GZA
Proactive by Design

Geotechnical
Environmental
Ecological
Water
Construction Management

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MEMORANDUM

TO: Mr. Tom Kendrick, P.E., Theresa D. McAuliffe, P.E.
McFarland-Johnson, Inc.

FROM: Blaine Cardali, Christopher Snow, P.E.,
Andrew Blaisdell, P.E.

DATE: May 23, 2018

FILE NO.: 09.0025887.01

SUBJECT: Geotechnical Design Basis Memorandum for Bedrock Dowels at Pier 2
Rehabilitation of Barter's Island Bridge No. 2039 over the Back River
Maine Department of Transportation WIN 22607.00
Boothbay, Maine

STATE OF MAINE
CHRISTOPHER L. SNOW
No. 7275
5-23-2018
LICENSED
PROFESSIONAL ENGINEER

GZA GeoEnvironmental, Inc. (GZA) has prepared this geotechnical design basis memorandum for bedrock dowels for the proposed Bridge No. 2039 over the Back River in Boothbay, Maine. Our work was completed in accordance with the Subconsultant Agreement between McFarland-Johnson, Inc. and GZA GeoEnvironmental, Inc. (GZA) dated December 19, 2016, which incorporates GZA's proposal No. 09.P000005.17, dated March 21, 2016, and Contract Amendment No. 1, dated May 4, 2017. This memorandum is subject to the *Limitations* included in **Appendix A**.

REQUESTED ADDITIONAL SCOPE

In addition to the Geotechnical Design Report (GDR) that was submitted on August 15, 2017, per your request, the following additional scope was completed to evaluate foundation stability and constructability at the new center pier (Pier 2) of the proposed rehabilitation of Barter's Island Bridge:

- Evaluate the existing subsurface information to assess the possible presence of sloping rock at the proposed new center pier.
- Develop design recommendations for dowels to provide supplemental shear resistance for the foundation against sliding at the tremie seal / bedrock interface.
- Evaluate constructability of the proposed tremie seal considering sloping rock and variable seal thickness.
- Summarize results in this design basis memorandum to include: typical length, embedment, and grid spacing for shear dowels; constructability considerations for bedrock preparation, and dowel installation as appropriate.

SLOPING BEDROCK

The replacement bridge will be supported on two existing abutments, two existing approach piers, and a new center pier. GZA evaluated the potential presence of sloping bedrock at the proposed center pier. If the exposed bedrock surface is steeper than 4 horizontal to 1 vertical (4H:1V), then anchoring, doweling, benching or other means should be employed to improve sliding resistance as required by the Maine Department of Transportation Bridge Design Guide (BDG). The proposed tremie seal dimensions from McFarland-Johnson, Inc. are 26.0 feet by 31.5 feet.

GZA evaluated the inclination of the bearing surface at the bedrock and seal interface in both the longitudinal and transverse directions using the top of rock elevations from borings BB-BTBR-102, -202, -203, and -204. The boring logs, boring location plan, and interpretive subsurface profile can be found in the GDR for reference. The longitudinal direction is defined as the length of the seal parallel to the proposed roadway alignment and the transverse is the direction perpendicular to the proposed alignment. From the boring data, the average slope across the seal was interpolated and tabulated below:

Sliding Resistance Direction	Average Bedrock Surface Slope Inclination
Longitudinal	21° (approx. 2.6H:1V)
Transverse	14° (approx. 4H:1V)

Since the average bedrock slopes are greater than or equal to 4H:1V, rock dowels are recommended. GZA's evaluation of the rock dowel requirements for Pier 2 is described in the sections that follow.

LOAD AND RESISTANCE FACTORS

From the 2017 AASHTO LRFD Bridge Design Specifications, 8th Edition (AASHTO) Table 3.4.1-2, the load factors that were applied to the vertical and horizontal components of the of the normal force due to the dead load at the bedrock interface were 0.9 and 1.25, respectively.

For proposed foundations bearing on bedrock, we recommend that sliding resistance be assessed using a nominal sliding resistance coefficient ($C \tan \phi_f$) equal to 0.55 for cast-in-place concrete on Silty Sand and Gravel, which assumes that the bedrock surface is not perfectly clean, a resistance factor (ϕ_τ) of 0.8, and a factored sliding resistance coefficient of 0.44 for the Strength Limit State.

LOADING ON FOUNDATION

The loads at the bottom of the tremie seal were provided by McFarland-Johnson, Inc. and are attached to the sliding resistance calculation in **Appendix B**. The applied dead load at bottom of the seal is 3,030 kips. The applied lateral loads on the footing in the longitudinal and the transverse directions are 72.8 and 78.6 kips, respectively. When combined with the transverse component due to sloping rock, the total lateral load is 1422 kips.

SLIDING ANALYSIS

The sliding analysis utilized the bedrock elevations from the borings BB-BTBR-102, 202, -203, and -204 and average inclinations that were previously discussed. Nominal and factored sliding resistances were calculated for bedrock-bearing footings using the methodology from AASHTO 10.6.3.4 and the American Concrete institute (ACI) shear friction design with the supplied loads.

Our analyses indicated that the total factored load exceeded the total factored resistance. Therefore, rock dowels are required to resist sliding. GZA assessed the dowel requirements using No. 9 reinforcing bar, consistent with the typical shear dowel detail presented in the BDG. Based on the shear friction design method, from the ACI 2011 Building Code Requirements for Structural Concrete, we estimate that 12 dowels will be required to provide supplemental shear resistance of the foundation against sliding. Refer to the attached Rock Dowel Calculation in **Appendix B** for additional details.

RECOMMENDATIONS

Due to the bedrock slope, dowels are recommended at the center pier to achieve the required sliding resistance. The proposed pier may be supported on a doweled spread footing including the following components to provide a geotechnical factored lateral resistance of 1422 kips and to meet the lateral loading requirements:

<u>TYPICAL BEDROCK DOWELS FOR CENTER PIER</u>

- A minimum of 12 bedrock dowels should be installed through the top of the tremie seal and into bedrock prior to constructing the footing.
- Dowels should be equally spaced within the footprint of the proposed footing and should be offset a minimum of 1 foot inside the limits of the footing.
- Dowels should consist of hot-dipped galvanized or epoxy-coated or stainless steel No. 9 deformed bar conforming to ASTM A615, with minimum yield strength of 75 ksi.
- The bars should extend at least 3 feet into the bedrock.
- The bars should extend to top of the tremie seal, or into the footing in accordance with requirements of the structural engineer.
- Dowels should be installed in a minimum 3-inch diameter drill hole.
- Grout should consist of a water-cement mixture with a maximum water-cement ratio of 0.45 and a minimum 28-day compressive strength of 5,000 psi.

CONSTRUCTION CONSIDERATIONS

We anticipate that bedrock bearing surfaces will be prepared in the wet, with water depths up to 30 feet or more during high tide, and that the bedrock surface will be variable in terms of elevation, slope and localized weathering. A combination of standard excavation equipment, hydraulic hoe-ramming equipment, and/or air lifting may be needed to clean the rock surface. All soil and loose, decomposed, highly weathered and fractured bedrock should be removed from the footing bearing surface prior to placement of the tremie seal. The prepared bearing surface 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 cross-sections showing the prepared rock surface geometry prior to placement of concrete. We recommend that the Geotechnical Engineer be on site to observe bedrock preparation, bedrock surface inspection, and dowel installation to ensure that the intent of the design and special provisions are met.

CLOSURE

We trust that this information meets current project needs. Please feel free to call Christopher Snow at (207) 358-5118 or Blaine Cardali at (207) 358-5131 for additional information.

BMC/CLS/ARB:erc
P:\09 Jobs\0025800s\09.0025887.00 - Barters Island Bridge\09.0025887.01 Final Design\Report\Rock dowel Memo\FINAL 25887 01 Barters Island Rock Dowel Memo 052318.docx

Attachments: Appendix A – Limitations
Appendix B – Rock Dowel Calculation

APPENDIX A – LIMITATIONS

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 agreement, 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.

Subsurface Conditions

4. 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.

5. 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.

6. Water level readings have been made in test holes (as described in the 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.

7. 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.

Compliance with Codes and Regulations

8. 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.

APPENDIX B – ROCK DOWEL CALCULATION

Longitudinal Sliding and Dowel Evaluation

GZA
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Portland, Maine 04101
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Fax 207-879-0099
http://www.gza.com

Engineers and Scientists

Objective

Evaluate rock dowels to provide shear resistance for tremie seal and footing construction over sloping bedrock. Perform evaluation using LRFD approach and shear-friction design method.

References

1. American Concrete Institute (ACI) (2011), Building Code Requirements for Structural Concrete(ACI 318-11) and Commentary (ACI 318-11)
2. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 8th edition, 2017. (AASHTO LRFD).

Analysis

Definitions, inputs and variables

Width of seal	$b_s := 26.0ft$	
Length of seal	$l_s := 31.5 \cdot ft$	
Concrete placment type factor	$C := 1.0$	Assumes cast-in-place concrete on soil or rock
Interface Friction Angle	$\phi_f = 29\,deg$	Assumes base is not perfectly clean (concrete on silty sand or gravel)
Sliding resistance factor of tremie concrete on rock	$\phi_\tau := 0.8$	
Yield strength of dowels	$f_{y.d} := 75ksi$	
Area of shear dowel	$A_{vf9} := 1.0in^2$	#9 bar
coefficient of friction at interface	$\mu := 0.6$	ACI 11.7.4.3
resistance factor for shear of steel	$\phi_{st} := 0.6$	

Longitudinal Sliding Evaluation

Rock Slope

$EL_{max} := -16.4ft$

$EL_{min} := -23.3 \cdot ft$

$d := 18.1 \cdot ft$

$$\alpha := atan\left[\frac{(EL_{max} - EL_{min})}{d}\right]$$

$\alpha = 20.9 \cdot deg$ Typical inclination of rock surface nearest to horizontal (measured from rock contour sections at pier)

Normal force

$DC := 0.9 \cdot 3030 \cdot kip = 2727 \cdot kip$

Factored Dead load at bottom of seal using a load factor of 0.9 from AASHTO Table 3.4.1-2. Dead load from structural engineer 3030 kips.

$V_n := DC \cdot \cos(\alpha) = 2548.1 \cdot kip$

Factored vertical component of the dead load

Sliding Resistance

$R_\tau := C \cdot \tan(\phi_f) \cdot V_n = 1401.4 \cdot kip$

Nominal resistance of sliding at rock interface

$R_N := R_\tau \cdot \phi_\tau = 1121.1 \cdot kip$

Factored resistance of sliding at rock interface

Factored Lateral Loads

$Q_f := 72.8 \cdot kip$

Factored Lateral loads provided by the structural engineer.

$DC_f := 3030 \cdot kip \cdot 1.25 = 3787.5 \cdot kip$

Factored Dead load at bottom of seal using a load factor of 1.25 from AASHTO Table 3.4.1-2

$Q_{D.\alpha} := DC_f \cdot \sin(\alpha) = 1349.1 \cdot kip$

Factored Lateral component of the dead load

$Q := (Q_{D.\alpha} + Q_f) = 1421.9 \cdot kip$

Total driving force at bottom of seal.

Check adequacy sliding without dowels

$\text{if}(R_N \geq Q, \text{"Okay"}, \text{"Not Okay"}) = \text{"Not Okay"}$

Since Q larger use difference to find required area of steel

$R_{d.req} := Q - R_N = 300.8 \cdot kip$

Required Shear resistance from dowels

$$A_{vf.req} := \frac{R_{d.req}}{\phi_{st} \cdot f_{y.d} \cdot \mu} = 11.1 \cdot in^2$$

ACI eq. 11-25 required area of steel

$$n_{bars} := \frac{A_{vf.req}}{A_{vf9}} = 11.1$$

Say 12 bars from structural engineer sketch

Check number of dowels

$n_{bar.design} := 12$

$A_{vf.design} := n_{bar.design} \cdot A_{vf9} = 12 \cdot in^2$

$R_{d.design} := \phi_{st} \cdot A_{vf.design} \cdot f_{y.d} \cdot \mu = 324 \cdot kip$

$R_R := R_N + R_{d.design} = 1445.1 \cdot kip$

$if(R_R \geq Q, \text{"Okay"}, \text{"Not Okay"}) = \text{"Okay"}$

Transverse Sliding and Dowel Evaluation

Objective

Evaluate rock dowels to provide shear resistance for tremie seal and footing construction over sloping bedrock. Perform evaluation using LRFD approach and shear-friction design method.

References

1. American Concrete Institute (ACI) (2011), Building Code Requirements for Structural Concrete(ACI 318-11) and Commentary (ACI 318-11)
2. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 8th edition, 2017. (AASHTO LRFD).

Analysis

Definitions, inputs and variables

Width of seal $b_s := 26.0ft$

Length of seal $l_s := 31.5 \cdot ft$

Concrete placement type factor $C := 1.0$ Assumes cast-in-place concrete on soil or rock

Interface friction angle $\phi_f = 29\,deg$ Assumes base is not perfectly clean (concrete on silty sand and gravel)

Sliding resistance factor of tremie concrete on rock $\phi_\tau := 0.8$

Yield strength of dowels $f_{y.d} := 75ksi$

Area of shear dowel $A_{vf9} := 1.0in^2$ #9 bar

coefficient of friction at interface $\mu := 0.6$ ACI 11.7.4.3

resistance factor for shear of steel $\phi_{st} := 0.6$

Transverse Sliding Evaluation

Rock Slope

$EL_{max} := -19ft$

$EL_{min} := -23 \cdot ft$

$d := 16 \cdot ft$ distance between max and min elevation points in transverse direction

$$\alpha := atan\left[\frac{(EL_{max} - EL_{min})}{d}\right]$$

$\alpha = 14 \cdot deg$ Typical inclination of rock surface nearest to horizontal (measured from rock contour sections at pier)

Normal force

$DC := 0.9 \cdot 3030 \cdot kip = 2727 \cdot kip$

Factored Dead load at bottom of seal using a load factor of 0.9 from AASHTO Table 3.4.1-2. Dead load from structural engineer 3030 kips.

$V_n := DC \cdot \cos(\alpha) = 2645.6 \cdot kip$

Factored vertical (normal) component of the dead load

Sliding Resistance

$R_\tau := C \cdot \tan(\phi_f) \cdot V_n = 1455 \cdot kip$

Interface sliding resistance

$R_N := R_\tau \cdot \phi_\tau = 1164 \cdot kip$

Nominal resistance of sliding at rock interface

Factored Lateral Loads

$Q_f := 78.6 \cdot kip$

Factored Lateral loads provided by the structural engineer.

$DC_f := 3030 \cdot kip \cdot 1.25 = 3787.5 \cdot kip$

Factored Dead load at bottom of seal using a load factor of 1.25 from AASHTO Table 3.4.1-2

$Q_{D.\alpha} := DC_f \cdot \sin(\alpha) = 918.6 \cdot kip$

Factored Lateral component of the dead load

$Q := (Q_{D.\alpha} + Q_f) = 997.2 \cdot kip$

Total driving force at bottom of seal.

Check adequacy sliding without dowels

$\text{if}(R_N \geq Q, \text{"Okay"}, \text{"Not Okay"}) = \text{"Okay"}$

Sliding resistance is adequate in the transverse direction

Load Data From Client

From:	Theresa D. McAuliffe
To:	Blaine Cardali
Cc:	Tom T. Kendrick; Christopher Snow
Subject:	Boothbay - Dead Loads at Pier 2
Date:	Tuesday, April 17, 2018 9:03:54 AM
Attachments:	image001.png

Hi Blaine,

The total Dead Load is 3030 kips at Pier 2.

This is the breakdown:

Superstructure:

- Steel = 200 kips
- Counterweight = 71 kips
- Deck = 241 kips

Miscellaneous attachments:

- Access Platforms = 4 kips
- Fender System = 22 kips

Substructure:

- Pier 2 Dead Load = 980 kips
- Seal Dead Load = 1512 kips

Seal dimensions are 26ft x 31.5ft x 12.3ft thick (thickness does not account for sloping bedrock).

Please let me know if you need more information.

Thanks,

Theresa McAuliffe, P.E.
Transportation Manager • Structures
5 Depot Street, Suite 25 • Freeport, ME 04032
(207) 869-5419 x181 • tmcauliffe@mjinc.com
McFarland Johnson
www.mjinc.com

From:	Theresa D. McAuliffe
To:	Christopher Snow
Cc:	Blaine Cardali
Subject:	RE: Boothbay - Sloping Bedrock
Date:	Tuesday, April 24, 2018 10:24:09 AM
Attachments:	image001.png

Yes, 2 pm it is.

Here are the Pier 2 unfactored loads and where they are applied. The loads have been broken down into the bridge longitudinal and transverse components.

Reference Elevations:
Q1.1 = 4.23 ft (This is also the Mean High Water)
Q50 = 9.5 ft
Bottom of Footing = -10.75 ft
Top of Footing = -8.5 ft
Top of Column = 5.81 ft
Top of Cap = 7.81 ft

Z-Axis: Along Bridge (roadway stationing in this direction)
Braking Force = 21.6 kips @ Top of Cap
Wind on Superstructure = 33.243 kips (max) @ Top of Cap
Wind on Substructure = 7.15 kips (max) @ EL -1.345
Wind on LiveLoad = 4.201 kips (max) @ Top of Cap
~~Q1.1 Ice = 282.04 kips @ EL 4.23~~
~~Q50 Ice = 145.403 kips @ Top of Cap~~
Q1.1 Water = 6.564 kip @ EL -2.13 (resultant of the pressure)
~~Q50 Water = 9.281 kip @ EL 0.502 (resultant of the pressure)~~
Total = 72.8 Kips

X-Axis: Perpendicular to Bridge (flow of the river)
Wind on Superstructure = 49.839 kips (max) @ Top of Cap
Wind of Substructure = 7.52 kips (max) @ EL -1.345
Wind on LiveLoad = 11.056 kips (max) @ Top of Cap
~~Q1.1 Ice = 940.145 kips @ EL 4.23~~
~~Q50 Ice = 484.678 kips @ Top of Cap~~
Q1.1 Water = 10.211 kip @ EL -2.13 (resultant of the pressure)
~~Q50 Water = 14.438 kip @ EL 0.502 (resultant of the pressure)~~
~~Vessel = 357.647 kips @ EL 4.23~~
Total = 78.6 Kips

Things to note:
-Ice and vessel loads are not applied to service cases.
-The wind loads given here are the maximum for the Strength III case. They vary depending on angle of attack and on which load case we use.
-The wind on live, wind on superstructure, and braking forces cause overturning moments at the pier cap. Those values are not given here.

Proactive By Design.
Our Company Commitment

GEOTECHNICAL DESIGN REPORT BARTER'S ISLAND BRIDGE NO. 2039 OVER THE BACK RIVER

MAINE DOT WIN 22607.00
BOOTHBAY, MAINE

Prepared for:
McFarland Johnson Inc.
Freeport, Maine

August 2017
09.0025887.01

Prepared by:
GZA GeoEnvironmental, Inc.
477 Congress Street | Suite 700 | Portland, Maine 04101
207.879.9190

Proactive by Design

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VIA EMAIL

August 15, 2017
File No. 09.0025887.01

Mr. Tom Kendrick, P.E.
McFarland-Johnson, Inc.
5 Depot Street, Suite 25
Freeport, ME 04032

Re: Geotechnical Design Report
Rehabilitation of Barter's Island Bridge No. 2039 over the Back River
MaineDOT WIN 22607.00
Boothbay, Maine

Dear Tom:

We are pleased to provide this Geotechnical Design Report (GDR) to McFarland Johnson, Inc. for Bridge No. 2039 over the Back River in Boothbay, Maine. Our work was completed in accordance with the Subconsultant Agreement between McFarland Johnson, Inc. and GZA GeoEnvironmental, Inc. (GZA) dated December 19, 2016, which incorporates GZA's proposal No. 09.P000005.17, dated March 21, 2016, Contract Amendment No.1, dated May 4, 2017, and the *Limitations* included in **Appendix A** of this report. GZA is providing geotechnical engineering services as a Subconsultant to McFarland Johnson, who is under contract with Maine Department of Transportation for design of the proposed bridge.

It has been a pleasure serving McFarland Johnson, Inc. 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, or if we can provide further assistance, please do not hesitate to contact the undersigned.

Very truly yours,

GZA GEOENVIRONMENTAL, INC.

Blaine Cardali, E.I.T.
Project Engineer

Russell J. Morgan
Consultant Reviewer

STATE OF MAINE
CHRISTOPHER L. SNOW
No. 7276
LICENSED
PROFESSIONAL ENGINEER
8/15/2017

Christopher L. Snow, P.E.
Associate Principal

BMC/CLS/ARB:erc
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FIGURES

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APPENDICES

1.0 INTRODUCTION

This report presents the results of GZA GeoEnvironmental, Inc.'s (GZA's) geotechnical evaluation for the proposed rehabilitation of Maine Department of Transportation (MaineDOT) Barter's Island Bridge No. 2039 over the Back River in Boothbay, Maine. Our work was completed in accordance with the Subconsultant Agreement between McFarland Johnson, Inc. (MJ) and GZA dated December 19, 2016, which incorporates GZA's proposal No. 09.P000005.17, dated March 21, 2016, Contract Amendment No.1, dated May 4, 2017, and the *Limitations* included in **Appendix A** of this report.

GZA is providing geotechnical engineering services as a Subconsultant to MJ, who is under contract with the State of Maine, MaineDOT for design of the proposed bridge.

1.1 BACKGROUND

The Barter's Island Bridge No. 2039 carries West Barter's Island Road over the Back River, in Boothbay, Maine, as shown on the *Locus Plan*, **Figure 1**. The existing bridge is 275 feet long, including two fixed approach spans with lengths of approximately 81 and 85 feet and a 110-foot-long, manually operated, pony truss swing span.

The bridge was originally constructed in 1931 with five piers, including two spans at each approach and the central swing span. The approach span piers consisted of timber bents supported directly on bedrock, or on unreinforced concrete placed directly on bedrock. The approach spans were both demolished in 1981 and replaced with single-span approaches. The current bridge configuration consists of two concrete abutments bearing on bedrock, two approach piers that support the swing span and the adjacent approach span, and the original circa-1931 central pier. Plans indicate that the circa-1931 approach piers were partially demolished in 1981 and that new piers were constructed around the original unreinforced concrete at those locations.

The 1981 bridge plans describe the central pier as a dry laid granite stone structure. The plans indicate that the pier substructure was intrusion grouted. Bridge inspection reports from 2006 and 2010 indicated an undermining of the northwest corner and that several stones at the central pier appeared "kicked out" or were missing.

1.2 PROPOSED CONSTRUCTION

The April 28, 2017 rehabilitation plans show reuse of the superstructure of both approach spans, and replacement of the movable span superstructure. Existing substructures and foundations will be reused at both abutments and the two approach span piers (Piers 1 and 3). Pier caps will be modified at Piers 1 and 3. The central pier, Pier 2, will be replaced with a precast concrete pier bearing on a cast-in-place spread footing with a tremie seal.

During construction, traffic will be detoured to a temporary bridge to be constructed south of the existing bridge.

1.3 OBJECTIVES AND SCOPE OF SERVICES

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

- Conducted site visits to observe surficial conditions, traffic and boring access;
- Coordinated and observed a subsurface exploration program, consisting of eight test borings to evaluate subsurface conditions;
- Conducted a laboratory testing program to evaluate engineering properties of the site soils and bedrock;
- Conducted geotechnical engineering analyses for refinement of subsurface profiles and engineering properties; evaluation of bedrock properties relative to stability and foundation support; bearing resistance of the new Pier 2 footing bearing on rock; and assessment of technically feasible alternatives for temporary bridge foundations;
- Developed geotechnical engineering considerations; and
- Prepared this report summarizing our findings and design considerations.

2.0 SUBSURFACE EXPLORATIONS

2.1 PREVIOUS MAINE DOT EXPLORATIONS

Subsurface data from three test borings (CT-15-78, CT-16-78, and CT-17-78) were shown in the 1982 bridge rehabilitation plan set. Two of the borings were drilled at Pier 2 and one was drilled near the center of the east approach span. The plans indicate subsurface conditions in the river generally consisted of silty sand and gravel, overlying a silt and clay deposit at one boring location, overlying biotite gneiss and pegmatite bedrock.

Subsurface data presented in the 1982 bridge plans are included in **Appendix B**.

2.2 PREVIOUS GZA SUBSURFACE EXPLORATIONS

GZA completed a preliminary subsurface exploration program in 2015 consisting of three (3) test borings (BB BTBR-101 through BB-BTBR-103). One boring was drilled behind each abutment and one through the existing bridge deck at the central pier. MaineDOT personnel cut a hole in the metal grate deck at the center pier prior to drill and patched it upon completion. The two remaining borings were drilled behind each abutment, using a truck mounted drill rig. Upon completion, the abutment borings were backfilled with cuttings and asphalt cold patch. The center pier boring was backfilled with bentonite chips. The as drilled boring locations and ground surface or deck elevations were surveyed by MaineDOT, and are shown on **Figure 2**.

The abutment test borings were drilled to depths of approximately 15.6 and 19.0 feet below ground surface (bgs). The central pier test boring was drilled to a depth of 47.8 feet below bridge deck surface, through the stone pier. The three (3) borings were each terminated after coring approximately 10 feet into bedrock. New England Boring Contractors, Inc. of Hermon, Maine provided drilling services and coordinated utility clearance for the project. Drilling was completed between May 26, 2015 and May 27, 2015. GZA personnel monitored the drilling work and prepared logs of each boring that are included in **Appendix C**.

Test borings were drilled using a 3-inch spun casing and drive and wash drilling techniques. Standard penetration testing (SPT) and split-spoon sampling were performed continuously at each abutment using a 24-inch-long, 1-3/8-inch inside diameter sampler. Bedrock and pier concrete/block masonry cores were obtained using a 2-inch nominal diameter, NQ2, core barrel.

2.3 FINAL DESIGN SUBSURFACE EXPLORATIONS

GZA completed a final design subsurface exploration program consisting of eight (8) test borings (BB-BTBR-201 through -208). The test boring program included five borings drilled with a truck rig: two through the bridge deck west of Pier 2; one through the bridge deck east of Pier 2; and one at each end of the temporary bridge alignment where it rejoins the existing roadway to provide data for the contractors use in design of the temporary detour. Three borings (BB-BTBR-206 through BB-BTBR-208) were drilled from a barge-mounted drill rig along the temporary bridge alignment to provide data for the contractor's use in designing the temporary bridge foundations.

The as-drilled boring locations and ground surface or deck elevations were surveyed by MaineDOT, and are shown on **Figures 2** and **3**. Elevations referenced in this report are in feet and refer to the National American Vertical Datum of 1988 (NAVD88).

The borings were drilled to depths of approximately 2.0 to 27.0 feet bgs. The borings were cored approximately 5.0 to 12.6 feet into bedrock, except boring BB-BTBR-201, which was carried to practical refusal. New England Boring Contractors of Hermon, Maine provided drilling services and coordinated utility clearance. The drilling was completed between June 12, 2017 and June 21, 2017. GZA personnel monitored the drilling work and prepared logs of each boring that are included in **Appendix C**.

The borings were drilled using 3- and 4-inch driven casing and drive-and-wash drilling techniques. Standard penetration testing (SPT) and split-spoon sampling were performed at 5-foot typical intervals in the overburden using a 24-inch-long, 1-3/8-inch inside-diameter sampler, driven with a 140-lb automatic hammer with a 30-inch drop. Bedrock cores were obtained using NQ2 coring equipment. New England Boring provided calibration data for the automatic hammer No. NEBC 1 indicating an efficiency factor of 0.75.

3.0 LABORATORY TESTING

GZA retained Thielsch Engineering's Geotechnical Laboratory in Cranston, Rhode Island to complete a soil and bedrock testing program to assess the gradation and engineering characteristics of the soil and strength of the bedrock. The testing program consisted of two gradation analysis/AASHTO Classification/Frost Classification assessments on soil and two unconfined compression strength tests with strain measurements on bedrock core samples. Results of the testing are included in **Appendix D**.

4.0 SUBSURFACE CONDITIONS

4.1 SURFICIAL AND BEDROCK GEOLOGY

Surficial geologic units mapped in the Barter's Island Bridge area include sand, gravel and silt, marine sand and silty clay (Presumpscot Formation), and glacial till, overlying bedrock. Bedrock at the site is mapped as the Bucksport Formation bedrock unit. The Bucksport Formation is described as beds of fine to medium grained, medium gray to purplish gray granofels and gneiss, with interbeds of greenish gray calc-silicate and some rusty biotite schist zones and pegmatite intrusions.

4.2 SUBSURFACE PROFILE

Two soil units were encountered in the test borings beneath surficial pavement and overlying bedrock: Fill and Harbor Bottom Deposit. Stone Masonry was encountered in one boring that was drilled through the central pier. 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 C**. The subsurface conditions are also shown in relation to the bridge alignment on the interpretive subsurface profile on **Figure 3**.

Subsurface Unit	Approximate Encountered Thickness (ft)	Generalized Description
Asphalt Pavement	0.8	Asphalt Pavement • Encountered in borings BB-BTBR-101, -103, -201, and -205
Fill	0.9 to 6.0	Brown, medium dense, fine to coarse SAND, little Silt, trace Gravel to Brown, medium dense, fine to coarse SAND, little Gravel, trace Silt (SM). • MaineDOT Frost Classification = II • Encountered in borings BB-BTBR-101, -103, -201, and -205
Concrete over	5.0	Portland Cement Concrete Cap
Stone Masonry	26.3	Stone masonry of variable rock types and block sizes typically ranging from approximately 5- to 18-inches, with 2- to 4-inch shim stones, and 2- to 6-inch voids, occasionally grout-filled between stones. • Encountered at Pier 2 in BB-BTR-102 only.
Harbor Bottom Deposit	0.2 to 13.2	Gray, soft, SILT, trace Sand, trace Gravel to Brown-gray, very loose, fine to coarse SAND, little Gravel, trace Silt. • Encountered in borings BB-BTBR-202, -203, -204, -207, -208 and -208A
Top of Bedrock Elevation	Abutment 1 Encountered Top of Rock: Approx. El. 13.6 Pier 2 Encountered Top of Rock: Approx. El. -16 to El.-25 Abutment 2 Encountered Top of Rock: Approx. El. 7.3	

4.2.1 Bedrock

Bedrock consisted of interlayered hard to moderately hard, fresh to moderately weathered, coarse grained, black to light gray, GNEISS. Joints are very close to close, low angle to high angle, planar to stepped, rough, tight to moderately wide, discolored; and very hard, fresh, coarse to very coarse grained, light gray to pink, PEGMATITE. Joints are very close to close, low angle to moderately dipping, stepped to undulating, rough, tight to open, fresh to discolored. The Rock Quality Designation (RQD) ranged from 0 to 100 percent. Lower RQD cores were encountered from 0 to 5 feet below the top of rock and are anticipated to be excavated to prepare footing. The remaining samples have an average RQD of 65 percent, indicating fair quality rock.

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

SUMMARY OF BEDROCK STRENGTH TEST RESULTS							
Boring	**Depth below Existing Grade (ft)**	**Depth below Top of Rock (ft)**	**Elevation (ft NAVD 88)**	**Unconfined Compressive Strength (psi)**	**Secant Modulus @ 50% of Failure Stress (ksi)**	**Unit Weight (pcf)**	**Rock Type**
BB-BTBR-101	6.3	1.4	12.2	5,980	2,160	171	GNEISS
BB-BTBR-102	43.5	6.7	-30.0	7,193	4,510	172	GNEISS

4.2.2 Groundwater

The test borings were drilled using drive-and-wash techniques, which introduce water into the borehole during drilling. As a result, stabilized groundwater levels were not determined at the boring locations. The pier test borings were drilled in the Back River where groundwater fluctuates with the tidal level.

The groundwater observations were made at the times and under the conditions stated in the borings logs. Fluctuations in groundwater will occur due to variations in temperature, season, precipitation, tide and other factors. Consequently, water levels during and after construction are likely to vary from those encountered at the time the observations were made.

5.0 ENGINEERING EVALUATIONS

5.1 GENERAL

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

5.2 APPROACH EMBANKMENTS

The existing bridge approaches were constructed with maximum side slope angles of 2 horizontal to 1 vertical (2H:1V), or flatter. No modifications are currently anticipated in the horizontal or vertical alignments of the approach embankments

5.3 LOAD AND RESISTANCE FACTORS

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

SUMMARY OF LOAD FACTORS – STRENGTH I		
Horizontal Load Factor Type	**Load Factor Symbol**	**Load Factor**
Earth Pressure from Retained Backfill – Active	γ_{EHMAX}	1.5
Max Earth Pressure from Road Base	γ_{ESMAX}	1.5
Min Earth Pressure from Road Base	γ_{ESMIN}	0.75
Live Load – Roadway	γ_{Ls}	1.75
Live Load – Superstructure	γ_{LL}	1.75

Recommended LRFD resistance factors for strength limit state design of the bedrock-bearing foundations were derived from LRFD Tables 10.5.5.2.2-1 and 11.5.7-1, and are presented in the following table.

RESISTANCE FACTORS – STRENGTH I		
Foundation Resistance Type	**Method/Condition**	**Resistance Factor (ϕ)**
Bearing	Footing on Rock	0.45
Sliding	Tremie Concrete on Rock	0.8

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

5.4 SPREAD FOOTINGS BEARING ON ROCK

Pier 1, Pier 3, and both abutment substructures are planned to be reused, and they are believed to be supported by concrete tremie seals bearing on bedrock. The new Pier 2 and control house foundations are proposed to consist of tremie seals and spread footings bearing on rock.

Preliminary 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 (7th Edition) of the AASHTO Design Specifications does not include the Rock Mass Rating (RMR) formulation included in the previous version (6th Edition). However, Articles C10.4.6.4 and 10.6.2.6.2 of the 7th Edition refer to RMR-based design procedures for footings on rock, so the 6th Edition methodology was followed.

GZA used bedrock data obtained in test borings BB-BTBR-101 and BB-BTBR-102 to develop foundation design parameters at Pier 2. The bedrock properties used in the bearing resistance evaluation are presented below:

DESIGN BEDROCK PROPERTIES FOR BEARING RESISTANCE EVALUATION					
Rock Type	**RQD (percent)**	**Unconfined Compressive Strength (ksi)**	**Rock Mass Rating (RMR)**	**m**	**s**
Gneiss	65	6.0	44	0.458	0.00009

It is anticipated that the highly fractured (RQD = 0) rock encountered near the proposed pier will be removed during subgrade preparation. Consequently, these data were not included in the analyses.

Based on the available data and the stated methodology, the calculated nominal bearing resistance is 74 kips per square foot (ksf). This will provide a factored bearing resistance of 33 ksf for the strength loading condition.

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

5.5 TEMPORARY BRIDGE

An off-alignment, temporary bridge is the preferred alternative for maintenance of traffic. The temporary bridge centerline is proposed to be located 65 feet south of the exiting centerline at the movable span, beginning the transition at approximately STA. 102+54.6 and connecting back to the proposed bridge alignment at approximately STA 108+11.3. The bridge concept plan includes two temporary abutments, 5 piers, and 6 spans.

Borings BB-BTBR-201, and BB-BTBR-205 through BB-BTBR-208 were completed along the alignment of the proposed temporary bridge. These data should be made available to the bidders for use in evaluating abutments and foundations for the proposed temporary bridge. Logs of these explorations are included in **Appendix C.**

6.0 RECOMMENDATIONS

6.1 SPREAD FOOTINGS

The replacement for Pier 2 may be supported on spread footing foundations bearing on a tremie seal that bears directly on intact bedrock free of all loose soil and rock material. We recommend a nominal bearing resistance of 74 ksf. This will provide a factored bearing resistance of 33 ksf for the strength loading condition.

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

Based on the test borings we anticipate sloping rock conditions at Pier 2. If the exposed bedrock surface is steeper than 4 horizontal to 1 vertical (4H:1V), then anchoring, doweling, benching or other means should be employed to improve sliding resistance.

Existing substructures are proposed to remain in place for both abutments and at Piers 1 and 3. Given the acceptable performance of these substructures and the factored bearing resistance recommended above, the existing spread footing foundations are judged suitable for continued use.

For proposed foundations bearing on bedrock, we recommend that sliding resistance be assessed using a nominal sliding resistance coefficient (C tan ϕ_f) equal to 0.7 for cast-in-place concrete on sound rock, a resistance factor (ϕ_τ) of 0.8, and a factored sliding resistance coefficient is 0.56 for the Strength Limit State.

6.2 SEISMIC DESIGN

The rehabilitated bridge will be supported on new or existing footings bearing on bedrock. Therefore, in general accordance with LRFD Table C3.10.3.1, the bridge should be assigned to Site Class B.

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

SITE CLASS B SEISMIC DESIGN PARAMETERS	
Parameter	**Design Value**
Fpga	1.0
Fa	1.0
Fv	1.0
As (Period = 0.0 sec)	0.069 g
SDs (Period = 0.2 sec)	0.146 g
SD1 (Period = 1.0 sec)	0.042 g

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

6.3 EVALUATION OF EXISTING STRUCTURES

The recommendations in this section are provided should the designer wish to evaluate the existing structures based on current AASHTO and MaineDOT methodology. The existing substructures may be evaluated using the following geotechnical parameters:

- Recommended soil properties for existing abutment backfill soils are as follows:
  - Internal Friction Angle of Soil = 33°
  - Soil Total Unit Weight = 120 pcf
  - Coefficient Rankine Active Earth Pressure, K_a = 0.295

- Live load surcharge should be applied as a uniform lateral surcharge pressure using the equivalent fill height (Heq) values developed in accordance with AASHTO Article 3.11.6.4 based on the abutment/wingwall height and distance from the wall backface to the edge of traffic.
- For existing foundations bearing on bedrock, we recommend that sliding resistance be assessed using a nominal sliding resistance coefficient (C tan ϕ_f) equal to 0.7 for cast-in-place concrete on sound rock, a resistance factor (ϕ_τ) of 0.8, and a factored sliding resistance coefficient is 0.56 for the Strength Limit State.

7.0 CONSTRUCTION CONSIDERATIONS

This section describes geotechnical-related issues that have the potential to impact design and cost considerations for bridge construction. These items are provided in the bullets that follow.

7.1 SUPPORT OF EXCAVATION

We anticipate that temporary sheet pile cofferdams will be necessary for the construction of the proposed central pier. Given the irregular surface of the bedrock and the lack of potential toe-in, we anticipate that the system will most likely require two levels of internal bracing.

7.2 SUBGRADE PREPARATION

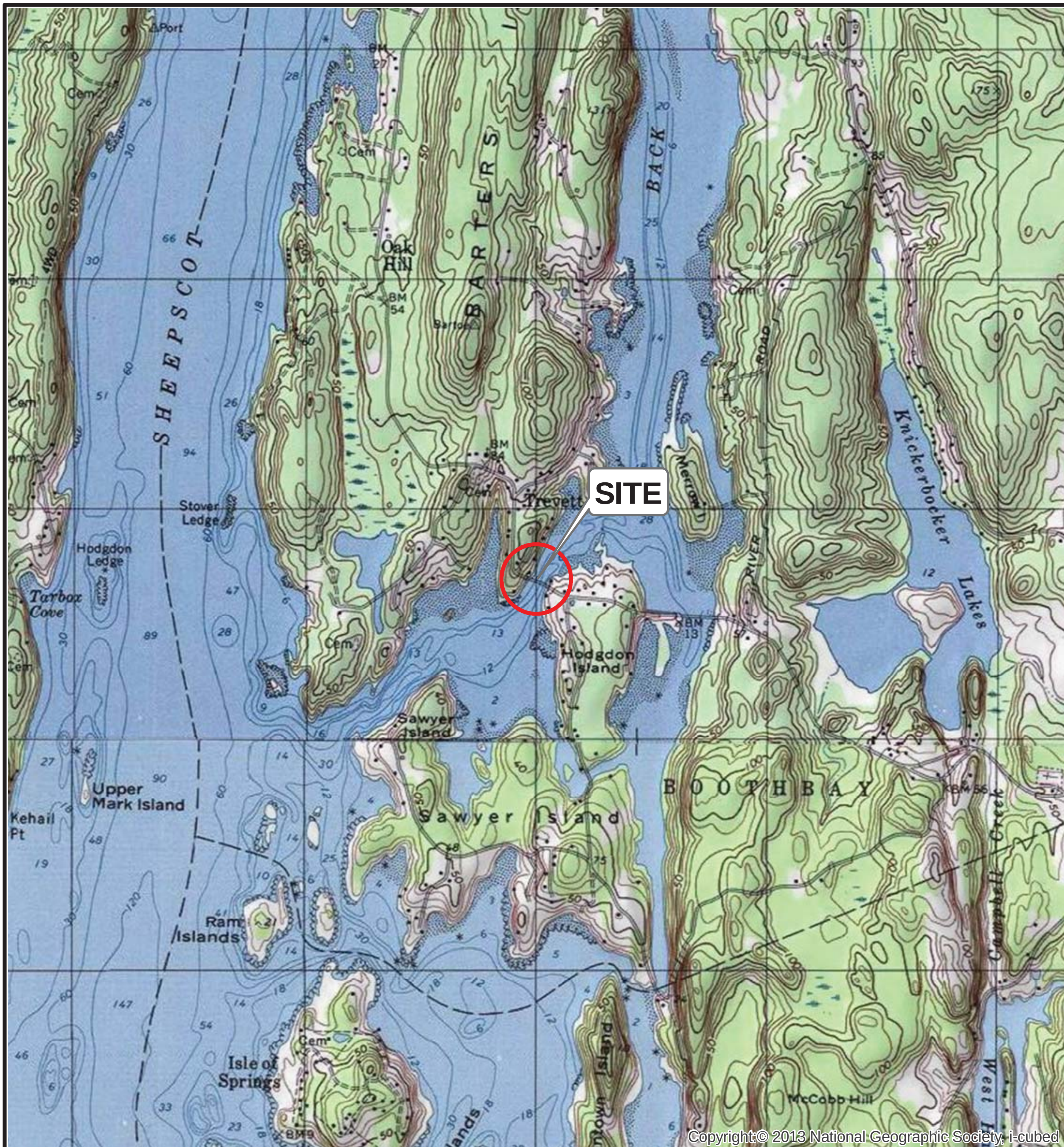
We anticipate that bedrock bearing surface preparation will be conducted in the wet, with water depths up to 30 feet or more during high tide, and that the bedrock surface will be variable in terms of elevation, slope and localized weathering. A combination of standard excavation equipment, hydraulic hoe-ramming equipment, and/or air lifting may be needed to remove the overburden and fractured/weathered rock. All soil and loose, decomposed, highly weathered and fractured bedrock should be removed from the footing bearing surface prior to placement of tremie seals. The prepared bearing surfaces should be checked by depth probing in conjunction with visual means such as diver and/or remotely operated vehicle (ROV) video inspection. A Special Provision should be prepared to define the project-specific requirements for subgrade preparation and quality assurance/quality control.

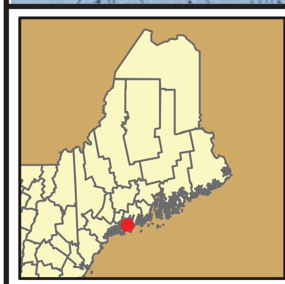
The Geotechnical Engineer and Designer should be provided cross-sections showing the prepared rock surface geometry prior to placement of concrete to evaluate whether benching, doweling, or subfooting reinforcement are needed for that foundation location. Based on the test borings we anticipate sloping rock conditions at Pier 2. If the exposed bedrock surface is steeper than 4 horizontal to 1 vertical (4H:1V), then anchoring, doweling, benching or other means should be employed to improving sliding resistance.

7.3 ENVIRONMENTAL CONCERNS

We understand that Section 7 environmental requirements may be imposed on the project, including schedule restrictions for in-water work. The cofferdams for Pier 2 and any temporary bridge piers will extend below mean tide level. Therefore, cofferdam installation may be affected by Section 7 requirements.

FIGURES





USGS QUADRANGLE LOCATION

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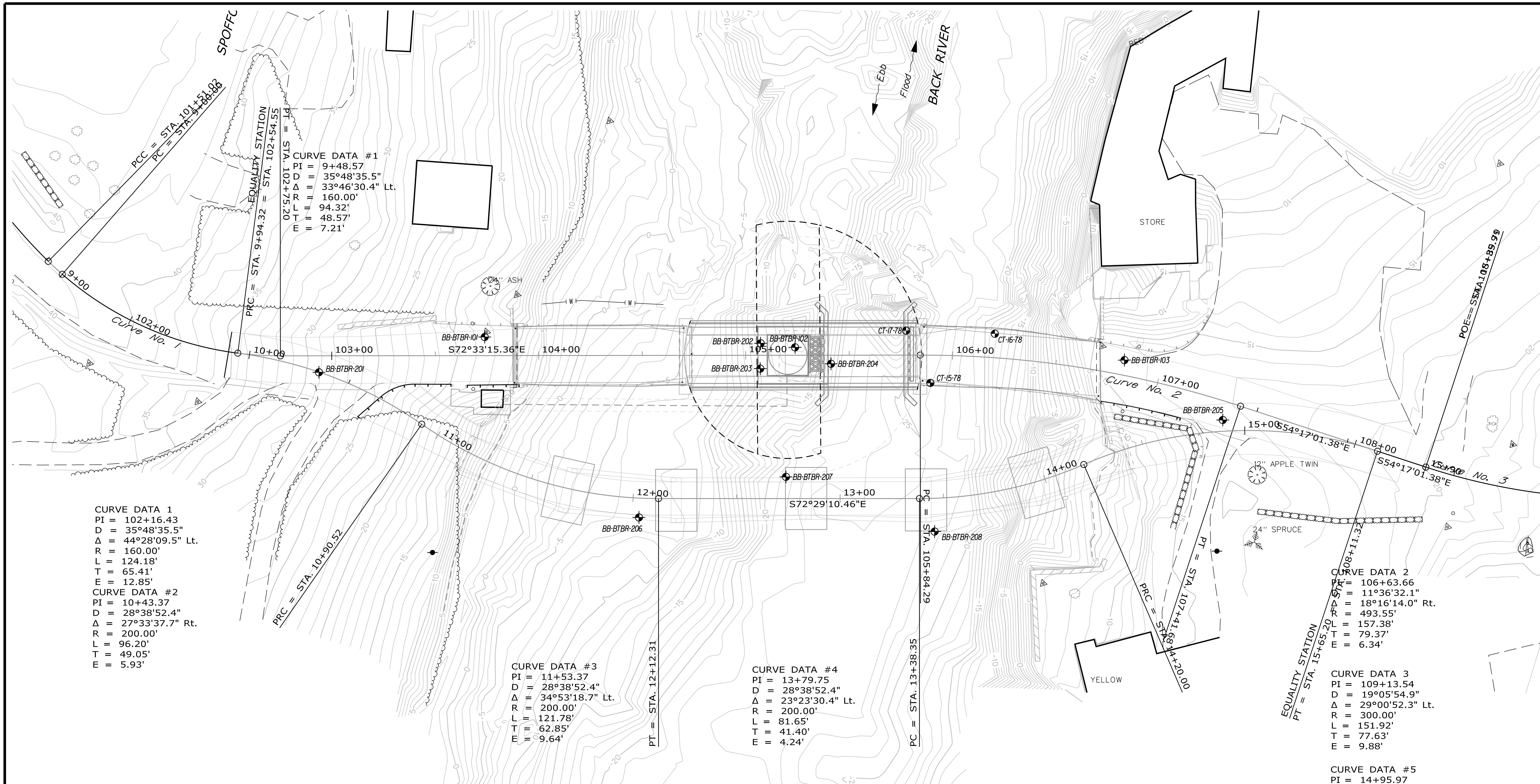
SOURCE : THIS MAP CONTAINS THE ESRI ARCGIS ONLINE USA TOPOGRAPHIC MAP SERVICE, PUBLISHED DECEMBER 12, 2009 BY ESRI ARCIMS SERVICES AND UPDATED AS NEEDED. THIS SERVICE USES UNIFORM NATIONALLY RECOGNIZED DATUM AND CARTOGRAPHY STANDARDS AND A VARIETY OF AVAILABLE SOURCES FROM SEVERAL DATA PROVIDERS. THIS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

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

SCALE IN FEET

Data Supplied by : esri USGS science for a changing world

PROJ. MGR.: BMC DESIGNED BY: BMC REVIEWED BY: CLS OPERATOR: ADM DATE: 08-02-2017	**LOCUS PLAN** **BARTER'S ISLAND BRIDGE #2039**	JOB NO. 09.0025887.01
	BOOTHBAY/TREVETT, MAINE MAINEDOT WIN 22607.00	FIGURE NO. 1



NOTES

1) Base map developed from electronic files provided by McFarland Johnson on July 11, 2017 (File: BDPLAN.dgn).

2) The as-drilled locations of the test borings were surveyed by a MaineDOT survey crew and supplied to GZA on June 28, 2017 (File: Borings_28jun17.dgn).

3) BB-BTBR-100 and BB-BTBR-200 series bridge borings were performed by New England Boring Contractors and observed by GZA personnel between May 26 and 27, 2015 and between June 15 and 25, 2017, respectively.

4) The CT-series boring locations and soil strata descriptions were taken from the State of Maine, Department of Transportation, Barter's Island over Back Bay, Foundation Survey Plan, dated November 1978.

BORING LOCATION PLAN LEGEND

BB-BTBR-208	Location and designation of cased wash boring
CT-17-78	Location and designation of 1978 Boring

PLAN

25 0 25 50

Scale

GZA Report Figure 2

PRELIMINARY
NOT FOR CONSTRUCTION

PREPARED BY: GZA

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2260(700)

WIN 22607.00

BRIDGE NO. 2039

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	XXXX	
DESIGN-DETAILED	BMC	JULY 2017
CHECKED-REVIEWED	CLS	JULY 2017
DESIGN2-DETAILED2	ARB	
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

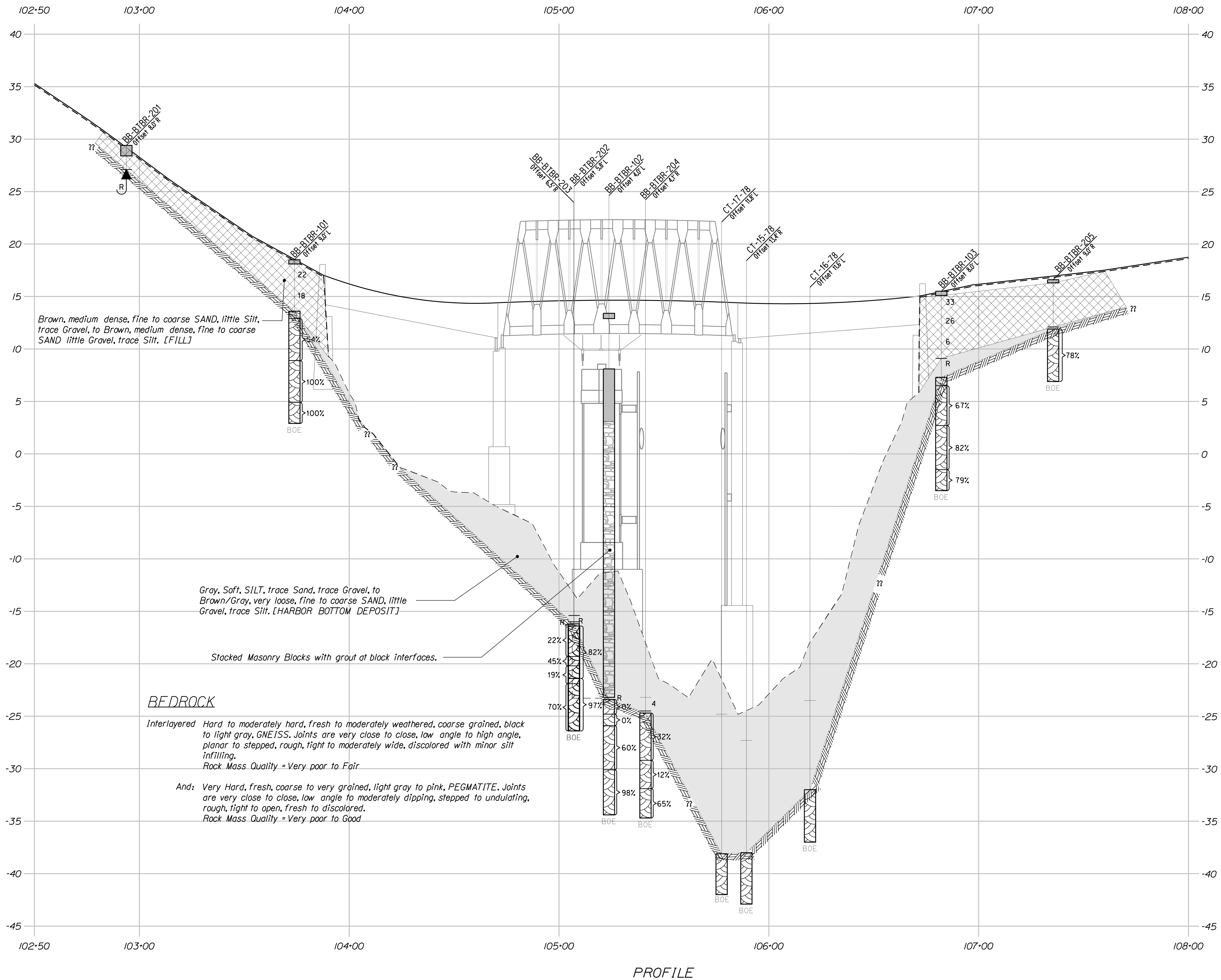
BARTER'S ISLAND BRIDGE
OVER BACK RIVER
BOOTHBAY LINCOLN COUNTY

BORING LOCATION PLAN

SHEET NUMBER
8
OF 12

Filename: ...\00\Geotech\MSTA008_BLP.dgn
Division: HIGHWAY
Username: common
Date:8/2/2017

Date:8/15/2017 common Division: HIGHWAY Username: Filename: ...\MSTA\009_ISP Bridge.dgn



NOTES

1) Profile developed from electronic files provided by McFarland Johnson on July 11, 2017 (File: Profile.dgn).

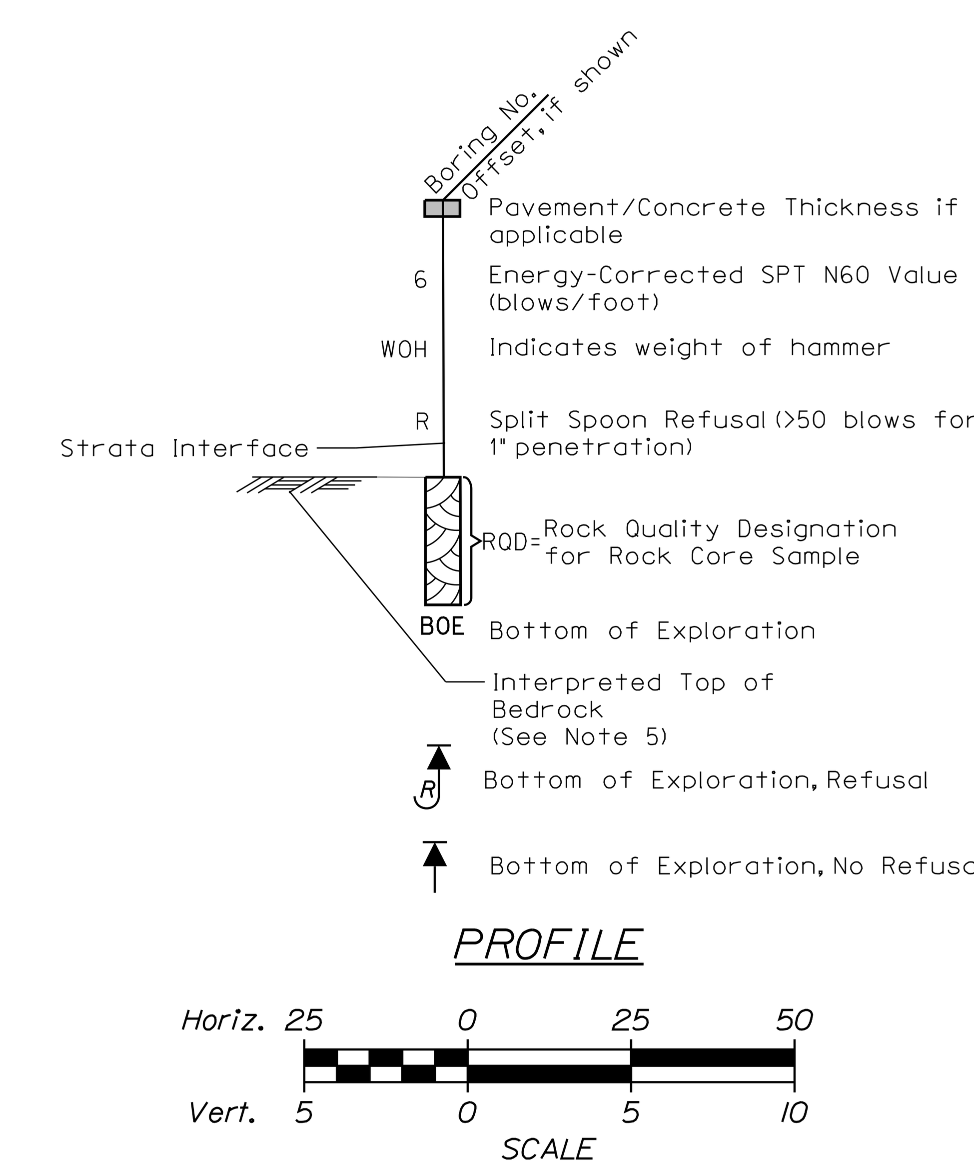
2) The as-drilled locations of the test borings were surveyed by a MaineDOT survey crew and supplied to GZA on June 28, 2017 (File: Borings_28jun17.dgn).

3) BB-BTBR-100 and BB-BTBR-200 series bridge borings were performed by New England Boring Contractors and observed by GZA personnel between May 26 and 27, 2015 and between June 15 and 25, 2017, respectively.

4) The CT-series boring locations and soil strata descriptions were taken from the State of Maine, Department of Transportation, Barter's Island over Back Bay, Foundation Survey Plan, dated November 1978.

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

INTERPRETIVE SUBSURFACE PROFILE LEGEND



PRELIMINARY
NOT FOR CONSTRUCTION

PREPARED BY: GZA

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2260(700)

WIN 22607.00

BRIDGE PLANS

BRIDGE NO. 2039

		BY	DATE
PROJ. MANAGER			
DESIGN-DETAILED	BMC	BMC	JULY 2017
CHECKED-REVIEWED	CLS	ARB	JULY 2017
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

BARTER'S ISLAND BRIDGE
OVER BACK RIVER
BOOTHBAY LINCOLN COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

9

OF 12

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.

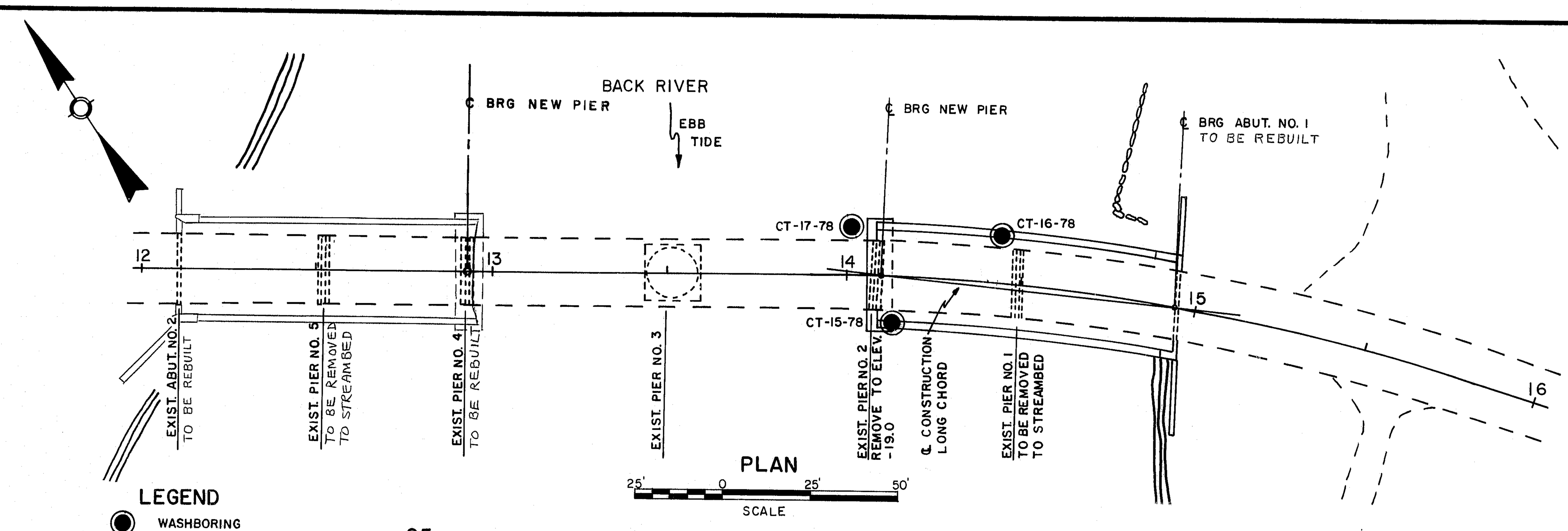
Additional Services

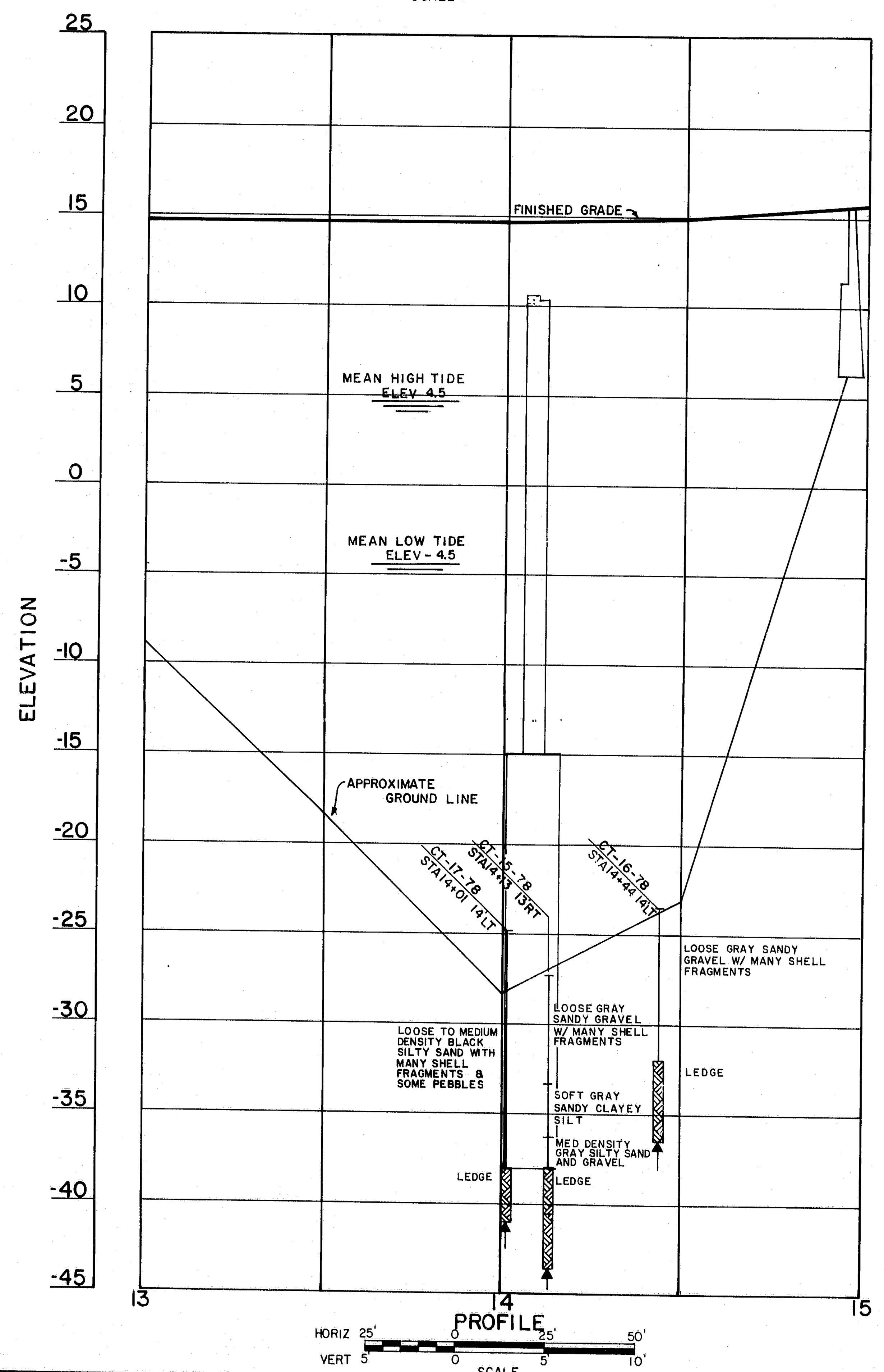
12. GZA recommends that we be retained to provide services during any future: site observations, design, implementation activities, construction and/or property development/redevelopment. This will allow us the opportunity to: i) observe conditions and compliance with our design concepts and opinions; 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.

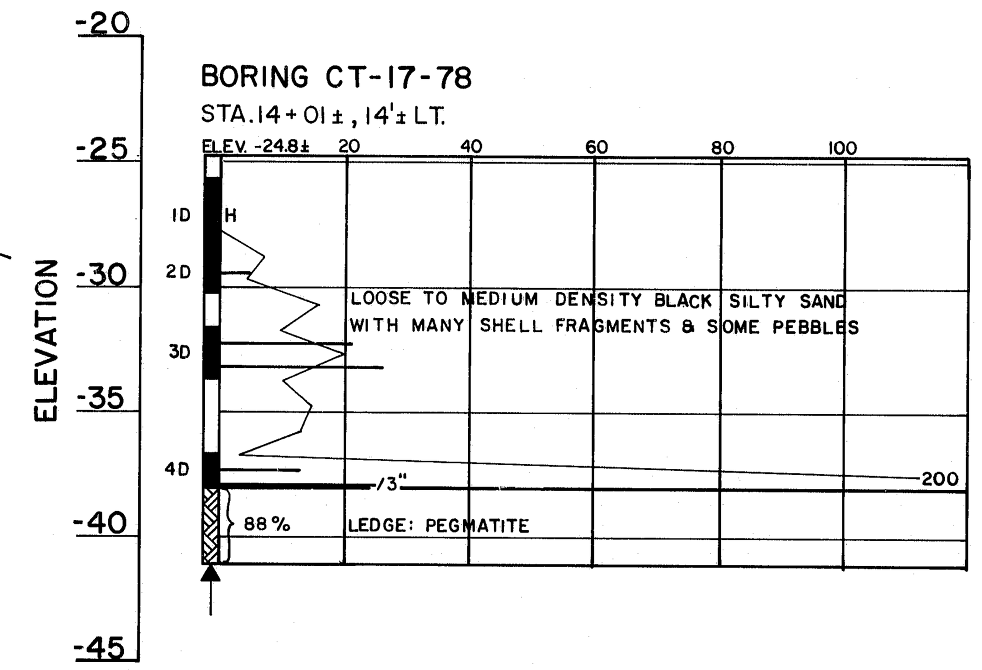
\\GZAPort1\Jobs\09 Jobs\0025800s\09.0025887.00 - Barters Island Bridge\09.0025887.01 Final Design\Report\Final DRAFT 25887.01 Barters Island Bridge GDR 081117.docx

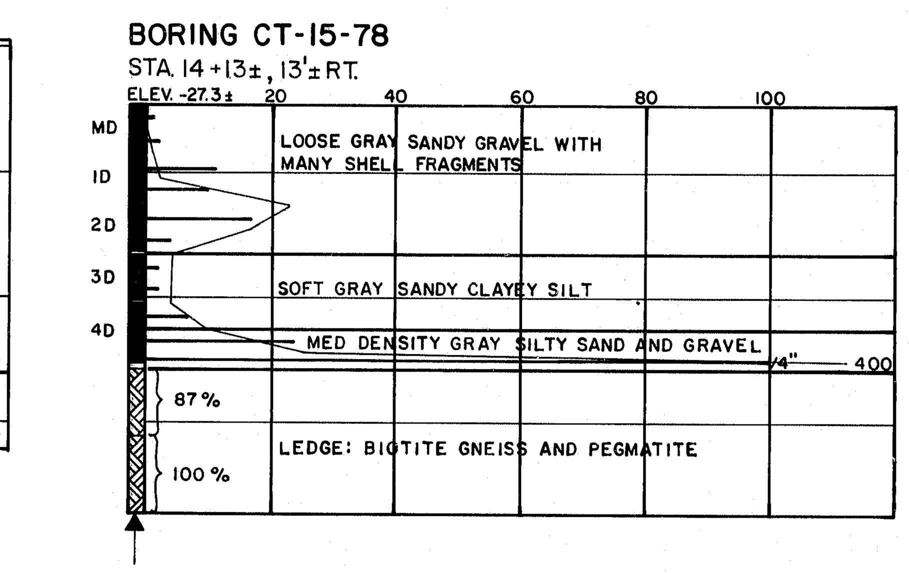
APPENDIX B – 1978 TEST BORING LOGS

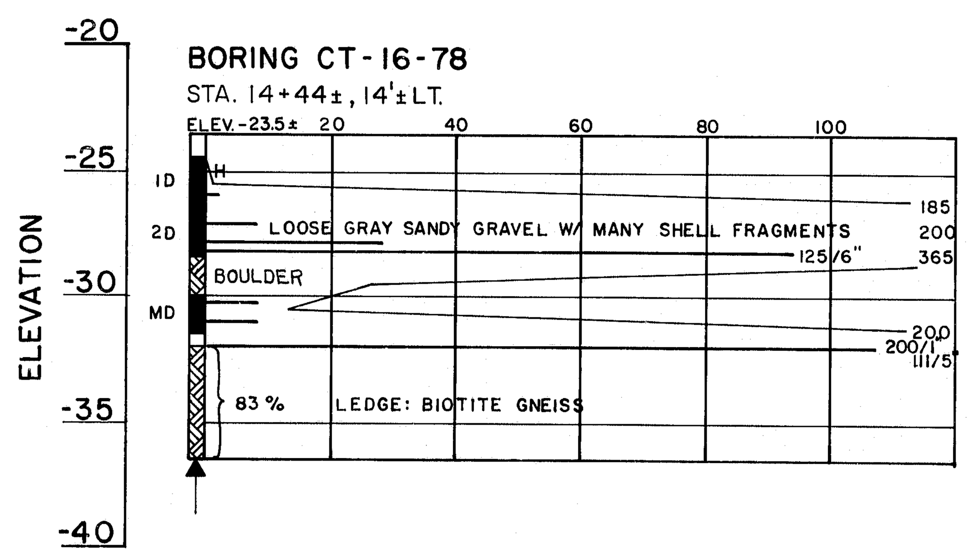
F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	0005(4)	8	44











BORING NOTES

All samples are made ahead of casing

Number of blows required to drive extra heavy casing one foot with 400 ft. lbs. of energy per blow

Location of sample or sample attempt

Number and type of dry sample
1D S & H Sampler #1290's

MD Unsuccessful sample attempt and type of sampler

Number of blows required to drive spoon or tubing one foot with 350 ft. lbs. of energy per blow

H Sampling spoon or seamless tubing driven by static weight of drill rods and hammer

Bottom of boring (may not be bottom of soil strata)

71% Locations cored by diamond bit and per cent recovery of rock

PLANS		BY	DATE
	PROJECT DESIGN ENGINEER	CDH	
	DESIGN-DETAILED	Soils	11-78
	CHECKED	Soils	11-78
	REVISIONS		
	FIELD CHANGES		

R92-346

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BARTER'S ISLAND BRIDGE
OVER
BACK RIVER
IN THE TOWN OF
BOOTHBAY
LINCOLN COUNTY
FOUNDATION SURVEY

SHEET 8 OF 44 AUGUSTA, MAINE NOV. 1978

APPENDIX C – GZA TEST BORING LOGS

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Barters Island Bridge #2039
Location: Boothbay, Maine
Boring No.: BB-BTBR-101
PIN: 22607.00

Driller:	New England Boring	**Elevation (ft.)**	18.5'	**Auger ID/OD:**	4"
Operator:	Brad Enos	**Datum:**		**Sampler:**	Split Spoon
Logged By:	Blaine Cardali	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	5/27/15-5/27/15	**Drilling Method:**	Drive & Wash / SSA	**Core Barrel:**	NQ
Boring Location:		**Casing ID/OD:**	3"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/13	0.4 - 2.4	10-11-11-15	22	22	SSA	18.1		PAVEMENT ——0.4	
										Brown, dry, medium dense, fine to coarse SAND, some Gravel. -FILL- (SM)	
	2D	24/16	2.4 - 4.4	13-11-7-9	18	18				Brown, moist, medium dense, fine to coarse SAND, some Gravel, little Silt. -FILL- (SM) SSA auger refusal at 4.9' bgs. Spun casing to 6.0' bgs. Set up to start core at 5.6' bgs.	G#3 A-1-b, SM WC=9.2
							SPUN	13.6		——4.9	
5	R1	48/45	5.6 - 9.6	RQD = 54%			NQ			Hard, fresh to slightly weathered, fine to medium grained, gray, GNEISS. Joints are close to moderately close, low to high angle, undulating, rough, fresh, tight to open. Rock Mass Quality = Fair Rock Core Times (min/ft): 1.75, 1.15, 2. 0, 1.5	q_p=6.0 ksi
10	R2	48/51	9.6 - 13.6	RQD = 100%						Hard, fresh, fine to medium grained, gray, GNEISS. Joints are close to wide, undulating, rough, fresh, tight to open. Rock Mass Quality = Excellent Rock Core Times (min/ft): 1.5, 1.5, 1.5, 1.75	
15	R3	24/24	13.6 - 15.6	RQD = 100%				2.9		Hard, fresh, fine to medium grained, gray, GNEISS. Joints are moderately close, low angle, undulating, rough, open. Rock Mass Quality = Excellent Rock Core Times (min/ft): 1.0, 1.5 ——15.6	
										Bottom of Exploration at 15.60 feet below ground surface.	
20											
25											

Remarks:

1. Spun 3" casing to 6.0' bgs, NQ to 15.6' bgs.

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:** Barters Island Bridge #2039 **Location:** Boothbay, Maine	**Boring No.:** BB-BTBR-102 **PIN:** 22607.00

Driller:	New England Boring	**Elevation (ft.)**	13.5'	**Auger ID/OD:**	4"
Operator:	Brad Enos	**Datum:**		**Sampler:**	Split Spoon
Logged By:	Blaine Cardali	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	5/26/15-5/27/15	**Drilling Method:**	Drive & Wash / SSA	**Core Barrel:**	NQ
Boring Location:		**Casing ID/OD:**	3"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SPUN	13.0		Bridge Deck (0.5)	
										Set up to core at 5.3' below top of deck.	
5	R1	60/55	5.3 - 10.3				NQ	8.2		CONCRETE (Bridge Structure) (5.3) Rock Core Times (min/ft): 1.5, 2.25, 2.5, 3.0	
10	R2	73/65	10.3 - 16.4					3.2		BLOCK FOUNDATION (10.3) Varying color/type of rock. Block thickness typically between 10"-15".	
15	R3	14/8	16.4 - 17.6							BLOCK FOUNDATION Blcok thickness 4" or less.	
	R4	40/11	17.6 - 20.9							BLOCK FOUNDATION. Block thickness 3" or less. Some Sand infilling.	
20	R5	24/22	21.1 - 23.1							BLOCK FOUNDATION. Blocks ranged in thickness from 3"-13". Grout encountered in voids.	
	R6	32/40	23.1 - 25.8							BLOCK FOUNDATION Block thickness ranged from 5"-12". Grout encountered in voids.	
25											

Remarks:

1. 5.3' from top of deck to top of concrete surface, spun 3" casing 6" into concrete and set up to core. NQ casing removed due to possible blocking collapse at approximately 20.0' below deck surface (bds). Spun 3" casing to 35.5' bds.
2. Based on recovery and drilling effort, TOR is approximately 36.8' below Top of Bridge Deck.
3. Casing spun to 39.1' bds, then cored to 47.8' bds.
4. Mean water height 17.0' bds at time of drilling.

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: Barters Island Bridge #2039	**Boring No.:** BB-BTBR-102	
Location: Boothbay, Maine	**PIN:** 22607.00	

Driller:	New England Boring	**Elevation (ft.)**	13.5'	**Auger ID/OD:**	4"
Operator:	Brad Enos	**Datum:**		**Sampler:**	Split Spoon
Logged By:	Blaine Cardali	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	5/26/15-5/27/15	**Drilling Method:**	Drive & Wash / SSA	**Core Barrel:**	NQ
Boring Location:		**Casing ID/OD:**	3"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
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MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	R7	14/6	25.8 - 27.0							BLOCK FOUNDATION (6" thick block)	
	R8	20/7	27.0 - 28.7							BLOCK FOUNDATION Block pieces 3" or less.	
	R9	88/46	28.8 - 36.1							BLOCK FOUNDATION Block thickness ranged from 5"-18". Voids encountered varying from 2"-6" based on drilling action.	
30											
35											
	1D	2/2	36.6 - 36.8	50/2"				-23.1		36.6	
	R10	17/14	36.8 - 38.2	RQD = 0%	R			-23.3		Gray, wet, dense, fine SAND and shells (likely wash material). 36.8	
	R11	13/18	38.2 - 39.3	RQD = 0%						Hard to medium hard, fresh to slightly weathered, fine to medium grained, gray, GNEISS. Joints are very close to close, low to high angle, undulating, rough, partially open to open. Rock Mass Quality = Very Poor Rock Core Times (min/ft): 1.5, 6.0, 2.0/2"	
	R12	50/48	39.3 - 43.5	RQD = 60%						Hard to medium hard, fresh to slightly weathered, fine to medium grained, gray, GNEISS. Joints are very close to close, low to high angle, undulating, rough, partially open to open. Rock Mass Quality = Very Poor Rock Core Times (min/ft): 2.5/13"	
40											
	R13	51/53	43.5 - 47.8	RQD = 98%						Hard to medium hard, fresh, fine to medium grained, gray, GNEISS. Joints are close to moderately close, low to high angle, undulating, rough, fresh, tight to partially open. Rock Mass Quality = Fair Rock Core Times (min/ft): 2.0/8", 2.5, 2.25, 2.0, 1.75/6".	q_p=7.2 ksi
45											
								-34.3		Hard to medium hard, fresh, fine to medium grained, gray, GNEISS. Joints are close to moderately close, low to high angle, undulating, rough, fresh, tight to partially open. Rock Mass Quality = Excellent Rock Core Times (min/ft): .5/6", 3.5, 2. 0, 2.5/9" 47.8	
50										**Bottom of Exploration at 47.80 feet below ground**	

Remarks:

1. 5.3' from top of deck to top of concrete surface, spun 3" casing 6" into concrete and set up to core. NQ casing removed due to possible blocking collapse at approximately 20.0' below deck surface (bds). Spun 3" casing to 35.5' bds.
2. Based on recovery and drilling effort, TOR is approximately 36.8' below Top of Bridge Deck.
3. Casing spun to 39.1' bds, then cored to 47.8' bds.
4. Mean water height 17.0' bds at time of drilling.

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: Barters Island Bridge #2039
Location: Boothbay, Maine

Boring No.: BB-BTBR-102
PIN: 22607.00

Driller:	New England Boring	**Elevation (ft.)**	13.5'	**Auger ID/OD:**	4"
Operator:	Brad Enos	**Datum:**		**Sampler:**	Split Spoon
Logged By:	Blaine Cardali	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	5/26/15-5/27/15	**Drilling Method:**	Drive & Wash / SSA	**Core Barrel:**	NQ
Boring Location:		**Casing ID/OD:**	3"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										**surface.**	
55											
60											
65											
70											
75											

Remarks:

1. 5.3' from top of deck to top of concrete surface, spun 3" casing 6" into concrete and set up to core. NQ casing removed due to possible blocking collapse at approximately 20.0' below deck surface (bds). Spun 3" casing to 35.5' bds.
2. Based on recovery and drilling effort, TOR is approximately 36.8' below Top of Bridge Deck.
3. Casing spun to 39.1' bds, then cored to 47.8' bds.
4. Mean water height 17.0' bds at time of drilling.

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: Barters Island Bridge #2039
Location: Boothbay, Maine

Boring No.: BB-BTBR-103
PIN: 22607.00

Driller:	New England Boring	**Elevation (ft.)**	15.5'	**Auger ID/OD:**	4"
Operator:	Brad Enos	**Datum:**		**Sampler:**	Split Spoon
Logged By:	Blaine Cardali	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	5/26/15-5/27/15	**Drilling Method:**	Drive & Wash / SSA	**Core Barrel:**	NQ
Boring Location:		**Casing ID/OD:**	3"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/16	0.4 - 2.4	14-15-18-19	33	33	SPUN	15.1		PAVEMENT — 0.4	
										Brown, dry, dense, fine to coarse SAND, some Gravel, trace Silt. -FILL- (SM)	
	2D	24/10	2.4 - 4.4	20-14-12-8	26	26				Brown, dry, medium dense, fine to coarse SAND, some Gravel, trace Silt. -FILL- (SM)	
	3D	24/5	4.4 - 6.4	5-4-2-1	6	6				Brown, dry, loose, fine to coarse SAND, some Gravel, trace Silt. -FILL- (SM)	
5											
	4D	21/7	6.4 - 8.2	3-3-2-50/3"	R			9.1		6.4 — Dark brown, moist, loose, fine to coarse SAND, some Gravel, some Silt. -SAND- (SM)	G#4 A-1-b, SM wc=20.9
							NQ	7.3		Split spoon refusal at 8.2' bgs. Spun casing to 9.0' bgs and setup to core. — 8.2	
	R1	46/45	9.0 - 12.8	RQD = 67%						Hard, fresh, fine to medium grained, gray, GNEISS. Joints are moderately close, high angle, undulating, rough, tight to partially open. Rock Mass Quality = Fair Rock Core Times (min/ft): 1.75, 1.5, 1.5, 1.5/9"	
10											
	R2	50/50	12.8 - 17.0	RQD = 82%						Hard, fresh to slightly weathered, fine to medium grained, gray, GNEISS. Joints are close to moderately close, low angle, undulating, rough, fresh, partially open to moderately wide. Rock Mass Quality = Good Rock Core Times (min/ft): 0.5/2", 2.0, 1.5, 1.5, 1.25	
15											
	R3	24/24	17.0 - 19.0	RQD = 79%				-3.5		Hard, fresh to slightly weathered, fine to medium grained, gray, gneiss. Joints are close to moderately close, undulating, rough, fresh, partially open. Rock Mass Quality = Good Rock Core Times (min/ft): 1.0, 1.25 — 19.0	
										Bottom of Exploration at 19.00 feet below ground surface.	
20											
25											

Remarks:

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Barter's Island Bridge #2039 Over Back River Location: Boothbay, Maine	Boring No.: BB-BTBR-201 PIN: 22607.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	29.4	**Auger ID/OD:**	3.5"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	E. Friede	**Rig Type:**	Truck Mobile B-53	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	06/15/17 - 06/15/17	**Drilling Method:**	SSA	**Core Barrel:**	--
Boring Location:	N139569.1, E1496092.9	**Casing ID/OD:**	-	**Water Level*:**	Not observed
Hammer Efficiency Factor:		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			-ASPHALT-	
								28.4		1.0	
	1D	15/15	1.0 - 2.3	8-30-50/2"	R					1.0'-1.9': Brown and black, moist, fine to coarse SAND, little Silt, trace Gravel, with Mica fragments. (SM) -FILL-	
								27.5		1.9	
								27.1		1.9'-2.3': White and gray, dry, weathered Rock (PEGMATITE), pulverized to fine to coarse Sand, little Gravel/Rock fragments.	
										2.3	
5										**Bottom of Exploration at 2.30 feet below ground surface.**	
10											
15											
20											
25											

Remarks:

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Barter's Island Bridge #2039 Over Back River
Location: Boothbay, Maine

Boring No.: BB-BTBR-202

PIN: 22607.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	-15.4	**Auger ID/OD:**	-
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	E. Friede	**Rig Type:**	Truck Mobile B-53	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	06/12/17 - 06/12/17	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ Wireline
Boring Location:	N139518.2, E1496300.6	**Casing ID/OD:**	3"	**Water Level*:**	Tidal
Hammer Efficiency Factor:		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	18/6	0.0 - 1.5	WOH-1-80			R/C	-16.4		Gray, medium to coarse SAND, little Gravel (rock fragments), trace organics (shells), trace Silt, wet. (SP) -HARBOR BOTTOM DEPOSIT- Casing refusal at 0.9'. Roller bit to 1.0', begin core. 1.0	
	R1	60/60	1.0 - 6.0	RQD = 82%			NQ			R1: Hard, fresh to discolored, gray and black, coarse grained, GNEISS. Joints are low angle to moderately dipping, planar, undulating, rough to smooth, close, medium wide to partially open, discolored to slightly weathered (rusty), trace brown Silt infilling at 4.7'. Fractured zone at 4.7'. Rock Mass Quality = Good Recovery = 100% Rock Core Times (min:sec): 1.0-2.0 (0:51) 2.0-3.0 (1:11) 3.0-4.0 (1:32) 4.0-5.0 (2:46) 5.0-6.0 (1:00)	
5											
	R2	60/60	6.0 - 11.0	RQD = 97%						R2: Hard, fresh to slightly weathered, gray and black, medium to coarse grained, GNEISS. Joints are horizontal to low angle, close to moderately spaced, moderately wide to tight, slightly weathered, trace silt infilling. Rock Mass Quality = Excellent Recovery = 100% Rock Core Times (min:sec): 6.0-7.0 (1:22) 7.0-8.0 (1:49) 8.0-9.0 (1:35) 9.0-10.0 (1:52) 10.0-11.0 (1:34)	
10								-26.4		11.0 **Bottom of Exploration at 11.00 feet below ground surface.**	
15											
20											
25											

Remarks:

1. Deck surveyed at El. 13.6', 29.0' to mudline.
2. Casing refusal at 0.9'. Split spoon drove to 1.5' without refusal (likely walking down rock slope).
3. Advanced roller bit to 1.0', began core.
4. No water return during coring.

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: Barter's Island Bridge #2039 Over Back River
Location: Boothbay, Maine

Boring No.: BB-BTBR-203

PIN: 22607.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	-16.0	**Auger ID/OD:**	-
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	E. Friede	**Rig Type:**	Truck Mobile B-53	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	06/12/17 - 06/12/17	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ Wireline
Boring Location:	N139506.6, E1496296.7	**Casing ID/OD:**	3"	**Water Level*:**	Tidal
Hammer Efficiency Factor:		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	2/1	0.0 - 0.2	50/2"			R/C	-16.2		Glass fragments, little fine Sand, little Silt, trace Gravel (rock fragments). Split spoon refusal at 0.2'. Begin core.	
	R1	37/37	0.2 - 3.3	RQD = 32%			NQ				
										0.2 — R1: 0.2'-3.1': Hard, discolored to slightly weathered (rusty, black), black and light gray, medium to coarse grained, GNEISS. Joints are low angle to high angle, very close to close, partially open to moderately wide, slightly weathered (discolored), dark gray and rusty, with minor gray silt infilling.	
	R2	11/10	3.3 - 4.2	RQD = 45%				-19.1		3.1 — R1: 3.1'-3.3': Very hard, discolored, light gray and pink, coarse to very coarse grained, PEGMATITE (quartz feldspar). Rock Mass Quality = Poor Recovery = 100% Rock Core Times (min:sec): 0.2-1.2 (1:54) 1.2-2.2 (2:51) 2.2-3.2 (3:52) 3.2-3.3 (2:20)	
	R3	20/21	4.2 - 5.9	RQD = 19%						R2: Hard, discolored (rusty), light gray/white, coarse to very coarse grained, PEGMATITE. Joints are horizontal, rough, planar, very close to close, moderately wide, with minor silt infilling. Rock Mass Quality = Poor Recovery = 90% Rock Core Times (min:sec): 3.3-4.2 (3:30)	
5	R4	54/51	5.9 - 10.4	RQD = 70%						R3: Hard, discolored (rusty), light gray/white, coarse to very coarse grained, PEGMATITE. Joints are low angle to moderately dipping, rough to smooth, planar, very close to close, open, slightly weathered, discolored with minor sand infilling. Biotite present at 4.7'. Rock Mass Quality = Very Poor Recovery = 105% Rock Core Times (min:sec): 4.2-5.2 (3:40) 5.2-5.9 (1:55)	
								-25.0		R4: 5.9'-9.0': Hard, discolored (rusty), light gray/white, coarse to very coarse grained, PEGMATITE. Joints are low angle, planar, rough, moderately close, discolored to fresh.	
10								-26.4		9.0 — R4: 9.0'-10.4': Hard, slightly to moderately weathered, black and gray, coarse grained, GNEISS. Joints are low angle to high angle, undulating and stepped, rough, close, partially open, moderately weathered, with minor silt infilling. Rock Mass Quality = Fair Recovery = 94% Rock Core Times (min:sec): 5.9-6.9 (1:13) 6.9-7.9 (2:18)	
15											
20											
25											

Remarks:

1. Deck surveyed at El. 13.5', 29.5' to mudline.

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: Barter's Island Bridge #2039 Over Back River
Location: Boothbay, Maine

Boring No.: BB-BTBR-203

PIN: 22607.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	-16.0	**Auger ID/OD:**	-
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	E. Friede	**Rig Type:**	Truck Mobile B-53	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	06/12/17 - 06/12/17	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ Wireline
Boring Location:	N139506.6, E1496296.7	**Casing ID/OD:**	3"	**Water Level*:**	Tidal
Hammer Efficiency Factor:		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										7.9-8.9 (2:13) 8.9-9.9 (1:51) 9.9-10.4 (1:30)	
										10.4	
										Bottom of Exploration at 10.40 feet below ground surface.	
30											
35											
40											
45											
50											

Remarks:

1. Deck surveyed at El. 13.5', 29.5' to mudline.

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* 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: Barter's Island Bridge #2039 Over Back River
Location: Boothbay, Maine

Boring No.: BB-BTBR-204
PIN: 22607.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	-23.2	**Auger ID/OD:**	-
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	E. Friede	**Rig Type:**	Truck Mobile B-53	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	06/13/17 - 06/13/17	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ Wireline
Boring Location:	N139498.5, E1496330.1	**Casing ID/OD:**	3"	**Water Level*:**	Tidal
Hammer Efficiency Factor:		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	18/7	0.0 - 1.5	9-3-1-4	4	0	15		0 .0'-1.3': Brown, wet, very loose, fine to coarse SAND, little organics (shells), little Silt, trace Gravel (rock fragments), poorly graded (coarse). (SP-SM)	
								-24.5	1.3	
	R1	54/41	1.5 - 6.0	RQD = 37%			4		1.3'-1.5': Blue-gray, wet, SILT, little fine to coarse Sand, trace Gravel (rock fragments), trace organics (shells). (ML) -HARBOR BOTTOM DEPOSIT- Roller bit to 1.5', begin to core.	
							53	-24.7	1.5	
							NQ		R1: 1.5'-3.6': Hard, fresh, black and light gray, medium to coarse grained, GNEISS. Joints are low angle, planar, undulating, smooth, close to moderately spaced, partially open, fresh to discolored (rusty).	
								-26.8	3.6	
									R1: 3.6'-3.7': Very hard, discolored to slightly weathered, light gray to gray, coarse grained, PEGMATITE.	
								-26.9	3.7	
5									R1: 3.7'-6.0': Soft, highly weathered, brown, fine to medium grained, GNEISS. Joints are very close to close, weathered to gravel, brown sand, and clay. Rock Mass Quality = Poor Recovery = 76% Rock Core Times (min:sec): 1.5-2.5 (1:48) 2.5-3.5 (1:31) 3.5-4.5 (0:54) 4.5-5.5 (2:58) 5.5-6.0 (3:12)	
	R2	32/32	6.0 - 8.7	RQD = 12%					R2: 6.0'-8.7': Hard, moderately weathered, black and light gray and brown, medium to coarse grained, GNEISS. Joints are very close to close, moderately dipping to steep, rough, undulating, open, discolored, slightly weathered, with minor silt infilling. Rock Mass Quality = Very Poor Recovery = 100% Rock Core Times (min:sec): 6.0-7.0 (1:08) 7.0-8.0 (2:20) 8.0-8.7 (3:30)	
								-31.9	8.7	
	R3	34/34	8.7 - 11.5	RQD = 65%					R3: 8.7'-8.9': Hard, fresh, gray and light gray, coarse grained, PEGMATITE.	
								-32.1	8.9	
10									R3: 8.9'-11.5': Hard, fresh to discolored, black to light gray, coarse grained, GNEISS. Joints are low angle to steep, rough, planar to stepped, very close to close, moderately wide to tight, discolored, with minor gray silt infilling. Rock Mass Quality = Fair Recovery = 100% Rock Core Times (min:sec): 8.7-9.7 (2:59) 9.7-10.7 (1:54)	
								-34.7		
15										
20										
25										

Remarks:

1. Deck surveyed at El. 13.6', 36.8' to mudline.
2. Casing refusal at 1.5'. Split spoon drove to 2.0' without refusal (likely walking down rock slope).

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Boring No.: BB-BTBR-204

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Barter's Island Bridge #2039 Over Back River
Location: Boothbay, Maine

Boring No.: BB-BTBR-204

PIN: 22607.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	-23.2	**Auger ID/OD:**	-
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	E. Friede	**Rig Type:**	Truck Mobile B-53	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	06/13/17 - 06/13/17	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ Wireline
Boring Location:	N139498.5, E1496330.1	**Casing ID/OD:**	3"	**Water Level*:**	Tidal
Hammer Efficiency Factor:		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										10.7-11.5 (2:40)	
										11.5	
										Bottom of Exploration at 11.50 feet below ground surface.	
30											
35											
40											
45											
50											

Remarks:

1. Deck surveyed at El. 13.6', 36.8' to mudline.
2. Casing refusal at 1.5'. Split spoon drove to 2.0' without refusal (likely walking down rock slope).

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
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Boring No.: BB-BTBR-204

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Barter's Island Bridge #2039 Over Back River
Location: Boothbay, Maine

Boring No.: BB-BTBR-205
PIN: 22607.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	16.6	**Auger ID/OD:**	-
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	E. Friede	**Rig Type:**	Truck Mobile B-53	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	06/13/17 - 06/13/17	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ Wireline
Boring Location:	N139415.7, E1496502.4	**Casing ID/OD:**	3"	**Water Level*:**	Tidal
Hammer Efficiency Factor:		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/15	0.3 - 2.3	11-17-12-16	29	0	R/C AHEAD	16.3		Top 4": ASPHALT (0.3)	
										Brown, moist, medium dense, fine to coarse SAND, little Gravel, trace Silt. -FILL- (4.5)	
	S2	2/1	4.5 - 4.7	50/2"			NQ	12.1		Brown/black, sandy SILT, trace Gravel. Split spoon refusal at 4.7', begin to core. (4.7)	
5	R1	60/58	4.7 - 9.7	RQD = 78%				11.9		R1: Hard to moderately hard, fresh to slightly weathered, black and light gray, medium to coarse grained, GNEISS. Joints are wide to very close, low angle to horizontal, rough, stepped, partially open to wide, slightly weathered, mica in joint surface. (8.6)	
								8.0		R1: 8.6'-9.7': Very hard, fresh, light gray and pink, coarse to very coarse grained, PEGMATITE. Joints are low angle, rough, stepped, very close to close, tight, fresh (quartz, feldspar, mica).	
								6.9		Rock Mass Quality = Good Recovery = 96% Rock Core Times (min:sec): 4.7-5.7 (1:50) 5.7-6.7 (1:15) 6.7-7.7 (1:39) 7.7-8.7 (1:41) 8.7-9.7 (1:56) (9.7)	
10										**Bottom of Exploration at 9.70 feet below ground surface.**	
15											
20											
25											

Remarks:

1. 3.0'-4.0': Sandy clay in auger cuttings.
2. Boring relocated approximately 4' to west of original location for road access.
3. 4.5': Spoon broken upon removal, penetration likely less than 2".
4. Casing at 4.7'.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Barter's Island Bridge #2039 Over Back River
Location: Boothbay, Maine

Boring No.: BB-BTBR-206

PIN: 22607.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	-4.9	**Auger ID/OD:**	-
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	E. Friede	**Rig Type:**	Truck Mobile B-53	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	06/20/17 - 06/20/17	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ Wireline
Boring Location:	N139455.6, E1496219.2	**Casing ID/OD:**	3"	**Water Level*:**	Tidal
Hammer Efficiency Factor:		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	14/0	0.0 - 1.2	WOR-WOR-50/3"			R/C			0'-1.3': No recovery. Roller bit to 2.0' and began to core.	
								-6.9		2.0	
	R1	60/60	2.0 - 7.0	RQD = 43%			NQ			R1: Hard, discolored to slightly weathered, gray and light gray, coarse to very coarse grained, PEGMATITE. Joints are horizontal to steep, rough, undulating, very close to close, open to moderately wide. Brown sand and clay infilling, weathered to gravel from 3.7'-4.5' (feldspar, quartz, mica). Rock Mass Quality = Poor Recovery = 100% Rock Core Times (min:sec): 2.0-3.0 (2:30) 3.0-4.0 (2:27) 4.0-5.0 (2:20) 5.0-6.0 (2:24) 6.0-7.0 (2:40)	
5											
	R2	60/60	7.0 - 12.0	RQD = 50%						R2: Hard, fresh to discolored, gray and light gray to black, coarse to very coarse grained, PEGMATITE. Joints are horizontal to moderately dipping, rough, undulating, very close to close, open to wide. Sand infilling, discolored to moderately weathered. Fractured zone from 9.9'-10.3'. Rock Mass Quality = Poor Recovery = 100% Rock Core Times (min:sec): 7.0-8.0 (2:21) 8.0-9.0 (1.56) 9.0-10.0 (2:05) 10.0-11.0 (2:18) 11.0-12.0 (2:35)	
10											
								-16.9		12.0	
										Bottom of Exploration at 12.00 feet below ground surface.	
15											
20											
25											

Remarks:

1. Drove 3" casing to 2.0' below mudline.

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: Barter's Island Bridge #2039 Over Back River
Location: Boothbay, Maine

Boring No.: BB-BTBR-207

PIN: 22607.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	-25.6	**Auger ID/OD:**	-
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	E. Friede	**Rig Type:**	CME 45	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	06/20/17 - 06/20/17	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ Wireline
Boring Location:	N139453.0, E1496292.9	**Casing ID/OD:**	3"	**Water Level*:**	Tidal
Hammer Efficiency Factor:		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/4	0.0 - 2.0	7-4-1-0	5	0	R/C			Brown, wet, loose, medium to coarse SAND, poorly graded, little organics (shells), trace glass fragments. (SP)	
5								-30.6		5.0	
	R1	18/18	5.0 - 6.5	RQD = 0%			NQ			R1: Medium hard, gray and dark gray, medium to coarse grained, GNEISS. Joints are weathered, high angle, rough, undulating, planar, very close to close, open to wide; greenish gray silty infilling, moderately weathered. Rock Mass Quality = Very Poor Recovery = 100% Rock Core Times (min:sec): 5.0-6.0 (2:20) 6.0-6.5 (1:50)	
	R2	21/18	6.5 - 8.3	RQD = 43%						R2: Medium hard, greenish gray to dark gray, medium to coarse grained, GNEISS. Joints are moderately weathered, low angle to high angle, smooth to rough, planar, undulating, very close to close, open to wide; silt and coarse sand infilling, moderately weathered. Rock Mass Quality = Poor Recovery = 86% Rock Core Times (min:sec): 6.5-7.0 (1:12) 7.0-8.0 (3:18) 8.0-8.3 (2:00)	
	R3	43/43	8.3 - 11.9	RQD = 33%						R3: Medium to hard, moderately weathered, greenish gray and dark gray, medium to very coarse grained, GNEISS. Joints are with zones of coarse grained, steep to low angle, rough to smooth, undulating to planar, very close to close, open to wide. Rock Mass Quality = Poor Recovery = 100% Rock Core Times (min:sec): 8.3-9.0 (1:15) 9.0-10.0 (3:20) 10.0-11.0 (2:20) 11.0-11.9 (3:00)	
10											
	R4	17/14	11.9 - 13.3	RQD = 0%						R4: Greenish gray and dark gray, medium to very coarse grained, GNEISS. Joints are very high angle, smooth, rough, undulating, very close. Gray silt infilling. Rock Mass Quality = Very Poor Recovery = 82% Rock Core Times (min:sec): 11.9-12.0 (0:10) 12.0-13.0 (1:46) 13.0-13.3 (3:00)	
								-38.9		13.3	
15											
20											
25											

Remarks:

1. Mudline at El -25.6'.

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: Barter's Island Bridge #2039 Over Back River
Location: Boothbay, Maine

Boring No.: BB-BTBR-207

PIN: 22607.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	-25.6	**Auger ID/OD:**	-
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	E. Friede	**Rig Type:**	CME 45	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	06/20/17 - 06/20/17	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ Wireline
Boring Location:	N139453.0, E1496292.9	**Casing ID/OD:**	3"	**Water Level*:**	Tidal
Hammer Efficiency Factor:		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										**Bottom of Exploration at 13.30 feet below ground surface.**	
30											
35											
40											
45											
50											

Remarks:

1. Mudline at El -25.6'.

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* 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: Barter's Island Bridge #2039 Over Back River
Location: Boothbay, Maine

Boring No.: BB-BTBR-208A

PIN: 22607.00

Driller:	New England Boring Contractors	**Elevation (ft.)**		**Auger ID/OD:**	-
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	E. Friede	**Rig Type:**	CME 45	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	06/21/17 - 06/21/17	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ Wireline
Boring Location:		**Casing ID/OD:**	3"	**Water Level*:**	Tidal
Hammer Efficiency Factor:		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/24	0.0 - 2.0	WOR-WOR-WOR-WOR			SS			Blue gray, wet, very soft, SILT & CLAY, trace fine Sand, trace organics (shells, slightly (medium) plastic. (ML)	
								-2.0		2.0	
										Bottom of Exploration at 2.00 feet below ground surface.	
5											
10											
15											
20											
25											

Remarks:

1. Current pushed drill off location after first sample interval. Reset barge and began new hole.

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: Barter's Island Bridge #2039 Over Back River
Location: Boothbay, Maine

Boring No.: BB-BTBR-208
PIN: 22607.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	-26.3	**Auger ID/OD:**	-
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	E. Friede	**Rig Type:**	CME 45	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	06/21/17 - 06/21/17	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ Wireline
Boring Location:	N139406.1, E1496353.4	**Casing ID/OD:**	3"	**Water Level*:**	Tidal
Hammer Efficiency Factor:		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0										Advanced boring to 4.0' below mudline; see BB-BTBR-208(A) for drilling details.	
								-30.3		4.0	
	S1	24/24	4.0 - 6.0	WOR-WOR-WOR-WOR			R/C			Blue-gray, wet, very soft, SILT & CLAY.	
5											
	2D	24/24	9.0 - 11.0	WOR-WOR-WOR-WOR						Blue-gray, wet, very soft, SILT & CLAY, some fine Sand.	
10											
								-39.5		13.2 -WEATHERED ROCK-	
								-40.3		14.0	
	R1	15/13	14.0 - 15.3	RQD = 0%			NQ			R1: Hard, slightly weathered, light g ray, weathered, coarse grained, PEGMATITE. Joints are low angle to moderately dipping, rough, undulating, very close, open to wide. Coarse sand and gray clay infilling, moderately weathered. Rock Mass Quality = Very Poor Recovery = 86% Rock Core Times (min:sec): 14.0-15.0 (2:09) 15.0-15.3 (1:00)	
15	R2	44/15	15.3 - 19.0	RQD = 0%						R2: Hard, moderately weathered, gray and dark gray, coarse grained, PEGMATITE. Joints are moderately dipping, rough, undulating, stepped, very close, open to wide. Sand and light gray clay infilling (possible weathered material). Moderately weathered. Rock Mass Quality = Very Poor Recovery = 34% Rock Core Times (min:sec): 15.3-16.3 (2:13) 16.3-17.3 (2:16) 17.3-18.3 (2:20) 18.3-19.0 (2:04)	
	R3	15/12	19.0 - 20.3	RQD = 0%						R3: Hard, moderately weathered, gray and light gray,	
20	R4	34/25	20.3 - 23.1	RQD = 12%				-47.9			
	R5	42/42	23.1 - 26.6	RQD = 36%				-49.4			
25											

Remarks:

1. Advanced rollerbit from 13.2' to 14.0', begin core.

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: Barter's Island Bridge #2039 Over Back River
Location: Boothbay, Maine

Boring No.: BB-BTBR-208

PIN: 22607.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	-26.3	**Auger ID/OD:**	-
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	E. Friede	**Rig Type:**	CME 45	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	06/21/17 - 06/21/17	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ Wireline
Boring Location:	N139406.1, E1496353.4	**Casing ID/OD:**	3"	**Water Level*:**	Tidal
Hammer Efficiency Factor:		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
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V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25								-52.9		coarse grained, PEGMATITE. Joints are moderately dipping to steep, rough, undulating, very close, wide. Light gray clay and coarse sand infilling. Joints highly weathered. Rock Mass Quality = Very Poor Recovery = 80% Rock Core Times (min:sec): 19.0-20.0 (1:58) 20.0-20.3 (2:06)	
30										R4: 20.3'-21.6': Hard, discolored, gray to light gray, coarse to very coarse grained, PEGMATITE. Joints are low angle to moderately dipping, rough, undulating, very close to close, open to wide. Coarse sand and gray clay infilling. Rock Mass Quality =Very Poor Recovery = 74% Rock Core Times (min:sec): 20.3-21.0 (1:30) 21.0-22.0 (2:10) 22.0-23.0 (3:00) 23.0-23.1 (2:20) 21.6	
35										R4: 21.6'-23.1: Highly weathered zone. Weathered to gray clay, coarse sand, and rock fragments. 23.1	
										R5: 23.1'-24.8': Hard, discolored, dark to light gray, medium to very coarse, GNEISS. Joints are low angle to moderately dipping, rough, stepped, planar, very close to close, partially open to moderately wide. Trace gray clay infilling at interface (24.8').	
40										R5: 24.8'-26.6': Hard, discolored, gray to light gray, coarse to very coarse, PEGMATITE. Joints are moderately dipping to high angle, rough, undulating, close to very close, partially open to wide. Gray clay infilling at 24.8' interface. Highly fractured zone from 25.7'- 26.1'. Rock Mass Quality = Poor Recovery = 100% Rock Core Times (min:sec): 23.1-24.0 (2:18) 24.0-25.0 (2:30) 25.0-26.0 (2:24) 26.0-26.6 (2:10) 26.6	
45										**Bottom of Exploration at 26.60 feet below ground surface.**	
50											

Remarks:

1. Advanced rollerbit from 13.2' to 14.0', begin core.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

APPENDIX D – LABORATORY TESTING RESULTS

THIELSCH ENGINEERING

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Barters Island Bridge

MDOT Project Number:

GZA Project Number: 09.0025887.00

Town(s): Boothbay, ME

Boring & Sample Identification Number	Station (Feet)	Sample No.	Depth (Feet)	Lab Number	Organic %	W.C.	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-BTBR-101		2D	2.4-4.4	3		9.2			SM	A-1-b	II
BB-BTBR-103		4D	6.4-8.2	4		20.9			SM	A-1-b	II

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

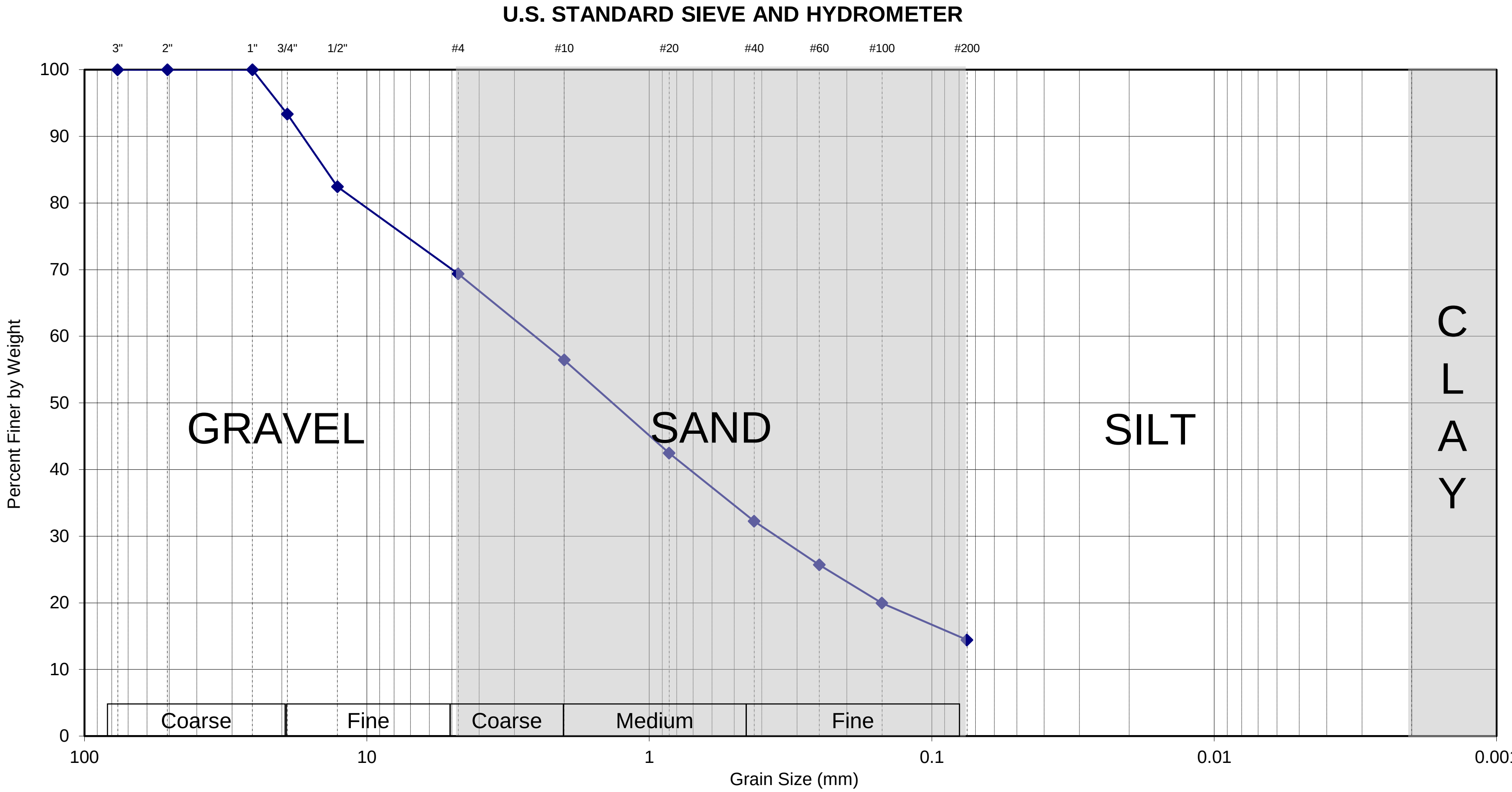
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

ASTM D422



Gravel	Sand	Fines
30.6%	55.0%	14.4%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
3	BB-BTBR-101	2D	2.4-4.4'	Brown f-c SAND, some f-c Gravel, little Silt (SM)	9.2			

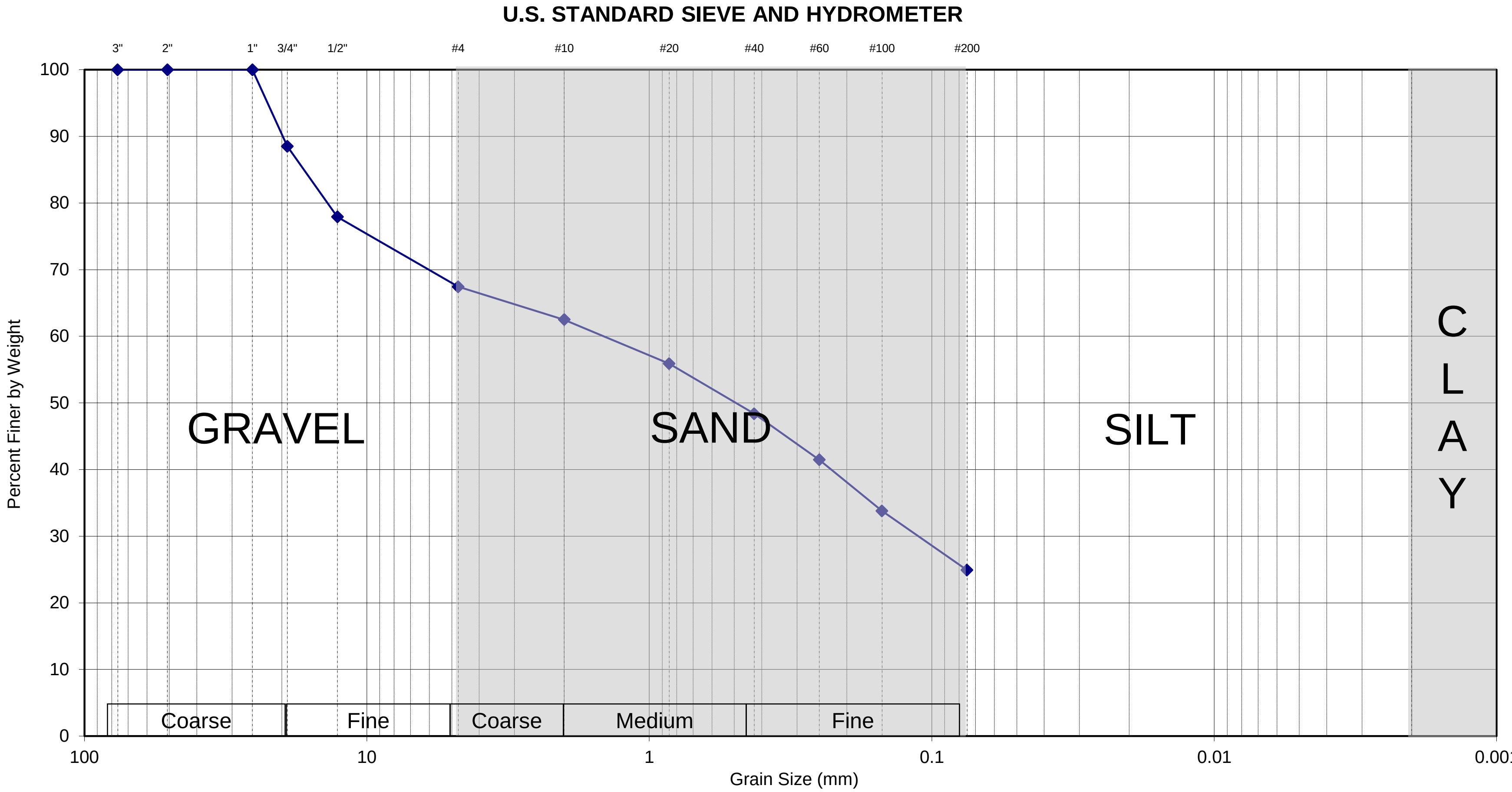
Sieve Size	% Passing
¾"	93.3
½"	82.4
#4	69.4
#10	56.4
#20	42.5
#40	32.2
#60	25.7
#100	20.0
#200	14.4

74-15-0003
Barters Island Bridge
Boothbay, ME
GZA Project # 09.0025887.00

Tested by: AS/MS Date: 6/12/15
Reviewed by: MBP Date: 6/16/15

THIELSCH ENGINEERING
195 Frances Ave., Cranston, RI 02910
401-467-6454

ASTM D422



Gravel	Sand	Fines
32.6%	42.5%	24.9%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
4	BB-BTBR-103	4D	6.4-8.2'	Brown f-c SAND, some f-c Gravel, some Silt (SM)	20.9			

Sieve Size	% Passing
¾"	88.5
½"	77.9
#4	67.4
#10	62.5
#20	55.9
#40	48.4
#60	41.5
#100	33.8
#200	24.9

74-15-0003
Barters Island Bridge
Boothbay, ME
GZA Project # 09.0025887.00

Tested by:	AS/MS	Date:	6/12/15
Reviewed by:	MBP	Date:	6/16/15

THIELSCH ENGINEERING
195 Frances Ave., Cranston, RI 02910
401-467-6454

LABORATORY TESTING DATA SHEET

Project Name Barters Island Bridge
Project No. 09.0025887.00
Project Manager J. Baron

Location Boothbay, ME
Assigned By B.Cardali
Report Date 6/16/2015

Reviewed By [signature]
Date Reviewed 6/16/2015

Boring No.	Sample No.	Depth Ft.	Lab No.	Sample Data: Moh's Hard-ness	Do in.	L in.	(1) Unit Wt. PCF	(2) Wet Density PCF	Bulk Gs.	Compression Tests: (3) Other Tests	(4) Strength PSI	(5) Strain %	(6) Conf. Stress	(7) E sec PSI EE+06	(8) Poisson's Ratio	στ PSI	Is_{50} KSI	Rock Formation or Description or Remarks
BB-BTBR-101	R-1	6.3-6.7	1		1.974	4.597	170.6			U	5,980	0.30		2.16	0.29			
BB-BTBR-102	R-13	43.5-43.9	2		1.977	4.509	171.7			U	7,193	0.14		4.51	0.18			

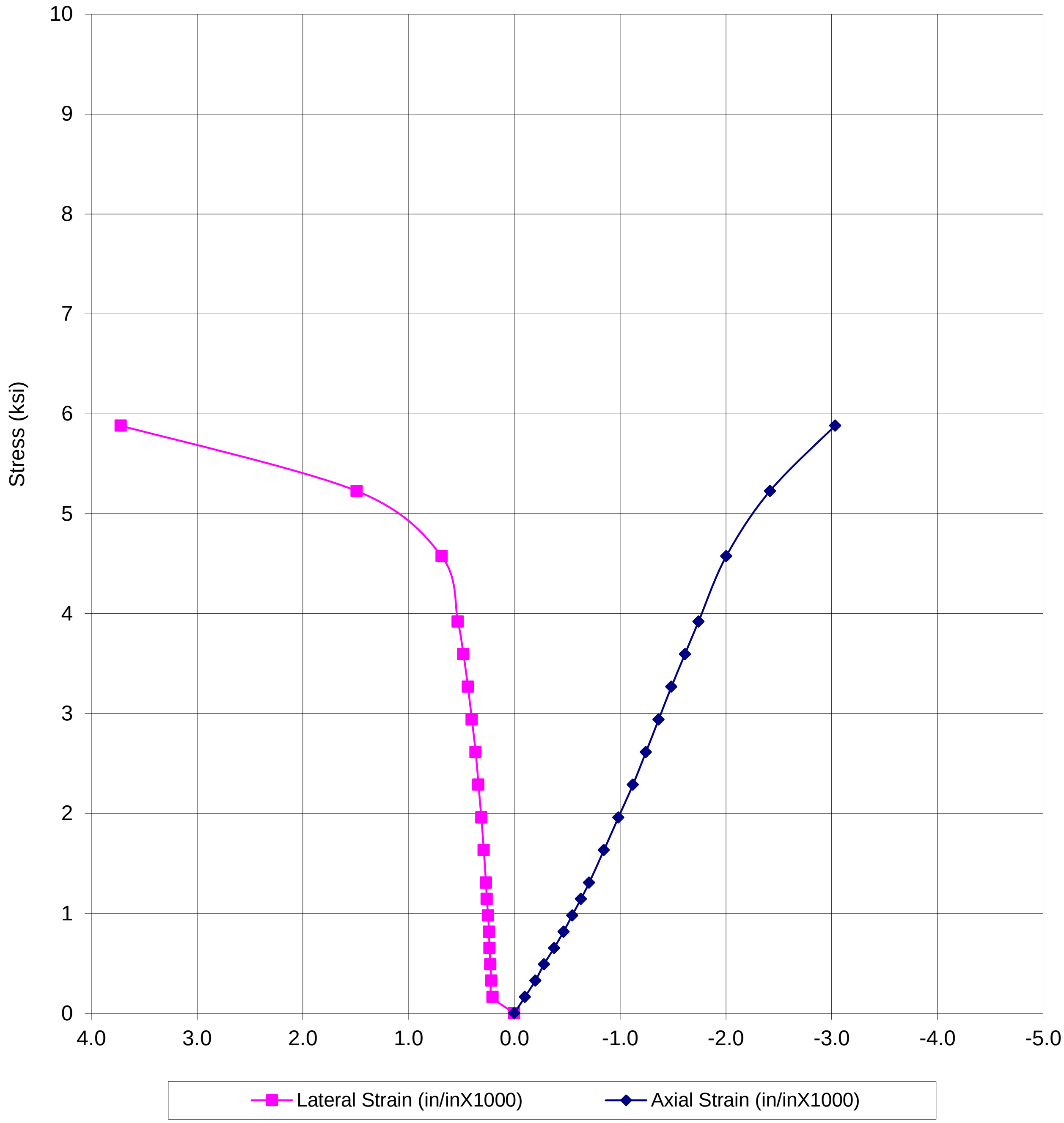
(1) Volume Determined By Measuring Dimensions
(2) Determined by Measuring Dimensions and Weight of Saturated Sample

(3) P=Petrographic PLD=Point Load (diametrical), PLA= Point Load (Axial) RST= Splitting Tensile U= Unconfined Compressive Strength
(4) Taken at Peak Deviator Stress

(5) Strain at Peak Deviator Stress
(6) Represents Confining Stress on Triaxial Tests
(7) Represents Secant Modulus at 50% of Total Failure Stress
(8) Represents Secant Poisson's Ratio at 50% of Total Failure Stress

THIELSCH ENGINEERING
195 Frances Avenue
Cranston, RI 02910
401-467-6454

Barters Island Bridge
Boothbay, ME



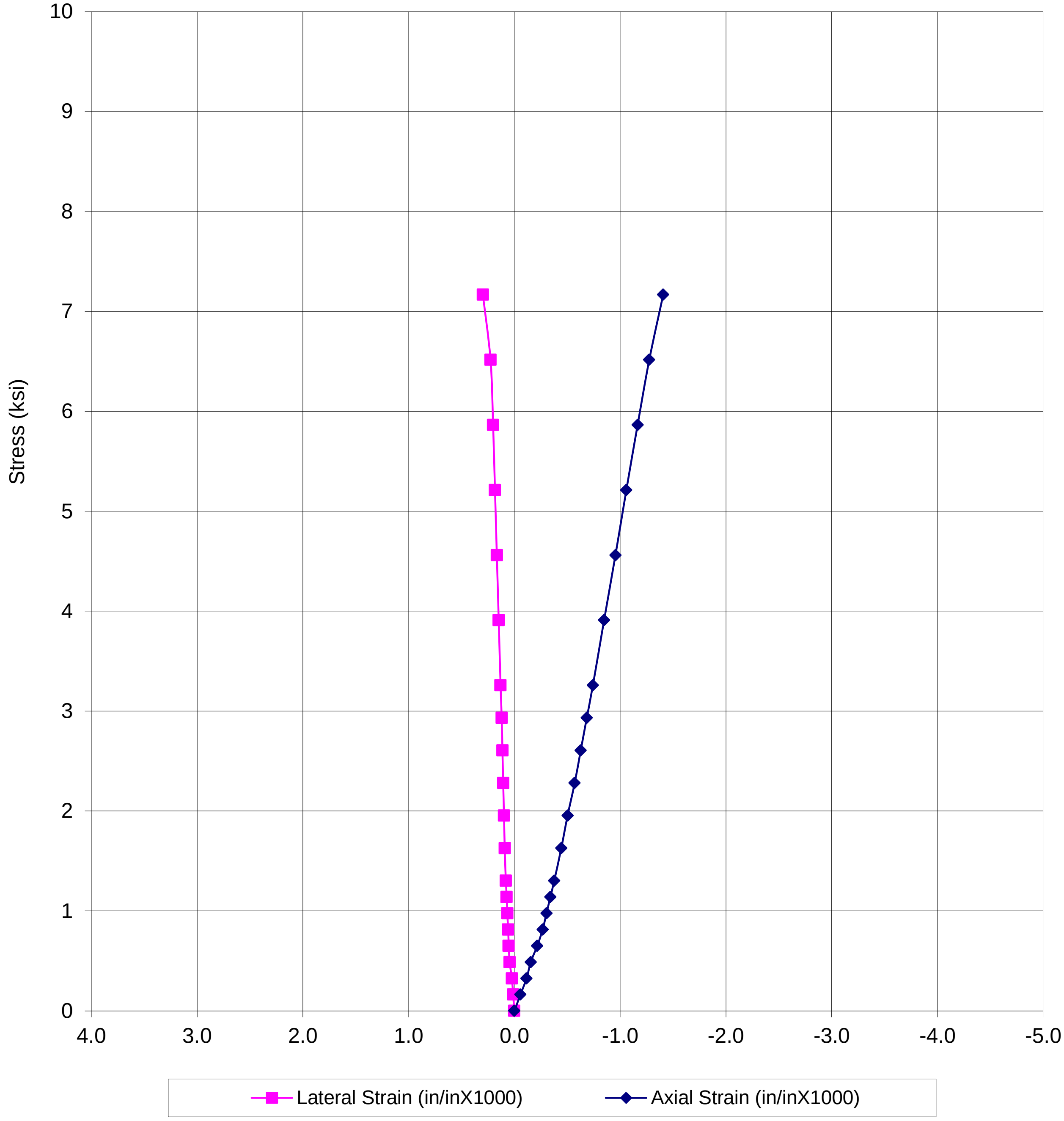
Rock Unconfined Compression Testing - ASTM D7012

Boring No. BB-BTBR-101
Sample No. R-1
Depth: 6.3-6.7'

File No. 09.0025887.00
Date: 06/11/15
Test No. U 1

THIELSCH ENGINEERING

Barters Island Bridge
Boothbay, ME



Rock Unconfined Compression Testing - ASTM D7012

Boring No. BB-BTBR-102
Sample No. R-13
Depth: 43.5-43.9'

File No. 09.0025887.00
Date: 06/11/15
Test No. U 2

THIELSCH ENGINEERING

APPENDIX E – ROCK CORE PHOTOGRAPHS

MaineDOT Barters Island
Bridge #2039 over the Back River
Boothbay, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)			Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-BTBR-101	R1	5.6	-	9.6	45	94%	26	54%	GNEISS	1
BB-BTBR-101	R2	9.6	-	13.6	51	106%	48	100%	GNEISS	1,2
BB-BTBR-101	R3	13.6	-	15.6	24	100%	24	100%	GNEISS	2
BB-BTBR-103	R1	9.0	-	12.8	45	98%	31	67%	GNEISS	3
BB-BTBR-103	R2	12.8	-	17.0	50	100%	41	82%	GNEISS	3,4
BB-BTBR-103	R3	17.0	-	19.0	24	100%	19	79%	GNEISS	4

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

MaineDOT Barters Island
Bridge #2039 over the Back River
Boothbay, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-BTBR-102	R1	5.3 - 10.3	55	92%	--	--	Masonry/Concrete	1
BB-BTBR-102	R2	10.3 - 16.4	65	89%	--	--	Masonry/Concrete	2
BB-BTBR-102	R3	16.4 - 17.6	8	57%	--	--	Masonry/Concrete	3
BB-BTBR-102	R4	17.6 - 21.1	11	26%	--	--	Masonry/Concrete	3
BB-BTBR-102	R5	21.1 - 23.1	22	92%	--	--	Masonry/Concrete	3
BB-BTBR-102	R6	23.1 - 25.8	38	119%	--	--	Masonry/Concrete	4
BB-BTBR-102	R7	25.8 - 27.0	6	43%	--	--	Masonry/Concrete	4
BB-BTBR-102	R8	27.0 - 28.8	7	35%	--	--	Masonry/Concrete	4

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

MaineDOT Barters Island
Bridge #2039 over the Back River
Boothbay, ME

Rock Core Photographs

Boring No.	Run	Depth (ft)			Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-BTBR-102	R9	28.8	-	36.1	46	52%	--	--	Masonry/Concrete	1
BB-BTBR-102	R10	36.1	-	38.2	14	56%	0	0%	GNEISS	1,2
BB-BTBR-102	R11	38.2	-	39.3	18	138%	0	0%	GNEISS	2
BB-BTBR-102	R12	39.3	-	43.5	48	96%	30	60%	GNEISS	2,3
BB-BTBR-102	R13	43.5	-	47.8	53	104%	50	98%	GNEISS	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. Top photo is dry, bottom photo is wet.
3. Transition between core runs within a row are marked by wood separators.

MaineDOT Barters Island
Bridge #2039 over the Back River
Boothbay, ME

Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-BTBR-202	R1	30.0 - 35.0	60	100%	49	82%	GNEISS	1,2
BB-BTBR-202	R2	35.0 - 40.0	60	100%	58	97%	GNEISS	2,3
BB-BTBR-203	R1	29.7 - 32.8	37	100%	12	32%	GNEISS, PEGMATITE	3
BB-BTBR-203	R2	32.8 - 33.7	10	91%	5	45%	PEGMATITE	4
BB-BTBR-203	R3	33.7 - 35.4	21	100%	4	19%	PEGMATITE	4
BB-BTBR-203 (Top 22”)	R4	35.4 - 39.9	51	96%	37	70%	PEGMATITE	4

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

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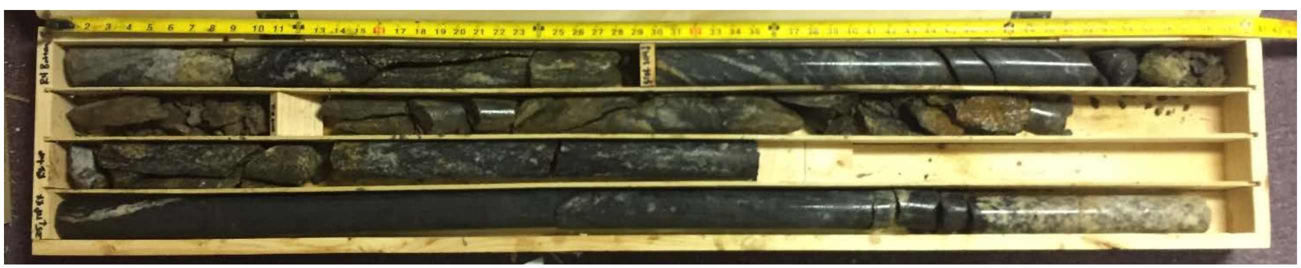
Bridge #2039 over the Back River
Boothbay, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)			Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-BTAR-203 (Bottom 30")	R4	35.4	-	39.9	51	96%	37	70%	PEGMATITE, GNEISS	1
BB-BTBR-204	R1	38.3	-	42.8	41	76%	20	37%	GNEISS, PEGMATITE	1,2
BB-BTBR-204	R2	42.8	-	45.5	32	100%	4	12%	GNEISS	2
BB-BTBR-204	R3	45.5	-	48.3	34	100%	22	65%	PEGMATITE/GNEISS	3
BB-BTBR-205	R1	4.7	-	9.7	58	96%	47	78%	GNEISS, PEGMATITE	4



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

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Rock Core Photographs

Boring No.	Run	Depth (ft)			Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-BTBR-206	R1	2.0	-	7.0	60	100%	26	43%	PEGMATITE	1
BB-BTBR-206	R2	7.0	-	12.0	60	100%	30	50%	PEGMATITE	2
BB-BTBR-207	R1	5.0	-	6.5	18	100%	0	0%	GNEISS	3
BB-BTBR-207	R2	6.5	-	8.0	18	86%	0	0%	GNEISS	3
BB-BTBR-207	R3	8.3	-	11.9	43	100%	14	33%	GNEISS	3,4
BB-BTBR-207	R4	11.9	-	13.3	14	82%	0	0%	GNEISS	4

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

MaineDOT Barters Island
Bridge #2039 over the Back River
Boothbay, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)			Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-BTBR-208	R1	14.0	-	15.3	13	100%	0	0%	GNEISS	1
BB-BTBR-208	R2	15.3	-	19	15	100%	0	0%	GNEISS	1
BB-BTBR-208	R3	19.0	-	20.3	12	100%	0	0%	GNEISS, PEGMATITE	1
BB-BTBR-208	R4	20.3	-	23.1	25	91%	5	12%	PEGMATITE	1,2
BB-BTBR-208	R5	23.1	-	26.6	42	100%	4	36%	PEGMATITE	2

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

APPENDIX F – CALCULATIONS

Objective

Assess nominal and factored bearing resistance of a foundation on rock at Central Pier location.

Methodology

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

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

References

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

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

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

1. Rock Properties

Bedrock properties were obtained from rock core specimens and logs completed for the Barter's Island Bridge Project. The following table presents the data for the Pier test boring where spread footings on bedrock will be utilized.

Run	Depth	RQD(%)	Rock Type	Joint Spacing Desc.	Corr. Spacing (in)	Apeture Desc	Corr. Apeture (in)
B101R1	5.6-9.6	54	Gneiss	Close to Moderate	2.5-24	Tight-Open	0.004-0.1
B101R2	9.6-13.6	100	Gneiss	Close to Wide	2.5-80	Tight-Open	0.004-0.1
B101R3	13.6-15.6	100	Gneiss	Moderate	8-24	Open	0.02-0.1
B102R10	36.6-38.2	0	Gneiss	Very Close to Close	0.75-8	P.Open-Open	0.01-0.1
B102R11	38.2-39.3	0	Gneiss	Close to Moderate	2.5-24	P. Open-Open	0.01-0.1
B102R12	39.3-43.5	60	Gneiss	Close to Moderate	2.5-24	Tight-P.Open	0.004-0.02
B102R13	43.5-47.8	98	Gneiss	Close to Moderate	2.5-24	Tight-P.Open	0.004-0.02
B103R1	9.0-12.8	67	Gneiss	Moderate	8-24	Tight-P.Open	0.004-0.02
B103R2	12.8-17	82	Gneiss	Close to Moderate	2.5-24	P.Open-M.wide	0.01-0.4
B103R3	17-19	79	Gneiss	Close to Moderate	2.5-24	P.Open	0.01-0.02
B202R1	1.0-6.0	82	Gneiss	Close	2.5-8	P.Open-M.wide	0.01-0.4
B202R2	6.0-11.0	97	Gneiss	Close to Moderate	2.5-24	Tight-M.wide	0.0004-0.4
B203R1	0.2-3.3	32	Gneiss	Very Close to Close	0.75-8	P.Open-M.wide	0.01-0.4
B203R2	3.3-4.2	45	Pegmatite	Very Close to Close	0.75-8	M.wide	
B203R3	4.2-5.9	19	Pegmatite	Very Close to Close	0.75-8	Open	0.02-0.1
B203R4	5.9-10.4	70	Pegmatite	Close to Moderate	2.5-24	P.Open-Open	0.01-0.1
B204R1	1.5-6.0	37	Gneiss	Close to Moderate	2.5-24	P.Open	0.01-0.02
B204R2	6.0-8.7	12	Gneiss	Very Close to Close	0.75-8	Open	0.02-0.1
B204R3	8.7-11.5	65	Gneiss	Very Close to Close	0.75-8	Tight-M.wide	0.0004-0.4

B205R1	4.7-9.7	78	Gneiss	Very Close to Close	0.75-8	P.Open-M.wide	0.01-0.4

Anticipate that 0 RQD rock will be excavated to prepare footing. Average RQD of remaining samples = 65

2. Calculation of Rock Mass Rating (RMR)

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

Parameter 1- Uniaxial Compressive Strength

Uniaxial compressive strength tests were performed on two core specimens at or in the vicinity of the Penobscot River Bridge.

Boring	Run	Depth	Rock Type	qp (ksi)
101	R1	6.3-6.7	Gneiss	5.98
102	R13	43.5-43.9	Gneiss	7.19

Select design unconfined compressive strength of 6,000 psi.

Representative unconfined compressive strength of intact rock. $\sigma_{u.r} := 6.0\text{ksi}$

$\sigma_{u.r} = 864\cdot\text{ksf}$

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_1 := 4$ for $\sigma_{u.r}$=520 to 1080 ksf

Parameter 2- Drill Core Quality

Representative RQD from table above: 65%; choose 50-75%

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_2 := 13$ for RQD = 50% to 75%

Parameter 3- Spacing of Joints

From Boring Logs, generally close = 2.5 in to 8 in

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_3 := 10$ for 2 in to 1 ft spacing

Parameter 4- Condition of Joints

From boring logs, aperture generally less than 0.1 inches and hard joint walls.

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_4 := 20$ for slightly rough surfaces, separation <0.05 in, hard joint wall rock

Parameter 5- Ground Water Conditions

Groundwater Conditions

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_5 := 4$ for Moderate water pressure. joint water pressure = 0.2 to 0.5 total vertical stress

Adjustment for joint orientation (Parameter 6)

The joint sets are generally moderate to high angle and generally rough and tight to open. Considering rock will remain embedded below bearing level and steep joints tend to compress less, joint orientation is considered fair.

From AASHTO LRFD Table 10.4.6.4-2

Relative Rating $RR_6 := -7$ for foundations - fair conditions

Total RMR Rating

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

$$RMR = 44$$

From AASHTO LRFD Table 10.4.6.4-3 RMR= 41-60 is indicative of Fair Rock Quality (Class No. 3)

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 E (Gneiss), RMR=44, using s and m values interpolated from the logarithmic trend of plotted values from AASHTO Table 10.4.6.4-4 (plots on sheet 10).

$m := .458$

$s := .00009$

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

From Wyllie "Foundations on Rock"

Eq. 5.4 Pg.138

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

Where

$C_{f1} := 1.12$ From Wyllie Table 5.4 Pg. 138 Correction factor for foundation shape for rectangular foundation:

$s = 0.00009$ For L/B>5, use factor C_{fl}=1.05,

$m = 0.46$ For L/B=2, use factor C_{fl}=1.12,

$\sigma_{u.r} = 6 \cdot ksi$ Estimate footing is roughly L/B=2, use 1.12.

Nominal Bearing Resistance

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

$q_n = 73.6 \cdot ksf$ Say 74 ksf

Factored Bearing Resistance

Bearing Resistance Factor is specified in Table 10.5.5.2.2-1

$\phi_b := 0.45$ Footing on rock

$q_R := \phi_b \cdot q_n$

$q_R = 33.1 \cdot ksf$ Say 33 ksf

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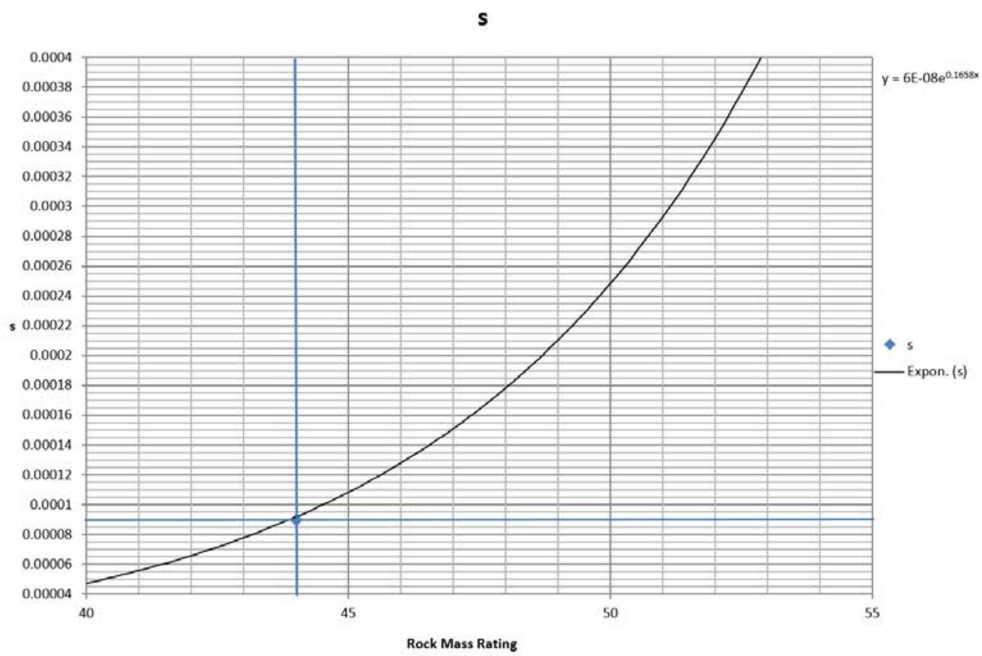

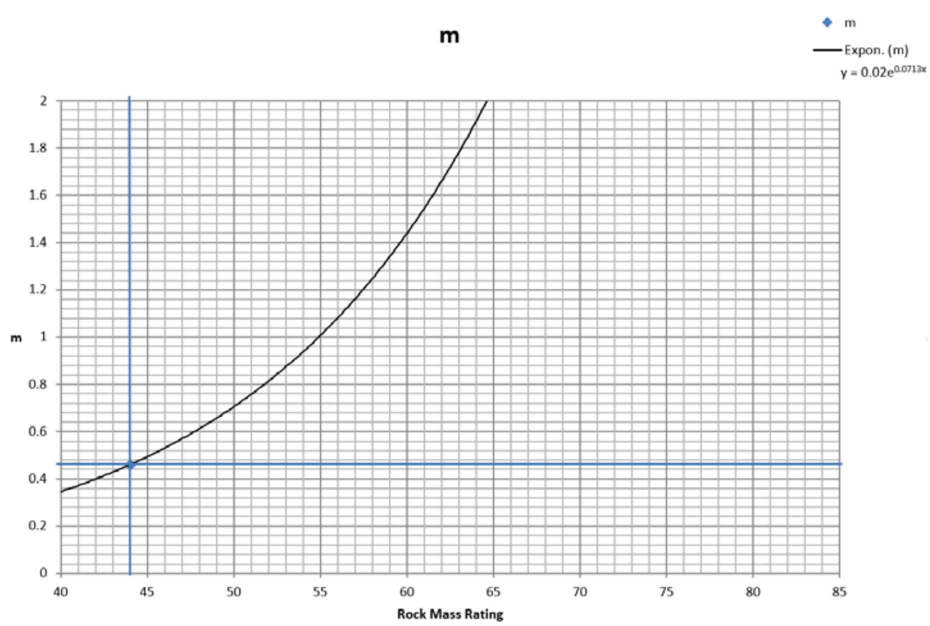

Reference:M:\FILES\GEOTECH\Design Calculations\Units v7.xmcd

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Engineers and Scientists

JOB: 09.0025887.01 Barter's Island Bridge
SUBJECT: Bearing Resistance
SHEET: 7 OF 9
CALCULATED BY: BC 08-1-17
CHECKED BY: C.Snow
REVIEWED BY: C.Snow

Table 10.4.6.4-1 Geomechanics Classification of Rock Masses.

	Parameter		Ranges of Values						
1	Strength of intact rock material	Point load strength index	>175 ksf	85–175 ksf	45–85 ksf	20–45 ksf	For this low range, uniaxial compressive test is preferred		
		Uniaxial compressive strength	>4320 ksf	2160–4320 ksf	1080–2160 ksf	520–1080 ksf	215–520 ksf	70–215 ksf	20–70 ksf
	Relative Rating		15	12	7	4	2	1	0

	Parameter					
2	Drill core quality RQD	90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%
	Relative Rating	20	17	13	8	3
3	Spacing of joints	>10 ft.	3–10 ft.	1–3 ft.	2 in.–1 ft.	<2 in.
	Relative Rating	30	25	20	10	5
4	Condition of joints	• Very rough surfaces • Not continuous • No separation • Hard joint wall rock	• Slightly rough surfaces • Separation <0.05 in. • Hard joint wall rock	• Slightly rough surfaces • Separation <0.05 in. • Soft joint wall rock	• Slicken-sided surfaces or • Gouge <0.2 in. thick or • Joints open 0.05–0.2 in. • Continuous joints	• Soft gouge >0.2 in. thick or • Joints open >0.2 in. • Continuous joints
	Relative Rating	25	20	12	6	0

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

Table 10.4.6.4-2 Geomechanics Rating Adjustment for Joint Orientations.

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

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

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

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

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