GEOTECHNICAL DESIGN REPORT
TANNERY BROOK BRIDGE NO. 3511
MARIAVILLE ROAD OVER TANNERY BROOK
MARIAVILLE, MAINE

August 2025
09.0026171.01

Prepared for:
Stantec
Portland, Maine

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

Offices Nationwide
www.gza.com

Known for excellence.
Built on trust.

GEOTECHNICAL
ENVIRONMENTAL
ECOLOGICAL
WATER
CONSTRUCTION MANAGEMENT

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

VIA EMAIL

August 21, 2025
File No. 09.0026171.01

Ms. Sarah Williams, P.E.
Stantec
2211 Congress Street, Suite 380
Portland, ME 04096

Re: Geotechnical Design Report
Replacement of Tannery Brook Bridge No. 3511, Mariaville Road over Tannery Brook
MaineDOT WIN 26107.00
Mariaville, Maine

Dear Sarah:

We are pleased to provide this Geotechnical Design Report (GDR) to Stantec for the subject project. Our work was completed in accordance with the Agreement for Professional Services between Stantec and GZA GeoEnvironmental, Inc. (GZA) dated December 13, 2024, which incorporates our September 19, 2024 proposal, and the *Limitations* included in **Appendix A** of this report. GZA is providing geotechnical engineering services as a subconsultant to Stantec, who is under contract with the Maine Department of Transportation for design of the proposed bridge replacement.

It has been a pleasure serving Stantec 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.

Nicholas V. Williams, P.E.
Project Manager

Christopher L. Snow, P.E.
Consultant Reviewer

Andrew R. Blaisdell, P.E.
Associate Principal

NVW/ARB/CLS:cc
j:\09 jobs\0026100s\09.0026171.00 - stantec-medot, tannery bridge, mariaville\01 - final design\report\draft 09.0026171.01 - mariaville_tannery brook bridge gdr 07.01.25.docx

Enc: Geotechnical Design Report

TABLE OF CONTENTS

Page

TABLE OF CONTENTS (*continued*)

TABLES

FIGURES

APPENDICES

1.0 INTRODUCTION

This report presents the results of the final design geotechnical evaluation by GZA GeoEnvironmental, Inc. (GZA) for the replacement of Tannery Brook Bridge No. 3511 in Mariaville, Maine. Our services were provided in accordance with GZA's proposal No. 09.P000108.25, dated September 19, 2024, and the *Limitations* included in **Appendix A** of this report. Stantec is serving as the bridge designer for MaineDOT.

1.1 BACKGROUND

The project includes replacement of the Tannery Brook Bridge No. 3511 carrying Mariaville Road over Tannery Brook in Mariaville, Maine. The existing bridge was constructed circa 1937 and is an approximately 26-foot-long, single-span bridge with reinforced concrete girders and concrete abutments bearing on spread footings. Large grout bags were placed along both abutments to reduce scour issues. Additional repairs to the bridge structure included rehabilitation of the fascia, a new curb and rail in 1996, and rehabilitation of the substructures of 2014.

We understand that a replacement bridge is proposed to consist of a 48-foot-long, single-span bridge with a 20-degree skew consisting of precast NEXT beams supporting a cast-in-place structural concrete deck with integral wearing surface and semi-integral end diaphragm wall. The superstructure will be supported by cast-in-place concrete abutments over spread footings bearing on bedrock. A raise in grade of approximately 2 to 3 feet is planned at the approaches to the replacement bridge. We understand that a bridge closure and temporary detour are planned during construction.

1.2 OBJECTIVES AND SCOPE OF SERVICES

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

- Conducted site visits to observe surficial conditions and reviewed mapped surficial and bedrock geology of the site;
- Reviewed existing subsurface data;
- Coordinated and observed subsurface explorations, consisting of seven (7) test borings (including five unsuccessful attempts) to evaluate subsurface conditions;
- Conducted a laboratory testing program to evaluate engineering and index properties of the site soils and bedrock;
- Conducted final design geotechnical engineering analyses to evaluate foundation 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 an exploration program consisting of seven test borings (BB-MTB-101, BB-MTB-102, and BB-MTB-102A through -102E). The borings were drilled near the proposed abutment locations for the replacement bridge.

Borings BB-MTB-101 and BB-MTB-102E were drilled to depths of approximately 24.5 to 26.8 feet below ground surface (bgs) and terminated approximately 10 feet into bedrock. There were five unsuccessful attempts to drill the Abutment 2 boring using solid stem auger and/or drive-and-wash techniques (BB-MTB-102 and -102A through -102D) that encountered refusal on probable cobbles and boulders or could not maintain vertical casing alignment at depths ranging between 0.7 and 9.0 feet. BB-MTB-102E switched to spin-and-wash technique and penetrated the depth that created the refusals. New England Boring Contractors of Derry, New Hampshire provided drilling services and coordinated utility clearance. The drilling was completed on October 25 and October 26, 2022, using a truck-mounted drill rig. GZA personnel monitored the drilling work and prepared logs of each boring that are included in **Appendix B**.

The borings were drilled using solid stem augers and 4-inch spin-and-wash (BB-MTB-102E only) and drive-and-wash cased drilling techniques. Standard penetration testing and split-spoon sampling were performed at 5-foot typical intervals in the borings using a 24-inch-long, 1-3/8-inch inside diameter sampler, driven with an automatic hammer with a rated hammer efficiency factor of 0.87. Between 10 and 10.3 feet of 2-inch-diameter rock core was collected from two borings using an NX core barrel. Wet and dry photographs of the collected rock core specimens are included in **Appendix D**.

The as-drilled boring locations and elevations were surveyed by MaineDOT and provided to GZA and are shown on the logs in **Appendix B** and in **Figure 2.** Elevations referenced in this report are in feet and refer to North American Vertical Datum of 1988 (NAVD 88).

3.0 LABORATORY TESTING

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

- Six (6) gradation analyses (including two (2) with hydrometers);
- Seven (7) moisture content tests;
- One (1) Atterberg limits analysis;
- Two (2) unconfined compressive strength / secant modulus tests; and
- Two (2) point load tests (one [1] axial and one [1] diametrical).

Results of the testing are included in **Appendix C** and summarized in **Section 4.2**.

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 glacial marine deposits (Qp; silt and clay) at the bridge and glacial stream deposits (Qg; sand and gravel) which are mapped in upland areas to the north and south of the site and can be overlain by glacial marine deposits.

Available bedrock geologic mapping[2] indicates that the site is near a contact between the Devonian-Ordovician Bucksport formation (described as Metasandstone and Metalimestone with Greenschist facies) and the Penobscot Formation (described as Slate, Schist, and Quartzite with Greenschist facies).

4.2 SUBSURFACE PROFILE

Up to three soil units were encountered in the test borings above bedrock: Fill, Marine Deposit and Glacial Stream Deposit. Approximately 0.5 to 0.8 feet of asphalt pavement was encountered in the test borings. The thicknesses and generalized descriptions of the soil units are presented in the following table, in descending order from existing ground surface. Detailed descriptions of the materials encountered at specific locations are provided in the boring logs in **Appendix B**. An interpretive subsurface profile was developed representing generalized stratification along the proposed bridge alignment and is presented on **Figure 2**. The strata thicknesses and elevations are summarized for each boring in **Table 1**.

---

[1] Borns, Harold W., Jr., and Andersen, Bjorn, 1982, Reconnaissance surficial geology of the Ellsworth quadrangle, Maine: Maine Geological Survey, Open-File Map 82-3, map, scale 1:62,500.

[2] Osberg, Philip H., Hussey, Arthur M., II, and Boone, Gary M., (Editors), 1985, Bedrock geologic map of Maine: Maine Geological Survey, 1 plate, correlation chart, tectonic inset map, metamorphic inset map, color geologic map, cross sections, scale 1:500,000.

GENERALIZED SUBSURFACE CONDITIONS		
Subsurface Unit	**Approximate Encountered Thickness (ft)**	**Generalized Description and Laboratory Testing Results**
Fill	2.9 to 8.3	Brown, medium dense to very dense, fine to coarse SAND, some gravel, little silt, to; brown, Sandy GRAVEL, little silt. Cobbles/boulders present at all BB-MTB-102 locations; oversized materials may have influenced blow counts and density interpretations. MaineDOT Frost Classification Range = II to III Results of 2 Grain Size and 2 Moisture Content Analyses: • AASHTO Classification: A-1-b • USCS Classification: SM • Moisture Content: 3.9 to 6.3% *Encountered in all borings (penetrated full thickness in BB-MTB-101 and -102E).*
Marine Deposit	4.7	Brown and grey, very stiff, Silty CLAY, trace sand, to; Grey, very stiff, Sandy Clayey SILT. MaineDOT Frost Classification = III to IV Results of 1 Grain Size/Hydrometer, 1 Atterberg Limits and 2 Moisture Content Analyses: • USCS Classification: ML, CL • Moisture Content: 16.7 to 22.0% • Liquid Limit: 33 • Plastic Limit: 18 • Plasticity Index: 15 *Only encountered in boring BB-MTB-101*
Glacial Stream Deposit	5.0 to 7.2	Brown to grey, very dense, fine to coarse SAND, some gravel, some to little silt, to; brown to grey, very dense, GRAVEL, some fine to coarse sand, trace silt. Contains cobbles and boulders. MaineDOT Frost Classification Range = I to II Results of 3 Grain Size and 3 Moisture Content Analyses: • AASHTO Classification: A-1-a, A-1-b • USCS Classification: GW-GM, SM • Moisture Content: 5.4 to 12.3% *Encountered in both borings that penetrated the Fill (BB-MTB-101 and -102E).*
Top of Bedrock Elevation	Abutment 1: Approximately El. 102.3 (15.7 feet bgs) Abutment 2: Approximately El. 103.4 (14.0 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 two test borings and was described hard, fresh, aphanitic to fine grained, grey, SLATE with occasional calcite stringers. Primary joints are extremely close to moderately spaced, low angle to vertical, planar to undulating, smooth to rough, fresh to discolored, very tight to tight. Secondary joints are very close to moderately spaced, low angle to moderately dipping, planar, smooth to rough, fresh, tight to partially open. The Rock Quality Designation in the core runs ranged from 36 to 88 percent, indicating a Rock Quality of Poor to Good. The bedrock core data are summarized in **Table 2**. Wet and dry photographs of the collected rock core are included in **Appendix D**.

Unconfined compressive strength (UCS) testing was conducted on two core samples. The results of the UCS strength ranged from approximately 5,960 to 6,820 psi. Both tests broke along the foliation.

Axial and diametral point load testing were conducted on two samples. The size corrected point load index, $I_{s(50mm)}$, ranged from 297 to 330 psi, which suggests an unconfined compressive strength ranging from approximately 7,100 to 7,900 psi based on correlation factors between $I_{s(50mm)}$ and UCS from ASTM D5731. Laboratory test results are included in **Appendix C**.

4.2.2 Groundwater

Groundwater depth was measured in borings BB-MTB-101 and -102E at approximately 8.3 and 8.6 feet bgs, corresponding to El. 109.4 to 109.0 respectively, which is approximately at the brook level. Groundwater levels in the borings were measured immediately after drilling and may have been affected by drilling procedures, which included introduction of water for drilling purposes.

Fluctuations in groundwater levels will occur due to variations in river level, season, precipitation, and construction activity in the area. Consequently, water levels during and after construction are likely to vary from those encountered in the borings at the time the observations were made.

5.0 ENGINEERING EVALUATIONS

5.1 GENERAL

GZA conducted geotechnical engineering evaluations in accordance with 2024 AASHTO LRFD Bridge Design Specifications, 10th Edition (herein designated as AASHTO) and the *MaineDOT Bridge Design Guide, 2003 Edition*, with updates through 2018 (MaineDOT BDG). The sections that follow describe the evaluations and the geotechnical basis for each element. Supporting calculations are included in **Appendix E**.

5.2 PROPOSED CONSTRUCTION

We understand that a full bridge replacement is planned for the project. The current alternative includes increasing the span length to 48 feet. The new abutments will be located on the current alignment, approximately 10 to 13 feet behind the existing abutments.

5.3 APPROACH EMBANKMENTS

Typical grade raises of 2.5 feet or less are shown on the drawings at the approaches to the new bridge. Where embankment widening is proposed, proposed grade raises of 3.0 feet or less are shown. The approach embankments are proposed with typical side slope angles of 2 horizontal to 1 vertical (2H:1V), or flatter, including the ground surface in front of each abutment, which will slope down to the river level at an inclination of approximately 3H:1V and will be protected by riprap.

We anticipate that the proposed embankment fills will be constructed primarily over medium dense to dense fills, stiff Silt and Clay, 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 built within a cofferdam over tremie seals on bedrock are the most appropriate foundation system for the abutments and wing walls. Recommendations for spread footing design are provided in **Section 6.3**.

5.5 SEISMIC DESIGN CONSIDERATIONS

AASHTO LRFD Section 3.10.3.1 was used to conduct seismic design evaluations. The 10*th* Edition references the site class provisions included in ASCE 7-22, using the average shear wave velocity in the upper 100 feet of soil and bedrock (V*s-BAR*) to determine the site class. Because the bridge is a single-span structure bearing on bedrock, the bedrock will control seismic response and design. Article 20.2.4 of ASCE 7-22 specifies that Site Class BC shall be assumed in the absence of shear wave velocity measured on-site for competent bedrock with moderate fracturing and weathering.

Spectral design accelerations were developed using the AASHTO-USGS Seismic Design Ground Motion Database (2023). In accordance with AASHTO Section 3.10.3.1, the maximum spectral values of the range in site classes were selected for evaluation of Seismic Zone and (if necessary) for seismic analysis.

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 Tables 3.4.1-1 and 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 (10th 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 10th 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**
Slate	50-75	5.96	47	0.228	0.00015

Based on these parameters, the calculated nominal bearing resistance is 57 kips per square foot (ksf), resulting in a factored bearing resistance of 25.5 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 or on concrete with a raked surface. 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), resulting in a factored sliding resistance coefficient of 0.56 for the strength limit state.

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 1,500, and with low to moderate moisture content (±10 percent) soils, the estimated depth of frost penetration is 5.8 feet. However, where abutment foundations bear directly on sound rock, there is no minimum requirement for footing embedment.

5.8.2 Lateral Earth Pressures

The material properties will be controlled by the backfill material, which is proposed to consist of BDG Type 4 soil. In accordance with the requirements of the BDG Section 5.4.3, the semi-integral 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 superstructure 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. Stantec provided a maximum expansion deflection of approximately 0.2 inches for use in end diaphragm design. The end diaphragm height is approximately 2 feet resulting in a calculated abutment rotation of 0.0083 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 Seismic Zone for the bridge was evaluated in accordance with AASHTO 3.10.6. S_{D1} was calculated to be 0.04. Based on AASHTO Table 3.10.6-1, the bridge should be assigned to Seismic Zone 1. Therefore, based on AASHTO 4.7.4, seismic analysis is not required. Minimum horizontal design connection forces should be determined in accordance with AASHTO 3.10.9.

6.3 ABUTMENT AND WINGWALL DESIGN

- Abutment backfill should consist of MaineDOT 703.19 Granular Borrow for Underwater Backfill, MaineDOT BDG Type 4 soil. Recommended soil properties for Type 4 soils are as follows:
  - Internal Friction Angle of Soil = 32°
  - Soil Total Unit Weight = 125 pcf
  - Coefficient of Passive Earth Pressure, K_p (use for design of end diaphragms), K_p= 4.96;
  - Coefficient of Active Earth Pressure, K_a=0.31 (use for design of abutments and wingwalls):

- Live load surcharge should be applied as a uniform lateral surcharge pressure using the equivalent fill height (H_{eq}) values developed in accordance with LRFD Section 3.11.6.4, based on the abutment/wingwall height and distance from the wall backface to the edge of traffic. A minimum H_{eq} of 2 feet is recommended.

Foundation drainage should be provided in accordance with Section 5.4.1.9 of the MaineDOT BDG. We recommend the use of French drains on the uphill side of abutments and wing walls to prevent buildup of differential hydrostatic pressure. The drains should be sloped to drain by gravity and should outlet through a series of 4-inch-diameter weep holes, spaced approximately 10 feet center-to-center.

6.3.1 Spread Footing Design

- Footings designed to bear on intact bedrock, subfooting concrete, or on tremie seals bearing on intact bedrock should be designed using a nominal bearing resistance, q_n, of 57 ksf. At the strength limit state, footings should be designed for a maximum factored bearing resistance of 25.5 ksf. A bearing resistance of 25.5 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 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 Bedrock Elevation Based on Nearest Boring (feet, NAVD 88)**
Abutment 1	El. 102
Abutment 2	El. 103

  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 exposed bedrock surface after cleaning is below the design footing bearing level, subfooting concrete or increased tremie seal thickness may be used up to the proposed footing bearing level.

- Concrete used for cofferdam seals, subfootings and footings should consist of Class A Concrete in accordance with Specification Section 502.05.
- Anchoring, doweling, benching or other means of improving sliding resistance are recommended if the prepared bedrock surface is steeper than 4H:1V in any direction.
- Since the footings or subfootings 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

Excavation will be required to depths of approximately 16 to 18 feet below existing roadway level to expose the bedrock subgrade, which is about 7 to 8 feet below typical brook level and 8 to 10 feet below measured groundwater levels.

Depending on staging, excavations for abutment foundation construction may be achieved using sloped open cut techniques, temporary excavation support or a combination of the two. If sufficient space is available, sloped open cut excavations may provide the most economical method to achieve the proposed excavations at this site. If feasible, sloped open cuts would likely require temporary damming and diversion of the brook for foundation construction to proceed in the dry. It may also be feasible to utilize portions of the existing abutments to assist excavation support and/or dewatering.

Technically feasible temporary excavation support systems for this site include internally-braced, steel sheet pile cofferdams and drilled micropiles and lagging. Steel sheet piling would likely require pre-excavation through as much as 10 feet of the overburden soil due to the presence of cobbles and boulders, most significantly at Abutment 2. Although more expensive, drilled micropiles with wood lagging are an alternative solution to sheet pile cofferdams, but they also may not control water from the brook if diversion is not implemented.

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. However, supplemental pumping from sumps may be required to control water in the excavation. Sumps should be fitted with geotextile or sand filters to prevent loss of subgrade fines during pumping. Pumping may not be feasible if dam-and-divert is not utilized, which would require placement of a tremie seal in the wet for foundation construction. Dewatering discharge should be managed in accordance with the contractor's Stormwater Prevention Plan and MaineDOT Best Management Practices.

7.2 SUBGRADE PREPARATION

If possible, bedrock bearing surface preparation should be completed in the dry. Without suitable groundwater control, a tremie seal on rock may be required. 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 subgrade preparation is completed in the wet, the prepared bearing surfaces should be checked by depth probing in conjunction with visual means such as a diver and/or remotely operated vehicle video inspection. A Special Provision should be prepared to define the project-specific requirements for subgrade preparation and quality assurance/quality control.

7.3 REUSE OF ON-SITE MATERIALS

Based on the test boring results, all fill samples and two of the three glacial stream deposit samples tested had more than 10 percent to 20 percent passing the No. 200 sieve, indicating the material will not meet MaineDOT Specification requirements Granular Borrow for Underwater Backfill and would not be suitable for use as structural backfill, but it could potentially be used as Granular Borrow for embankment construction. The material is also considered suitable for use as Common Borrow. The Marine deposit sample tested had more than 50 percent passing the No. 200 sieve indicating that it will not meet MaineDOT specifications for Granular Borrow, but it may be considered suitable for use as Common Borrow.

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

TABLES

TABLE 1

Summary of Subsurface Explorations

Tannery Brook Bridge #3511

Mariaville, Maine

WIN 026107.00

Exploration Designation	Ground Surface Elev. [1,2]	Northing [2]	Easting [2]	Station[5]			Offset		Top of Stratum Elevation (ft)			Approximate Encountered Thickness of Stratum[4]			Top of Rock		Total Exploration Depth (ft)	Groundwater	
							(feet)	Dir.	Fill	Marine Deposit	Glacial Stream Deposit	Fill	Marine Deposit	Glacial Stream Deposit	Depth (ft)	Elev. [1]		Depth (ft)	Elev. [1]
BB-MTB-101	118.0	316768.5	2156699.2	254	+	20.5	5.8	L	118.0	114.189	109.489	3.8	4.7	7.2	15.7	102.3	26.8	8.6	109.4
BB-MTB-102	117.4	316839.2	2156727.2	254	+	95.6	6.3	R	117.4	NE	NE	>2.5	NE	NE	NE	NE	2.5	NM	NM
BB-MTB-102A	117.3	316844.3	2156728.3	255	+	0.9	6.3	R	117.3	NE	NE	>2.5	NE	NE	NE	NE	2.5	NM	NM
BB-MTB-102B	117.4	316835.1	2156726.2	254	+	91.5	6.3	R	117.4	NE	114.4	3.0	NE	>5.0	NE	NE	8.0	NM	NM
BB-MTB-102C	117.4	316826.6	2156724.1	254	+	82.7	6.0	R	117.4	NE	NE	>0.7	NE	NE	NE	NE	9.0	NM	NM
BB-MTB-102D	117.4	316842.3	2156727.5	254	+	98.8	5.9	R	117.4	--	--	--	--	--	--	--	--	--	--
BB-MTB-102E	117.4	316840.5	2156727.1	254	+	96.9	6.0	R	117.4	NE	108.4	9.0	NE	5.0	14.0	103.4	24.5	8.3	109.1

Notes:

1) Elevations are in feet and reference the North American Vertical Datum of 1988 (NAVD 88). Northing and Easting are in feet and reference the North American Datum of 1983 (NAD83) (2011) Maine 2000 East.

2) As-completed exploration locations and elevations for the borings were surveyed by MaineDOT, and were provided on the boring logs.

3) "NE" indicates stratum not encountered, "NM" indicates Groundwater not measured.

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.

5) Boring stations and offsets from the centerline alignment were measured based on surveyed locations.

6) 100 series borings were conducted using a truck - mounted rig by NEBC. SPT samples were conducted using calibrated automatic hammers.Top of Rock elevations were confirmed with bedrock coring.

7) BB-MTB-102D was abandoned after encountering refusal just below the asphalt (Asphalt thickness not measured in -102D).

8) See boring logs in Appendix B for additional details of all borings.

TABLE 2
Summary of Rock Core and Laboratory Test Results
Tannery Brook Bridge #3511
Mariaville, Maine
WIN 026107.00

Boring ID	Core Run	Ground Surface El. (ft)	Depth of Core Run below GS (ft)			Depth to Rock	Depth Below Top of Rock (ft)				Length of Core Run (in)	Rec (in)	Rec (%)	RQD (in)	RQD %	Joint Spacing (in)	Joint Aperture (in)	Elev. (ft)		LAB									Rock Type
			Top		Bottom		Top		Bottom	Midpoint Depth								Top	Bottom	Depth to Top of Sample (ft)	Depth to Top of Sample into Rock (ft)	Elev Top of Sample (ft)	UCS (psi)	Poissons Ratio	Point Load Index, Is50 (psi)	Correlated UCS from Point Load Tests (psi)	Young's Modulus (ksi)	Unit Wt (pcf)	
BB-MTB-101	R1	118.0	16.5	-	20.0	15.7	0.8	-	4.3	2.6	42	42	100%	18	42%	<0.75-8	<0.004-0.01	101.5	98.0										SLATE
BB-MTB-101	R2	118.0	20.0	-	25.0	15.7	4.3	-	9.3	6.8	60	60	100%	38	63%	<0.75-24	<0.004-0.01	98.0	93.0	20.5	4.8	97.5	5,961	0.31			6,100	171.2	SLATE
BB-MTB-101	R3	118.0	25.0	-	26.8	15.7	9.3	-	11.1	10.2	22	22	100%	8	36%	<0.75-8	0.004-0.01	93.0	91.2										SLATE
BB-MTB-102E	R1	117.4	14.5	-	19.5	14.0	0.5	-	5.5	3.0	60	60	100%	53	88%	24	0.004-0.01	102.9	97.9	16.8	2.8	100.6	6,818	0.16	330 (D) 297 (A)	7,920 (D) 7,128 (A)	1,800	173.8	SLATE
BB-MTB-102E	R2	117.4	19.5	-	24.5	14.0	5.5	-	10.5	8.0	60	60	100%	53	88%	2.5-24	0.004-0.01	97.9	92.9										SLATE

Notes:

1) UCS is Unconfined Compressive Strength. UCS values marked with "D" (Diametrical) and "A" (Axial) are correlated form the point load test results.

2) Refer to the boring logs in Appendix B for additional information.

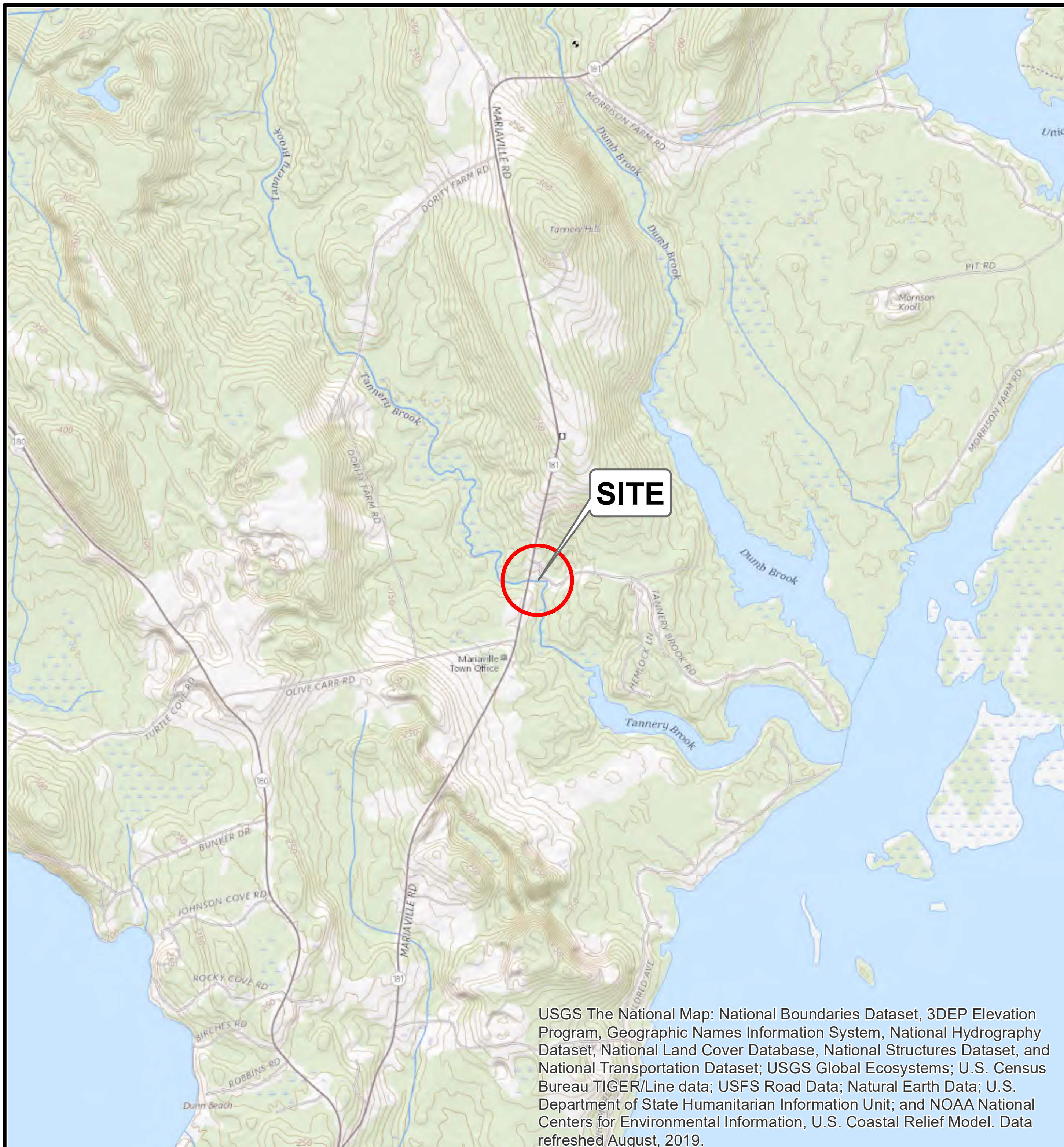
3) Laboratory test results included Young's Modulus values at three different stress ranges. The Young's Modulus values included in the table are the minimum values from the lab results.

4) UCS values marked with "D" (Diametrical) and "A" (Axial) are correlated form the point load test results.

5) Project Elevation datum is North American Vertical Datum (NAVD 88), unless noted otherwise.

6) As-drilled locations and elevations 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 :

USGS science for a changing world

PROJ. MGR.: NVW
DESIGNED BY: EF
REVIEWED BY: ARB
OPERATOR: EF

DATE: 11-30-2022

LOCUS PLAN

TANNERY BROOK BRIDGE #3511
MARIAVILLE, MAINE

JOB NO. 09.0026171.01

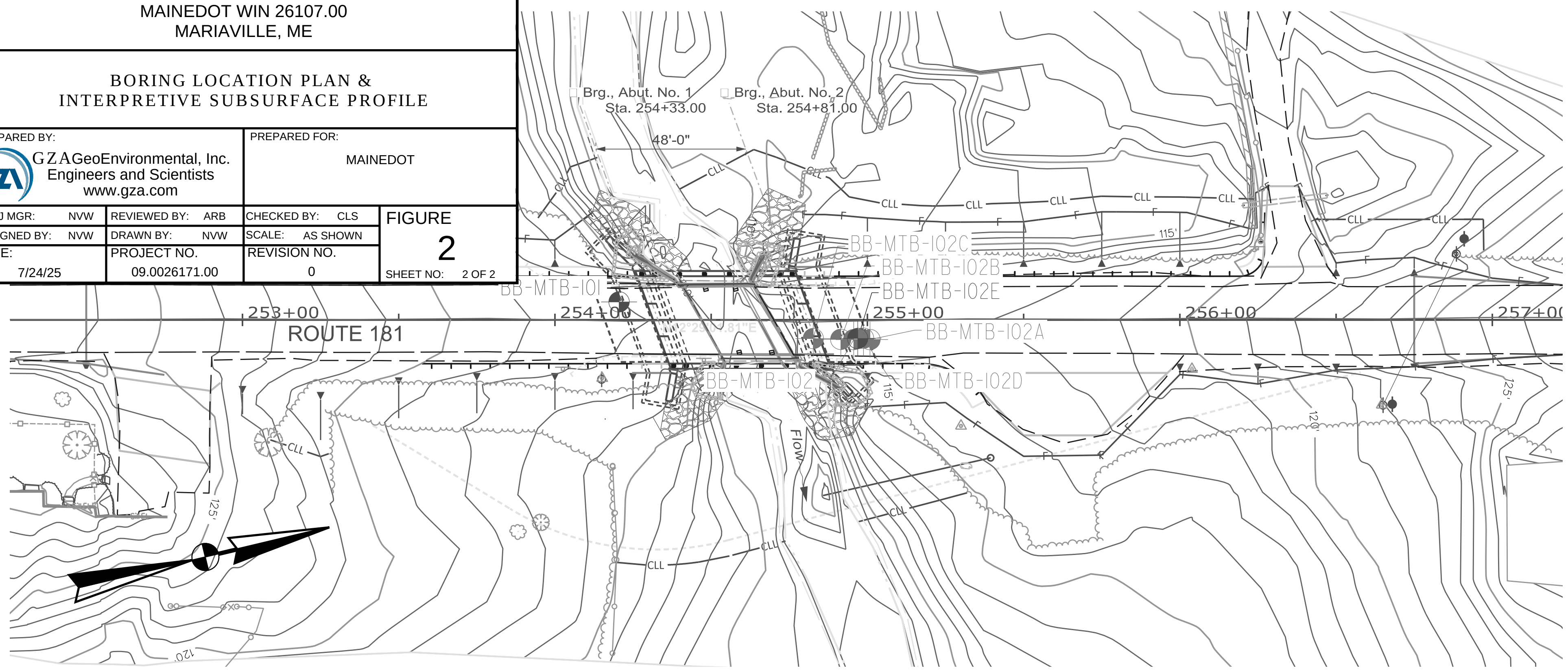
FIGURE NO. 1

TANNERY BROOK BRIDGE #3511
MAINEDOT WIN 26107.00
MARIAVILLE, ME

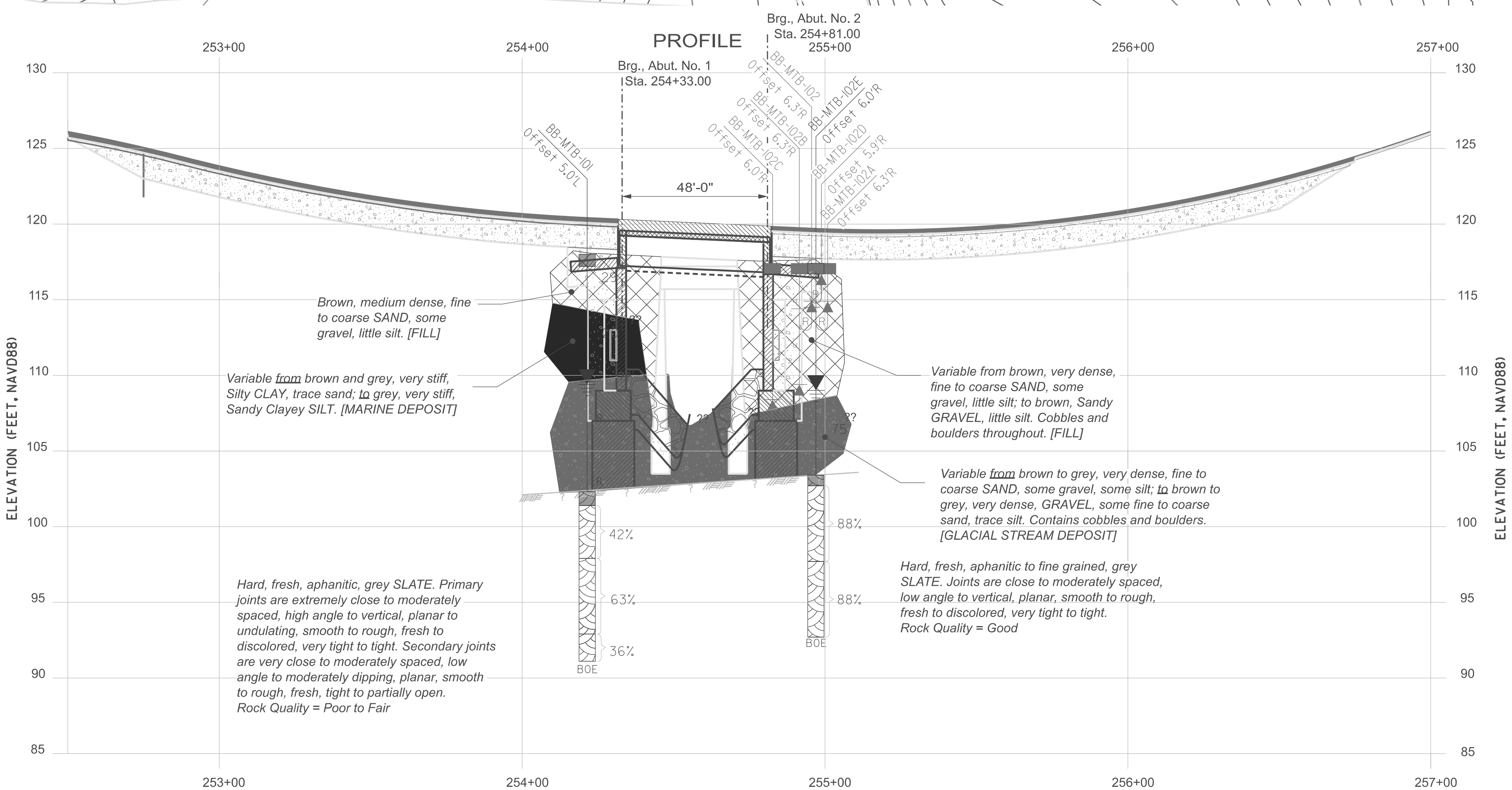
BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

PREPARED BY: GZA GeoEnvironmental, Inc. Engineers and Scientists www.gza.com		PREPARED FOR: MAINEDOT	
PROJ MGR: NVW	REVIEWED BY: ARB	CHECKED BY: CLS	FIGURE 2
DESIGNED BY: NVW	DRAWN BY: NVW	SCALE: AS SHOWN	
DATE: 7/24/25	PROJECT NO. 09.0026171.00	REVISION NO. 0	SHEET NO: 2 OF 2

PLAN



PROFILE



PRELIMINARY
NOT FOR CONSTRUCTION

NOTES

1) Base map developed from electronic files provided by Stantec on March 18, 2025 (Alignment_route181.dgn, Bridge.dgn, Ground.dgn, Topo.dgn, and Profile_route181.dgn),

2) The as-drilled locations of the test borings were surveyed and provided by MaineDOT in an electronic file (Borings.dgn) on May 15, 2023.

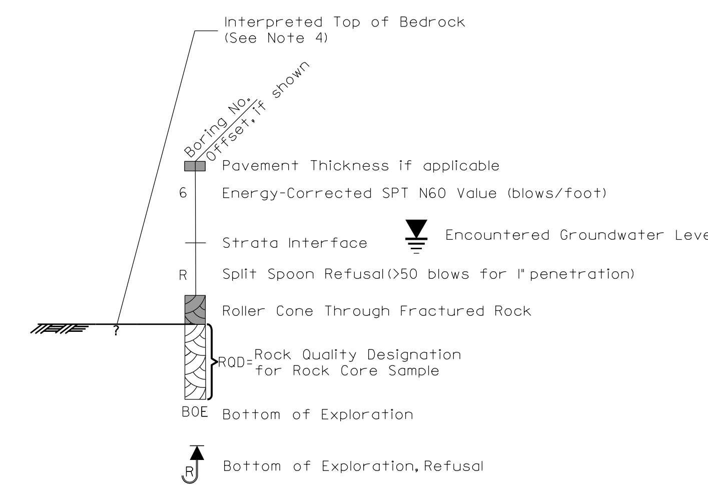
3) BB-MTB-100 series borings were performed by New England boring Contractors and observed by GZA personnel between October 25 and October 26, 2022.

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

BORING LOCATION PLAN LEGEND

BB-MTB-102 Location and designation of borings

INTERPRETIVE SUBSURFACE PROFILE LEGEND



PLAN

25 0 25 50
SCALE IN FEET

PROFILE

HORIZ 25 0 25 50
VERT 5 0 5 10
SCALE IN FEET

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
2610700
WIN 26107.00
BRIDGE PLANS
BRIDGE NO. 3511

	BY	DATE
PROJ. MANAGER D. SMALL		
DESIGN-DETAILED	NVW	MAR 2025
CHECKED-REVIEWED	ARB	MAR 2025
DESIGN2-DETAILED2	CLS	
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

MARIAVILLE
TANNERY BRIDGE
BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
5
OF 29

Date: $date$
Username: $user$

Filename: P:\09 Jobs\0026100s\09.0026171.00 - Stantec-MEDOT, Tannery Bridge, Mariaville\01 - Final Design\Figures\Microstation\BLP_ISP_OpenRds_Grayscale.dgn

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):

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

*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))

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

Sample Container Labeling Requirements:

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

Modified ISRM Rock Classification (GZA)

Rock cores are visually classified by the Modified ISRM System using the following format and order: Field hardness, weathering, grain size, color, ROCK TYPE, joint description (spacing, dip angle, type, shape and roughness, weathering, aperture, infilling, condition of joint surfaces, other features such as minerals.

FIELD HARDNESS:
Very Hard – Cannot be scratched with knife or sharp pick. Breaking of hand specimens requires several hard blows of geologists pick.
Hard – Can be scratched with knife or pick only with difficulty. Hard blow of hammer required to detach hand specimen.
Medium – Can be grooved or gouged 1/16 in. deep by firm pressure on knife or pick point. Can be excavated in small chips to pieces about 1 in. maximum size by hard blows from the point of a geologist's pick.
Soft – Can be gouged or grooved readily with knife or pick point. Can be excavated in chips to pieces several inches in size by moderate blows of a pick point. Small thin pieces can be broken by finger pressure.
Very Soft – Can be carved with knife. Can be excavated readily with point of pick. Pieces 1 in. or more in thickness can be broken with finger pressure. Can be scratched readily by fingernail.

WEATHERING:
Fresh – Rock fresh, crystals bright, few joints may show slight staining. Rock rings under hammer if crystalline.
Slightly Weathered – Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering and may be somewhat weaker externally than in its fresh condition. In granitoid rocks some occasional feldspar crystals are dull and discolored. Crystalline rocks ring under hammer.
Moderately Weathered – Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a continuous framework or as corestones. In granitoid rock, most feldspars are dull and discolored; some show clayey. Rock has dull sound under hammer and shows significant loss of strength as compared with fresh rock.
Highly Weathered – More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones. In granitoid rocks, all feldspars kaolinized to some extent. Some fragments of strong rock usually left.
Completely Weathered – All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact. Quartz may be present as dikes or Stringers.
Residual Soil – All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

GRAIN SIZE:
Fine Grained – Barely seen with naked eye.
Coarse Grained: 1/8 in. to 1/4 in.
Aphanitic: Too small to be seen with naked eye.
Medium Grained: Barely seen with naked eye to 1/8 in.
Very Coarse Grained: >1/4 in.

COLOR and ROCK TYPE

JOINT DESCRIPTION:
Spacing and Dip Angle:

Joints	Spacing	Dip	Angle
Extremely Close	Less than ¾ in.	Horizontal	0° - 5°
Very Close	¾ in. – 2 ½ in.	Low Angle	5° - 35°
Close	2 ½ in. - 8 in.	Moderately dipping	35° - 55°
Moderate	8 in. – 24 in.	High Angle	55° - 85°
Wide	24 in. – 80 in.	Vertical	85° - 90°
Very Wide	80 in. – 20 ft.		
Extremely Wide	Greater than 20 ft.		

Type of Discontinuities:
Joint – A break of geologic origin in the continuity of a body of rock along which there has been no visible displacement. May form sets (parallel joints).
Shear – A zone of fractures along which differential movement has taken place parallel to the surface sufficient to produce slickensides, striations, or polishing. May be accompanied by a zone of fractured rock up to a few inches wide.
Fault – Major discontinuity along which there has been appreciable displacement and accompanied by gouge and/or severely fractured adjacent zone of rock.
Shear or Fault Zone – A band or zone of parallel, closely spaced discontinuities along which differential movement has occurred, accompanied by gouge, maylonite, and breccia.
Bedding – A surface parallel to the surface of the deposition
Foliation – A parallel orientation of platy minerals, or mineral banding in metamorphic rocks.

Shape and Roughness:

Shape	Roughness
Stepped	Rough
Undulating	Smooth
Planar	Slickensided

Weathering of Joints:
Fresh – No visible sign of weathering of the rock material
Discolored – The color of the original fresh rock material is changed. The degree of change from the original color should be indicated. If the color change is confined to particular mineral constituents this should be documented.
Decomposed – The rock is weathered to the condition of soil in which the original material fabric is still intact, but some or all of the mineral grains are decomposed
Disintegrated – The rock is weathered to the condition of soil in which the original fabric is still intact. The rock is friable, but the mineral grains are not decomposed.

Aperture:

Tight – Core pieces on either side of a discontinuity can be fitted together by hand so that no visible void spaces remain.
Open – Core pieces on either side of a discontinuity cannot be fitted tightly together and voids are remain.

		Opening
Very Tight		<0.004 in.
Tight	"Closed features"	0.004-0.01 in.
Partially Open		0.01-0.02 in.
Open		0.02 – 0.1 in.
Moderately Wide	"Gapped features"	0.1 – 0.4 in.
Wide		>0.4 in.
Very Wide		0.4 – 4.0 in.
Extremely Wide	"Open features"	4.0 – 40.0 in.
Cavernous		>40 in.

Infilling:
Silt, Sand, Clay, Calcite

Miscellaneous Features:
Pit – Barely seen with the naked eye, to ¼ inch in diameter
Vug – ¼ inch to 2 inches in maximum diameter
Cavity – 2 inches to 2 feet in maximum diameter
Cave – larger than 2 feet in maximum diameter

ROCK OUTCROP CHARACTERIZATION
Also include the following parameters when describing rock outcrops and rock masses:
Persistence:

	Dimensions
Very low persistence	<3.3 ft
Low persistence	3.3 – 9.8 ft
Medium persistence	9.8 -32.8 ft
High persistence	32.8 -65.6 ft
Very high persistence	>65.6 ft

Number of Sets (occurring locally):

I	Massive, occasional random joints
II	One joint set
III	One joint set plus random
IV	Two joint sets
V	Two joint sets plus random
VI	Three joint sets
VII	Three joint sets plus random
VIII	Four or more joint sets
IX	Crushed rock, earth-like

GZA reports the total core recovery and rock quality designation for each core run* on the boring logs. The definitions of these terms are as follows:

TOTAL CORE RECOVERY (REC)

$$\textbf{REC (\%)} = \frac{\text{Sum of Recovered Core}}{\text{Length of Core Run}} \times 100$$

ROCK QUALITY DESIGNATION (RQD)

$$\textbf{RQD (\%)} = \frac{\text{Sum of Lengths of intact Core with Full Diameter in Pieces 4 in. and Longer}}{\text{Length of Core Run}} \times 100$$

The RQD is in general accordance with methodology described by Deere and Deere (1988). In addition, significant vertical to sub-vertical foliation/cross-foliation joints/fractures occur within the rock mass and influence ground behavior. The length of core exhibiting the vertical to sub-vertical joints/fractures has been deducted from the RQD, which is consistent with the "pieces of intact rock core" criteria. The vertical to sub-vertical joints/fractures have been identified on the rock core or the upside divider in the core box with permanent "dots" spaced every 0.1 feet apart. These dots have been counted and entered in the fractures per foot column on the boring log.

* - RQD not reported for severely and/or completely weathered rock or core runs with length of 2.0 feet or less.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Mariaville Road over Tannery Brook Bridge #3511
Location:	Mariaville, Maine
Boring No.:	BB-MTB-101
WIN:	26107.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	118.0	**Auger ID/OD:**	6.0"
Operator:	M. D'Ambrosio	**Datum:**	NAVD88	**Sampler:**	Standard
Logged By:	L. Navarrete	**Rig Type:**	G-Tech GT8 Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/25/22-10/25/22	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ
Boring Location:	254+20.5, 5.8' LT	**Casing ID/OD:**	4.25/4.5", 3.25/3.5"	**Water Level*:**	8.6
Hammer Efficiency Factor:	0.87	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/8	0.0 - 2.0	12-11-9-6	20	29	SSA			0'-0.8': Asphalt	
							50	117.1		0.9 Bottom 5": Brown, moist, medium dense, fine to coarse SAND, some gravel, little silt, (Fill).	22-S-4335 A-1-b, SM WC=6.3%
							69				
							60	114.2		3.8	
							63				22-L-4336 CL, WC=22% LL=33, PL=18
5	2D	24/24	5.0 - 7.0	5-6-8-16	14	20	75			Top 12": Brown and grey, moist, very stiff, Silty CLAY, trace sand, (Marine Deposit). Bottom 12": Grey, wet, very stiff, Sandy Clayey SILT, (Marine Deposit).	22-S-4337 A-4(0), ML WC=16.7%
							120				
							RC				
								109.5		8.5	
10	3D	7/4	10.0 - 10.6	29-56/1"	R					Brown, wet, very dense, fine to coarse SAND, some gravel, some silt, (Glacial Stream Deposit). Note: bottom 2" of spoon contained gravel-size pieces.	
15	4D	8/8	15.0 - 15.7	28-50/2"	R			102.3		Grey, wet, very dense, GRAVEL, some fine to coarse sand, trace silt, (Glacial Stream Deposit). 15.7	22-S-4338 A-1-a, GW-GM WC=5.4%
	R1	42/42	16.5 - 20.0	RQD = 42%						Advanced roller bit to 16.5'. Roller cone penetration resistance increase throughout. Dark grey rock in wash return. Set up to core at 16.5'. R1: Hard, fresh, aphanitic, grey, SLATE, with calcite stringers. Primary joints are extremely close to close, high angle to vertical, planar to undulating, smooth to rough, fresh to discolored, very tight to tight. Secondary joints are close, low angle to moderately dipping, planar, smooth to rough, fresh to discolored, tight to partially open. Recovery = 100% Rock Quality = Poor Rock Core Times (min:sec): 16.5-17.5' (5:02), 17.5-18.5' (3:45), 18.5-19.5' (3:50), 19.5-20.0' (2:33)	
20	R2	60/60	20.0 - 25.0	RQD = 63%						R2: Hard, fresh, aphanitic, grey, SLATE. Primary joints are extremely close to moderately spaced, high angle to vertical, planar to undulating, smooth to rough, fresh, very tight to tight. Secondary joints are very close to moderately spaced, low angle to moderately dipping,	22-S-4407 q_p=858.4 ksf
25											

Remarks:

1. Coordinates N316768.5, E2156699.2. Datum NAD83 (2011) Maine 2000 East. As-drilled boring locations were surveyed by MaineDOT.
2. 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.
3. Automatic hammer NEB #4712-A-GT8-P energy transfer ratio = 0.87.
4. Water level measured immediately after removal of casing.

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

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Mariaville Road over Tannery Brook Bridge #3511
Location: Mariaville, Maine

Boring No.: BB-MTB-101

WIN: 26107.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	118.0	**Auger ID/OD:**	6.0"
Operator:	M. D'Ambrosio	**Datum:**	NAVD88	**Sampler:**	Standard
Logged By:	L. Navarrete	**Rig Type:**	G-Tech GT8 Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/25/22-10/25/22	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ
Boring Location:	254+20.5, 5.8' LT	**Casing ID/OD:**	4.25/4.5", 3.25/3.5"	**Water Level*:**	8.6
Hammer Efficiency Factor:	0.87	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	R3	22/22	25.0 - 26.8	RQD = 36%						planar, rough to smooth, fresh, tight to partially open. Recovery = 100% Rock Quality = Fair Rock Core Times (min:sec): 20.0-21.0' (3:22), 21.0-22.0' (4:22), 22.0-23.0' (3:43), 23.0-24.0' (4:02), 24.0-25.0' (3:58)	
								91.2		R3: Hard, fresh, aphanitic, grey, SLATE, with calcite stringers. Primary joints are extremely close to close, high angle to vertical, planar, smooth to rough, fresh to discolored, tight. Secondary joints are close, low angle, planar, rough, fresh to discolored, partially open. Recovery = 100% Rock Quality = Poor Rock Core Times (min:sec): 25.0-26.0' (3:00), 26.0-26.8' (2:24)	
										26.8	
										Bottom of Exploration at 26.8 feet below ground surface.	
30											
35											
40											
45											
50											

Remarks:

1. Coordinates N316768.5, E2156699.2. Datum NAD83 (2011) Maine 2000 East. As-drilled boring locations were surveyed by MaineDOT.
2. 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.
3. Automatic hammer NEB #4712-A-GT8-P energy transfer ratio = 0.87.
4. Water level measured immediately after removal of casing.

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Mariaville Road over Tannery Brook Bridge #3511
Location: Mariaville, Maine

Boring No.: BB-MTB-102
WIN: 26107.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	117.4	**Auger ID/OD:**	6.0"
Operator:	M. D'Ambrosio	**Datum:**	NAVD88	**Sampler:**	Standard
Logged By:	L. Navarrete	**Rig Type:**	G-Tech GT8 Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/25/22-10/25/22	**Drilling Method:**	Drive & Wash	**Core Barrel:**	--
Boring Location:	254+95.6, 6.3' RT	**Casing ID/OD:**	--	**Water Level*:**	--
Hammer Efficiency Factor:	0.87	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	6/6	0.5 - 1.0	17/6"	--		119	116.9		0'-0.5': Asphalt. 0.5	22-S-4339 A-1-b, SM WC=3.9%
							120			Brown, dry, fine to coarse SAND, some gravel, little silt, hydrocarbon odor, (Fill). Refusal on probable cobble.	
								114.9		2.5 **Bottom of Exploration at 2.5 feet below ground surface.**	
5											
10											
15											
20											
25											

Remarks:

1. Coordinates N316839.2, E2156727.2. Datum NAD83 (2011) Maine 2000 East. As-drilled boring locations were surveyed by MaineDOT.
2. Automatic hammer NEB #4712-A-GT8-P energy transfer ratio = 0.87.

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

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Mariaville Road over Tannery Brook Bridge #3511
Location: Mariaville, Maine

Boring No.: BB-MTB-102A
WIN: 26107.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	117.3	**Auger ID/OD:**	6.0"
Operator:	M. D'Ambrosio	**Datum:**	NAVD88	**Sampler:**	Standard
Logged By:	L. Navarrete	**Rig Type:**	G-Tech GT8 Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/25/22-10/25/22	**Drilling Method:**	Drive & Wash	**Core Barrel:**	--
Boring Location:	255+00.9, 6.3' RT	**Casing ID/OD:**	--	**Water Level*:**	--
Hammer Efficiency Factor:	0.87	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D						SSA	116.6		0'-0.7': Asphalt. 0.7	
								114.8		No sample collected. Soil visible is brown, dry, fine to coarse SAND, some gravel, little silt, (Fill). Auger refusal on cobble. Cobbles visible 4"-7" size in bore hole. 2.5	
										Bottom of Exploration at 2.5 feet below ground surface.	
5											
10											
15											
20											
25											

Remarks:

1. Coordinates N316844.3, E2156728.3. Datum NAD83 (2011) Maine 2000 East. As-drilled boring locations were surveyed by MaineDOT.
2. Automatic hammer NEB #4712-A-GT8-P energy transfer ratio = 0.87.

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

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Mariaville Road over Tannery Brook Bridge #3511
Location: Mariaville, Maine

Boring No.: BB-MTB-102B
WIN: 26107.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	117.4	**Auger ID/OD:**	6.0"
Operator:	M. D'Ambrosio	**Datum:**	NAVD88	**Sampler:**	Standard
Logged By:	L. Navarrete	**Rig Type:**	G-Tech GT8 Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/25/22-10/25/22	**Drilling Method:**	Drive & Wash	**Core Barrel:**	--
Boring Location:	254+91.5, 6.3' RT	**Casing ID/OD:**	4.25/4.5"	**Water Level*:**	--
Hammer Efficiency Factor:	0.87	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	116.7		0'-0.7': Asphalt — 0.7	
								114.4		(Fill), based on BB-MTB-102. — 3.0	
5	1D	18/11	5.0 - 6.5	7-20-50/6"	--					Brown, moist, very dense, fine to coarse SAND, some gravel, little silt, (Glacial Stream Deposit). Rock piece at spoon tip.	22-S-4340 A-1-b, SM WC=8.7%
							180			Attempted to drive casing. Casing would not remain straight after several attempts to correct orientation.	
							223	109.4		Abandoned borehole. — 8.0	
										Bottom of Exploration at 8.0 feet below ground surface.	
10											
15											
20											
25											

Remarks:

1. Coordinates N316835.1, E2156726.2. Datum NAD83 (2011) Maine 2000 East. As-drilled boring locations were surveyed by MaineDOT.
2. Automatic hammer NEB #4712-A-GT8-P energy transfer ratio = 0.87.

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

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Mariaville Road over Tannery Brook Bridge #3511
Location: Mariaville, Maine

Boring No.: BB-MTB-102C
WIN: 26107.00

Driller: New England Boring Contractors		**Elevation (ft.)** 117.4		**Auger ID/OD:** 6.0"	
Operator: M. D'Ambrosio		**Datum:** NAVD88		**Sampler:** Standard	
Logged By: L. Navarrete		**Rig Type:** G-Tech GT8 Truck		**Hammer Wt./Fall:** 140#/30"	
Date Start/Finish: 10/25/22-10/25/22		**Drilling Method:** Drive & Wash		**Core Barrel:** --	
Boring Location: 254+82.7, 6.0' RT		**Casing ID/OD:** 4.25/4.5"		**Water Level*:** --	
Hammer Efficiency Factor: 0.87		**Hammer Type:** Automatic ☒ Hydraulic ☐ Rope & Cathead ☐			

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	116.7		0'-0.7': Asphalt 0.7	
							45			No samples taken. Casing removed; auger spoils consisted of: Brown, dry, GRAVEL and fine to coarse sand, little silt, (Fill). Straight casing orientation could not be maintained. At 9.0' casing broke off threads.	
							50				
							70				
							34				
5							21				
							42				
							131				
								108.4		9.0 **Bottom of Exploration at 9.0 feet below ground surface.**	
10											
15											
20											
25											

Remarks:

1. Coordinates N316826.6, E2156724.1. Datum NAD83 (2011) Maine 2000 East. As-drilled boring locations were surveyed by MaineDOT.
2. Automatic hammer NEB #4712-A-GT8-P energy transfer ratio = 0.87.
3. Offset 3.5' north of BB-MTB-102 for BB-MTB-102D. Encountered refusal just below the asphalt, so the boring was abandoned and relocated to BB-MTB-102E.

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

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Mariaville Road over Tannery Brook Bridge #3511
Location: Mariaville, Maine

Boring No.: BB-MTB-102E

WIN: 26107.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	117.4	**Auger ID/OD:**	6.0"
Operator:	M. D'Ambrosio	**Datum:**	NAVD88	**Sampler:**	Standard
Logged By:	L. Navarrete	**Rig Type:**	G-Tech GT8 Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/26/22-10/26/22	**Drilling Method:**	Spin & Wash	**Core Barrel:**	NQ
Boring Location:	254+96.9, 6.0' RT	**Casing ID/OD:**	4.25/4.5", 3.25/3.5"	**Water Level*:**	8.3
Hammer Efficiency Factor:	0.87	**Hammer Type:**	Automatic ☒	Hydraulic ☐	Rope & Cathead ☐

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SPIN			0'-0.7': Asphalt	
								116.7		0.7	
										Casing advancement difficult; probable cobbles.	
5											
										Casing jammed at 9.0'. Roller cone cuttings consist of dark grey rock, wood, and silt (Fill).	
								108.4		9.0	
							RC				
10	1D	19/11	10.0 - 11.6	14-19-33-40/1"	52	75	SPIN			Grey, wet, very dense, Gravelly fine to coarse SAND, little silt, (Glacial Stream Deposit).	22-S-4341 A-1-b, SM WC=12.3%
							RC				
								103.4		14.0	
	R1	60/60	14.5 - 19.5	RQD = 88%			NQ			Casing spin refusal at 14.0'. Placed 3" casing and roller bit. Advanced roller bit to 14.5'. Dark grey rock in wash return. Set up to core at 14.5'. R1: Hard, fresh, aphanitic to fine grained, grey, SLATE. Joints are moderately spaced, high angle to vertical, planar, smooth, fresh, tight. Recovery = 100% Rock Quality = Good Rock Core Times (min:sec): 14.5-15.5' (3:10), 15.5-16.5' (3:25), 16.5-17.5' (4:15), 17.5-18.5' (4:29), 18.5-19.5' (5:17)	22-S-4408 q_p=982 ksf 22-S-4409 PLD=47.5 ksf PLA=42.8 ksf
15											
	R2	60/60	19.5 - 24.5	RQD = 88%						R2: Hard, fresh, aphanitic to fine grained, grey, SLATE. Primary joints are close to moderately spaced, low angle, planar, rough, fresh to discolored, tight, with calcite stringers and silt infilling. One joint is vertical, planar, smooth, fresh, very tight. Recovery = 100% Rock Quality = Good Rock Core Times (min:sec): 19.5-20.5' (2:31), 20.5-21.5' (3:15), 21.5-22.5' (3:43), 22.5-23.5' (2:38), 23.5-24.5' (3:41)	
20											
								92.9		24.5	
25											

Remarks:

1. Coordinates N316840.5, E2156727.1. Datum NAD83 (2011) Maine 2000 East. As-drilled boring locations were surveyed by MaineDOT.
2. Automatic hammer NEB #4712-A-GT8-P energy transfer ratio = 0.87.
3. 6" solid stem finless auger utilized to drill through pavement.
5. Offset 1.5' north of BB-MTB-102.

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

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Mariaville Road over Tannery Brook Bridge #3511 **Location:** Mariaville, Maine	**Boring No.:** BB-MTB-102E **WIN:** 26107.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	117.4	**Auger ID/OD:**	6.0"
Operator:	M. D'Ambrosio	**Datum:**	NAVD88	**Sampler:**	Standard
Logged By:	L. Navarrete	**Rig Type:**	G-Tech GT8 Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/26/22-10/26/22	**Drilling Method:**	Spin & Wash	**Core Barrel:**	NQ
Boring Location:	254+96.9, 6.0' RT	**Casing ID/OD:**	4.25/4.5", 3.25/3.5"	**Water Level*:**	8.3
Hammer Efficiency Factor:	0.87	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										**Bottom of Exploration at 24.5 feet below ground surface.**	
30											
35											
40											
45											
50											

Remarks:

1. Coordinates N316840.5, E2156727.1. Datum NAD83 (2011) Maine 2000 East. As-drilled boring locations were surveyed by MaineDOT.
2. Automatic hammer NEB #4712-A-GT8-P energy transfer ratio = 0.87.
3. 6" solid stem finless auger utilized to drill through pavement.
5. Offset 1.5' north of BB-MTB-102.

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

APPENDIX C – LABORATORY TEST RESULTS

Thielsch DIVISION OF THE RISE GROUP	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 South Portland, ME PM: Michael Johnescu Assigned By: Michael Johnescu Collected By: LN	Project Information: **Tannery Brook Bridge** **Mariaville, Maine** Project Number: 09.0026171.00 Summary Page: 1 of 1 Report Date: 11.11.22

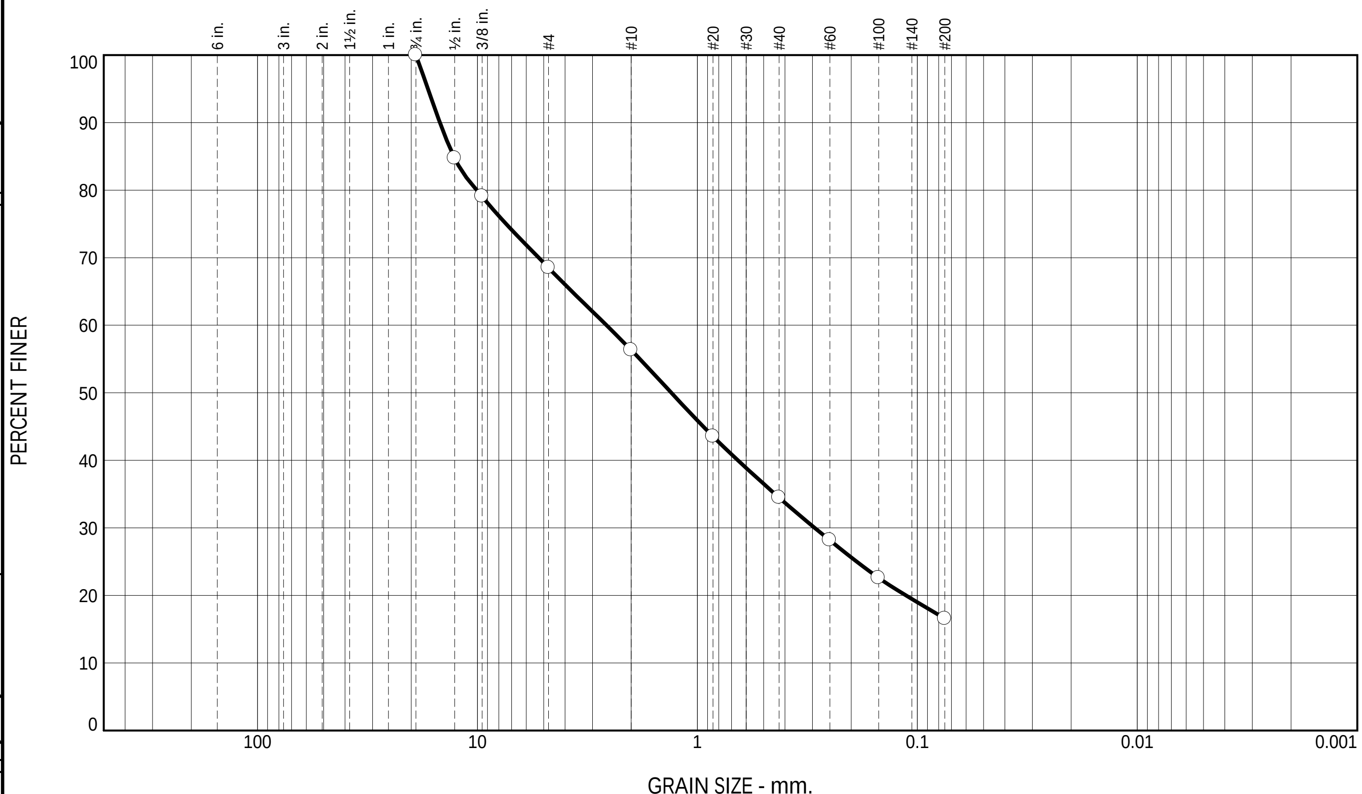
LABORATORY TESTING DATA SHEET, Report No.: 7422-L-109

Boring	Sample No.	Depth (ft)	Laboratory No.	Identification Tests								Proctor / CBR / Permeability Tests								Laboratory Log and Soil Description
				As Rcvd 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						
101	1D	0.5-2.5	22-S-4335	6.3			31.5	51.9	16.6											Brown f-c SAND, some fine Gravel, little Silt
101	2D	5-7 (Top 12")	22-S-4336	22.0	33	18														Brown CLAY & SILT
101	2D	5-7 (Btm 12")	22-S-4337	16.7			0.0	43.9	56.1											Brown CLAYEY SILT and fine SAND
101	4D	15-15.7	22-S-4338	5.4			66.7	26.6	6.7											Gray f-c GRAVEL, some f-c Sand, trace Silt
102	1D	0.5-2.5	22-S-4339	3.9			31.8	54.9	13.3											Dark Brown f-c SAND, some f-c Gravel, little Silt
102B	1D	5-6.5	22-S-4340	8.7			26.4	61.0	12.6											Dark Brown f-c SAND, some fine Gravel, little Silt
102E	1D	10-11.6	22-S-4341	12.3			35.9	48.1	16.0											Brown f-c SAND and f-c GRAVEL, little Silt

Date Received: *11.02.22*　　Reviewed By: ______　　Date Reviewed: *11.11.22*

Particle Size Distribution Report

ASTM D6913



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	31.5	12.2	21.8	17.9	16.6	

Test Results (ASTM D6913)

Sieve Size or Diam. (mm.)	Finer (%)	Spec.* (%)	Out of Spec. (%)	Pct. of Fines
3/4"	100.0			
1/2"	84.7			
3/8"	79.1			
#4	68.5			
#10	56.3			
#20	43.5			
#40	34.5			
#60	28.2			
#100	22.6			
#200	16.6			

* (no specification provided)

Material Description

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

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 14.8163 D_{85}= 12.8194 D_{60}= 2.5914
D_{50}= 1.3136 D_{30}= 0.2930 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-1-b

Test Remarks

Source of Sample: SPT Depth: 0.5-2.5'
Sample Number: 101 / 1D

Sample Date: 11.08.22

Thielsch Engineering Inc.

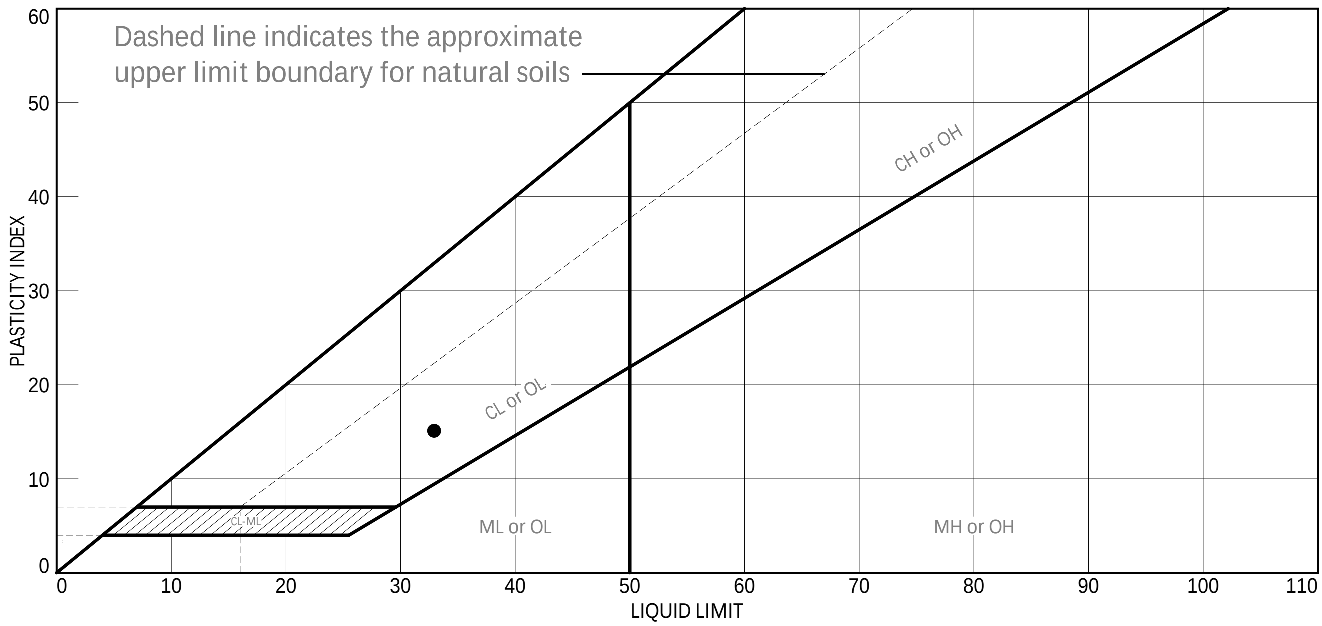
Cranston, RI

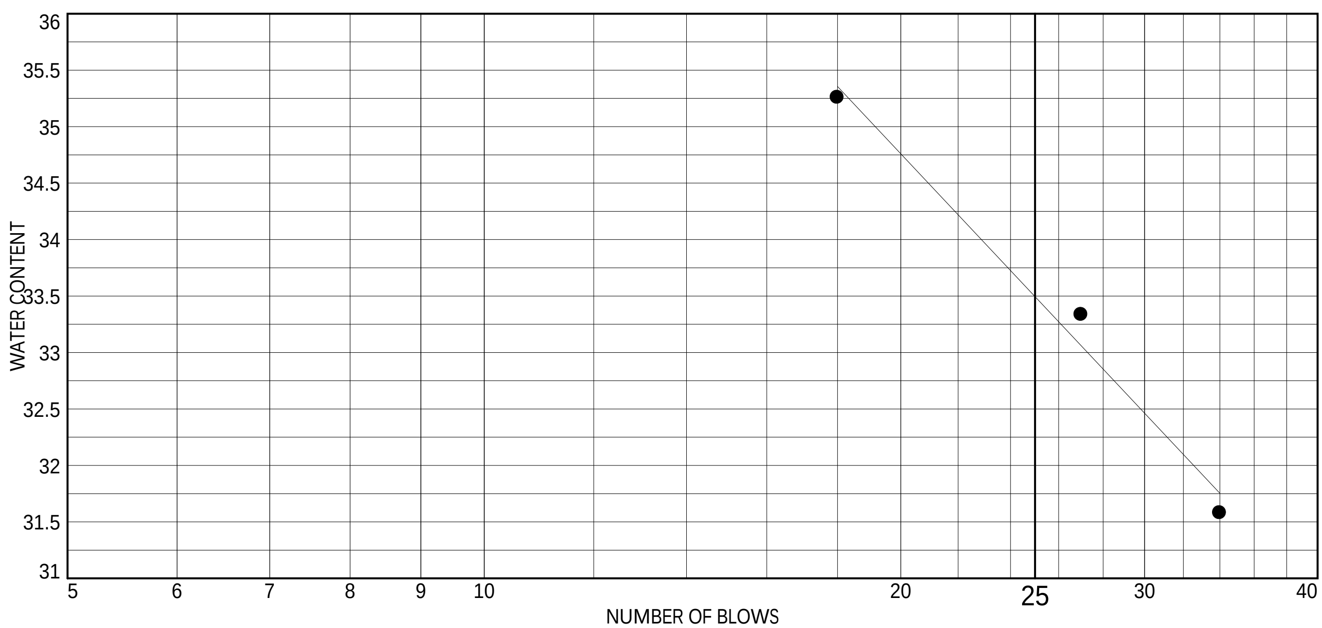
Client: GZA GeoEnvironmental
Project: Tannery Brook Bridge
Mariaville, ME
Project No: 09.0026171.00

Figure 22-S-4335

Tested By: RB Checked By:

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Brown CLAY & SILT	33	18	15			

Project No. 09.0026171.00 Client: GZA GeoEnvironmental

Project: Tannery Brook Bridge

Mariaville, ME

Source of Sample: SPT Depth: 5-7 (Top 12")

Sample Number: 101 / 2D

Thielsch Engineering Inc.

Cranston, RI

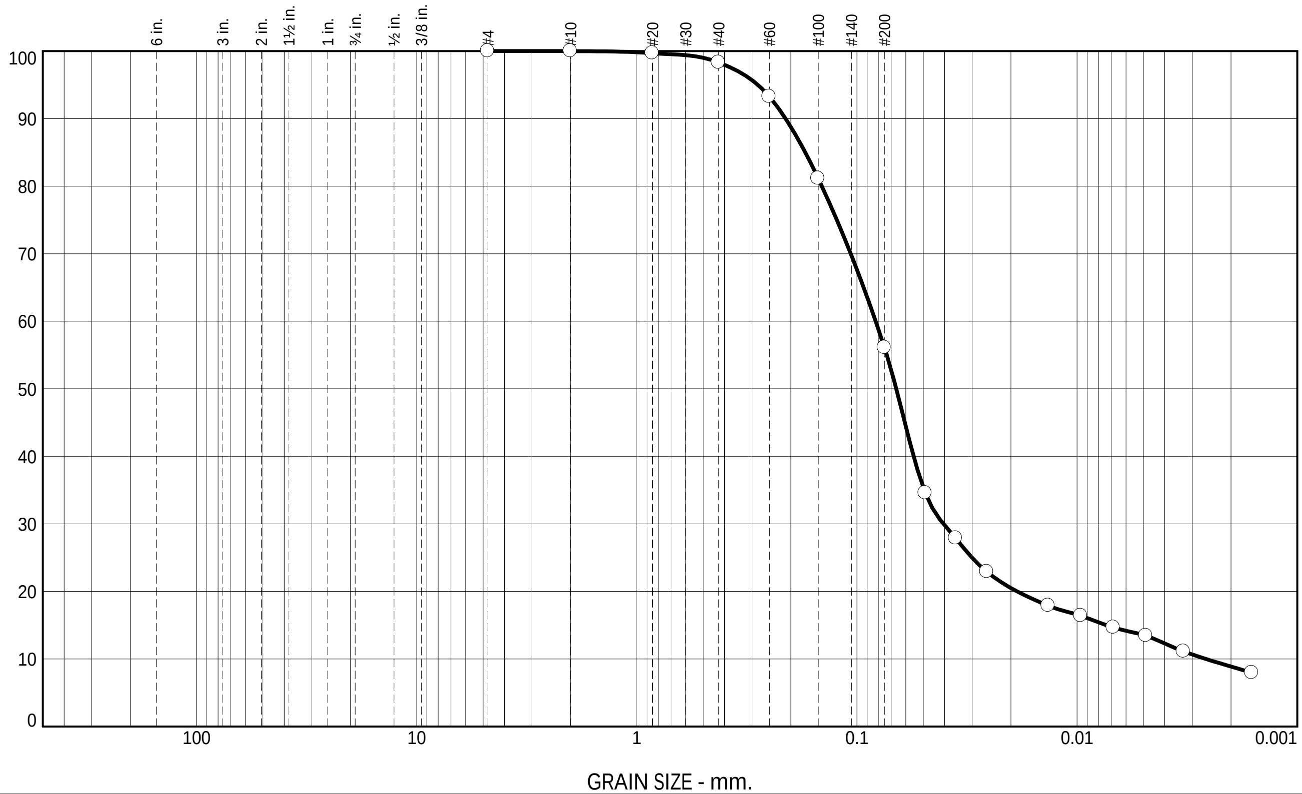
Remarks:

Figure 22-L-4336

Tested By: SL Checked By:

Particle Size Distribution Report

ASTM D6913 and ASTM D7928



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	1.7	42.2	47.2	8.9

Test Results (ASTM D6913 and ASTM D7928)

Sieve Size or Diam. (mm.)	Finer (%)	Spec.* (%)	Out of Spec. (%)	Pct. of Fines
#4	100.0			
#10	100.0			
#20	99.7			
#40	98.3			
#60	93.3			
#100	81.1			
#200	56.1			
0.0489 mm.	34.6			
0.0356 mm.	27.9			
0.0257 mm.	22.9			
0.0135 mm.	17.9			
0.0096 mm.	16.4			
0.0068 mm.	14.7			
0.0049 mm.	13.4			
0.0033 mm.	11.1			
0.0016 mm.	7.9			

* (no specification provided)

Material Description

Brown CLAYEY SILT and fine SAND

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 0.2110 D_{85}= 0.1718 D_{60}= 0.0823
D_{50}= 0.0664 D_{30}= 0.0405 D_{15}= 0.0074
D_{10}= 0.0026 C_u= 31.66 C_c= 7.66

Classification

USCS= ML AASHTO= A-4(0)

Test Remarks

Source of Sample: SPT Depth: 5-7 (Btm 12")
Sample Number: 101 / 2D Sample Date: 11.9.22

Thielsch Engineering Inc.
Cranston, RI

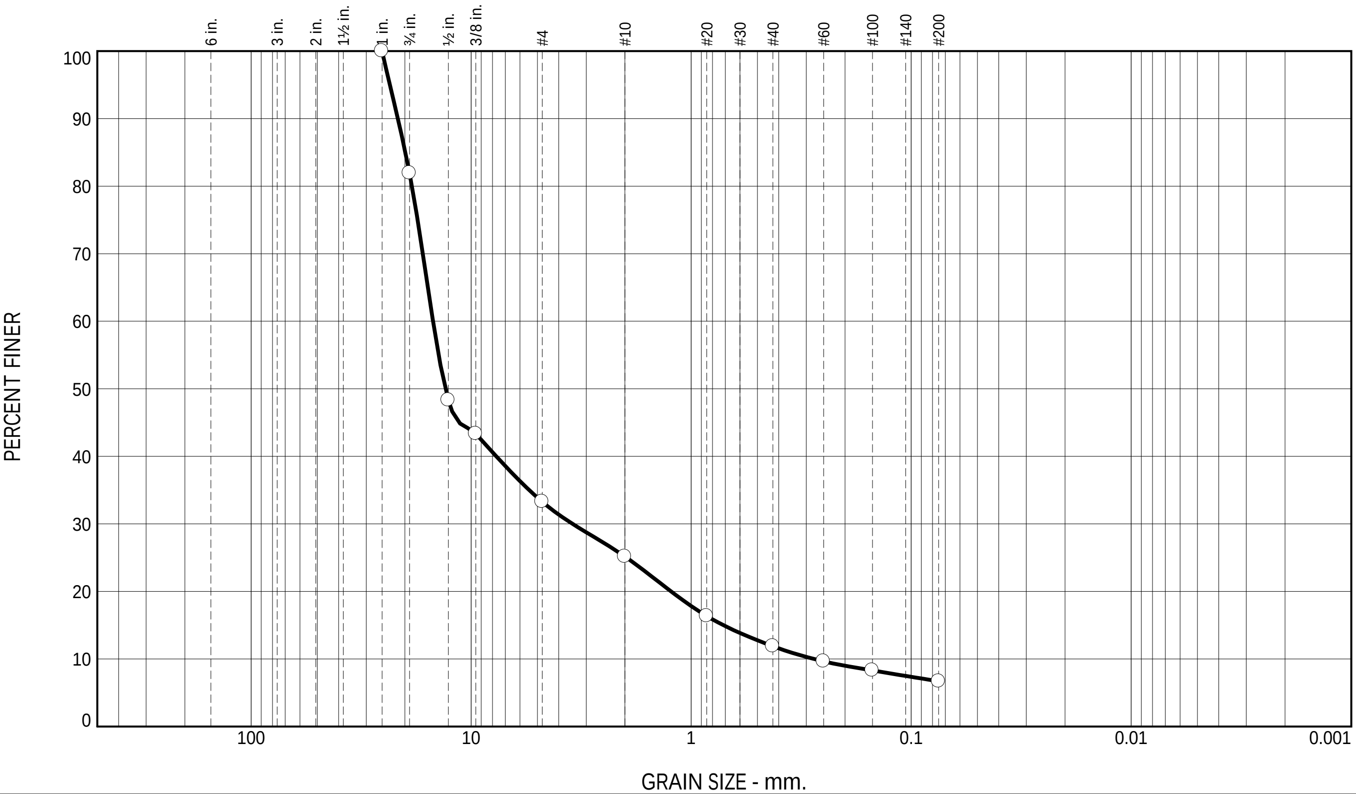
Client: GZA GeoEnvironmental
Project: Tannery Brook Bridge
Mariaville, ME
Project No: 09.0026171.00 Figure 22-S-4337

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: RB Checked By:

Particle Size Distribution Report

ASTM D6913



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	18.1	48.6	8.1	13.3	5.2	6.7	

Test Results (ASTM D6913)				
Sieve Size or Diam. (mm.)	Finer (%)	Spec.* (%)	Out of Spec. (%)	Pct. of Fines
1"	100.0			
3/4"	81.9			
1/2"	48.3			
3/8"	43.4			
#4	33.3			
#10	25.2			
#20	16.4			
#40	11.9			
#60	9.7			
#100	8.3			
#200	6.7			

* (no specification provided)

Material Description

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

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 21.5196 D_{85}= 19.9009 D_{60}= 14.9020
D_{50}= 13.0931 D_{30}= 3.4578 D_{15}= 0.7132
D_{10}= 0.2760 C_u= 53.99 C_c= 2.91

Classification

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

Test Remarks

Source of Sample: SPT Depth: 15-15.7'
Sample Number: 101 / 4D

Sample Date: 11.8.22

Thielsch Engineering Inc.

Cranston, RI

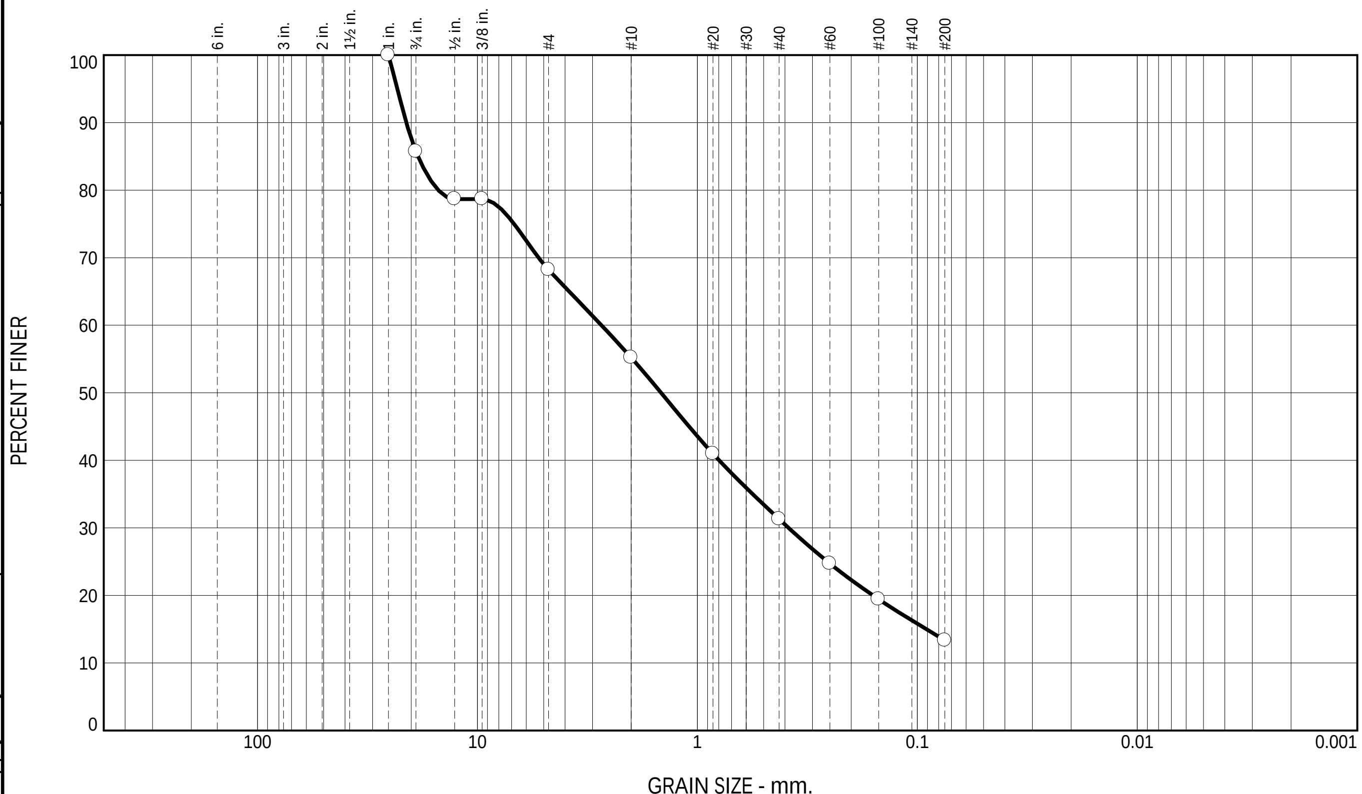
Client: GZA GeoEnvironmental
Project: Tannery Brook Bridge
Mariaville, ME
Project No: 09.0026171.00

Figure 22-S-4338

Tested By: RB Checked By:

Particle Size Distribution Report

ASTM D6913



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	14.3	17.5	13.0	23.9	18.0	13.3	

Test Results (ASTM D6913)

Sieve Size or Diam. (mm.)	Finer (%)	Spec.* (%)	Out of Spec. (%)	Pct. of Fines
1"	100.0			
3/4"	85.7			
1/2"	78.7			
3/8"	78.7			
#4	68.2			
#10	55.2			
#20	41.0			
#40	31.3			
#60	24.7			
#100	19.4			
#200	13.3			

* (no specification provided)

Material Description

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

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 21.0549 D_{85}= 18.6351 D_{60}= 2.7325
D_{50}= 1.4620 D_{30}= 0.3848 D_{15}= 0.0909
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-1-b

Test Remarks

Source of Sample: SPT Depth: 0.5-2.5"
Sample Number: 102 / 1D
Sample Date: 11.8.22

Thielsch Engineering Inc.

Cranston, RI

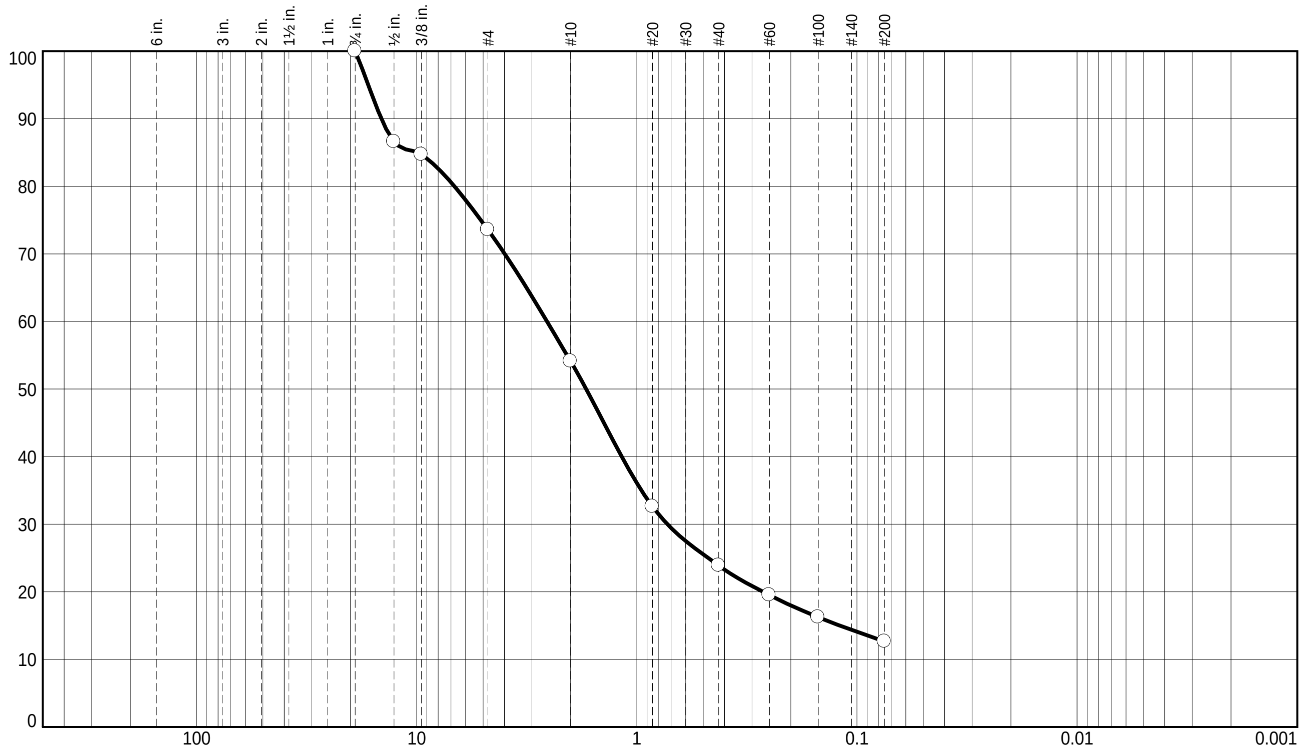
Client: GZA GeoEnvironmental
Project: Tannery Brook Bridge
Mariaville, ME
Project No: 09.0026171.00
Figure 22-S-4339

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: RB Checked By:

Particle Size Distribution Report

ASTM D6913



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	26.4	19.5	30.2	11.3	12.6	

Test Results (ASTM D6913)

Sieve Size or Diam. (mm.)	Finer (%)	Spec.* (%)	Out of Spec. (%)	Pct. of Fines
3/4"	100.0			
1/2"	86.6			
3/8"	84.7			
#4	73.6			
#10	54.1			
#20	32.6			
#40	23.9			
#60	19.5			
#100	16.2			
#200	12.6			

* (no specification provided)

Material Description

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

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 14.4896 D_{85}= 9.8987 D_{60}= 2.5619
D_{50}= 1.6996 D_{30}= 0.7268 D_{15}= 0.1197
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-1-b

Test Remarks

Source of Sample: SPT Depth: 5-6.5"
Sample Number: 102B / 1D Sample Date: 11.8.22

Thielsch Engineering Inc.
Cranston, RI

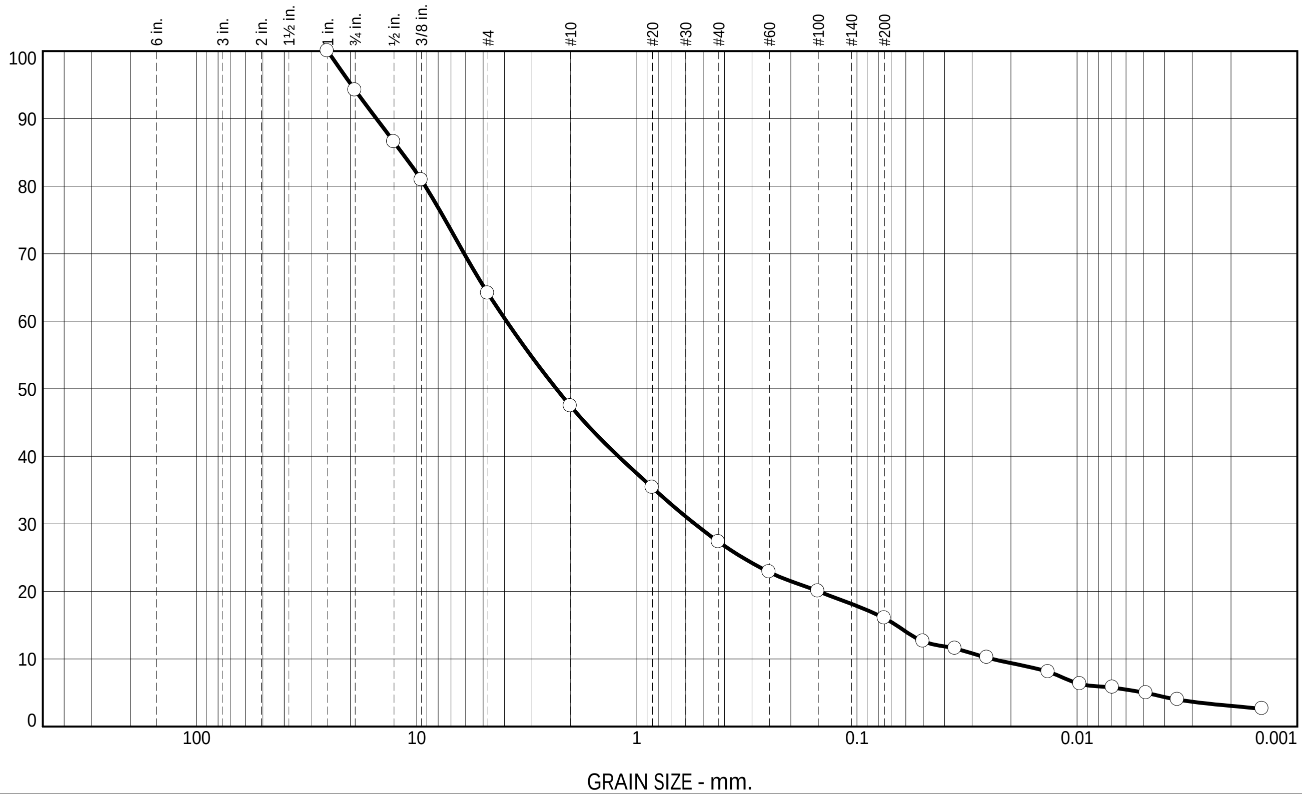
Client: GZA GeoEnvironmental
Project: Tannery Brook Bridge
Mariaville, ME
Project No: 09.0026171.00 Figure 22-S-4340

Tested By: RB Checked By:

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.

Particle Size Distribution Report

ASTM D6913 and ASTM D7928



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	5.8	30.1	16.7	20.1	11.3	12.9	3.1

Test Results (ASTM D6913 and ASTM D7928)

Sieve Size or Diam. (mm.)	Finer (%)	Spec.* (%)	Out of Spec. (%)	Pct. of Fines
1"	100.0			
3/4"	94.2			
1/2"	86.5			
3/8"	80.9			
#4	64.1			
#10	47.4			74.0
#20	35.4			55.1
#40	27.3			42.6
#60	22.8			35.6
#100	20.0			31.2
#200	16.0			25.0
0.0501 mm.	12.6			
0.0358 mm.	11.5			
0.0256 mm.	10.2			
0.0135 mm.	8.1			
0.0097 mm.	6.3			
0.0069 mm.	5.8			
0.0048 mm.	4.9			
0.0035 mm.	4.0			
0.0014 mm.	2.6			

* (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}= 15.2622 D_{85}= 11.7149 D_{60}= 3.9086
D_{50}= 2.3227 D_{30}= 0.5459 D_{15}= 0.0663
D_{10}= 0.0243 C_u= 160.55 C_c= 3.13

Classification

USCS= SM AASHTO= A-1-b

Test Remarks

Source of Sample: SPT Depth: 10-11.6"
Sample Number: 102E / 1D

Sample Date: 11.9.22

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Tannery Brook Bridge
Mariaville, ME
Project No: 09.0026171.00

Figure 22-S-4341

Tested By: SL / RB / SL Checked By:

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.

Thielsch
DIVISION OF THE RISE GROUP

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
South Portland, ME
PM: Michael Johnescu
Assigned By: Michael Johnescu
Collected By: LN

Project Information:
Tannery Brook Bridge
Mariaville, Maine
Project Number: 09.0026171.00
Summary Page: 1 of 1
Report Date: 11.15.22

LABORATORY TESTING DATA SHEET, Report No.: 7422-L-126

Boring No.	Sample No.	Depth (ft)	Laboratory No.	Specimen Data						Compressive Strength Tests								Rock Formation or Description or Remarks
				Mohs Hard-ness	Diameter (in)	Length (in)	(1) Unit Weight (PCF)	(2) Wet Density (PCF)	Bulk G_s	(3) Other Tests	(4) Strength PSI	(5) Strain %	(6) E sec PSI EE+06	(7) Poisson's Ratio	στ PSI	Is_{50} PSI	(8) s_c PSI	
BB-MTB-101	R2	20.5-21.05	22-S-4407		1.987	4.344	171.2				5961	0.336	6.10	0.31				Grey Slate
Broke along foliation, minor break at 2904 psi																		
BB-MTB-102 E	R1	16.8-17.65	22-S-4408		1.985	4.387	173.8				6818	0.312	1.80	0.16				Grey Slate
Broke along foliation, minor break at 3555 psi																		
BB-MTB-102 E	R1	16.0-17.65	22-S-4409		1.906	1.986	172.0			PLD						330		Grey Slate
BB-MTB-102 E	R1	16.0-17.65	22-S-4409		1.534	1.976	173.9			PLA						297		Grey Slate

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

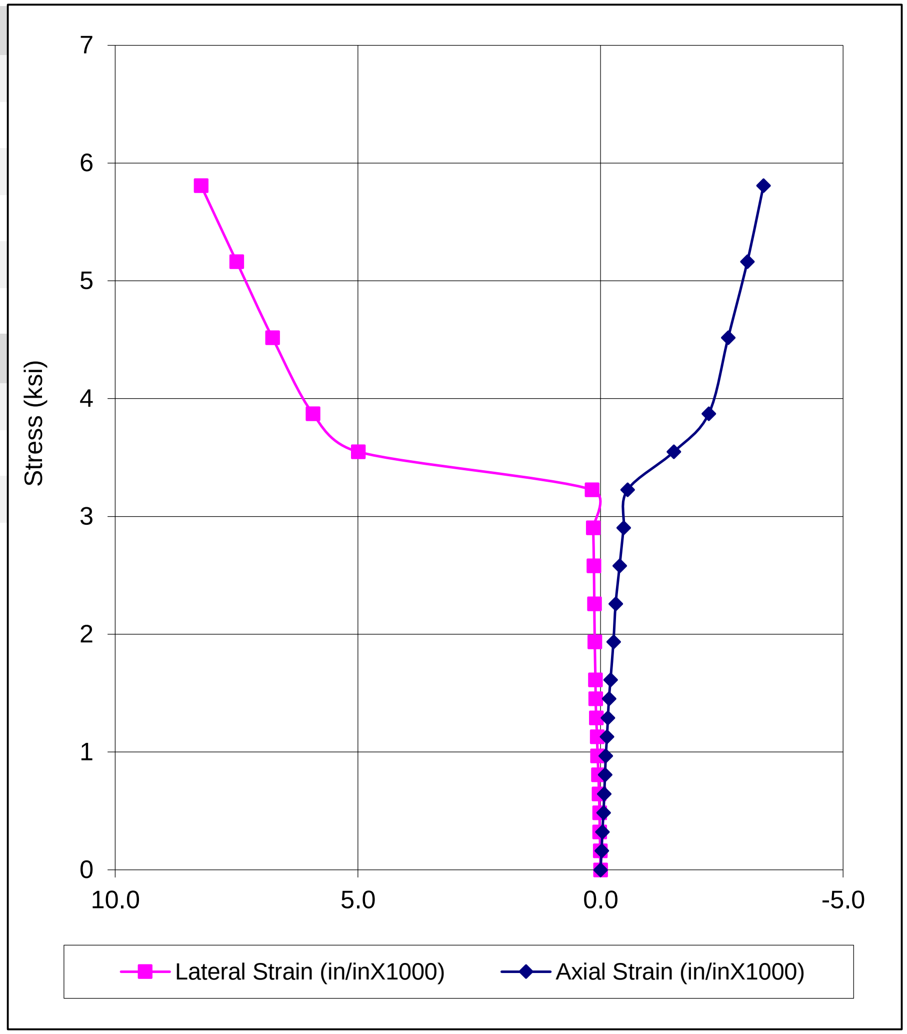
(8) Estimated UCS from Table 1 of ASTM D5731 for NX cores (Is x 24)

Date Received: *11.04.22* Reviewed By: ____ Date Reviewed: *11.15.22*

THIELSCH ENGINEERING	195 Frances Avenue Cranston, Rhode Island 02910 Phone: (401) 467-6454 Fax: (401) 467-2398 www.thielsch.com *Let's Build a Solid Foundation*	**Client Information:** GZA GeoEnvironmental South Portland, ME PM: Michael Johnescu Assigned by: Michael Johnescu Collected by: LN	**Project Information:** Tannery Brook Bridge Mariaville, ME Project Number: 09.0026171.00 Technician: AV Report Date: 11.10.22

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

Sample Information		Compressive Test Information	
Boring ID:	BB-MTB-101	**Unit Weight (pcf):**	171.2
Sample #:	R2	**Failure Stress (psi):**	5,961
Depth (ft):	20.5-21.05	**Failure Mode:**	Fresh
Tested Depth (ft):		**Time to Failure (min)**	4.88
Rock Type:	Grey Slate		
Features:	Broke along foliation		
Test Specimen Information		**Elastic Moduli Test Information**	
Diameter, D (in):	1.987	**Poisson's Ratio @ 50%:**	0.31
Length, L (in):	4.344	**Strain %:**	0.336
L:D Ratio:	2.19	**E sec PSI @ 50%:**	6.10E+06



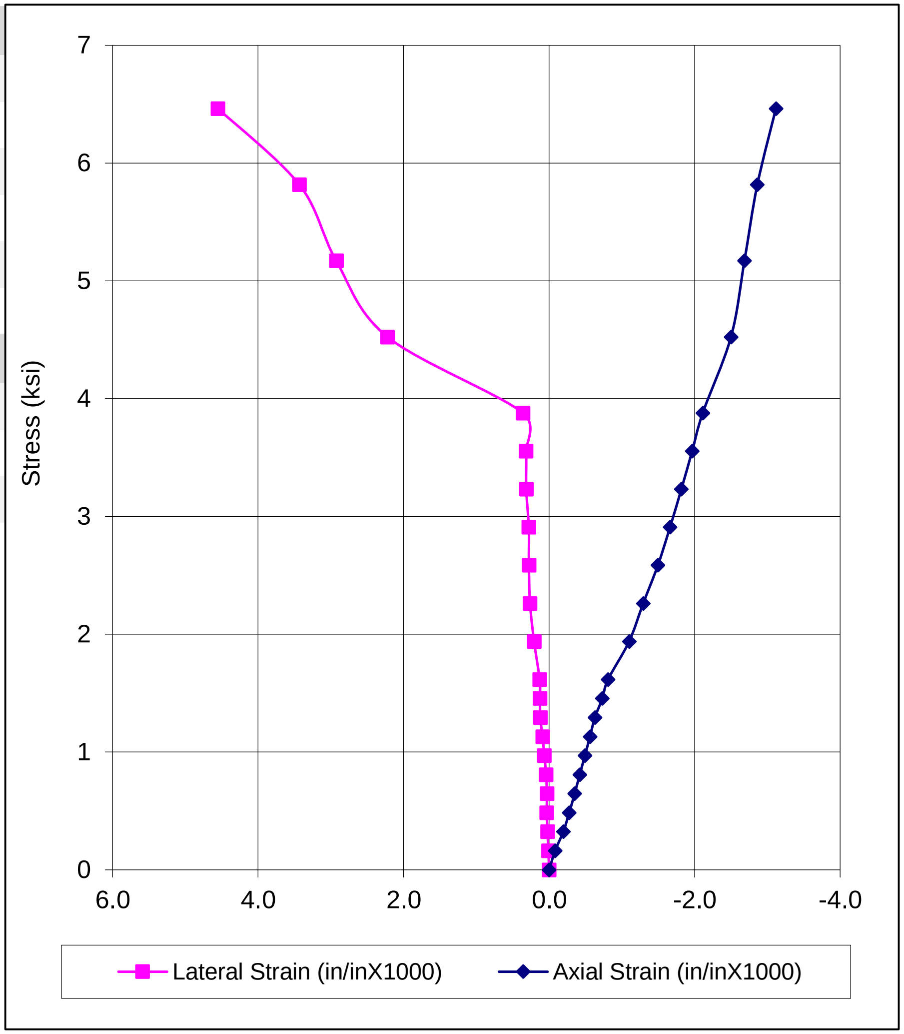
Testing Notes: **Minor break at 2904 psi**

THIELSCH ENGINEERING	195 Frances Avenue Cranston, Rhode Island 02910 Phone: (401) 467-6454 Fax: (401) 467-2398 www.thielsch.com *Let's Build a Solid Foundation*	**Client Information:** GZA GeoEnvironmental South Portland, ME PM: Michael Johnescu Assigned by: Michael Johnescu Collected by: LN	**Project Information:** Tannery Brook Bridge Mariaville, ME Project Number: 09.0026171.00 Technician: AV Report Date: 11.10.22

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

Sample Information		Compressive Test Information	
Boring ID:	BB-MTB-102E	**Unit Weight (pcf):**	173.8
Sample #:	R1	**Failure Stress (psi):**	6,818
Depth (ft):	16.8-17.65	**Failure Mode:**	Fresh
Tested Depth (ft):		**Time to Failure (min)**	5.45
Rock Type:	Grey Slate		
Features:	Broke along foliation		

Test Specimen Information		Elastic Moduli Test Information	
Diameter, D (in):	1.985	**Poisson's Ratio @ 50%:**	0.16
Length, L (in):	4.387	**Strain %:**	0.312
L:D Ratio:	2.21	**E sec PSI @ 50%:**	1.80E+06



Testing Notes: **Intial Break at 3555 psi**

APPENDIX D – ROCK CORE PHOTOGRAPHS

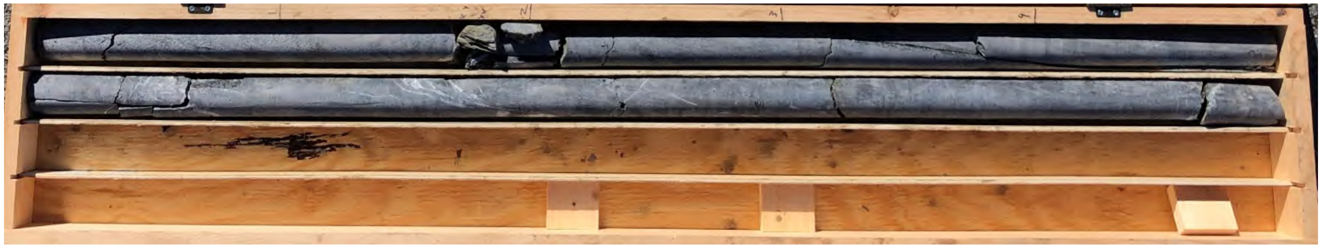
Mariaville Road Over Tannery Brook No. 3511
Mariaville, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)			Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-MTB-101	R1	16.5	-	20	42	100	18	42	SLATE	1
BB-MTB-101	R2	20	-	25	60	100	38	63	SLATE	1,2
BB-MTB-101	R3	25	-	26.8	22	100	8	36	SLATE	2,3

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

Mariaville Road Over Tannery Brook No. 3511
Mariaville, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-MTB-102E	R1	14.5 - 19.5	60	100	53	88	SLATE	1
BB-MTB-102E	R2	19.5 - 24.5	60	100	53	88	SLATE	2



Notes: 1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 2=Bottom.
2. Top photo is dry, bottom photo is wet.

APPENDIX E – GEOTECHNICAL ENGINEERING CALCULATIONS

Objective

Assess nominal and factored bearing resistance of a foundation on rock based on support in slate rock from borings BB-MTB-101 and -102E.

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 Tannery Brook Bridge Project in Mariaville, ME. This calculation is based on the data from borings BB-MTB-101, and -102.

Bedrock Quality

Representative RQD's are shown in the table below.

Boring ID	Core Run	Ground Surface El. (ft)	Depth of Core Run below GS (ft)			Length of Core Run (in)	Rec (in)	Rec (%)	RQD (in)	RQD %	Joint Spacing Desc.	Corr. Spacing (in)	Aperture Desc.	Corr. Aperture (in)
			Top		Bottom									
BB-MTB-101	R1	118.0	16.5	-	20.0	42	42	100%	18	42%	Extremely Close to Close	<0.75-8	Very Tight to Tight	<0.004-0.01
BB-MTB-101	R2	118.0	20.0	-	25.0	60	60	100%	38	63%	Extremely Close to Moderate	<0.75-24	Very Tight to Tight	<0.004-0.01
BB-MTB-101	R3	118.0	25.0	-	26.8	22	22	100%	8	36%	Extremely Close to Close	<0.75-8	Tight	0.004-0.01
BB-MTB-102E	R1	117.4	14.5	-	19.5	60	60	100%	53	88%	Moderately Spaced	24	Tight	0.004-0.01
BB-MTB-102E	R2	117.4	19.5	-	24.5	60	60	100%	53	88%	Close to Moderate	2.5-24	Tight	0.004-0.01

RQD between 36% and 88% for core runs at each location. Representative RQD of 50-75% range selected.

Bedrock Strength

Boring ID	Core Run	LAB									Rock Type
		Depth to Top of Sample (ft)	Depth to Top of Sample into Rock (ft)	Elev Top of Sample (ft)	UCS (psi)	Poissons Ratio	Point Load Index, Is50 (psi)	Correlated UCS from Point Load Tests (psi)	Young's Modulus (ksi)	Unit Wt (pcf)	
BB-MTB-101	R1										SLATE
BB-MTB-101	R2	20.5	4.8	97.5	5,961	0.31			6,100	171.2	SLATE
BB-MTB-101	R3										SLATE
BB-MTB-102E	R1	16.8	2.8	100.6	6,818	0.16	330 (D) 297 (A)	7,920 (D) 7,128 (A)	1,800	173.8	SLATE
BB-MTB-102E	R2										SLATE

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

$$\sigma_{u.r} := 5960\text{psi} = 858\cdot\text{ksf}$$

Unconfined compressive strength varies from approximately 5.96 to 6.82 ksi and correlated strength from point load tests ranges from 7.13 to 7.92 ksi. Take the minimum UCS value for design.

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_1 := 4$ for $\sigma_{u.r}$ between 520 and 1,080 ksf

Parameter 2- Drill Core Quality

Representative RQD from table above: 36 - 88% 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 extremely close to moderately spaced = <0.75 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

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

$$RMR = 47$$

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 empirircal rock property constants

Slate 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).

$m := 0.228$

$s := 0.00015$

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

$s = 0.00015$

$m = 0.228$

$\sigma_{u.r} = 5.96 \cdot ksi$

Nominal Bearing Resistance

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

$q_n = 57.1 \cdot ksf$ Say 57 ksf

Factored Bearing Resistance (Strength Condition)

Bearing Resistance Factor is specified in Table 10.5.5.2.2-1

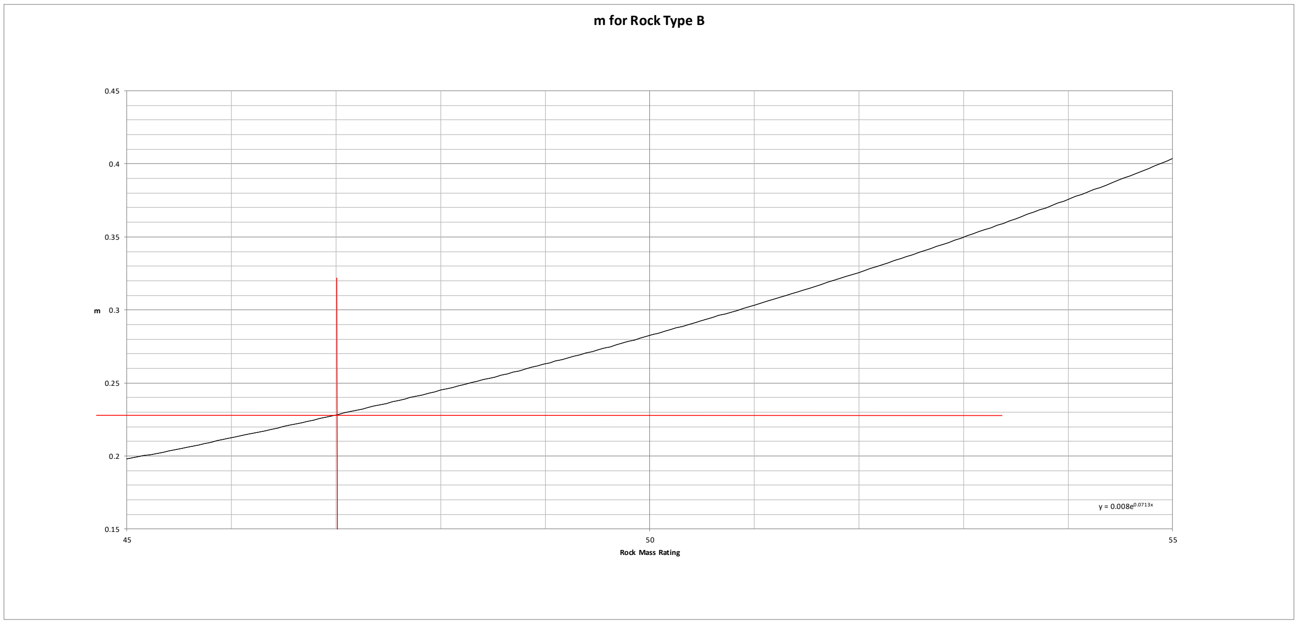
$\phi_b := 0.45$ Footing on rock

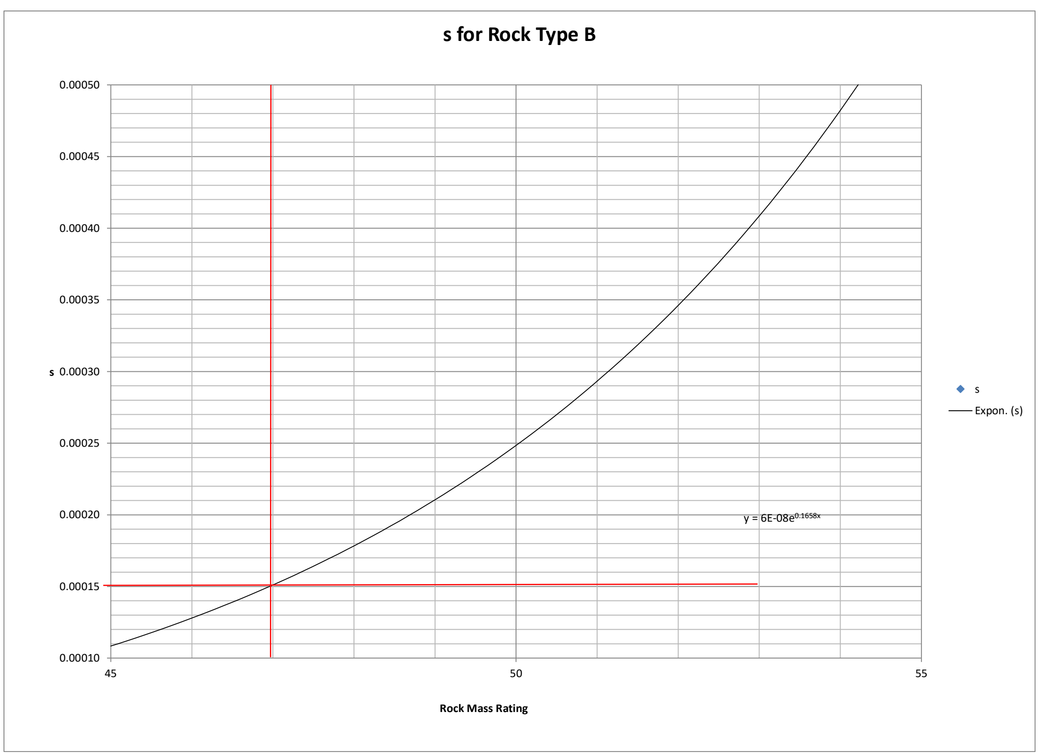
$q_R := \phi_b \cdot q_n$

$q_R = 25.7 \cdot ksf$ Say 25.5 ksf

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

Engineers and Scientists





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

Engineers and Scientists

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}

5.2 General

5.2.1 Frost

Any foundation placed on seasonally frozen soils must be embedded below the depth of frost penetration to provide adequate frost protection and to minimize the potential for freeze/thaw movements. Fine-grained soils with low cohesion tend to be most frost susceptible. Soils containing a high percentage of particles smaller than the No. 200 sieve also tend to promote frost penetration.

In order to estimate the depth of frost penetration at a site, Table 5-1 has been developed using the Modified Berggren equation and Figure 5-1 Maine Design Freezing Index Map. The use of Table 5-1 assumes site specific, uniform soil conditions where the Geotechnical Designer has evaluated subsurface conditions. Coarse-grained soils are defined as soils with sand as the major constituent. Fine-grained soils are those having silt and/or clay as the major constituent. If the make-up of the soil is not easily discerned, consult the Geotechnical Designer for assistance. In the event that specific site soil conditions vary, the depth of frost penetration should be calculated by the Geotechnical Designer.

Table 5-1 Depth of Frost Penetration

Design Freezing Index	Frost Penetration (in)					
	Coarse Grained			Fine Grained		
	w=10%	w=20%	w=30%	w=10%	w=20%	w=30%
1000	66.3	55.0	47.5	47.1	40.7	36.9
1100	69.8	57.8	49.8	49.6	42.7	38.7
1200	73.1	60.4	52.0	51.9	44.7	40.5
1300	76.3	63.0	54.3	54.2	46.6	42.2
1400	79.2	65.5	56.4	56.3	48.5	43.9
1500	82.1	67.9	58.4	58.3	50.2	45.4
1600	84.8	70.2	60.3	60.2	51.9	46.9
1700	87.5	72.4	62.2	62.2	53.5	48.4
1800	90.1	74.5	64.0	64.0	55.1	49.8
1900	92.6	76.6	65.7	65.8	56.7	51.1
2000	95.1	78.7	67.5	67.6	58.2	52.5
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

Average Moisture Content = 10.8%
Design Freezing Index = 1500
Soil: Coarse & Fine Grained
Depth = (82.1+58.3)/2 = 70.2" = 5' 9"

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.

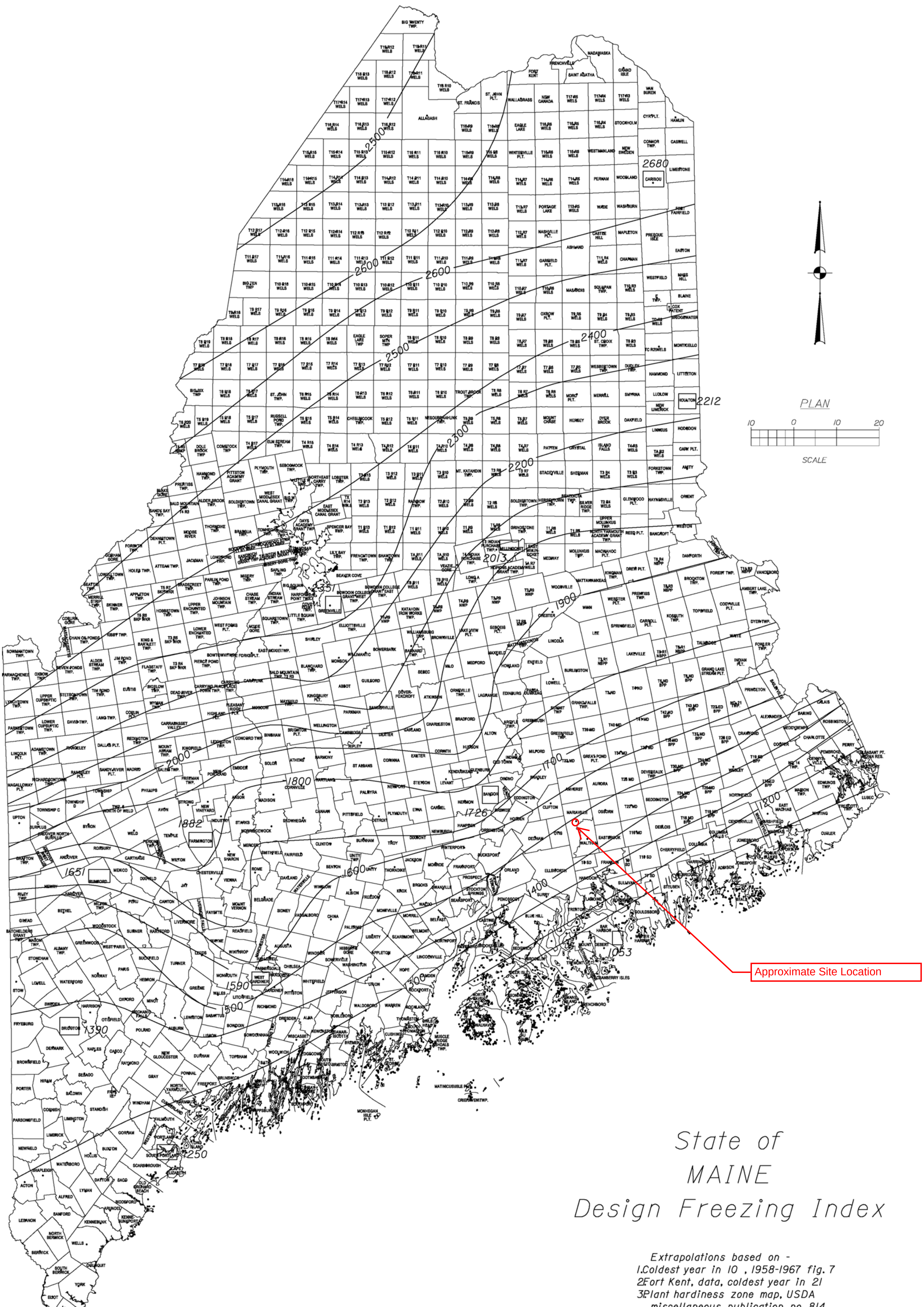


Figure 5-1 Maine Design Freezing Index Map

Example 5-1 illustrates how to use Table 5-1 and Figure 5-1 to determine the depth of frost penetration:

Example 5-1 Depth of Frost Penetration

Given: Site location is Freeport, Maine
Soil conditions: Silty fine to coarse Sand

Step 1. From Figure 5-1 Design Freezing Index = 1300 degree-days
Step 2. From laboratory results: soil water content = 28% and major constituent Sand
Step 3. From Table 5-1: Depth of frost penetration = 56 inches = 4.7 feet

Spread footings founded on bedrock require no minimum embedment depth. Pile supported footings will be embedded for frost protection. The minimum depth of embedment will be calculated using the techniques discussed in Example 5-1. Pile supported integral abutments will be embedded no less than 4.0 feet for frost protection.

Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection.

The final depth of footing embedment may be controlled by the calculated scour depth and be deeper than the depth required for frost protection. Refer to Section 2.3.11 Scour for information regarding scour depth.

5.2.2 Seal Cofferdams

Seal cofferdams are used when a substructure unit must be constructed with its foundation more than 4 feet below the water table, to counteract the buoyant forces produced during pumping of the cofferdam. Once the cofferdam is constructed, the seal is placed under water and water is then pumped out of the cofferdam. This provides a dry platform for construction of the spread footing, or in the case of a pile foundation, the distribution slab. When a seal is needed, the top of footing or distribution slab is located approximately at streambed, and the depth of seal is calculated based upon the buoyancy of the concrete under the expected water surface during construction. The following formula can be used:

$$145 \cdot y = 62.4 \cdot z$$

where:

$145\ lb/ft^3$ = unit weight of concrete
$62.4\ lb/ft^3$ = unit weight of water
y = the depth of seal from top of seal to bottom of seal
z = the depth of water from water surface to bottom of seal

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

Engineers and Scientists

JOB: 09.0026171.01 Tannery Brook Bridge
SUBJECT: Lateral Earth Pressures
SHEET: 1 OF 1
CALCULATED BY N. Williams 7/24/25
CHECKED BY_

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, 10th Edition (2024)

Input Parameters:

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

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

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

$\theta := 90 \cdot deg$ — 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.

$y := \frac{0.4 \cdot in}{2}$ — 1/2 of the Maximum deflection from thermal expansion of the entire bridge provided by structural engineer.

$H_b := 2ft$ — End Diaphragm Height

$\frac{y}{H_b} = 0.0083$ — Ratio of lateral movement to abutment height

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

$$\omega := \frac{y}{H_b} = 0.0083$$

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

$$K_{p.mass} = 4.96$$

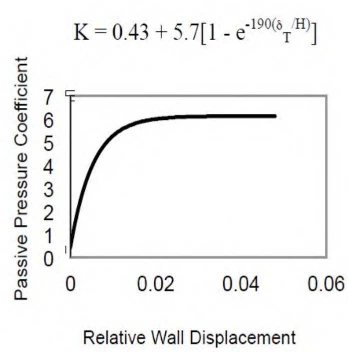


Figure 3.10.8-1: Plot of Passive Pressure Coefficient, K, vs. Relative Wall Displacement, δ_T/H.

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

Engineers and Scientists

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.

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

$$heel := 4.3ft$$

$$Intersection_{height} := \tan(90deg - \alpha)\cdot heel = 7.8\cdot ft$$

The abutment height is 7.5 feet (below the end diaphragm). Based on Figure C3.11.5.3-1 of LRFD, the abutment is considered to be a long-heeled wall. Therefore, Rankine theory should be used to calculate active earth pressures.

Rankine Active Earth Pressure Coefficient (Long-Heeled Wall)

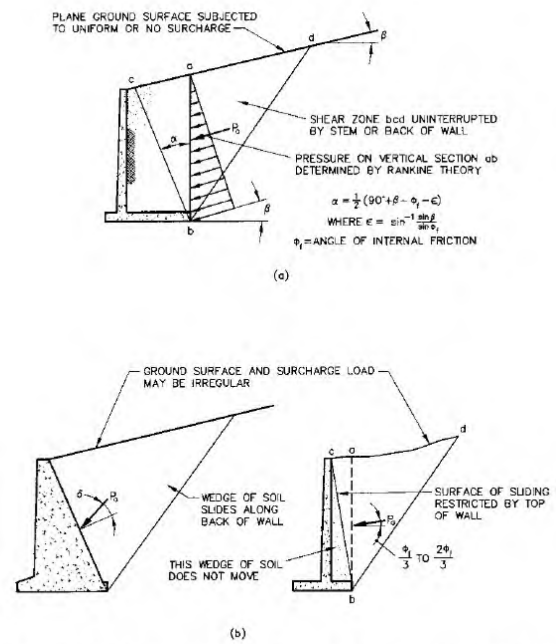


Figure C3.11.5.3-1—Application of (a) Rankine and (b) Coulomb Earth Pressure Theories in Retaining Wall Design

Rankine Active Earth Pressure Coefficient

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

$K_{ar} = 0.31$

Note - Need increased Ka if there will be sloping backfill behind wingwalls, consult GZA if so.