file : AB1-Vert.ap8o
Name of plot output file : AB1-Vert.ap8p

Time and Date of Analysis

Date: February 13, 2020 Time: 09:32:03

1

* INPUT INFORMATION *

Mattawamkeag Abutment 1

DESIGNER : M. Johnescu

JOB NUMBER : 09.0025975.01

METHOD FOR UNIT LOAD TRANSFERS :

- USACE (U.S. Army Corps of Engineers)
Unfactored Unit Side Friction and Unit Side Resistance are used.

COMPUTATION METHOD(S) FOR PILE CAPACITY :

- USACE (U.S. Army Corps of Engineers)
Critical Depth Method for Sand:
10 to 20 Pile Diameter based on the Density
Use Long Pile Option

TYPE OF LOADING :

- COMPRESSION

PILE TYPE :

H-Pile/Steel Pile

DATA FOR AXIAL STIFFNESS :

- MODULUS OF ELASTICITY = 0.290E+08 PSI
- CROSS SECTION AREA = 26.10 IN2

NONCIRCULAR PILE PROPERTIES :

- TOTAL PILE LENGTH, TL = 18.50 FT.
- BATTER ANGLE = 0.00 DEG
- PILE STICKUP LENGTH, PSL = 1.00 FT.
- ZERO FRICTION LENGTH, ZFL = 1.00 FT.
- PERIMETER OF PILE = 57.00 IN.
- TIP AREA OF PILE = 26.10 IN2
- INCREMENT OF PILE LENGTH
USED IN COMPUTATION = 1.00 FT.

SOIL INFORMATIONS :

DEPTH FT.	SOIL TYPE	LATERAL EARTH PRESSURE	EFFECTIVE UNIT WEIGHT LB/CF	FRICTION ANGLE DEGREES	BEARING CAPACITY FACTOR
0.00	SAND	1.25	67.60	30.00	0.00

30.00 SAND 1.25 67.60 30.00 0.00

MAXIMUM UNIT FRICTION	MAXIMUM UNIT BEARING	UNDISTURB SHEAR STRENGTH	REMOLDED SHEAR STRENGTH	BLOW COUNT	UNIT SKIN FRICTION	UNIT END BEARING
KSF	KSF	KSF	KSF		KSF	KSF
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00

* MAXIMUM UNIT FRICTION AND/OR MAXIMUM UNIT BEARING WERE SET TO BE 0.10E+08 BECAUSE THE USER DOES NOT PLAN TO LIMIT THE COMPUTED DATA.

DEPTH FT.	LRFD FACTOR ON UNIT FRICTION	LRFD FACTOR ON UNIT BEARING
	0.00	1.000
30.00	1.000	1.000

1

 * COMPUTATION RESULT *

 * ARMY CORPS METHOD *

PILE PENETRATION FT.	TOTAL SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	0.2	0.2
1.00	0.1	0.3	0.4
2.00	0.4	0.5	0.9
3.00	0.9	0.8	1.7
4.00	1.6	1.1	2.7
5.00	2.5	1.4	3.9
6.00	3.6	1.7	5.3
7.00	4.8	2.0	6.8
8.00	6.3	2.3	8.6
9.00	8.0	2.5	10.6

10.00	9.9	2.8	12.7
11.00	12.0	3.1	15.1
12.00	14.3	3.4	17.6
13.00	16.7	3.7	20.4
14.00	19.4	3.9	23.3
15.00	22.3	4.2	26.5
16.00	25.2	4.3	29.6
17.00	28.2	4.4	32.6

NOTES:

- AN ASTERISK IS PLACED IN THE END-BEARING COLUMN IF THE TIP RESISTANCE IS CONTROLLED BY THE FRICTION OF SOIL PLUG INSIDE AN OPEN-ENDED PIPE PILE.

 * COMPUTE LOAD-DISTRIBUTION AND LOAD-SETTLEMENT *
 * CURVES FOR AXIAL LOADING *

T-Z CURVE NO.	NO. OF POINTS	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
1	10	0.0000E+00	0.0000E+00	0.0000E+00
			0.0000E+00	0.1000E-01
			0.0000E+00	0.2000E-01
			0.0000E+00	0.4000E-01
			0.0000E+00	0.6000E-01
			0.0000E+00	0.8000E-01
			0.0000E+00	0.1200E+00
			0.0000E+00	0.1600E+00
			0.0000E+00	0.5000E+00
			0.0000E+00	0.1000E+02
2	10	0.1503E+02	0.0000E+00	0.0000E+00
			0.5286E+00	0.1000E-01
			0.9424E+00	0.2000E-01
			0.1549E+01	0.4000E-01
			0.1971E+01	0.6000E-01
			0.2283E+01	0.8000E-01
			0.2712E+01	0.1200E+00
			0.2992E+01	0.1600E+00
			0.3794E+01	0.5000E+00
			0.4310E+01	0.1000E+02
3	10	0.2996E+02	0.0000E+00	0.0000E+00

0.5291E+00	0.1000E-01
0.9440E+00	0.2000E-01
0.1553E+01	0.4000E-01
0.1978E+01	0.6000E-01
0.2292E+01	0.8000E-01
0.2725E+01	0.1200E+00
0.3008E+01	0.1600E+00
0.3820E+01	0.5000E+00
0.4344E+01	0.1000E+02

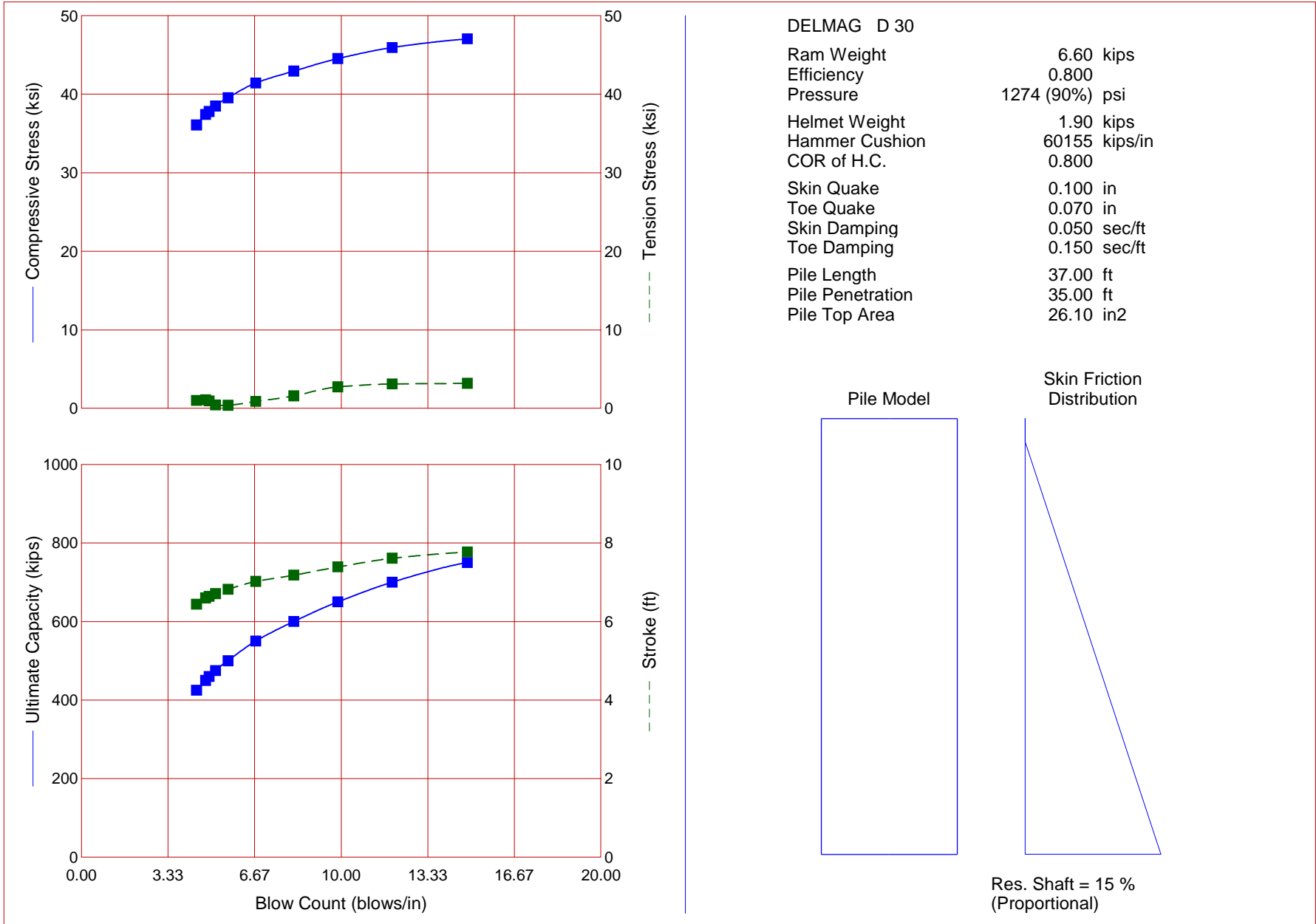
TIP LOAD KIP	TIP MOVEMENT IN.
0.0000E+00	0.0000E+00
0.4436E-01	0.1000E-03
0.3137E+00	0.5000E-02
0.4436E+00	0.1000E-01
0.9919E+00	0.5000E-01
0.1403E+01	0.1000E+00
0.1984E+01	0.2000E+00
0.3137E+01	0.5000E+00
0.4436E+01	0.1000E+01
0.6273E+01	0.2000E+01

LOAD VERSUS SETTLEMENT CURVE

TOP LOAD KIP	TOP MOVEMENT IN.	TIP LOAD KIP	TIP MOVEMENT IN.
0.8609E-01	0.1200E-03	0.4436E-01	0.1000E-03
0.4954E+00	0.1096E-02	0.9382E-01	0.1000E-02
0.2315E+01	0.5432E-02	0.3137E+00	0.5000E-02
0.4410E+01	0.1081E-01	0.4436E+00	0.1000E-01
0.1338E+02	0.5253E-01	0.9919E+00	0.5000E-01
0.1854E+02	0.1036E+00	0.1403E+01	0.1000E+00
0.2888E+02	0.5060E+00	0.3137E+01	0.5000E+00
0.3037E+02	0.1006E+01	0.4436E+01	0.1000E+01
0.3260E+02	0.2007E+01	0.6273E+01	0.2000E+01

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Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
425.0	36.07	1.01	4.4	6.44	20.62
450.0	37.41	1.09	4.8	6.60	21.03
460.0	37.78	0.94	4.9	6.64	21.18
475.0	38.48	0.44	5.2	6.71	21.36
500.0	39.53	0.39	5.6	6.82	21.53
550.0	41.42	0.88	6.7	7.02	22.03
600.0	42.93	1.60	8.2	7.18	22.61
650.0	44.52	2.74	9.9	7.39	23.39
700.0	45.94	3.11	12.0	7.61	24.20
750.0	47.03	3.19	14.9	7.77	24.84

09.0025975.01 - HNTB - Mattawamkeag Bridge
Abutment 2, 35' Pile

=====

APILE for Windows, Version 2018.8.5

Serial Number : 653550831

A Program for Analyzing the Axial Capacity
and Short-term Settlement of Driven Piles
under Axial Loading.
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=====

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Path to file locations : P:\09 Jobs\0025900s\09.0025975.00 - HNTB -
Mattawamkeag Bridge\09.0025975.01 - Final Design\Work\Calcs\Apile\AB2 (35' pile)\
Name of input data file : AB2 Vert.ap8d
Name of output file : AB2 Vert.ap8o
Name of plot output file : AB2 Vert.ap8p

Time and Date of Analysis

Date: February 13, 2020 Time: 09:44:15

1

* INPUT INFORMATION *

Mattawamkeag Abutment 2

DESIGNER : M. Johnescu

JOB NUMBER : 09.0025975.01

METHOD FOR UNIT LOAD TRANSFERS :

- USACE (U.S. Army Corps of Engineers)
Unfactored Unit Side Friction and Unit Side Resistance are used.

COMPUTATION METHOD(S) FOR PILE CAPACITY :

- USACE (U.S. Army Corps of Engineers)
Critical Depth Method for Sand:
10 to 20 Pile Diameter based on the Density

TYPE OF LOADING :

- COMPRESSION

PILE TYPE :

H-Pile/Steel Pile

DATA FOR AXIAL STIFFNESS :

- MODULUS OF ELASTICITY = 0.290E+08 PSI
- CROSS SECTION AREA = 26.10 IN2

NONCIRCULAR PILE PROPERTIES :

- TOTAL PILE LENGTH, TL = 37.50 FT.
- BATTER ANGLE = 0.00 DEG
- PILE STICKUP LENGTH, PSL = 1.00 FT.
- ZERO FRICTION LENGTH, ZFL = 5.00 FT.
- PERIMETER OF PILE = 57.00 IN.
- TIP AREA OF PILE = 26.10 IN2
- INCREMENT OF PILE LENGTH
USED IN COMPUTATION = 1.00 FT.

SOIL INFORMATIONS :

DEPTH FT.	SOIL TYPE	LATERAL EARTH PRESSURE	EFFECTIVE UNIT WEIGHT LB/CF	FRICTION ANGLE DEGREES	BEARING CAPACITY FACTOR
0.00	CLAY	0.00	55.60	0.00	0.00
3.50	CLAY	0.00	55.60	0.00	0.00

3.50	SAND	1.25	67.60	30.00	0.00
40.00	SAND	1.25	67.60	30.00	0.00

MAXIMUM UNIT FRICTION KSF	MAXIMUM UNIT BEARING KSF	UNDISTURB SHEAR STRENGTH KSF	REMOLDED SHEAR STRENGTH KSF	BLOW COUNT	UNIT SKIN FRICTION KSF	UNIT END BEARING KSF
0.10E+08*	0.10E+08*	0.40	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.40	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00

* MAXIMUM UNIT FRICTION AND/OR MAXIMUM UNIT BEARING WERE SET TO BE 0.10E+08 BECAUSE THE USER DOES NOT PLAN TO LIMIT THE COMPUTED DATA.

DEPTH FT.	LRFD FACTOR ON UNIT FRICTION	LRFD FACTOR ON UNIT BEARING
0.00	1.000	1.000
3.50	1.000	1.000
3.50	1.000	1.000
40.00	1.000	1.000

1

 * COMPUTATION RESULT *

 * ARMY CORPS METHOD *

PILE PENETRATION FT.	TOTAL SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	0.3	0.3
1.00	0.0	0.3	0.3
2.00	0.0	0.5	0.5
3.00	0.0	0.8	0.8
4.00	0.0	0.9	0.9

5.00	0.4	1.2	1.6
6.00	1.4	1.5	2.8
7.00	2.5	1.8	4.3
8.00	3.9	2.1	5.9
9.00	5.4	2.3	7.7
10.00	7.1	2.6	9.8
11.00	9.1	2.9	12.0
12.00	11.2	3.2	14.4
13.00	13.6	3.5	17.0
14.00	16.1	3.7	19.8
15.00	18.8	4.0	22.8
16.00	21.7	4.1	25.8
17.00	24.5	4.2	28.7
18.00	27.4	4.2	31.5
19.00	30.2	4.1	34.3
20.00	33.1	4.1	37.1
21.00	35.9	4.1	40.0
22.00	38.8	4.1	42.8
23.00	41.6	4.1	45.7
24.00	44.5	4.1	48.5
25.00	47.3	4.1	51.4
26.00	50.2	4.1	54.2
27.00	53.0	4.1	57.1
28.00	55.9	4.1	59.9
29.00	58.7	4.1	62.8
30.00	61.6	4.1	65.6
31.00	64.4	4.1	68.5
32.00	67.3	4.1	71.3
33.00	70.1	4.1	74.2
34.00	73.0	4.1	77.1
35.00	75.8	4.1	79.9
36.00	78.7	4.1	82.8

NOTES:

- AN ASTERISK IS PLACED IN THE END-BEARING COLUMN IF THE TIP RESISTANCE IS CONTROLLED BY THE FRICTION OF SOIL PLUG INSIDE AN OPEN-ENDED PIPE PILE.

 * COMPUTE LOAD-DISTRIBUTION AND LOAD-SETTLEMENT *
 * CURVES FOR AXIAL LOADING *

T-Z CURVE NO.	NO. OF POINTS	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
---------------	---------------	--------------------	-------------------	-------------------

1	10	0.0000E+00	0.0000E+00	0.0000E+00
			0.0000E+00	0.1000E-01
			0.0000E+00	0.2000E-01
			0.0000E+00	0.4000E-01
			0.0000E+00	0.6000E-01
			0.0000E+00	0.8000E-01
			0.0000E+00	0.1200E+00
			0.0000E+00	0.1600E+00
			0.0000E+00	0.5000E+00
			0.0000E+00	0.1000E+02
2	10	0.1775E+01	0.0000E+00	0.0000E+00
			0.0000E+00	0.1000E-01
			0.0000E+00	0.2000E-01
			0.0000E+00	0.4000E-01
			0.0000E+00	0.6000E-01
			0.0000E+00	0.8000E-01
			0.0000E+00	0.1200E+00
			0.0000E+00	0.1600E+00
			0.0000E+00	0.5000E+00
			0.0000E+00	0.1000E+02
3	10	0.3458E+01	0.0000E+00	0.0000E+00
			0.0000E+00	0.1000E-01
			0.0000E+00	0.2000E-01
			0.0000E+00	0.4000E-01
			0.0000E+00	0.6000E-01
			0.0000E+00	0.8000E-01
			0.0000E+00	0.1200E+00
			0.0000E+00	0.1600E+00
			0.0000E+00	0.5000E+00
			0.0000E+00	0.1000E+02
4	10	0.3500E+01	0.0000E+00	0.0000E+00
			0.0000E+00	0.1000E-01
			0.0000E+00	0.2000E-01
			0.0000E+00	0.4000E-01
			0.0000E+00	0.6000E-01
			0.0000E+00	0.8000E-01
			0.0000E+00	0.1200E+00
			0.0000E+00	0.1600E+00
			0.0000E+00	0.5000E+00
			0.0000E+00	0.1000E+02
5	10	0.2178E+02	0.0000E+00	0.0000E+00
			0.5259E+00	0.1000E-01
			0.9341E+00	0.2000E-01
			0.1526E+01	0.4000E-01
			0.1935E+01	0.6000E-01

			0.2235E+01	0.8000E-01
			0.2644E+01	0.1200E+00
			0.2910E+01	0.1600E+00
			0.3662E+01	0.5000E+00
			0.4141E+01	0.1000E+02
6	10	0.3996E+02		
			0.0000E+00	0.0000E+00
			0.5259E+00	0.1000E-01
			0.9341E+00	0.2000E-01
			0.1526E+01	0.4000E-01
			0.1935E+01	0.6000E-01
			0.2235E+01	0.8000E-01
			0.2644E+01	0.1200E+00
			0.2910E+01	0.1600E+00
			0.3662E+01	0.5000E+00
			0.4141E+01	0.1000E+02

TIP LOAD KIP	TIP MOVEMENT IN.
0.0000E+00	0.0000E+00
0.4228E-01	0.1000E-03
0.2990E+00	0.5000E-02
0.4228E+00	0.1000E-01
0.9454E+00	0.5000E-01
0.1337E+01	0.1000E+00
0.1891E+01	0.2000E+00
0.2990E+01	0.5000E+00
0.4228E+01	0.1000E+01
0.5979E+01	0.2000E+01

LOAD VERSUS SETTLEMENT CURVE

TOP LOAD KIP	TOP MOVEMENT IN.	TIP LOAD KIP	TIP MOVEMENT IN.
0.1396E+00	0.1568E-03	0.4228E-01	0.1000E-03
0.9928E+00	0.1352E-02	0.8943E-01	0.1000E-02
0.4785E+01	0.6666E-02	0.2990E+00	0.5000E-02
0.9201E+01	0.1319E-01	0.4228E+00	0.1000E-01
0.3059E+02	0.6084E-01	0.9454E+00	0.5000E-01
0.4386E+02	0.1159E+00	0.1337E+01	0.1000E+00
0.7069E+02	0.5265E+00	0.2990E+01	0.5000E+00
0.7254E+02	0.1027E+01	0.4228E+01	0.1000E+01

0.7552E+02

0.2029E+01

0.5979E+01

0.2000E+01

Bearing Resistance Calculation



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Engineers and
 Scientists

JOB: 09.0025975.01 Mattawamkeag Bridge
 SUBJECT: Pier Footing Bearing Resistance on Rock
 SHEET: 1 OF 8
 CALCULATED BY: B. Cardali 5/11/20
 REVIEWED BY: A. Blaisdell 5/11/20

Objective

Evaluate nominal and factored bearing resistance of a spread footing foundation on rock for the proposed piers using Unconfined Compressive Strength (UCS) and Rock Quality Designation (RQD) data.

Methodology

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

1. Use Bedrock Properties From Test Borings BB-MMR-101 through BB-MMR-103 and BB-MMR-201 through BB-MMR-204.
2. Calculate Rock Mass Rating (RMR)
3. Determine Rock Property Constants s and m
4. Calculate Nominal and Factored Bearing Resistance of Bedrock (q_n and q_f)

References

1. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 6th Edition 2012. (AASHTO LRFD)
2. Wyllie, Duncan C., "Foundations on Rock", Second edition, 1992.

Evaluation

1. Rock Mass Properties

A. Unconfined Compressive Strength

Unconfined compressive strength (UCS) laboratory test results.

Boring	Run	LAB						Rock Type
		Depth of Sample (ft)	Depth of Sample into Rock (ft)	Elev Top of Sample (ft)	UCS (psi)	Modulus (ksi)	Unit Wt (pcf)	
BB-MMR-101	R1	34.4	0	174.5	23,157	4,020	169.2	SLATE
BB-MMR-102	R3	13.2	3	171.5	25,336	6,230	164.9	SLATE
BB-MMR-103	R5	61.8	8.8	147.4	13,092	2,870	170.2	SCHIST
BB-MMR-201	R3	12.5	4.2	171.9	9,861	3,680	167.5	SLATE
BB-MMR-204	R4	15	3.8	169.2	1,976	2,700	166.2	SLATE

Results at BB-MMR-204 appear to be an outlier and not representative of the overall rock mass. Consider a representative strength value for design as the average of the lower bound values at the pier locations identified in the table above.

$$\sigma_u := \frac{(1.98 \cdot \text{ksi} + 9.86 \cdot \text{ksi})}{2} = 5.92 \cdot \text{ksi}$$



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 SHEET: 2 OF 8
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B. Rock Quality Designation

Select representative RQD based on data collected in the river (BB-MMR-102, -201, -202, -203, and -204). Highlighted cells indicate rock that is likely removed from the cofferdams, not considered representative of supporting rock mass.

Boring	Run	Depth of Core Run below GS (ft)		Depth Below Top of Rock (ft)		Rec (%)	RQD %	Joint Spacing (in)	Joint Aperture (in)	Elev. (ft)		Rock Type
		Top	Bottom	Top	Bottom					Top	Bottom	
BB-MMR-102	R1	8.9	- 11.4	-1.3	- 1.2	57%	0%	N/A	N/A-N/A	175.8	173.3	SLATE
BB-MMR-102	R2	11.4	- 12.2	1.2	- 2.0	100%	0%	<0.75	0.1-0.4	173.3	172.5	SLATE
BB-MMR-102	R3	12.2	- 15.8	2.0	- 5.6	100%	42%	0.75-8	0.004-0.1	172.5	168.9	SLATE
BB-MMR-102	R4	15.8	- 17.0	5.6	- 6.8	100%	0%	<0.75-0.75	0.004-0.1	168.9	167.7	SLATE
BB-MMR-102	R5	17.0	- 19.8	6.8	- 9.6	100%	71%	8-24	0.02-0.1	167.7	164.9	SLATE
BB-MMR-102	R6	19.8	- 21.3	9.6	- 11.1	100%	33%	<0.75-8	0.02-0.1	164.9	163.4	SLATE
BB-MMR-102	R7	21.3	- 25.5	11.1	- 15.3	100%	94%	<0.75-24	0.02-0.1	163.4	159.2	SLATE
BB-MMR-102	R8	25.5	- 30.5	15.3	- 20.3	100%	90%	24	0.1-0.4	159.2	154.2	SLATE / SCHIST
BB-MMR-201	R1	8.4	- 10.1	0.1	- 1.8	100%	30%	0.75-80	0.02-0.1	176.0	174.3	SLATE
BB-MMR-201	R2	10.1	- 11.9	1.8	- 3.6	91%	64%	0.75-24	0.02-0.1	174.3	172.5	SLATE
BB-MMR-201	R3	11.9	- 16.7	3.6	- 8.4	81%	47%	0.75-24	0.02-0.1	172.5	167.7	SLATE
BB-MMR-201	R4	16.7	- 21.7	8.4	- 13.4	100%	77%	0.75-24	0.01-0.02	167.7	162.7	SLATE
BB-MMR-202	R1	10.0	- 12.1	0.2	- 2.3	84%	0%	<0.75-8	0.1-0.4	174.9	172.8	SLATE
BB-MMR-202	R2	12.1	- 16.7	2.3	- 6.9	78%	78%	0.75-80	0.02-0.4	172.8	168.2	SLATE
BB-MMR-202	R3	16.7	- 21.0	6.9	- 11.2	96%	58%	0.75-24	0.02-0.1	168.2	163.9	SLATE
BB-MMR-203	R2	10.0	- 11.5	0.0	- 1.5	100%	0%	<0.75-0.75	0.02-0.1	172.8	171.3	SLATE
BB-MMR-203	R3	11.5	- 16.5	1.5	- 6.5	100%	89%	8-24	0.01-0.1	171.3	166.3	SLATE
BB-MMR-203	R4	16.5	- 18.0	6.5	- 8.0	100%	100%	#N/A	#N/A	166.3	164.8	SLATE
BB-MMR-203	R5	18.0	- 20.0	8.0	- 10.0	63%	46%	#N/A	#N/A	164.8	162.8	SLATE
BB-MMR-204	R2	10.9	- 11.5	-0.3	- 0.3	86%	0%	#N/A	#N/A	173.3	172.7	SLATE
BB-MMR-204	R3	11.5	- 13.6	0.3	- 2.4	68%	0%	<0.75-8	0.01-0.02	172.7	170.6	SLATE
BB-MMR-204	R4	13.6	- 16.9	2.4	- 5.7	95%	65%	8-24	0.01-0.1	170.6	167.3	SLATE
BB-MMR-204	R5	16.9	- 21.7	5.7	- 10.5	96%	23%	<0.75-24	0.01-0.1	167.3	162.5	SLATE

Average RQD for all Pier borings 64%

2. Calculation of Rock Mass Rating (RMR)

From AASHTO LRFD Tables 10.4.6.4-1 and 10.4.6.4-2, determine the RMR (see sheets 10 and 11 for reference tables)

Parameter 1- Uniaxial Compressive Strength

$$\sigma_{u,r} := 5.92 \text{ksi}$$

$$\sigma_{u,r} = 852 \cdot \text{ksf}$$

Selected in Section 1, see Sheet 1.

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating

$$RR_1 := 4$$

Parameter 2- Drill Core Quality

Average RQD=64% from table in Section 1. Use RQD 50 to 75% as design basis.

From AASHTO LRFD Table 10.4.6.4-1

$$\text{Relative Rating } RR_2 := 13 \text{ for RQD} = 50 \text{ to } 75\%$$



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 SUBJECT: Pier Footing Bearing Resistance on Rock
 SHEET: 3 OF 8
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 REVIEWED BY: A. Blaisdell 5/11/20

Parameter 3- Spacing of Joints

From boring logs, generally extremely close to moderately spaced = < 0.75 in to 2 feet. On average spacing generally 2 to 12 inches.

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating

$$RR_3 := 10$$

Parameter 4- Condition of Joints

From boring logs, aperture 0.02-0.1 inches and hard joint walls.

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.

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 high angle to low angle and generally rough and tight to open. Considering rock will remain embedded below foundation bearing level, joint orientation is considered favorable.

From AASHTO LRFD Table 10.4.6.4-2

Relative Rating

$$RR_6 := -2$$

Total RMR Rating

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

$$RMR = 49$$

From AASHTO LRFD Table 10.4.6.4-3 RMR= 49 is indicative of fair Rock Quality

3. Determine Rock Property Constants s and m

From AASHTO LRFD Table 10.4.6.4-4 for fair Quality Rock Mass

Categorized as rock type B, Slate, using s and m values interpolated from the logarithmic trend of plotted values from AASHTO Table 10.4.6.4-4 (plot specific to this project on sheet 5, plot showing trend lines through all RMR data on sheet 8).

$$m := .263$$

$$s := 0.00021$$

for RMR=49



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

From Wyllie "Foundations on Rock"

Eq. 5.4 Pg.138

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

Where

$$C_{f1} := 1.0$$

From Wyllie Table 5.4 Pg. 138 Correction factor for foundation shape for rectangular foundation:

For $L/B > 6$, use factor $C_{f1} = 1.0$,

For $L/B = 1$, use factor $C_{f1} = 1.12$

Assume long strip, lowest C_{f1} .

$$\sigma_u = 5.92 \cdot \text{ksi}$$

Nominal Bearing Resistance

$$q_n := C_{f1} \cdot \sqrt{s} \cdot \sigma_u \cdot \left[1 + \sqrt{m \cdot \left(\frac{1}{s} \right) + 1} \right] = 66 \cdot \text{ksf}$$

$$q_n = 66 \cdot \text{ksf}$$

Factored Bearing Resistance

Bearing Resistance Factor is specified in Table 10.5.5.2.2-1

$$\phi_b := 0.45 \quad \text{Footing on rock (Strength Limit State)}$$

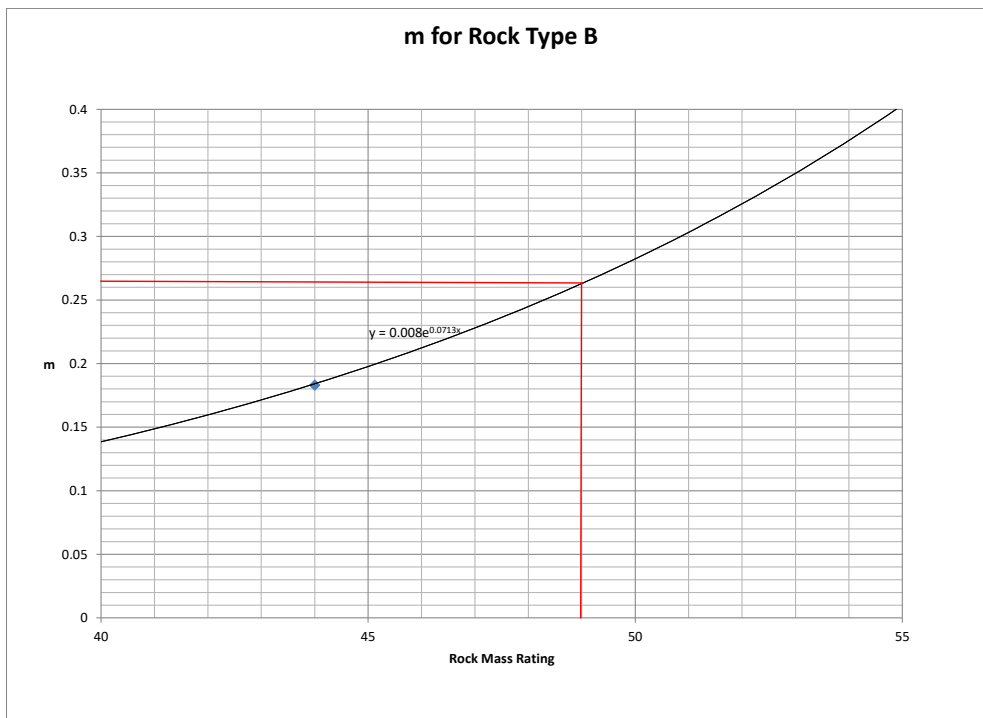
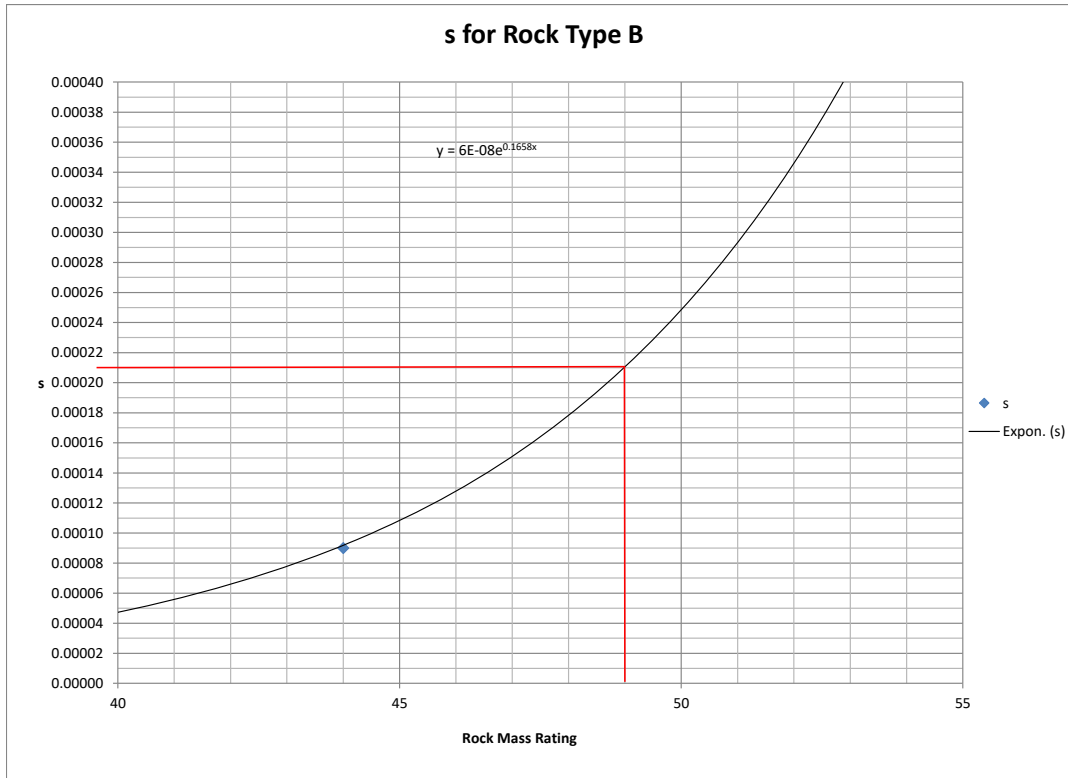
$$q_R := \phi_b \cdot q_n \quad q_R = 30 \cdot \text{ksf}$$



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SUBJECT: Pier Footing Bearing Resistance on Rock
SHEET: 5 OF 8
CALCULATED BY: B. Cardali 5/11/20
REVIEWED BY: A. Blaisdell 5/11/20





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JOB: 09.0025975.01 Mattawamkeag Bridge
 SUBJECT: Pier Footing Bearing Resistance on Rock
 SHEET: 6 OF 8
 CALCULATED BY: B. Cardali 5/11/20
 REVIEWED BY: A. Blaisdell 5/11/20

➔ Reference: I:\Mathcad\units.xmcd

10-22

AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

Table 10.4.6.4-1 Geomechanics Classification of Rock Masses.

Parameter		Ranges of Values							
1	Strength of intact rock material	Point load strength index	>175 ksf	85–175 ksf	45–85 ksf	20–45 ksf	For this low range, uniaxial compressive test is preferred		
		Uniaxial compressive strength	>4320 ksf	2160–4320 ksf	1080–2160 ksf	520–1080 ksf	215–520 ksf	70–215 ksf	20–70 ksf
		Relative Rating	15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>10 ft.	3–10 ft.	1–3 ft.	2 in.–1 ft.	<2 in.		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		<ul style="list-style-type: none"> • Very rough surfaces • Not continuous • No separation • Hard joint wall rock 	<ul style="list-style-type: none"> • Slightly rough surfaces • Separation <0.05 in. • Hard joint wall rock 	<ul style="list-style-type: none"> • Slightly rough surfaces • Separation <0.05 in. • Soft joint wall rock 	<ul style="list-style-type: none"> • Slicken-sided surfaces or • Gouge <0.2 in. thick or • Joints open 0.05–0.2 in. • Continuous joints 	<ul style="list-style-type: none"> • Soft gouge >0.2 in. thick or • Joints open >0.2 in. • Continuous joints 		
	Relative Rating		25	20	12	6	0		
	5	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				



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JOB: 09.0025975.01 Mattawamkeag Bridge
 SUBJECT: Pier Footing Bearing Resistance on Rock
 SHEET: 7 OF 8
 CALCULATED BY: B. Cardali 5/11/20
 REVIEWED BY: A. Blaisdell 5/11/20

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 10.5.5.2.2-1—Resistance Factors for Geotechnical Resistance of Shallow Foundations at the Strength Limit State

		Method/Soil/Condition	Resistance Factor
Bearing Resistance	ϕ_b	Theoretical method (Munfakh et al., 2001), in clay	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using <i>CPT</i>	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using <i>SPT</i>	0.45
		Semi-empirical methods (Meyerhof, 1957), all soils	0.45
		Footings on rock	0.45
		Plate Load Test	0.55
Sliding	ϕ_c	Precast concrete placed on sand	0.90
		Cast-in-Place Concrete on sand	0.80
		Cast-in-Place or precast Concrete on Clay	0.85
		Soil on soil	0.90
	ϕ_{ep}	Passive earth pressure component of sliding resistance	0.50



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AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

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

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