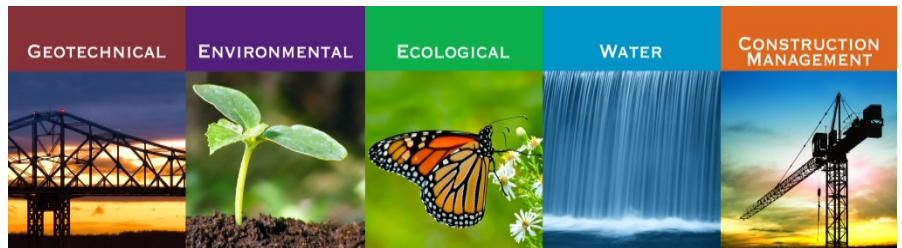




Known for excellence.
Built on trust.



GEOTECHNICAL DESIGN REPORT

CANAAN BRIDGE NO. 2120

ROUTE 2/23 OVER CARRABASSETT STREAM

MAINE DOT WIN 21878.00

CANAAN, MAINE

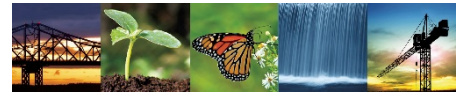
Prepared for:
Erdman Anthony
Rochester, New York

March 2021
09.0025926.01

Prepared by:
GZA GeoEnvironmental, Inc.
707 Sable Oaks Drive | Suite 150 | South Portland, Maine 04106
207.879.9190

30 Offices Nationwide
www.gza.com

Copyright© 2021 GZA GeoEnvironmental, Inc.



Known for excellence.
Built on trust.

GEOTECHNICAL
ENVIRONMENTAL
ECOLOGICAL
WATER
CONSTRUCTION
MANAGEMENT

707 Sable Oaks Drive
Suite 150
South Portland, ME 04106
T: 207.879.9190
F: 207.536.1173
www.gza.com

VIA EMAIL

March 8, 2021
File No. 09.0025926.01

Mr. Christopher Sichak, PE
Erdman Anthony
145 Culver Road, Suite 200
Rochester, New York 14620

Re: Geotechnical Design Report
Replacement of Canaan Bridge No. 2120, Route 2/23 over Carrabassett Stream
MaineDOT WIN 21878.00
Canaan, Maine

Dear Chris:

We are pleased to provide this Geotechnical Design Report (GDR) to Erdman Anthony for the replacement of Bridge No. 2120 carrying Route 2/23 over Carrabassett Stream in Canaan, Maine. Our services were provided in accordance the Subconsultant Agreement between Erdman Anthony (EA) and GZA GeoEnvironmental, Inc. (GZA) dated May 20, 2020, which incorporates GZA's proposal No. 09.P000022.21, dated April 22, 2020, and the attached *Limitations* included in **Appendix A**. GZA is providing geotechnical engineering services as a Subconsultant to EA, who is under contract with Maine Department of Transportation for design of the proposed bridge replacement.

It has been a pleasure serving Erdman Anthony 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 M. Cardali, P.E.
Assistant Project Manager

Andrew R. Blaisdell, P.E.
Consultant Reviewer



Christopher L. Snow, P.E.
Principal

BMC/CLS/ARB:erc

p:\09 jobs\0025900s\09.0025926.00 - canaan bridge replacement\09.0025926.01 - final design\report\final 25926.01 canaan bridge replacement gdr 03082021.docx

Attachment: Draft Geotechnical Report



TABLE OF CONTENTS

	<u>Page</u>
1.0 INTRODUCTION	1
1.1 BACKGROUND	1
1.2 OBJECTIVES AND SCOPE OF SERVICES	1
2.0 SUBSURFACE EXPLORATIONS	2
2.1 BORINGS	2
2.1.1 Preliminary Borings	2
2.1.2 Final Borings	2
2.1.3 Hand Probes	3
2.2 GEOPHYSICAL SURVEY	3
3.0 LABORATORY TESTING	3
3.1 SOIL	3
3.2 ROCK	4
4.0 SUBSURFACE CONDITIONS	4
4.1 SURFICIAL AND BEDROCK GEOLOGY	4
4.2 SUBSURFACE PROFILE	4
4.2.1 Weathered/Fractured Rock	5
4.2.2 Bedrock	5
4.2.3 Groundwater	6
5.0 ENGINEERING EVALUATIONS	6
5.1 GENERAL	6
5.2 APPROACH EMBANKMENTS	6
5.3 FOUNDATION DESIGN CONSIDERATIONS	7
5.3.1 Abutment Foundations	7
5.4 SEISMIC DESIGN CONSIDERATIONS	7
5.5 LOAD AND RESISTANCE FACTORS	7
5.6 SPREAD FOOTING DESIGN CONSIDERATIONS	8
5.6.1 Footing Bearing Resistance	8
5.7 ADDITIONAL FOUNDATION CONSIDERATIONS	8
5.7.1 Frost Protection	8
5.7.2 Lateral Earth Pressures	9
6.0 RECOMMENDATIONS	9
6.1 EMBANKMENT DESIGN CONSIDERATIONS	9
6.2 SEISMIC DESIGN	9
6.3 ABUTMENT AND WINGWALL DESIGN	10
6.4 RECOMMENDATIONS FOR FOUNDATIONS	10
6.4.1 Spread Footing Design	10



TABLE OF CONTENTS (*continued*)

7.0	CONSTRUCTION CONSIDERATIONS	12
7.1	SUPPORT OF EXCAVATION AND DEWATERING	12
7.2	SUBGRADE PREPARATION	12
7.3	REUSE OF ON-SITE MATERIALS	13

FIGURES

FIGURE 1	Locus Plan
FIGURE 2	Boring Location Plan & Interpretive Subsurface Profile

APPENDICES

APPENDIX A	Limitations
APPENDIX B	Boring Logs
APPENDIX C	Seismic Refraction Survey
APPENDIX D	Rock Core Photographs
APPENDIX E	Laboratory Test Results
APPENDIX F	Calculations



1.0 INTRODUCTION

This report presents the results of GZA GeoEnvironmental, Inc.'s (GZA's) geotechnical evaluation for the proposed replacement of Maine Department of Transportation (MaineDOT) Canaan Bridge No. 2120 carrying Route 2/23 over Carrabassett Stream in Canaan, Maine. Our work was completed in accordance with the Subconsultant Agreement between Erdman Anthony (EA) and GZA dated May 20, 2020, which incorporates GZA's proposal No. 09.P000022.21, dated April 22, 2020, and the attached *Limitations* included in **Appendix A** of this report.

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

1.1 BACKGROUND

The existing Canaan Bridge No. 2120 carries Route 2/23 over Carrabassett Stream in Canaan, Maine at the location shown on **Figure 1, Locus Plan**. The existing bridge was constructed in 1941 and consists of a 33-foot, single-span, concrete, rigid frame structure. The bridge is supported by spread footings bearing on bedrock. The abutments are cantilever-type with cast-in-place concrete back walls. We understand the bridge needs replacement due primarily to the poor condition of the deck and superstructure, with advanced deterioration, and the minor section loss of the concrete substructure units.

The proposed replacement bridge will consist of an approximately 60-foot-long, 37-foot-wide, single-span bridge, supported by semi-integral abutments. The bridge will be constructed on the existing alignment. It is anticipated that the abutments will be constructed on spread footings bearing directly on bedrock. The Route 2/23 approach roadway elevation is proposed to be within about 1 foot of existing grades.

1.2 OBJECTIVES AND SCOPE OF SERVICES

The objectives of our work were to evaluate subsurface conditions and provide final geotechnical design recommendations and construction considerations for bridge replacement. 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 preliminary and final subsurface exploration programs, consisting of a total of four test borings, two hand probes, and four seismic refraction lines, to evaluate subsurface conditions;
- Conducted a laboratory testing program to evaluate classification and engineering properties of the site soil and bedrock;
- Conducted geotechnical engineering analyses for soil and bedrock properties; stability and settlement of approach embankments; frost susceptibility and drainage of approach embankments; AASHTO LRFD load and resistance factors associated with geotechnical design elements; nominal resistance of spread footings bearing on rock; and seismic design considerations;



- Developed geotechnical engineering recommendations including foundation design recommendations for spread footings, lateral earth pressures and seismic design parameters; and
- Prepared this report summarizing our findings and design recommendations.

2.0 SUBSURFACE EXPLORATIONS

2.1 BORINGS

GZA completed a preliminary exploration program in 2017 consisting of two borings, designated as BB-CCS-101 and -102, a final design exploration program in 2020 consisting of two borings designated as BB-CCS-201 and -202, and two hand probes designated as HP-CCS-201 and -202. The approximate as-drilled 100-series boring locations were estimated by tape measurement from existing bridge components and the as-drilled 200-series boring and probe locations and elevations were surveyed by MaineDOT and provided via electronic file (borings.dgn) and are shown on **Figure 2**.

New England Boring Contractors of Hermon, Maine provided drilling services and coordinated utility clearance. All 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. Bedrock cores were obtained using NX coring equipment. Photographic logs of the recovered rock core specimens are included in **Appendix D**. GZA personnel monitored the drilling work and prepared logs of each boring that are included in **Appendix B**. Elevations referenced in this report are in feet and refer to the National American Vertical Datum of 1988. Additional details of each program are described below.

2.1.1 Preliminary Borings

GZA completed BB-CCS-101 and BB-CCS-102 on March 2, 2017. The borings were drilled in the westbound lanes/shoulder behind each existing abutment. The borings were drilled using a truck-mounted drill rig and backfilled with cuttings, crushed stone, and asphalt cold patch. SPT sampling was completed using a safety hammer operated by a rope and cathead.

The borings were drilled to depths of approximately 20.4 to 29.5 feet below ground surface. Both borings were cored 10-plus feet into bedrock.

2.1.2 Final Borings

GZA completed BB-CCS-201 and BB-CCS-202 on June 8, 2020. The borings were drilled on the southern side of the eastbound lane behind each existing abutment. The borings were drilled using a track-mounted drill rig and backfilled with sand. SPT sampling was completed with automatic hammer NEBC Drill Rig No. 23, which had a rated hammer efficiency factor of 0.842 at the time of drilling.

The borings were drilled to depths of approximately 16 to 26.5 feet below ground surface (bgs) and cored 10-plus feet into bedrock.



2.1.3 Hand Probes

GZA conducted two hand probes (HP-CCS-201 and HP-CCS-202) on June 8, 2020. The hand probes were completed using a nominal 1/2-inch-diameter hand probe driven with a sledgehammer to refusal. The probes were located on the embankment slopes between the 200-series borings and the stream bed to evaluate the top of rock profile adjacent to the existing bridge. Refusals were encountered between 2.6 and 2.8 feet bgs.

2.2 GEOPHYSICAL SURVEY

Northeast Geophysical Services (NGS) completed a seismic refraction survey to assess possible bedrock depth profiles in the area between the -100 and -200 series borings where the overhead lines limited access for the drill rig. Four, approximately 92-foot-long seismic refractions lines, identified as L-1 through L-4, were completed on the south of Route 2. The locations of the beginning and end of each seismic refraction line were surveyed by a MaineDOT crew and provided to GZA as an electronic data file and are shown on **Figure 2**. The surveyed locations were shared with NGS for profile development. The seismic data were acquired via a Geometrics Geode, 24-channel seismograph. The seismic refraction surveys measure the travel time of sound waves from a 16-lb hammer and a metal plate to refract waves at subsurface layers and differentiate material compressive wave velocities. Each seismic line contained 24 geophones and included six to seven impacts from the hammer at various locations.

The recorded field data was interpreted using the Hobson-Overton method. The results identified two layers, a dry soil and bedrock. NGS indicated that the bedrock velocity values at this site are lower than typical bedrock values in Maine which could represent a weak or highly fractured zone of bedrock.

The results of the seismic refraction survey are documented in a report prepared by NGS dated August 17, 2020, which is presented in **Appendix C**. The report provides additional details on the locations and interpretations.

3.0 **LABORATORY TESTING**

GZA retained Thielsch Engineering's Geotechnical Laboratory in Cranston, Rhode Island to complete a soil testing program to assess the gradation and engineering characteristics of the soil; and GeoTesting of Acton, Massachusetts to perform the bedrock testing to evaluate the strength of the bedrock. The testing program consisted of the following:

3.1 SOIL

- Eight (8) gradation analysis / AASHTO Classification / Unified Soil Classification System / Frost Classification assessments; and
- Eight (8) moisture content tests.



3.2 ROCK

- Two (2) unconfined compressive strength / secant modulus tests.

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

4.0 SUBSURFACE CONDITIONS

4.1 SURFICIAL AND BEDROCK GEOLOGY

Surficial geologic units mapped¹ in the area include fine to coarse sand and gravel Eskers, and gravelly sand Glacial Till deposits.

Bedrock at the site is mapped as slate, schist and marble of the Silurian Sangerville Formation, based on the available the bedrock geologic data².

4.2 SUBSURFACE PROFILE

Two soil units were encountered beneath surficial pavement or topsoil in the test borings: Fill and Glacial Till. Approximately 8 inches of asphalt pavement was encountered in the borings drilled through the approaches. The approximate thicknesses and generalized descriptions of the subsurface units are presented in the following table, in descending order from existing ground surface. Detailed descriptions of the materials encountered at specific locations are provided in the boring logs in **Appendix B**. The subsurface conditions are also shown in relation to the bridge alignment on the interpretive subsurface profile on **Figure 2**.

¹ Hanson, Lindley S. and Caldwell, Dabney W., 1986, Reconnaissance surficial geology of the Skowhegan [15-minute] quadrangle, Maine: Maine Geological Survey, Open-File Map 86-38, map, scale 1:62,500. Maine Geological Survey Maps. 625. http://digitalmaine.com/mgs_maps/625

² Ludman, Allan, 1977, Geologic map of the Skowhegan [15-minute] quadrangle, Maine: Maine Geological Survey, Geologic Map GM-5, 25 p. report, color map, cross section, scale 1:62,500. Maine Geological Survey Maps. 295. http://digitalmaine.com/mgs_maps/295



GENERALIZED SUBSURFACE CONDITIONS		
Soil Unit	Approximate Encountered Thickness (ft)	Generalized Description
Fill	4 to 9	Variable ranging from: Grey to Dark Brown, very loose to very dense, fine to coarse SAND, trace to some Gravel, little Silt <u>to</u> : silty fine to coarse SAND, little to some Gravel. (USCS: SM). <ul style="list-style-type: none">• MaineDOT Frost Classification = I-III• Encountered in all borings• Possible cobbles and boulders
Glacial Till	2 to 8	Variable ranging from: Grey/Brown, medium dense to very dense, angular GRAVEL, trace to some fine to coarse Sand, trace Silt <u>to</u> : gravelly fine to coarse SAND, little Silt (gravel material moderately weathered) (USCS: SM). <ul style="list-style-type: none">• MaineDOT Frost Classification = 0-II• Encountered in all borings
Weathered/ Fractured Rock	0.9	Dark grey, very dense, angular GRAVEL, trace silt (remnant bedding evident). <ul style="list-style-type: none">• Encountered in Boring BB-CCS-202
Encountered Top of Bedrock Elevation	Abutment 1: Approx. El. 221 to El. 218 Abutment 2: Approx. El. 220 to 226	

4.2.1 Weathered/Fractured Rock

Weathered/fractured rock descriptions and encountered locations are described in the table above. The glacial till appeared to transition into a very dense weathered/fractured rock layer. The split spoon driven within the stratum met refusal at 5 inches in boring BB-CCS-202. The recovered weathered/fractured rock in the spoon had remnant bedrock structure, but was easily broken with a field knife or by hand. Some harder rock pieces fractured into gravel were collected in the sample and increases in resistance during the advancement of the roller cone showed variable hardness within the stratum. The transition from weathered/fractured rock to rock was interpreted to occur at the depth of split spoon refusal.

The seismic refraction survey identified potential weathered/fractured rock zone between approximately EL. 223 and EL. 228 in the area of BB-CCS-202.

4.2.2 Bedrock

GZA evaluated the top of bedrock based on observations made by GZA's field engineer. The top of bedrock was defined based on split-spoon refusal and drill performance during advancement of roller bit, casing refusal, or rock coring.

Bedrock was cored in all test borings and was described as moderately hard to hard, fresh to slightly weathered, aphanitic, grey, SLATE. Joints are extremely close to moderately spaced, moderately dipping to vertical, planar to undulating, rough to smooth, fresh, and tight to partially open. The Rock Quality Designation (RQD) of the Slate ranged from 0 to 87 percent. Photographic logs of the recovered rock core specimens are included in **Appendix D**.

The top of bedrock elevations were found to be variable at the abutment locations, appearing to slope down moderately to the south and down steeply from both abutments toward the center of Carrabassett Stream. The average inclination of the bedrock surface from both abutments toward the center of the



stream appears to be approximately 1.5H:1V to 2H:1V. The actual inclination of the bedrock surface between explorations is unknown, but it is likely stepped with steeper and flatter portions, rather than following a consistent inclination.

Unconfined compressive strength 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 Ground (ft bgs)	Depth below Top of Rock (ft bgs)	Elevation (ft NAVD 88)	Unconfined Compressive Strength (psi)	Secant Modulus @ 50% of Failure Stress (ksi)	Unit Weight (pcf)	Rock Type
BB-CCS-101	21.9	4.4	212.8	9,022	2,900	188	SLATE
BB-CCS-102	14.4	4.8	220.7	17,786	8,370	179	SLATE

4.2.3 Groundwater

Groundwater was encountered at depths of approximately 9.2 to 7.9 feet bgs in BB-CCS-101 and -102, respectively. These depths correspond to approximately El. 225.5 to 227.2. Groundwater was not observed in the BB-CCS-201 or -202. Water levels were measured in completed boreholes within about 20 minutes after completion of drilling. The groundwater observations were made at the times and under the conditions stated on the boring logs. Fluctuations in groundwater levels will occur due to variations in season, precipitation, river level and other factors. Consequently, water levels during and after construction are likely to vary from those encountered in the borings at the time the observations were made.

5.0 ENGINEERING EVALUATIONS

5.1 GENERAL

GZA conducted geotechnical engineering evaluations in accordance with *2020 AASHTO LRFD Bridge Design Specifications, 9th Edition*, with Interims (herein designated as AASHTO), the *2011 AASHTO Guide Specifications for LRFD Bridge Design*, and the *MaineDOT Bridge Design Guide*, with revisions through 2018 (MaineDOT BDG). The sections that follow describe the evaluations and the geotechnical basis for each element. Supporting calculations developed by GZA for the project are attached in **Appendix F** of this report.

5.2 APPROACH EMBANKMENTS

The proposed bridge replacement is to be constructed on or close to the existing horizontal and vertical alignments. Grading within the limits of the existing roadway will be limited to minor cuts and fills on the order of 1 foot or less. Widening is not anticipated in the vicinity of the bridge except for an area near Easy Street where an existing structure will be removed. Slopes for the project will be 2H:1V or flatter.



The subsurface conditions beneath the approaches include fill and glacial till over bedrock. Therefore, it is our judgement that post-construction embankment settlement and global stability are not concerns for the project.

5.3 FOUNDATION DESIGN CONSIDERATIONS

5.3.1 Abutment Foundations

Spread footings bearing on bedrock were selected as the preferred foundation alternative during preliminary design. Recommendations for spread footing design are provided in **Section 6.4.1**.

Assessment of the foundation types was influenced by thickness and strength of soil, and depth to and strength of the bedrock. Subsurface conditions relevant to foundation type considerations are summarized in the table below.

Boring/Hand Probe	Station	Offset (ft)	Approximate Top of Rock Depth (ft)	Approximate Top of Rock Elevation (ft)	Substructure
BB-CCS-101	572+99.1	13.1 Lt.	17.5	217.2	Abutment 1
BB-CCS-201	572+64.2	29.8 Rt.	6.0	228.5	Abutment 1
HP-CCS-201	572+92.0	30.4 Rt.	2.6	221.9	Abutment 1
BB-CCS-102	573+51.3	12.4 Lt.	9.6	225.5	Abutment 2
BB-CCS-202	573+70.1	32.9 Rt.	15.4	220.7	Abutment 2
HP-CCS-202	573+45.1	31.4 Rt.	2.8	223.8	Abutment 2

5.4 SEISMIC DESIGN CONSIDERATIONS

Seismic site class was determined in general accordance with LRFD Table C3.10.3.1. Considering the bridge will be supported by spread footings bearing directly on bedrock, the bridge is assigned to Site Class B.

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

5.5 LOAD AND RESISTANCE FACTORS

AASHTO LRFD load factors should be applied to horizontal earth pressure (EH), vertical earth pressure (EV), earth surcharge (ES), and live load surcharge (LS) loads, using the load factors for permanent loads (γ_p) provided in LRFD Table 3.4.1-2 for strength limit state foundation design. Load factors are not provided for passive earth pressure because this is considered a resistance in AASHTO LRFD. A load factor of 1.5 may be applied to the passive soil reaction used to design the integral backwall (end diaphragm) to account for deformation of the backwall into the soil as a result of thermal expansion of the integral bridge deck, consistent with the load factor provided for active earth pressure in AASHTO Table 3.4.1-2.

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



GEOTECHNICAL RESISTANCE FACTORS – STRENGTH LIMIT STATE			
Foundation Resistance Type	Method/Condition	Resistance Factor (ϕ)	AASHTO Reference
Bearing	Footing on Rock	0.45	10.5.5.2.2-1
Sliding	Footing on Rock	0.8	10.5.5.2.2-1

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

5.6 SPREAD FOOTING DESIGN CONSIDERATIONS

5.6.1 Footing Bearing Resistance

Nominal and factored bearing resistances were calculated for bedrock-bearing footings using the Rock Mass Rating- (RMR-) based empirical correlation presented in “Foundations on Rock,” by Duncan Wyllie. RMR was evaluated in accordance with Table 10.4.6.4-1 of the 2012 AASHTO LRFD Bridge Design Specifications, 6th Edition (AASHTO). The current (9th) Edition of the *AASHTO Design Specifications* does not include the RMR formulation included in the previous version (6th Edition). However, Articles C10.4.6.4 and 10.6.2.6.2 of the 8th Edition refer to RMR-based design procedures for footings on rock, so the 6th Edition methodology was followed.

GZA used bedrock data obtained in test borings drilled at or near the proposed abutments to develop foundation design parameters at the abutment locations. The bedrock properties used in the bearing resistance evaluation are presented below:

DESIGN BEDROCK PROPERTIES FOR BEARING RESISTANCE EVALUATION					
Rock Type	RQD (percent)	Unconfined Compressive Strength (ksi)	Rock Mass Rating (RMR)	m	s
Slate	37	9.0	42	0.16	0.000063

Based on these parameters, the calculated nominal bearing resistance is 58 kips per square foot (ksf), resulting in a factored bearing resistance of 26 ksf for the strength limit state. Supporting calculations are provided in **Appendix F**.

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

5.7 ADDITIONAL FOUNDATION CONSIDERATIONS

5.7.1 Frost Protection

Based on the MaineDOT BDG, Section 5.2.1, the Freezing Index for the site is 1,740, and with low-moisture content (<10 percent) soils, the estimated depth of frost penetration is approximately 7.4 feet. Where abutment foundations bear directly on sound rock, there is no minimum requirement for footing embedment.



Granular fill soils encountered near the surface at the abutments typically were classified as AASHTO A-1-b, A-4 and A-2-4(0) with MaineDOT Frost Classification from I to III, indicating they are considered to exhibit low to moderate frost susceptibility. Since there was no evidence of significant pavement distress or heave, these materials are judged to be suitable for continued use beneath the approach roadway after reconstruction. In accordance with MaineDOT Standards, new backfill placed behind abutments will consist of non-frost-susceptible materials.

5.7.2 Lateral Earth Pressures

The material properties will be controlled by the backfill material, which is proposed to consist of BDG Type 4 soil. In accordance with the requirements of the BDG Section 5.4.3, the semi-integral abutments and wingwalls will be free to rotate and therefore should be designed for active earth pressure. For the planned abutment with footing bearing on rock, an active earth pressure coefficient, $K_a=0.31$, is recommended.

Thermal expansion of the bridge will cause the superstructure backwall (end diaphragm) to move toward the backfill, which will result in earth pressures ranging from at-rest to passive earth pressure. Therefore, the superstructure backwall should be designed for full passive pressure. A Coulomb passive earth pressure coefficient, $K_{pc}=6.73$, is recommended. Developing full passive pressure assumes that the ratio of lateral movement to backwall height (y/H) exceeds 0.005. If the calculated displacements are significantly less than that required to develop full passive pressure, the designer may consider using Rankine theory passive earth pressure coefficient, $K_{pr}=3.25$.

Design lateral earth pressure recommendations are provided in **Section 6.3** of this report.

6.0 RECOMMENDATIONS

6.1 EMBANKMENT DESIGN CONSIDERATIONS

Embankment side slopes that are not riprap-covered should be designed with MaineDOT-typical slope angles of 2H:1V or flatter. Soil slopes should be provided with loam and seed for permanent erosion protection. Steeper slopes should be covered with riprap. Riprap should also be provided where the embankment side slopes will be near or below typical water levels, to protect from scour.

6.2 SEISMIC DESIGN

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



SITE CLASS B SEISMIC DESIGN PARAMETERS	
Parameter	Design Value
F _{pga}	1.0
F _a	1.0
F _v	1.0
A _s (Period = 0.0 sec)	0.075 g
SD _s (Period = 0.2 sec)	0.159 g
SD ₁ (Period = 1.0 sec)	0.047 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 apply as specified in AASHTO Articles 4.7.4.4 and 3.10.9.

6.3 ABUTMENT AND WINGWALL DESIGN

- Abutment backfill should consist of MaineDOT 703.19 Granular Borrow for Underwater Backfill, MaineDOT BDG Type 4 soil. Recommended soil properties for Type 4 soils are as follows:
 - Internal Friction Angle of Soil = 32°
 - Soil Total Unit Weight = 125 pcf
 - Coefficient of Passive Earth Pressure, K_p (use for design of end diaphragms):
 - For a ratio of lateral movement to backwall height (y/H) equal to or exceeding 0.005, use Coloumb theory coefficient, $K_p = 6.73$;
 - For a ratio of y/H significantly less than 0.005, use Rankine theory coefficient, $K_p = 3.25$;
 - Coefficient of Active Earth Pressure, $K_a = 0.31$ (use for design of abutments and wingwalls):
- Live load surcharge should be applied as a uniform lateral surcharge pressure using the equivalent fill height (H_{eq}) values developed in accordance with LRFD Section 3.11.6.4, based on the abutment/wingwall height and distance from the wall backface to the edge of traffic. A minimum H_{eq} of 2 feet is recommended.
- Foundation drainage should be provided in accordance with Section 5.4.1.9 of the MaineDOT BDG. We recommend the use of French drains on the uphill side of abutments and wing walls to prevent buildup of differential hydrostatic pressure. The drains should be sloped to drain by gravity and should outlet through a series of 4-inch-diameter weep holes, spaced approximately 10 feet center-to-center.

6.4 RECOMMENDATIONS FOR FOUNDATIONS

6.4.1 Spread Footing Design

- The proposed abutments should be supported on spread footing foundations bearing on sound, intact bedrock. Footings designed to bear on intact bedrock should be designed using a nominal bearing resistance, q_n , of 58 ksf. At the strength limit state, footings should be designed for a maximum factored bearing resistance of 26 ksf. A bearing resistance of 26 ksf should be used for service limit state design.



- Spread footings founded on bedrock should be checked for eccentricity with AASHTO Article 10.6.3.3. Eccentricity of the footing reaction at the strength limit state should be limited such that the resultant reaction on the base of the footing is no further than 0.45 B from the centerline of the footing, where B is the footing width perpendicular to the axis of rotation.
- The base resistance against sliding may be based on NAVFAC DM7.02-63, Table 1, which indicates the sliding resistance coefficient ($\tan \delta$) is equal to 0.7 for cast-in-place concrete on sound rock. Therefore, the nominal sliding resistance between footings and bedrock subgrades is equal to the vertical force multiplied by 0.7. The factored sliding resistance coefficient is 0.56 for Strength Limit State.
- The bedrock surface should be cleaned of loose soil or rock at the time of concrete placement for subfooting concrete or the footing. Bearing surface preparation should be in accordance with **Section 7.2**.
- The following table summarizes the top of bedrock elevations encountered in the borings. This data, combined with the interpreted subsurface profile shown in **Figure 2**, is provided to assist the designer in developing bottom-of-footing elevations for the abutments.

ESTIMATED BEDROCK LEVELS FOR FOOTING DESIGN	
Foundation Element	Estimated Range in Bedrock Elevation (feet, NAVD 88)
Abutment 1	El. 217 to 225
Abutment 2	El. 217 to 226

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

- If the bedrock level extends above the design bottom of footing elevation, the footing may be raised and vertical reinforcement shortened in the wall, subject to review and approval of the Designer to limit over-excavation of bedrock.
- If the exposed bedrock surface after cleaning is below the design footing bearing level, fill concrete may be placed up to the bottom of footing level.
- Concrete used for fill concrete beneath footings and for footings should consist of Class A Concrete in accordance with MaineDOT Standard Specification Section 502.05.
- Anchoring, doweling, benching or other means of improving sliding resistance are recommended at locations where the prepared bedrock surface is steeper than 4H:1V in any direction.
- Rock dowels may be used to supplement the sliding resistance for the footing to resist design lateral loads. If used, the dowels should be grouted a minimum of 2 feet into intact bedrock and embedded at least 1.5 feet into concrete. The unconfined compressive strength of the bedrock should be assumed to be 9.0 ksi for design of rock dowels.
- Dowels should be grouted with a cementitious grout on the MaineDOT Qualified Products List of Grout Materials for Keyways and Anchoring (pre-qualified for anchoring). Epoxy grout should not be used.



- Since the footings will be founded on bedrock, there is no minimum embedment required for frost protection per BDG Article 5.2.1.
- Existing substructures should be completely removed prior to new foundation construction where they interfere with new foundations.

7.0 CONSTRUCTION CONSIDERATIONS

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

7.1 SUPPORT OF EXCAVATION AND DEWATERING

Excavations for abutment foundations will extend up to about 10 to 18 feet below existing grade to expose bedrock. The anticipated bedrock surface elevation ranges from approximately El. 221 to 225 at Abutment 1 and El. 217 to 227 at Abutment 2, corresponding to depths of approximately 2 to 3 feet below the Q1.1 (El. 220.4) at Abutment 1, and approximately at or 5 to 6 feet above Q1.1 at Abutment 2. Sandbags with a membrane are likely to be a suitable flow diversion system at this site for the anticipated relatively shallow water depths.

Sloped open cut excavation techniques are considered feasible between the abutments and the approach fills. Depending on the sequencing and construction durations, temporary lateral support systems such as braced steel sheet piling may be necessary to provide grade separation between the temporary detour roadway and the new footing excavations.

The contractor should be responsible for design of all temporary support of excavation. In all cases, temporary excavations should comply with Occupational Safety and Health Administration excavation safety requirements.

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

7.2 SUBGRADE PREPARATION

We anticipate it will be feasible to complete final bedrock subgrade preparation in-the-dry. The bedrock surface is known to be variable in terms of elevation, slope and localized weathering. Conventional excavation equipment is anticipated to be sufficient to complete excavations. All soil and loose, decomposed, highly-weathered and fractured bedrock should be removed from the footing bearing surface prior to placement of subfootings or footings.

The prepared bearing surfaces should be observed by the geotechnical engineer prior to placing concrete. The Geotechnical Engineer and Designer should also be provided cross-sections showing the prepared rock surface geometry prior to placement of concrete to evaluate whether benching, doweling, or subfooting concrete fill are needed for that foundation location. If the exposed bedrock surface is steeper



than 4H:1V, then anchoring, doweling, benching or other means should be employed to improve sliding resistance.

7.3 REUSE OF ON-SITE MATERIALS

Based on the test boring results, none of the six fill samples tested had less than 10 percent passing the No. 200 sieve, indicating the fill will not meet MaineDOT specifications for Granular Borrow and/or Granular Borrow for Underwater Backfill. The material is considered suitable for use as Common Borrow.

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



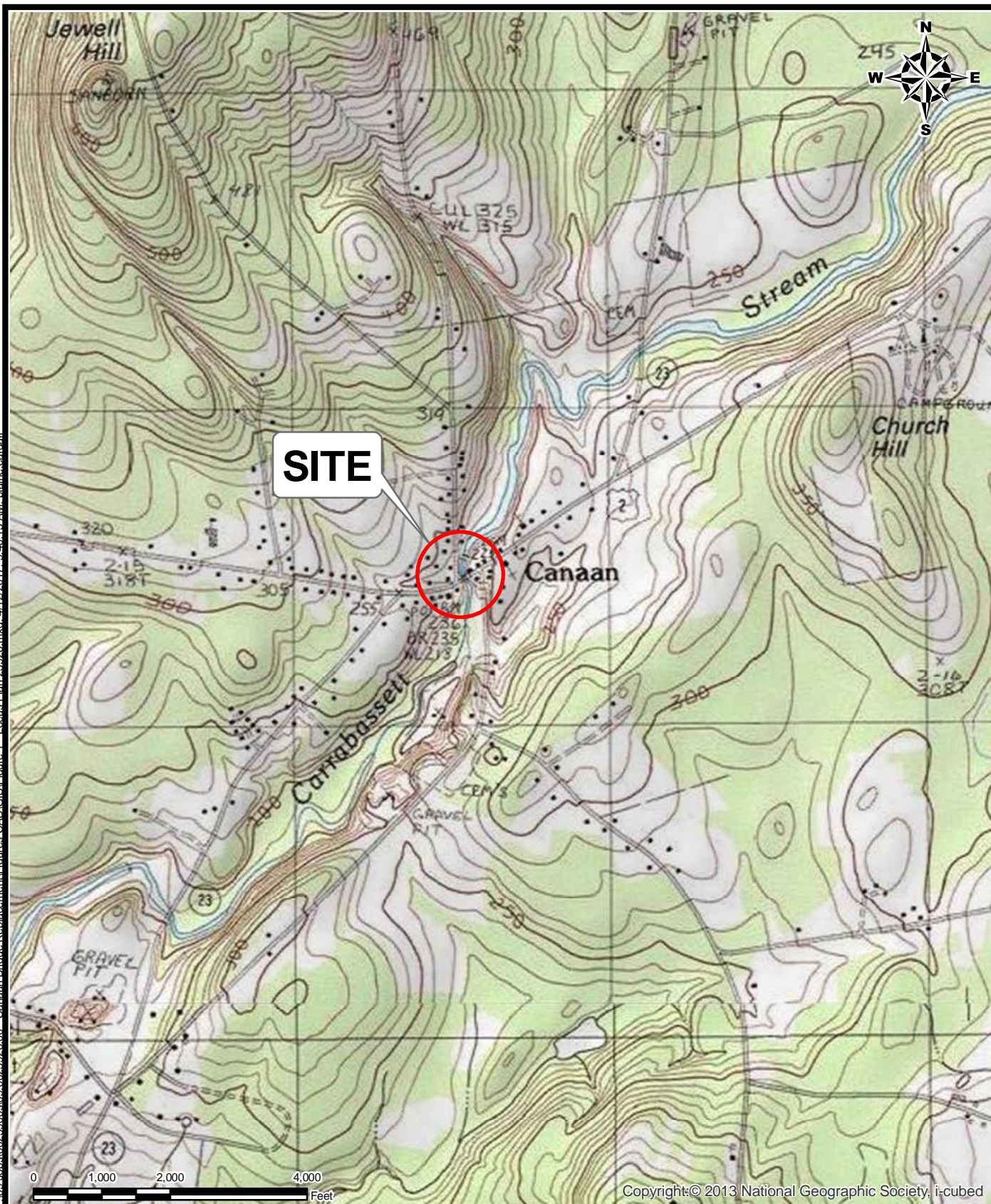
03/08/21

**CANAAN BRIDGE NO. 2120 OVER CARRABASSETT STREAM
GEOTECHNICAL DESIGN REPORT**

09.0025926.01

FIGURES

© 2017 - GZA GeoEnvironmental, Inc. P:\09 Jobs\0025900s\09.0025926.mxd - Canaan Bridge Replacement\Figures-CAD\GIS\Figure 1 - Locus Plan 25926.mxd 4/12/2017 9:40:15 AM blaine.ccardelli



UNLESS SPECIFICALLY STATED BY WRITTEN AGREEMENT, THIS DRAWING IS THE SOLE PROPERTY OF GZA GEOENVIRONMENTAL, INC. (GZA). THE INFORMATION SHOWN ON THE DRAWING IS SOLELY FOR THE USE BY GZA'S CLIENT OR THE CLIENT'S DESIGNATED REPRESENTATIVE FOR THE SPECIFIC PROJECT AND LOCATION IDENTIFIED ON THE DRAWING. THE DRAWING SHALL NOT BE TRANSFERRED, REUSED, COPIED, OR ALTERED IN ANY MANNER FOR USE AT ANY OTHER LOCATION OR FOR ANY OTHER PURPOSE WITHOUT THE PRIOR WRITTEN CONSENT OF GZA. ANY TRANSFER, REUSE, OR MODIFICATION TO THE DRAWING BY THE CLIENT OR OTHERS, WITHOUT THE PRIOR WRITTEN EXPRESS CONSENT OF GZA, WILL BE AT THE USER'S SOLE RISK AND WITHOUT ANY RISK OR LIABILITY TO GZA.

SOURCE: THIS MAP CONTAINS THE ESRI/ARC GIS ONLINE USA TOPOGRAPHIC MAP SERVICE, PUBLISHED DECEMBER 12, 2009 BY ESRI/ARC GIS 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

CANAAN BRIDGE #2120 OVER CARABASSET STREAM
CANAAN, MAINE
WIN 21878.00

PREPARED BY:
 **GZA GeoEnvironmental, Inc.**
Engineers and Scientists
www.gza.com

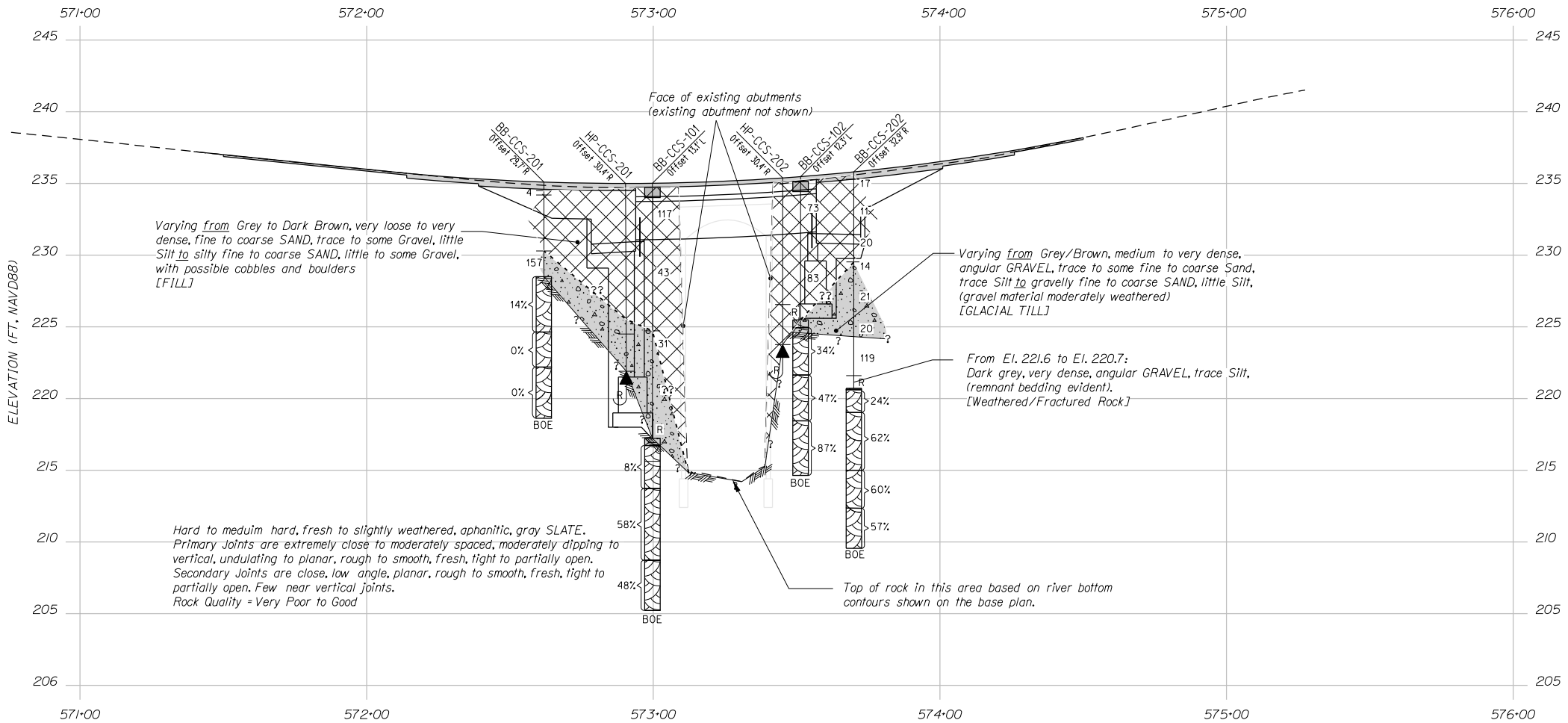
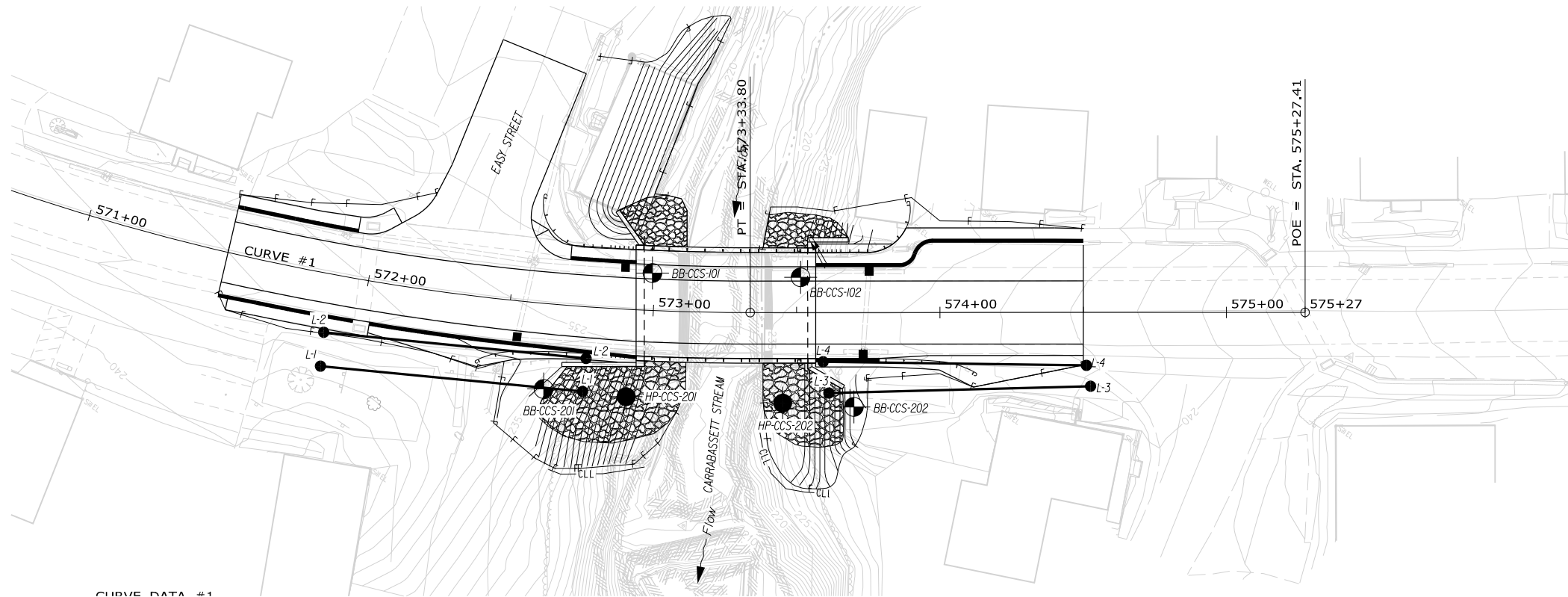
PREPARED FOR:

ERDMAN ANTHONY

LOCUS PLAN

PROJ MGR:	CLS	REVIEWED BY:	CLS	CHECKED BY:	ARB
DESIGNED BY:	BMC	DRAWN BY:	ADM	SCALE:	1 in = 2,000 ft
DATE:	OCTOBER 2020	PROJECT NO.	09.0025926.01	REVISION NO.	

FIGURE
1



PROFILE

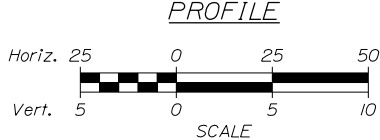
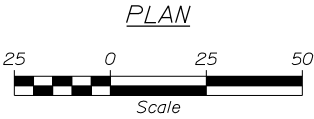
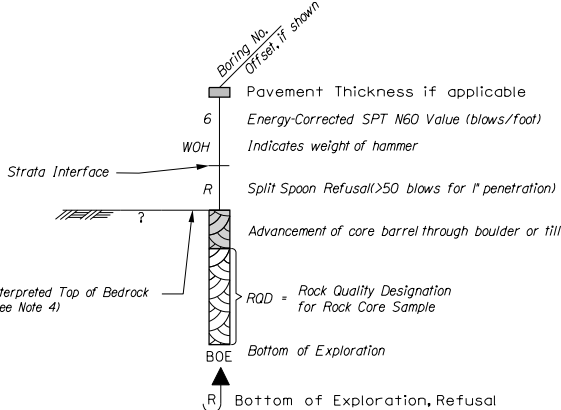
NOTES

- 1) Base map developed from electronic files provided by Erdman Anthony on August 24, 2020 (Files included 3DContours.dgn, Alignment.dgn, Bridge.dgn, 3DTopo_13Oct16.dgn and Profile.dgn).
- 2) The as drilled locations of the -100 series test borings were estimated using measured ties from existing structures. The elevations were derived from the 3D ground surface object within the microstation file from Erdman Anthony. The as-drilled locations of the -200 series borings, probes, and seismic lines were surveyed and provided by MaineDOT in an electronic file (Borings.dgn).
- 3) This generalized interpretive soil profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil and rock transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

BORING LOCATION PLAN LEGEND

- BB-CCS-102 Indicates borings performed by New England Boring Contractors of Hermon, Maine on March 3, 2017 and observed by GZA personnel.
- BB-CCS-202 Indicates borings performed by New England Boring Contractors of Hermon, Maine on June 8, 2020 and observed by GZA personnel.
- HP-CCS-202 Indicates hand probe performed by GZA personnel on June 8, 2020.
- L-4 Indicates geophysical survey line performed by Northeast Geophysical Services on June 10, 2020 and observed by GZA personnel.

INTERPRETIVE SUBSURFACE PROFILE LEGEND



PREPARED BY:



03/08/21

**CANAAN BRIDGE NO. 2120 OVER CARRABASSETT STREAM
GEOTECHNICAL DESIGN REPORT**

09.0025926.01

TABLES



TABLE 1
Summary of Subsurface Explorations
 Canaan Bridge Replacement No. 2120, Route 2/23 over Carrabassett Stream
 Canaan, ME
 WIN 21878.00

Boring ID	Ground Surface El. (ft)	Top of Stratum Elevation (ft)						Stratum Thickness (ft)					Depth to Top of Possible Bedrock (ft)	Bottom of Boring Depth (ft)	Bottom of Boring El. (ft)	Groundwater	
		Pavement	Topsoil	Fill	Glacial Till	Weathered/ Fractured Rock	Possible Bedrock	Pavement	Topsoil	Fill	Glacial Till	Weathered/ Fractured Rock				El. (ft)	Depth (ft)
BB-CCS-101	234.7	234.7	NE	234.0	224.7	NE	217.2	0.7	NE	9.3	7.5	NE	17.5	29.5	205.2	225.5	9.2
BB-CCS-102	235.1	235.1	NE	234.4	NE	NE	225.5	0.7	NE	8.8	NE	NE	9.6	20.4	214.7	227.2	7.9
BB-CCS-201	234.5	NE	234.5	234.2	230.3	NE	228.5	NE	0.3	3.9	1.8	NE	6.0	16.0	218.5	NE	NE
BB-CCS-202	236.1	NE	236.1	235.8	229.3	221.6	220.7	NE	0.3	6.5	7.7	0.9	15.4	26.5	209.6	NE	NE
HP-CCS-201	224.5	NE	NE	NE	224.5	NE	221.9	NE	NE	NE	2.6	NE	2.6	2.6	221.9	NE	NE
HP-CCS-202	226.6	NE	NE	NE	226.6	NE	223.8	NE	NE	NE	2.8	NE	2.8	2.8	223.8	NE	NE

El. = Elevation, NE = Not Encountered, NM = Not Measured, NP = Not Penetrated, > = Boring Terminated in Stratum

Notes:

1. Refer to the boring logs in Appendix B for additional information.
2. Project elevation datum is North American Vertical Datum (NAVD 88), unless noted otherwise.
3. As-drilled elevations and locations were surveyed by MaineDOT and provided to GZA by Erdman Anthony.
4. Stratum depths, thickness and elevations are rounded to the nearest 0.1 foot as interpreted on the boring logs, but this does not represent the precision of the data.



TABLE 2
Summary of Bedrock Data
Canaan Bridge Replacement No. 2120, Route 2/23 over Carrabassett Stream
Canaan, ME
WIN 21878.00

Boring ID	Core Run	Ground Surface El. (ft)	Depth of Core Run below GS (ft)			Depth to Rock (ft)	Depth Below Top of Rock (ft)			Length of Core Run (in)	Rec (in)	Rec (%)	RQD (in)	RQD %	Joint Spacing (in)	Joint Aperture (in)	Elev. (ft)		LAB						Rock Type
			Top		Bottom		Top		Bottom								Top	Bottom	Depth of Sample (ft)	Depth of Sample into Rock (ft)	Elev Top of Sample (ft)	UCS (psi)	Modulus (ksi)	Unit Wt (pcf)	
BB-CCS-101	R1	234.7	18.0	-	21.0	17.5	0.5	-	3.5	3.0	36	100%	3	8%	0.75-8	0.004-0.01	216.7	213.7							SLATE
BB-CCS-101	R2	234.7	21.0	-	26.0	17.5	3.5	-	8.5	5.0	60	100%	35	58%	0.75-24	0.004-0.01	213.7	208.7	21.9	4.4	212.8	9,022	2,900	188.0	SLATE
BB-CCS-101	R3	234.7	26.0	-	29.5	17.5	8.5	-	12.0	3.5	42	100%	20	48%	0.75-24	0.004-0.01	208.7	205.2							SLATE
BB-CCS-102	R1	235.1	10.2	-	13.5	9.6	0.6	-	3.9	3.3	38	97%	13	34%	0.75-8	0.004-0.01	224.9	221.6							SLATE
BB-CCS-102	R2	235.1	13.5	-	16.7	9.6	3.9	-	7.1	3.2	38	100%	18	47%	2.5-24	0.004-0.01	221.6	218.4	14.4	4.8	220.7	17,786	8,370	179.0	SLATE
BB-CCS-102	R3	235.1	16.7	-	20.4	9.6	7.1	-	10.8	3.7	45	101%	39	87%	2.5-24	0.004-0.01	218.4	214.7							SLATE
BB-CCS-201	R1	234.5	6.0	-	9.7	6.0	0.0	-	3.7	3.7	38	86%	10	23%	<0.75-8	0.004-0.1	228.5	224.8							SLATE
BB-CCS-201	R2	234.5	9.7	-	12.0	6.0	3.7	-	6.0	2.3	28	100%	4	14%	<0.75-8	0.004-0.1	224.8	222.5							SLATE
BB-CCS-201	R3	234.5	12.0	-	14.3	6.0	6.0	-	8.3	2.3	26	93%	0	0%	<0.75-8	0.02-0.1	222.5	220.2							SLATE
BB-CCS-201	R4	234.5	14.3	-	16.0	6.0	8.3	-	10.0	1.7	15	75%	0	0%	<0.75-8	0.02-0.1	220.2	218.5							SLATE
BB-CCS-202	R1	236.1	15.5	-	17.6	15.4	0.1	-	2.2	2.1	22	88%	6	24%	<0.75-8	0.004-0.01	220.6	218.5							SLATE
BB-CCS-202	R2	236.1	17.6	-	21.1	15.4	2.2	-	5.7	3.5	42	100%	26	62%	2.5-24	0.004-0.01	218.5	215.0							SLATE
BB-CCS-202	R3	236.1	21.1	-	23.6	15.4	5.7	-	8.2	2.5	30	100%	18	60%	<0.75-24	0.004-0.01	215.0	212.5							SLATE
BB-CCS-202	R4	236.1	23.6	-	26.5	15.4	8.2	-	11.1	2.9	35	100%	20	57%	2.5-24	0.004-0.01	212.5	209.6							SLATE



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.



APPENDIX B – BORING LOGS

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

Desired Soil Observations (in this order, if applicable):

Color (Munsell color chart)

Moisture (dry, damp, moist, wet)

Density/Consistency (from above right hand side)

Texture (fine, medium, coarse, etc.)

Name (Sand, Silty Sand, Clay, etc., including portions - trace, little, etc.)

Gradation (well-graded, poorly-graded, uniform, etc.)

Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)

Structure (layering, fractures, cracks, etc.)

Bonding (well, moderately, loosely, etc.,)

Cementation (weak, moderate, or strong)

Geologic Origin (till, marine clay, alluvium, etc.)

Groundwater level

Maine Department of Transportation

Geotechnical Section

Key to Soil and Rock Descriptions and Terms

Field Identification Information

MODIFIED BURMISTER SYSTEM			
<u>Descriptive Term</u>		<u>Portion of Total (%)</u>	
trace		0 - 10	
little		11 - 20	
some		21 - 35	
adjective (e.g. Sandy, Clayey)		36 - 50	

TERMS DESCRIBING DENSITY/CONSISTENCY

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

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

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

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

Rock Quality Designation (RQD):

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

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

Rock Quality Based on RQD	
<u>Rock Quality</u>	<u>RQD (%)</u>
Very Poor	≤25
Poor	26 - 50
Fair	51 - 75
Good	76 - 90
Excellent	91 - 100

Desired Rock Observations (in this order, if applicable):

Color (Munsell color chart)

Texture (aphanitic, fine-grained, etc.)

Rock Type (granite, schist, sandstone, etc.)

Hardness (very hard, hard, mod. hard, etc.)

Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)

Geologic discontinuities/jointing:

- dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.)
- spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet)
- tightness (tight, open, or healed)
- infilling (grain size, color, etc.)

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)

RQD and correlation to rock quality (very poor, poor, etc.)

ref: ASTM D6032 and FHWA NHI-16-072 GEC 5 - Geotechnical Site Characterization, Table 4-12

Recovery (inch/inch and percentage)

Rock Core Rate (X.X ft - Y.Y ft (min:sec))

Sample Container Labeling Requirements:

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

<div>Maine Department of Transportation</div>						Project: <div>Canaan Bridge Replacement09.0025926.00</div>							Boring No.: <div>BB-CCS-101</div>							
<div>Soil/Rock Exploration LogUS CUSTOMARY UNITS</div>						Location: <div>Route 2/23 Over Carrabassett StreamCanaan, Maine</div>							WIN: <div>21878.00</div>							
Driller: <div>New England Boring</div>						Elevation (ft.): <div>234.7</div>							Auger ID/OD: <div>2.25</div>							
Operator: <div>Brad Enos</div>						Datum: <div>NAVD 88</div>							Sampler: <div>Standard Splitspoon</div>							
Logged By: <div>Blaine Cardali</div>						Rig Type: <div>Truck - Mobile Drill</div>							Hammer Wt./Fall: <div>140lbs/30"</div>							
Date Start/Finish: <div>3/2/17-3/2/17</div>						Drilling Method: <div>SSA / Drive & Wash</div>							Core Barrel: <div>NX2</div>							
Boring Location: <div>N460432.2, E1527083.0</div>						Casing ID/OD: <div>4/4.5/3/3.5"</div>							Water Level*: <div>9.2'</div>							
Hammer Efficiency Factor: <div>0.6</div>						Hammer Type:														
						Automatic <input type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input checked="" type="checkbox"/>														
Definitions: <div>D = Split Spoon SampleMD = Unsuccessful Split Spoon Sample AttemptU = Thin Wall Tube SampleMU = Unsuccessful Thin Wall Tube Sample AttemptV = Field Vane Shear Test, PP = Pocket PenetrometerMV = Unsuccessful Field Vane Shear Test Attempt</div> R = Rock Core SampleSSA = Solid Stem AugerHSA = Hollow Stem AugerRC = Roller ConeWOH = Weight of 140 lb. HammerWOR/C = Weight of Rods or CasingWO1P = Weight of One PersonSu = Peak/Remolded Field Vane Undrained Shear Strength (psf)Su(lab) = Lab Vane Undrained Shear Strength (psf)qp = Unconfined Compressive Strength (ksf)N-uncorrected = Raw Field SPT N-valueHammer Efficiency Factor = Rig Specific Annual Calibration ValueN60 = SPT N-uncorrected Corrected for Hammer EfficiencyN60 = (Hammer Efficiency Factor/60%)*N-uncorrectedTv = Pocket Torvane Shear Strength (psf)WC = Water Content, percentLL = Liquid LimitPL = Plastic LimitPI = Plasticity IndexG = Grain Size AnalysisC = Consolidation Test																				
Sample Information																				
Depth (ft.)Sample No.Pen./Rec. (in.)Sample Depth (ft.)Blows (/6 in.)Shear Strength (psf)or RQD (%)N-uncorrectedN60Casing BlowsElevation (ft.)Graphic LogVisual Description and RemarksLaboratory Testing Results/AASHTO Unified Class.																				
25 <div>R342/4226.0 - 29.5RQD = 48%</div> <div>Elevation: 205.2</div> <div>Bottom of Exploration at 29.5 feet below ground surface.</div>																				
Remarks: <div>1. 4" casing to 17.5' bgs, then 3" casing to 18.0' bgs. 2. Water level measured approximately 20 minutes after completion of drilling. 3. Automatic Hammer NEBC #23, Energy Transfer Ratio = 0.6</div>																				
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.																				
Page 2 of 2 Boring No.: BB-CCS-101																				

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Location: Route 2/23 Over Carrabassett Stream
Canaan, Maine

WIN: 21878.00

C = Consolidation Test

Remarks:

- Boring No.:** BB-CCS-201

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Canaan Bridge Replacement Location: Route 2/23 Over Carrabassett Stream Canaan, Maine				Boring No.: BB-CCS-202 WIN: 21878.00																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
Driller: New England Boring Contractors				Elevation (ft.): 236.1				Auger ID/OD: 4.25"																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
Operator: Brad Enos				Datum: NAVD88				Sampler: Standard Splitspoon																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
Logged By: Erin Tome				Rig Type: ATV				Hammer Wt./Fall: 140lbs/30"																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
Date Start/Finish: 6/8/20 - 6/8/20				Drilling Method: SSA / Drive & Wash				Core Barrel: NX																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
Boring Location: N460432.7, E1527167.3				Casing ID/OD: 4/4.5/3/3.5"				Water Level*: Not Observed																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
Hammer Efficiency Factor: 0.842				Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
<div>Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt</div> <div>R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140 lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person</div> <div>S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S_u(lab) = Lab Vane Undrained Shear Strength (psf) q_p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected</div> <div>T_v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test</div>																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
<table><tr><th colspan="9">Sample Information</th><th rowspan="2">Graphic Log</th><th rowspan="2">Visual Description and Remarks</th><th rowspan="2">Laboratory Testing Results/ AASHTO and Unified Class.</th></tr><tr><th>Depth (ft.)</th><th>Sample No.</th><th>Pen./Rec. (in.)</th><th>Sample Depth (ft.)</th><th>Blows (6 in.) Shear Strength (psf) or RQD (%)</th><th>N-uncorrected</th><th>N₆₀</th><th>Casing Blows</th><th>Elevation (ft.)</th></tr><tr><td>25</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td rowspan="10">209.6</td><td rowspan="10"><div>Recovery = 100%</div><div>Rock Core Times (min:sec): 21.1-22.1' (2:00), 22.1-23.1' (5:45), 23.1-23.6' (1:45)</div><div>R4: Hard, fresh, aphanitic, grey, SLATE. Joints are close to moderately spaced, high angle, undulating to planar, rough to smooth, discolored, tight to partially open.</div><div>Rock Quality = Fair</div><div>Recovery = 100%</div><div>Rock Core Times (min:sec): 23.6-24.6' (3:00), 24.6-25.6' (3:30), 25.6-26.5' (3:30)</div><div>26.5</div><div>Bottom of Exploration at 26.5 feet below ground surface.</div></td><td rowspan="10"></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>30</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>40</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>45</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>50</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr></table>												Sample Information									Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.	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.)	25									209.6	<div>Recovery = 100%</div> <div>Rock Core Times (min:sec): 21.1-22.1' (2:00), 22.1-23.1' (5:45), 23.1-23.6' (1:45)</div> <div>R4: Hard, fresh, aphanitic, grey, SLATE. Joints are close to moderately spaced, high angle, undulating to planar, rough to smooth, discolored, tight to partially open.</div> <div>Rock Quality = Fair</div> <div>Recovery = 100%</div> <div>Rock Core Times (min:sec): 23.6-24.6' (3:00), 24.6-25.6' (3:30), 25.6-26.5' (3:30)</div> <div>26.5</div> <div>Bottom of Exploration at 26.5 feet below ground surface.</div>																																																																																			30																																																																																																																																																												40																																																																																																																																				45																																																																																																												50											
Sample Information									Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
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.)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
25									209.6	<div>Recovery = 100%</div> <div>Rock Core Times (min:sec): 21.1-22.1' (2:00), 22.1-23.1' (5:45), 23.1-23.6' (1:45)</div> <div>R4: Hard, fresh, aphanitic, grey, SLATE. Joints are close to moderately spaced, high angle, undulating to planar, rough to smooth, discolored, tight to partially open.</div> <div>Rock Quality = Fair</div> <div>Recovery = 100%</div> <div>Rock Core Times (min:sec): 23.6-24.6' (3:00), 24.6-25.6' (3:30), 25.6-26.5' (3:30)</div> <div>26.5</div> <div>Bottom of Exploration at 26.5 feet below ground surface.</div>																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
30																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
40																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
45																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
50																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
Remarks: 1. Automatic Hammer NEBC #23, Energy Transfer Ratio = 0.842.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										Page 2 of 2																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
Boring No.: BB-CCS-202																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					



03/08/21

**CANAAN BRIDGE NO. 2120 OVER CARRABASSETT STREAM
GEOTECHNICAL DESIGN REPORT**

09.0025926.01

APPENDIX C – SEISMIC REFRACTION SURVEY

**SEISMIC REFRACTION SURVEY
CANAAAN BRIDGE SITE
CANAAAN, MAINE**

**For:
GZA Inc.
August, 2020**

Northeast Geophysical Services
Division of NGS, Inc.
4 Union Street
Bangor, Maine

**SEISMIC REFRACTION SURVEY
CANAAN BRIDGE SITE
CANAAN, MAINE**

INTRODUCTION

At the request of GZA Inc., a seismic refraction survey was completed at the Canaan Bridge Site located on Route 2 in Canaan, Maine. The objective of this survey was to determine the bedrock depth and configuration beneath the survey area. The field survey was undertaken on June 10, 2020. Four seismic lines were surveyed. This report describes the equipment and methods used and the results of the survey, and includes profiles of the interpreted seismic lines.

LOCATION AND SITE CONDITIONS

The survey lines are located along the south side of Route 2 in Canaan, Maine with Seismic Lines 1 and 2 located to the west of the Canaan Bridge and Lines 3 and 4 located to the east of the bridge. The approximate locations of the seismic lines with respect to the Canaan Bridge are shown on the Seismic Line Location map (following page). Surface conditions along the lines were generally lawn and a gravel driveway.

SUMMARY OF RESULTS

The seismic refraction results are attached as profiles of each survey line. The seismic results show the seismically interpreted depths to bedrock and configurations. The seismically calculated bedrock depths range from approximately 2 feet to 14 feet deep over the survey area.

SEISMIC METHODS AND INSTRUMENTATION

The seismic refraction method relies on travel times of sound waves, measured in milliseconds, traveling through and refracting from subsurface layers with contrasting densities. The seismic refraction lines were surveyed using a Geometrics Geode, 24-channel seismograph. Surface elevations for the end points of each line were obtained from GZA and the intermediate elevations were interpolated.

Each survey line was 92 feet long containing 24 geophones that were spaced 4 feet apart. Each segment was tested with six to seven shots. The general shot configuration consisted of one shot at either end of the segment, one off each end about 20 feet, and two or more within the segment. The energy source consisted of a 16-lb hammer striking a metal plate.

The seismic data were processed and interpreted using the SeisImager 2D seismic interpretation program by Geometrics. This program calculates seismic velocities by regression and by Hobson-Overton method, and solves for layer thicknesses using the delay-time method and iterative ray tracing modeling. The data was also interpreted using tomographic analysis (also by SeisImager). Tomography better accommodates horizontal velocity variations.

Seismic Line Location Map Canaan Bridge Site



Locations are approximate

SEISMIC SURVEY RESULTS

Profiles of the four lines showing the seismically interpreted bedrock depths and configurations and tabulated data are attached.

The survey identified two velocity layers. The Layer 1 velocity for the survey averaged about 1,260 feet per second (fps) and is interpreted to represent compact dry soil. The Layer 2 velocity averaged about 11,920 fps and is interpreted to represent bedrock. Typically bedrock velocity in Maine is faster than this, ranging from 14,000 to 20,000 fps. It is possible the lower bedrock velocities measured at this site are because the bedrock is less competent (softer or more fractured).

DISCUSSION OF SEISMIC RESULTS

In order for the seismic refraction method to accurately estimate velocity layer depths, certain natural conditions should exist:

- a.) Layers should increase in velocity and in thickness with depth. A typical example would be ten feet of unsaturated soil at 1,100 fps overlying 50 feet of saturated soil at 5,000 fps that overlies bedrock at 15,000 fps.
- b.) There should be a sufficient velocity contrast between different layers. Ideally, each velocity layer would be 2 to 3 times faster than the overlying layer.
- c.) The velocity within a layer should be relatively constant throughout that layer (lateral homogeneity).

At the Canaan site these conditions were generally met, however, it is suspected that there are some lateral velocity variations in the Layer 1 and 2 velocities which may have affected the models. If velocities are used in the model that are lower than the actual velocities traveling through the subsurface, the depth to bedrock would be underestimated. And, conversely, if higher velocities are used in the model, the depth to bedrock would be overestimated.

In addition to these conditions, it is also important that there be a low level of background noise at the site. Background noise was very troublesome at this site because of traffic but we tried to collect data during lulls in the traffic. It is also very helpful if there is some ground truth data, such as borehole data, to compare and calibrate the seismic information. At the Canaan site there were 2 boreholes, one located near the east end of Line 1 (BB-CCS-201) and one located near the west end of Line 3 (BB-CCS-202).

The 2-velocity model for bedrock depth matched the known bedrock depth fairly closely in BB-CCS-201 at the east end of Line 1. However, the 2-velocity model for Line 3 was shallower than the depth to bedrock in BB-CCS-202 and more closely matched the depth to weathered rock. Accordingly, a 3-velocity model was used for Lines 3 and 4. This 3-layer model brings the bedrock surface closer to the depth to competent bedrock measured in BB-CCS-202.

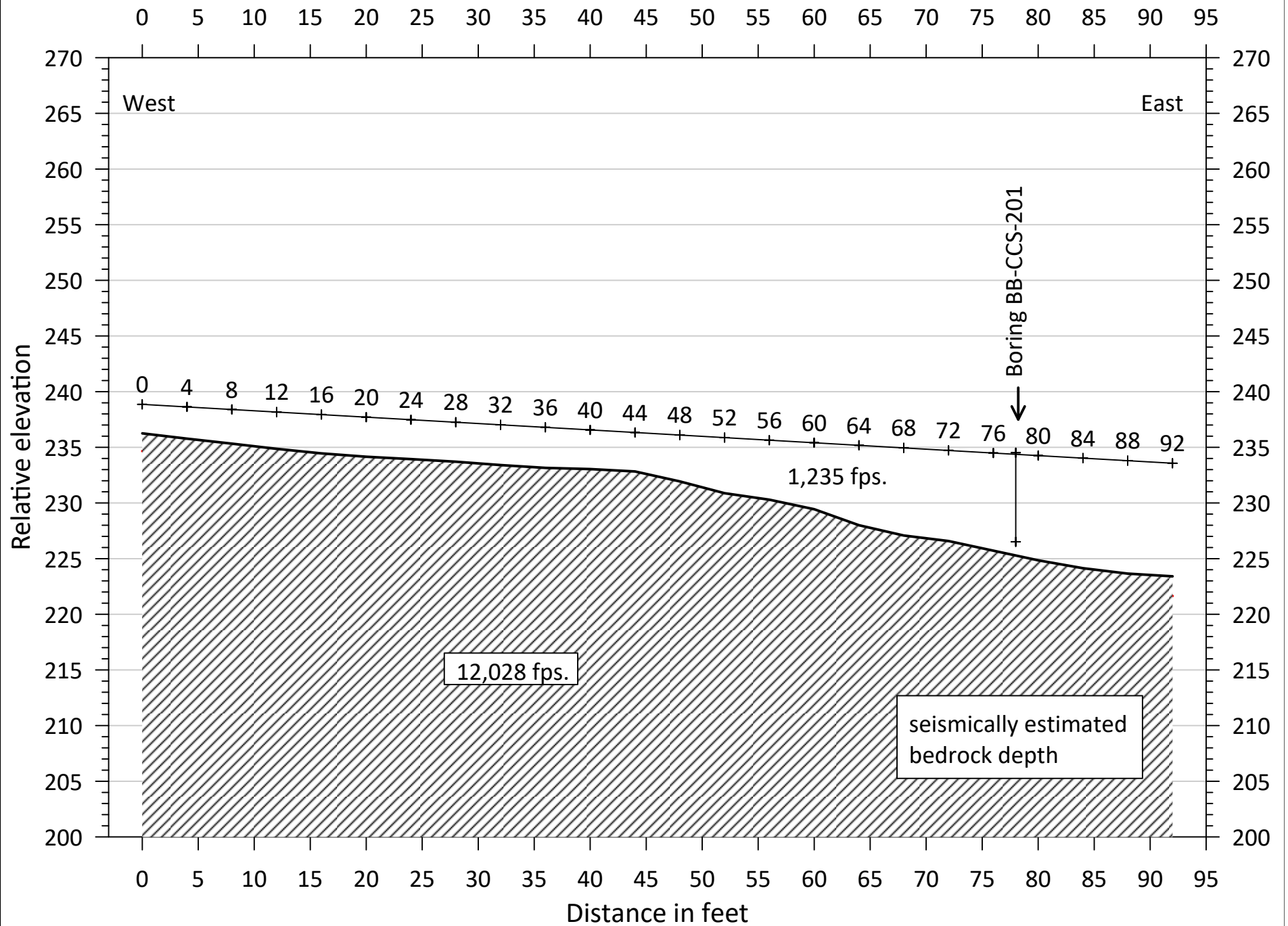
Under favorable conditions seismic refraction results can be fairly precise, within +/- 10 percent or within 5 feet. The conditions at the Canaan site were challenging mainly because of the high background noise and also possible lateral changes in layer velocities. Based on the borehole information there are varying thicknesses of weathered bedrock in the survey area. Ideally the soil and bedrock would be firm and homogeneous.

Northeast Geophysical Services

As with any indirect method it is possible that the seismically interpreted depths may not be accurate, however, it is believed that the seismic survey at the Canaan site fairly accurately depicts the bedrock configuration.

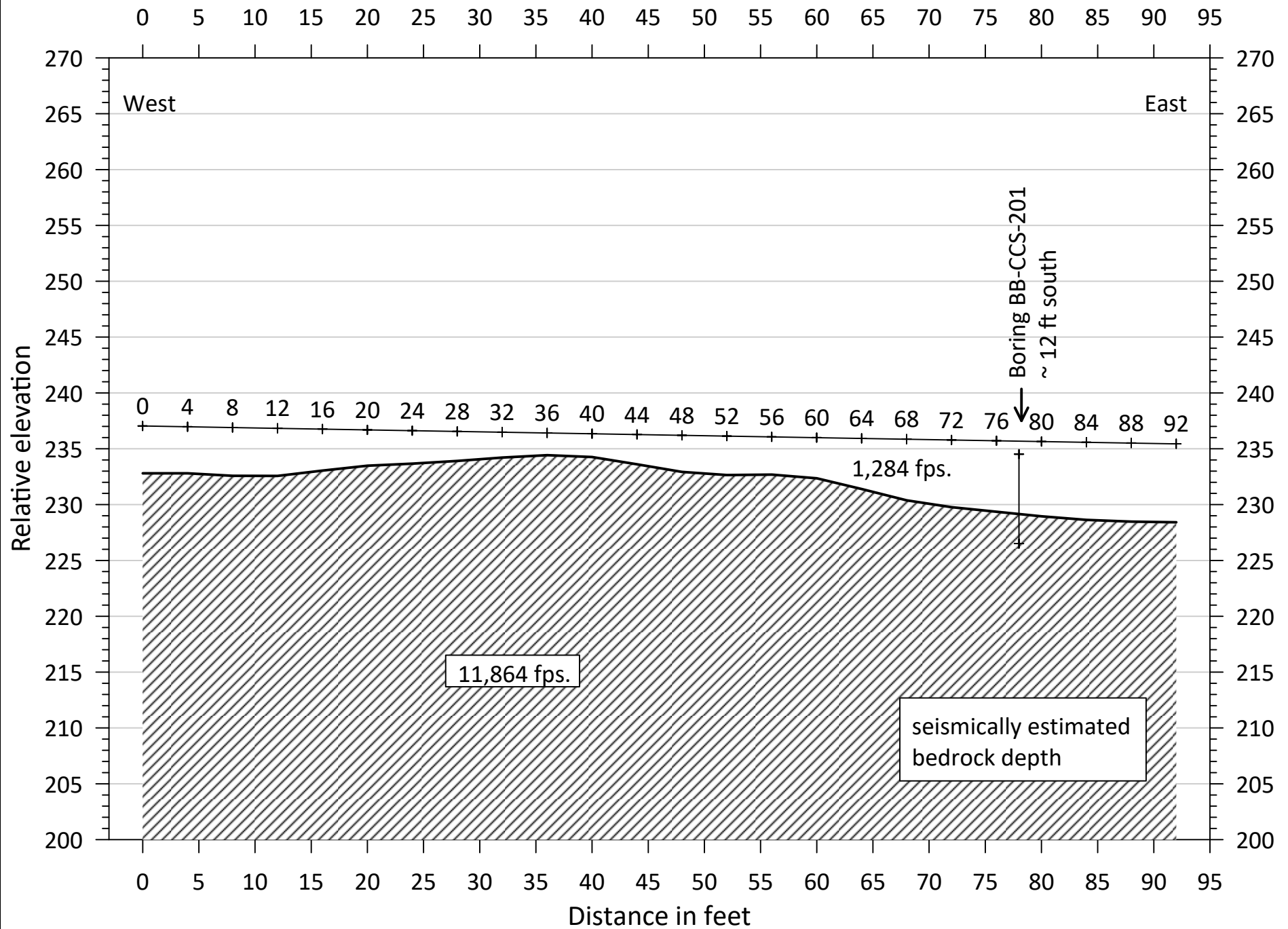
ATTACHMENTS
SEISMIC REFRACTION PROFILES
AND TABULATED DATA

Seismic Line 1



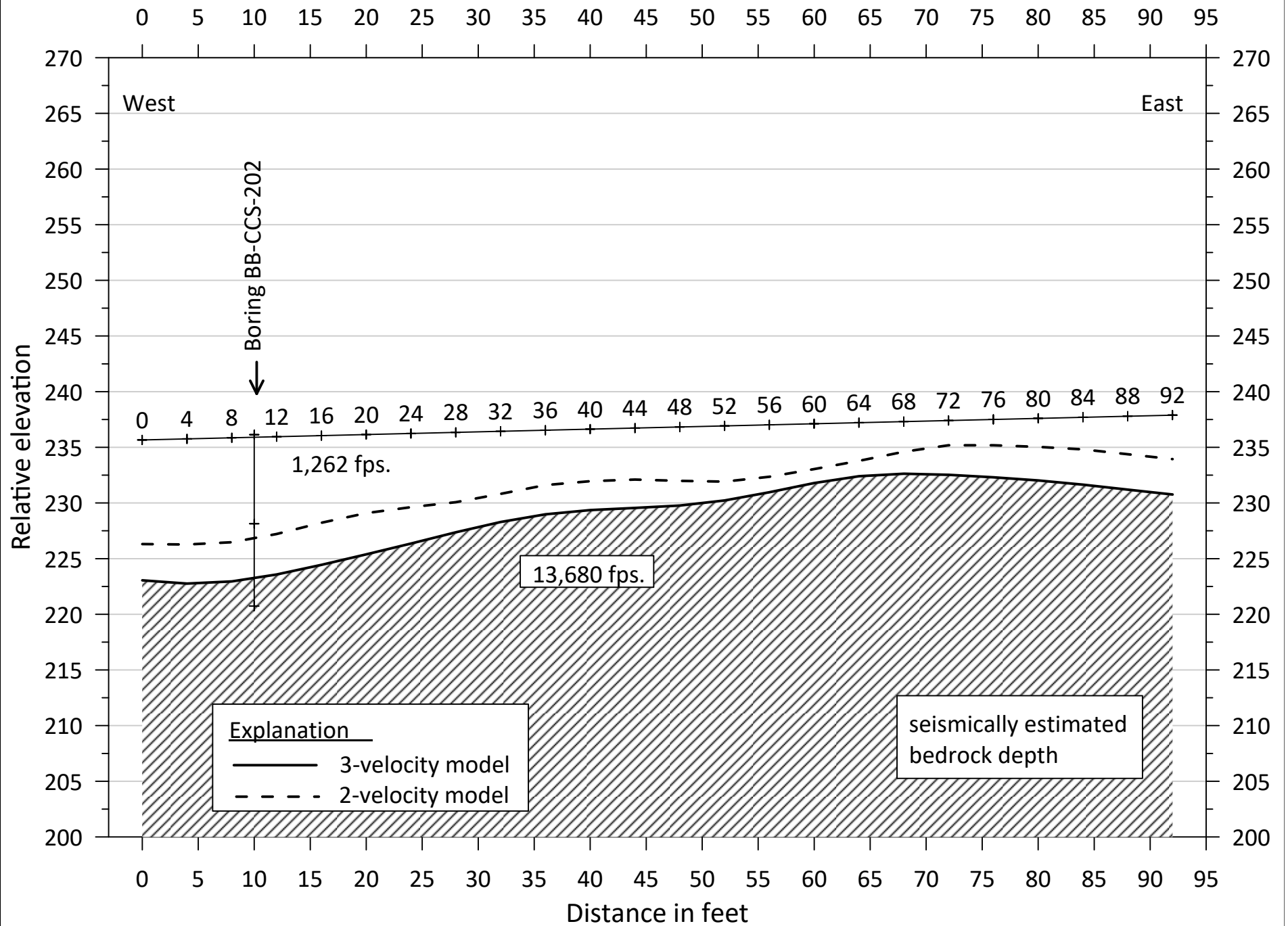
Line 1 Seismic Model - Canaan Brdge Site - Canaan, Maine				
Seismic Line	Geophone number	X distance	Surface elevation	Bedrock elevation
1	1	0	238.9	236
1	2	4	238.6	236
1	3	8	238.4	235
1	4	12	238.2	235
1	5	16	237.9	234
1	6	20	237.7	234
1	7	24	237.5	234
1	8	28	237.3	234
1	9	32	237.0	233
1	10	36	236.8	233
1	11	40	236.6	233
1	12	44	236.3	233
1	13	48	236.1	232
1	14	52	235.9	231
1	15	56	235.6	230
1	16	60	235.4	229
1	17	64	235.2	228
1	18	68	234.9	227
1	19	72	234.7	227
1	20	76	234.5	226
1	21	80	234.3	225
1	22	84	234.0	224
1	23	88	233.8	224
1	24	92	233.6	223

Seismic Line 2



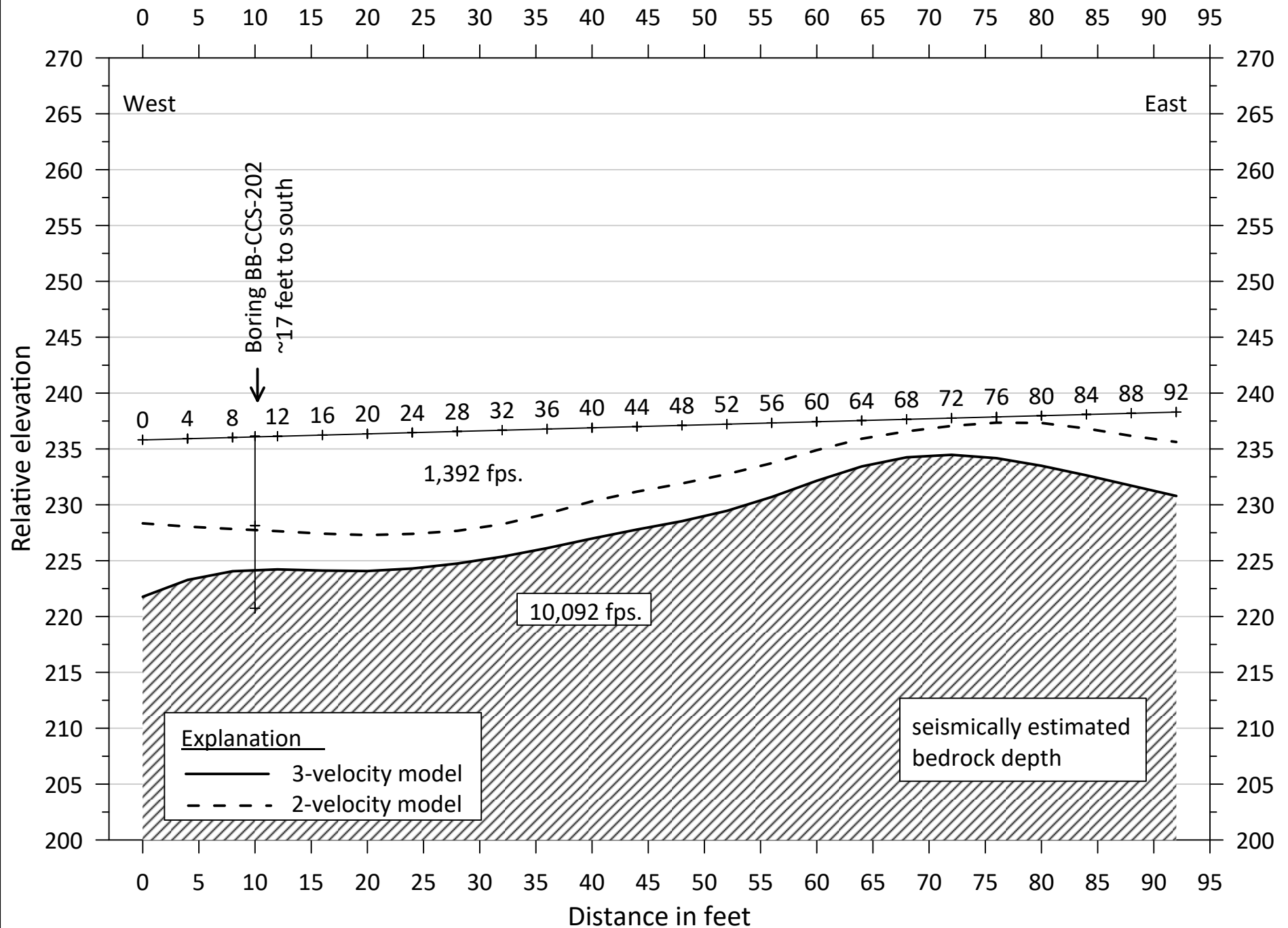
Line 2 Seismic Model - Canaan Brdge Site - Canaan, Maine				
Seismic Line	Geophone number	X distance	Surface elevation	Bedrock elevation
2	1	0	237.0	233
2	2	4	237.0	233
2	3	8	236.9	233
2	4	12	236.8	233
2	5	16	236.8	233
2	6	20	236.7	233
2	7	24	236.6	234
2	8	28	236.6	234
2	9	32	236.5	234
2	10	36	236.4	234
2	11	40	236.3	234
2	12	44	236.3	234
2	13	48	236.2	233
2	14	52	236.1	233
2	15	56	236.1	233
2	16	60	236.0	232
2	17	64	235.9	231
2	18	68	235.9	230
2	19	72	235.8	230
2	20	76	235.7	229
2	21	80	235.6	229
2	22	84	235.6	229
2	23	88	235.5	228
2	24	92	235.4	228

Seismic Line 3



Line 3 Seismic Model - Canaan Brdge Site - Canaan, Maine				
Seismic Line	Geophone number	X distance	Surface elevation	Bedrock elevation
3	1	0	235.7	223
3	2	4	235.8	223
3	3	8	235.9	223
3	4	12	236.0	224
3	5	16	236.0	224
3	6	20	236.1	225
3	7	24	236.2	226
3	8	28	236.3	227
3	9	32	236.4	228
3	10	36	236.5	229
3	11	40	236.6	229
3	12	44	236.7	230
3	13	48	236.8	230
3	14	52	236.9	230
3	15	56	237.0	231
3	16	60	237.1	232
3	17	64	237.2	232
3	18	68	237.3	233
3	19	72	237.4	233
3	20	76	237.5	232
3	21	80	237.6	232
3	22	84	237.7	232
3	23	88	237.8	231
3	24	92	237.9	231

Seismic Line 4



Line 4 Seismic Model - Canaan Brdge Site - Canaan, Maine				
Seismic Line	Geophone number	X distance	Surface elevation	Bedrock elevation
4	1	0	235.8	222
4	2	4	235.9	223
4	3	8	236.0	224
4	4	12	236.1	224
4	5	16	236.2	224
4	6	20	236.4	224
4	7	24	236.5	224
4	8	28	236.6	225
4	9	32	236.7	225
4	10	36	236.8	226
4	11	40	236.9	227
4	12	44	237.0	228
4	13	48	237.1	229
4	14	52	237.2	229
4	15	56	237.3	231
4	16	60	237.4	232
4	17	64	237.5	233
4	18	68	237.7	234
4	19	72	237.8	234
4	20	76	237.9	234
4	21	80	238.0	233
4	22	84	238.1	233
4	23	88	238.2	232
4	24	92	238.3	231



APPENDIX D – ROCK CORE PHOTOGRAPHS



**Canaan Bridge No. 2120 Replacement
Route 2/23 over Carrabasset Stream
Canaan, ME
Rock Core Photographs**

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-CCS-101	R1	18.0 - 21.0	36	100	3	8	SLATE	1
BB-CCS-101	R2	21.0 - 26.0	60	100	35	58	SLATE	1,2
BB-CCS-101	R3	26.0 - 29.5	42	100	20	48	SLATE	2,3



- 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.



**Canaan Bridge No. 2120 Replacement
Route 2/23 over Carrabasset Stream
Canaan, ME
Rock Core Photographs**

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-CCS-102	R1	10.2 - 13.5	38	96	13	34	SLATE	1
BB-CCS-102	R2	13.5 - 16.7	38	100	18	47	SLATE	1,2
BB-CCS-102	R3	16.7 - 20.4	45	101	39	87	SLATE	2,3



- 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.



**Canaan Bridge No. 2120 Replacement
Route 2/23 over Carrabasset Stream
Canaan, ME
Rock Core Photographs**

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-CCS-201	R1	6.0 - 9.7	38	86	10	23	SLATE	1
BB-CCS-201	R2	9.7 - 12.0	28	100	4	14	SLATE	1,2
BB-CCS-201	R3	12.0 - 14.3	26	93	0	0	SLATE	2
BB-CCS-201	R4	14.3 - 16	15	75	0	0	SLATE	2
BB-CCS-202	R1	15.5 - 17.6	22	88	6	24	SLATE	3
BB-CCS-202	R2	17.6 - 21.1	42	100	26	62	SLATE	3,4
BB-CCS-202	R3	21.1 - 23.6	30	100	18	60	SLATE	4
BB-CCS-202	R4	23.6 - 26.5	35	100	20	57	SLATE	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.



APPENDIX E – LABORATORY TEST RESULTS



Laboratory Testing Summary Sheet

Replacement

MDOT Project Number:

GZA Project Number: 09.0025926.00

Town(s): Canaan, Maine

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

LABORATORY TESTING DATA SHEET

Matthew Roberson

Project Name **Canaan Bridge Replacement**
 Project No. **09.0025926.00**
 Project Manager T. Blair

Location Canaan, Maine
 Assigned By B. Cardali
 Report Date 03.21.17

Reviewed By _____
 Date Reviewed **03.22.17**

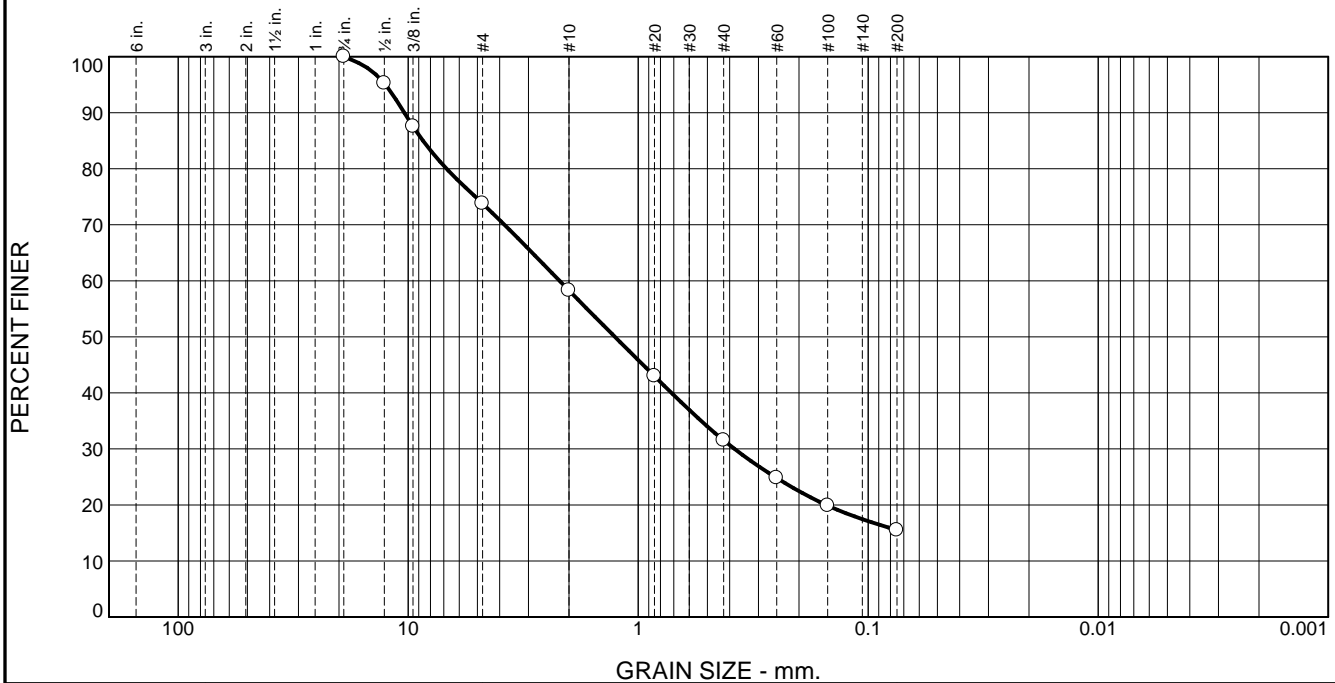
Boring/ Test Pit No.	Sample No.	Depth ft.	Lab No.	Identification Tests						Corrosivity					Laboratory Log and Soil Description
				Water Content %	LL %	PL %	Gravel %	Sand %	Fines (<#200) %	pH	Sulfate (mg/kg)	Chloride (mg/kg)	Resistivity (Mohms-cm)	GTL Resist	
BB-CCS-101	1D	2-4	1	10.8			26.2	58.3	15.5						Dark Grey f-c SAND, some fine Gravel, little Silt
BB-CCS-101	3D	10-12	2	7.5			38.6	48.5	12.9						Dark Grey f-c SAND and fine GRAVEL, little Silt
BB-CCS-102	1D	2-4	3	9.0			20.7	41.3	38.0						Grey f-c SAND and SILT, some f-c Gravel
BB-CCS-102	2D	5-7	4	11.7			13.7	42.2	44.1						Grey f-c SAND and SILT, little f-c Gravel



195 Frances Avenue
 Cranston, RI 02910

401-467-6454

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	26.2	15.5	26.8	16.0	15.5	

TEST RESULTS (D6913)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
.75	100.0		
0.5	95.2		
.375	87.5		
#4	73.8		
#10	58.3		
#20	43.0		
#40	31.5		
#60	24.8		
#100	19.9		
#200	15.5		

* (no specification provided)

Material Description

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

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

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

Coefficients

D₉₀= 10.4005 D₈₅= 8.6237 D₆₀= 2.1969
D₅₀= 1.2620 D₃₀= 0.3812 D₁₅=
D₁₀= C_u= C_c=

Remarks

Date Received: 03.13.17 Date Tested: 03.16.17

Tested By: IA

Checked By: Matthew Colman, P.E.

Title: Laboratory Manager

Source of Sample: Borings Depth: 2-4'
Sample Number: BB-CCS-101 / 1D

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

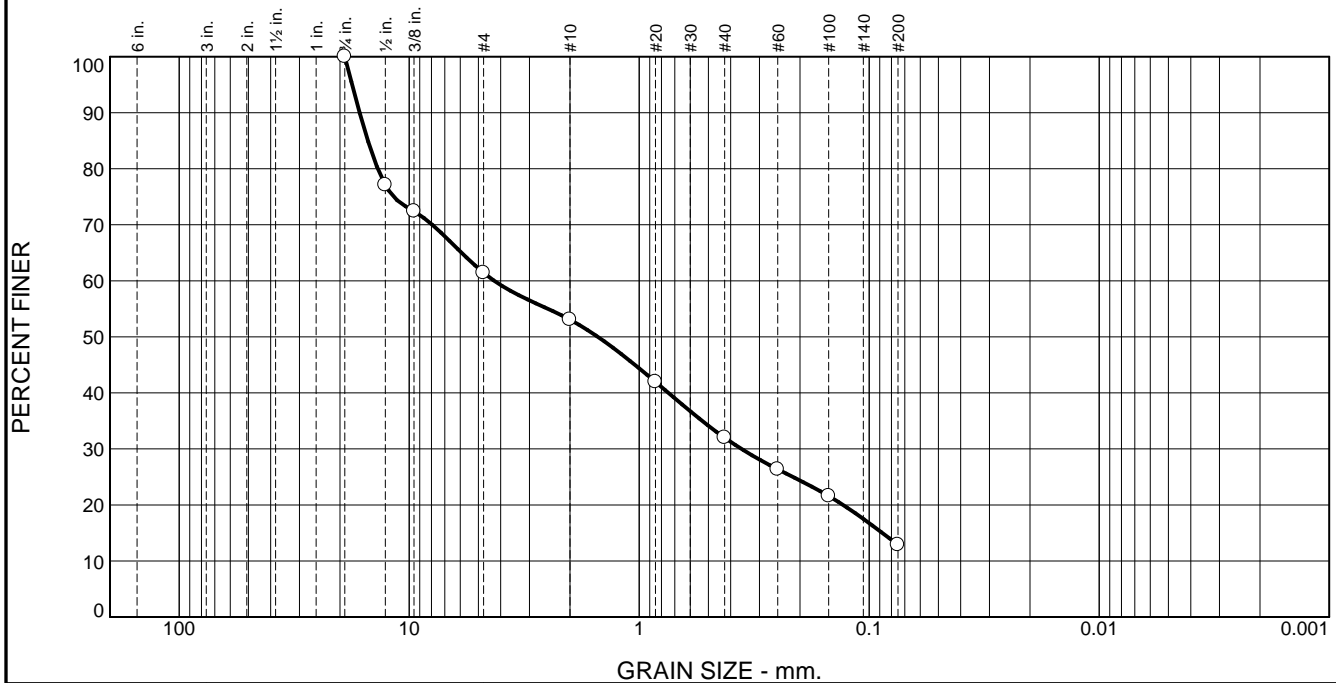
Client: GZA GeoEnvironmental/Maine Department of Transportation

Project: Canaan Bridge Replacement
Canaan, Maine

Project No: 09.0025926.00

Figure S-1

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	38.6	8.3	21.1	19.1	12.9	

TEST RESULTS (D6913)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
.75	100.0		
0.5	77.1		
.375	72.4		
#4	61.4		
#10	53.1		
#20	42.0		
#40	32.0		
#60	26.4		
#100	21.6		
#200	12.9		

* (no specification provided)

Material Description
Dark Grey f-c SAND and fine GRAVEL, little Silt

Atterberg Limits (ASTM D 4318)
PL= NP LL= NV PI= NP

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

Coefficients
D₉₀= 16.4100 D₈₅= 15.1078 D₆₀= 4.2645
D₅₀= 1.5127 D₃₀= 0.3589 D₁₅= 0.0876
D₁₀= C_u= C_c=

Remarks

Date Received: 03.13.17 Date Tested: 03.16.17
Tested By: IA
Checked By: Matthew Colman, P.E.
Title: Laboratory Manager

Source of Sample: Borings Depth: 10-12'
Sample Number: BB-CCS-101 / 3D

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

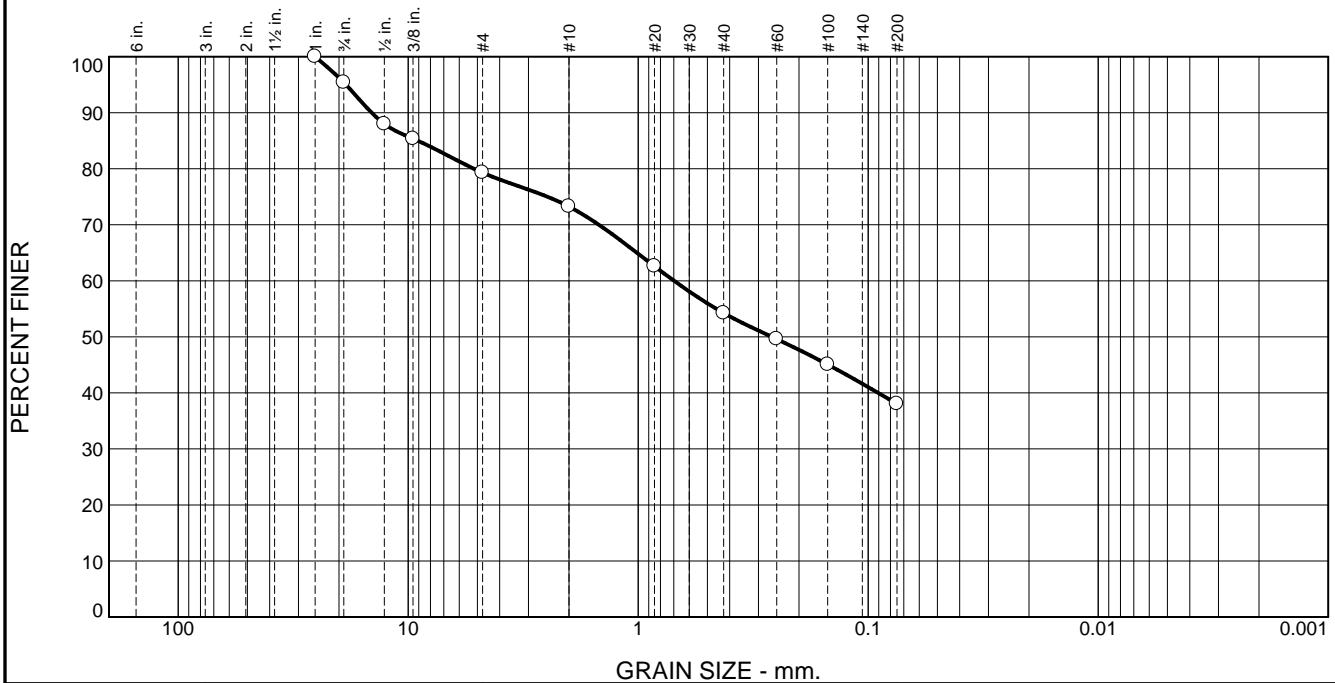
Client: GZA GeoEnvironmental/Maine Department of Transportation

Project: Canaan Bridge Replacement
Canaan, Maine

Project No: 09.0025926.00

Figure S-2

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	4.6	16.1	6.0	19.0	16.3	38.0	

TEST RESULTS (D6913)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1	100.0		
.75	95.4		
0.5	88.0		
.375	85.4		
#4	79.3		
#10	73.3		
#20	62.6		
#40	54.3		
#60	49.6		
#100	45.1		
#200	38.0		

* (no specification provided)

Material Description
Grey f-c SAND and SILT, some f-c Gravel

Atterberg Limits (ASTM D 4318)
PL= NP LL= NV PI= NP

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

Coefficients
D₉₀= 14.4124 D₈₅= 9.0754 D₆₀= 0.6977
D₅₀= 0.2622 D₃₀= C_u= D₁₅= C_c=

Remarks

Date Received: 03.13.17 Date Tested: 03.16.17
Tested By: IA
Checked By: Matthew Colman, P.E.
Title: Laboratory Manager

Source of Sample: Borings Depth: 2-4'
Sample Number: BB-CCS-101 / 1D

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

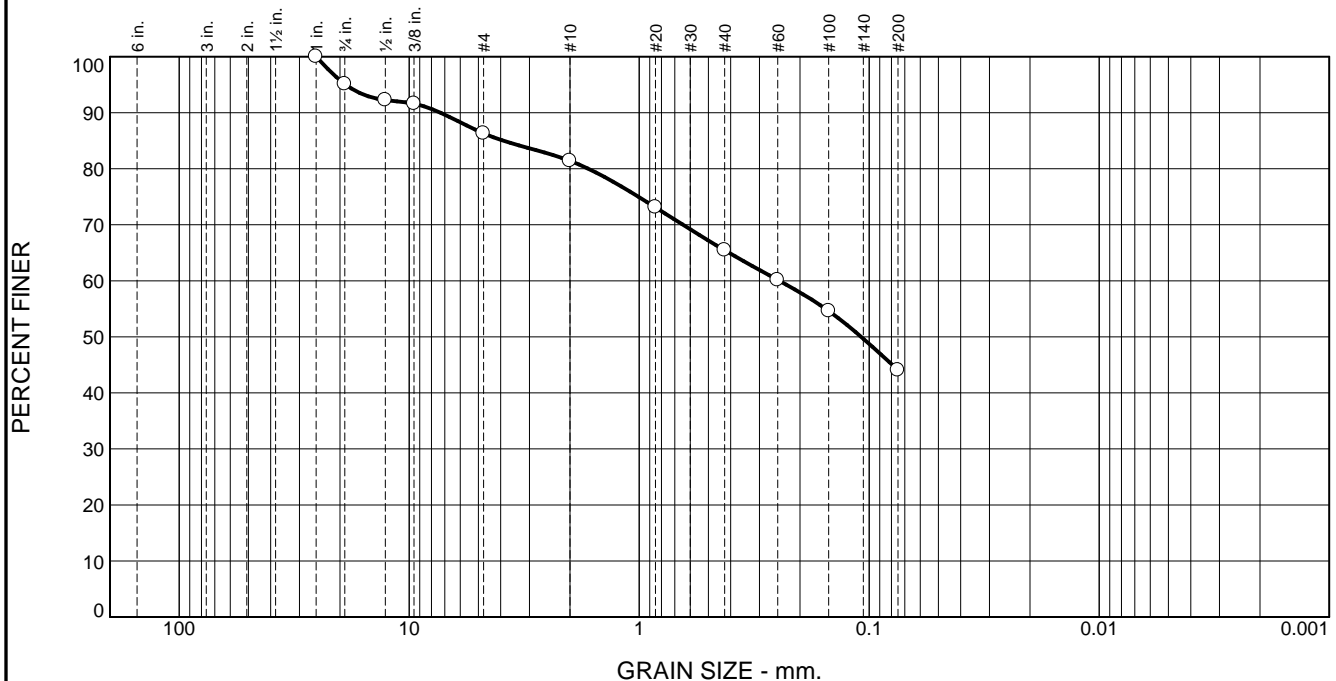
Client: GZA GeoEnvironmental/Maine Department of Transportation

Project: Canaan Bridge Replacement
Canaan, Maine

Project No: 09.0025926.00

Figure S-3

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	4.9	8.8	4.9	15.9	21.4	44.1	

TEST RESULTS (D6913)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1	100.0		
.75	95.1		
0.5	92.3		
.375	91.6		
#4	86.3		
#10	81.4		
#20	73.1		
#40	65.5		
#60	60.2		
#100	54.6		
#200	44.1		

* (no specification provided)

Material Description
Grey f-c SAND and SILT, little f-c Gravel

Atterberg Limits (ASTM D 4318)
PL= NP LL= NV PI= NP

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

Coefficients
D₉₀= 7.3113 D₈₅= 3.8856 D₆₀= 0.2461
D₅₀= 0.1083 D₃₀= C_u= D₁₅= C_c=

Remarks

Date Received: 03.13.17 Date Tested: 03.16.17
Tested By: IA
Checked By: Matthew Colman, P.E.
Title: Laboratory Manager

Source of Sample: Borings Depth: 5-7'
Sample Number: BB-CCS-101 / 2D

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental/Maine Department of Transportation

Project: Canaan Bridge Replacement
Canaan, Maine

Project No: 09.0025926.00

Figure S-4



Technologies to manage risk
for infrastructure

Boston
Atlanta
Chicago
Los Angeles
New York

www.geotesting.com

Transmittal

TO:

Blaine Cardali

GZA GeoEnvironmental, Inc.

477 Congress Street, Suite 700,

Portland, ME 04101

DATE: 3/24/2017

GTX NO: 306143

RE: Canaan Bridge Replacement

COPIES	DATE	DESCRIPTION
	3/24/2017	March 2017 Laboratory Test Report

REMARKS:

CC:

SIGNED:

Jonathan Campbell, Assistant Laboratory Manager

APPROVED BY:

Mark Dobday, P.G., Laboratory Manager

March 24, 2017

Blaine Cardali
GZA GeoEnvironmental, Inc.
477 Congress Street, Suite 700
Portland, ME 04101

RE: Canaan Bridge Replacement, Canaan, ME (GTX-306143)

Dear Blaine Cardali:

Enclosed are the test results you requested for the above referenced project. GeoTesting Express, Inc. (GTX) received two samples from you on 3/15/2017. These samples were labeled as follows:

Boring Number	Sample Number	Depth
BB-CCS-101, R2	2	21.5-22.6 ft
BB-CCS-102, R2	2	14.2-15.3 ft

GTX performed the following tests on these samples:

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

A copy of your test request is attached.

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

Respectfully yours,



Jonathan Campbell
Assistant Laboratory Manager



*Technologies to manage risk
for infrastructure*

Boston
Atlanta
Chicago
Los Angeles
New York

www.geotesting.com

Geotechnical Test Report

3/24/2017

GTX-306143

Canaan Bridge Replacement

Canaan, ME

Client Project No.: 09.0025926.00

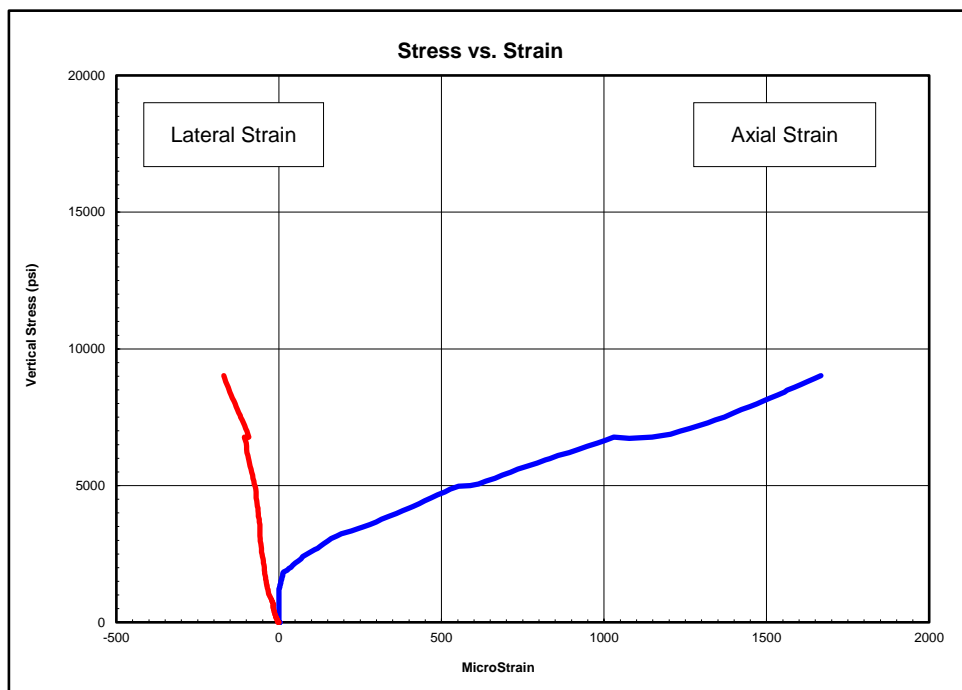
Prepared for:

GZA GeoEnvironmental, Inc.



Client:	GZA GeoEnvironmental, Inc.
Project Name:	Canaan Bridge Replacement
Project Location:	Canaan, ME
GTX #:	306143
Test Date:	3/21/2017
Tested By:	daa/trm
Checked By:	jsc
Boring ID:	BB-CCS-101, R2
Sample ID:	2
Depth, ft:	21.92-22.29
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

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



Peak Compressive Stress: 9,022 psi

One lateral strain gauge failed to record meaningful data. Poisson's Ratio reported based on results of a single lateral strain gauge. The strain gauges picked up an initial failure within the specimen and then continued reading until total failure occurred.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
900-3300	11,400,000	0.15
3300-5700	4,440,000	0.05
5700-8100	2,900,000	0.06

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

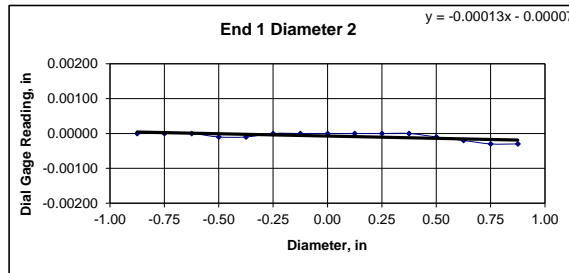
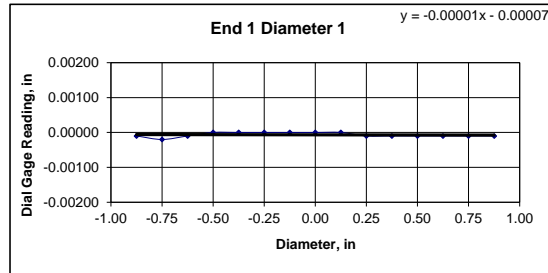


Client:	GZA GeoEnvironmental, Inc.	Test Date:	3/17/2017
Project Name:	Canaan Bridge Replacement	Tested By:	daa/trm
Project Location:	Canaan, ME	Checked By:	jsc
GTX #:	306143		
Boring ID:	BB-CCS-101, R2		
Sample ID:	2		
Depth:	21.92-22.29 ft		
Visual Description:	See photographs		

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

BULK DENSITY				DEVIATION FROM STRAIGHTNESS (Procedure S1)	
	1	2	Average	Maximum gap between side of core and reference surface plate: Is the maximum gap \leq 0.02 in.? YES	
Specimen Length, in:	4.34	4.33	4.34	Maximum difference must be $<$ 0.020 in.	
Specimen Diameter, in:	1.98	1.98	1.98	Straightness Tolerance Met? YES	
Specimen Mass, g:	658.61				
Bulk Density, lb/ft ³	188				
Length to Diameter Ratio:	2.2	Minimum Diameter Tolerance Met? YES			
		Length to Diameter Ratio Tolerance Met? YES			

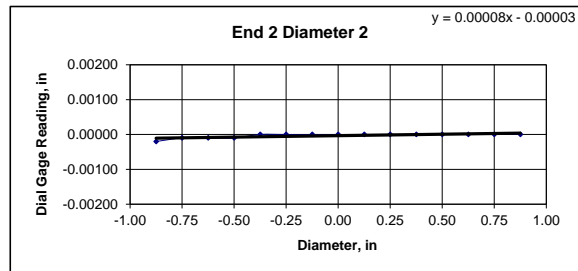
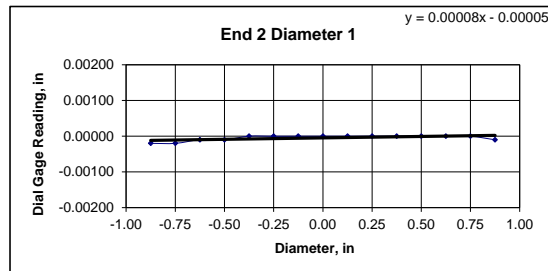
END FLATNESS AND PARALLELISM (Procedure FP1)																
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875	
Diameter 1, in	-0.00010	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00030	
Difference between max and min readings, in:																
0° =												0.00020		90° =		0.00030
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875	
Diameter 1, in	-0.00020	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	
Diameter 2, in (rotated 90°)	-0.00020	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	
Difference between max and min readings, in:																
0° =												0.0002		90° =		0.0002
Maximum difference must be < 0.0020 in.																
Flatness Tolerance Met? YES																



DIAMETER 1

End 1:	Slope of Best Fit Line	0.00001
	Angle of Best Fit Line:	0.00057
End 2:	Slope of Best Fit Line	0.00008
	Angle of Best Fit Line:	0.00458
Maximum Angular Difference:		0.00401

Parallelism Tolerance Met? YES
Spherically Seated



DIAMETER 2

End 1:	Slope of Best Fit Line	0.00013
	Angle of Best Fit Line:	0.00745
End 2:	Slope of Best Fit Line	0.00008
	Angle of Best Fit Line:	0.00458
Maximum Angular Difference:		0.00286

Parallelism Tolerance Met? YES
Spherically Seated

PERPENDICULARITY (Procedure P1)						(Calculated from End Flatness and Parallelism measurements above)	
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?	Maximum angle of departure must be \leq 0.25°	
Diameter 1, in	0.00020	1.980	0.00010	0.006	YES		
Diameter 2, in (rotated 90°)	0.00030	1.980	0.00015	0.009	YES	Perpendicularity Tolerance Met? YES	
END 2							
Diameter 1, in	0.00020	1.980	0.00010	0.006	YES		
Diameter 2, in (rotated 90°)	0.00020	1.980	0.00010	0.006	YES		

Client:	GZA GeoEnvironmental, Inc.
Project Name:	Canaan Bridge Replacement
Project Location:	Canaan, ME
GTX #:	306143
Test Date:	3/21/2017
Tested By:	daa/trm
Checked By:	jsc
Boring ID:	BB-CCS-101, R2
Sample ID:	2
Depth, ft:	21.92-22.29



After cutting and grinding

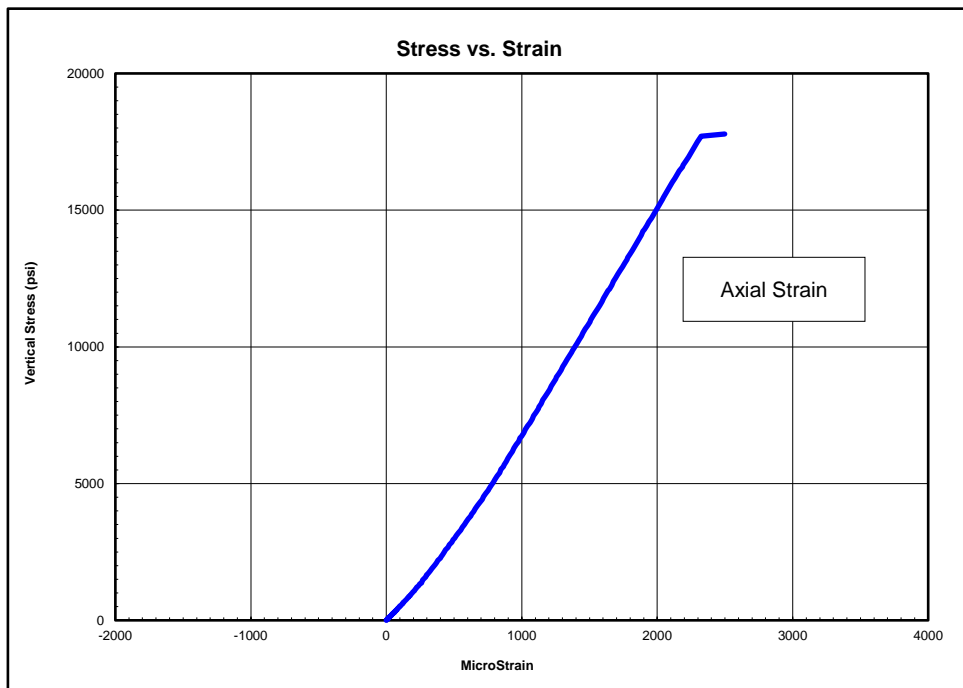


After break



Client:	GZA GeoEnvironmental, Inc.
Project Name:	Canaan Bridge Replacement
Project Location:	Canaan, ME
GTX #:	306143
Test Date:	3/21/2017
Tested By:	daa/trm
Checked By:	jsc
Boring ID:	BB-CCS-102, R2
Sample ID:	2
Depth, ft:	14.40-14.77
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

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



Peak Compressive Stress: 17,786 psi

Both lateral strain gauges failed to record meaningful data. Poisson's Ratio could not be determined.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1800-6500	7,200,000	---
6500-11300	8,320,000	---
11300-16000	8,370,000	---

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

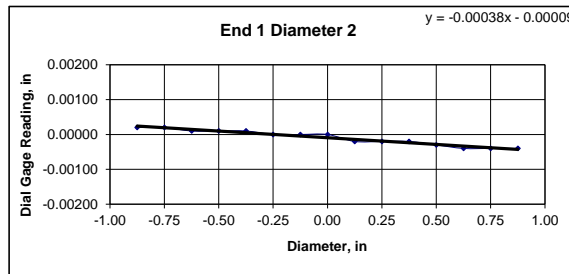
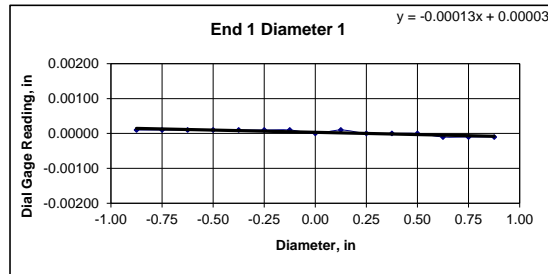


Client:	GZA GeoEnvironmental, Inc.	Test Date:	3/16/2017
Project Name:	Canaan Bridge Replacement	Tested By:	daa/trm
Project Location:	Canaan, ME	Checked By:	jsc
GTX #:	306143		
Boring ID:	BB-CCS-102, R2		
Sample ID:	2		
Depth:	14.40-14.77 ft		
Visual Description:	See photographs		

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

BULK DENSITY				DEVIATION FROM STRAIGHTNESS (Procedure S1)	
	1	2	Average	Maximum gap between side of core and reference surface plate: Is the maximum gap \leq 0.02 in.? YES	
Specimen Length, in:	4.32	4.32	4.32	Maximum difference must be $<$ 0.020 in.	
Specimen Diameter, in:	1.99	1.99	1.99	Straightness Tolerance Met? YES	
Specimen Mass, g:	633.38				
Bulk Density, lb/ft ³ :	179				
Length to Diameter Ratio:	2.2	Minimum Diameter Tolerance Met? YES			
		Length to Diameter Ratio Tolerance Met? YES			

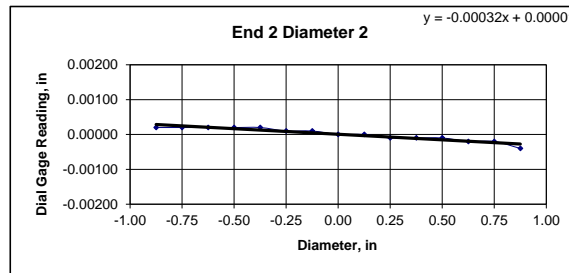
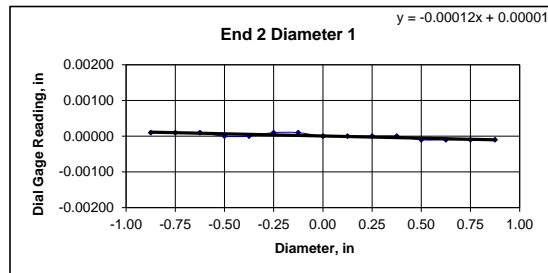
END FLATNESS AND PARALLELISM (Procedure FP1)															
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010	0.00000	0.00010	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010
Diameter 2, in (rotated 90°)	0.00020	0.00020	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	-0.00020	-0.00020	-0.00020	-0.00030	-0.00040	-0.00040	-0.00040
Difference between max and min readings, in:															
0° = 0.00020 90° = 0.00060															
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00010	0.00010	0.00010	0.00000	0.00000	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010	-0.00010
Diameter 2, in (rotated 90°)	0.00020	0.00020	0.00020	0.00020	0.00020	0.00010	0.00010	0.00000	0.00000	-0.00010	-0.00010	-0.00010	-0.00020	-0.00020	-0.00040
Difference between max and min readings, in:															
0° = 0.0002 90° = 0.0006															
Maximum difference must be $<$ 0.0020 in. Difference = \pm 0.00030															
Flatness Tolerance Met? YES															



DIAMETER 1

End 1:		
Slope of Best Fit Line	-0.00013	
Angle of Best Fit Line:	-0.00745	
End 2:		
Slope of Best Fit Line	-0.00012	
Angle of Best Fit Line:	-0.00688	
Maximum Angular Difference:	0.00057	

Parallelism Tolerance Met? YES
Spherically Seated



DIAMETER 2

End 1:		
Slope of Best Fit Line	-0.00038	
Angle of Best Fit Line:	-0.02177	
End 2:		
Slope of Best Fit Line	-0.00032	
Angle of Best Fit Line:	-0.01833	
Maximum Angular Difference:	0.00344	

Parallelism Tolerance Met? YES
Spherically Seated

PERPENDICULARITY (Procedure P1)						(Calculated from End Flatness and Parallelism measurements above)	
END 1		Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?	Maximum angle of departure must be \leq 0.25°
Diameter 1, in		0.00020	1.990	0.00010	0.006	YES	
Diameter 2, in (rotated 90°)		0.00060	1.990	0.00030	0.017	YES	Perpendicularity Tolerance Met? YES
END 2							
Diameter 1, in		0.00020	1.990	0.00010	0.006	YES	
Diameter 2, in (rotated 90°)		0.00060	1.990	0.00030	0.017	YES	



Client:	GZA GeoEnvironmental, Inc.
Project Name:	Canaan Bridge Replacement
Project Location:	Canaan, ME
GTX #:	306143
Test Date:	3/21/2017
Tested By:	daa/trm
Checked By:	jsc
Boring ID:	BB-CCS-102, R2
Sample ID:	2
Depth, ft:	14.40-14.77



After cutting and grinding



After break

WARRANTY and LIABILITY


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

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

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

Commonly Used Symbols


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

	195 Frances Avenue Cranston RI, 02910 Phone: (401)-467-6454 Fax: (401)-467-2398 thielsch.com <i>Let's Build a Solid Foundation</i>	Client Information: GZA GeoEnvironmental South Portland, ME PM: Blaine Cardali Assigned By: B. Cardali Collected By: B. Cardali	Project Information: Canna Bridge Replacement No. 2120, MEDOT WIN 21878.00 Canaan, ME GZA Project Number: 09.0025926.01 Summary Page: 1 of 1 Report Date: 09.09.2020
--	---	--	---

LABORATORY TESTING DATA SHEET, Report No.: 7420-H-203 Rev.1

Boring No.	Sample No.	Depth (Ft)	Laboratory No.	Identification Tests								Proctor / CBR / Permeability Tests								Laboratory Log and Soil Description
				As Received Water Content %	LL %	PL %	Gravel %	Sand %	Fines %	Org. %	G _s	Dry unit wt. pcf	Test Water Content %	γ_d MAX (pcf) $\frac{W_{opt}}{W_{opt}(\%)}(\text{Corr.})$	γ_d MAX (pcf) $\frac{W_{opt}}{W_{opt}(\%)}(\text{Corr.})$	Target Test Setup as % of Proctor	CBR @ 0.1"	CBR @ 0.2"	Permeability cm/sec	
				D2216	D4318		D6913			D2974	D854			D1557						
BB-CCS-201	1D	0-2	20-S-2571	9.4			7.5	67.6	24.9											Brown f-m SAND, some Silt, trace fine Gravel
BB-CCS-202	1D	0-2	20-S-2572	2.3			35.8	49.2	15.0											Brown Gravelly fine to coarse SAND, little Silt
BB-CCS-202	3D	4-6	20-S-2573	5.8			28.3	59.1	12.6											Brown f-c SAND, some f-c Gravel, little Silt
BB-CCS-202	6D	10-12	20-S-2574	12.5			49.5	33.3	17.2											Brown f-c GRAVEL, some f-c Sand, little Silt

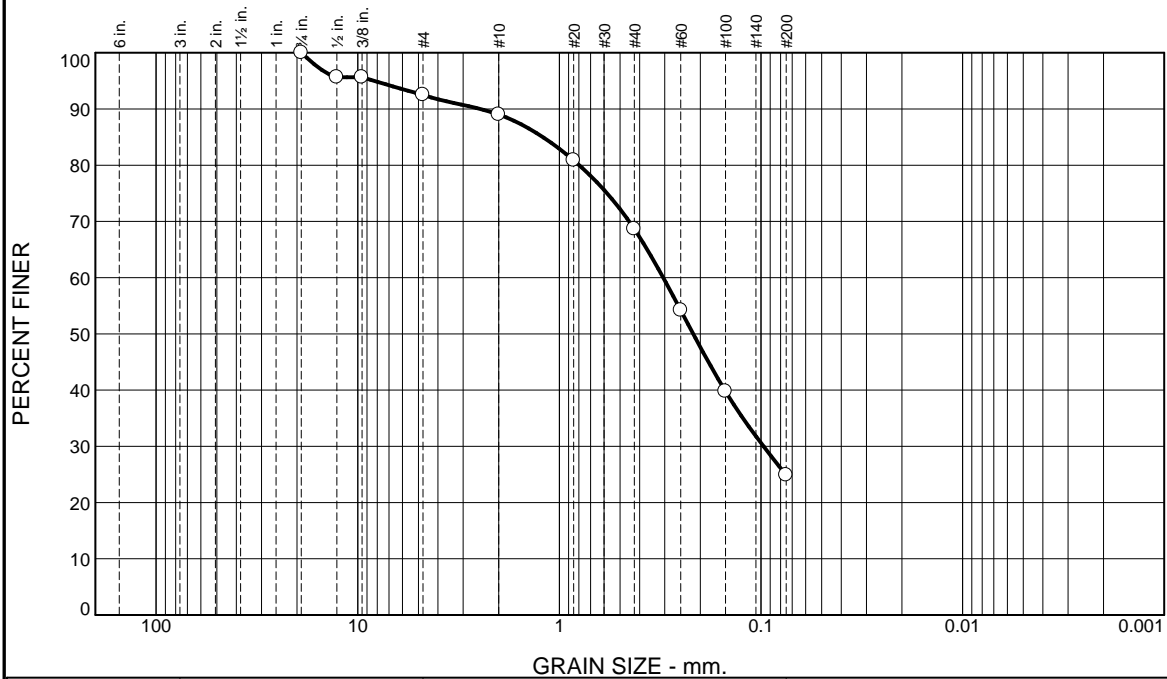
Date Received:
08.31.2020

Reviewed By:


Date Reviewed:
09.09.2020

This report only relates to items inspect and/or tested. No warranty, expressed or implied, is made.
This report shall not be reporduced, except in full, without prior written approval from the Agency, as defined in ASTM E329.

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	7.5	3.5	20.3	43.8	24.9	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.75"	100.0		
0.5"	95.7		
0.375"	95.7		
#4	92.5		
#10	89.0		
#20	80.9		
#40	68.7		
#60	54.2		
#100	39.8		
#200	24.9		

* (no specification provided)

Material Description

Brown f-m SAND, some Silt, trace fine Gravel

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

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

Coefficients

D₉₀= 2.4655 D₈₅= 1.2061 D₆₀= 0.3054
D₅₀= 0.2168 D₃₀= 0.0972 D₁₅=
D₁₀= C_u= C_c=

Remarks

Date Received: 08.31.2020 Date Tested: 09.02.2020

Tested By: RR

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: Borings Depth: 0-2'
Sample Number: BB-CCS-201 / 1D

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

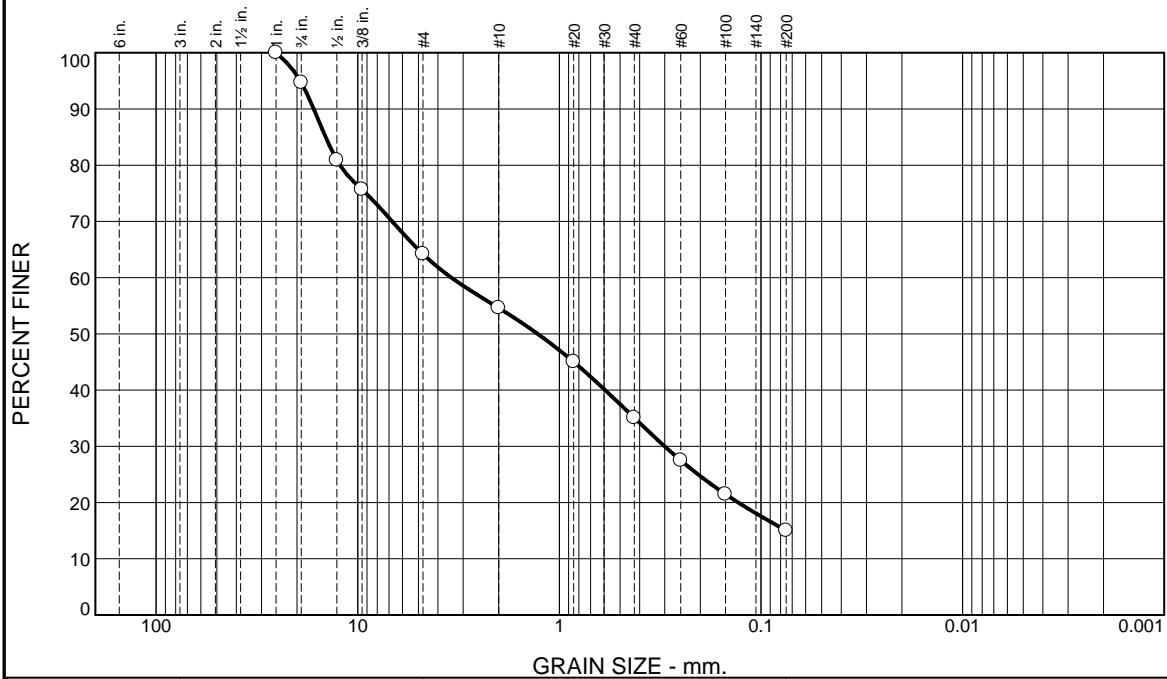
Client: GZA GeoEnvironmental

Project: Canaan Bridge Replacement No 2120 MEDOT WIN 21878.00
Canaan, ME

Project No: 09.0025926.01

Figure 20-S-2571

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	5.3	30.5	9.6	19.5	20.1	15.0	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	94.7		
0.5"	80.9		
0.375"	75.7		
#4	64.2		
#10	54.6		
#20	45.0		
#40	35.1		
#60	27.5		
#100	21.5		
#200	15.0		

* (no specification provided)

Material Description

Brown Gravelly fine to coarse SAND, little Silt

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

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

Coefficients

D₉₀= 16.5170 D₈₅= 14.4274 D₆₀= 3.4291
D₅₀= 1.2808 D₃₀= 0.3004 D₁₅=
D₁₀= C_u= C_c=

Remarks

Date Received: 08.31.2020 Date Tested: 09.02.2020

Tested By: RR

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: Borings Depth: 0-2'
Sample Number: BB-CCS-202 / 1D

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

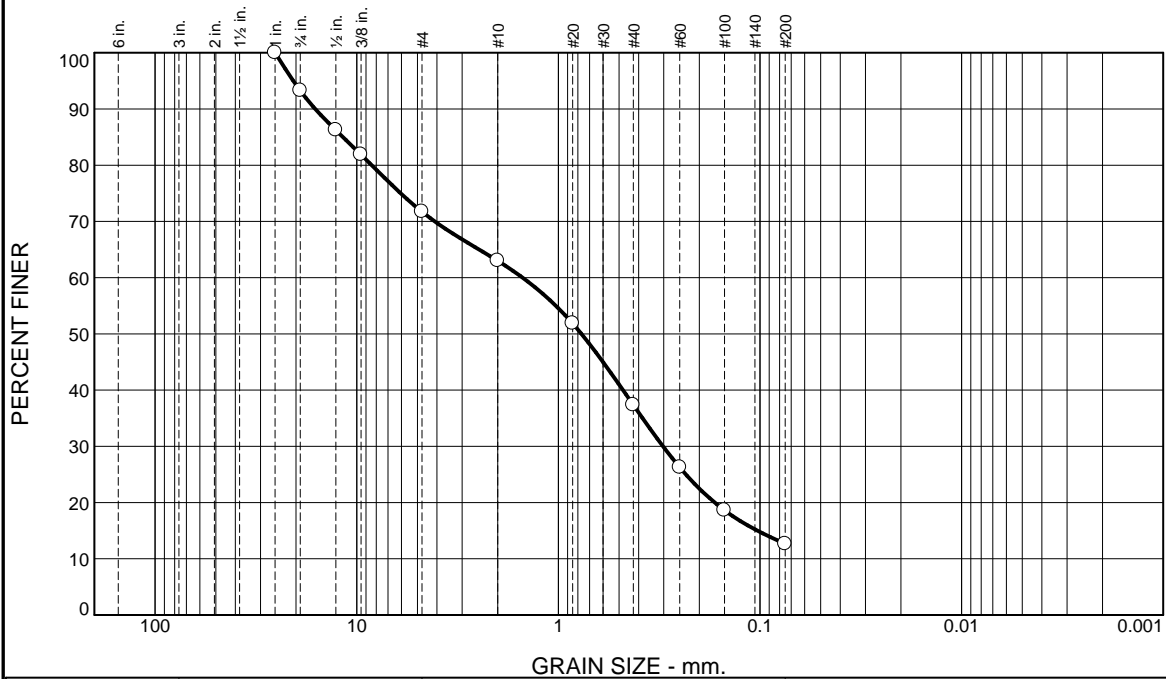
Client: GZA GeoEnvironmental

Project: Canaan Bridge Replacement No 2120 MEDOT WIN 21878.00
Canaan, ME

Project No: 09.0025926.01

Figure 20-S-2572

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	6.7	21.6	8.7	25.6	24.8	12.6	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	93.3		
0.5"	86.2		
0.375"	81.9		
#4	71.7		
#10	63.0		
#20	51.9		
#40	37.4		
#60	26.3		
#100	18.6		
#200	12.6		

* (no specification provided)

Material Description

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

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

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

Coefficients

D₉₀= 16.0531 D₈₅= 11.7022 D₆₀= 1.5047
D₅₀= 0.7680 D₃₀= 0.3026 D₁₅= 0.1029
D₁₀= C_u= C_c=

Remarks

Date Received: 08.31.2020 Date Tested: 09.02.2020

Tested By: RR

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: Borings Depth: 4-6'
Sample Number: BB-CCS-202 / 3D

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

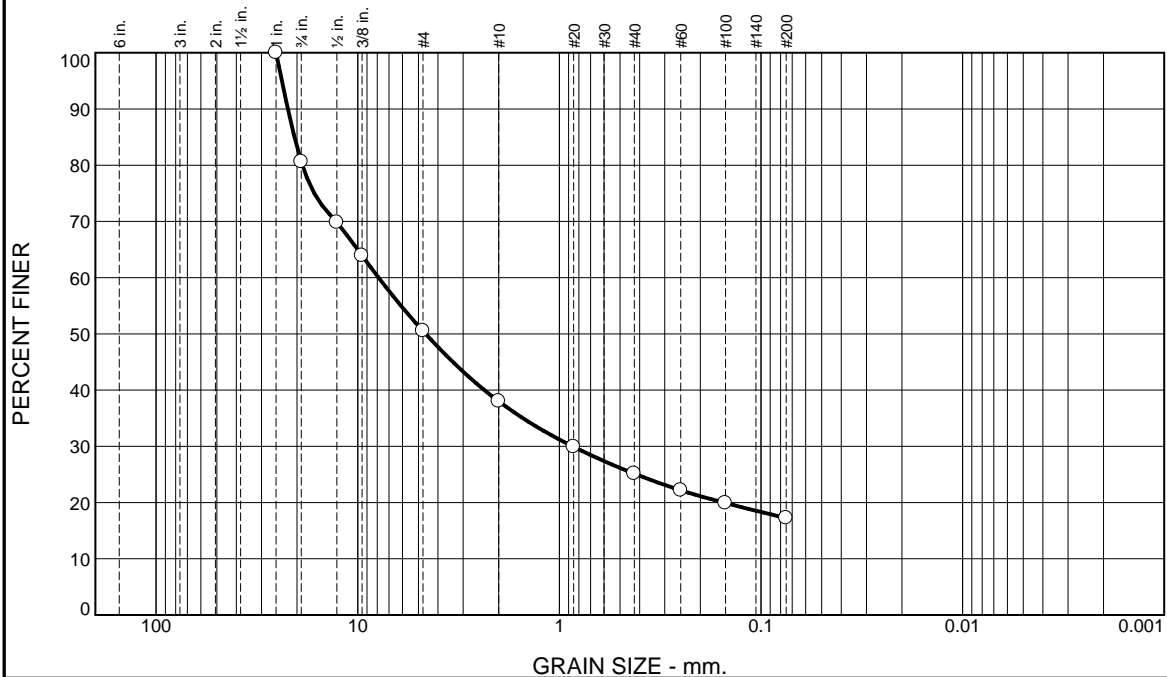
Client: GZA GeoEnvironmental

Project: Canaan Bridge Replacement No 2120 MEDOT WIN 21878.00
Canaan, ME

Project No: 09.0025926.01

Figure 20-S-2573

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	19.4	30.1	12.5	12.9	7.9	17.2	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	80.6		
0.5"	69.8		
0.375"	63.9		
#4	50.5		
#10	38.0		
#20	29.9		
#40	25.1		
#60	22.2		
#100	19.9		
#200	17.2		

* (no specification provided)

Material Description

Brown f-c GRAVEL, some f-c Sand, little Silt

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

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

Coefficients

D₉₀= 22.1993 D₈₅= 20.5919 D₆₀= 7.8810
D₅₀= 4.6070 D₃₀= 0.8585 D₁₅=
D₁₀= C_u= C_c=

Remarks

Date Received: 08.31.2020 Date Tested: 09.02.2020

Tested By: RR

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: Borings Depth: 10-12'
Sample Number: BB-CCS-202 / 6D

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental

Project: Canaan Bridge Replacement No 2120 MEDOT WIN 21878.00
Canaan, ME

Project No: 09.0025926.01

Figure 20-S-2574



03/08/21

**CANAAN BRIDGE NO. 2120 OVER CARRABASSETT STREAM
GEOTECHNICAL DESIGN REPORT**

09.0025926.01

APPENDIX F – CALCULATIONS

CALCULATIONS

Seismic



Design Maps Detailed Report

2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design (44.76225°N, 69.5612°W)

Site Class B – “Rock”

Article 3.4.1 — Design Spectra Based on General Procedure

Note: Maps in the 2009 AASHTO Specifications are provided by AASHTO for Site Class B.
Adjustments for other Site Classes are made, as needed, in Article 3.4.2.3.

From Figure 3.4.1-2 ^[1]	PGA = 0.075 g
From Figure 3.4.1-3 ^[2]	S _s = 0.159 g
From Figure 3.4.1-4 ^[3]	S ₁ = 0.047 g

Article 3.4.2.1 — Site Class Definitions

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class B, based on the site soil properties in accordance with Article 3.4.2.

Table 3.4.2.1-1 Site Class Definitions

SITE CLASS	SOIL PROFILE NAME	Soil shear wave velocity, \bar{v}_s , (ft/s)	Standard penetration resistance, \bar{N}	Soil undrained shear strength, \bar{s}_u , (psf)
A	Hard rock	$\bar{v}_s > 5,000$	N/A	N/A
B	Rock	$2,500 < \bar{v}_s \leq 5,000$	N/A	N/A
C	Very dense soil and soft rock	$1,200 < \bar{v}_s \leq 2,500$	$\bar{N} > 50$	>2,000 psf
D	Stiff soil profile	$600 \leq \bar{v}_s < 1,200$	$15 \leq \bar{N} \leq 50$	1,000 to 2,000 psf
E	Stiff soil profile	$\bar{v}_s < 600$	$\bar{N} < 15$	<1,000 psf
E	—	Any profile with more than 10 ft of soil having the characteristics: 1. Plasticity index $PI > 20$, 2. Moisture content $w \geq 40\%$, and 3. Undrained shear strength $\bar{s}_u < 500$ psf		
F	—	Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays ($H > 10$ feet of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays ($H > 25$ feet with plasticity index $PI > 75$) 4. Very thick soft/medium stiff clays ($H > 120$ feet)		
For SI: 1ft/s = 0.3048 m/s 1lb/ft ² = 0.0479 kN/m ²				

Article 3.4.2.3 — Site Coefficients

Table 3.4.2.3-1 (for F_{pga})—Values of F_{pga} as a Function of Site Class and Mapped Peak Ground Acceleration Coefficient

Site Class	Mapped Peak Ground Acceleration				
	$PGA \leq 0.10$	$PGA = 0.20$	$PGA = 0.30$	$PGA = 0.40$	$PGA \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See AASHTO Article 3.4.3				

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

For Site Class = B and PGA = 0.075 g, $F_{PGA} = 1.000$

Table 3.4.2.3-1 (for F_a)—Values of F_a as a Function of Site Class and Mapped Short-Period Spectral Acceleration Coefficient

Site Class	Spectral Response Acceleration Parameter at Short Periods				
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See AASHTO Article 3.4.3				

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

For Site Class = B and $S_s = 0.159$ g, $F_a = 1.000$

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

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

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

For Site Class = B and $S_1 = 0.047$ g, $F_v = 1.000$

Equation (3.4.1-1):

$$A_S = F_{PGA} \text{ PGA} = 1.000 \times 0.075 = 0.075 \text{ g}$$

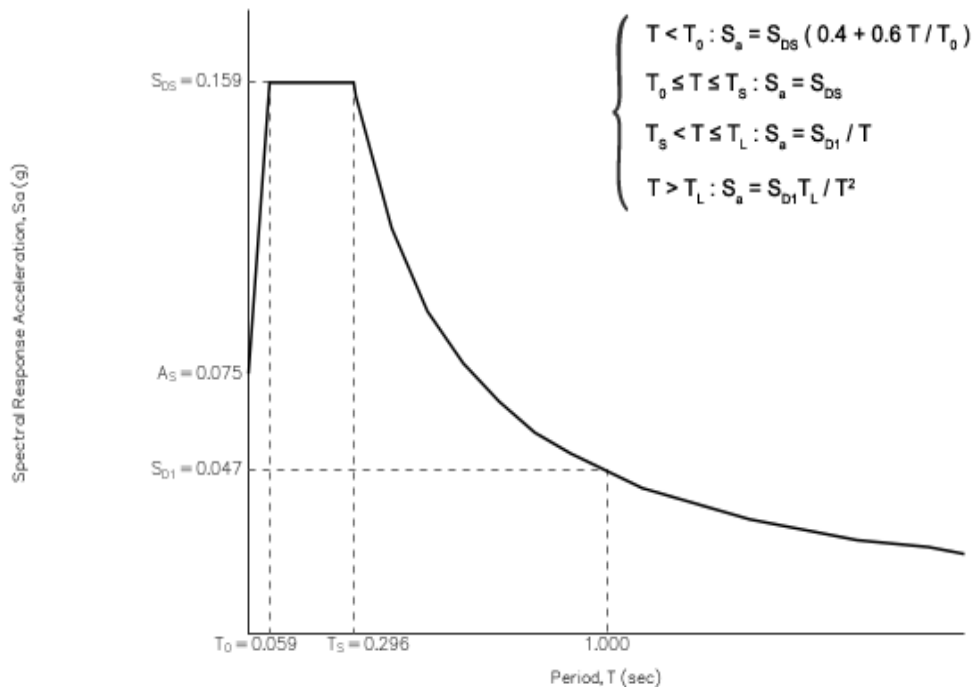
Equation (3.4.1-2):

$$S_{DS} = F_a S_S = 1.000 \times 0.159 = 0.159 \text{ g}$$

Equation (3.4.1-3):

$$S_{D1} = F_v S_1 = 1.000 \times 0.047 = 0.047 \text{ g}$$

Figure 3.4.1-1: Design Response Spectrum



Article 3.5 - Selection of Seismic Design Category (SDC)

Table 3.5-1—Partitions for Seismic Design Categories A, B, C, and D

VALUE OF S_{D1}	SDC
$S_{D1} < 0.15g$	A
$0.15g \leq S_{D1} < 0.30g$	B
$0.30g \leq S_{D1} < 0.50g$	C
$0.50g \leq S_{D1}$	D

For $S_{D1} = 0.047\text{ g}$, Seismic Design Category = A

Seismic Design Category \equiv “the design category in accordance with Table 3.5-1” = A

References

1. *Figure 3.4.1-2*: <https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-2.pdf>
2. *Figure 3.4.1-3*: <https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-3.pdf>
3. *Figure 3.4.1-4*: <https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-4.pdf>

CALCULATIONS

Frost Penetration

Figure 5-1 Maine Design Freezing Index Map

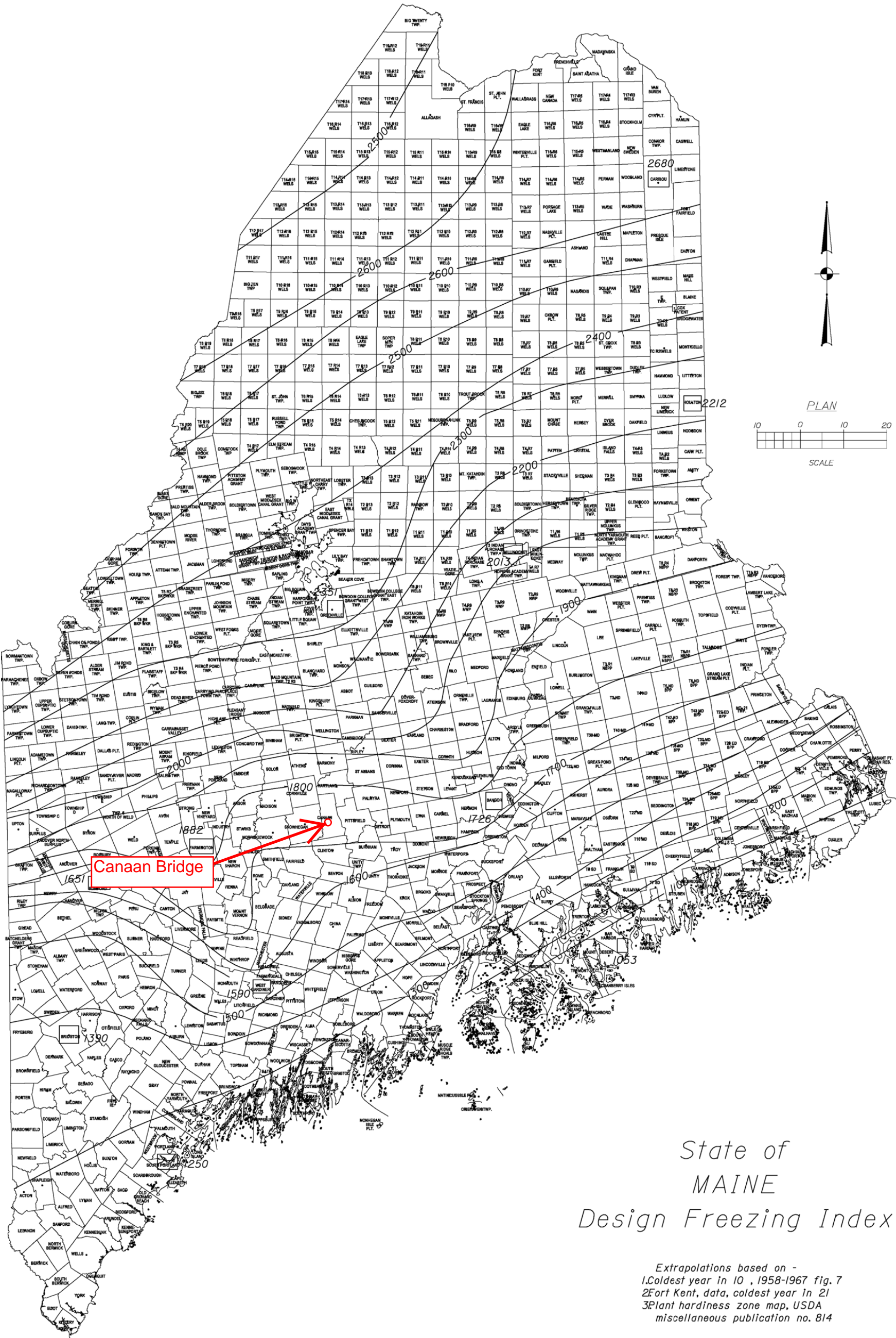


Table 5-1 Depth of Frost Penetration

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

1740

88.5 = 7.4'

- Notes: 1. w = water content
2. Where the Freezing Index and/or water content is between the presented values, linear interpretation may be used to determine the frost penetration.

Existing Fill soils Glacial Till deposits are anticipated to be present at the abutments near the elevation of the footings. The material is coarse-grained with water contents less than 10% to 15%. Based on the MaineDOT BDG, Section 5.2.1 and a Freezing Index of 1740 the estimated depth of frost penetration is 88.5 inches.

CALCULATIONS

Bedrock Bearing Resistance



GZA
GeoEnvironmental, Inc
 707 Sable Oaks Drive, Suite 150
 South Portland, Maine 04106
 207-879-9190
 Fax 207-879-0099
<http://www.gza.com>

Engineers and
 Scientists

Canaan Bridge Replacement, Canaan, ME
 JOB: 09.0025926.01
 SUBJECT: Bearing Resistance on Bedrock
 SHEET: 1 OF 8
 CALCULATED BY: E. Tome 9/10/20
 CHECKED BY: B. Cardali 9/15/20
 REVIEWED BY: C. Snow 9/29/20

Objective

Assess nominal and factored bearing resistance of a foundation on rock based on support in SLATE from borings BB-CCS-101, -102, -201, and -202.

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 9th 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 9th 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 Canaan Bridge Project in Canaan, ME. This calculation is based on the data from borings BB-CCS-101, -102, -201, and -202.

Bedrock Quality

Boring ID	Core Run	Length of Core Run (ft)	Rec (%)	RQD %	Joint Spacing Desc.	Joint Spacing (in)	Aperture Desc.	Joint Aperture (in)	Joint Weathering
BB-CCS-101	R1	3.0	100%	8%	Very Close to Close	0.75-8	Tight to Partially Open	0.004-0.02	Fresh
BB-CCS-101	R2	5.0	100%	58%	Very Close to Moderate	0.75-24	Tight to Partially Open	0.004-0.02	Fresh
BB-CCS-101	R3	3.5	100%	48%	Very Close to Moderate	0.75-24	Tight to Partially Open	0.004-0.02	Fresh
BB-CCS-102	R1	3.3	97%	34%	Very Close to Close	0.75-8	Tight to Partially Open	0.004-0.02	Fresh
BB-CCS-102	R2	3.2	100%	47%	Close to Moderate	2.5-24	Tight to Partially Open	0.004-0.02	Fresh
BB-CCS-102	R3	3.7	101%	87%	Close to Moderate	2.5-24	Tight to Partially Open	0.004-0.02	Fresh
BB-CCS-201	R1	3.7	86%	23%	Extremely Close to Close	<0.75-8	Tight to Open	0.004-0.1	Fresh to Slightly Weathered
BB-CCS-201	R2	2.3	100%	14%	Extremely Close to Close	<0.75-8	Tight to Open	0.004-0.1	Fresh to Slightly Weathered
BB-CCS-201	R3	2.3	93%	0%	Extremely Close to Close	<0.75-8	Open	0.02-0.1	Fresh to Slightly Weathered
BB-CCS-201	R4	1.7	75%	0%	Extremely Close to Close	<0.75-8	Open	0.02-0.1	Fresh to Slightly Weathered
BB-CCS-202	R1	2.1	88%	24%	Extremely Close to Close	<0.75-8	Tight to Partially Open	0.004-0.02	Fresh
BB-CCS-202	R2	3.5	100%	62%	Close to Moderate	2.5-24	Tight to Partially Open	0.004-0.02	Fresh
BB-CCS-202	R3	2.5	100%	60%	Extremely Close to Moderate	<0.75-24	Tight to Partially Open	0.004-0.02	Fresh
BB-CCS-202	R4	2.9	100%	57%	Close to Moderate	2.5-24	Tight to Partially Open	0.004-0.02	Fresh
				Avg RQD	37%				
				St. Dev RQD	26%				

RQD between 11% and 63% representative of rock for BB-CCS-101, -102, -201, and -202 (mean-1 std deviation to mean).



GZA
GeoEnvironmental, Inc
 707 Sable Oaks Drive, Suite 150
 South Portland, Maine 04106
 207-879-9190
 Fax 207-879-0099
<http://www.gza.com>

Engineers and
 Scientists

Canaan Bridge Replacement, Canaan, ME
 JOB: 09.0025926.01
 SUBJECT: Bearing Resistance on Bedrock
 SHEET: 2 OF 8
 CALCULATED BY: E. Tome 9/10/20
 CHECKED BY: B. Cardali 9/15/20
 REVIEWED BY: C. Snow 9/29/20

Bedrock Strength

Boring ID	Core Run	Elev. (ft)		LAB						Rock Type
		Top	Bottom	Depth of Sample (ft)	Depth of Sample into Rock (ft)	Elev Top of Sample (ft)	UCS (psi)	Modulus (ksi)	Unit Wt (pcf)	
BB-CCS-101	R2	213.7	208.7	21.9	4.4	212.8	9,022	2,900	188.0	SLATE
BB-CCS-102	R2	221.6	218.4	14.4	4.8	220.7	17,786	8,370	179.0	SLATE

To be conservative, utilize the lowest strength from lab testing for design, 9 ksi.

2. Calculation of Rock Mass Rating (RMR)

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

Parameter 1- Uniaxial Compressive Strength

$$\sigma_{u,r} := 9 \text{ ksi} = 1296 \text{ ksf} \quad \text{Representative unconfined compressive strength of rock at BB-CCS-101.}$$

From AASHTO LRFD Table 10.4.6.4-1

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

Parameter 2- Drill Core Quality

Representative RQD from table above, average RQD of 37%

From AASHTO LRFD Table 10.4.6.4-1

$$\text{Relative Rating} \quad RR_2 := 8$$

Parameter 3- Spacing of Joints

From Boring Logs, generally very close to moderately spaced = < .75 in to 2 feet, Typical spacing was 6 in. to 10 in.

From AASHTO LRFD Table 10.4.6.4-1

$$\text{Relative Rating} \quad RR_3 := 10$$

Parameter 4- Condition of Joints

From boring logs, generally hard joint walls and rough to smooth surface, with joint separation less than 0.05 in., and described fresh to slightly weathered.

From AASHTO LRFD Table 10.4.6.4-1

$$\text{Relative Rating} \quad RR_4 := 20$$



GZA
GeoEnvironmental, Inc
707 Sable Oaks Drive, Suite 150
South Portland, Maine 04106
207-879-9190
Fax 207-879-0099
<http://www.gza.com>

*Engineers and
Scientists*

Canaan Bridge Replacement, Canaan, ME
JOB: 09.0025926.01
SUBJECT: Bearing Resistance on Bedrock
SHEET: 3 OF 8
CALCULATED BY: E. Tome 9/10/20
CHECKED BY: B. Cardali 9/15/20
REVIEWED BY: C. Snow 9/29/20

Parameter 5- Ground Water Conditions

Hydrostatic Conditions- Water under moderate pressure

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_5 := 4$

Parameter 6-Adjustment for joint orientation

The joint sets are generally moderately dipping to high angle and generally rough to smooth and partially open.
Therefore the joint orientation is considered Fair.

From AASHTO LRFD Table 10.4.6.4-2

Relative Rating $RR_6 := -7$

Total RMR Rating

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

$$RMR = 42$$

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

3. Determine Rock Property Constants s and m

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

Schist is categorized as rock type B, Lithified argillaceous rocks, RMR=42, using s and m values interpolated from the logarithmic trend of plotted values from AASHTO Table 10.4.6.4-4 (plots on sheet 8).

$$m := 0.16$$

$$s := 0.000063$$



GZA
GeoEnvironmental, Inc
 707 Sable Oaks Drive, Suite 150
 South Portland, Maine 04106
 207-879-9190
 Fax 207-879-0099
 http://www.gza.com

Engineers and
 Scientists

Canaan Bridge Replacement, Canaan, ME

JOB: 09.0025926.01

SUBJECT: Bearing Resistance on Bedrock

SHEET: 4 OF 8

CALCULATED BY: E. Tome 9/10/20

CHECKED BY: B. Cardali 9/15/20

REVIEWED BY: C. Snow 9/29/20

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

From Wyllie "Foundations on Rock"

Eq. 5.4 Pg.138

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

Where

$$C_{fl} := 1.0$$

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

$$s = 0.000063$$

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

$$m = 0.16$$

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

$$\sigma_{u,r} = 9 \cdot \text{ksi}$$

For conservatism, assume long strip, lowest C_{fl} .

Nominal Bearing Resistance

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

$$q_n := 58 \cdot \text{ksf}$$

Factored Bearing Resistance

Bearing Resistance Factor is specified in Table 10.5.5.2.2-1

$$\phi_b := 0.45 \quad \text{Footings on rock}$$

$$q_R := \phi_b \cdot q_n$$

$$q_R := 26 \cdot \text{ksf}$$



GZA
GeoEnvironmental, Inc
 707 Sable Oaks Drive, Suite 150
 South Portland, Maine 04106
 207-879-9190
 Fax 207-879-0099
<http://www.gza.com>

Engineers and
 Scientists

Canaan Bridge Replacement, Canaan, ME

JOB: 09.0025926.01

SUBJECT: Bearing Resistance on Bedrock

SHEET: 5 OF 8

CALCULATED BY: E. Tome 9/10/20

CHECKED BY: B. Cardali 9/15/20

REVIEWED BY: C. Snow 9/29/20

➔ Reference: I:\Mathcad\units.xmcd

10-22

AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

Table 10.4.6.4-1 Geomechanics Classification of Rock Masses.

Parameter			Ranges of Values						
1	Strength of intact rock material	Point load strength index	>175 ksf	85–175 ksf	45–85 ksf	20–45 ksf	For this low range, uniaxial compressive test is preferred		
		Uniaxial compressive strength	>4320 ksf	2160–4320 ksf	1080–2160 ksf	520–1080 ksf	215–520 ksf	70–215 ksf	20–70 ksf
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>10 ft.	3–10 ft.	1–3 ft.	2 in.–1 ft.	<2 in.		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		<ul style="list-style-type: none">• Very rough surfaces• Not continuous• No separation• Hard joint wall rock	<ul style="list-style-type: none">• Slightly rough surfaces• Separation <0.05 in.• Hard joint wall rock	<ul style="list-style-type: none">• Slightly rough surfaces• Separation <0.05 in.• Soft joint wall rock	<ul style="list-style-type: none">• Slicken-sided surfaces or• Gouge <0.2 in. thick or• Joints open 0.05–0.2 in.• Continuous joints	<ul style="list-style-type: none">• Soft gouge >0.2 in. thick or• Joints open >0.2 in.• Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Ground water conditions (use one of the three evaluation criteria as appropriate to the method of exploration)	Inflow per 30 ft. tunnel length	None	<400 gal./hr.	400–2000 gal./hr.	>2000 gal./hr.			
		Ratio = joint water pressure/major principal stress	0	0.0–0.2	0.2–0.5	>0.5			
		General Conditions	Completely Dry	Moist only (interstitial water)	Water under moderate pressure	Severe water problems			
	Relative Rating		10	7	4	0			



GZA
GeoEnvironmental, Inc
707 Sable Oaks Drive, Suite 150
South Portland, Maine 04106
207-879-9190
Fax 207-879-0099
<http://www.gza.com>

*Engineers and
Scientists*

Canaan Bridge Replacement, Canaan, ME

JOB: 09.0025926.01

SUBJECT: Bearing Resistance on Bedrock

SHEET: 6 OF 8

CALCULATED BY: E. Tome 9/10/20

CHECKED BY: B. Cardali 9/15/20

REVIEWED BY: C. Snow 9/29/20

Table 10.4.6.4-2 Geomechanics Rating Adjustment for Joint Orientations.

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

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

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



GZA
GeoEnvironmental, Inc
 707 Sable Oaks Drive, Suite 150
 South Portland, Maine 04106
 207-879-9190
 Fax 207-879-0099
<http://www.gza.com>

Engineers and
 Scientists

Canaan Bridge Replacement, Canaan, ME
 JOB: 09.0025926.01
 SUBJECT: Bearing Resistance on Bedrock
 SHEET: 7 OF 8
 CALCULATED BY: E. Tome 9/10/20
 CHECKED BY: B. Cardali 9/15/20
 REVIEWED BY: C. Snow 9/29/20

10-24

AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

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

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



GZA
GeoEnvironmental, Inc
707 Sable Oaks Drive, Suite 150
South Portland, Maine 04106
207-879-9190
Fax 207-879-0099
<http://www.gza.com>

*Engineers and
Scientists*

Canaan Bridge Replacement, Canaan, ME

JOB: 09.0025926.01

SUBJECT: Bearing Resistance on Bedrock

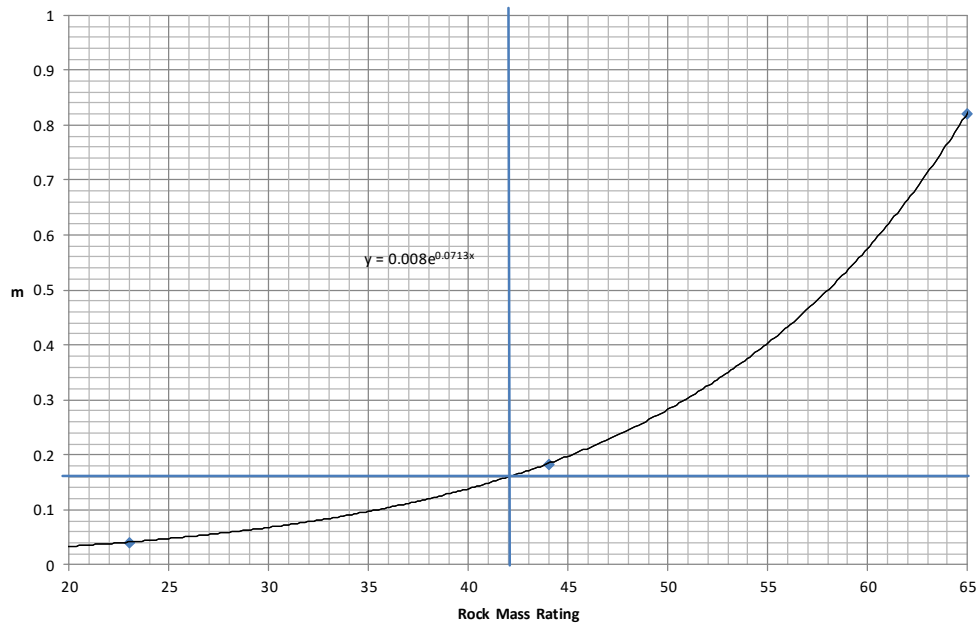
SHEET: 8 OF 8

CALCULATED BY: E. Tome 9/10/20

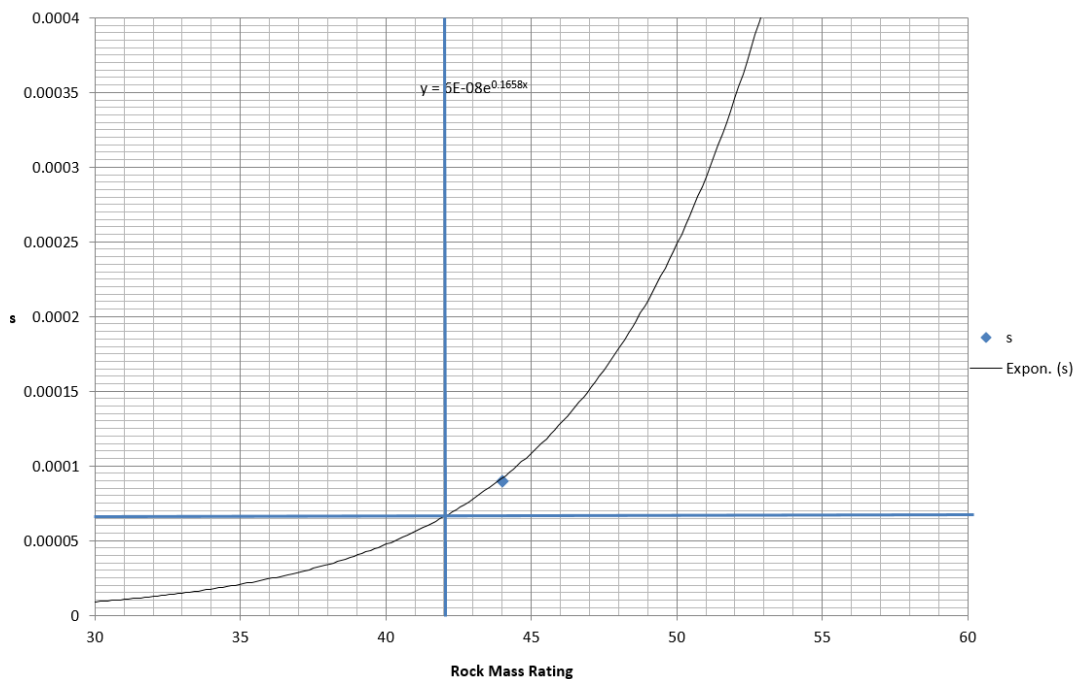
CHECKED BY: B. Cardali 9/15/20

REVIEWED BY: C. Snow 9/29/20

m for Rock Type B



s for Rock Type B



CALCULATIONS

Lateral Earth Pressures



GZA
GeoEnvironmental, Inc
 707 Sable Oaks Drive
 Suite 150
 South Portland, Maine 04106
 207-879-9190
 Fax 207-879-0099

Engineers and
 Scientists

JOB: 09.0025926.01 Canaan Br.
 SUBJECT: Lateral Earth Pressures
 SHEET: 1 OF 1
 CALCULATED BY B.Cardali 11/10/2020
 CHECKED BY C. Snow 11/10/2020

Subject:

Evaluate lateral earth pressure coefficients for proposed cast-in-place abutment with a semi-integral backwall

References:

1. MaineDOT Bridge Design Guide, Chapters 3 and 5 (BDG)
2. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 8th edition, 2017. (AASHTO LRFD).

Input Parameters:

$\beta := 0\text{deg}$	Angle of backfill to the horizontal
$\theta := 90\text{deg}$	Angle of backface of wall to the horizontal
$\delta_f := 19.5\text{deg}$	Average value, formed concrete, clean sand/silty sand (AASHTO LRFD Table 3.11.5.3-1)
$\phi := 32\text{deg}$	Effective angle of internal friction (backfill material, granular backfill MaineDOT 703.19)

Earth Pressure Coefficients:

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

Passive Earth Pressure (End Diaphragms)

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

Coloumb Passive Earth Pressure Coefficient

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

$K_{pc} = 6.73$

Rankine Passive Earth Pressure Coefficient

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

$K_{pr} = 3.25$

Active Earth Pressure (Abutments and Wingwalls)

Rankine Active Earth Pressure Coefficient

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

$K_{ar} = 0.31$