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GEOTECHNICAL DESIGN REPORT

DAY'S MILL BRIDGE NO. 2221

MAINE DOT WIN 26226.00

KENNEBUNK-ARUNDEL, MAINE

October 2024
File No. 09.0026198.01

Prepared for:
HNTB Corporation
South Portland, Maine

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VIA EMAIL

October 24, 2024
File No. 09.0026198.01

Ms. Ashley Stephens
HNTB Corporation
82 Running Hill Road
South Portland, ME 04106

Re: Geotechnical Design Report
Replacement of Day's Mill Bridge No. 2221
MaineDOT WIN 26226.00
Kennebunk-Arundel, Maine

Dear Ashley:

We are pleased to provide this Final Geotechnical Design Report, which includes geotechnical design recommendations for the replacement of the Day's Mill Bridge No. 2221 in Kennebunk-Arundel, Maine. Our work was completed in accordance with GZA GeoEnvironmental, Inc.'s Project Contract for Task Order No. GZA622.01 which incorporates our March 11, 2024 proposal, HNTB File No. 67328-DS-622-001-E008, dated March 3, 2024, our Master/Task Order Agreement dated December 8, 2020, and the attached Limitations contained in **Appendix A** of this report. HNTB is serving as the bridge designer for MaineDOT.

It has been a pleasure serving HNTB/MaineDOT 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.

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Attachment: Geotechnical Design Report

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1.0 INTRODUCTION

This report presents the results of the final design geotechnical evaluation by GZA GeoEnvironmental, Inc. (GZA) for the replacement of Maine Department of Transportation (MaineDOT) Day's Mill Bridge No. 2221 in Kennebunk-Arundel, Maine. Our work was completed in accordance with GZA GeoEnvironmental, Inc.'s Project Contract for Task Order No. GZA622.01 which incorporates our March 11, 2024 proposal, HNTB File No. 67328-DS-622-001-E008, dated March 3, 2024, our Master/Task Order Agreement dated December 8, 2020, and the attached Limitations contained in **Appendix A** of this report.

1.1 BACKGROUND

The project includes replacement of the Day's Mill Bridge No. 2221 carrying State Route 35 over Kennebunk River from Kennebunk to Arundel, Maine. The project location is shown on **Figure 1**. The existing bridge is a single span bridge with a span length of approximately 26 feet. The bridge was rebuilt in 1932 and consists of a 25-foot wide, simple-span, concrete bridge deck supported by concrete T-Beams founded on concrete gravity abutments. The abutments are understood to have been cast against existing stacked stone foundations at both abutments and wingwalls. Existing foundations are believed to bear directly on bedrock.

The selected bridge alternative is a single span bridge with a span length of 58 feet and a width of approximately 32 feet. The proposed bridge centerline will be approximately 16 feet east (downstream) from the existing bridge. The new abutments are anticipated to be designed with semi-integral abutment substructures supported on spread footings bearing directly on bedrock. We understand that Route 35 will be closed to traffic and a detour will be required to maintain traffic during construction.

1.2 OBJECTIVES AND SCOPE OF SERVICES

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

- Conducted a site visit to observe surficial and reviewed mapped surficial and bedrock geology of the site;
- Reviewed existing subsurface data and as-built plans;
- Coordinated and observed subsurface exploration programs for preliminary and final design to evaluate subsurface conditions and collect samples for laboratory testing;
- Requested and were provided with rock outcrop survey data for additional consideration of bedrock elevations for footing evaluations;
- Conducted laboratory testing programs to evaluate engineering and index properties of the site soils and bedrock;
- Conducted final design geotechnical engineering analyses to evaluate feasible foundation types; final design parameters; considerations for widened embankments; and seismic design parameters;
- Developed geotechnical construction considerations; and
- Prepared this geotechnical design report summarizing our findings and design recommendations.

2.0 SUBSURFACE EXPLORATIONS

GZA completed a preliminary design exploration program in 2023 consisting of two test borings designated as BB-KAKR-101 and -102, and a final design supplemental exploration program in 2024 consisting of three borings designated as BB-KAKR-201 through -203. GZA's representative also marked and gave designations for survey points designated as TR-1 through TR-15, on the bedrock outcrops along the riverbanks in the vicinity of the new abutments, to provide additional top of rock data points. The points were subsequently surveyed by MaineDOT and are summarized in **Table 1** and shown on **Figure 2.**

Borings were drilled using 4-inch casing, and drive- or spin-and-wash drilling techniques, as noted on the boring logs. Standard penetration testing (SPT) and split spoon sampling were performed continuously or at standard 5-foot intervals using a 24-inch-long, 1-3/8-inch inside diameter sampler. The borings were backfilled with ¾-inch crushed stone and/or soil cuttings and topped with asphalt cold patch. GZA personnel monitored the drilling work and prepared logs of each boring that are included in **Appendix B**. Additional details of each program are described below.

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

2.1 PRELIMINARY DESIGN BORINGS

Borings BB-KAKR-101 and BB-KAKR-102 were drilled between September 13, 2023 and September 14, 2023, by New England Boring Contractors of Hermon, Maine. The test borings were completed using a Mobile B-53 drill carried on a CME track-mounted rig. The borings were drilled to depths of 26.5 and 33.5 feet below ground surface (bgs). Ten feet of bedrock was cored in each boring. SPTs were conducted using automatic hammer NEBC No. D-20, which had a rated hammer energy transfer ratio of 0.742 at the time of drilling, except for the upper 18 feet of BB-KAKR-101, which were conducted with a 140 lb. safety hammer with a rope and cathead, which has an assumed energy transfer ratio of 0.6.

2.2 FINAL DESIGN BORINGS

Borings BB-KAKR-201 though BB-KAKR-203 were drilled between April 10, 2024 and April 11, 2024, by New England Boring Contractors of Hermon, Maine. The test borings were completed using a Mobile B-53 drill carried on a CME track-mounted rig. The borings were drilled to depths of 18.0 to 21.0 feet bgs. Ten feet of bedrock was cored from each boring location. SPTs were conducted using automatic hammer NEBC No. D-20, which had a rated hammer energy transfer ratio of 0.742 at the time of drilling.

3.0 LABORATORY TESTING

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

- Twelve (12) gradation analysis / MaineDOT Frost Classification / AASHTO Soil Classifications;
- Twelve (12) moisture content tests;
- Five (5) unconfined compression tests on bedrock core samples; and
- Three (3) hydrometer tests.

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

4.0 SUBSURFACE CONDITIONS

4.1 SURFICIAL AND BEDROCK GEOLOGY

Based on available geologic mapping[1], the surficial units in the vicinity of the site consist of Presumpscot Formation marine silty clay; sand and gravel Marine Delta Deposits; and mixed silt, sand, gravel, cobble, and boulder Glacial Till; with bedrock exposures and anticipated shallow overburden depths.

Based on available bedrock geologic mapping[2], bedrock in the vicinity of the site consists of medium brownish-gray feldspathic quartz-biotite granofels, greenish calc-silicate granofels and subordinate quartz-biotite schist and is mapped as the Berwick Formation.

4.2 SUBSURFACE PROFILE

The existing bridge foundation appears to have been constructed directly on bedrock, with no record of marine silty clay shown on historic plans. Since the 1932 concrete appears to have been placed in front of pre-existing stone masonry abutments and based on our experience with similar structures, we anticipate that rock fill may be present behind the older stone masonry abutment above natural soils.

Two soil units were encountered above bedrock at the site: Fill and Glacial Outwash. The approximate thicknesses and generalized descriptions of the subsurface units are presented in the following table, in descending order from existing ground surface.

---

[1] Smith, Geoffrey W., 1999, Surficial geology of the Kennebunk 7.5-minute quadrangle, York County, Maine: Maine Geological Survey, Open-File Report 99-117, 9 p.. *Maine Geological Survey Publications*. 258. http://digitalmaine.com/mgs_publications/258

[2] Hussey, Arthur M., II, Bothner, Wallace A., and Thompson, Peter J., 2008, Bedrock geology of the Kittery 1:100,000 quadrangle, Maine and New Hampshire: Maine Geological Survey, Geologic Map 08-78 (Superseded by Hussey, Bothner, and Thompson, 2016, Maine Geological Survey Open-File 16-6), 1 plate, photographs, color map, cross section, scale 1:100,000. *Maine Geological Survey Maps*. 2043. http://digitalmaine.com/mgs_maps/2043

Soil Unit	Approximate Encountered Thickness (ft)	Generalized Description
Fill	2.6 to 16	Brown, loose to dense, fine to coarse SAND, trace to little silt, trace to some gravel, with occasional cobbles. (USCS: SM, SP-SM, SW-SM) Typical MaineDOT Frost Classification Range= 0 to II *Encountered in all borings*
Glacial Outwash	4.0 to 7.5	Varying from: Brown, loose to medium dense, GRAVEL, some fine to coarse sand, trace to little silt to: Grey, loose, Silty medium to fine SAND. (USCS: GM,SM, SP-SM, SW-SM). Probable cobbles and boulders throughout. Typical MaineDOT Frost Classification = II to IV *Encountered in borings BB-KAKR-102 and BB-KAKR-201 through BB-KAKR-203*
Estimated Top of Bedrock*	Abutment 1: El. 121.8 to 128.4 Abutment 2: El. 123.8 to 128.5	

* Estimated top of rock considers boring data and survey data for points nearest to the project baseline. See **Figure 2** for survey points included in the range in top of bedrock elevations.

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 and top of rock survey results 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 cored in each test boring was generally identified as a Schist and was described as hard, fresh to slightly weathered, fine to medium grained, and grey. In boring BB-KAKR-101, a Granofels intrusion was also encountered within the Schist bedrock and was described as hard, fresh to slightly weathered, fine to coarse grained, and grey. The joints are very close to moderately spaced, low angle to high angle, stepped to planar, smooth to rough, fresh to decomposed, and tight to open. The Rock Quality Designation (RQD) in the bedrock ranged from 0 to 93 percent (weighted average of 58 percent), corresponding to a rock quality of very poor to good. Dry and wet photographs of the collected rock core are presented in **Appendix D**.

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

SUMMARY OF BEDROCK STRENGTH TEST RESULTS

Boring	Depth below Existing Ground (ft bgs)	Depth below Top of Rock (ft bgs)	Elevation (ft NAVD 88)	Unconfined Compressive Strength (psi)	Secant Modulus @ 50% of Failure Stress (ksi)	Unit Weight (pcf)	Rock Type
BB-KAKR-101	23.7	0.2	121.6	3,201	1,180	173.9	SCHIST
BB-KAKR-101	25.8	2.3	119.5	5,138	1,490	165.9	GRANOFELS
BB-KAKR-102	20.6	4.6	123.9	4,072	2,450	173.8	SCHIST
BB-KAKR-201	13.3	2.8	142.0	4,766	3,090	163.6	SCHIST
BB-KAKR-203	14.8	6.8	143.9	6,286	4,560	174.1	SCHIST

4.2.2 Groundwater

Groundwater depths were measured in borings BB-KAKR-101 and -102 at depths between approximately 14.2 and 16 feet bgs, corresponding to approximately El. 129.3 to 130.3. Groundwater levels in the 100 series borings were measured immediately after removal of drill casing and may have been affected by drilling procedures, which included introduction of water for drilling purposes. Groundwater was not observed in borings BB-KAKR-201 through BB-KAKR-203.

The ground water observations were made at the varying times and under the conditions stated in the boring logs. Fluctuations in groundwater level occur due to variations in season, precipitation, stream levels and construction activities in the area. Consequently, water levels during construction are likely to vary from those encountered at the time the observations were made. Due to the shallow depth to bedrock, perched water conditions are anticipated to occur seasonally.

5.0 ENGINEERING EVALUATIONS

5.1 GENERAL

GZA conducted preliminary geotechnical engineering evaluations in accordance with *2020 AASHTO LRFD Bridge Design Specifications, 9th Edition* (herein designated as AASHTO) and the *MaineDOT Bridge Design Guide, 2003 Edition,* with 2018 updates (MaineDOT BDG).

5.2 PROPOSED CONSTRUCTION

We understand that a full bridge replacement is planned for the project. The current alternative includes shifting the centerline of the bridge approximately 16 feet to the east and increasing the span length to 58 feet. The new abutments will be located approximately 16 feet behind the existing abutments, where the existing and new bridge foundation footprints are aligned.

5.3 APPROACH EMBANKMENTS

Typical grade raises of 1 foot or less are shown on the drawings at the approaches to the new bridge. Where embankment widening is proposed on the right side, proposed grade raises of 5 feet are typical, with a maximum fill height of about 11 feet behind the right wingwall at Abutment 2. The approach embankments are proposed with typical side slope angles of 2 horizontal to 1 vertical (2H:1V), or flatter, except for the ground surface in front of each abutment, which will slope down to the river level at an inclination of approximately 1.5H:1V and will be protected by riprap.

We anticipate that the proposed embankment fills will be constructed primarily over medium dense Glacial Outwash or bedrock. Due to the typical strength and low compressibility, embankment settlement and global stability are not considered to be concerns for the project.

5.4 FOUNDATION DESIGN CONSIDERATIONS

5.4.1 Abutment Foundations

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

5.5 SEISMIC DESIGN CONSIDERATIONS

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

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

5.6 LOAD AND RESISTANCE FACTORS

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

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

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

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

5.7 SPREAD FOOTING DESIGN CONSIDERATIONS

5.7.1 Footing Bearing Resistance

Nominal and factored bearing resistances have been developed for the abutments using the Rock Mass Rating- (RMR-) based empirical correlation presented in "Foundations on Rock," by Duncan Wyllie. RMR was evaluated in accordance with Table 10.4.6.4-1 of the *2012 AASHTO LRFD Bridge Design Specifications, 6th Edition* (AASHTO). The current version (9th Edition) of the AASHTO Design Specifications does not include the RMR formulation that is included in the 6th Edition version. However, Articles C10.4.6.4 and 10.6.2.6.2 of the 9th Edition refer to RMR-based design procedures for footings on rock, so the 6th Edition methodology was utilized here.

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

DESIGN BEDROCK PROPERTIES FOR BEARING RESISTANCE EVALUATION					
Rock Type	**RQD (percent)**	**Unconfined Compressive Strength (ksi)**	**Rock Mass Rating (RMR)**	**m**	**s**
Schist	58	4.0	47	0.388	0.000145

Based on these parameters, the calculated nominal bearing resistance is 47 kips per square foot (ksf), resulting in a factored bearing resistance of 21 ksf for the strength limit state. Supporting calculations are provided in **Appendix E**.

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

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

5.8 ADDITIONAL FOUNDATION CONSIDERATIONS

5.8.1 Frost Penetration

Fill soils are anticipated to be present at the abutments and embankments, either as existing fill or imported backfill. Based on the MaineDOT BDG, Section 5.2.1, the Freezing Index for the site is 1,250, and with low-moisture content (<10 percent) soils, the estimated depth of frost penetration is approximately 6.2 feet. However, where abutment foundations bear directly on sound rock, there is no minimum requirement for footing embedment.

5.8.2 Lateral Earth Pressure

The material properties will be controlled by the backfill material, which is proposed to consist of BDG Type 4 soil. In accordance with the requirements of the BDG Section 5.4.3, the semi-integral abutments and wingwalls will be free to rotate and therefore should be designed for active earth pressure.

Thermal expansion of the bridge will cause the superstructure backwall (end diaphragm) to move toward the backfill, which will result in earth pressures ranging from at-rest to passive earth pressure. Therefore, the superstructure backwall should be designed for full passive pressure. HNTB provided a maximum expansion deflection of 0.38 inches for use in end diaphragm design. The end diaphragm height is approximately 4 feet resulting in a calculated abutment rotation of 0.0079 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.

The riprap detail in front of the abutments shows a standard keyway detail. Bedrock is likely to be present at or near ground surface, which will make creation of the keyway impractical.

6.2 SEISMIC DESIGN

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

SITE CLASS B SEISMIC DESIGN PARAMETERS	
Parameter	**Design Value**
Fpga	1.0
Fa	1.0
Fv	1.0
As (Period = 0.0 sec)	0.10 g
SDs (Period = 0.2 sec)	0.18 g
SD1 (Period = 1.0 sec)	0.05 g

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

6.3 ABUTMENT AND WINGWALL DESIGN

- Abutment backfill should consist of MaineDOT 703.19 Granular Borrow for Underwater Backfill, MaineDOT BDG Type 4 soil. Recommended soil properties for Type 4 soils are as follows:
  - Internal Friction Angle of Soil = 32°
  - Soil Total Unit Weight = 125 pcf
  - Coefficient of Passive Earth Pressure, K_p (use for design of end diaphragms), K_p= 4.86;
  - Coefficient of Active Earth Pressure, K_a=0.28 (use for design of abutments and wingwalls):
- Live load surcharge should be applied as a uniform lateral surcharge pressure using the equivalent fill height (H_{eq}) values developed in accordance with LRFD Section 3.11.6.4, based on the abutment/wingwall height and distance from the wall backface to the edge of traffic. A minimum H_{eq} of 2 feet is recommended.

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

6.3.1 Spread Footing Design

- The proposed abutments should be supported on spread footing foundations bearing on sound, intact bedrock. Footings designed to bear on intact bedrock should be designed using a nominal bearing resistance, q_n, of 47 ksf. At the strength limit state, footings should be designed for a maximum factored bearing resistance of 21 ksf. A bearing resistance of 21 ksf should also be used for service limit state design.
- Spread footings founded on bedrock should be checked for eccentricity with AASHTO Article 10.6.3.3. Eccentricity of the footing reaction at the strength limit state should be limited such that the resultant reaction on the base of the footing is no further than 0.45 B from the centerline of the footing, where B is the footing width perpendicular to the axis of rotation.
- The base resistance against sliding may be based on NAVFAC DM7.02-63, Table 1, which indicates the nominal sliding resistance coefficient (tan δ) is equal to 0.7 for cast-in-place concrete on sound rock. The factored sliding resistance coefficient is 0.56 for Strength Limit State.
- Existing substructures should be completely removed prior to new foundation construction where they interfere with new foundations.
- The bedrock surface should be cleaned of loose soil or rock prior to concrete placement for subfooting concrete or the footing. Bearing surface preparation should be in accordance with **Section 7.2**.
- The following table summarizes the top of bedrock elevations encountered in the borings and survey points located within or adjacent to foundation locations . These data, combined with the interpreted subsurface profile shown in **Figure 2**, are provided to assist the designer in developing bottom-of-footing elevations for the abutments.

ESTIMATED BEDROCK LEVELS FOR FOOTING DESIGN	
Foundation Element	**Estimated Range in Bedrock Elevation (feet, NAVD 88)**
Abutment 1	El. 121.8 to 128.4
Abutment 2	El. 123.8 to 128.5

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

- If the bedrock level extends above the design bottom of footing elevation, the footing may be raised and vertical reinforcement shortened in the wall, subject to review and approval of the Designer to limit the volume of bedrock excavation.
- If the exposed bedrock surface after cleaning is below the design footing bearing level, fill concrete may be placed up to the bottom of footing level with a minimum thickness of 6 inches.
- Concrete used for fill concrete beneath footings and for footings should consist of Class A Concrete in accordance with MaineDOT Standard Specification Section 502.05.

- Anchoring, doweling, benching or other means of improving sliding resistance is recommended at locations where the prepared bedrock surface is steeper than 4H:1V in any direction. However, it is GZA's understanding that the abutment and footing designer requires the threshold to be 12H:1V.
- Rock dowels may be used to supplement the sliding resistance for the footing. If used, the dowels should be grouted a minimum of 2 feet into intact bedrock and embedded at least 2 feet into concrete. The unconfined compressive strength of the bedrock should be assumed to be 4.0 ksi for design of rock dowels.
- Dowels should be grouted with a cementitious grout on the MaineDOT Qualified Products List of Grout Materials for Keyways and Anchoring (pre-qualified for anchoring). Epoxy grout should not be used.
- Since the footings will be founded on bedrock, there is no minimum embedment required for frost protection per BDG Article 5.2.1.

7.0 CONSTRUCTION CONSIDERATIONS

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

7.1 SUPPORT OF EXCAVATION AND DEWATERING

Excavations for abutment foundations will extend approximately 0 to 23.5 feet below existing grade to expose bedrock. The anticipated bedrock surface elevation ranges from approximately El. 121.8 to 128.4 at Abutment 1 and El. 123.8 to 128.5 at Abutment 2, corresponding to depths of approximately 2 feet below to 4 feet above the Q1.1 water level (El. 124.1) at Abutment 1, and at or 4 feet above the Q1.1 water level at Abutment 2. A water diversion system, such as sandbags with a membrane may be used as a flow diversion system at this site if the water depths allow this approach.

Sloped open cut excavation techniques are considered feasible between the abutments and the approach fills.

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

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

7.2 SUBGRADE PREPARATION

We anticipate it will be feasible to complete final bedrock subgrade preparation in-the-dry. The bedrock surface is known to be variable in terms of elevation, slope and localized weathering. Conventional excavation equipment such as hydraulic excavators and hydraulic rock breakers are anticipated to be sufficient to complete excavations. All soil and loose, decomposed, highly weathered and fractured bedrock should be removed from the footing bearing surface prior to placement of subfootings or footings. We anticipate that high-pressure air and or water will be used to clean the prepared bedrock surface.

The prepared bearing surfaces should be observed by the geotechnical engineer prior to placing concrete. The Geotechnical Engineer and Designer should also be provided cross-sections or contour plans showing the prepared rock surface geometry prior to placement of concrete to evaluate whether benching, doweling, or subfooting concrete fill are needed for that foundation location. If the exposed bedrock surface is steeper than 4H:1V, then anchoring, doweling, benching or other means should be designed by HNTB and/or GZA based on the exposed inclinations to provide sufficient sliding resistance for the design loads.

7.3 REUSE OF ON-SITE MATERIALS

Soil samples recovered from the existing approach fills typically had approximately 10 percent passing the No. 200 sieve, indicating the fill may meet MaineDOT specifications for Granular Borrow. Soil samples recovered from areas outside of the existing approach fills typically had 20 to 40 percent passing the No. 200 sieve, indicating that it will not meet MaineDOT specifications for Granular Borrow, but that 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 and Bedrock Survey Data
Day's Mill Bridge Replacement
Kennebunk-Arundel, Maine
GZA job#: 09.0026198.01

Exploration/Survey Point ID	Northing	Easting	Station	Offset	Ground Surface El. (ft)	Top of Stratum Elevation				Stratum Thickness (ft)			Depth to Bedrock (ft)	Bottom of Boring Depth (ft)	Bottom of Boring El. (ft)	Groundwater	
						Asphalt	Fill	Glacial Outwash	Bedrock	Asphalt	Fill	Glacial Outwash				El. (ft)	Depth (ft)
TEST BORINGS																	
BB-KAKR-101	227572.8	923039.6	549+82.1	6.8' L	145.3	145.3	144.3	129.3	121.8	1.0	15.0	7.5	23.5	33.5	111.8	129.3	16.0
BB-KAKR-102	227633.3	923019.5	550+50.3	6.9' L	144.5	144.5	143.5	NE	128.5	1.0	15.0	NE	16.0	26.5	118.0	130.3	14.2
BB-KAKR-201	227565.6	923088.3	549+65.6	36.2' R	142.0	NE	142.0	139.4	132.2	NE	2.6	7.2	9.8	20.5	121.5	NE	NE
BB-KAKR-202	227550.4	923021.3	549+68.0	32.6' L	144.5	NE	144.5	NE	133.7	NE	10.8	NE	10.8	21.0	123.5	NE	NE
BB-KAKR-203	227702.7	923026.1	551+10.3	24.6' R	143.9	NE	143.9	139.9	135.9	NE	4.0	4.0	8.0	18.0	125.9	NE	NE
BEDROCK SURVEY POINTS																	
TR-1	227586.1	923059.1	549+90.3	18.9' R					128.4								
TR-2	227591.8	923068.9	549+92.8	29.9' R					126.6								
TR-3	227595.3	923093.3	549+89.2	54.2' R					132.7								
TR-4	227594.2	923062.5	549+96.9	24.6' R					121.2								
TR-5	227591.3	923056.6	549+96.0	18.0' R					122.2								
TR-6	227597.5	923081.1	549+94.6	43.3' R					123.7								
TR-7	227652.9	923070.0	550+47.0	51.3' R					119.3								
TR-8	227649.2	923066.9	550+44.9	47.1' R					120.6								
TR-9	227650.8	923063.5	550+47.5	44.5' R					121.8								
TR-10	227644.3	923058.3	550+43.6	37.3' R					125.7								
TR-11	227644.3	923054.9	550+44.7	34.1' R					122.6								
TR-12	227642.2	923050.4	550+44.4	29.2' R					123.4								
TR-13	227637.3	923046.8	550+41.3	24.0' R					124.4								
TR-14	227633.9	923046.2	550+38.5	22.3' R					123.8								
TR-15	227629.8	923039.3	550+37.0	14.4' R					128.3								

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

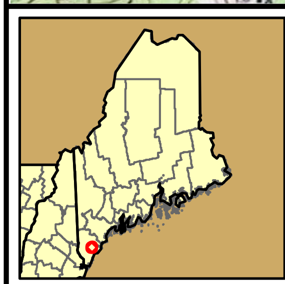
Notes:
1. Refer to the boring logs in Appendix B for additional information.
2. Project elevation datum is North American Vertical Datum (NAVD 88), unless noted otherwise.
3. As-drilled boring locations and elevations were surveyed by MaineDOT and provided to GZA.
4. Stratum depths, thickness and elevations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

TABLE 2
Summary of Bedrock Data
Day's Mill Bridge No. 2221 over Kennebunk River
Kennebunk-Arundel, Maine
MaineDOT WIN 26226.00

Boring ID	Core Run	Ground Surface Elevation (ft)	Depth of Core Run below Ground Surface (ft)			Depth to Rock (ft)	Depth (ft) Below Top of Rock			Length of Core Run (ft)	Rec (in)	Rec (%)	RQD (in)	RQD %	Joint Spacing (in)	Joint Aperture (in)	Elevation (ft)		LABORATORY TESTING						Rock Type
			Top		Bottom		Top		Bottom								Top	Bottom	Depth of Sample (ft)	Depth of Sample into Rock (ft)	Elev Top of Sample (ft)	UCS (psi)	Modulus (ksi)	Unit Wt (pcf)	
BB-KAKR-101	R1	145.3	23.5	-	28.5	23.5	0.0	-	5.0	5.0	60	100%	39	65%	0.75-24	0.004-0.4	121.8	116.8	*23.7*	*0.2*	*121.6*	*3,201*	*1,180*	*173.9*	Schist/Granofels
BB-KAKR-101	R2	145.3	28.5	-	33.5	23.5	5.0	-	10.0	5.0	60	100%	48	80%	2.5-24	0.004-0.1	116.8	111.8	*25.8*	*2.3*	*119.5*	*5,138*	*1,490*	*165.9*	Schist/Granofels
BB-KAKR-102	R1	144.5	16.5	-	21.5	14.5	2.0	-	7.0	5.0	58	96%	37	64%	0.75-8	0.004-0.1	128.0	123.0							Schist
BB-KAKR-102	R2	144.5	21.5	-	26.5	14.5	7.0	-	12.0	5.0	60	100%	49	81%	0.75-24	0.004-0.1	123.0	118.0	*16.8*	*2.3*	*127.7*	*4,070*	*2,450*	*173.8*	Schist
BB-KAKR-201	R1	142.0	10.5	-	15.5	10.5	0.0	-	5.0	5.0	57	95%	20	33%	0.75-8	0.004-0.1	131.5	126.5	*13.3*	*2.8*	*128.7*	*4,766*	*3,090*	*163.6*	Schist/Granofels
BB-KAKR-201	R2	142.0	15.5	-	20.5	10.5	5.0	-	10.0	5.0	59	98%	35	58%	0.75-8	0.004-0.1	126.5	121.5							Schist/Granofels
BB-KAKR-202	R1	144.5	11.0	-	14.7	11.0	0.0	-	3.7	3.7	42	93%	0	0%	0.75-8	0.004-0.1	133.5	129.8							Schist
BB-KAKR-202	R2	144.5	14.7	-	19.7	11.0	3.7	-	8.7	5.0	58	96%	47	78%	0.75-24	0.004-0.1	129.8	124.8							Schist
BB-KAKR-202	R3	144.5	19.7	-	21.0	11.0	8.7	-	10.0	1.3	14	99%	13	93%	8	0.004-0.01	124.8	123.5							Schist
BB-KAKR-203	R1	143.9	8.0	-	10.0	8.0	0.0	-	2.0	2.0	20	83%	0	0%	8	0.004-0.1	135.9	133.9							Schist
BB-KAKR-203	R2	143.9	10.0	-	14.0	8.0	2.0	-	6.0	4.0	48	100%	14	29%	0.75-8	0.004-0.1	133.9	129.9							Schist
BB-KAKR-203	R3	143.9	14.0	-	18.0	8.0	6.0	-	10.0	4.0	48	100%	43	89%	8	0.004-0.1	129.9	125.9	*14.8*	*6.8*	*129.1*	*6,286*	*4,560*	*174.1*	Shist/Granofels

Notes:
1. Refer to the boring logs in Appendix B for additional information.
2. Project elevation datum is North American Vertical Datum (NAVD 88), unless noted otherwise.
3. As-drilled locations were surveyed by MaineDOT and provided to GZA.

FIGURES



USGS QUADRANGLE LOCATION

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

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

SCALE IN FEET

Data Supplied by :

esri

USGS
science for a changing world

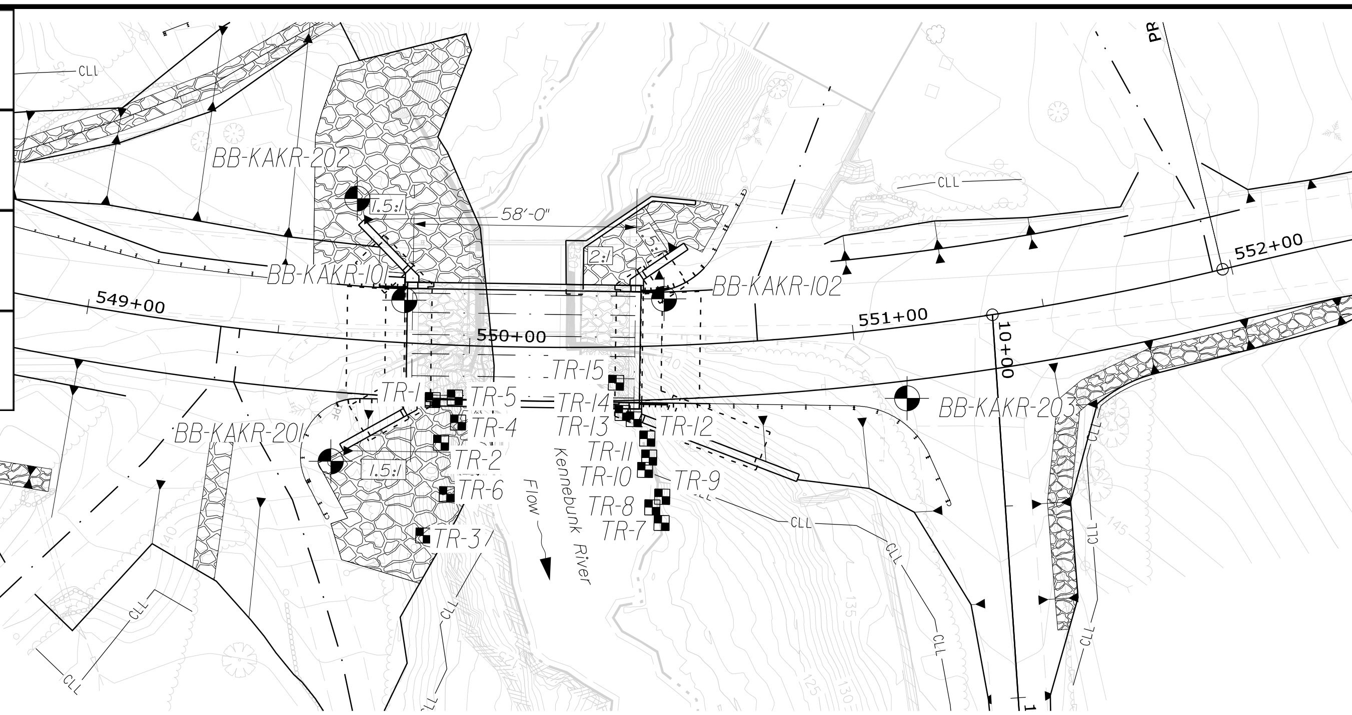
PROJ. MGR.: BMC DESIGNED BY: EAF REVIEWED BY: ARB OPERATOR: EAF DATE: 09-05-2024	LOCUS PLAN	JOB NO. 09.0026198.01
	DAY'S MILL BRIDGE KENNEBUNK-ARUNDEL, MAINE	FIGURE NO. 1

DAY'S MILL BRIDGE
MAINEDOT WIN 26226.00
KENNEBUNK, ME

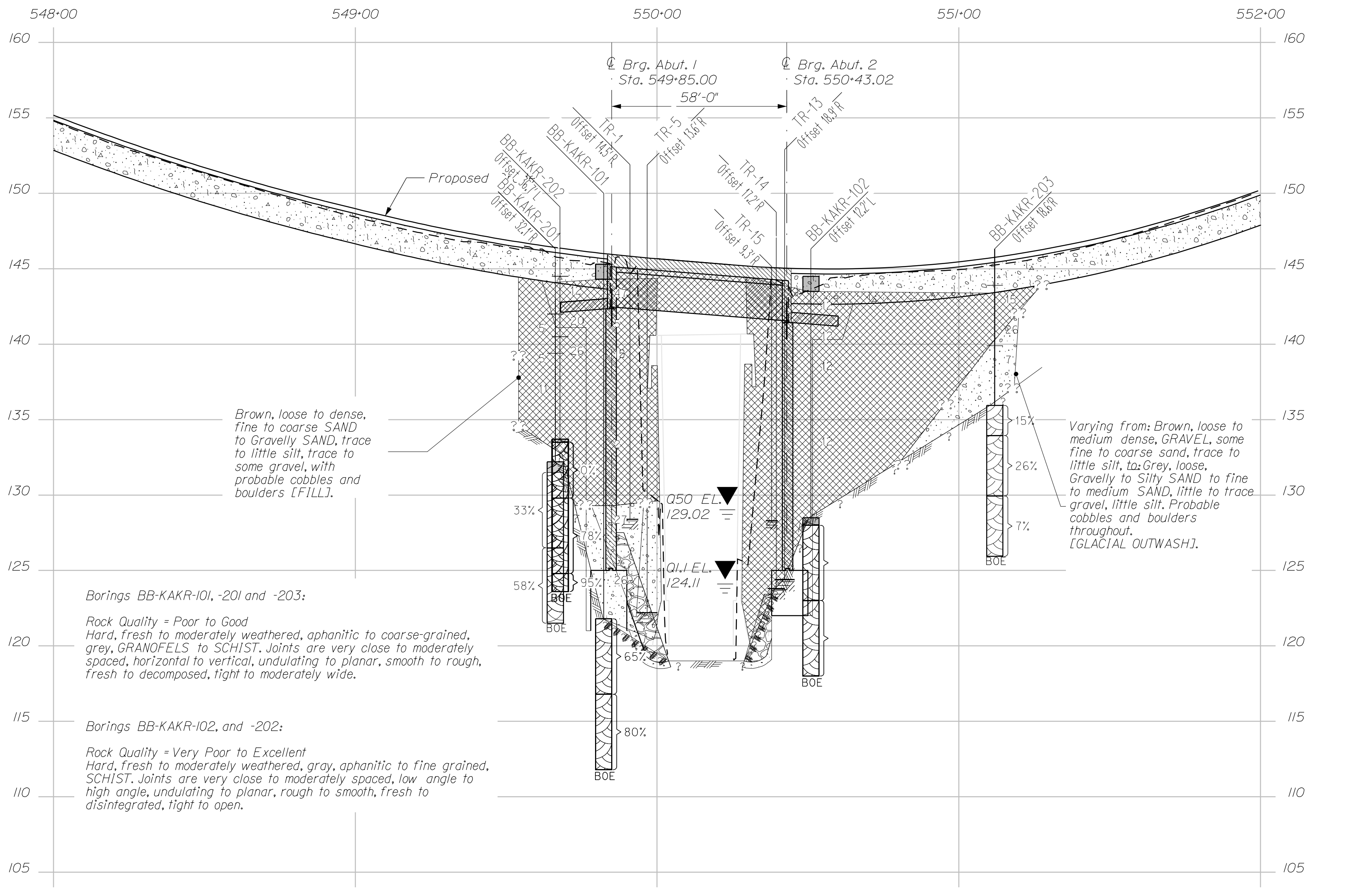
BRIDGE BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

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

PROJ MGR: BMC	REVIEWED BY: CLS	CHECKED BY: ARB	FIG 2
DESIGNED BY: BMC	DRAWN BY: BMC	SCALE: AS SHOWN	
DATE: 10/23/24	PROJECT NO. 09.0026198.01	REVISION NO. 0	SHEET NO. 2 OF 2



BORING LOCATION PLAN



INTERPRETIVE SUBSURFACE PROFILE

NOTES

1) Base map developed from electronic files (Alignments.dgn, Bridge.dgn, Contours.dgn, Profile.dgn, and Topo.dgn) provided by HNTB on September 17, 2024.

2) The as-drilled locations of the BB-KAKR-100 series test borings, and the top of rock elevations were surveyed and provided by MaineDOT in an electronic table (26226 Boring Date 11-2-23) on November 2, 2023.

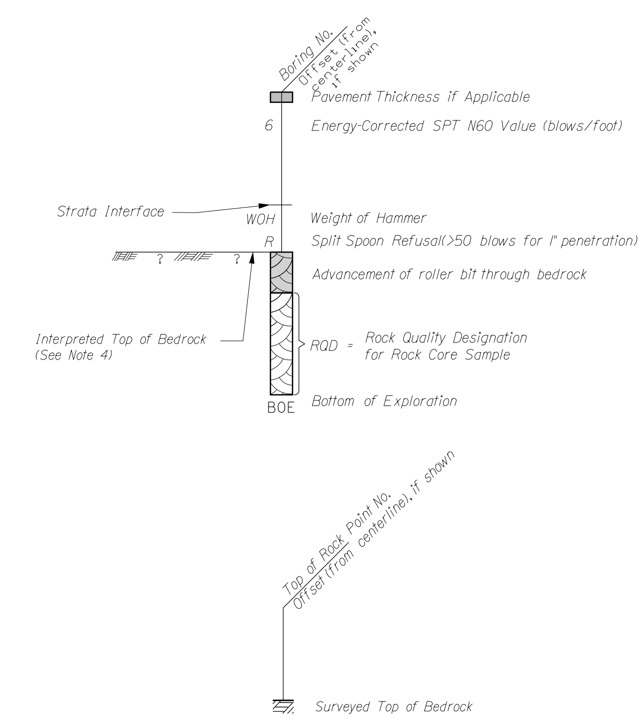
3) The as-drilled locations of the BB-KAKR-200 series test borings were surveyed and provided by MaineDOT in an electronic table (Boring Loc R26226A9-Edit) on May 7, 2024.

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

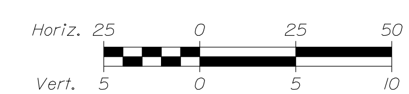
BORING LOCATION PLAN LEGEND

BB-KAKR-102	Location and designation of BB-KAKR-100 series borings performed by New England Boring Contractors of Hermon, Maine between September 13 and September 14, 2023
BB-KAKR-203	Location and designation of BB-KAKR-200 series borings performed by New England Boring Contractors of Hermon, Maine between April 10 and April 11, 2024
TR-15	Location and designation of exposed rock outcrop survey point collected by MaineDOT between September 13 and September 14, 2023.

INTERPRETIVE SUBSURFACE PROFILE LEGEND



SCALE



Scale of Feet

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2622600

WIN 26226.00

BRIDGE PLANS

BRIDGE NO. 2221

		BY	DATE
PROJ. MANAGER	M. WIGHT		
DESIGN-DETAILED	L. NAVARRETE	B. CARDALI	10/2024
CHECKED-REVIEWED	C. SNOW	A. BLAISDELL	10/2024
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE
P.E. NUMBER
DATE

DAY'S MILL BRIDGE
KENNEBUNK RIVER
KENNEBUNK-ARUNDEL
YORK

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
6
OF 51

Filename: ...\BLP_ISP 10.23.24 combined.dgn Division: HIGHWAY Username: common Date:10/23/2024

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

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Day's Mill Bridge No. 2221		**Boring No.:**	BB-KAKR-101
Location: Kennebunk / Arundel, Maine		**WIN:**	26226.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	145.3	**Auger ID/OD:**	4.25" OD SSA
Operator:	Tom Schaefer	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	S. Doyle - GZA	**Rig Type:**	ATV B-53	**Hammer Wt./Fall:**	140#/30
Date Start/Finish:	09-13-23/9-13-23	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NX
Boring Location:	Sta. 549+82.1, 6.8'L	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	16.0'
Hammer Efficiency Factor:	0.742/0.6	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			0'-1.0': Asphalt.	
	1D	24/11	1.0 - 3.0	37-20-18-13	38	38		144.3		1.0 Brown, dense, fine to coarse SAND, some gravel, little silt, (Fill).	23-S-4024 A-1-b, SP-SM WC=5.0%
	2D	24/6	3.0 - 5.0	3-2-2-4	4	4				Brown, moist, loose, fine to medium SAND, little gravel, trace silt, (Fill).	
5	3D	24/13	5.0 - 7.0	2-2-10-13	12	12				Top 6": Brown, moist, fine to medium SAND, little silt, trace gravel, (Fill). Bottom 7": Brown to light brown, medium dense, fine to medium SAND, little silt, little Gravel (Fill).	
							11				
							78				
							83				
							47			Intermittent increased roller bit resistance encountered from 9.0-11.0', probable cobbles.	
10							53				
	4D	24/7	11.0 - 13.0	3-1-1-2	2	2	15			Brown, wet, very loose, fine to coarse SAND, some gravel, little silt, (Fill).	23-S-4025 A-1-b, SM WC=13.6%
							14				
							21				
							23				
15							21			Increased roller bit resistance from approximately 15.0-16.0', probable cobbles or boulders.	
	5D	24/0	16.0 - 18.0	19-7-15-18	22	22	21	129.3		16.0 No Recovery. Rock in splitspoon tip; used 3" diameter spoon to get recovery.	
							83			5D: Brown, wet, medium dense, GRAVEL, some fine to medium Sand, trace Silt (Glacial Outwash).	
							86				
							207				
20	6D	24/6	20.0 - 22.0	26-10-11-7	21	26	139			Brown, wet, medium dense, fine to coarse GRAVEL, some fine to coarse sand, little silt, (Glacial Outwash). Casing met refusal at 20.5', probable cobble. Roller bit advanced to 23.5', probable top of rock. Set up to core at 23.5'.	23-S-4026 A-1-b, GW-GM WC=6.0%
							R/C				
	R1	60/60	23.5 - 28.5	RQD = 65%			NX	121.8		23.5 R1: 23.5-25.5': Hard, fresh, fine to medium grained, SCHIST.	
25										25.5-28.5', Hard, fresh to slightly weathered, fine to coarse grained,	

Remarks:

1. Automatic hammer NEBC #D20 with an energy transfer ratio = 0.742.
2. Due to automatic hammer malfunction used 300 lbs. hammer to drive casing to 15 feet below ground surface (bgs).
3. Automatic hammer fixed and used to drive casing from 15 feet bgs.
4. Due to automatic hammer malfunction used 140 lbs safety hammer with rope and cathead to drive splitspoon from 0 to 18 ft bgs. Automatic hammer fixed and used to drive splitspoon 20.0'-22.0' bgs.
5. Water level reading was taken immediately after drilling, after casing was removed.
6. As-drilled boring locations were surveyed by MaineDOT in the field (227572.8N, 923039.6E).

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: Day's Mill Bridge No. 2221 Location: Kennebunk / Arundel, Maine	Boring No.: BB-KAKR-101 WIN: 26226.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	145.3	**Auger ID/OD:**	4.25" OD SSA
Operator:	Tom Schaefer	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	S. Doyle - GZA	**Rig Type:**	ATV B-53	**Hammer Wt./Fall:**	140#/30
Date Start/Finish:	09-13-23/9-13-23	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NX
Boring Location:	Sta. 549+82.1, 6.8'L	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	16.0'
Hammer Efficiency Factor:	0.742	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							NX			GRANOFELS. Joints are very close to moderately spaced, low angle to moderately dipping, planer to undulating, rough to smooth, fresh to discolored, tight to moderately wide. Recovery = 100% Rock Quality = Fair Core times (min:sec): 23.5-24.5' (2:09), 24.5-25.5' (1:49), 25.5-26.5' (2:09), 26.5-27.5' (1:39), 21.5-28.5' (1:43)	
	R2	60/60	28.5 - 33.5	RQD = 80%						R2: 28.5'-28.9': Hard, fresh to slightly weathered, fine to coarse grained, GRANOFELS. 28.9'-33.5': Hard, fresh, fine to medium grained, SCHIST. Joints are very close to moderately spaced, moderately dipping to high angle, stepped to undulating, rough to smooth, fresh to decomposed, tight to open. Recovery = 100% Rock Quality = Good Core times (min:sec): 28.5-29.5' (1:38), 29.5-30.5' (1:44), 30.5-31.5' (1:48), 31.5-32.5' (1:44), 32.5-33.5' (1:46)	
30											
										Bottom of Exploration at 33.5 feet below ground surface.	
35											
40											
45											
50											

Remarks:

1. Automatic hammer NEBC #D20 with an energy transfer ratio = 0.742.
2. Due to automatic hammer malfunction used 300 lbs. hammer to drive casing to 15 feet below ground surface (bgs).
3. Automatic hammer fixed and used to drive casing from 15 feet bgs.
4. Due to automatic hammer malfunction used 140 lbs safety hammer with rope and cathead to drive splitspoon from 0 to 18 ft bgs. Automatic hammer fixed and used to drive splitspoon 20.0'-22.0' bgs.
5. Water level reading was taken immediately after drilling, after casing was removed.
6. As-drilled boring locations were surveyed by MaineDOT in the field (227572.8N, 923039.6E).

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: Day's Mill Bridge No. 2221
Location: Kennebunk / Arundel, Maine

Boring No.: BB-KAKR-102
WIN: 26226.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	144.5	**Auger ID/OD:**	4.25" OD SSA
Operator:	Tom Schaefer	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	S. Doyle - GZA	**Rig Type:**	ATV B-53	**Hammer Wt./Fall:**	140#/30
Date Start/Finish:	09-14-23/9-14-23	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NX
Boring Location:	Sta. 550+50.3, 6.9'L	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	14.2'
Hammer Efficiency Factor:	0.742	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			0'-1.0': Asphalt.	
	1D	24/12	1.0 - 3.0	19-24-20-23	44	54		143.5		1.0 Brown, dense, fine to coarse SAND, some fine gravel, trace silt, (Fill).	23-S-4027 A-1-b, SP-SM WC=3.9%
	2D	24/15	3.0 - 5.0	11-13-9-7	22	27				Brown, medium dense, fine to coarse SAND, little gravel, little silt, (Fill).	
5	3D	24/12	5.0 - 7.0	3-3-2-3	5	6				Brown, loose, fine to coarse SAND, little gravel, trace silt, (Fill).	23-S-4028 A-1-b, SW-SM WC=5.1%
							20				
							21				
							17				
							17				
10	4D	10/12	10.0 - 10.8	3-3-3-4	6	7	17			Brown, loose, fine to coarse SAND, little gravel, trace silt, (Fill).	23-S-4029 A-1-b, SW-SM WC=11.7%
							7				
							18				
							38				
							56/6"			Casing met refusal at 14.5'. Roller bit advancement from 14.5 to 16.0' indicate probable cobbles and boulders. Roller bit advanced smooth at 16.0', probable top of rock. Advanced roller bit to 16.5' and set up to core.	
15							Spin	128.5		16.0	
	R1	60/58	16.5 - 21.5	RQD=64%			NX			R1: Hard, slightly weathered, gray, fine grained, SCHIST, joints very close to close, moderately dipping, undulating, rough to smooth, fresh to discolored, tight to partially open. Recovery = 96% Rock Quality = Fair Core Times (min:sec): 16.5-17.5' (1:20), 17.5-18.5' (1:14), 18.5-19.5' (1:07), 19.5-20.5' (1:15), 20.5-21.5' (1:16)	
20											
	R2	60/60	21.5 - 26.5	RQD=81%						R2: Hard, fresh, gray, fine grained, SCHIST, joints very close to moderately spaced, moderately dipping, undulating, fresh to discolored, tight to partially open. Recovery = 100% Rock Quality: Good Core Times (min:sec): 21.5-22.5' (1:32), 22.5-23.5' (1:46), 23.5-24.5' (1:16), 24.5-25.5' (1:17), 25.5-26.5' (1:21)	
25											

Remarks:

1. Automatic hammer NEBC #D20 with an energy transfer ratio = 0.742.
2. Water level reading was taken immediately after removal of casing.
3 As-drilled boring locations were surveyed by MaineDOT in the field (227633.3N, 923019.5E).

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: Day's Mill Bridge No. 2221
Location: Kennebunk / Arundel, Maine

Boring No.: BB-KAKR-102
WIN: 26226.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	144.5	**Auger ID/OD:**	4.25" OD SSA
Operator:	Tom Schaefer	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	S. Doyle - GZA	**Rig Type:**	ATV B-53	**Hammer Wt./Fall:**	140#/30
Date Start/Finish:	09-14-23/9-14-23	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NX
Boring Location:	Sta. 550+50.3, 6.9'L	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	14.2'
Hammer Efficiency Factor:	0.742	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25											
										Bottom of Exploration at 26.5 feet below ground surface.	
30											
35											
40											
45											
50											

Remarks:

1. Automatic hammer NEBC #D20 with an energy transfer ratio = 0.742.
2. Water level reading was taken immediately after removal of casing.
3 As-drilled boring locations were surveyed by MaineDOT in the field (227633.3N, 923019.5E).

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: Day's Mill Bridge No. 2221
Location: Kennebunk / Arundel, Maine

Boring No.: BB-KAKR-201
WIN: 26226.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	142.0	**Auger ID/OD:**	4.25" OD SSA
Operator:	T. Schaefer	**Datum:**	NAVD88	**Sampler:**	Standard Splitspoon
Logged By:	J. Cozens	**Rig Type:**	ATV CME-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/11/24 - 4/11/24	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NX
Boring Location:	Sta. 549+65.6, 36.2'R	**Casing ID/OD:**	4.0/4.5"	**Water Level*:**	Not Encountered
Hammer Efficiency Factor:	0.742	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	2/6	0.0 - 0.2	1-1-3-3	4	5	SSA			Brown, moist, loose, fine to coarse SAND, trace silt, trace gravel, with grass and rootlets, (Fill).	
	2D	24/12	2.0 - 4.0	2-2-2-5	4	5		139.4		2D (Top 7"): Brown, moist, loose, fine to coarse SAND, trace silt, trace gravel, with grass and rootlets, (Fill). 2.6	#24-S-1352 A-4(0), SM MC=21.6%
										2D (Bottom 5"): Gray, loose, Silty fine to coarse SAND, little gravel, with rootlets, (Glacial Outwash).	
	3D	24/8	4.0 - 6.0	4-4-5-7	9	11				Brown, moist, loose, SAND & GRAVEL, little silt, (Glacial Outwash).	#24-S-1353 A-1-b, SM MC=8.8%
5											
							3				
							5				
							7				
							15/8"	132.2		Casing refusal at 9.8'. Roller coned to 10.5', and set up to core. 9.8	
10	R1	60/57	10.5 - 15.5	RQD = 33%			NX			R1 (10.5'-13.5'): Hard, slightly to moderately weathered, aphanitic to fine grained, gray, SCHIST. Primary joints are closely spaced, moderately dipping to vertical, planar, smooth, fresh to decomposed, tight to open, with silt and sand infilling.	
										R1 (13.5'-15.5'): Very hard, fresh to moderately weathered, medium grained, GRANOFELS. Joints are very close to closely spaced, horizontal to low angle, planar, rough to smooth, discolored to decomposed, tight to open, with silt and sand infilling.	q_p = 677 ksf
										Rock Quality = Poor	
										Recovry = 95%	
										Rock Core Times (min:sec): 10.5-11.5' (2.09), 11.5-12.5' (1:50), 12.5-13.5' (1:54), 13.5-14.5' (2:12), 14.5-15.5' (2:32)	
15	R2	60/55	15.5 - 20.5	RQD = 58%						R2: Very hard, fresh, fine to medium grained, grey to brown, SCHIST / GRANOFELS. Joints are very close to close, horizontal to low angle, planar, smooth to rough, fresh to discolored, tight to open. One high angle to vertical joint is undulating, rough, discolored, tight to open.	
										Rock Quality = Fair	
										Recovery = 93%	
										Rock Core Times (min:sec): 15.5-16.5' (1:59), 16.5-17.5' (1:50), 17.5-18.5' (1:43), 18.5-19.5' (1:21), 19.5-20.5' (1:39)	
20								121.5		20.5 **Bottom of Exploration at 20.5 feet below ground surface.**	
25											

Remarks:

1. Automatic hammer NEBC # D-230 with an energy transfer ratio = 0.742.
2. As-drilled boring locations were surveyed by MaineDOT in the field (227565.6N, 923088.3E).

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
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Boring No.: BB-KAKR-201

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Day's Mill Bridge No. 2221		**Boring No.:**	BB-KAKR-202
Location: Kennebunk / Arundel, Maine		**WIN:**	26226.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	144.5	**Auger ID/OD:**	4.25"
Operator:	T. Schaefer	**Datum:**	NAVD88	**Sampler:**	Standard Splitspoon
Logged By:	J. Cozens	**Rig Type:**	ATV CME-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/10/24 - 4/11/24	**Drilling Method:**	SSA and Drive & Wash	**Core Barrel:**	NX
Boring Location:	Sta. 549+68.0, 32.6'L	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Not Encountered
Hammer Efficiency Factor:	0.742	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:

D = Split Spoon Sample	R = Rock Core Sample	S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)	T_v = Pocket Torvane Shear Strength (psf)
MD = Unsuccessful Split Spoon Sample Attempt	SSA = Solid Stem Auger	$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)	WC = Water Content, percent
U = Thin Wall Tube Sample	HSA = Hollow Stem Auger	q_p = Unconfined Compressive Strength (ksf)	LL = Liquid Limit
MU = Unsuccessful Thin Wall Tube Sample Attempt	RC = Roller Cone	N-uncorrected = Raw Field SPT N-value	PL = Plastic Limit
V = Field Vane Shear Test, PP = Pocket Penetrometer	WOH = Weight of 140lb. Hammer	Hammer Efficiency Factor = Rig Specific Annual Calibration Value	PI = Plasticity Index
MV = Unsuccessful Field Vane Shear Test Attempt	WOR/C = Weight of Rods or Casing	N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency	G = Grain Size Analysis
	WO1P = Weight of One Person	N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected	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	24/14	0.0 - 2.0	WOH-WOH-10-10	10	12	SSA			1D (Top 6"): Black, loose, fine to coarse SAND, little silt, with rootlets, petroleum odor, (Fill). 1D (Bottom 8"): Brown, moist, medium dense, fine to coarse SAND, little gravel, petroleum odor, (Fill).	
	2D	24/18	2.0 - 4.0	8-8-8-9	16	20				Light brown, moist, medium dense, fine to coarse SAND, little silt, trace gravel, (Fill).	#24-S-1354 A-1-b, SW-SM MC=8.6%
	3D	24/4	4.0 - 6.0	5-11-10-7	21	26				Light brown, moist, medium dense, fine to coarse SAND, little silt, trace gravel, gravel blocked spoon, (Fill).	#24-S-1355 A-2-4(0), SM MC=8.2%
5							R/C				
10										Intermittent resistance from 9.0'-10.8', probable cobbles/boulder. Increased roller bit resistance at 10.8', probable top of rock.	
								133.7		10.8	
	R1	45/42	11.0 - 14.8	RQD = 0%			NX			Advanced roller bit from 10.8'-11.0' and set up to core. R1: Hard, fresh to moderately weathered, fine grained to aphanitic, grey, SCHIST. Joints are very close to close, low angle to moderately dipping, planar, smooth, fresh to disintegrated, tight to open, with sand and silt infilling. Rock Quality = Very Poor Recovery = 93% Rock Core Times (min:sec): 11.0-12.0' (1:32), 12.0-13.0' (1:19), 13.0-14.0' (1:23), 14.0-14.7' (1:42)	
15	R2	60/58	14.7 - 19.7	RQD = 78%						R2: Hard, fresh, aphanitic to fine grained, grey, SCHIST. Primary joints are very close to moderately spaced, moderately dipping to high angle, planar, smooth to rough, fresh to discolored, tight to open, with silt infilling. One low angle joint is planar, smooth, fresh to discolored, tight. Rock Quality = Good Recovery = 96% Rock Core Times (min:sec): 14.7-15.7' (1:00), 15.7-16.7' (1:05), 16.7-17.7' (1:14), 17.7-18.7' (1:26), 18.7-19.7' (1:39)	
20	R3	15/14	19.7 - 21.0	RQD = 95%				123.5		R3: Hard, fresh, aphanitic to fine grained, grey, SCHIST. Joints are close, moderately dipping, planar, smooth, fresh, tight. Rock Quality = Excellent Recovery = 99% Rock Core Times (min:sec): 19.7-20.7' (2:05), 20.7-21.0' (0:39)	
										21.0 **Bottom of Exploration at 21.0 feet below ground surface.**	
25											

Remarks:

1. Automatic hammer NEBC # D-230 with an energy transfer ratio = 0.742.
2. 300-lb hammer used to drive casing.
3. As-drilled boring locations were surveyed by MaineDOT in the field (227550.4N, 923021.3E).

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Day's Mill Bridge No. 2221
Location: Kennebunk / Arundel, Maine

Boring No.: BB-KAKR-203
WIN: 26226.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	143.9	**Auger ID/OD:**	4.25"
Operator:	T. Schaefer	**Datum:**	NAVD88	**Sampler:**	Standard Splitspoon
Logged By:	J. Cozens	**Rig Type:**	ATV CME-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/10/24 - 4/10/24	**Drilling Method:**	SSA and Drive & Wash	**Core Barrel:**	NX
Boring Location:	Sta. 551+10.3, 24.6'R	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	Not Encountered
Hammer Efficiency Factor:	0.742	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/13	0.0 - 2.0	2-6-6-5	12	15	SSA			Brown, moist, medium dense, gravelly fine to coarse SAND, trace silt, (Fill).	
	2D	24/12	2.0 - 4.0	6-11-10-7	21	26				2D (Top 6"): Brown, moist, medium dense, gravelly fine to coarse SAND, trace silt, (Fill). 2D (Bottom 6"): Light brown, moist, medium dense, fine to coarse SAND, little silt, (Fill).	#24-S-1356 A-2-4(0), SM MC=8.2%
	3D	24/17	4.0 - 6.0	3-3-3-3	6	7		139.9		4.0 Brown, moist, loose, fine to medium SAND, little silt, trace gravel, with rootlets, (Glacial Outwash).	#24-S-1357 A-4(0), SM MC=21.1%
5											
							R/C			Intermittent resistance from 7.0'-8.0', probable cobbles/boulder. Increased roller bit resistance at 8.0', probable top of rock. Set up to core at 8.0'.	
	R1	24/20	8.0 - 10.0	RQD = 0%			NX	135.9		8.0 R1: Hard, moderately weathered, fine to coarse grained, grey, SCHIST. Joints are close, moderately dipping, planar, smooth, fresh to discolored, tight to open, with silt and sand infilling. Rock Quality = Very Poor Recovery = 85% Rock Core Times (min:sec): 8.0-9.0' (1:31), 9.0-10.0' (1:40)	
10	R2	48/48	10.0 - 14.0	RQD = 29%						R2: Hard, moderately weathered, fine grained, grey, SCHIST. Primary joings are very close to close, moderately dipping, planar, smooth, fresh to discolored, tight to open, with silt infilling. Secondary joints are close, low angle, planar, smooth, discolored, tight. One vertical joint is planar, smooth, discolored, tight. Rock Quality = Poor Recovery = 100% Rock Core Times (min:sec): 10.0-11.0' (1:11), 11.0-12.0' (1:29), 12.0-13.0' (1:56), 13.0-14.0' (1:48)	
	R3	48/48	14.0 - 18.0	RQD = 89%						R3: Hard, fresh to slightly weathered, aphanitic to fine grained, grey, SCHIST /GRANOFELS. Joints are close, low angle to moderately dipping, planar, smooth to rough, fresh to discolored, tight to open. Rock Quality = Good Recovery = 100% Rock Core Times (min:sec): 14.0-15.0' (2:04), 15.0-16.0' (2:05), 16.0-17.0' (3:16), 17.0-18.0' (1:40)	q_p = 893 ksf
15											
								125.9		18.0 **Bottom of Exploration at 18.0 feet below ground surface.**	
20											
25											

Remarks:

1. Automatic hammer NEBC # D-230 with an energy transfer ratio = 0.742.
2. 300-lb hammer used to drive casing.
3. As-drilled boring locations were surveyed by MaineDOT in the field (227702.8N, 923026.1E).

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

Boring No.: BB-KAKR-203

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 cts.thielsch.com *Let's Build a Solid Foundation*	Client Information: GZA GeoEnvironmental South Portland, ME Project Manager: Blaine Cardali Assigned By: Blaine Cardali Collected By: B. Cardali	Project Information: **Days Mill Bridge No. 2221 Replacement, MEDOT WIN 26226.00** **Kennebunk, ME** Project Number: 09.0026198.00 Summary Page: 1 of 1 Report Date: 10.17.23

LABORATORY TESTING DATA SHEET, Report No.: 7423-K-117

Boring No.	Sample ID	Depth (ft)	Laboratory No.	Identification Tests: As Rcvd Moisture Content %	LL %	PL %	Gravel %	Sand %	Fines %	Org. %	pH	Proctor / CBR / Permeability Tests: g_d MAX (pcf) W_{opt} (%)	g_d MAX (pcf) W_{opt} (%) (Corr.)	Dry unit wt. (pcf)	Test Moisture Content %	Target Test Setup as % of Proctor	CBR @ 0.1"	CBR @ 0.2"	Permeability cm/sec	Laboratory Log and Soil Description
				D2216	D4318		D6913			D2974	D4792	D1557								
BB-KAKR-101	1D	1-3	23-S-4024	5.0			20.9	69.0	10.1											Brown f-c SAND, some fine Gravel, little Silt
BB-KAKR-101	4D	11-13	23-S-4025	13.6			20.2	65.0	14.8											Brown f-c SAND, some f-c Gravel, little Silt
BB-KAKR-101	6D	20-22	23-S-4026	6.0			59.7	29.2	11.1											Brown f-c GRAVEL, some f-c Sand, little Silt
BB-KAKR-102	1D	1-3	23-S-4027	3.9			20.8	69.2	10.0											Brown f-c SAND, some fine Gravel, trace Silt
BB-KAKR-102	3D	5-7	23-S-4028	5.1			13.9	79.3	6.8											Brown f-c SAND, little fine Gravel, trace Silt
BB-KAKR-102	4D	10-12	23-S-4029	11.7			13.4	76.9	9.7											Brown f-c SAND, little fine Gravel, trace Silt

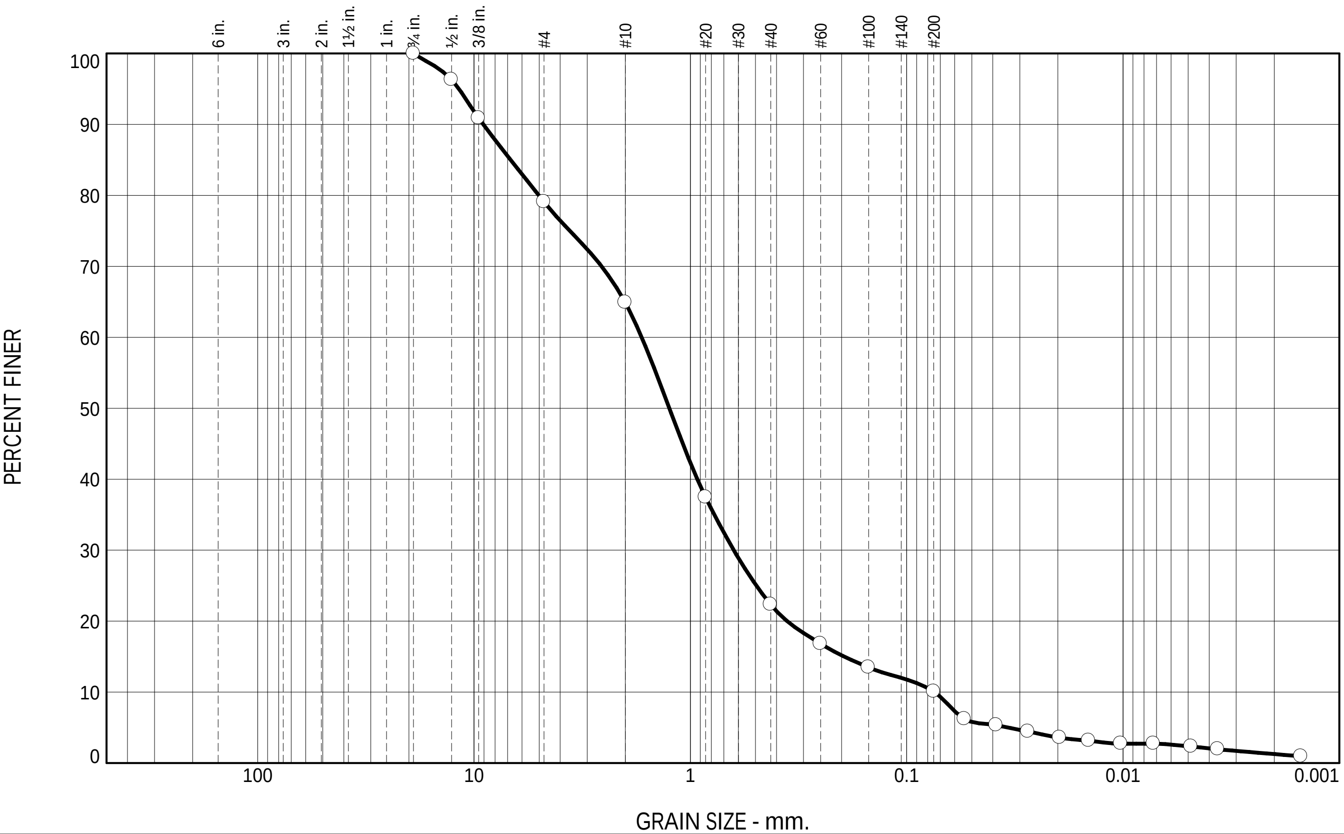
Date Received: **10.06.23**

Reviewed By:

Date Reviewed: **10.17.23**

These results are for the exclusive use of the client for whom they were obtained. This report only relates to items inspect and/or tested. No warranty, expressed or implied, is made.

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	20.9	14.2	42.6	12.2	8.8	1.3

SIEVE SIZE OR DIAMETER	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
1/2"	96.3		
3/8"	90.9		
#4	79.1		
#10	64.9		
#20	37.4		
#40	22.3		
#60	16.8		
#100	13.5		
#200	10.1		
0.0540 mm.	6.2		
0.0385 mm.	5.3		
0.0275 mm.	4.4		
0.0196 mm.	3.6		
0.0144 mm.	3.1		
0.0102 mm.	2.7		
0.0072 mm.	2.7		
0.0048 mm.	2.3		
0.0036 mm.	1.9		
0.0015 mm.	1.0		

* (no specification provided)

Soil Description

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

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 9.0591 D_{85}= 6.7713 D_{60}= 1.6790
D_{50}= 1.2512 D_{30}= 0.6301 D_{15}= 0.1943
D_{10}= 0.0744 C_u= 22.57 C_c= 3.18

Classification

USCS= SP-SM AASHTO= A-1-b

Remarks

Source of Sample: BB-KAKR-101 Depth: 1-3'
Sample Number: 1D

Date: 10.13.23

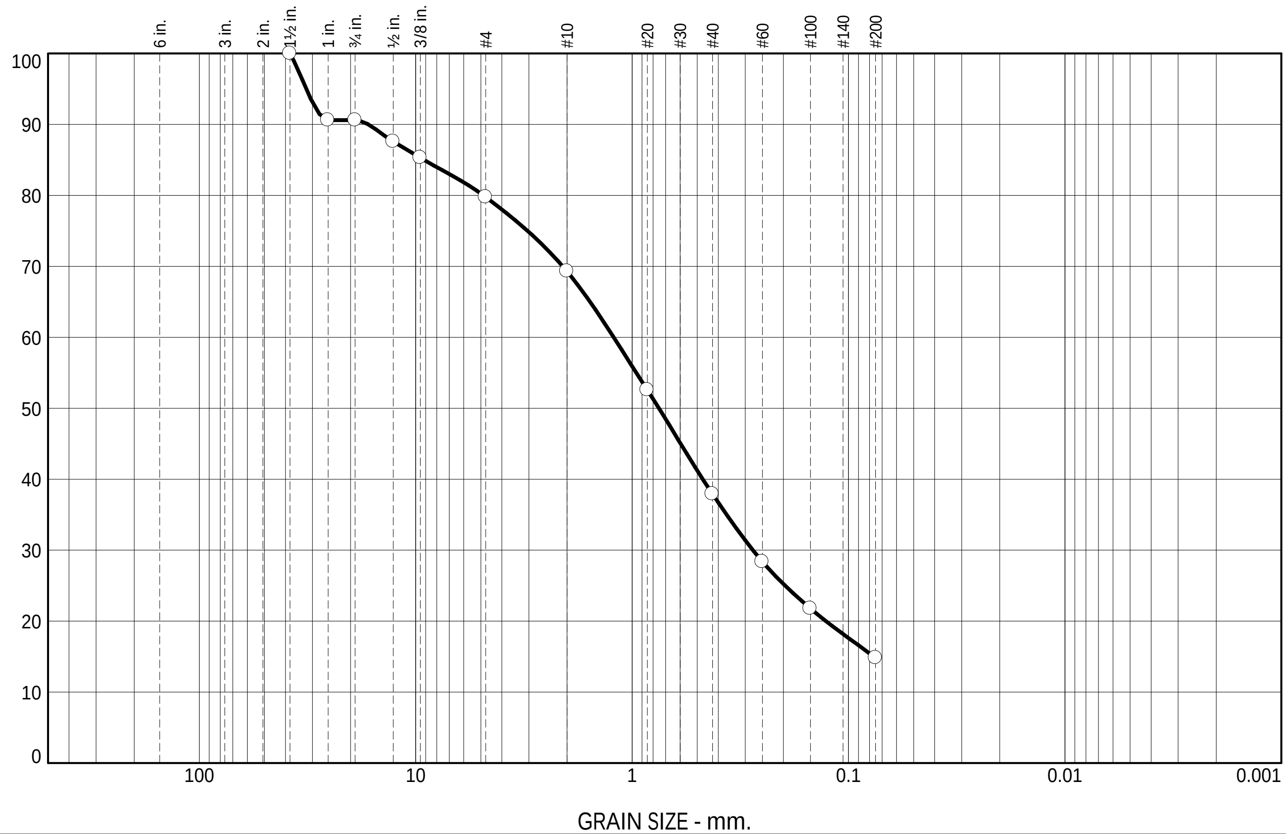
Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Days Mill Bridge No. 2221 Replacement
Kennebunk, ME
Project No: 09.0026198.00 Fig. 23-S-4024

Tested By: RB Checked By:

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	9.4	10.8	10.5	31.4	23.1	14.8	

SIEVE SIZE OR DIAMETER	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/2"	100.0		
1"	90.6		
3/4"	90.6		
1/2"	87.6		
3/8"	85.3		
#4	79.8		
#10	69.3		
#20	52.6		
#40	37.9		
#60	28.4		
#100	21.8		
#200	14.8		

* (no specification provided)

Soil Description

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

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 16.5282 D_{85}= 9.1587 D_{60}= 1.2170
D_{50}= 0.7518 D_{30}= 0.2768 D_{15}= 0.0764
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-1-b

Remarks

Source of Sample: BB-KAKR-101 Depth: 11-13'
Sample Number: 4D

Date: 10.12.23

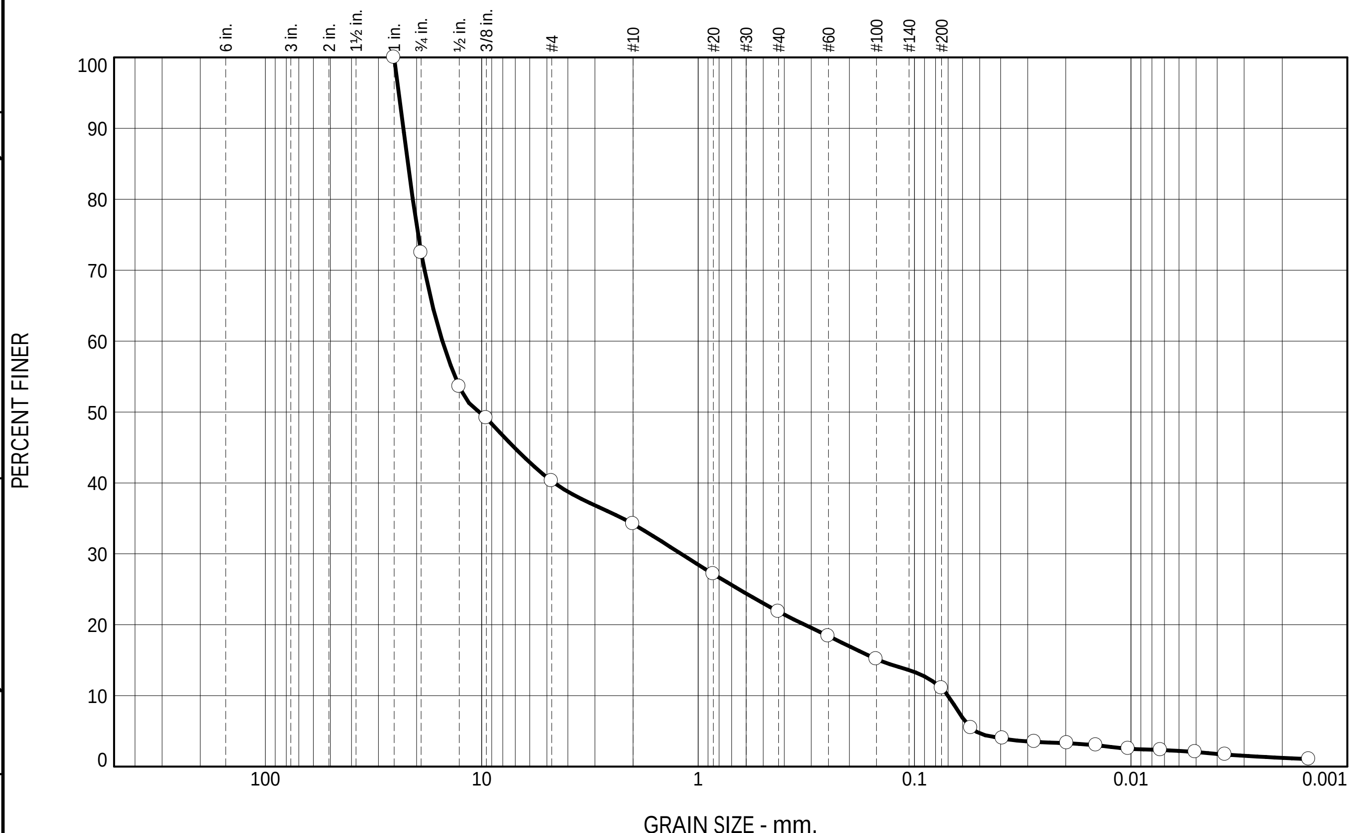
Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Days Mill Bridge No. 2221 Replacement
Kennebunk, ME
Project No: 09.0026198.00 Fig. 23-S-4025

Tested By: RB Checked By:

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	27.5	32.2	6.1	12.3	10.8	9.9	1.2

SIEVE SIZE OR DIAMETER	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1"	100.0		
3/4"	72.5		
1/2"	53.6		
3/8"	49.1		
#4	40.3		
#10	34.2		
#20	27.1		
#40	21.9		
#60	18.4		
#100	15.2		
#200	11.1		
0.0548 mm.	5.4		
0.0393 mm.	4.0		
0.0279 mm.	3.5		
0.0198 mm.	3.3		
0.0145 mm.	3.0		
0.0103 mm.	2.5		
0.0073 mm.	2.3		
0.0050 mm.	2.0		
0.0037 mm.	1.7		
0.0015 mm.	1.0		

* (no specification provided)

Soil Description

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

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 23.0112 D_{85}= 21.9216 D_{60}= 15.2015
D_{50}= 10.2489 D_{30}= 1.1989 D_{15}= 0.1462
D_{10}= 0.0700 C_u= 217.16 C_c= 1.35

Classification

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

Remarks

Source of Sample: BB-KAKR-101 Depth: 20-22'
Sample Number: 6D

Date: 10.13.23

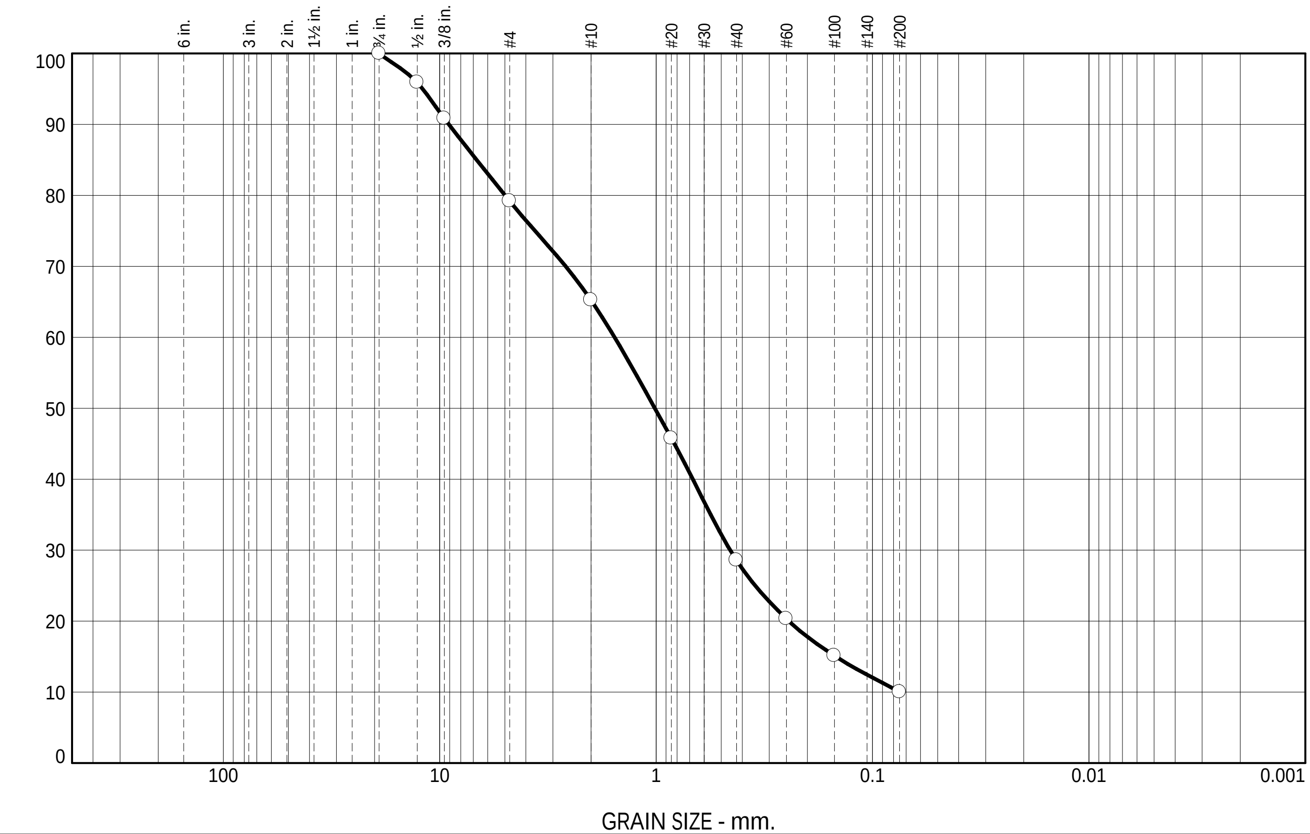
Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Days Mill Bridge No. 2221 Replacement
Kennebunk, ME
Project No: 09.0026198.00 Fig. 23-S-4026

Tested By: RB Checked By:

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	20.8	14.0	36.6	18.6	10.0	

SIEVE SIZE OR DIAMETER	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
1/2"	95.9		
3/8"	90.8		
#4	79.2		
#10	65.2		
#20	45.8		
#40	28.6		
#60	20.3		
#100	15.1		
#200	10.0		

* (no specification provided)

Soil Description

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

Atterberg Limits

PL= NP LL= NV PI=

Coefficients

D_{90}= 9.0738 D_{85}= 6.7465 D_{60}= 1.5561
D_{50}= 1.0139 D_{30}= 0.4549 D_{15}= 0.1478
D_{10}= C_u= C_c=

Classification

USCS= SP-SM AASHTO= A-1-b

Remarks

Source of Sample: BB-KAKR-102 Depth: 1-3'
Sample Number: 1D

Date: 10.12.23

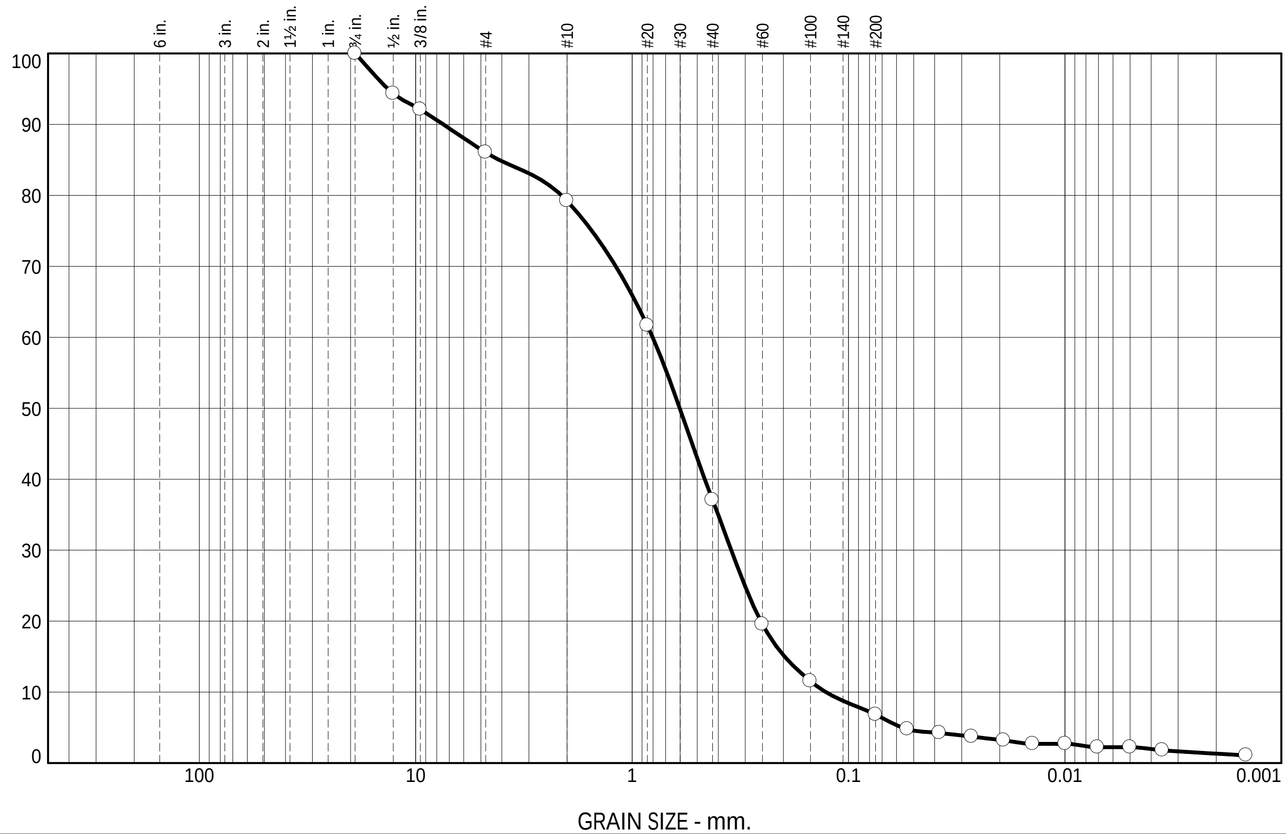
Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Days Mill Bridge No. 2221 Replacement
Kennebunk, ME
Project No: 09.0026198.00 Fig. 23-S-4027

Tested By: RB Checked By:

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	13.9	6.9	42.1	30.3	5.5	1.3

SIEVE SIZE OR DIAMETER	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
1/2"	94.4		
3/8"	92.1		
#4	86.1		
#10	79.2		
#20	61.7		
#40	37.1		
#60	19.6		
#100	11.5		
#200	6.8		
0.0535 mm.	4.8		
0.0380 mm.	4.2		
0.0270 mm.	3.7		
0.0192 mm.	3.2		
0.0141 mm.	2.7		
0.0099 mm.	2.7		
0.0071 mm.	2.2		
0.0050 mm.	2.2		
0.0035 mm.	1.8		
0.0015 mm.	1.1		

* (no specification provided)

Soil Description

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

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 7.4517 D_{85}= 4.1101 D_{60}= 0.8032
D_{50}= 0.6026 D_{30}= 0.3483 D_{15}= 0.1974
D_{10}= 0.1274 C_u= 6.30 C_c= 1.19

Classification

USCS= SW-SM AASHTO= A-1-b

Remarks

Source of Sample: BB-KAKR-102 Depth: 5-7'
Sample Number: 3D
Date: 10.13.23

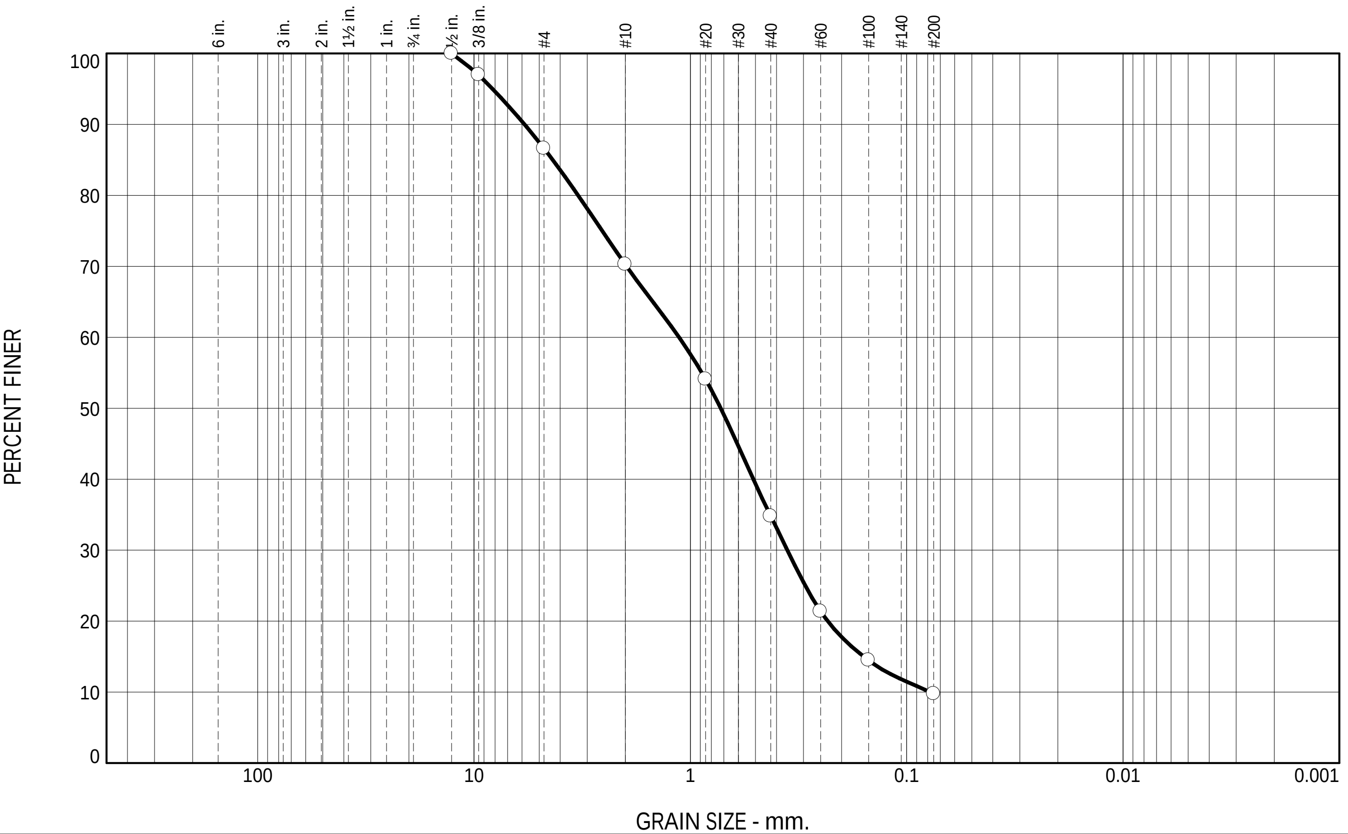
Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Days Mill Bridge No. 2221 Replacement
Kennebunk, ME
Project No: 09.0026198.00 Fig. 23-S-4028

Tested By: RB Checked By:

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	13.4	16.3	35.5	25.1	9.7	

SIEVE SIZE OR DIAMETER	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2"	100.0		
3/8"	97.0		
#4	86.6		
#10	70.3		
#20	54.1		
#40	34.8		
#60	21.4		
#100	14.5		
#200	9.7		

* (no specification provided)

Soil Description

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

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 5.8432 D_{85}= 4.3255 D_{60}= 1.1284
D_{50}= 0.7241 D_{30}= 0.3563 D_{15}= 0.1578
D_{10}= 0.0781 C_u= 14.44 C_c= 1.44

Classification

USCS= SW-SM AASHTO= A-1-b

Remarks

Source of Sample: BB-KAKR-102 Depth: 10-12'
Sample Number: 4D
Date: 10.12.23

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Days Mill Bridge No. 2221 Replacement
Kennebunk, ME
Project No: 09.0026198.00 Fig. 23-S-4029

Tested By: RB Checked By:

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 Project Manager: B. Cardali Assigned By: B. Cardali Collected By: B. Cardali	Project Information: **Days Mill Bridge No. 2221 Replacement** **Kennebunk, ME** Project Number: 09.0026198.00 Summary Page: 1 of 1 Report Date: 10.17.23

LABORATORY TESTING DATA SHEET, Report No.: 7423-K-116

Boring No.	Sample No.	Depth (ft/in)	Laboratory No.	Specimen Data: Mohs Hard-ness	Diameter (in)	Length (in)	(1) Unit Weight (PCF)	(2) Wet Density (PCF)	Bulk G_s	Compressive Strength Tests: (3) Other Tests	(4) Strength PSI	(5) Strain %	(6) E sec PSI EE+06	(7) Poisson's Ratio	st PSI	Is_{50} PSI	(8) s_c PSI	Rock Formation or Description or Remarks
BB-KAKR-101	R1	23.7-24.4	23-S-4021		1.978	4.062	173.9				3201	0.258	1.18	0.03				Grey Schist
Fresh Break along foliation																		
BB-KAKR-101	R1	25.8-26.3	23-S-4022		1.982	4.452	165.9				5138	0.354	1.49	0.13				Grey Schist
Fresh Break along foliation																		
BB-KAKR-102	R1	20.6-21.1	23-S-4023		1.978	4.743	173.8				4070	0.139	2.45	0.42				Grey Schist
Fresh Break along foliation, minor break at 0.32psi indicates the calculation of Poisson's Ratio is high																		

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

Notes:

(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)

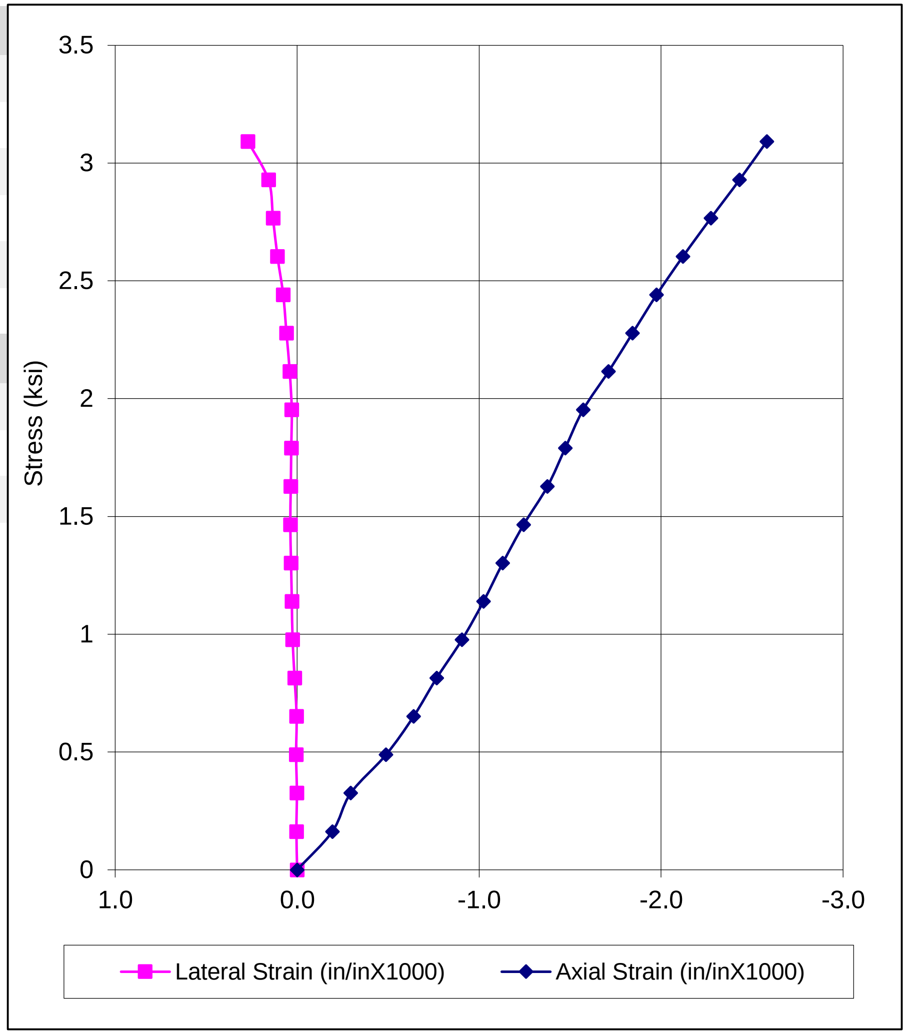
Date Received: **10.06.23** Reviewed By: Date Review **10.17.23**

Thielsch DIVISION OF THE RISE GROUP	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 Project Manager: B. Cardali Assigned by: B. Cardali Collected by: B. Cardali	**Project Information:** Days Mill Bridge No. 2221 Replacement Kennebunk, ME Project Number: 09.0026198.00 Technician: KW Report Date: 10.17.23

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

Sample Information		Compressive Test Information	
Boring ID:	BB-KAKR-101	**Unit Weight (pcf):**	173.9
Sample #:	R1	**Failure Stress (psi):**	3,201
Depth (ft):	23.7-24.4	**Failure Mode:**	Fresh
Tested Depth (ft):	24.05-24.38	**Time to Failure (min)**	1.73
Rock Type:	Grey Schist		
Features:	Fresh Break along foliation		

Test Specimen Information		Elastic Moduli Test Information	
Diameter, D (in):	1.978	**Poisson's Ratio @ 50%:**	0.03
Length, L (in):	4.062	**Strain %:**	0.258
L:D Ratio:	2.05	**E sec PSI @ 50%:**	1.18E+06



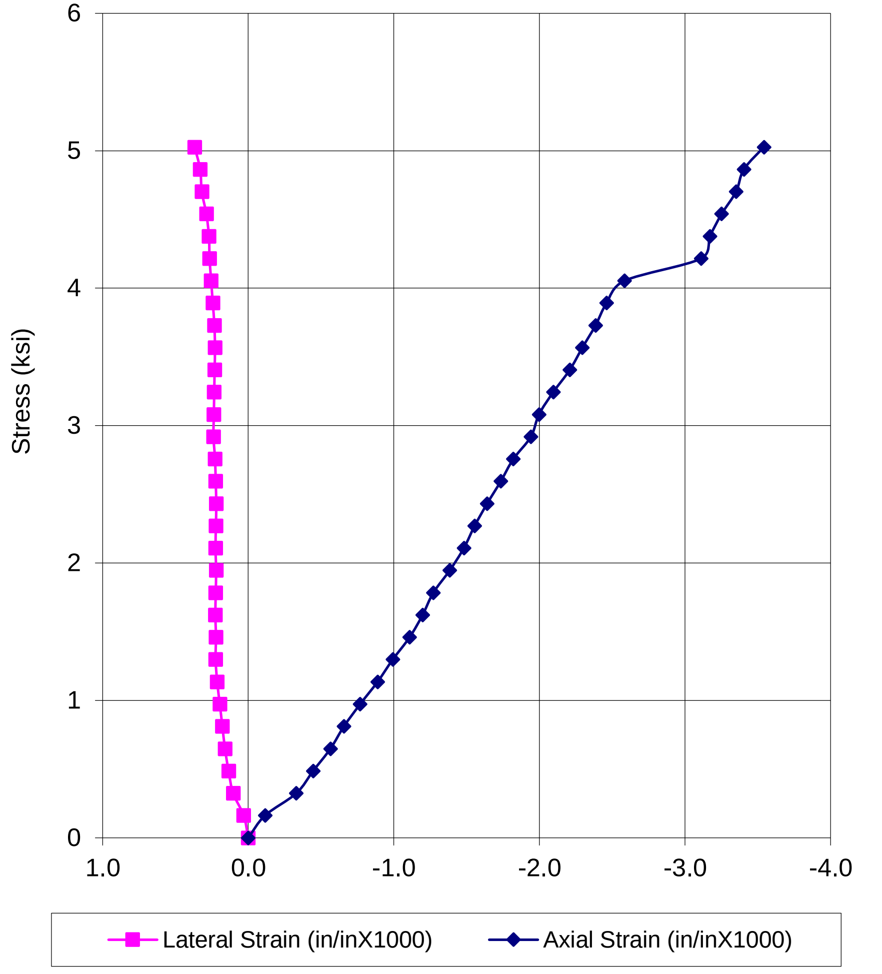
Testing Notes:

Thielsch DIVISION OF THE RISE GROUP	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 Project Manager: B. Cardali Assigned by: B. Cardali Collected by: B. Cardali	**Project Information:** Days Mill Bridge No. 2221 Replacement Kennebunk, ME Project Number: 09.0026198.00 Technician: KW Report Date: 10.17.23

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

Sample Information		Compressive Test Information	
Boring ID:	BB-KAKR-101	**Unit Weight (pcf):**	165.9
Sample #:	R1	**Failure Stress (psi):**	5,138
Depth (ft):	25.8-26.3	**Failure Mode:**	Fresh
Tested Depth (ft):	25.8-26.18	**Time to Failure (min)**	2.47
Rock Type:	Grey Schist		
Features:	Fresh Break along foliation		

Test Specimen Information		Elastic Moduli Test Information	
Diameter, D (in):	1.982	**Poisson's Ratio @ 50%:**	0.13
Length, L (in):	4.452	**Strain %:**	0.354
L:D Ratio:	2.25	**E sec PSI @ 50%:**	1.49E+06



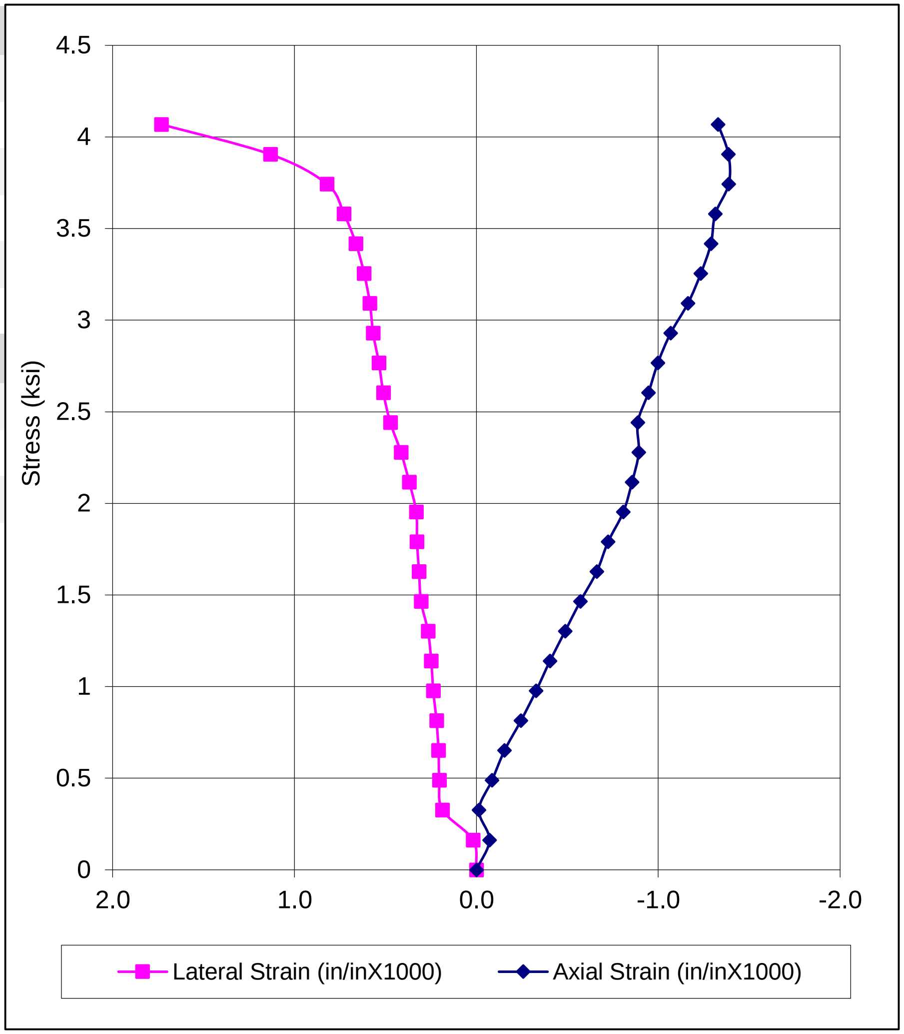
Testing Notes: **Minor break at about 4054 psi**

Thielsch DIVISION OF THE RISE GROUP	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 Project Manager: B. Cardali Assigned by: B. Cardali Collected by: B. Cardali	**Project Information:** Days Mill Bridge No. 2221 Replacement Kennebunk, ME Project Number: 09.0026198.00 Technician: KW Report Date: 10.17.23

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

Sample Information		Compressive Test Information	
Boring ID:	BB-KAKR-102	**Unit Weight (pcf):**	173.8
Sample #:	R1	**Failure Stress (psi):**	4,070
Depth (ft):	20.6-21.1	**Failure Mode:**	Fresh
Tested Depth (ft):	20.65-21.3	**Time to Failure (min)**	1.92
Rock Type:	Grey Schist		
Features:	Fresh Break along foliation		

Test Specimen Information		Elastic Moduli Test Information	
Diameter, D (in):	1.978	**Poisson's Ratio @ 50%:**	0.42
Length, L (in):	4.743	**Strain %:**	0.139
L:D Ratio:	2.40	**E sec PSI @ 50%:**	2.45E+06



Testing Notes: Minor break occurred at about 0.32psi indicating the calcuated Poissons Ration is too high

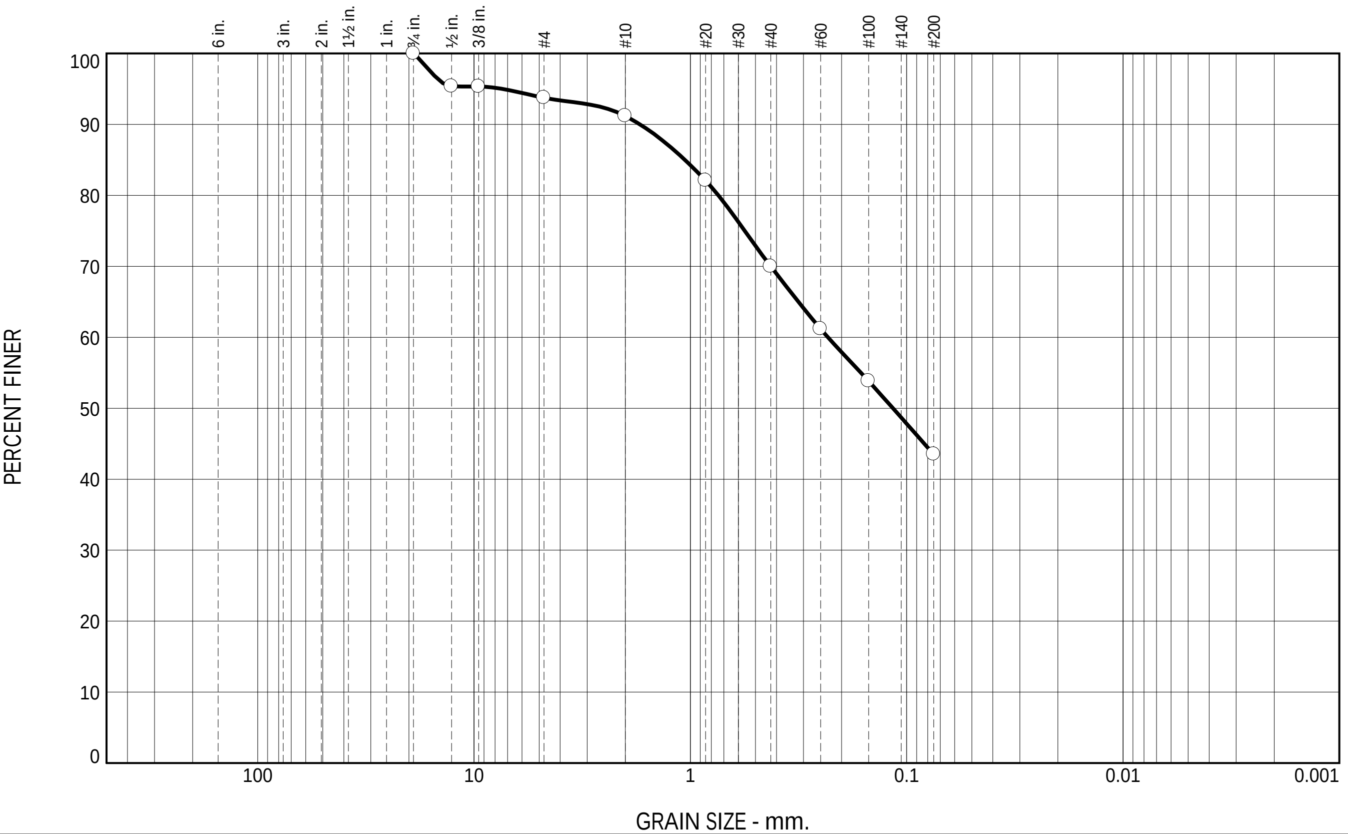
Thielsch DIVISION OF THE RISE GROUP	195 Frances Avenue Cranston RI, 02910 Phone: (401)-467-6454 Fax: (401)-467-2398 cts.thielsch.com *Let's Build a Solid Foundation*	Client Information: GZA GeoEnvironmental South Portland, ME Project Manager: John Cozens Assigned By: John Cozens Collected By: Client	Project Information: **Days Mills Bridge #2221 Replacement** **Bridge #2221 over Kennebuck River** Project Number: 09.0026198.01 Summary Page: 1 of 1 Report Date: 04.23.24

LABORATORY TESTING DATA SHEET, Report No.: 7424-D-180

Material Source	Sample ID	Depth (ft)	Laboratory No.	Identification Tests									Proctor / CBR / Permeability Tests								Laboratory Log and Soil Description
				As Rcvd Moisture Content %	LL %	PL %	OD LL	Gravel %	Sand %	Fines %	Org. %	pH	g_d MAX (pcf) W_{opt} (%)	g_d MAX (pcf) W_{opt} (%) (Corr.)	Dry unit wt. (pcf)	Test Moisture Content %	Target Test Setup as % of Proctor	CBR @ 0.1"	CBR @ 0.2"	Permeability cm/sec	
				D2216	D4318			D6913			D2974	D4792	D1557								
BB-KAKR-201	2D	2.0-4.0	24-S-1352	21.6				6.2	50.3	43.5											Brown f-m SAND and CLAYEY SILT, trace fine Gravel
BB-KAKR-201	3D	4.0-6.0	24-S-1353	8.8				39.0	46.2	14.8											Brown f-c SAND and fine GRAVEL, little Silt
BB-KAKR-202	2D	2.0-4.0	24-S-1354	8.6				32.7	58.3	9.0											Dark Brown f-c SAND, soem f-c Gravel, trace Silt
BB-KAKR-202	3D	4.0-6.0	24-S-1355	8.2				32.8	39.9	27.3											Brown f-c SAND, some f-c Gravel, some Silt
BB-KAKR-203	2D	2.0-4.0	24-S-1356	12.6				9.2	58.3	32.5											Brown f-m SAND, some Silt, trace f-c Gravel
BB-KAKR-203	3D	4.0-6.0	24-S-1357	21.1				4.9	51.6	43.5											Brown f-m SAND and SILT, trace fine Gravel

Date Received: **04.17.24** Reviewed By: Date Reviewed: **04.23.24**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	6.2	2.6	21.2	26.5	43.5	

SIEVE SIZE OR DIAMETER	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
1/2"	95.4		
3/8"	95.3		
#4	93.8		
#10	91.2		
#20	82.1		
#40	70.0		
#60	61.2		
#100	53.8		
#200	43.5		

* (no specification provided)

Soil Description

Brown f-m SAND and CLAYEY SILT, trace fine Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 1.7088 D_{85}= 1.0595 D_{60}= 0.2313
D_{50}= 0.1158 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-4(0)

Remarks

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

Source of Sample: Borings Depth: 2.0-4.0'
Sample Number: BB-KAKR-201 / 2D

Date: 04.22.24

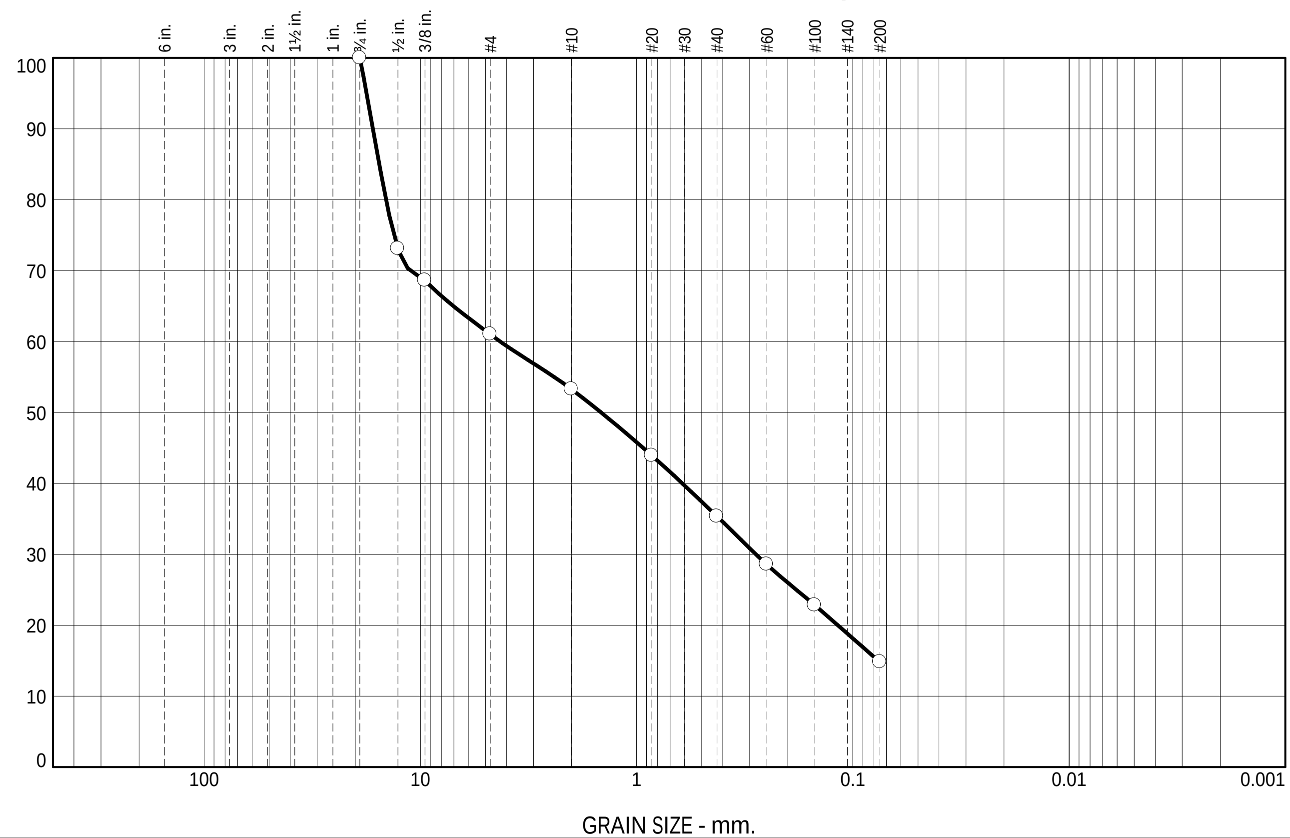
Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Days Mills Bridge #2221 Replacement
Kennebuck, ME
Project No: 09.0026198.01 Fig. 24-S-1352

Tested By: MCS Checked By: Rebecca Roth

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	39.0	7.7	17.9	20.6	14.8	

SIEVE SIZE OR DIAMETER	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
1/2"	73.1		
3/8"	68.6		
#4	61.0		
#10	53.3		
#20	43.9		
#40	35.4		
#60	28.6		
#100	22.8		
#200	14.8		

* (no specification provided)

Soil Description

Brown f-c SAND and fine GRAVEL, little Silt

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 16.5798 D_{85}= 15.4954 D_{60}= 4.2679
D_{50}= 1.4586 D_{30}= 0.2808 D_{15}= 0.0763
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-1-b

Remarks

Source of Sample: Borings Depth: 4.0-6.0'
Sample Number: BB-KAKR-201 / 3D

Date: 04.22.24

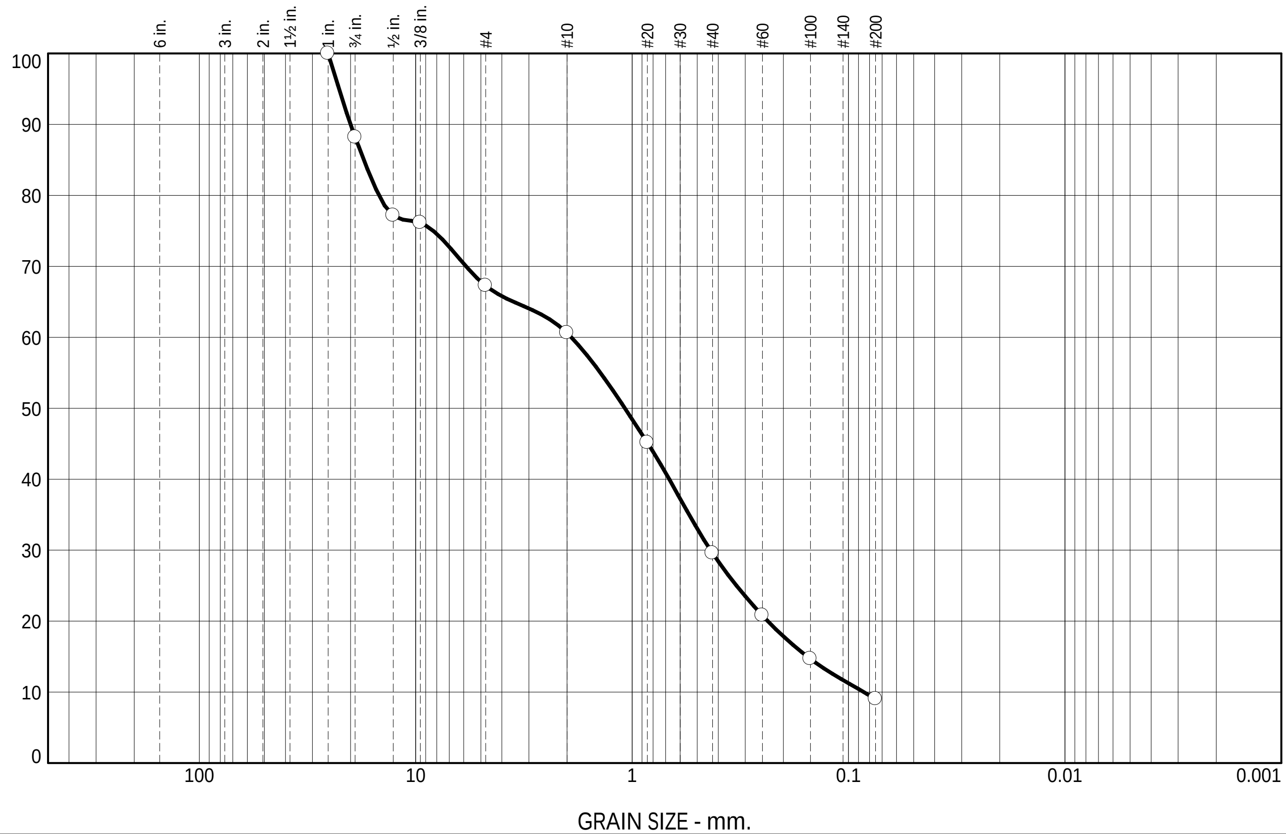
Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Days Mills Bridge #2221 Replacement
Kennebuck, ME
Project No: 09.0026198.01 Fig. 24-S-1353

Tested By: MCS Checked By: Rebecca Roth

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	11.8	20.9	6.7	31.0	20.6	9.0	

SIEVE SIZE OR DIAMETER	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1"	100.0		
3/4"	88.2		
1/2"	77.2		
3/8"	76.1		
#4	67.3		
#10	60.6		
#20	45.1		
#40	29.6		
#60	20.8		
#100	14.7		
#200	9.0		

* (no specification provided)

Soil Description

Dark Brown f-c SAND, soem f-c Gravel, trace Silt

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 20.0256 D_{85}= 17.3665 D_{60}= 1.9102
D_{50}= 1.0802 D_{30}= 0.4343 D_{15}= 0.1548
D_{10}= 0.0850 C_u= 22.48 C_c= 1.16

Classification

USCS= SW-SM AASHTO= A-1-b

Remarks

Source of Sample: Borings Depth: 2.0-4.0'
Sample Number: BB-KAKR-202 / 2D

Date: 04.22.24

Thielsch Engineering Inc.

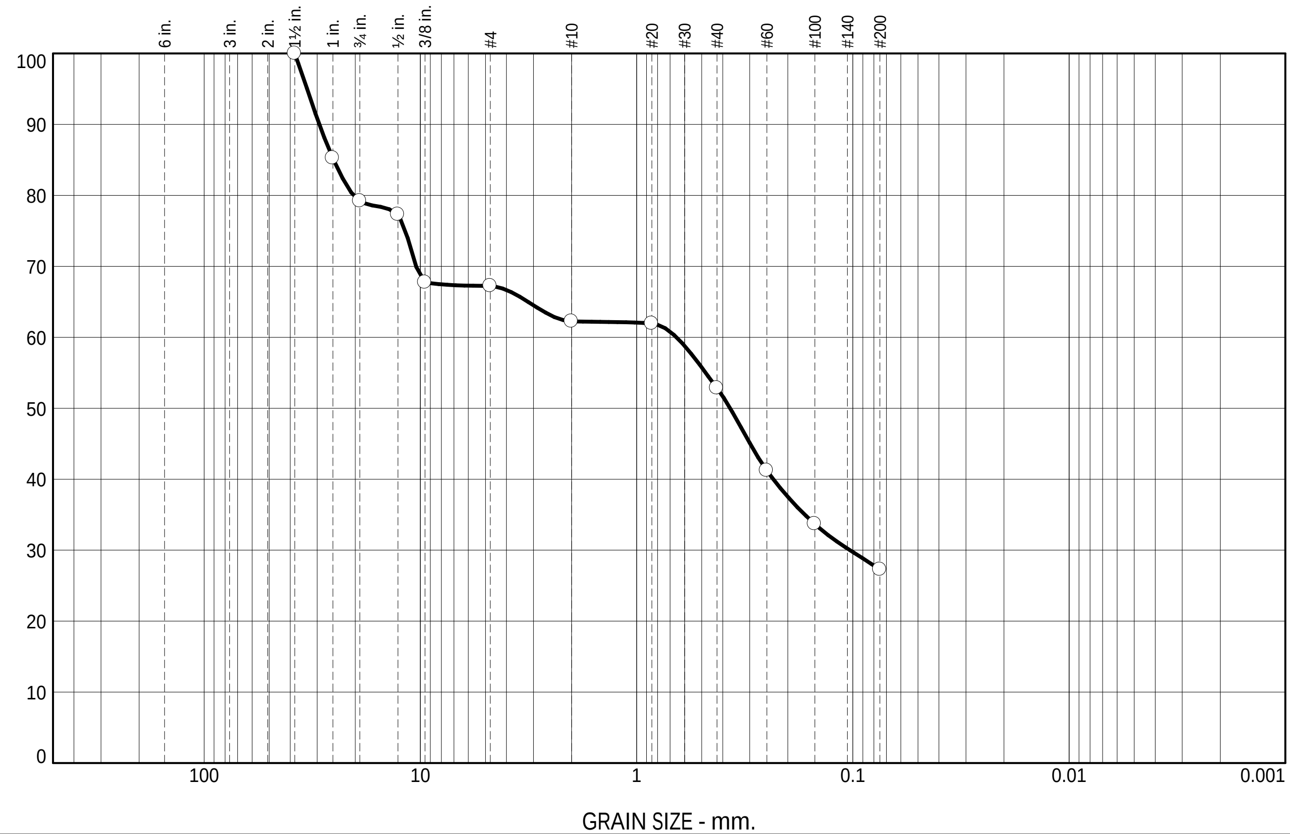
Cranston, RI

Client: GZA GeoEnvironmental
Project: Days Mills Bridge #2221 Replacement
Kennebuck, ME
Project No: 09.0026198.01 Fig. 24-S-1354

Tested By: MCS Checked By: Rebecca Roth

These results are for the exclusive use of the client for whom they were obtained. This report only relates to items inspect and/or tested. No warranty, expressed or implied, is made.

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	20.8	12.0	4.9	9.5	25.5	27.3	

SIEVE SIZE OR DIAMETER	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/2"	100.0		
1"	85.2		
3/4"	79.2		
1/2"	77.3		
3/8"	67.7		
#4	67.2		
#10	62.3		
#20	61.9		
#40	52.8		
#60	41.2		
#100	33.7		
#200	27.3		

* (no specification provided)

Soil Description

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

Atterberg Limits

PL= NP   LL= NV   PI= NP

Coefficients

D_{90}= 29.3148 D_{85}= 25.1858 D_{60}= 0.6537
D_{50}= 0.3695 D_{30}= 0.1032 D_{15}=
D_{10}= C_u= C_c=

Classification

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

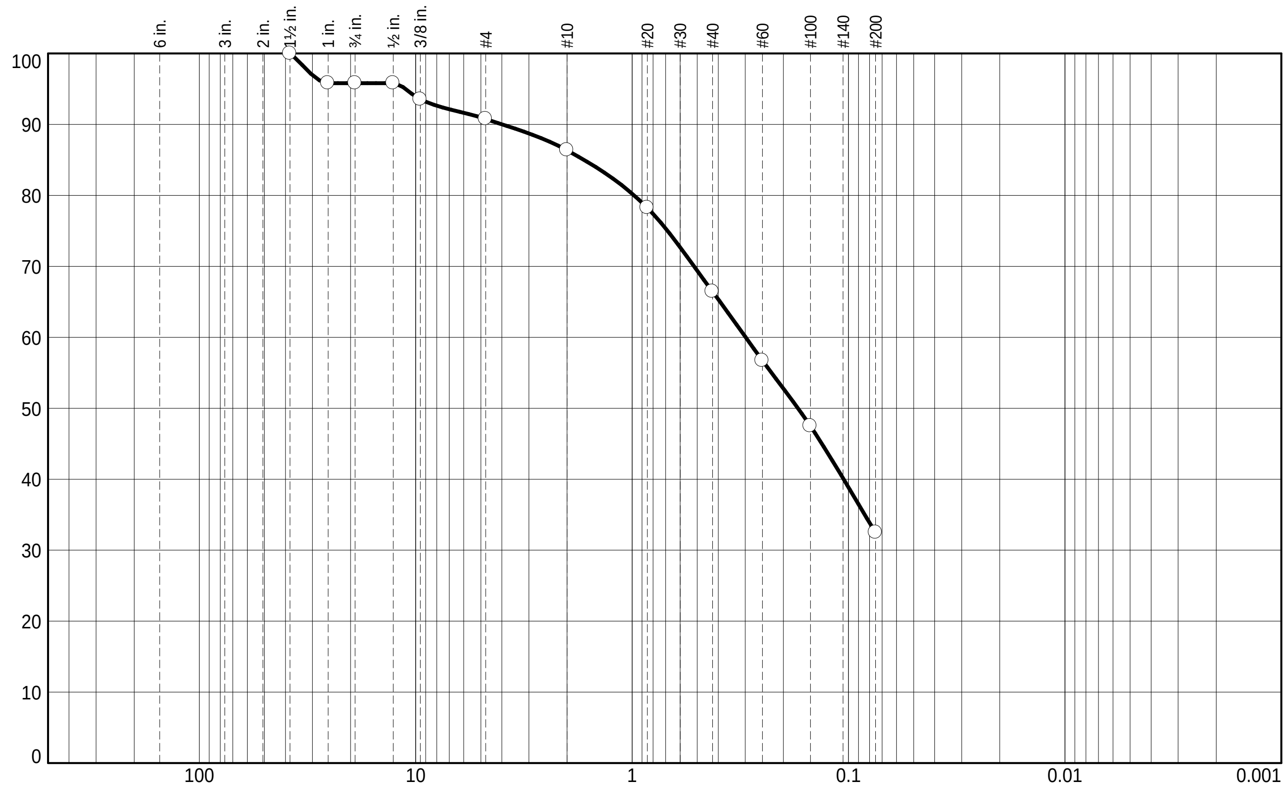
Remarks

Source of Sample: Borings   Depth: 4.0-6.0'
Sample Number: BB-KAKR-202 / 3D   Date: 04.22.24

Thielsch Engineering Inc. Cranston, RI	Client: GZA GeoEnvironmental Project: Days Mills Bridge #2221 Replacement Kennebuck, ME Project No: 09.0026198.01 Fig. 24-S-1355

Tested By: MCS   Checked By: Rebecca Roth

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	4.2	5.0	4.4	20.0	33.9	32.5	

SIEVE SIZE OR DIAMETER	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/2"	100.0		
1"	95.8		
3/4"	95.8		
1/2"	95.8		
3/8"	93.5		
#4	90.8		
#10	86.4		
#20	78.2		
#40	66.4		
#60	56.7		
#100	47.5		
#200	32.5		

* (no specification provided)

Soil Description

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

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 3.9860 D_{85}= 1.6678 D_{60}= 0.2988
D_{50}= 0.1712 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

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

Remarks

Sample visually classified as non-plastic.

Source of Sample: Borings Depth: 2.0-4.0'
Sample Number: BB-KAKR-203 / 2D

Date: 04.22.24

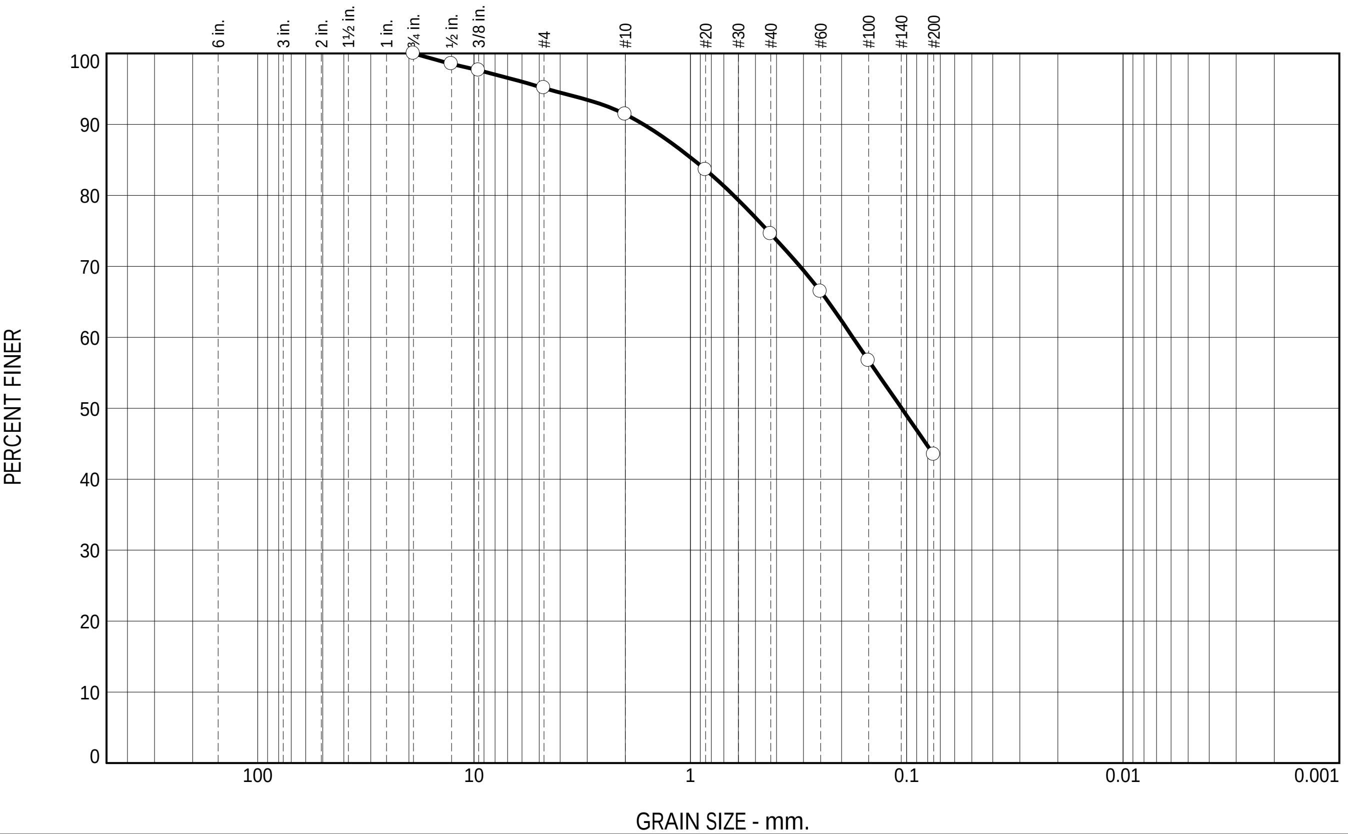
Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Days Mills Bridge #2221 Replacement
Kennebuck, ME
Project No: 09.0026198.01 Fig. 24-S-1356

Tested By: MCS Checked By: Rebecca Roth

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	4.9	3.7	16.8	31.1	43.5	

SIEVE SIZE OR DIAMETER	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
1/2"	98.5		
3/8"	97.6		
#4	95.1		
#10	91.4		
#20	83.6		
#40	74.6		
#60	66.4		
#100	56.7		
#200	43.5		

* (no specification provided)

Soil Description

Brown f-m SAND and SILT, trace fine Gravel

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 1.6464 D_{85}= 0.9667 D_{60}= 0.1776
D_{50}= 0.1056 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-4(0)

Remarks

Sample visually classified as non-plastic.

Source of Sample: Borings Depth: 4.0-6.0'
Sample Number: BB-KAKR-203 / 3D

Date: 04.22.24

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Days Mills Bridge #2221 Replacement
Kennebuck, ME
Project No: 09.0026198.01 Fig. 24-S-1357

Tested By: MCS Checked By: Rebecca Roth

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 Project Manager: John Cozens Assigned By: John Cozens Collected By: Client	Project Information: **Days Mills Bridge #2221 Replacement** **Bridge #2221 over Kennebuck River** Project Number: 09.0026198.01 Summary Page: 1 of 1 Report Date: 04.24.24

LABORATORY TESTING DATA SHEET, Report No.: 7424-D-179

Boring No.	Sample No.	Depth (ft)	Laboratory No.	Specimen Data: Mohs Hard-ness	Diameter (in)	Length (in)	(1) Unit Weight (PCF)	(2) Wet Density (PCF)	Bulk G_s	Compressive Strength Tests: (3) Other Tests	(4) Strength PSI	(5) Strain %	(6) E sec PSI EE+06	(7) Poisson's Ratio	st PSI	Is_{50} PSI	(8) s_c PSI	Rock Formation or Description or Remarks
BB-KAKR-201	1R	2'10"-3'7"	24-S-1350		1.983	4.518	163.6				4766	0.165	3.09	0.30				Grey Granite and Slate
Fresh Break																		
BB-KAKR-203	2R	2'7"-3'2"																
Sample broke in transit																		
BB-KAKR-203	3R	6'10"-7'8"	24-S-1351		1.981	4.535	174.1				6286	0.121	4.56	0.13				Grey Slate
Sample broke along foliation																		

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

Notes

(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

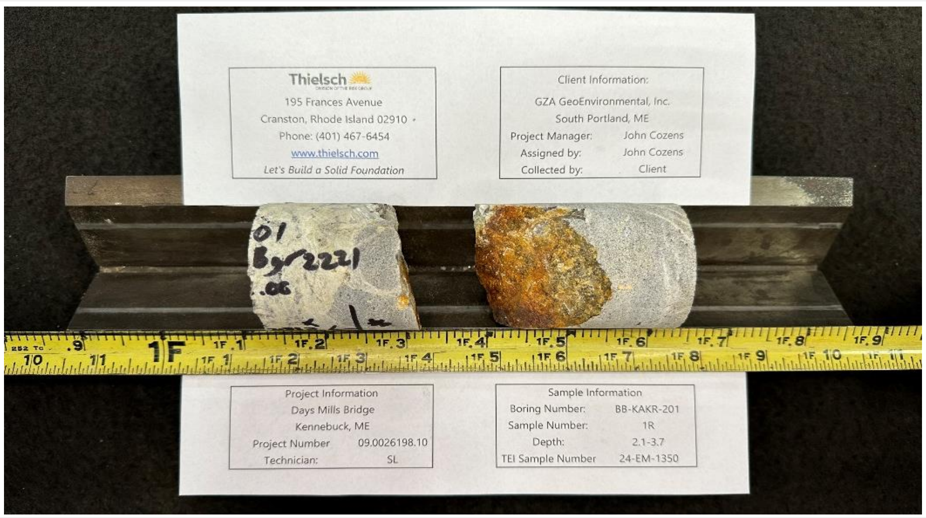
(8) Estimated UCS from Table 1 of ASTM D5731 for NX cores (Is x 24)

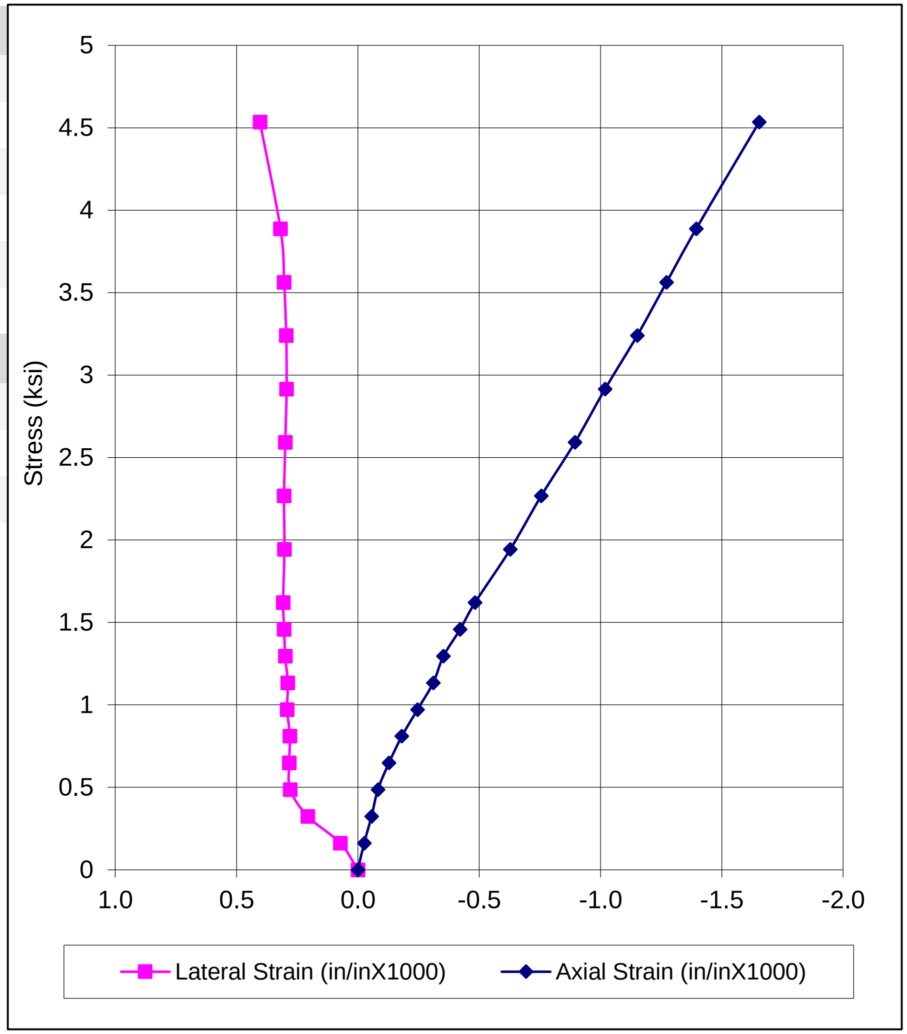
Date Received: **04.17.24** Reviewed By: Date Review **04.24.24**

Thielsch DIVISION OF THE RISE GROUP	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 Project Manager: John Cozens Assigned by: John Cozens Collected by: Client	**Project Information:** Days Mills Bridge #2221 Replacement Kennebuck,ME Project Number: 09.0026198.10 Technician: SL Report Date: 04.24.24

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

Sample Information		Compressive Test Information	
Boring ID:	BB-KAKR-201	**Unit Weight (pcf):**	163.6
Sample #:	1R	**Failure Stress (psi):**	4,766
Depth (ft):	2.1-3.7	**Failure Mode:**	Fresh
Tested Depth (ft):	3.15-3.55	**Time to Failure (min)**	2.82
Rock Type:	Grey Granite and Slate		
Features:	Fresh Break		
Test Specimen Information		**Elastic Moduli Test Information**	
Diameter, D (in):	1.983	**Poisson's Ratio @ 50%:**	0.30
Length, L (in):	4.518	**Strain %:**	0.165
L:D Ratio:	2.28	**E sec PSI @ 50%:**	3.09E+06





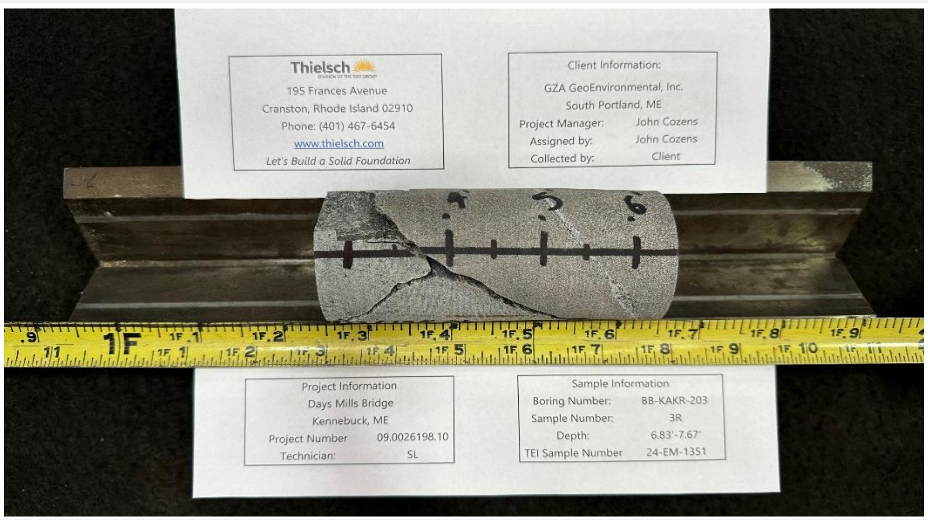
Testing Notes:

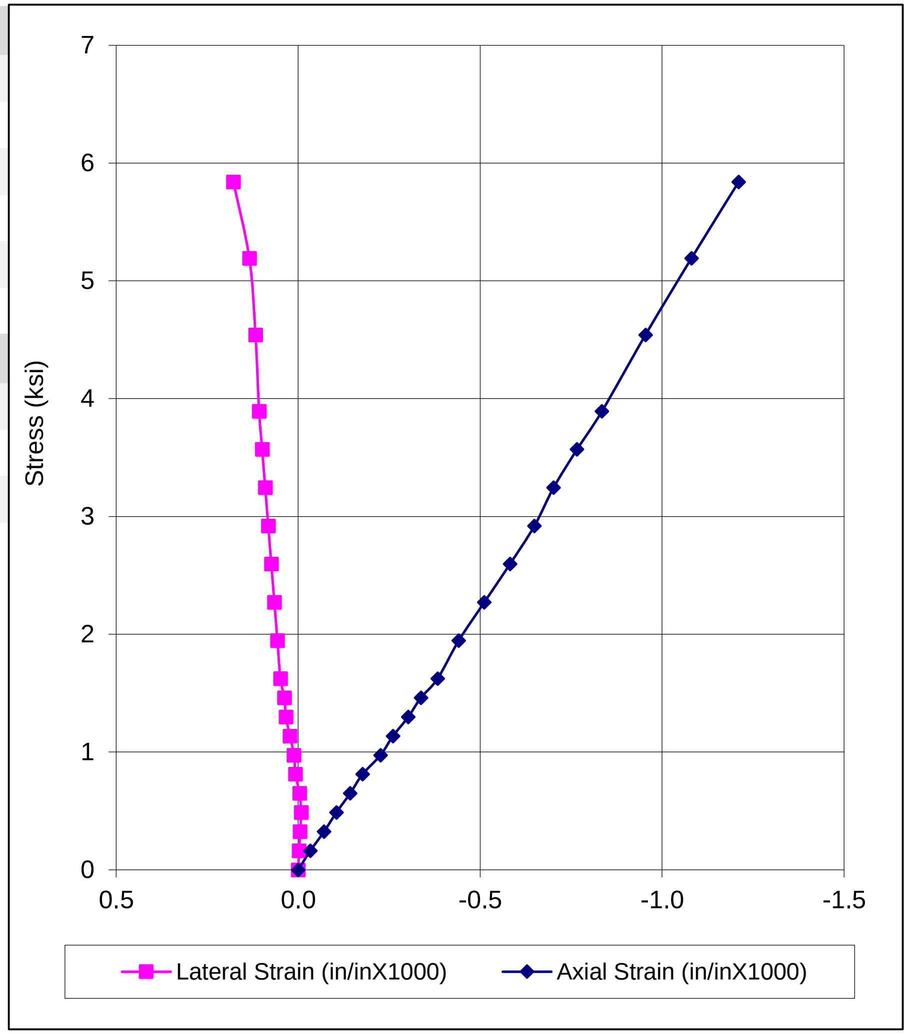
Thielsch DIVISION OF THE RISE GROUP	195 Frances Avenue Cranston, Rhode Island 02910 Phone: (401) 467-6454 Fax: (401) 467-2398 www.thielsch.com *Let's Build a Solid Foundation*	**Client Information:** GZA GeoEnvironmental South Portland, ME Project Manager: John Cozens Assigned by: John Cozens Collected by: Client	**Project Information:** Days Mills Bridge #2221 Replacement Kennebuck,ME Project Number: 09.0026198.10 Technician: SL Report Date: 04.24.24

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

Sample Information		Compressive Test Information	
Boring ID:	BB-KAKR-203	**Unit Weight (pcf):**	174.1
Sample #:	3R	**Failure Stress (psi):**	6,286
Depth (ft):	6.83-7.67	**Failure Mode:**	Fresh
Tested Depth (ft):	7.25-7.65	**Time to Failure (min)**	2.82
Rock Type:	Grey Slate		
Features:	Sample broke along foliation		

Test Specimen Information		Elastic Moduli Test Information	
Diameter, D (in):	1.981	**Poisson's Ratio @ 50%:**	0.13
Length, L (in):	4.535	**Strain %:**	0.121
L:D Ratio:	2.29	**E sec PSI @ 50%:**	4.56E+06





Testing Notes:

APPENDIX D – ROCK CORE PHOTOGRAPHS

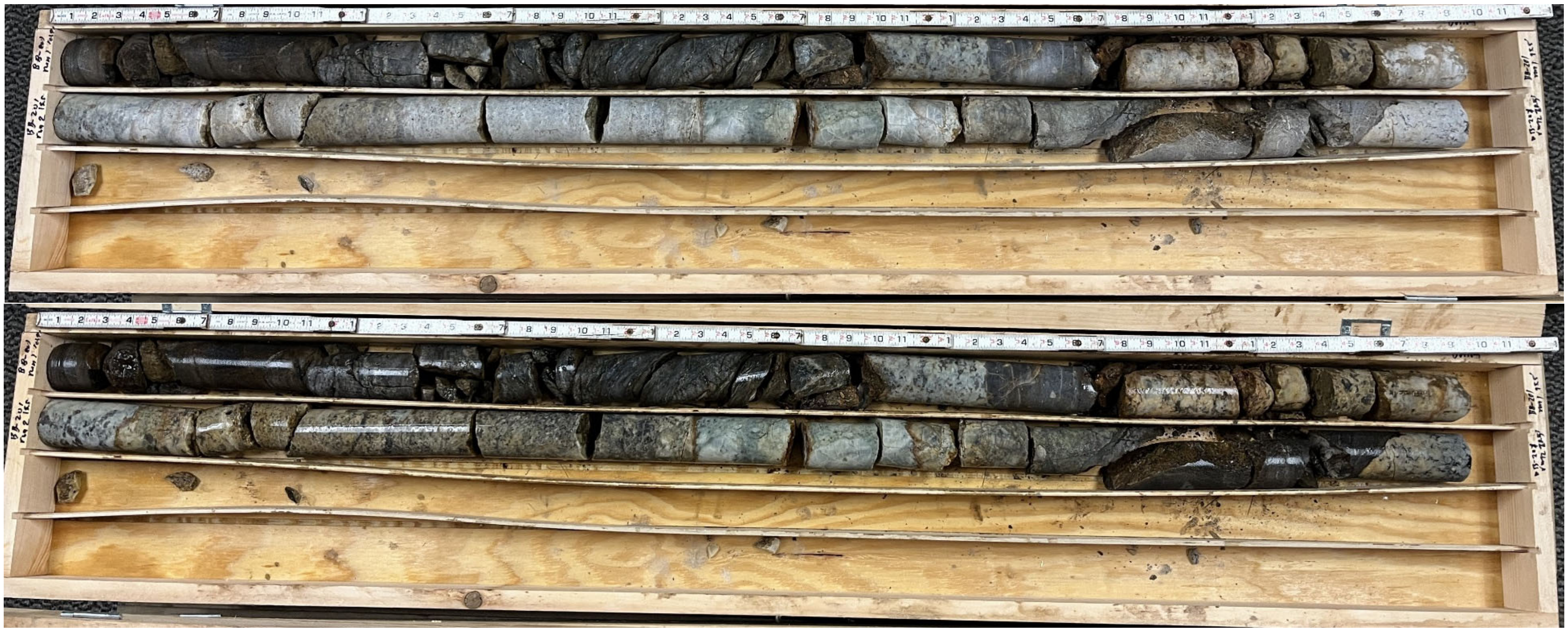
MaineDOT Day's Mill Bridge No. 2221
Route 35 over Kennebunk River
Kennebunk-Arundel, ME
WIN 26226.00
Rock Core Photographs

Boring No.	Run	Depth (ft)			Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB- KAKR -101	R1	23.5	-	28.5	60	100	39	65	SCHIST/GRANOFELS	1
BB- KAKR -101	R2	28.5	-	33.5	60	100	48	80	SCHIST/GRANOFELS	2
BB- KAKR -102	R1	16.5	-	21.5	58	96	37	64	SCHIST	3
BB- KAKR -102	R2	21.5	-	26.5	60	100	49	81	SCHIST	4

Notes: 1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 4=Bottom.
2. Top photo is dry, bottom photo is wet.

MaineDOT Day's Mill Bridge No. 2221
Route 35 over Kennebunk River
Kennebunk-Arundel, ME
WIN 26226.00
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB- KAKR -201	R1	10.5 - 15.5	57	95	20	33	SCHIST/GRANOFELS	1
BB- KAKR -201	R2	15.5 - 20.5	59	98	35	58	SCHIST/GRANOFELS	2



Notes:
1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 4=Bottom.
2. Top photo is dry, bottom photo is wet.

MaineDOT Day's Mill Bridge No. 2221
Route 35 over Kennebunk River
Kennebunk-Arundel, ME
WIN 26226.00
Rock Core Photographs

Boring No.	Run	Depth (ft)			Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB- KAKR -203	R1	8.0	-	10.0	20	83	0	0	SCHIST	1
BB- KAKR -203	R2	10.0	-	14.0	48	100	14	29	SCHIST	1 & 2
BB- KAKR -203	R3	14.0	-	18.0	48	100	43	89	SCHIST/GRANOFELS	2
BB- KAKR -202	R1	11.0	-	14.7	42	93	0	0	SCHIST	3
BB- KAKR -202	R2	14.7	-	19.7	58	96	47	78	SCHIST	3 & 4
BB- KAKR -202	R3	19.7	-	21.0	14	99	13	93	SCHIST	4

Notes: 1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 4=Bottom.
2. Top photo is dry, bottom photo is wet.

APPENDIX E – CALCULATIONS

Objective

Assess the nominal and factored bearing resistance of a foundation on rock based on support in SCHIST/GRANOFELS from borings BB-KAKR-101, 102, 201 through 203.

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 Day's Mill Bridge Replacement Project in Kennebunk-Arundel, ME. This calculation is based on the data from borings BB-KAKR-101/BB-KAKR-102 and BB-KAKR-201, BB-KAKR-202 and BB-KAKR-203.

Bedrock Strength

Boring	GS Elevation	RQD %	LAB							
			Depth of Sample (ft)	Depth of Sample Into Rock (ft)	ElevTop of Sample (ft)	UCS (psi)	UCS (ksi)	UCS (ksf)	Modulus (ksi)	Unit Wt (pcf)
BB-KAKR-101	145	65	23.5	0.2	121.3	3201	3.201	461	1180	173.9
BB-KAKR-101	145	65	23.5	2.3	119.2	5138	5.138	740	1490	165.9
BB-KAKR-102	144	64	16.5	4.1	123.4	4070	4.07	586	2450	173.8
BB-KAKR-201	142	95	13.3	2.8	132.2	4766	4.766	686	3090	163.6
BB-KAKR-203	144	75	14.8	6.8	135.9	6286	6.286	905	4560	174.1

Use a strength is 4 ksi for design

Boring ID	Core Run	Rec (%)	RQD %	Joint Spacing Desc.	Joint Spacing (in)	Aperture Desc.	Joint Aperture (in)	Joint Weathering
BB-KAKR-101	R1	100%	65%	Very Close to Close	0.75-8	Tigh to Moderately Wide	0.004-0.4	Fresh
BB-KAKR-101	R2	100%	80%	Close to Moderate	2.5-24	Tight	0.004-0.01	Fresh
BB-KAKR-102	R1	96%	64%	Very Close to Close	0.75-8	Tight to Partially Open	0.004-0.1	Fresh to Discolored
BB-KAKR-102	R2	100%	81%	/ery Close to Moderate	0.75-24	Tight to Partially Open	0.004-0.1	Fresh to Discolored
BB-KAKR-201	R1	95%	33%	Very Close to Close	0.75-8	Tight to Partially Open	0.004-0.1	Fresh to Decomposed
BB-KAKR-201	R2	98%	58%	Very Close to Close	0.75-8	Tight to Partially Open	0.004-0.1	Discolored
BB-KAKR-202	R1	93%	0%	Very Close to Close	0.75-8	Tight to Partially Open	0.004-0.1	Fresh to Disintegrated
BB-KAKR-202	R2	96%	78%	Close to Moderate	2.5-24	Tight to Partially Open	0.004-0.1	Fresh to Discolored
BB-KAKR-202	R3	99%	93%	/ery Close to Moderate	0.75-24	Tight to Partially Open	0.004-0.1	Fresh
BB-KAKR-203	R1	83%	0%	Close	8	Tight to Partially Open	0.004-0.1	Fresh to Discolored
BB-KAKR-203	R2	100%	29%	Very Close to Close	0.75-8	Tight to Partially Open	0.004-0.1	Discolored
BB-KAKR-203	R3	100%	89%	Close	8	Tight to Partially Open	0.004-0.1	Fresh to Discolored

Bedrock Quality

Average RQD of 58% representative of rock encountered in the borings.

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

$\sigma_{u.r} := 4\text{ksi} = 576 \cdot \text{ksf}$ Use assumed unconfined compressive strength of 4 ksi

From AASHTO LRFD Table 10.4.6.4-1

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

Parameter 2 - Drill Core Quality

Representative RQD from table above: approximately 58%, use a range of 50 to 75% for design.

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_2 := 13$

Parameter 3 - Spacing of Joints

From Boring Logs generally very close to moderately spaced, generally between 2 -12 inches.

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating

$$RR_3 := 10$$

Parameter 4 - Condition of Joints

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

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_4 := 20$

Parameter 5 - Ground Water Conditions

Hydrostatic Conditions- Interstitial water

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, and generally rough and tight. Therefore, the joint orientation is considered fair.

From AASHTO LRFD Table 10.4.6.4-2

Relative Rating $RR_6 := -7$

Total RMR Rating

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

$$RMR = 47$$

From AASHTO LRFD Table 10.4.6.4-3 RMR= 47 to 60 is indicative of Fair to Good Rock Quality

3. Determine Rock Property Constants s and m

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

SCHIST is categorized as rock type D, fine-grained polymineralic igneous & metamorphic crystalline rocks, RMR=47, using s and m values interpolated from the logarithmic trend of plotted values from AASHTO Table 10.4.6.4-4 (plots on sheet 7).

$m := .388$

$s := .000145$

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}.

$s = 0.000145$

$m = 0.388$

$\sigma_{u.r} = 4 \cdot ksi$

Nominal Bearing Resistance

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

$q_n = 46.9 \cdot ksf$ Say 47 ksf

Factored Bearing Resistance for Strength Condition

Bearing Resistance Factor is specified in Table 10.5.5.2.2-1

$\phi_b := 0.45$ Footing on rock

$q_R := \phi_b \cdot q_n$

$q_R = 21.1 \cdot ksf$ Say 21 ksf

Reference:I:\Mathcad\units.xmcd

10-22 AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

Table 10.4.6.4-1 Geomechanics Classification of Rock Masses.

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

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

	Parameter		Ranges of Values			
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

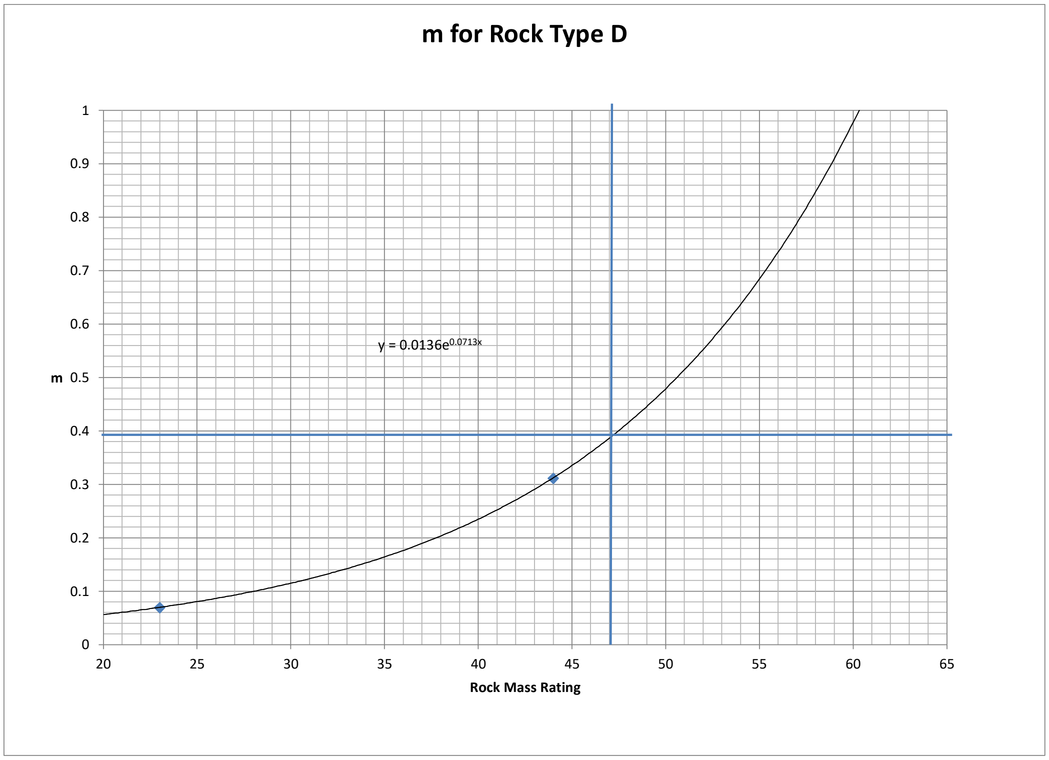
10-24 AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

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

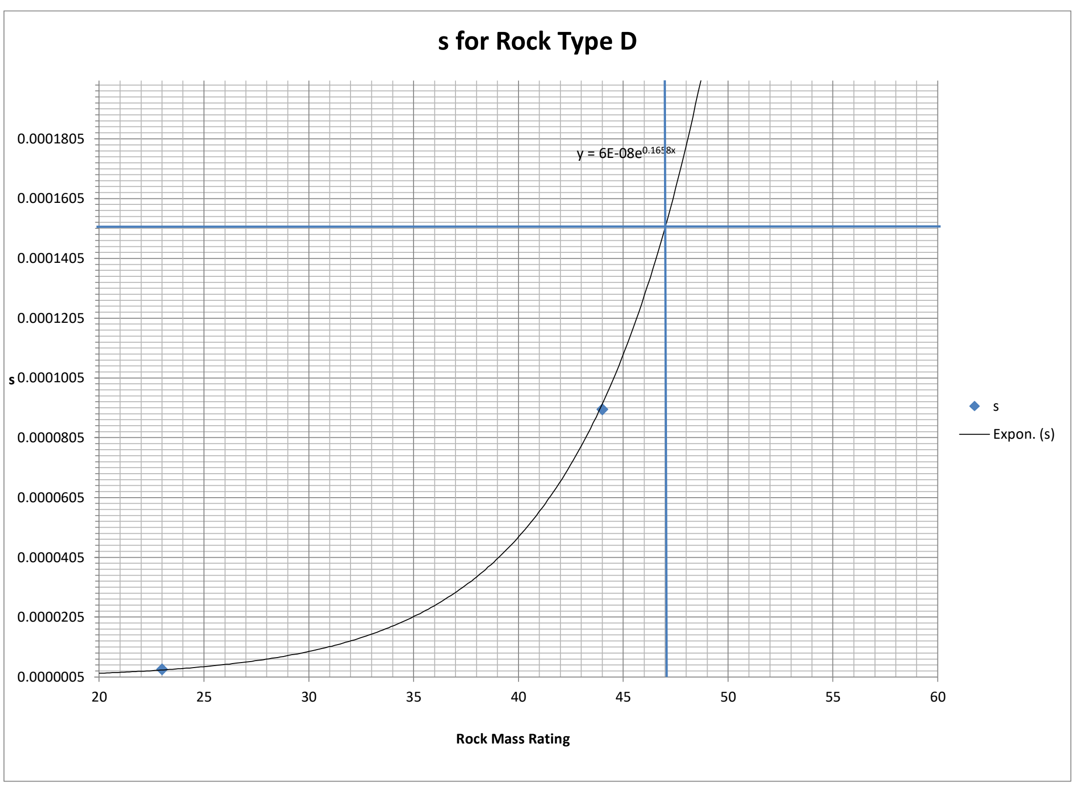
Rock Type:

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 polymineralic 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}



m for Rock Type D
y = 0.0136e^0.0713x
m
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
20
25
30
35
40
45
50
55
60
65
Rock Mass Rating



s for Rock Type D
y = 6E-08e^0.1658x
s
0.0000005
0.0000205
0.0000405
0.0000605
0.0000805
0.0001005
0.0001205
0.0001405
0.0001605
0.0001805
20
25
30
35
40
45
50
55
60
Rock Mass Rating
s
Expon. (s)

March 2014

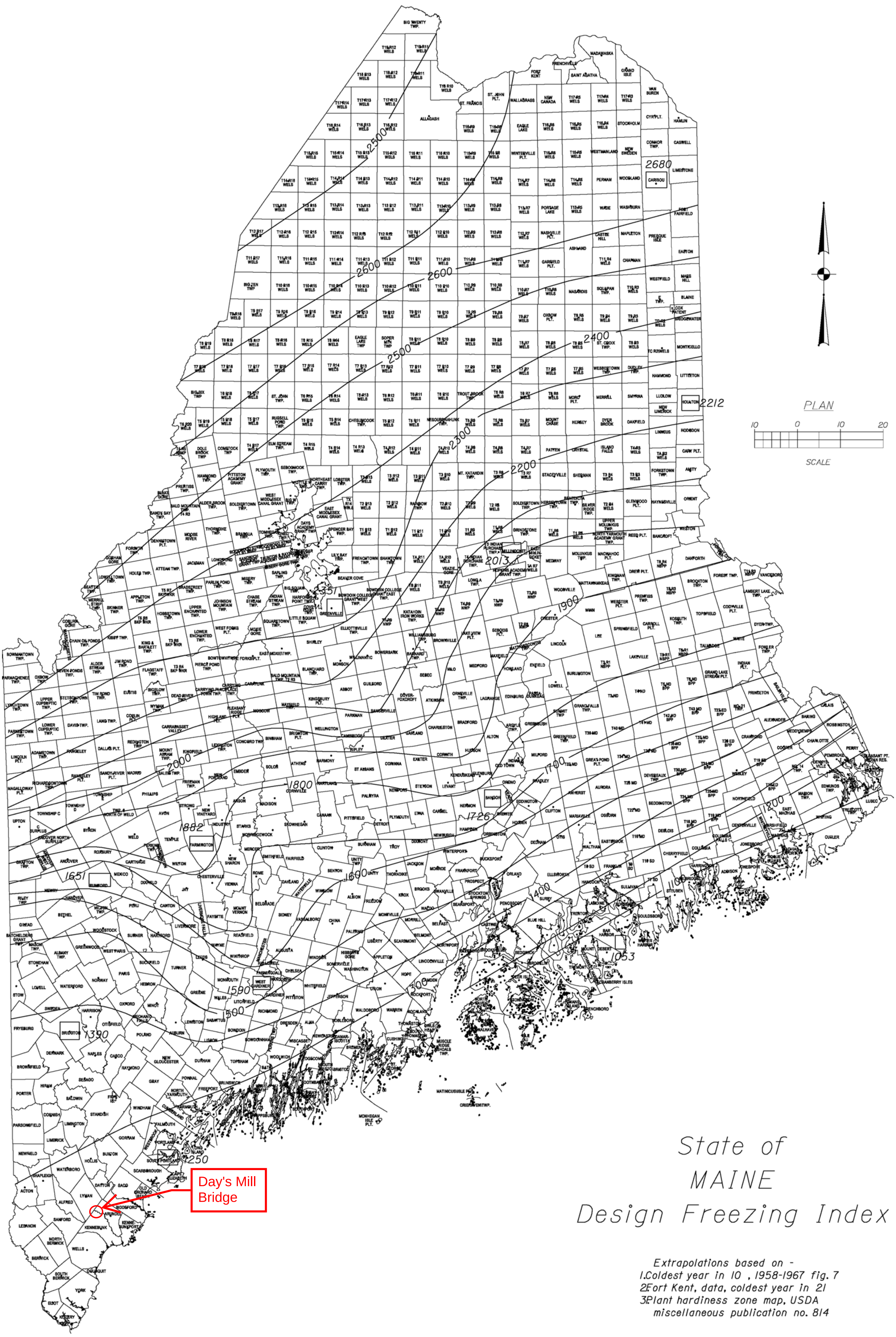


Figure 5-1 Maine Design Freezing Index Map

Table 5-1 Depth of Frost Penetration

Design Freezing Index	Frost Penetration (in)					
	Coarse Grained			Fine Grained		
	w=10%	w=20%	w=30%	w=10%	w=20%	w=30%
1000	66.3	55.0	47.5	47.1	40.7	36.9
1100	69.8	57.8	49.8	49.6	42.7	38.7
1200	73.1	60.4	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	[illegible]	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

1250

74.7" = 6.2'

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

The Freezing Index for the site is 1,250, and with low-moisture content (<10 percent) soils, the estimated depth of frost penetration is approximately 6.2 feet. Where abutment foundations bear directly on sound rock, there is no minimum requirement for footing embedment.

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

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Engineers and Scientists

JOB: 09.0026198.01 Day's Mill Bridge
SUBJECT: Lateral Earth Pressures
SHEET: 1 OF 1
CALCULATED BY B. Cardali 9/5/24
CHECKED BY C.Snow 9/5/24

Subject:

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

References:

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

Input Parameters:

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

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

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

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

Earth Pressure Coefficients:

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

Passive Earth Pressure (End Diaphragms)

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

$y := 0.38\text{in}$ — Maximum deflection from thermal expansion provided by structural engineer.

$H_b := 4\text{ft}$ — End Diaphragm Height

$\frac{y}{H_b} = 0.0079$ — Ratio of lateral movement to abutment height

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

$$\omega := \frac{y}{H_b} = 0.0079$$

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

$$\boxed{K_{p.mass} = 4.86}$$

$K = 0.43 + 5.7[1 - e^{-190(\delta_T/H)}]$

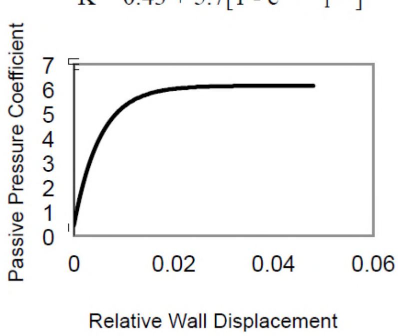


Figure 3.10.8-1: Plot of Passive Pressure Coefficient, K, vs. Relative Wall Displacement, δ_T/H.

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.

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

$$heel := 5ft$$

$$Intersection_{height} := \tan(90deg - \alpha)\cdot heel = 9\cdot ft$$

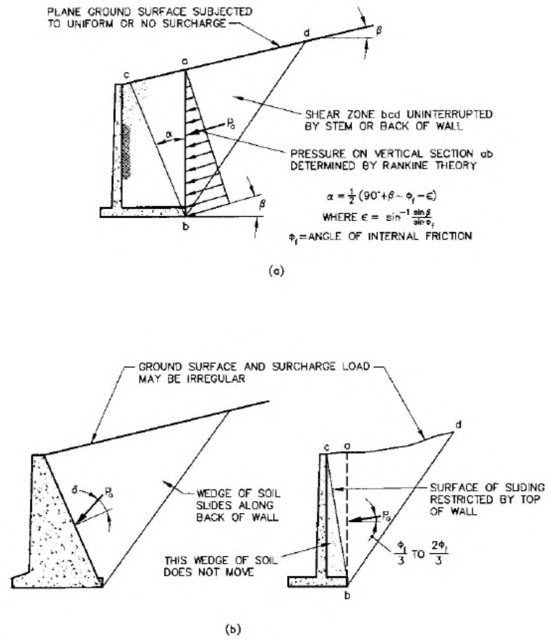


Figure C3.11.5.3-1—Application of (a) Rankine and (b) Coulomb Earth Pressure Theories in Retaining Wall Design

The abutment height is 17.5 feet. Based on Figure C3.11.5.3-1 of LRFD, the abutment is considered to be a short-heeled wall. Therefore, Coulomb theory should be used to calculate active earth pressures.

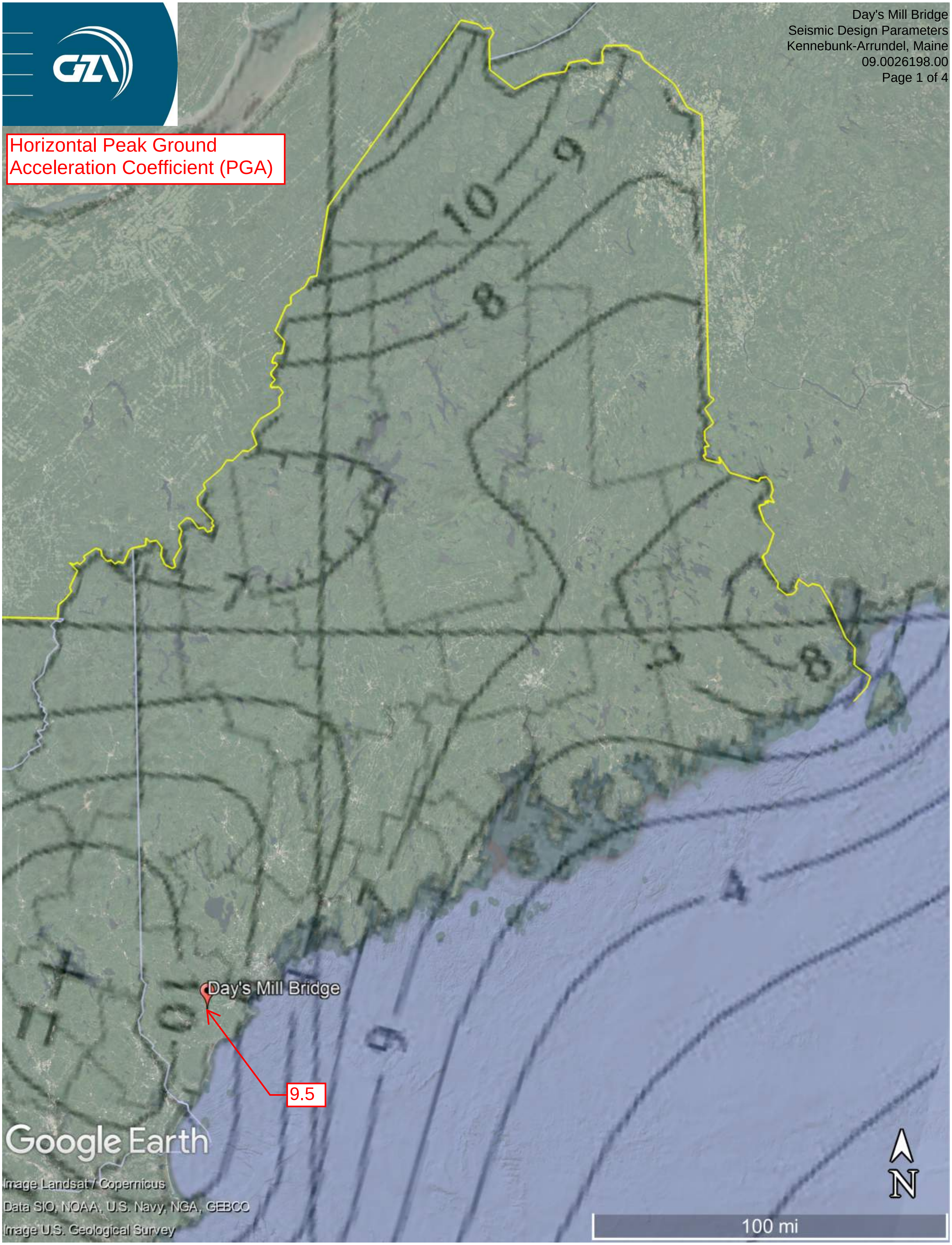
Coloumb Active Earth Pressure Coefficient (Short-Heeled Wall)

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

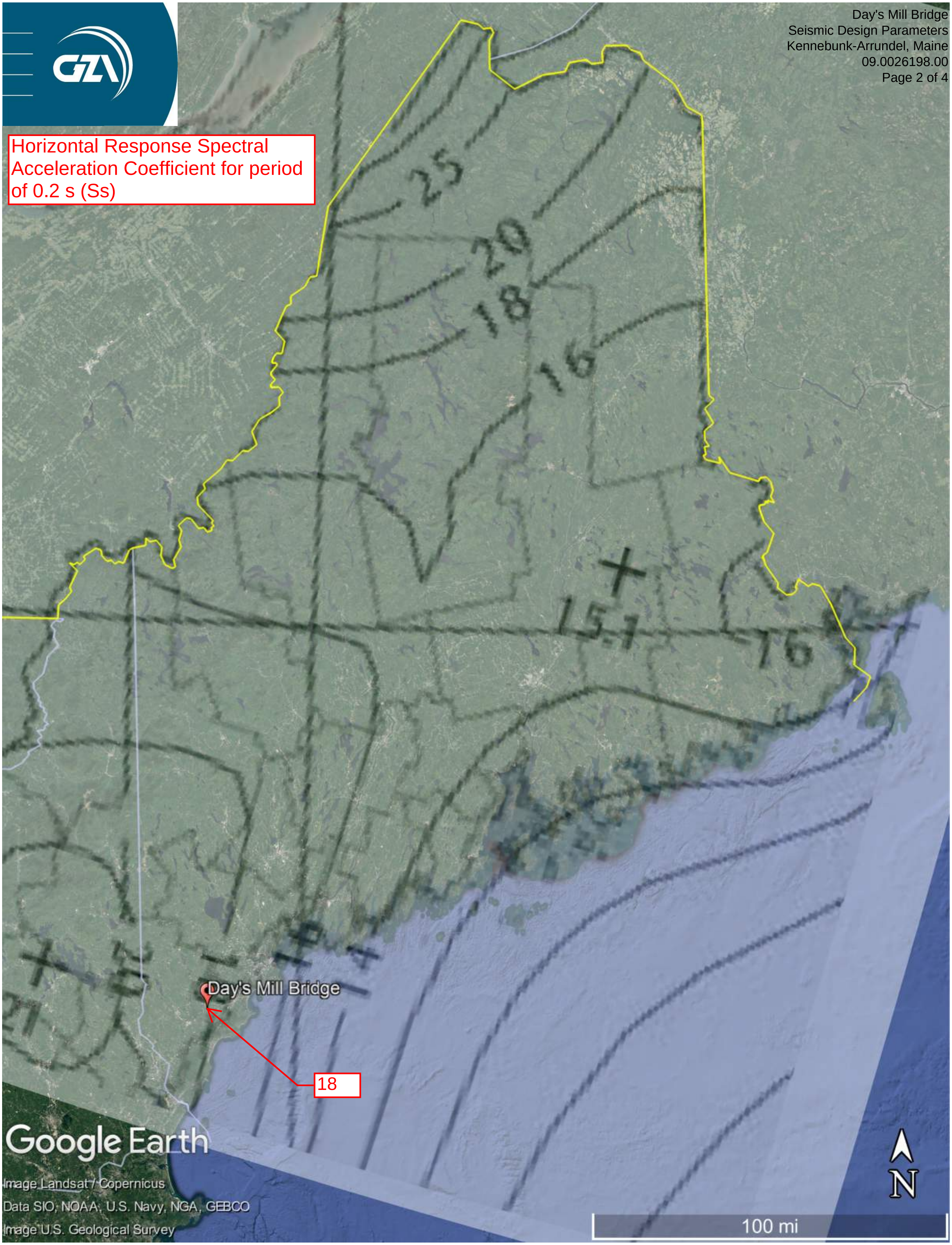
$$K_{ac} := \frac{(\sin(\theta + \phi))^2}{\Gamma\cdot\left[(\sin(\theta))^2\cdot\sin(\theta - \delta_f)\right]}$$

$$\boxed{K_{ac} = 0.28}$$

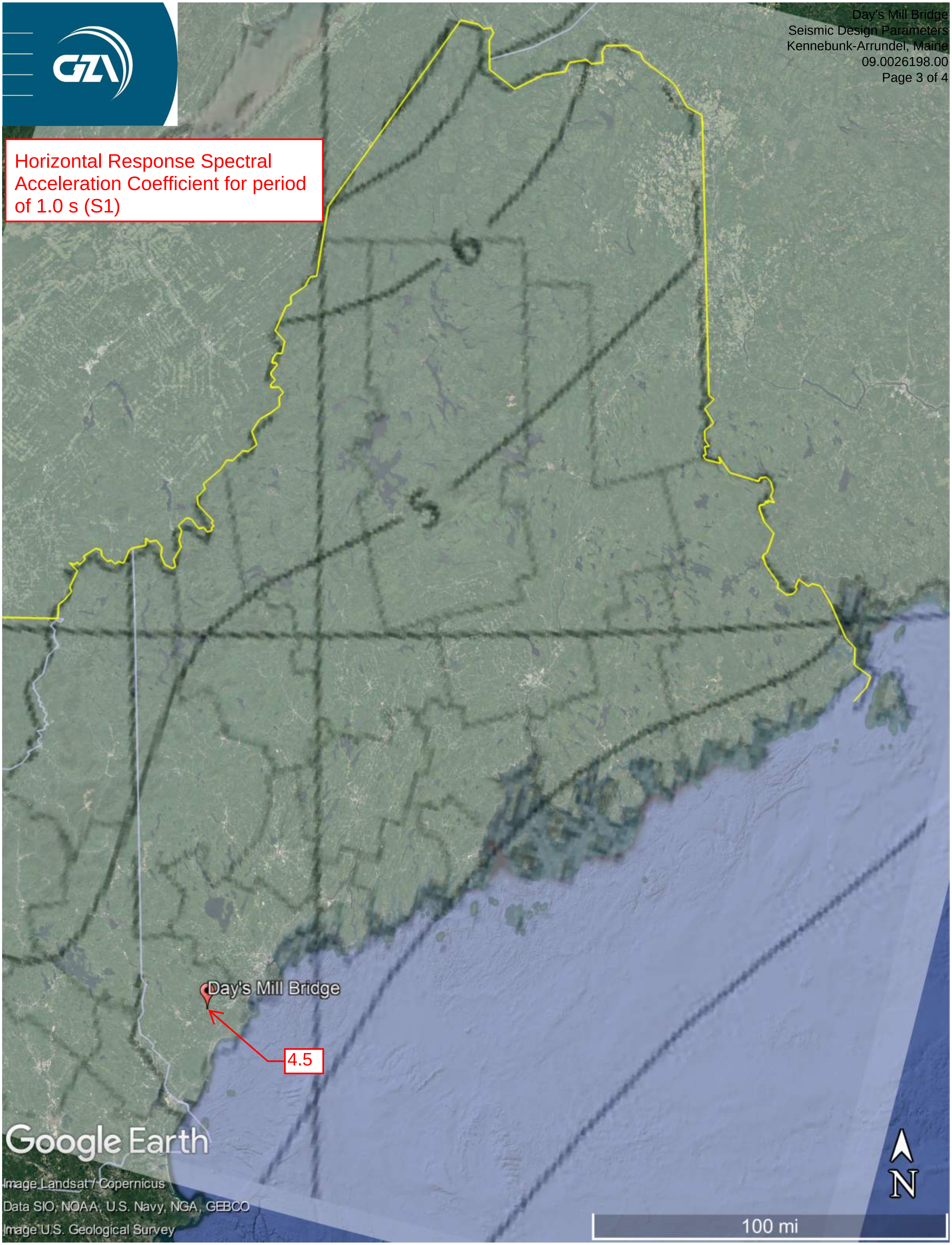
Horizontal Peak Ground Acceleration Coefficient (PGA)



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



Horizontal Response Spectral Acceleration Coefficient for period of 1.0 s (S1)



Day's Mill Bridge Seismic Interpolation for Coefficients		
Seismic Parameter	**Interpolated Value from Maps[1]**	**Design Parameter**
Horizontal Peak ground Acceleration Coefficient	9.5	$PGA = .095$
Horizontal Response Spectral Acceleration Coefficient for Period of 0.2s	18	$S_s = 0.180$
Horizontal Response Spectral Acceleration Coefficient for Period of 1.0s	4.5	$S_1 = .045$

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

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

Therefore:

$$A_s = F_{PGA} \times PGA = 1.0 \times 0.095 = 0.095\ g$$

$$S_{DS} = F_a \times S_s = 1.0 \times 0.180 = 0.180\ g$$

$$S_{D1} = F_v \times S_1 = 1.0 \times 0.045 = 0.045\ g$$

Summary:

SITE CLASS B SEISMIC DESIGN PARAMETERS	
Parameter	**Design Value**
Fpga	1.0
Fa	1.0
Fv	1.0
As (Period = 0.0 sec)	0.10 g
SDs (Period = 0.2 sec)	0.18 g
SD1 (Period = 1.0 sec)	0.05 g