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GEOTECHNICAL DESIGN REPORT

CROMWELL BROOK No. 3 BRIDGE No. 0452

LEDGELAWN AVENUE EXTENSION
OVER CROMWELL BROOK
BAR HARBOR, MAINE

November 2024
File No. 09.0026155.01

Prepared for:
Vanasse Hangen Brustlin, Inc.
Scarborough, Maine

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VIA EMAIL

November 7, 2024
File No. 09.0026155.01

Mr. Carl Ayers, P.E.
Vanasse Hangen Brustlin, Inc.
500 Southborough Drive, Suite 105B
South Portland, ME 04106

Re: Geotechnical Design Report
Cromwell Brook No. 3 Bridge No. 0452
Ledgelawn Avenue Extension over Cromwell Brook
Maine Department of Transportation WIN 26574.00
Bar Harbor, Maine

Dear Carl:

We are pleased to provide this Geotechnical Design Report (GDR) to Vanasse Hangen Brustlin, Inc. (VHB) for the subject project. Our work was completed in accordance with the Agreement for Professional Services between VHB and GZA GeoEnvironmental, Inc. (GZA) dated April 15, 2024, which incorporates our January 22, 2024 proposal, and the *Limitations* included in **Appendix A** of this report. GZA is providing geotechnical engineering services as a Subconsultant to VHB, who is under contract with the Maine Department of Transportation for design of the proposed bridge replacement.

It has been a pleasure serving VHB 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.
Senior Project Manager

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Attachment: Geotechnical Design Report

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1.0 INTRODUCTION

This report presents the results of the geotechnical evaluation by GZA GeoEnvironmental, Inc. (GZA) for the subject project. Our work was completed in accordance with the Agreement for Professional Services between Vanasse Hangen Brustlin, Inc. (VHB) and GZA dated April 15, 2024, which incorporates our January 22, 2024 proposal, and the *Limitations* included in **Appendix A** of this report. GZA is providing geotechnical engineering services as a Subconsultant to VHB, who is under contract with the Maine Department of Transportation (MaineDOT) for design of the proposed bridge replacement.

1.1 BACKGROUND

The project includes replacement of the Cromwell Brook No. 3 Bridge No. 0452 carrying Ledgelawn Avenue Extension over Cromwell Brook in Bar Harbor, Maine, the location of which is shown on **Figure 1.** Built in 1945, the original bridge is an approximately 29-foot-long, single-span bridge with reinforced concrete girders supported on stone masonry abutments. In 2020, an approximately 40-foot-long, single-span Mabey steel truss bridge was constructed directly over the existing bridge due to concerns with the structural integrity of the substructures. The 2020 inspection report notes that there is undermining at the south abutment, movement of stones under the concrete cap that the beams are set on, and signs of movement along all four of the wingwalls.

The selected bridge alternative is a single span bridge with a span length of 50 feet and a width of approximately 24 feet, and a 20-degree skew. The superstructure is considered detail-build and will have a composite concrete deck. The plans indicate three superstructure alternatives including: Next Beams, composite tub girders, press-brake formed tub girders, and a slab beam alternative. The proposed bridge centerline will be approximately 6 feet east (downstream) from the existing bridge. The new abutments are anticipated to be designed with semi-integral abutment substructures supported on spread footings bearing directly on bedrock. We understand that a temporary single lane bridge with alternating traffic will be used during construction.

1.2 OBJECTIVES AND SCOPE OF SERVICES

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

- Conducted a site visit to observe surficial and reviewed mapped surficial and bedrock geology of the site;
- Reviewed existing subsurface data and as-built plans;
- Coordinated and observed subsurface exploration programs to evaluate subsurface conditions and collect samples for laboratory testing;
- Conducted laboratory testing programs to evaluate engineering and index properties of the site soils and bedrock;
- Conducted final design geotechnical engineering analyses to evaluate feasible foundation types; final design parameters; considerations for widened embankments; and seismic design parameters;

- Developed geotechnical construction considerations; and
- Prepared this geotechnical design report summarizing our findings and design recommendations.

2.0 SUBSURFACE EXPLORATIONS

GZA completed a subsurface exploration program in 2022 consisting of three (3) test borings designated as BB-BHCB-101 through -103 and one (1) auger probe designated as BB-BHCB-104. One boring was drilled behind each existing abutment through the roadway (BB-BHCB-101 and -103) and one probe was drilled off-alignment beside each existing abutment (BB-BHCB-102 and -104). The explorations were drilled using a track-mounted drill rig. The as-drilled boring locations were surveyed by MaineDOT and are shown on **Figure 2**. Elevations referenced in this report are in feet and refer to the North American Vertical Datum of 1988 (NAVD88).

The borings were drilled to depths of approximately 18.0 to 22.0 feet below ground surface (bgs). New England Boring Contractors (NEBC) of Hermon, Maine provided drilling services and coordinated utility clearance. The drilling was completed between July 13, 2022 and August 11, 2022. GZA personnel monitored the drilling work and prepared logs of each boring that are included in **Appendix B**.

The borings were drilled using 3- and 4-inch driven casing and drive-and-wash drilling techniques. Standard penetration testing (SPT) and split-spoon sampling were performed at 5-foot typical intervals in the overburden using a 24-inch-long, 1-3/8-inch inside-diameter sampler, driven with an automatic hammer with hammer efficiencies at the time of drilling of 0.86 and 0.92 as provided in the boring logs. Bedrock cores were obtained using NX2 coring equipment in each test boring. Upon completion of the borings, NEBC backfilled the holes with drill spoils and placed cold patch asphalt at the surface for borings BB-BHCB-101 and -103.

Auger probe BB-BHCB-104 was advanced with solid stem augers (SSA) and no soil samples were collected from the exploration. The augers struck an apparent waterline at a depth of approximately 7.2 feet bgs. NEBC stopped drilling, and the Town of Bar Harbor handled repairing the water pipe and abandonment of the exploration after GZA had left the site.

3.0 LABORATORY TESTING

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

<u>Soil</u>

- Four (4) gradation analysis / MaineDOT Frost Classification / AASHTO Soil Classifications;
- Four (4) moisture content tests; and
- Two (2) hydrometer tests.

Rock

- Two (2) unconfined compressive strength / secant modulus tests; and
- Four (4) point-load tests (2 axial and 2 diametrical).

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

4.0 SUBSURFACE CONDITIONS

4.1 SURFICIAL AND BEDROCK GEOLOGY

Based on available geologic mapping[1], the surficial units in the vicinity of the bridge consist of artificial fill, marine shoreline deposits, glacial till, and bedrock. Marine shoreline deposits (Pms) are described as stratified pebble to boulder gravel and sand that has layering dipping downslope. Glacial Till (Pt) consists of a poorly sorted mixture of gravel, sand, silt and clay. Bedrock is designated as areas where the land surface is a combination of knobs of bare or vegetation-covered bedrock ledge and a thin, 1- to 3-foot-thick layer of glacial till overlying the bedrock between knobs.

Available bedrock geologic mapping[2] indicates that the site is near a contact between the Bar Harbor formation (described as layers of light gray to lavender quartzite and argillite and light gray metarhyolite tuff) and a complex contact zone of Silurian-age rock, designated as a Shatter zone which forms a belt around the eastern and southern margins of the Cadillac Granite. The portion of the Shatter zone closest to the Bar Harbor formation is described as extensively broken-up, tightly packed, deformed Bar Harbor formation.

4.2 SUBSURFACE PROFILE

Four soil units were encountered above bedrock at the site: Fill, Topsoil, Silt and Sand, and Gravel and Sand. At borings BB-BHCB-101 and -103 the soil was beneath 4 to 5 inches of asphalt, and a 3-inch-thick layer of buried asphalt was encountered in BB-BHCB-103 at a depth of 4.5 feet bgs. 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**.

[1] Braun, Duane D., 2015, Surficial materials of the southwestern portion of the Bar Harbor quadrangle, Maine: Maine Geological Survey, Open-File Map 15-16, map, scale 1:24,000.

[2] Braun, Duane D., 2019, Bedrock geology of the Southwestern Portion of the Bar Harbor quadrangle, Maine: Maine Geological Survey, Open-File Map 19-13, color map, scale 1:24,000. Maine Geological Survey Maps. 2115. https://digitalmaine.com/mgs_maps/2115

Soil Unit	Approximate Encountered Thickness (ft)	Generalized Description
Fill	5.3 to 7.4	Brown, loose to very dense, Gravelly fine to coarse SAND, trace to little silt. (USCS: SW, SM). MaineDOT Frost Classification = II *Encountered in BB-BHCB-101 and -103.* A hydrocarbon odor was noted in samples collected from 3 to 7 feet bgs in BB-BHCB-103.
Topsoil	0.3	Dark brown, loose, Silty fine SAND with roots. (USCS: SM) *Encountered in BB-BHCB-102 only.*
Silt and Sand	2.9 to 11.7	Layered brown-orange, medium stiff, CLAYEY SILT, little fine to medium sand, trace gravel; and Gray, medium dense, fine to coarse SAND, some Silt to some Clayey Silt, little to some Gravel. (USCS: ML, SM). MaineDOT Frost Classification = II, and III *Encountered in BB-BHCB-101 and -102.*
Gravel and Sand	3.2	Olive-brown, dense, GRAVEL, some fine to coarse Sand, trace Silt. (USCS: GW-GM). MaineDOT Frost Classification = I *Encountered in BB-BHCB-101 only.*
Estimated Top of Bedrock	Abutment 1: Approx. El. 39.8 to El. 33.2 (11.7 to 12.0 feet bgs) Abutment 2: Approx. El. 43.8 (7.8 feet bgs)	

Detailed descriptions of the materials encountered at specific locations are provided on the boring logs in **Appendix B**. An interpretive subsurface profile based on the test borings is presented as **Figure 2**. The approximate thickness and elevation of each stratum is summarized on the attached **Table 1**.

4.2.1 Bedrock

Bedrock was cored in each test boring and was described as Metasandstone. Photographic logs of the recovered rock core specimens are included in **Appendix D**. Metasandstone was typically described as hard, fresh to slightly weathered, aphanitic to medium grained and grey. Primary joints are very close to moderately spaced, low angle to vertical, planar to undulating, smooth to rough, fresh to decomposed, tight to wide. A diorite intrusion was encountered in BB-BHCB-103 R2 from a depth of 16.0 to 18.0 feet, and the majority of the core samples had calcite stringers.

Unconfined compressive strength (UCS) testing was conducted on two samples of fresh to slightly weathered 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-BHCB-101	14.3	2.6	37.2	26,522	4,730	169.2	METASANDSTONE
BB-BHCB-103	8.0	0.1	43.6	10,372	3,200	170.9	METASANDSTONE

Axial and diametral point load testing were conducted on two samples from borings BB-BHCB-102 and -103. The size-corrected point load index, $I_{s(50mm)}$, ranged from 234 to 2,266 psi, which suggests an unconfined compressive strength ranging from approximately 5,600 to 54,400 psi based on correlation factors between $I_{s(50mm)}$ and UCS from ASTM D5731.

4.2.2 Groundwater

Groundwater depth was measured in boring BB-BHCB-101 and BB-BHCB-102 at approximately 10.0 and 3.2 feet bgs, corresponding to El. 41.5 and El. 42.0, respectively, which is approximately at the brook level. Boring BB-BHCB-103 was dry at a depth of 8.0 feet bgs (El. 43.6) after removal of casing and borehole collapse to a depth of 8.0 feet bgs. Groundwater levels in the borings were measured during or immediately after drilling and may have been affected by drilling procedures, which included introduction of water for drilling purposes.

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

5.0 ENGINEERING EVALUATIONS

5.1 GENERAL

GZA has conducted geotechnical engineering evaluations in accordance with *2020 AASHTO LRFD Bridge Design Specifications, 9th Edition* (herein designated as AASHTO) and the *MaineDOT Bridge Design Guide, 2003 Edition*, with updates through 2018 (MaineDOT BDG). The sections that follow describe the evaluations and the geotechnical basis for each element. Geotechnical calculations will be submitted in the Final Geotechnical Design Report for the project.

5.2 PROPOSED CONSTRUCTION

We understand that a full bridge replacement is planned for the project. The proposed layout includes shifting the centerline of the bridge approximately 6 feet to the east and increasing the span length to 50 feet. The existing and new bridge foundation footprints have different skews; therefore, the new abutments will be located between 3 and 11 feet behind the existing abutments.

5.3 APPROACH EMBANKMENTS

Typical grade raises of 1.5 feet or less are shown on the drawings at the approaches to the new bridge. Where embankment widening is proposed on the right side, proposed grade raises of 5 feet are typical, with a maximum fill height of about 6 feet behind the right wingwall at Abutment 2. The approach embankments are proposed with typical side slope angles of 2 horizontal to 1 vertical (2H:1V), or flatter, except for the ground surface in front of each abutment, which will slope down to the river level at an inclination of approximately 1.5H:1V and will be protected by riprap.

We anticipate that the proposed embankment fills will be constructed primarily over medium dense to dense fills, Silt and Sand, Sand and Gravel or bedrock. Due to the typical strength and low compressibility, embankment settlement and global stability are judged to be acceptable for the project.

5.4 FOUNDATION DESIGN CONSIDERATIONS

5.4.1 Abutment Foundations

Given the shallow depth and relative quality of the bedrock, it is our opinion that spread footings bearing on intact bedrock are the most appropriate foundation system for the abutments and wing walls. Recommendations for spread footing design are provided in **Section 6.4**.

5.5 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 indicate that the natural materials encountered above bedrock at the site are sufficiently cohesive or dense that the potential for liquefaction is low.

5.6 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, Cast-in-Place	0.8	10.5.5.2.2-1

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

5.7 SPREAD FOOTING DESIGN CONSIDERATIONS

5.7.1 Footing Bearing Resistance

Nominal and factored bearing resistances have been developed for the abutments 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 version (9th Edition) of AASHTO LRFD does not include the RMR formulation that is included in the 6th Edition version. However, Articles C10.4.6.4 and 10.6.2.6.2 of the 9th Edition refer to RMR-based design procedures for footings on rock, so the 6th Edition methodology was utilized here.

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**
Metasandstone	50-75	7.8	50	0.285	0.00025

Based on these parameters, the calculated nominal bearing resistance is 94 kips per square foot (ksf), resulting in a factored bearing resistance of 42 ksf for the strength limit state. Supporting calculations are provided in **Appendix E**.

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

The resistance against sliding should be evaluated in accordance with AASHTO LRFD Article 10.6.3.4 using an interface friction angle (ϕ_f) of 35 degrees, representing mass concrete on clean sound rock. Nominal sliding resistance for footings is equal to the vertical force multiplied by the concrete placement type factor (1.0 for cast-in-place concrete), and the sliding resistance coefficient (tan ϕ_f), which is equal to 0.7.

5.8 ADDITIONAL FOUNDATION CONSIDERATIONS

5.8.1 Frost Penetration

Fill soils are anticipated to be present at the abutments and embankments, either as existing fill or imported backfill. Based on the MaineDOT BDG, Section 5.2.1, the Freezing Index for the site is 1,100, and with low to moderate moisture content (±15 percent) soils, the estimated depth of frost penetration is 5.25 feet. However, where abutment foundations bear directly on sound rock, there is no minimum requirement for footing embedment.

5.8.2 Lateral Earth Pressure

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 abutment backwalls (below the end diaphragm) and wingwalls will be free to rotate and therefore should be designed for active earth pressure.

Thermal expansion of the bridge super structure in all superstructure details proposed 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. VHB provided a maximum expansion deflection of 0.25 inches for use in end diaphragm design. The end diaphragm height is approximately 2 feet resulting in a calculated abutment rotation of 0.0104 feet/foot. It is GZA's understanding that recent practice is to utilize The *Massachusetts Department of Transportation LRFD Bridge Design Manual* methodology, which provides an empirical equation, to calculate lateral earth pressure coefficient (K) based on the ratio of deflection (δt) and wall height (H).

Design lateral earth pressure recommendations are provided in **Section 6.3** of this report and calculations are presented in **Appendix E**.

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 peak ground acceleration coefficient, short- and long-period spectral acceleration coefficients were interpolated from the AASHTO design guide maps (3.10.2.1-1 through -21 as appropriate). Based on the site coordinates, the recommended AASHTO Response Spectra (Site Class B) for a 7 percent probability of exceedance in 75 years are summarized for the site are as follows:

SITE CLASS B SEISMIC DESIGN PARAMETERS	
Parameter	**Design Value**
Fpga	1.0
Fa	1.0
Fv	1.0
As (Period = 0.0 sec)	0.06 g
SDs (Period = 0.2 sec)	0.12 g
SD1 (Period = 1.0 sec)	0.04 g

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

6.3 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), K_p= 5.34;
  - Coefficient of Active Earth Pressure, K_a=0.28 (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.

6.3.1 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 94 ksf. At the strength limit state, footings should be designed for a maximum factored bearing resistance of 42 ksf. A bearing resistance of 42 ksf should also 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 nominal sliding resistance coefficient (tan δ) is equal to 0.7 for cast-in-place concrete on sound rock. The factored sliding resistance coefficient is 0.56 for Strength Limit State.
- Existing substructures should be completely removed prior to new foundation construction where they interfere with new foundations.
- The bedrock surface should be cleaned of loose soil or rock prior to concrete placement of concrete for the subfooting 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 located within or adjacent to foundation locations. These data, combined with the interpreted subsurface profile shown in **Figure 2**, are 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. 40 to 33
Abutment 2	El. 44 to 40

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/or 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 the volume of bedrock excavation.
- 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 with a minimum thickness of 6 inches.
- 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 is recommended at locations where the prepared bedrock surface is steeper than 4H:1V in any direction or as directed by the structural engineer.
- We understand rock dowels are being included in the design to supplement the sliding resistance for the footing. The dowels should be grouted a minimum of 2 feet into intact bedrock and embedded at least 2 feet into concrete.
- Dowels should be grouted with a cementitious grout on the MaineDOT Qualified Products List of Grout Materials for Keyways and Anchoring (pre-qualified for anchoring). Epoxy grout should not be used.
- Since the footings will be founded on bedrock, there is no minimum embedment required for frost protection per BDG Article 5.2.1.

7.0 CONSTRUCTION CONSIDERATIONS

This section 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 approximately 9 to 19 feet below existing grade to expose bedrock. The anticipated bedrock surface elevation ranges from approximately El. 40 to 33 at Abutment 1 and El. 44 to 40 at Abutment 2, which is 3 to 9 feet below the bottom of brook level at Abutment 1 and 3 feet above bottom of brook level at Abutment 2. Abutment 1 excavation is anticipated to extend approximately 2 to 9 feet below measured groundwater levels, and Abutment 2 excavation may remain above groundwater.

Excavations for abutment foundation construction may be achieved by sloped open cut techniques or by use of temporary excavation support. Sloped open cuts would likely require temporary damming and diversion of the brook during construction for foundation construction to proceed in the dry, especially at Abutment 1, as discussed below.

Technically feasible temporary excavation support systems for this site include internally-braced, steel sheet pile cofferdams and socketed H-piles or drilled micropiles and lagging. It is anticipated that the variable depth to bedrock will make use of steel sheet piling difficult, especially along the brook side where the toe would be exposed and overburden support would be minimal. A seal may not be developed against water infiltration at the rock interface. Although constructability would be more reliable in this application, socketed H-piles or drilled micropiles with wood lagging are likely to be a more expensive solution than steel sheetpiling, and they also may not control water from the brook if diversion is not implemented. Sloped open cut excavations are anticipated to be the most economical method to achieve the proposed excavations at this site, provided a diversion is implemented, there is sufficient space, and existing utilities can be appropriately protected or relocated during construction. It may also be feasible to leave portions of the existing abutments in-place to assist excavation support and/or dewatering.

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.

Damming and diversion and/or temporary dewatering are anticipated to be necessary to control groundwater and/or stream inflow in excavations. Depending on permitting and water levels at the time of construction, we anticipate that it would be possible to dam the stream and temporarily divert the flow through a pipe so the contractor can construct foundations in the dry with localized pumping from sumps. Where the excavations are at/near measured groundwater levels, it is anticipated that inflow of surface water or runoff to excavations can be handled by open pumping from sumps installed at the bottoms of excavations. Sumps should be fitted with geotextile or sand filters to prevent loss of subgrade fines during pumping. Where deeper excavation is required to expose bedrock, pumping may not be feasible, which would require placement of a tremie seal if dam-and-divert is not utilized. Dewatering discharge should be managed in accordance with the contractor's Stormwater Prevention Plan and MaineDOT Best Management Practices.

7.2 SUBGRADE PREPARATION

We anticipate that bedrock bearing surface preparation may be conducted in the dry, and that the bedrock surface will be variable in terms of elevation, slope and localized weathering. A combination of standard excavation equipment, hydraulic hoe-ramming equipment, and/or air lifting may be needed to remove the overburden and fractured/weathered rock. All soil and loose, decomposed, highly weathered and fractured bedrock should be removed from the footing bearing surface prior to placement of tremie seals or leveling concrete. Excavation should be accomplished within appropriate containment to prevent siltation if it is conducted in an open excavation.

If dam-and-divert is not utilized, it is more likely that preparation in the wet would be required. In this case, the prepared bearing surfaces should be checked by depth probing in conjunction with visual means. A Special Provision should be prepared to define the project-specific requirements for subgrade preparation and quality assurance/quality control, which would require confirmation coring for seals placed in the wet.

7.3 REUSE OF ON-SITE MATERIALS

One soil sample was recovered from the existing approach fill and it had approximately 10 percent passing the No. 200 sieve, indicating the fill may meet MaineDOT specifications for Granular 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.

TABLES

TABLE 1
Summary of Subsurface Explorations
Cromwell Brook No. 3 Bridge No. 0452
Bar Habor, Maine
GZA Job No. 09.0026155.01

Exploration ID	Easting	Northing	Station	Offset (ft)	Ground Surface El. (ft)	Top of Stratum Elevation						Stratum Thickness (ft)					Depth to Bedrock (ft)	Top of Rock Elevation (ft)	Bottom of Exploration Depth (ft)	Bottom of Exploration El. (ft)	Groundwater	
						Topsoil	Asphalt	Fill	Silt and Sand	Gravel and Sand	Bedrock	Topsoil	Asphalt	Fill	Silt and Sand	Gravel and Sand					El. (ft)	Depth (ft)
BB-BHCB-101	2209863.1	198802.8	13+50.8	6.5' L	51.5	NE	51.5	51.2	45.9	43.0	39.8	NE	0.3	5.3	2.9	3.2	11.7	39.8	18.6	32.9	41.5	10.0
BB-BHCB-102	2209882.4	198821.3	13+66.0	15.6' R	45.2	45.2	NE	NE	44.9	NE	33.2	0.3	NE	NE	11.7	NE	12.0	33.2	22.0	23.2	42.0	3.2
BB-BHCB-103	2209853.3	198857.9	14+06.8	7.3' L	51.6	NE	51.6	51.2	NE	NE	43.8	NE	0.4	7.4	NE	NE	7.8	43.8	18.0	33.6	NE	NE
BB-BHCB-104	2209863.6	198881.8	14+28.8	6.8' R	47.4	NE	NE	<40.2	NE	NE	NE	NE	NE	>7.2	NE	NE	NE	NE	7.2	40.2	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 boring locations and elevations were surveyed by MaineDOT and provided to GZA.
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.

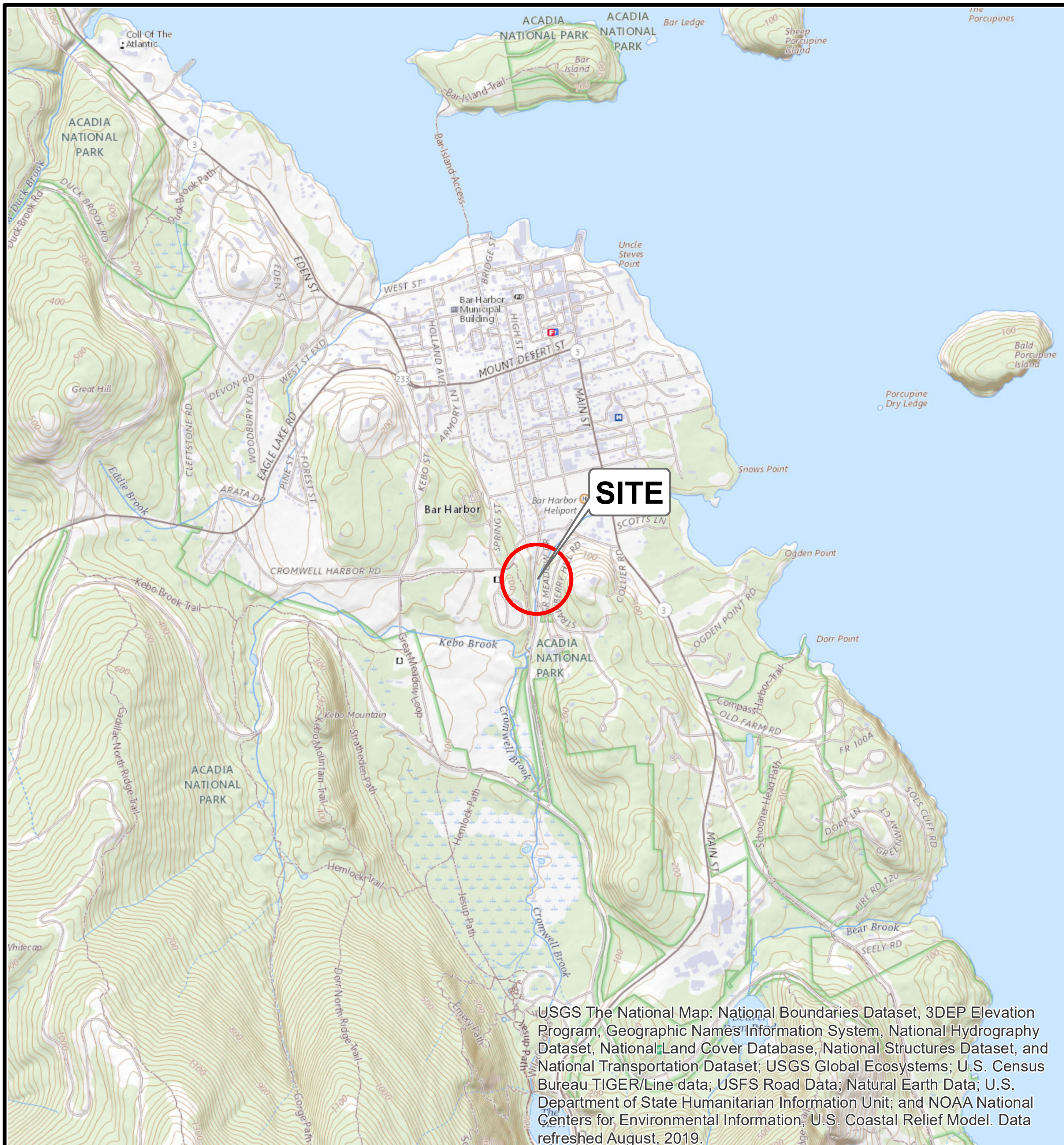
TABLE 2
Summary of Bedrock Data
Cromwell Brook No.3 Bridge #0452
Ledgelawn Avenue, Bar Harbor, Maine
WIN 026574.00

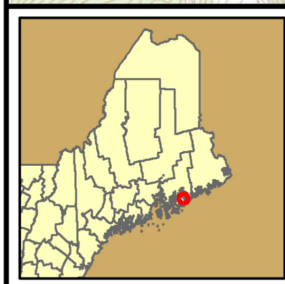
Boring ID	Core Run	Ground Suface Elevation (ft)	Depth of Core Run below Ground Suface (ft)			Depth to Rock (ft)	Depth (ft) Below Top of Rock			Length of Core Run (ft)	Rec (in)	Rec (%)	RQD (in)	RQD %	Joint Spacing (in)	Joint Aperture (in)	Elev. (ft)		LAB								Rock Type
			Top		Bottom		Top		Bottom								Top	Bottom	Depth of Sample (ft)	Depth of Sample into Rock (ft)	Elev Top of Sample (ft)	UCS (psi)	Point Load Index, Is50 (psi)	Correlated UCS from Point Load Tests (psi)	Modulus (ksi)	Unit Wt (pcf)	
BB-BHCB-101	R1	51.5	11.5	-	12.4	11.7	-0.2	-	0.7	0.9	3	28%	0	0%	--	--	40.0	39.1									METASANDSTONE
BB-BHCB-101	R2	51.5	12.4	-	16.0	11.7	0.7	-	4.3	3.6	38	88%	34	78%	2.5-24	.01-0.1	39.1	35.5	*14.3*	*2.6*	*37.2*	*26,522*	--		*4*	*169.2*	METASANDSTONE
BB-BHCB-101	R3	51.5	16.0	-	18.6	11.7	4.3	-	6.9	2.6	31	100%	22	71%	0.75-24	.004-0.1	35.5	32.9									METASANDSTONE
BB-BHCB-102	R1	45.2	12.5		15.5	12.0	0.5	-	3.5	3.0	36	100%	4	11%	2.5-8	.01-0.1	32.7	29.7	*14.1*	*1.6*	*31.2*	--	*234 (D), 303 (A)*	*5,616* (D), 7,272* (A)*	--	*178.0 (D), 168.6 (A)*	METASANDSTONE
BB-BHCB-102	R2	45.2	15.5		19.5	12.0	3.5	-	7.5	4.0	48	100%	23	48%	2.5-8	0.02-0.4	29.7	25.7									METASANDSTONE
BB-BHCB-102	R3	45.2	19.5		22.5	12.0	7.5	-	10.5	3.0	36	100%	35	96%	2.5-24	.01-0.1	25.7	22.7									METASANDSTONE
BB-BHCB-103	R1	51.6	8.0	-	13.0	7.8	0.2	-	5.2	5.0	60	100%	31	52%	0.75-8	0.02-0.4	43.6	38.6	*8.0*	*0.0*	*43.6*	*10,372*	--		*3*	*170.9*	METASANDSTONE
BB-BHCB-103	R2	51.6	13.0	-	18.0	7.8	5.2	-	10.2	5.0	48	80%	38	63%	0.75-8	0.01->0.4	38.6	33.6									METASANDSTONE

Notes:

1. "UCS" is unconfined compressive strength. UCS values marked with "*" are from diametrical (D) and axial (A) point load tests and are correlated from the point load test results.
2. Refer to the boring logs in Appendix B for additional information.
3. Project elevation datum is North American Vertical Datum (NAVD 88), unless noted otherwise.
4. As-drilled locations were surveyed by MaineDOT and provided to GZA.

FIGURES





USGS QUADRANGLE LOCATION

N W E S

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

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

SCALE IN FEET

Data Supplied by :

esri

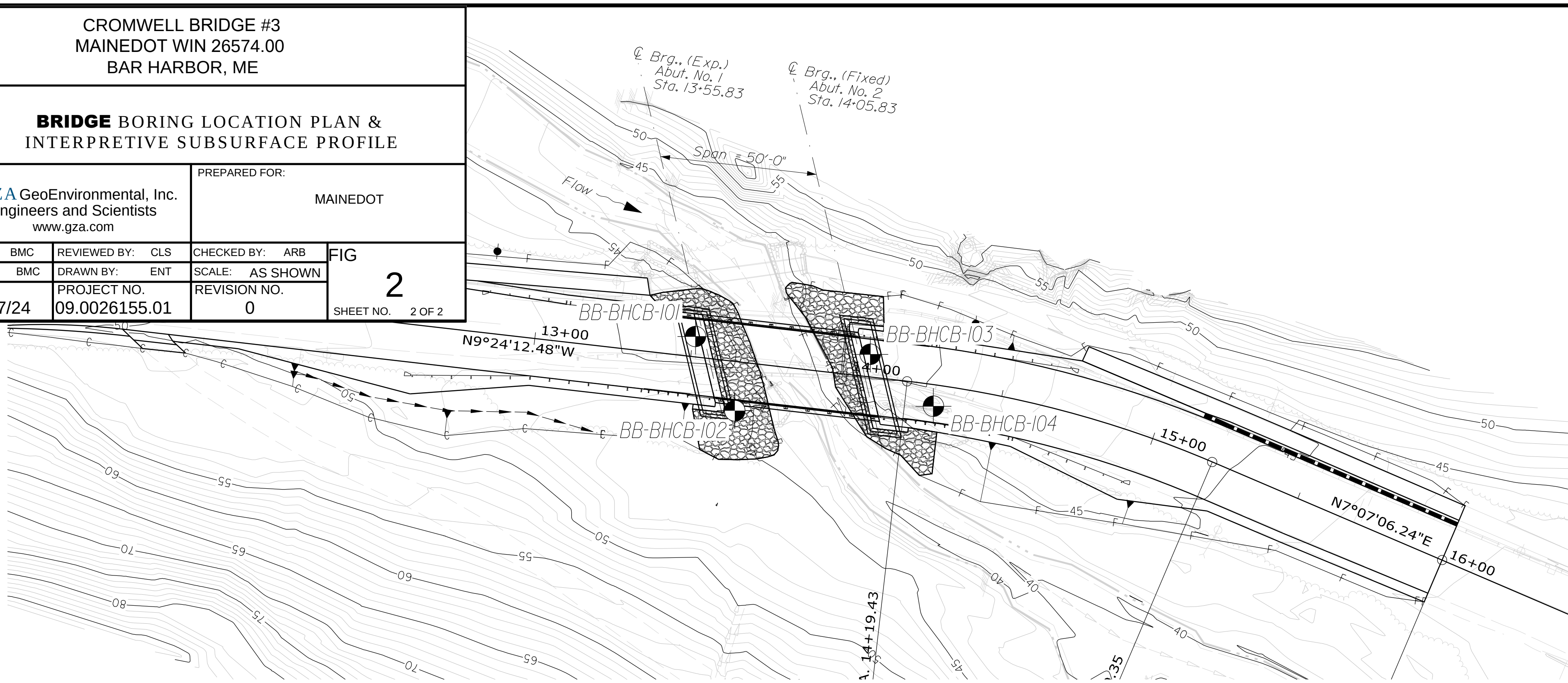
USGS science for a changing world

PROJ. MGR.: ARB DESIGNED BY: LCN REVIEWED BY: ARB OPERATOR: ADM DATE: 10-30-2024	**LOCUS PLAN**	JOB NO. 09.0026155.01
	CROMWELL BROOK #3 BRIDGE #0452 BAR HARBOR, MAINE	FIGURE NO. **1**

CROMWELL BRIDGE #3
MAINEDOT WIN 26574.00
BAR HARBOR, ME

BRIDGE BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

PREPARED BY: GZA GeoEnvironmental, Inc. Engineers and Scientists www.gza.com		PREPARED FOR: MAINEDOT	
PROJ MGR: BMC	REVIEWED BY: CLS	CHECKED BY: ARB	FIG 2
DESIGNED BY: BMC	DRAWN BY: ENT	SCALE: AS SHOWN	
DATE: 11/7/24	PROJECT NO. 09.0026155.01	REVISION NO. 0	SHEET NO. 2 OF 2



NOTES

1) Base map developed from electronic files provided by VHB on November 7, 2024 (Files included Alignments.dgn, Borings_10.6.22.dgn, BRIDGE.dgn, Highway.dgn, Contours.dgn, and Topo.dgn).

2) Profile developed from electronic file Profile.dgn and z-Bridge_Profile.dgn provided by VHB on November 7, 2024.

3) The as-drilled locations of the test borings were surveyed by a MaineDOT survey crew and supplied to GZA.

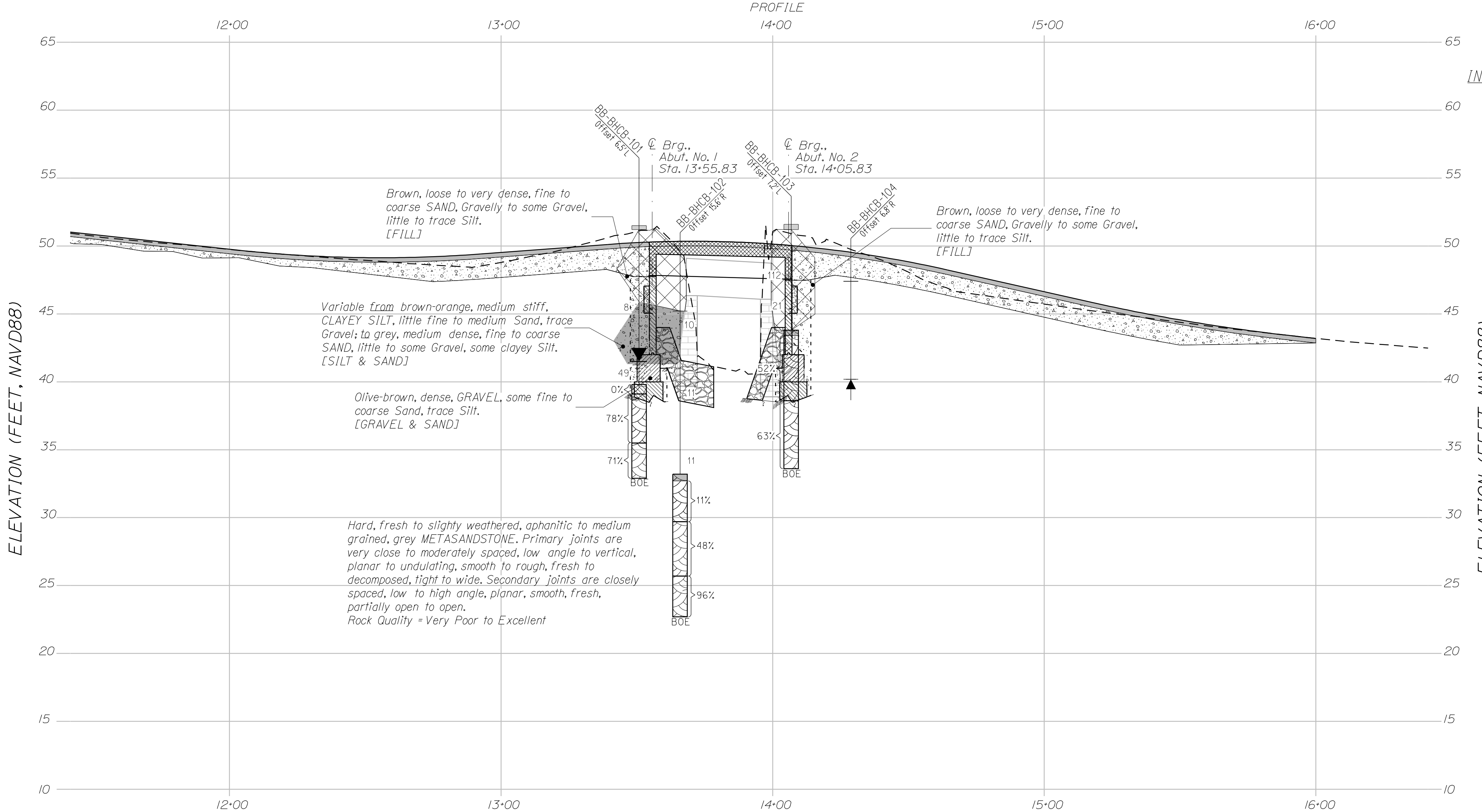
4) BB-BHCB-100 series borings were performed by New England Boring Contractors and observed by GZA personnel between July 13 and August 11, 2022.

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

BORING LOCATION PLAN LEGEND

BB-BHCB-104 — Location and designation of borings and roller probes

PROFILE



INTERPRETIVE SUBSURFACE PROFILE LEGEND

- Interpreted Top of Bedrock (See Note 5)
- Boring No., Offset, if shown
- Pavement Thickness if applicable
- 6 — Energy-Corrected SPT N60 Value (blows/foot)
- Encountered Groundwater Level
- Strata Interface
- R — Split Spoon Refusal (>50 blows for 1" penetration)
- Rock Core through Apparent Boulder
- RQD = Rock Quality Designation for Rock Core Sample
- BOE — Bottom of Exploration
- Bottom of Exploration, Refusal
- Bottom of Exploration, No Refusal

PLAN

25 0 25 50
Scale In Feet

PROFILE

Horiz. 25 0 25 50
Vert. 5 0 5 10
Scale In Feet

PREPARED BY: GZA

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2657400

WIN 026574.00

BRIDGE PLANS

BRIDGE NO. 0452

	BY	DATE
PROJ. MANAGER ANDREW LATHE		
DESIGN-DETAILED	BMC	SEPT. 2024
CHECKED-REVIEWED	CLS	
DESIGN2-DETAILED2	ENT	
DESIGN3-DETAILED3	ARB	
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

CROMWELL BRIDGE #3 LEDGELAWN AVENUE
OVER CROMWELL BROOK
BAR HARBOR HANCOCK COUNTY

BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
5
OF 22

Date:11/7/2024
common
Username:
Filename: ...\005_Cromwell_BLP_ISP_09_12_2024.dgn Division: HIGHWAY

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 – TEST BORING LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM

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

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

MODIFIED BURMISTER SYSTEM

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Density of Cohesionless Soils	Standard Penetration Resistance N_{60}-Value (blows per foot)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

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

Consistency of Cohesive soils	SPT N_{60}-Value (blows per foot)	Approximate Undrained Shear Strength (psf)	Field Guidelines
Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily penetrates
Soft	2 - 4	250 - 500	Thumb easily penetrates
Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort
Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort
Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail
Hard	>30	over 4000	Indented by thumbnail with difficulty

Rock Quality Designation (RQD):

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

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

Rock Quality Based on RQD

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

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

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock 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	

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Cromwell Brook #3 Bridge #0452 Location: Bar Harbor, Maine	Boring No.: BB-BHCB-101 WIN: 26574.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	51.5	**Auger ID/OD:**	4.5"
Operator:	G. McDougal	**Datum:**	NAVD88	**Sampler:**	Standard Splitspoon
Logged By:	E. Tome	**Rig Type:**	ATV-Mounted B53 Mobile Drill	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/11/22-8/11/22	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NX
Boring Location:	13+50.8, 6.5 LT	**Casing ID/OD:**	3.0/3.5"	**Water Level*:**	10'
Hammer Efficiency Factor:	0.92	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	51.2		0'-0.3': Temporary Asphalt. —0.3	
5	1D	24/17	5.0 - 7.0	8-3-2-1	5	8		45.9		Top 7": Brown, dry, fine to coarse SAND, some gravel, trace silt, (Fill). —5.6 Bottom 10": Brown/orange, moist, medium stiff, CLAYEY SILT, little fine to medium sand, trace gravel,(Silt and Sand).	22-S-3203 A-4(0), ML WC=35%
								43.0		—8.5	
10	2D	20/11	10.0 - 11.7	6-14-18-50/2"	32	49	R/C			Olive-brown, wet, dense, GRAVEL, some fine to coarse sand, trace silt, (Gravel and Sand).	22-S-3204 A-1-a, GW-GM WC=10.2%
	R1	11/3	11.5 - 12.4	RQD = 0%			NX	39.8		Spoon refusal at 11.7' on probable bedrock. Highly fractured rock indicated by RC advancement. Advanced RC to 12.5' and set up to core. Fractured rock caved to 11.5', began core at 11.5'. R1: Three rounded gravel pieces. —11.7	22-S-3228 q_p=3819 ksf
	R2	43/38	12.4 - 16.0	RQD = 78%						R2: Hard, fresh, aphanitic, grey, METASANDSTONE, with some calcite stringers. Joints are close to moderately spaced, moderately dipping with one vertical joint, undulating, rough, fresh, partially open to open. Rock Quality = Good Recovery = 88% Rock Core Times (min:sec): 12.4-13.4' (2:21), 13.4-14.4' (3:19), 14.4-15.4' (4:00), 15.4-16.0' (5:13)	
15	R3	31/31	16.0 - 18.6	RQD = 71%				32.9		R3: Hard, fresh, aphanitic, grey, METASANDSTONE, with calcite stringers. Joints are very close to moderately spaced, low angle with one vertical joint, undulating, rough, fresh, tight to open. Rock Quality = Fair Recovery = 100% Rock Core Times (min:sec): 16.0-17.0' (1:53), 17.0-18.0' (2:11), 18.0-18.6' (2:28) —18.6	
20										**Bottom of Exploration at 18.6 feet below ground surface.**	
25											

Remarks:

1. Fine grained soil descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterberg Limit tests if available, rather than the MaineDOT Standard-based percentages passing specific grain sizes.
2. Automatic Hammer NEBC #28 Energy Transfer Ratio = 0.92.
3. Water level measurements were taken immediately after removal of casing.
4. As-drilled boring locations were surveyed by MaineDOT in the field (N198802.78, E2209863.10).

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

Boring No.: BB-BHCB-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Cromwell Brook #3 Bridge #0452
Location: Bar Harbor, Maine

Boring No.: BB-BHCB-102
WIN: 26574.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	45.2	**Auger ID/OD:**	4.5"
Operator:	T. Schaefer	**Datum:**	NAVD88	**Sampler:**	Standard Splitspoon
Logged By:	E. Tombaugh	**Rig Type:**	ATV-Mounted B53 Mobile Drill	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/13/22-7/13/22	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NX
Boring Location:	13+66.0, 15.6 RT	**Casing ID/OD:**	4.0/4.5"	**Water Level*:**	3.2'
Hammer Efficiency Factor:	0.86	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/4	0.0 - 2.0	WOH-1-6-10	7	10	SSA	44.9		Dark brown, dry, loose, Silty fine SAND, some gravel, with roots, (Topsoil). 0.3	
5	2D	24/11	5.0 - 7.0	WOH-1-7-12	8	11				Grey, wet, medium dense, fine to coarse SAND, some silt, little gravel, (Silt and Sand).	
							35				
							58				
							57				
							28				
10	3D	24/19	10.0 - 12.0	6-4-4-6	8	11	25			Grey, wet, medium dense, fine to coarse SAND, some gravel, some clayey Silt, (Silt and Sand).	22-S-3205 A-2-4(0), SM WC=7.5%
							44	33.2		Casing refusal at 12.0' on probable bedrock. Advanced roller cone to 12.5' and set up to core. 12.0	
	R1	36/36	12.5 - 15.5	RQD = 11%						R1: Hard, fresh, medium grained, grey, METASANDSTONE. Primary joints are closely spaced, low angle to moderately dipping, planar, smooth, fresh, partially open to open. Secondary joints are closely spaced, high angle, planar, smooth, fresh, partially open. Rock Quality = Very Poor Recovery = 100% Rock Core Times (min:sec): 12.5-13.5' (2:54), 13.5-14.5' (2:00), 14.5-15.5' (2:22)	22-S-3225 PLD=33.7 ksf PLA=43.6 ksf
15	R2	48/48	15.5 - 19.5	RQD = 48%						R2: Hard, fresh, medium grained, grey, METASANDSTONE, with calcite stringers. Primary joints are closely spaced, high angle, planar, smooth, decomposed, open to moderately wide. Secondary joints are closely spaced, low angle, planar, smooth, fresh, partially open to open. Rock Quality = Poor Recovery = 100% Rock Core Times (min:sec): 15.5-16.5' (1:56), 16.5-17.5' (1:39), 17.5-18.5' (1:53), 18.5-19.5' (1:46)	
20	R3	36/36	19.5 - 22.5	RQD = 96%				23.2		R3: Hard, fresh, medium grained, grey, METASANDSTONE, with calcite stringers. Joints are close to moderately spaced, high angle, planar, smooth, discolored, partially open to open. Rock Quality = Excellent Recovery = 100% Rock Core Times (min:sec): 19.5-20.5' (2:10), 20.5-21.5' (1:52), 21.5-22.5' (2:22) 22.0	
										Bottom of Exploration at 22.0 feet below ground surface.	
25											

Remarks:

1. Fine grained soil descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterberg Limit tests if available, rather than the MaineDOT Standard-based percentages passing specific grain sizes.
2. Automatic Hammer NEBC #1 Energy Transfer Ratio = 0.86.
3. Water level measurements were taken immediately after removal of casing.
4. As-drilled boring locations were surveyed by MaineDOT in the field (N198821.33, E2209882.41).

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

Boring No.: BB-BHCB-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Cromwell Brook #3 Bridge #0452
Location: Bar Harbor, Maine

Boring No.: BB-BHCB-103
WIN: 26574.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	51.6	**Auger ID/OD:**	4.5"
Operator:	G. McDougal	**Datum:**	NAVD88	**Sampler:**	Standard Splitspoon
Logged By:	E. Tome	**Rig Type:**	ATV-Mounted B53 Mobile Drill	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/11/22-8/11/22	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NX
Boring Location:	14+06.8, 7.2 LT	**Casing ID/OD:**	3.0/3.5"	**Water Level*:**	Dry at 8.0'
Hammer Efficiency Factor:	0.92	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	51.2		0'-0.4': Temporary Asphalt. 0.4	
	1D	18/14	3.0 - 4.5	23-23-50	73	112				Brown, dry, very dense, Gravelly fine to coarse SAND, (Fill). Asphalt in spoon tip at 4.3'. Hydrocarbon odor.	
								47.1		4.5	
5								46.9		4.5'-4.7': Asphalt. 4.7	
	2D	24/11	5.0 - 7.0	12-8-6-1	14	21				Fill observed on auger flights at 4.7'. Brown, dry, medium dense, Gravelly fine to coarse SAND, little silt, (Fill). Strong Hydrocarbon odor.	22-S-3206 A-1-b, SM WC=6.8%
								43.8		Increase in auger resistance during advancement from 7.8'-7.9'. Advanced 3" casing and roller cone to 8.0' and set up to core. 7.8	
	R1	60/60	8.0 - 13.0	RQD = 52%			NX			R1: Hard, fresh to slightly weathered, aphanitic, grey, METASANDSTONE, with calcite stringers. Joints are very close to close, low angle with two vertical joints, undulating, rough, fresh to discolored, open to moderately wide. Rock Quality = Fair Recovery = 100% Rock Core Times (min:sec): 8.0-9.0' (3:48), 9.0-10.0' (3:07), 10.0-11.0' (1:37), 11.0-12.0' (1:44), 12.0-13.0' (1:41)	22-S-3226 q_p=1494 ksf 22-S-3227 PLD=326.3 ksf PLA=261.1 ksf
10											
	R2	60/48	13.0 - 18.0	RQD = 63%						R2: Hard, fresh to slightly weathered, aphanitic, grey, METASANDSTONE, with calcite stringers. Joints are very close to close, low angle with one vertical joint, undulating, rough, fresh to discolored, partially open to wide. Diorite intrusions at 16.0'-18.0'. Rock Quality = Fair Recovery = 80% Rock Core Times (min:sec): 13.0-14.0' (1:50), 14.0-15.0' (1:54), 15.0-16.0' (3:51), 16.0-17.0' (4:04), 17.0-18.0' (5:32)	
15											
								33.6		18.0 **Bottom of Exploration at 18.0 feet below ground surface.**	
20											
25											

Remarks:

1. Fine grained soil descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterberg Limit tests if available, rather than the MaineDOT Standard-based percentages passing specific grain sizes.
2. Automatic Hammer NEBC #1 Energy Transfer Ratio = 0.92.
3. Water level measurements were taken immediately after removal of casing.
4. Bottom 12" of R2 fell out of core barrel upon retrieval, could not recover.
5. Hole collapsed at 8.0' upon removal of casing, no water observed.
6. As-drilled boring locations were surveyed by MaineDOT in the field (N198857.90, E2209853.25).

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

Boring No.: BB-BHCB-103

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Cromwell Brook #3 Bridge #0452
Location: Bar Harbor, Maine

Boring No.: BB-BHCB-104
WIN: 26574.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	47.4	**Auger ID/OD:**	4.5"
Operator:	T. Schaefer	**Datum:**	NAVD88	**Sampler:**	Standard Splitspoon
Logged By:	E. Tombaugh	**Rig Type:**	ATV-Mounted B53 Mobile Drill	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/13/22-7/13/22	**Drilling Method:**	SSA	**Core Barrel:**	--
Boring Location:	14+28.8, 6.8 RT	**Casing ID/OD:**	3.0/3.5"	**Water Level*:**	Unknown
Hammer Efficiency Factor:	0.86	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			Auger encountered rip rap.	
5										Water flowing up out of hole at 7.4'; at 1430 the water tested positive for chlorine and the wter department concluded the flow was coming from a water line. Water department directed termination of the boring, likely nicked side of pipe.	
								40.2		7.2 **Bottom of Exploration at 7.2 feet below ground surface.**	
10											
15											
20											
25											

Remarks:

1. Automatic Hammer NEBC #1 Energy Transfer Ratio = 0.86.
2. Bar Harbor Water Department abandoned exploration after GZA left the site.
3. As-drilled boring locations were surveyed by MaineDOT in the field (N198881.79, E2209863.62).

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

Boring No.: BB-BHCB-104

APPENDIX C – LABORATORY TEST RESULTS

THIELSCH ENGINEERING	195 Frances Avenue Cranston RI, 02910 Phone: (401)-467-6454 Fax: (401)-467-2398 thielsch.com *Let's Build a Solid Foundation*	Client Information: GZA GeoEnvironmental, Inc. South Portland, ME PM: Michels Johnescu Assigned By: Michael Johnescu Collected By: Erin Tome	Project Information: **Cromwell Brook Bridge** **Bar Harbor, ME** GZA Project Number: 09.0026155.10 Summary Page: 1 of 1 Report Date: 08.29.22

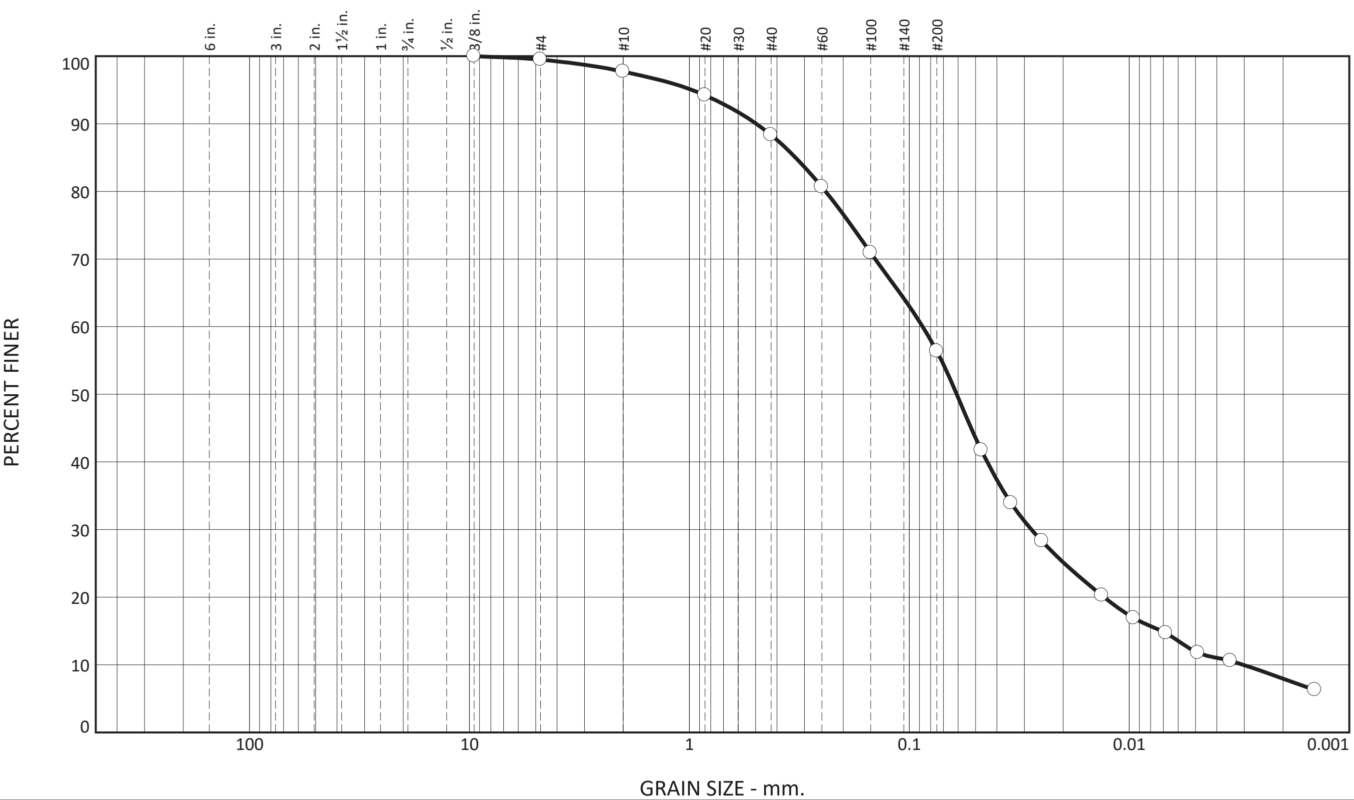
LABORATORY TESTING DATA SHEET, Report No.: 7422-H-177

Boring No.	Sample No.	Depth (ft)	Laboratory No.	Identification Tests								Proctor / CBR / Permeability Tests								Laboratory Log and Soil Description
				As Received Moisture Content %	LL %	PL %	Gravel %	Sand %	Fines %	Org. %	pH	Dry unit wt. (pcf)	Test Moisture Content %	γ_d MAX (pcf) W_{opt} (%)	γ_d MAX (pcf) W_{opt} (%) (Corr.)	Target Test Setup as % of Proctor	CBR @ 0.1"	CBR @ 0.2"	Permeability cm/sec	
				D2216	D4318		D6913			D2974	D4792			D1557						
BB-BHCB-101	1D	6.2-7	22-S-3203	35.0			0.5	43.1	56.4											Brown CLAYEY SILT, little f-m Sand, trace fine Gravel
BB-BHCB-101	2D	10-11.7	22-S-3204	10.2			56.9	34.2	8.9											Olive f-c GRAVEL, some f-c Sand, trace Silt
BB-BHCB-102	3D	10-12	22-S-3205	7.5			30.2	41.1	28.7											Grey f-c SAND, some f-c Gravel, some Clayey Silt
BB-BHCB-103	2D	5-7	22-S-3206	6.8			36.1	50.8	13.1											Brown f-c SAND and f-c GRAVEL, little Silt

Date Received: *08.22.22* Reviewed By: [signature] Date Reviewed: *08.29.22*

Particle Size Distribution Report

ASTM D6913 & D1140, ASTM D7928



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.5	1.8	9.4	31.9	48.4	8.0

Test Results (ASTM D6913 & D1140, ASTM D7928)				
Sieve Size or Diam. (mm.)	Finer (%)	Spec.* (%)	Out of Spec. (%)	Pct. of Fines
3/8"	100.0			
#4	99.5			
#10	97.7			98.2
#20	94.2			94.7
#40	88.3			88.8
#60	80.7			81.1
#100	70.9			71.3
#200	56.4			56.7
0.0471 mm.	41.7			
0.0345 mm.	33.9			
0.0250 mm.	28.3			
0.0133 mm.	20.3			
0.0096 mm.	16.9			
0.0068 mm.	14.7			
0.0049 mm.	11.8			
0.0035 mm.	10.6			
0.0014 mm.	6.3			

* (no specification provided)

Material Description

Brown CLAYEY SILT, little f-m Sand, trace fine Gravel

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 0.4967 D_{85}= 0.3292 D_{60}= 0.0870
D_{50}= 0.0609 D_{30}= 0.0279 D_{15}= 0.0071
D_{10}= 0.0030 C_u= 28.66 C_c= 2.94

Classification

USCS= ML AASHTO= A-4(0)

Test Remarks

Sample visually classified as plastic. Sample rolled to 1/4"

Source of Sample: BB-BHCB-xxx **Depth:** 6.2-7'
Sample Number: 101 / 1D **Sample Date:** 09.29.22

Thielsch Engineering Inc.

Cranston, RI

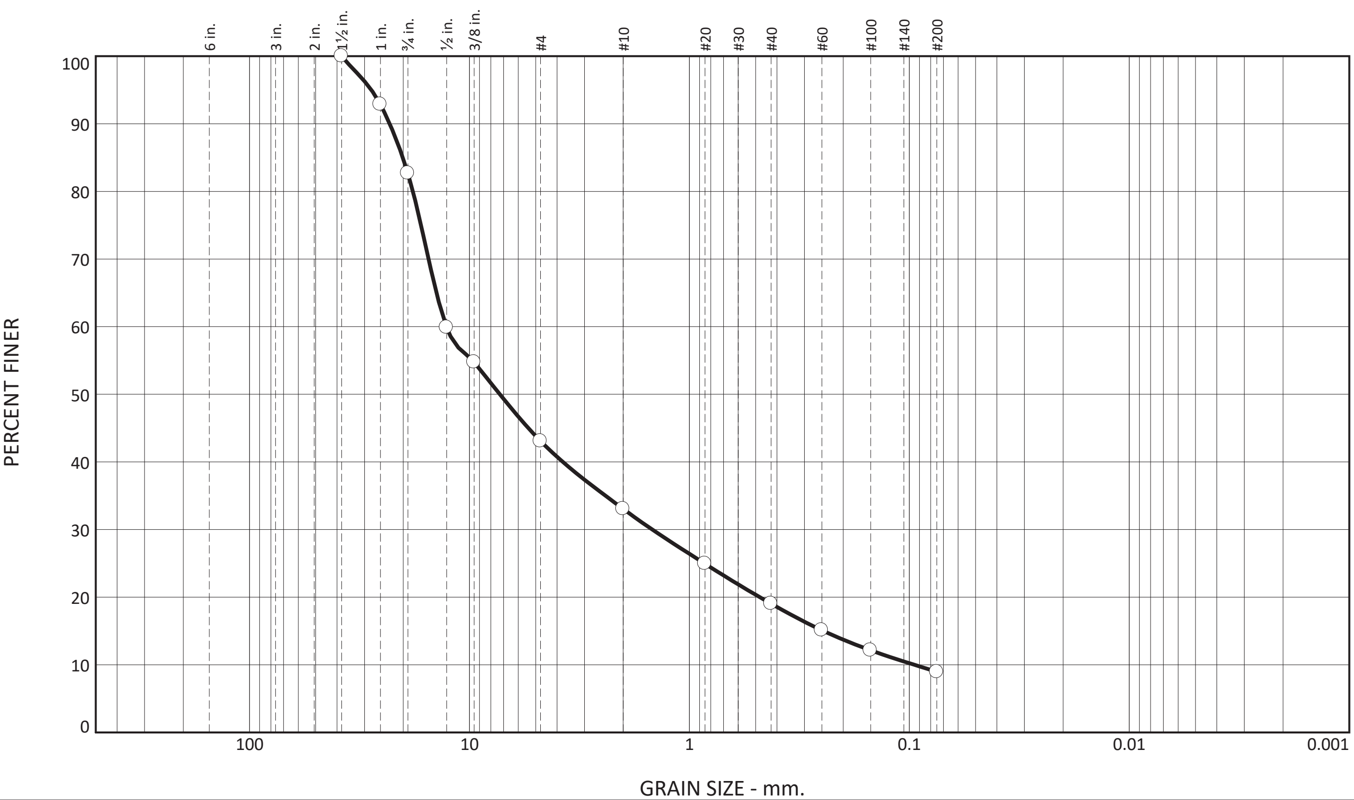
Client: GZA GeoEnvironmental
Project: Cromwell Brook Bridge
Bar Harbor, ME
Project No: 09.0026155.00 **Figure** 22-S-3203

These results are for the exclusive use of the client for whom they were obtained. They apply only to the samples tested and are not indicative of apparently identical samples.

Tested By: FR / JB **Checked By:** Rebecca Roth

Particle Size Distribution Report

ASTM D6913 & D1140



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	17.3	39.6	10.0	14.1	10.1	8.9	

Test Results (ASTM D6913 & D1140)				
Sieve Size or Diam. (mm.)	Finer (%)	Spec.* (%)	Out of Spec. (%)	Pct. of Fines
1 1/2"	100.0			
1"	92.8			
3/4"	82.7			
1/2"	59.9			
3/8"	54.7			
#4	43.1			
#10	33.1			
#20	24.9			
#40	19.0			
#60	15.1			
#100	12.1			
#200	8.9			

* (no specification provided)

Material Description

Olive f-c GRAVEL, some f-c Sand, trace Silt

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 23.0421 D_{85}= 20.0951 D_{60}= 12.7410
D_{50}= 7.2749 D_{30}= 1.4747 D_{15}= 0.2459
D_{10}= 0.0949 C_u= 134.25 C_c= 1.80

Classification

USCS= GW-GM AASHTO= A-1-a

Test Remarks

Source of Sample: BB-BHCB-xxx **Depth:** 10-11.7'
Sample Number: 101 / 2D **Sample Date:** 08.29.22

Thielsch Engineering Inc.

Cranston, RI

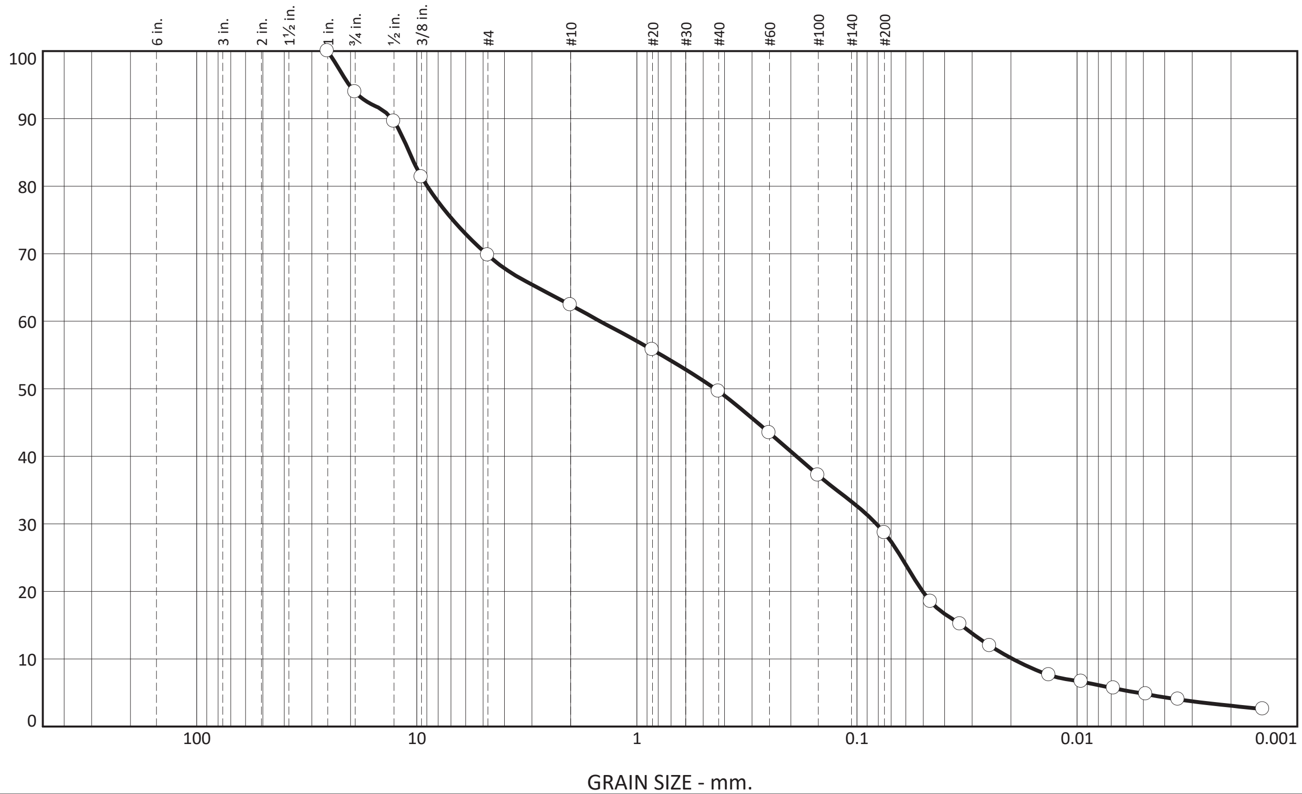
Client: GZA GeoEnvironmental
Project: Cromwell Brook Bridge
Bar Harbor, ME
Project No: 09.0026155.00 **Figure** 22-S-3204

These results are for the exclusive use of the client for whom they were obtained. They apply only to the samples tested and are not indicative of apparently identical samples.

Checked By: Rebecca Roth

Particle Size Distribution Report

ASTM D6913 & D1140, ASTM D7928



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	6.1	24.1	7.4	12.8	20.9	25.6	3.1

Test Results (ASTM D6913 & D1140, ASTM D7928)

Sieve Size or Diam. (mm.)	Finer (%)	Spec.* (%)	Out of Spec. (%)	Pct. of Fines
1"	100.0			
3/4"	93.9			
1/2"	89.6			
3/8"	81.3			
#4	69.8			
#10	62.4			89.4
#20	55.8			79.9
#40	49.6			71.1
#60	43.5			62.3
#100	37.2			53.3
#200	28.7			41.1
0.0463 mm.	18.5			
0.0340 mm.	15.2			
0.0249 mm.	11.9			
0.0134 mm.	7.6			
0.0096 mm.	6.6			
0.0068 mm.	5.7			
0.0049 mm.	4.8			
0.0035 mm.	4.0			
0.0014 mm.	2.6			

* (no specification provided)

Material Description

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

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 12.9842 D_{85}= 10.8047 D_{60}= 1.4707
D_{50}= 0.4411 D_{30}= 0.0817 D_{15}= 0.0335
D_{10}= 0.0196 C_u= 75.12 C_c= 0.23

Classification

USCS= SM AASHTO= A-2-4(0)

Test Remarks

Sample visually classified as plastic. Sample rolled to 1/4"

Source of Sample: BB-BHCB-xxx **Depth:** 10-12'
Sample Number: 102 / 3D **Sample Date:** 08.29.22

Thielsch Engineering Inc.

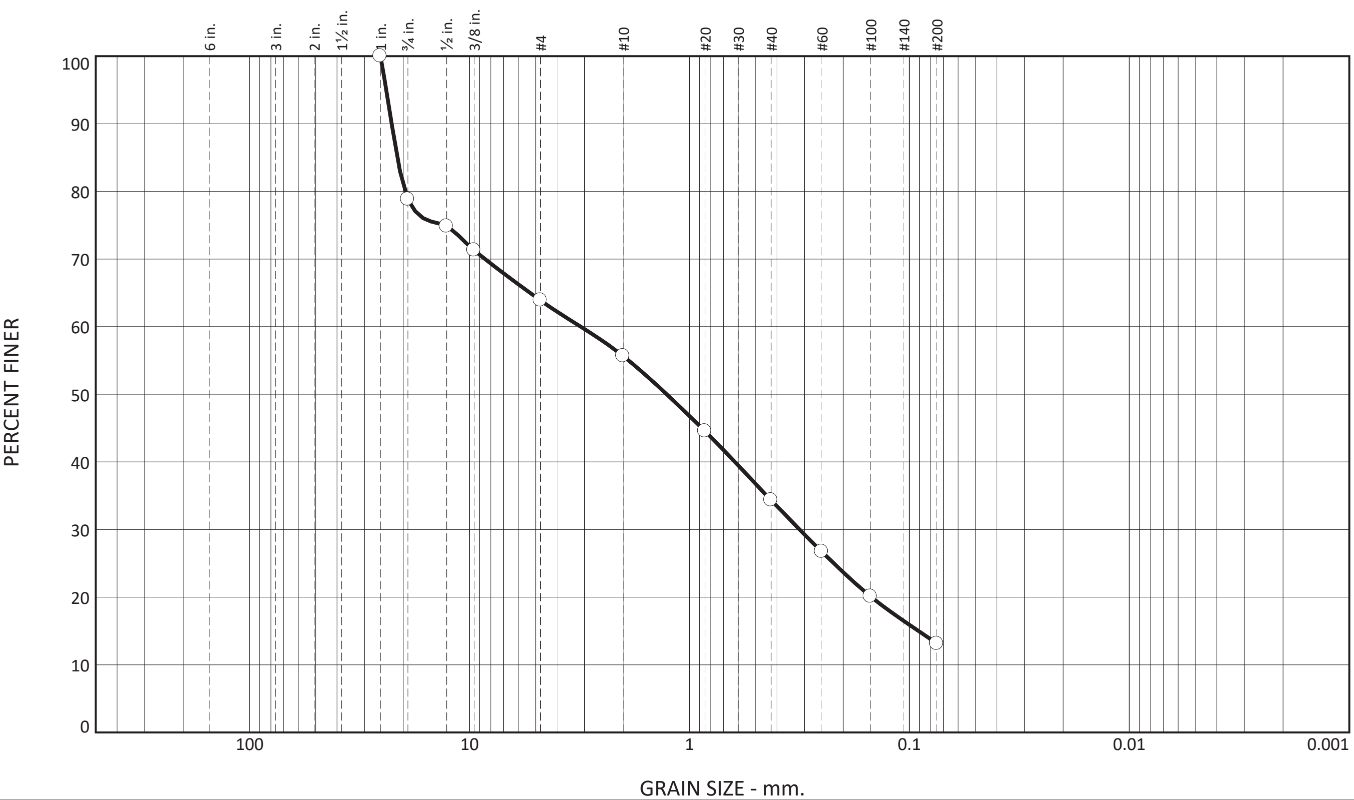
Cranston, RI

Client: GZA GeoEnvironmental
Project: Cromwell Brook Bridge
Bar Harbor, ME
Project No: 09.0026155.00 **Figure** 22-S-3205

Tested By: FR / JB **Checked By:** Rebecca Roth

Particle Size Distribution Report

ASTM D6913 & D1140



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	21.2	14.9	8.3	21.3	21.2	13.1	

Test Results (ASTM D6913 & D1140)

Sieve Size or Diam. (mm.)	Finer (%)	Spec.* (%)	Out of Spec. (%)	Pct. of Fines
1"	100.0			
3/4"	78.8			
1/2"	74.8			
3/8"	71.3			
#4	63.9			
#10	55.6			
#20	44.5			
#40	34.3			
#60	26.7			
#100	20.1			
#200	13.1			

* (no specification provided)

Material Description

Brown f-c SAND and f-c GRAVEL, little Silt

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 22.5701 D_{85}= 21.2638 D_{60}= 3.1208
D_{50}= 1.2678 D_{30}= 0.3154 D_{15}= 0.0908
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-1-b

Test Remarks

Source of Sample: BB-BHCB-xxx **Depth:** 5-7'
Sample Number: 103 / 2D **Sample Date:** 08.29.22

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Cromwell Brook Bridge
Bar Harbor, ME
Project No: 09.0026155.00 **Figure** 22-S-3206

These results are for the exclusive use of the client for whom they were obtained. They apply only to the samples tested and are not indicative of apparently identical samples.

Tested By: FR / JB **Checked By:** Rebecca Roth

THIELSCH ENGINEERING	195 Frances Avenue Cranston RI, 02910 Phone: (401)-467-6454 Fax: (401)-467-2398 thielsch.com *Let's Build a Solid Foundation*	Client Information: GZA GeoEnvironmental, Inc. South Portland, ME PM: Michels Johnescu Assigned By: Michael Johnescu Collected By: Erin Tome	Project Information: **Cromwell Brook Bridge** **Bar Harbor, ME** GZA Project Number: 09.0026155.10 Summary Page: 1 of 1 Report Date: 09.28.22

LABORATORY TESTING DATA SHEET, Report No.: 7422-H-182, Rev 2

Boring No.	Sample No.	Depth (ft)	Laboratory No.	Specimen Data: Mohs Hard-ness	Diameter (in)	Length (in)	(1) Unit Weight (PCF)	(2) Wet Density (PCF)	Bulk G_s	Compressive Strength Tests: (3) Other Tests	(4) Strength PSI	(5) Strain %	(6) E sec PSI EE+06	(7) Poisson's Ratio	$\sigma\tau$ PSI	Is_{50} PSI	(8) s_c PSI	Rock Formation or Description or Remarks
BB-BHCB-102	R1	14.05-14.35	22-S-3225		1.984	1.635	178.0			PLD	234					234	5616	Grey Metasandstone
BB-BHCB-102	R1	14.05-14.35	22-S-3225		1.986	1.484	168.6			PLA	303					303	7272	Grey Metasandstone
Broke along Foliation																		
BB-BHCB-103	R1	8.0-8.55	22-S-3226		1.986	4.322	170.9				*10372	0.339	3.20	0.75				Grey Metasandstone
*Minor break at about 4500 psi																		
BB-BHCB-103	R1	12.1-12.7	22-S-3227		1.993	1.904	179.2			PLD	2266					2266	54384	Grey Metasandstone
BB-BHCB-103	R1	12.1-12.7	22-S-3227		2.001	1.472	187.9			PLA	1813					1813	43512	Grey Metasandstone
Fresh Break																		
BB-BHCB-101	R2	14.3-15.05	22-S-3228		1.987	4.512	169.2				*26522	0.413	4.73	0.09				Grey Metasandstone
*Minor break at about 7100 psi - Broke along Discontinuity																		

	Notes		Notes	
(1) Volume Determined By Measuring Dimensions (2) Determined by Measuring Dimensions and Weight of Saturated Sample		(3) PLD=Point Load (diametrical), PLA= Point Load (Axial) ST= Splitting Tensile U= Unconfined Compressive Strength (4) Taken at Peak Deviator Stress		(5) Strain at Peak Deviator Stress (6) Represents Secant Modulus at 50% of Total Failure Stress (7) Represents Secant Poisson's Ratio at 50% of Total Failure Stress (8) Estimated UCS from Table 1 of ASTM D5731 for NX cores (Is x 24)

Date Received: *08.22.22* Reviewed By: Date Reviewed: *03.14.23*

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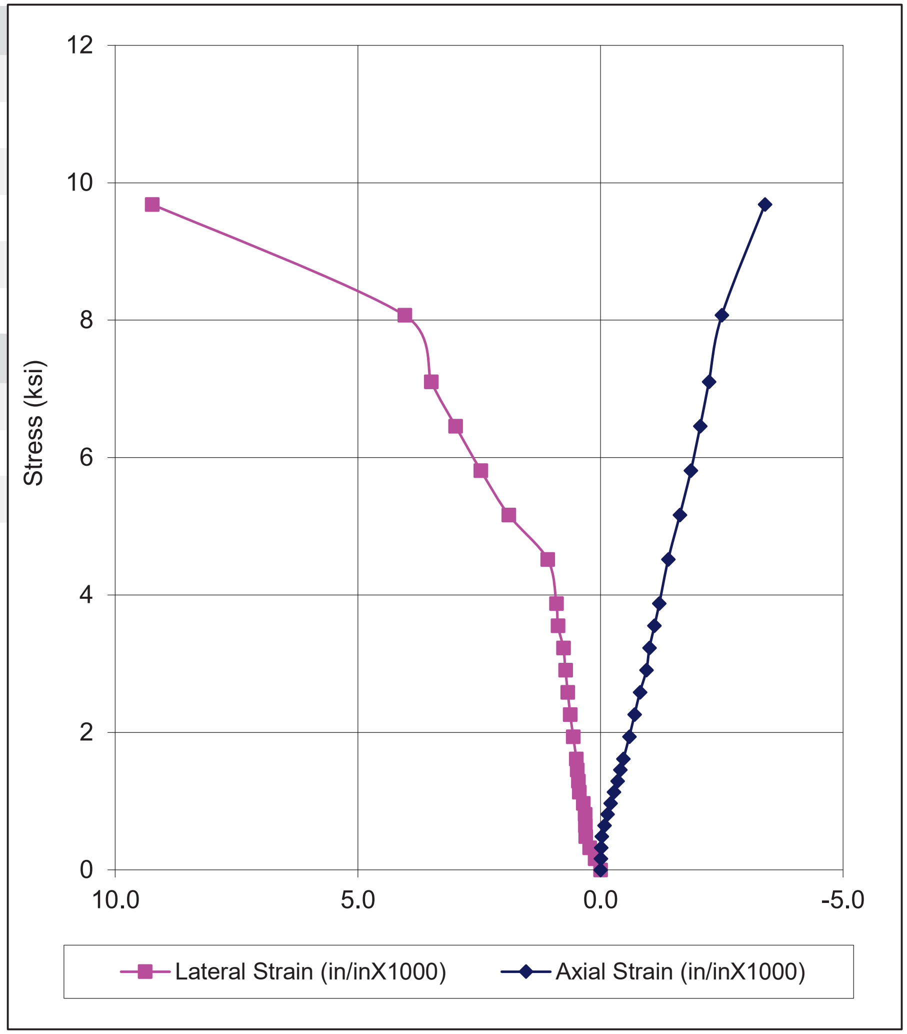
ASTM D7012 Compressive Strength and Elastic Moduli of Intact Rock Core Specimens

Sample Information		Compressive Test Information	
Boring ID:	BB-BHCB-103	**Unit Weight (pcf):**	170.9
Sample #:	R-1	**Failure Stress (psi):**	10,082
Depth (ft):	8-8.5	**Failure Mode:**	Discontinuity
Tested Depth (ft):		**Time to Failure (min)**	8.5
Rock Type:	Grey Metasandstone		
Features:	Broke along Discontinuity		

Test Specimen Information		Elastic Moduli Test Information	
Diameter, D (in):	1.986	**Poisson's Ratio @ 50%:**	0.75
Length, L (in):	4.322	**Strain %:**	0.339
L:D Ratio:	2.18	**E sec PSI @ 50%:**	3.20E+06



Testing Notes: **Sample had an early break at about 4500psi resulting in lower Poisson's Ratio and E.**
Early break possibly exhibited a strike/slip type shift which released pressure and may be the reason for the high Poisson's Ratio.

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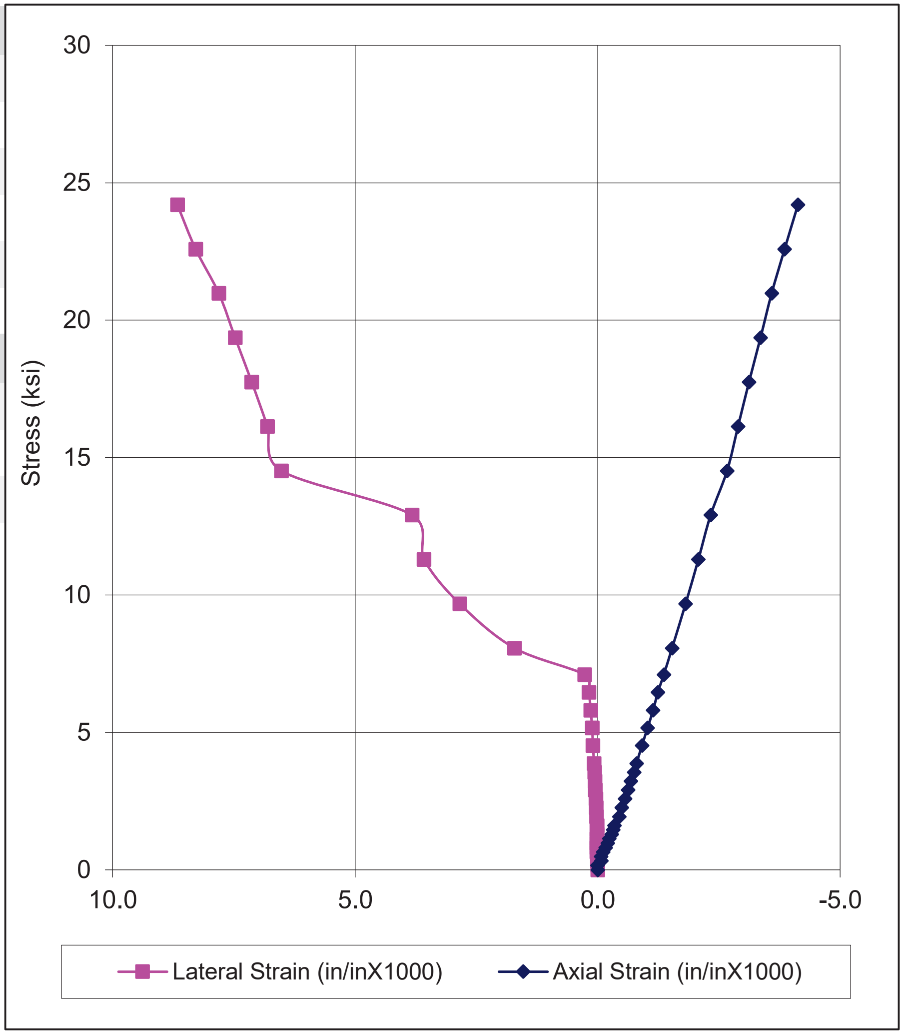
ASTM D7012 Compressive Strength and Elastic Moduli of Intact Rock Core Specimens

Sample Information		Compressive Test Information	
Boring ID:	BB-BHCB-101	**Unit Weight (pcf):**	169.2
Sample #:	R-2	**Failure Stress (psi):**	26,522
Depth (ft):	14.3-15.05	**Failure Mode:**	Discontinuity
Tested Depth (ft):		**Time to Failure (min)**	8.5
Rock Type:	Grey Metasandstone		
Features:	Broke along Discontinuity		

Test Specimen Information		Elastic Moduli Test Information	
Diameter, D (in):	1.987	**Poisson's Ratio @ 50%:**	0.09
Length, L (in):	4.512	**Strain %:**	0.413
L:D Ratio:	2.27	**E sec PSI @ 50%:**	4.73E+06



Testing Notes: **Sample had an early break at about 7098 psi resulting in a lower Poisson's Ratio.**

APPENDIX D – ROCK CORE PHOTOGRAPHS

MaineDOT Cromwell Brook #3 Bridge #0452
Ledgelawn Avenue Extension over Cromwell Brook
Bar Harbor, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-BHCB-102	R1	12.5 - 15.5	36	100	4	11	METASANDSTONE	1
BB-BHCB-102	R2	15.5 - 19.5	48	100	23	48	METASANDSTONE	1/2
BB-BHCB-102	R3	19.5 - 22.5	36	100	34.5	96	METASANDSTONE	2/3

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

MaineDOT Cromwell Brook #3 Bridge #0452
Ledgelawn Avenue Extension over Cromwell Brook
Bar Harbor, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)			Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-BHCB-103	R1	8	-	13	60	100	31	52	METASANDSTONE	1
BB-BHCB-103	R2	13	-	18	48	80	37.5	63	METASANDSTONE	2
BB-BHCB-101	R1	11.5	-	12.4	3	27	0	0	Gravel/Cobble	3
BB-BHCB-101	R2	12.4	-	16	38	88	33.5	78	METASANDSTONE	3
BB-BHCB-101	R3	16	-	18.6	31	100	22	71	METASANDSTONE	3/4

Notes:
1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 4=Bottom.
2. Top photo is dry, bottom photo is wet.
3. Transition between core runs within a row are marked by wood or paper separators.
4. Bottom 12” fell out of core barrel for BB-BHCB-103, R2, could not recover.

APPENDIX E – CALCULATIONS

Objective

Assess nominal and factored bearing resistance of a foundation on rock based on support in meta-sedimentary rock from borings BB-BHCB-101, -102 and -103.

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 Cromwell Brook Bridge Project in Bar Harbor, ME. This calculation is based on the data from borings BB-HBCB-101, -102, and -103.

Bedrock Quality

Representative RQD's are shown in the table below.

Boring	Run	GS Elevation	Depth to Rock (ft)	Depth (ft) Below Top of Rock			Length of Core Run (ft)	Rec (in)	Rec (%)	RQD (in)	RQD %	Joint Spacing Desc.	Corr. Spacing (in)	Aperture Desc.	Corr. Aperture (in)
				Top		Bottom									
BB-BHCB-101	R1	51.5	11.7	0.0	-	0.7	0.7	3	36%	0	0%				
BB-BHCB-101	R2	51.5	11.7	0.7	-	4.3	3.6	38	88%	34	78%	Close to Moderate	2.5-24	Partially Open to Open	.01-0.1
BB-BHCB-101	R3	51.5	11.7	4.3	-	6.9	2.6	31	99%	22	71%	Very Close to Moderate	0.75-24	Tight to Open	.004-0.1
BB-BHCB-102	R1	45.2	12.0	0.5	-	3.5	3.0	36	100%	4	11%	Close	2.5-8	Partially Open to Open	.01-0.1
BB-BHCB-102	R2	45.2	12.0	3.5	-	7.5	4.0	48	100%	23	48%	Close	2.5-8	Open to Moderate	0.02-0.4
BB-BHCB-102	R3	45.2	12.0	7.5	-	10.5	3.0	36	100%	35	96%	Close to Moderate	2.5-24	Partially Open to Open	.01-0.1
BB-BHCB-103	R1	51.6	7.8	0.2	-	5.2	5.0	60	100%	31	52%	Very Close to Close	0.75-8	Open to Moderate	0.02-0.4
BB-BHCB-103	R2	51.6	7.8	5.2	-	10.2	5.0	48	80%	38	63%	Very Close to Close	0.75-8	Partially Open to wide	0.01->0.4

Note: "UCS" is unconfined compressive strength. UCS values marked with "*" are from diametrical (D) and axial (A) point load tests and are correlated from the point load test results.

RQD between 11% and 96% for core runs at each location, neglect a 0% at top of rock for 0.7' run at BB-BHCB-101 R1. Representative RQD of 50-75% range selected.

Bedrock Strength

Boring	Run	LAB								Rock Type
		Depth of Sample (ft)	Depth of Sample into Rock (ft)	Elev Top of Sample (ft)	UCS (psi)	Point Load Index, Is50 (psi)	Correlated UCS from Point Load Tests (psi)	Modulus (ksi)	Unit Wt (pcf)	
BB-BHCB-101	R1									META-SEDIMENTARY
BB-BHCB-101	R2	14.3	2.6	37.2	26,522	--		4	169.2	META-SEDIMENTARY
BB-BHCB-101	R3									META-SEDIMENTARY
BB-BHCB-102	R1	14.1	1.6	31.2	--	234 (D), 303 (A)	5,616 (D), 7,272 (A)	--	178.0 (D), 168.6 (A)	META-SEDIMENTARY
BB-BHCB-102	R2									META-SEDIMENTARY
BB-BHCB-102	R3									META-SEDIMENTARY
BB-BHCB-103	R1	8.0	0.0	43.6	10,372	--		3	170.9	META-SEDIMENTARY
BB-BHCB-103	R1	12.1	4.1	39.5	--	2266 (D), 1813 (A)	54,384 (D), 43,512 (A)	--	179.2 (D), 187.9 (A)	META-SEDIMENTARY
BB-BHCB-103	R2									META-SEDIMENTARY

Note: "UCS" is unconfined compressive strength. UCS values marked with "*" are from diametrical (D) and axial (A) point load tests and are correlated from the point load test results.

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

$$\sigma_{u.r} := \frac{(5.62 \cdot ksi + 10.37 \cdot ksi + 7.27 \cdot ksi)}{3} = 1116.48 \cdot ksf$$

Unconfined compressive strength varies from approximately 10.4 to 26.5 ksi and correlated strength from point load tests ranges from 5.6 to 54.4 ksi. Take the average of the lowest UCS and two lowest PLTs for design.

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_1 := 7$ for $\sigma_{u.r}$ between 1,080 and 2,160 ksf

Parameter 2- Drill Core Quality

Representative RQD from table above: 11 -96% for abutment borings; choose 50-75%

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_2 := 13$

Parameter 3- Spacing of Joints

From Boring Logs, generally very close to moderately spaced = 0.3 in to 2 feet, Typical spacing was 2.5 in. to 8 in.

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating

$RR_3 := 10$

Parameter 4- Condition of Joints

From boring logs, hard joint walls and appeared smooth to rough on surface, and described fresh to discolored, joints are typically tight.

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_4 := 20$

Parameter 5- Ground Water Conditions

Hydrostatic Conditions- Tremie seals bearing on rock below the brook water level. Assume interstitial water pressure

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_5 := 7$

Parameter 6-Adjustment for joint orientation

The joint sets are generally moderately dipping to high angle and generally smooth and tight. Joints will not daylight below foundations because they will be at brook level. Assume fair conditions.

From AASHTO LRFD Table 10.4.6.4-2

Relative Rating $RR_6 := -7$

Total RMR Rating

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

$$RMR = 50$$

From AASHTO LRFD Table 10.4.6.4-3 RMR 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

Phyllite is categorized as rock type B, lithified argrillaceous rock rocks, 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).

$m := 0.285$

$s := 0.00025$

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

From Wyllie "Foundations on Rock"

Eq. 5.4 Pg.138

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

Where

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

$s = 0.00025$ For L/B>6, use factor C_{fl}=1.0,

$m = 0.285$ For L/B=1, use factor C_{fl}=1.12, therefore,

$\sigma_{u.r} = 7.753333 \cdot ksi$ For conservatism, assume long strip, lowest $C_{fl.}$

Nominal Bearing Resistance

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

$q_n = 94.7 \cdot ksf$ Say 94 ksf

Factored Bearing Resistance (Strength Condition)

Bearing Resistance Factor is specified in Table 10.5.5.2.2-1

$\phi_b := 0.45$ Footing on rock

$q_R := \phi_b \cdot q_n$

$q_R = 42.6 \cdot ksf$ Say 42 ksf

Reference:I:\Mathcad\units.xmcd

Table 10.4.6.4-1 Geomechanics Classification of Rock Masses.

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

Table 10.4.6.4-2 Geomechanics Rating Adjustment for Joint Orientations.

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

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

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

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

Rock Type

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

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

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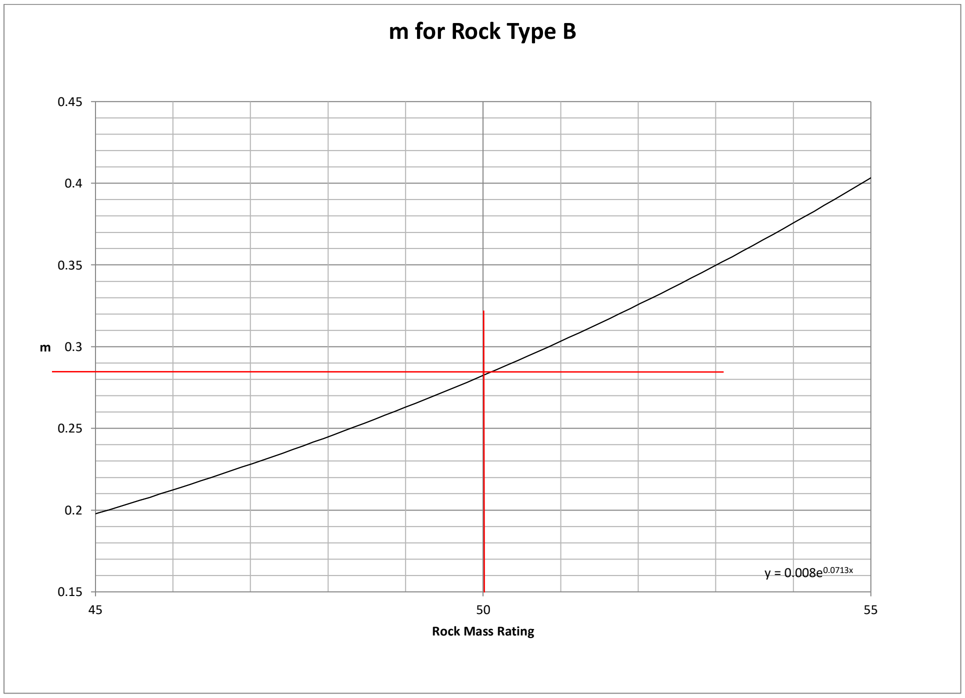
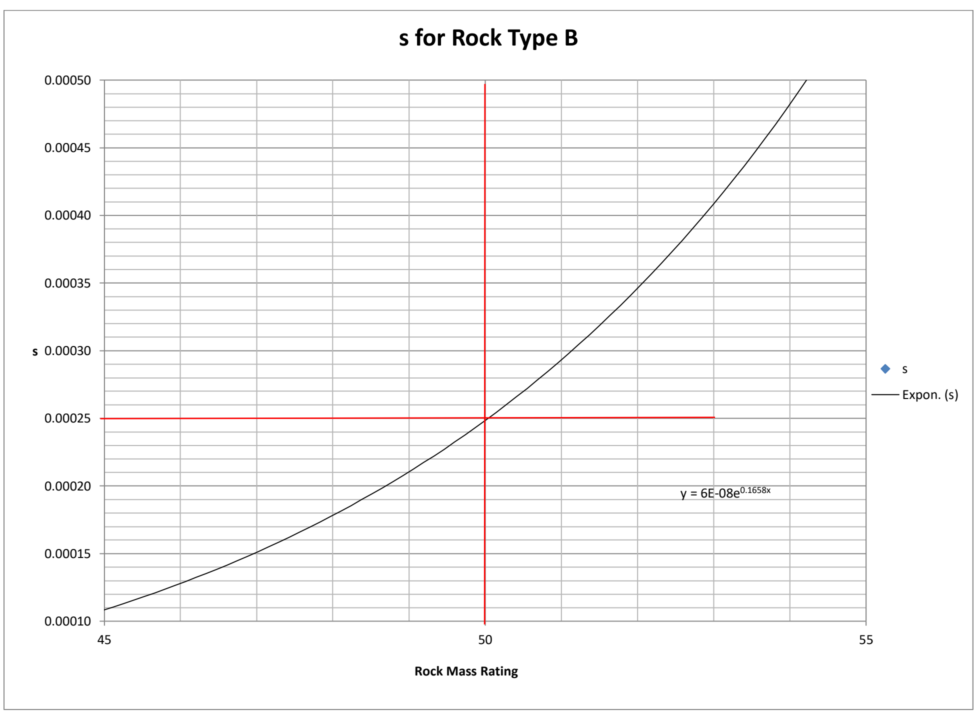
Engineers and Scientists





March 2014

Figure 5-1 Maine Design Freezing Index Map

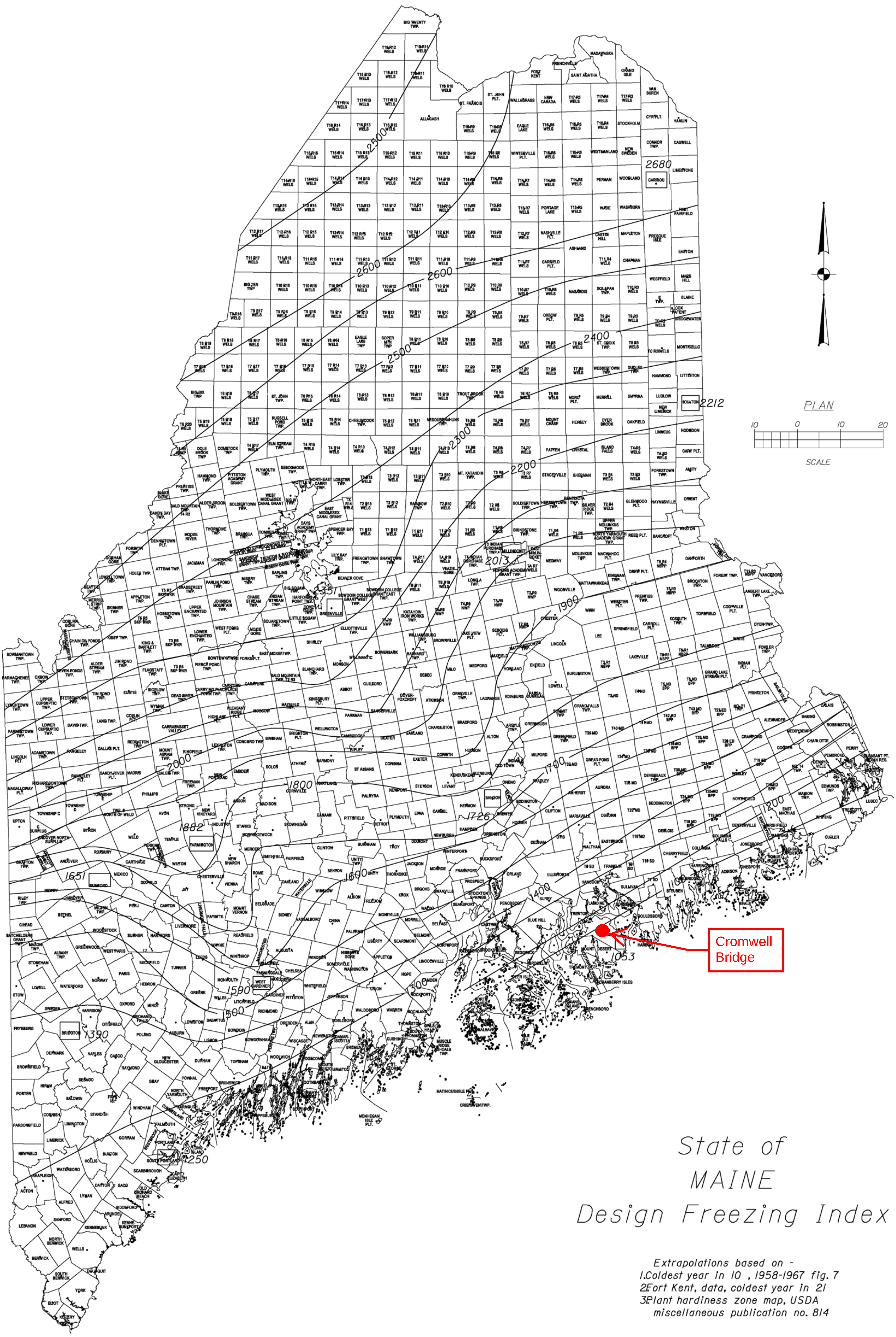


Table 5-1 Depth of Frost Penetration

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	[illegible]	51.9	44.7	40.5
1300	76.3	63.0	54.3	54.2	46.6	42.2
1400	79.2	65.5	56.4	56.3	48.5	43.9
1500	82.1	67.9	58.4	58.3	50.2	45.4
1600	84.8	70.2	60.3	60.2	51.9	46.9
1700	87.5	72.4	62.2	62.2	53.5	48.4
1800	90.1	74.5	64.0	64.0	55.1	49.8
1900	92.6	76.6	65.7	65.8	56.7	51.1
2000	95.1	78.7	67.5	67.6	58.2	52.5
2100	97.6	80.7	69.2	69.3	59.7	53.8
2200	100.0	82.6	70.8	71.0	61.1	55.1
2300	102.3	84.5	72.4	72.7	62.5	56.4
2400	104.6	86.4	74.0	74.3	63.9	57.6
2500	106.9	88.2	75.6	75.9	65.2	58.8
2600	109.1	89.9	77.1	77.5	66.5	60.0

63.8" = 5.25'

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.

The Freezing Index for the site is 1,100, and with low to moderate moisture content (<10 to 20 percent) soils, the estimated depth of frost penetration is approximately 5.25 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 and A-2-4(0) with MaineDOT Frost Classification from II, 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.

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Engineers and Scientists

JOB: 09.0026155.01 Cromwell Bridge
SUBJECT: Lateral Earth Pressures
SHEET: 1 OF 1
CALCULATED BY B. Cardali 11/4/24
CHECKED BY A.Blaisdell 11/5/24

Subject:

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

References:

1. MaineDOT Bridge Design Guide, Chapter 3 and 5 (BDG)
2. AASHTO LRFD Bridge Design Specifications, 9th Edition (2020)

Input Parameters:

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

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

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

$\theta := 90 \cdot \text{deg}$ — Angle of back face of wall to the horizontal

Earth Pressure Coefficients:

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

Passive Earth Pressure (End Diaphragms)

Per BDG Section 5.4.2.11, developing full passive pressure requires that ratio of lateral abutment movement (y) to abutment height (Hb) exceeds 0.005. If the calculated rotation is significantly less, Rankine earth pressure may be considered. However, we understand that recent practice by MaineDOT is to utilize methodology consistent with MassDOT Section 3.10.8.

$y := 0.25\text{in}$ — Maximum deflection from thermal expansion provided by structural engineer.

$H_b := 2\text{ft}$ — End Diaphragm Height

$\frac{y}{H_b} = 0.0104$ — Ratio of lateral movement to abutment height

MassDOT Section 3.10.8 presents the plot and calculation shown below for a gravel borrow material.

$$\omega := \frac{y}{H_b} = 0.0104$$

$$K_{p.mass} := 0.43 + 5.7\left(1 - \exp\left(-190 \cdot \frac{y}{H_b}\right)\right)$$

$$\boxed{K_{p.mass} = 5.34}$$

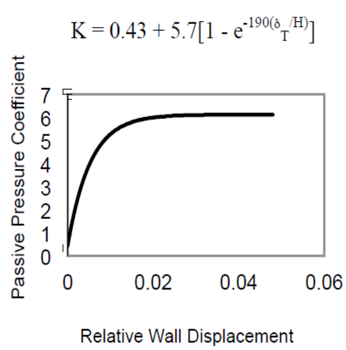

Figure 3.10.8-1: Plot of Passive Pressure Coefficient, K, vs. Relative Wall Displacement, δ_T/H.

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Engineers and Scientists

JOB: 09.0026155.01 Cromwell Bridge
SUBJECT: Lateral Earth Pressures
SHEET: 2 OF 1
CALCULATED BY B. Cardali 11/4/24
CHECKED BY A.Blaisdell 11/5/24

Active Earth Pressure (Abutments and Wingwalls)

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

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

$$\mathrm{heel} := 5.5\mathrm{ft}$$

$$\mathrm{Intersection}_{\mathrm{height}} := \tan(90\mathrm{deg} - \alpha)\cdot\mathrm{heel} = 10\cdot\mathrm{ft}$$

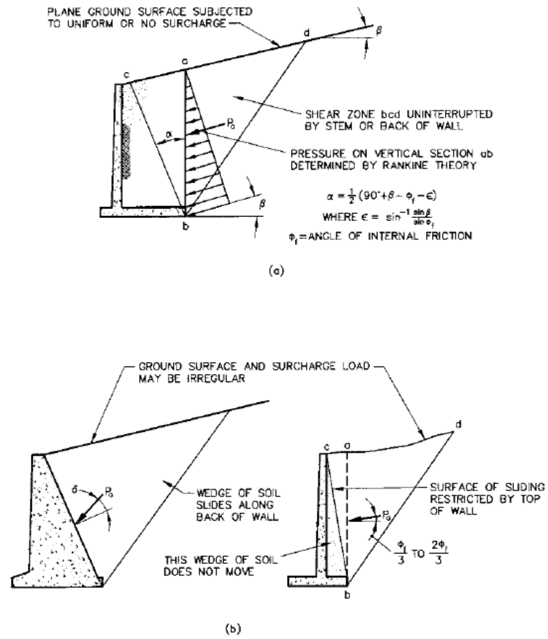


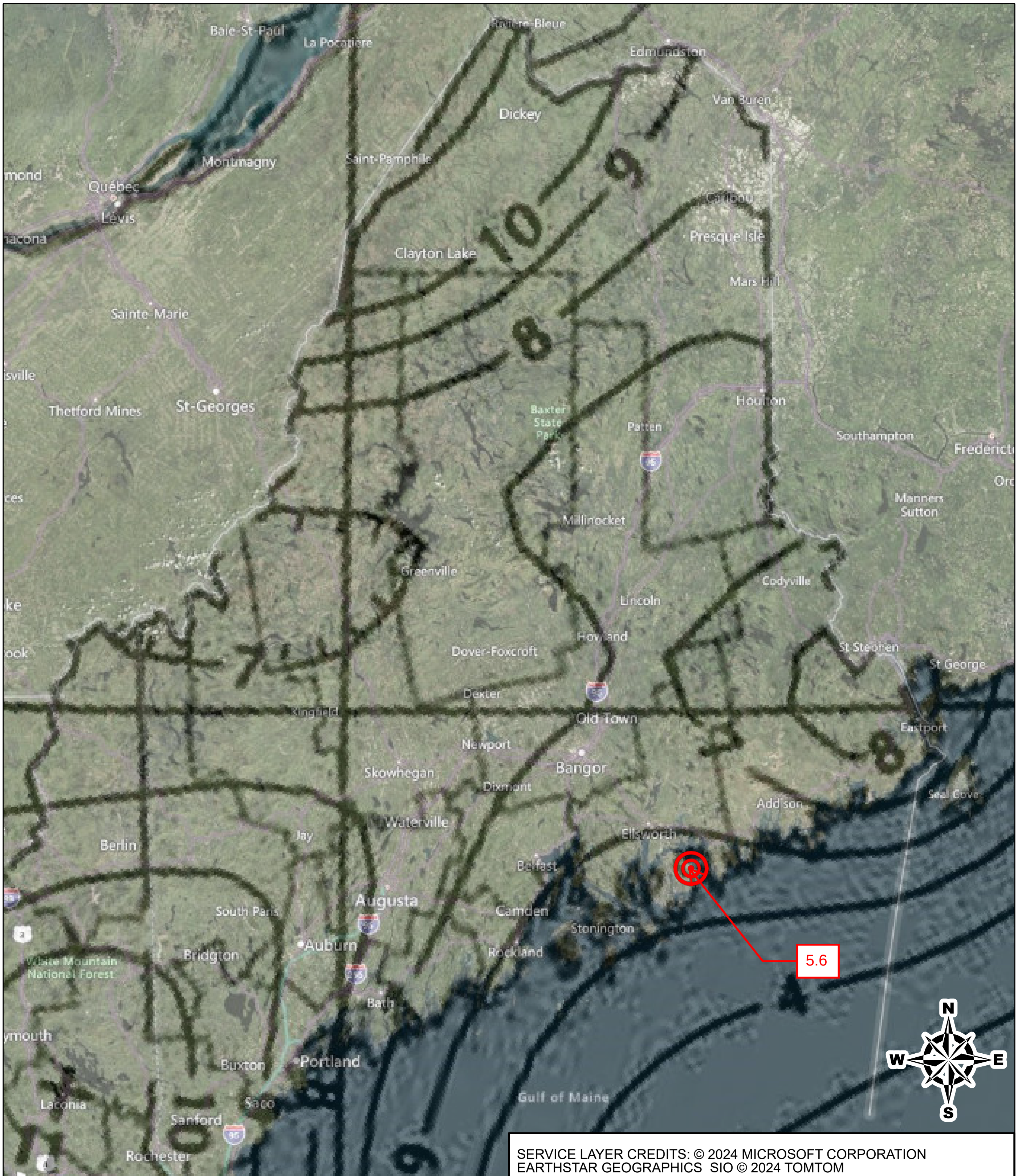
Figure C3.11.5.3-1—Application of (a) Rankine and (b) Coulomb Earth Pressure Theories in Retaining Wall Design

The abutment height is 10 feet (below the end diaphragm). Based on Figure C3.11.5.3-1 of LRFD, the abutment is considered to be a short-heeled wall. Therefore, Coulomb theory should be used to calculate active earth pressures.

Coloumb Active Earth Pressure Coefficient (Short-Heeled Wall)

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

$$K_{ac} := \frac{(\sin(\theta + \phi))^2}{\Gamma\cdot\left[(\sin(\theta))^2\cdot\sin(\theta - \delta_f)\right]} \qquad \boxed{K_{ac} = 0.28}$$



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0 15 30 60
mi

Legend

CROMWELL BROOK BRIDGE

140 GREAT MEADOW DR, BAR HARBOR, ME

PREPARED BY:

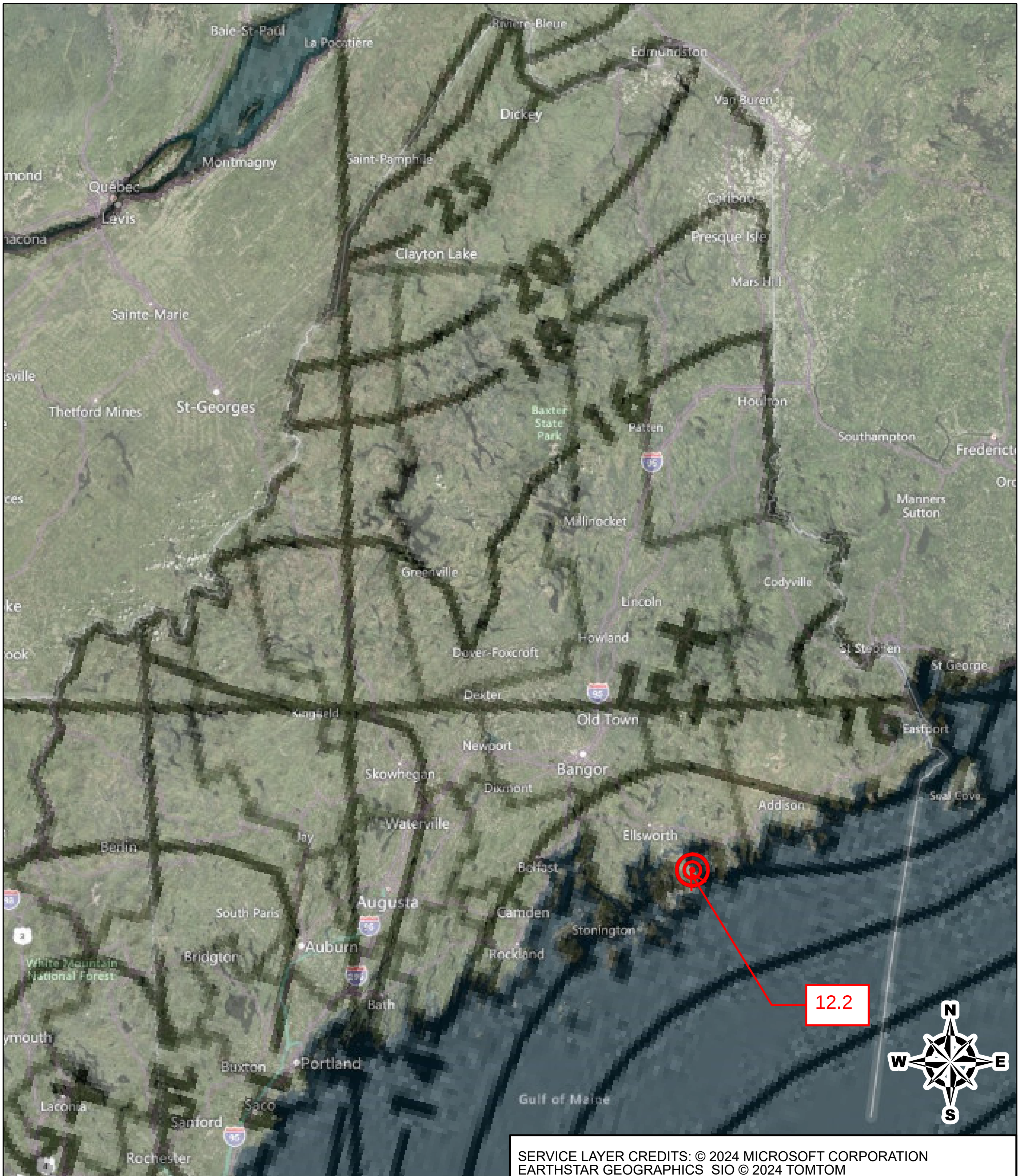
GZA GeoEnvironmental, Inc.
www.gza.com

PREPARED FOR:

MAINEDOT

HORIZONTAL PEAK GROUND ACCELERATION COEFFICIENT (PGA)

PROJ MGR: BMC	REVIEWED BY: CLS	CHECKED BY: BMC	FIGURE
DESIGNED BY:	DRAWN BY: ENT	SCALE: 1:3,174,534	1
DATE: 11/05/2024	PROJECT NO: 09.0026155.00	REVISION NO:	



Legend

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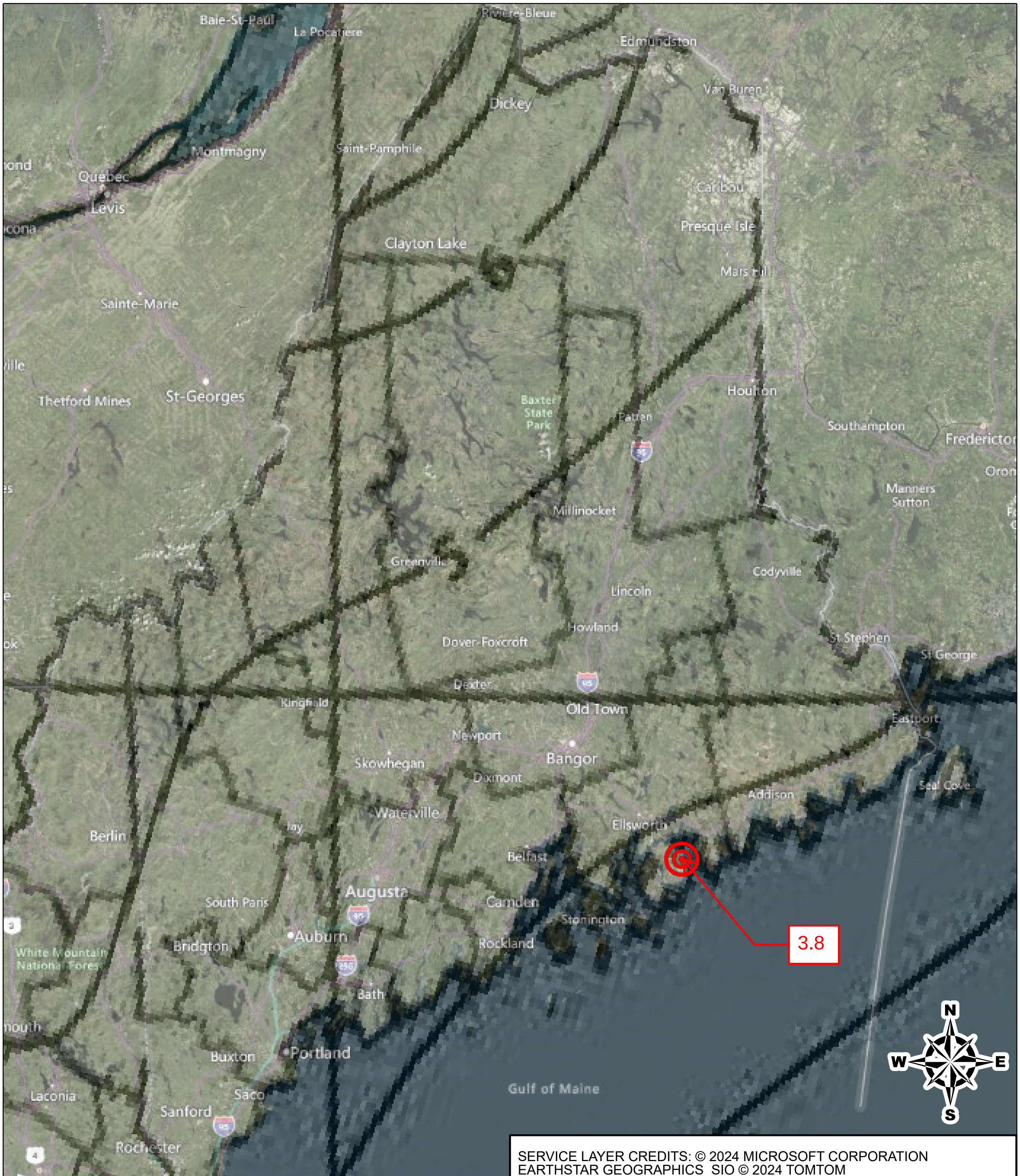
0 15 30 60
mi

CROMWELL BROOK BRIDGE

140 GREAT MEADOW DR, BAR HARBOR, ME

HORIZONTAL RESPONSE SPECTRAL ACCELERATION COEF. FOR PERIOD OF 0.2s (Ss)

PREPARED BY: GZA GeoEnvironmental, Inc. www.gza.com		PREPARED FOR: MAINEDOT	
PROJ MGR: BMC	REVIEWED BY: CLS	CHECKED BY: BMC	FIGURE 2
DESIGNED BY:	DRAWN BY: ENT	SCALE: 1:3,174,534	
DATE: 11/05/2024	PROJECT NO: 09.0026155.00	REVISION NO:	



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0 15 30 60
mi

Legend

CROMWELL BROOK BRIDGE

140 GREAT MEADOW DR, BAR HARBOR, ME

PREPARED BY: GZA GeoEnvironmental, Inc. www.gza.com

PREPARED FOR: MAINEDOT

HORIZONTAL RESPONSE SPECTRAL ACCELERATION COEF. FOR PERIOD OF 0.1s (S1)

PROJ MGR: BMC	REVIEWED BY: CLS	CHECKED BY: BMC	FIGURE
DESIGNED BY:	DRAWN BY: ENT	SCALE: 1:3,174,534	3
DATE: 11/05/2024	PROJECT NO: 09.0026155.00	REVISION NO:	

Seismic Parameter	Design Parameter[1]
Horizontal Peak ground Acceleration Coefficient	$PGA = .056$
Horizontal Response Spectral Acceleration Coefficient for Period of 0.2s	$S_s = 0.122$
Horizontal Response Spectral Acceleration Coefficient for Period of 1.0s	$S_1 = .038$

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

For Class B, values of F_{PGA}, F_a, and F_v = 1.0

Therefore:

$$A_s = F_{PGA} \times PGA = 1.0 \times 0.056 = 0.056\ g$$

$$S_{DS} = F_a \times S_s = 1.0 \times 0.122 = 0.122\ g$$

$$S_{D1} = F_v \times S_1 = 1.0 \times 0.038 = 0.038\ g$$

Summary:

SITE CLASS B SEISMIC DESIGN PARAMETERS	
Parameter	**Design Value**
Fpga	1.0
Fa	1.0
Fv	1.0
As (Period = 0.0 sec)	0.06 g
SDs (Period = 0.2 sec)	0.12 g
SD1 (Period = 1.0 sec)	0.04 g