Known for excellence.
Built on trust.

GEOTECHNICAL DESIGN REPORT

DETECTIVE BENJAMIN CAMPBELL BRIDGE OVER WEST BRANCH PENOBSCOT RIVER, ROUTE 11 BRIDGE NO. 3666

MAINE DOT WIN 23236.01

T3 INDIAN PURCHASE TOWNSHIP, MAINE

July 2020
09.0025976.01

Prepared for:
HNTB Corporation
Westbrook, Maine

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

31 Offices Nationwide
www.gza.com

Known for excellence.
Built on trust.

GEOTECHNICAL
ENVIRONMENTAL
ECOLOGICAL
WATER
CONSTRUCTION MANAGEMENT

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

VIA EMAIL

July 31, 2020
File No. 09.0025976.01

Ms. Lori Driscoll, P.E.
HNTB Corporation
340 Country Road, Suite 6C
Westbrook, Maine 04092

Re: Geotechnical Design Report
Replacement of Detective Benjamin Campbell Bridge No. 3666
Route 11 over West Branch Penobscot River
Maine Department of Transportation WIN 23236.01
T3 Indian Purchase Township, Maine

Dear Lori:

We are pleased to provide this Geotechnical Design Report, which includes geotechnical design recommendations for the Detective Benjamin Campbell Bridge No. 3666, Route 11 over the West Branch of the Penobscot River in T3 Indian Purchase Township, Maine. Our work was completed in accordance with GZA GeoEnvironmental, Inc.'s Project Contract for Task Order No. 775.01 which incorporates our December 11, 2019 proposal, HNTB Corporation (HNTB) File No. 75775-001-001-E008, dated February 4, 2020, our Master/Task Order Agreement dated February 4, 2020, and the attached Limitations contained in **Appendix A** of this report.

It has been a pleasure serving HNTB 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, please do not hesitate to contact the undersigned.

Very truly yours,

GZA GEOENVIRONMENTAL, INC.

Blaine M. Cardali, P.E.
Assistant Project Manager

Christopher L. Snow, P.E.
Consultant Reviewer

STATE OF MAINE
Andrew R. Blaisdell
10957
LICENSED
PROFESSIONAL ENGINEER
7/31/2020

Andrew R. Blaisdell, P.E.
Associate Principal

BMC/ARB/CLS:erc
p:\09 jobs\0025900s\09.0025976.00 - hntb - west branch indian purchase\09.0025976.01 - final design\report\final 25976.01 benjamin campbell bridge gdr 07312020.docx

Attachment: Geotechnical Design Report

TABLE OF CONTENTS

TABLE OF CONTENTS (*continued*)

TABLES

FIGURES

APPENDICES

1.0 INTRODUCTION

This report presents the results of the geotechnical evaluation by GZA GeoEnvironmental, Inc. (GZA) for the subject project. Our services were completed in accordance with GZA's Project Contract for Task Order No. 775.01 which incorporates our December 11, 2019 proposal, HNTB Corporation (HNTB) File No. 75775-001-001-E008, dated February 4, 2020, our Master/Task Order Agreement dated February 4, 2020, and the attached Limitations contained in **Appendix A** of this report.

GZA is providing geotechnical engineering services as a Subconsultant to HNTB, who is under contract with Maine Department of Transportation (MaineDOT) for design of the proposed bridge replacement.

1.1 BACKGROUND

The project includes replacement of the Detective Benjamin Campbell Bridge No. 3666 carrying Route 11 over the West Branch of the Penobscot River in T3 Indian Purchase Township, Maine, the location of which is shown on **Figure 1**. Our understanding of the existing bridge is based on drawings for the West Branch Bridge over Penobscot River, dated March 1948. The West Branch Bridge was built in 1948. It is a scour-critical, 360-foot-long, three-span bridge. The superstructure has two 60-foot-long steel with reinforced concrete deck approach spans, and a central 240-foot-long through-truss spanning the river. Existing substructures consist of full-height concrete abutments and concrete pier caps/columns supported by spread footings bearing on soil. The as-built drawings indicate the abutment footings bear approximately 5 feet below the original site grades, but embedment is significantly greater within the 10 to 15-foot-high approach embankments. Pier footings bear approximately 11 feet below current site grades, along the riverbanks, approximately coincident with the mid-river mudline elevation shown on the plans.

We understand plans are to construct a replacement bridge consisting of an approximately 380-foot-long, 45-foot-wide, two-span bridge, supported by stub abutments and one river pier, as shown on **Figure 2**. The proposed alignment is an on-alignment alternative, with the baseline matching the current roadway centerline. The proposed abutments will be skewed and will generally be within or behind the current abutment footprints. A temporary work trestle and single-lane temporary bridge are planned downstream of the existing bridge.

The Route 11 approach roadway elevation is proposed to be raised by about 4 to 6 feet above existing roadway grades at both approaches. The width of the roadway will be increased by about 8 feet to the east, where the maximum raise in grade will be approximately 10 feet. Additional grading is planned for a snowmobile trail approach to the bridge.

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. To meet these objectives, GZA completed the following Scope of Services:

- Conducted a site visit to observe surficial conditions and reviewed mapped surficial and bedrock geology of the site;
- Reviewed test borings, laboratory testing results and preliminary engineering completed as part of GZA's preliminary geotechnical design phase services;
- Coordinated and observed a supplemental subsurface exploration program, consisting of four test borings, to further evaluate subsurface conditions;
- Conducted additional laboratory testing to evaluate engineering and index properties of the site soils and bedrock;
- Conducted final design phase geotechnical engineering analyses for soil and bedrock properties; stability and settlement of approach embankments; frost susceptibility and drainage of approach subgrade materials; AASHTO LRFD load and resistance factors associated with geotechnical design elements; spread footing design considerations, including bearing resistance and resistance to lateral loading for footings bearing on rock and/or soil; and seismic design considerations;
- Developed geotechnical engineering recommendations including spread footings bearing on tremie seals over rock or soil, abutment backfill type and properties, earth pressures and seismic design parameters; recommended construction considerations; and
- Prepared this report summarizing our findings and design recommendations.

2.0 SUBSURFACE EXPLORATIONS

GZA completed a preliminary design exploration program in 2018 consisting of five test borings and a final design exploration program in 2020 consisting of four test borings. The as-drilled boring locations were surveyed by MaineDOT and provided to GZA and are shown on **Figure 2**. Elevations referenced in this report are in feet and refer to North American Vertical Datum of 1988.

New England Boring Contractors of Hermon, Maine provided drilling services and coordinated utility clearance. Borings were drilled using 3- and 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 in overburden soils, using a 24-inch-long, 1-3/8-inch inside diameter sampler. Bedrock cores were obtained using NX or NQ2 wire-line coring equipment in each test boring. Photographic logs of the recovered rock core specimens are included in **Appendix C**. 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.

2.1 PRELIMINARY BORINGS

Borings BB-IWBP-101 through BB-IWBP-105 were drilled between May 29, 2018 through June 7, 2018. The borings were completed using Mobile B-53 and CME track-mounted drill rigs. The borings were drilled to depths of approximately 18 to 47 feet below ground surface (bgs) and terminated after coring approximately 10 feet into competent bedrock.

SPTs were conducted using a safety hammer operated with a rope and cathead pulley system. Bedrock core runs were obtained using NQ2 wireline coring equipment. The borings were generally backfilled with ¾-inch crushed stone and/or soil cuttings and topped with asphalt cold patch in roadway areas and patched with concrete where drilled through the bridge deck.

2.2 FINAL DESIGN PHASE BORINGS

Borings BB-IWBP-201 through BB-IWBP-204 were drilled between February 25, 2020 and March 5, 2020. Two borings were drilled through the existing bridge in the river to the left and right of the proposed pier. The other two borings were drilled at each abutment. The borings were drilled using a truck-mounted Mobile B-53 drill rig.

Test borings were drilled to depths of 9 to 50 feet bgs and were terminated after coring approximately 5 to 23 feet into bedrock. SPTs were conducted using automatic hammer NEBC No. D24, which had a rated hammer energy transfer ratio of 0.904 at the time of drilling. The borings were generally backfilled with ¾-inch crushed stone and/or soil cuttings and topped with asphalt cold patch in roadway areas and patched with concrete where drilled through the bridge deck.

3.0 LABORATORY TESTING

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

Soil

- 19 gradation analysis / MaineDOT Frost Classification / Unified Soil Classification System (USCS) assessments;
- 19 moisture content tests; and
- Three hydrometer analyses.

Rock

- Five unconfined compression/secant modulus tests; and
- 15 point load tests (nine diametrical and six axial).

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

4.0 SUBSURFACE CONDITIONS

4.1 SURFICIAL AND BEDROCK GEOLOGY

Based on available surficial geologic mapping[1], the surficial unit in the vicinity of the site consists of end-moraine deposits, described as a mixture of silt, sand, gravel, cobbles and boulders deposited at the terminus of a glacial advance.

Based on available bedrock geologic mapping[2], the site is near the contact of two bedrock units. The site is mapped within and along the Northwest boundary of the Madrid Formation (Dsm), which is a metasedimentary formation consisting of quartzite and calc-silicate rock. The Carrabassett Formation (Dc) is mapped immediately west of the site, in which the major constituents are identified as slate and schist.

4.2 SUBSURFACE PROFILE

Two subsurface units were encountered beneath asphalt pavement or topsoil and above bedrock at the site: Fill and Glacial Moraine. In addition, a stratum of Fractured/Weathered Rock was encountered between the Moraine and bedrock in portions of the site. The approximate thicknesses and generalized descriptions of the subsurface units are presented in the following table, in descending order from existing ground surface. Detailed descriptions of the materials encountered at specific locations are provided in the boring logs in **Appendix B**. An interpretive subsurface profile based on the test boring results is presented as **Figure 3**, Interpretive Subsurface Profile.

Soil Unit	Approximate Encountered Thickness (ft)	Generalized Description
Fill	11 to 18	Brown to grey, loose to medium dense, sandy GRAVEL and gravelly SAND, trace silt. Possible cobbles and boulders. (USCS: GP, GW, GW-GM, GP-GM, SP-SM). MaineDOT Frost Classification = I to II *Encountered in all approach borings (BB-IWBP-101, -105, -201, and -204).*
Moraine	3 to 21	Brown to grey, dense to very dense, GRAVEL, some fine to coarse sand, some to trace silt, with cobbles and boulders. (USCS: GP-GM, GW, SM). Typical MaineDOT Frost Classification = I, Range = 0 to III *Encountered in all borings except BB-IWBP-103.*
Encountered Top of Bedrock Elevation	Approx. El. 429 to El. 454 (1.3 to 32.3 feet bgs) *Corresponds to bottom of Weathered/Fractured Rock in BB-IWBP-104, -105, and -204.*	

[1] Genes, Andrew N. and Newman, Willam A., 1986, Reconnaissance surficial geology of the Norcross [15-minute] quadrangle, Maine: Maine Geological Survey, Open-File Map 86-50, map, scale 1:62,500. Maine Geological Survey Maps. 638. http://digitalmaine.com/mgs_maps/638

[2] Osberg, P.H., Hussey, A.M., and Boone, G.M., 1985, Bedrock geologic map of Maine: Maine Geological Survey, Dept. of Conservation, scale 1:500,000.

4.2.1 Bedrock

Bedrock was cored in each test boring and was identified as schist of the Carrabassett Formation. The bedrock was generally described as hard, fresh to moderately weathered, fine to medium grained, and dark grey or grey-brown, with occasional white calcite banding. The joints were extremely close to moderately spaced, horizontal to vertical, planar to stepped, smooth to rough, fresh to discolored, tight to moderately wide, with occasional silt infilling. The RQD ranged from 0 to 85 percent (average of 23 percent), corresponding to a Rock Mass Quality of Very Poor to Good.

Three of the borings located near proposed abutment 2 (BB-IWBP-104, -105, and -205) encountered multiple 1- to 4-foot-thick zones of highly fractured Carrabassett Formation schist within the upper 10 to 16 feet of the rock. The highly fractured zones were interspersed with hard, fresh to moderately weathered SCHIST similar to the majority of the recovered rock. In some cases the fractured zones were sampled with a split spoon or drilled through using a roller cone because it was judged to be too fractured to core. Rock core recovery was generally between 11 and 100 percent and the Rock Quality Designation (RQD) was typically 0.

Five laboratory unconfined compressive strength / secant modulus tests and six point-load tests (nine diametrical and six axial) were conducted on typical bedrock core samples of Schist. The test results are included in **Appendix C**. The testing yielded:

- Unconfined compressive strengths ranging from 17.4 to 28.6 kips per square inch (ksi) and Young's modulus values ranging from 1,190 to 4,530 ksi; and
- Point-load index values correlated to compressive strengths ranging from 7.4 to 39.6 ksi.

In the fractured rock zone nine point-load tests were completed on rock core with the following results:

- Point-load index test values correlated to compressive strengths ranging from 7.4 to 33.3 ksi.

We note that these results show the rock in the fractured zones has similar strength to the typical rock tested from this site.

4.2.2 Groundwater

Groundwater depths were measured at depths of 10 and 15 feet (El. 470 and 459) in boreholes BB-IWBP-201 and -204, respectively. Ground water was not measured in the -100 series borings. Groundwater levels in the borings were measured during or immediately after drilling and may have been affected by drilling procedures, which included introduction of water for drilling purposes.

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

5.0 ENGINEERING EVALUATIONS

5.1 GENERAL

GZA conducted geotechnical engineering evaluations in accordance with *2017 AASHTO LRFD Bridge Design Specifications, 8th Edition* (herein designated as AASHTO) and the *MaineDOT Bridge Design Guide, 2003 Edition,* with updates through 2018 (MaineDOT BDG). The sections that follow describe the evaluations and the geotechnical design basis for each element. Supporting calculations are included in **Appendix F**.

5.2 APPROACH EMBANKMENTS

The proposed bridge approaches will be constructed with maximum side slope angles of approximately 2 horizontal to 1 vertical (2H:1V), or flatter. The proposed embankments will be raised up to about 6 feet above existing grades and widened toward the east.

We anticipate that the embankments will be primarily constructed over existing granular Fill and dense to very dense natural soil. Due to the strength and low compressibility of these soils, embankment settlement and global stability will not be a concern.

5.3 SCOUR CONSIDERATIONS

Available laboratory data for the soil at and near riverbed elevations were evaluated to estimate D50 values for use by others for scour depth evaluation, as summarized in the table below.

Boring & Sample ID	Soil Unit	Depth (feet)	Elevation (ft NAVD 88)	Water Content (%)	D_{50} (mm)	Classification	
						USCS	AASHTO
BB-IWBP-101, 5D	Moraine	20-22	460-458	5.5	6.97	GM	A-1-a
BB- IWBP-102, 2D	Moraine	5-7	455-453	3.6	15.5	GW	A-1-a
BB- IWBP-104, 1D	Topsoil	0-2	461-459	4.7	3.68	SP	A-1-a
BB- IWBP-104, 2D	Moraine	5.7-7.7	455.3-453.3	7.7	12.8	GP	A-1-a
BB- IWBP-105, 4D	Fill/Moraine	16.2-17.5	457.8-456.5	6.9	6.04	GM	A-1-a

5.4 EVALUATION OF FOUNDATION TYPES

5.4.1 Abutment Foundations

Spread footings bearing on tremie seals were identified as the preferred foundation type during preliminary design. The 98 percent plans indicate the following foundation and seal elevations at the abutments.

PROPOSED ABUTMENT FOOTING BEARING ELEVATIONS		
Abutment	Bottom of Footing El.	Bottom of Seal El.
1	461.5	454
2	458	452

The test borings indicate that bedrock will be exposed at or within a foot of the seal elevation at approximately El. 453 to El. 454 at Abutment 1. Therefore, the Abutment 1 footing should be founded on a tremie seal bearing on bedrock. Recommendations for bearing resistance are provided in **Section 6.4.1.**

At Abutment 2, the borings indicate a combination of very dense Moraine Gravel and/or fractured bedrock would be exposed at the seal elevation. The drilling characteristics and recovered samples in the material extending between El. 452 and El. 446 indicate that either very dense soil, fractured rock, or a combination of the two are present in this zone, extending to about 6 feet below the planned bottom of seal level (El. 452). Undisturbed moraine gravel and fractured bedrock are both considered suitable materials for support of spread footing foundations. To limit the extent of excavation required for foundation seal subgrade preparation at Abutment 2, the preferred approach is to prepare the excavations to a design seal elevation and allow support on fractured bedrock (see **Section 5.6.1**) or Moraine Gravel.

5.4.2 Pier Foundation

The interpreted top of rock profile shown on **Figure 3** indicates that rock is expected to be present within about 5 feet or less of the riverbed level. Therefore, we conclude that a spread footing constructed inside a cofferdam with a tremie seal on bedrock is the preferred foundation type for the pier. The bearing resistance for the pier will correspond to the bedrock evaluations included in **Section 5.6.1**.

5.5 LOAD AND RESISTANCE FACTORS

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

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 Fractured Rock or Sand	0.45	10.5.5.2.2-1
Sliding	Cast-in-Place Concrete on rock or soil	0.80	10.5.5.2.2-1

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

5.6 SPREAD FOOTING DESIGN CONSIDERATIONS

5.6.1 Footing Bearing Resistance on Rock, Pier and Abutments

Nominal and factored bearing resistances were calculated for bedrock-bearing footings using the Rock Mass Rating- (RMR-) based empirical correlation presented in "Foundations on Rock," by Duncan Wyllie. RMR was evaluated in accordance with Table 10.4.6.4-1 of the *2012 AASHTO LRFD Bridge Design*

Specifications, 6th Edition (AASHTO). The current (8th) Edition of the AASHTO Design Specifications does not include the RMR formulation included in the previous version (6th Edition). However, Articles C10.4.6.4 and 10.6.2.6.2 of the 8th Edition refer to RMR-based design procedures for footings on rock, so the 6th Edition methodology was followed.

GZA used bedrock data obtained in test borings to develop foundation design parameters at the abutment and pier locations. The unconfined compressive strength from the laboratory tests varied from 7.4 to 39.6 ksi. To select a representative strength for use in design, GZA considered the lowest unconfined compressive value from the five unconfined compressive tests as suitably conservative to represent the overall rock mass that will support the footings. This results in a representative unconfined compressive strength of 17.4 ksi. Point load tests and geologic hardness assessment of the fractured rock indicates similar or higher strengths. Therefore, the representative unconfined compressive strength is suitable for the fractured rock stratum, which is assumed to have an RQD of 0. The bedrock properties used in the bearing resistance evaluation are presented below:

DESIGN BEDROCK PROPERTIES FOR BEARING RESISTANCE EVALUATION					
Rock Type	**RQD (%)**	**Unconfined Compressive Strength (ksi)**	**Rock Mass Rating (RMR)**	**m**	**s**
Schist	0	17.4	29	0.064	0.000007

Based on these parameters, the calculated nominal bearing resistance is 40 kips per square foot (ksf), resulting in a factored bearing resistance of 18 ksf for the strength limit state. This bearing resistance is suitable for fractured (RQD = 0 percent) rock or sound bedrock. 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.

5.6.2 Footing Bearing Resistance on Soil, Abutment 2 Only

Bearing resistance values have been developed for the abutment footings bearing on Moraine Gravel using the theoretical method (Munfakh et al., 2001) based on SPT data. Bearing resistances were evaluated in accordance with Articles 10.6.3.1.1 and 10.6.3.1.2a of AASHTO LRFD.

Nominal, factored, and service bearing resistance values were developed for the proposed abutment footings which are planned to be 12 to 15 feet wide. The calculated bearing resistance values are presented in a bearing chart included in the calculations in **Appendix E** and are presented in the table below for the wingwall and abutment footing dimensions.

BEARING RESISTANCE VALUES FOR FOOTINGS ON SOIL (FOOTING 2 ONLY)				
Footing	**Footing Width (feet)**	**Nominal Bearing Resistance (ksf)**	**Factored Bearing Resistance, Strength Limit State (ksf)**	**Service Bearing Resistance (ksf)**
Abutment	15	42	19	8.5
Wingwall	12	37	16.5	9.5

Service limit design is based on a settlement of ¼-inch for the average bearing resistance stated in the table. Therefore, considering a triangular or trapezoidal pressure distribution, the maximum bearing pressure in the service condition could be greater than these values. This low settlement criteria for service limit state was selected in consideration of footing subgrade preparation under water and the potential to have some minor loosening of the material near the subgrade elevation, which could cause additional minor immediate settlement when loads are imposed. We estimate the post-construction foundation settlement associated with the service limit design will be ½ inch or less.

5.6.3 Design Footing Bearing Resistance, Abutment 2

To allow the Abutment 2 footing to be excavated to a target elevation, rather than exposing and cleaning a bedrock surface, the more conservative bearing resistance of the values for soil or rock should be utilized for design, as summarized in the table below.

ABUTMENT 2 DESIGN BEARING RESISTANCE VALUES			
Footing	**Nominal Bearing Resistance (ksf)**	**Factored Bearing Resistance, Strength Limit State (ksf)**	**Service Bearing Resistance (ksf)**
Abutment	40	18	8.5
Wingwall	37	16.5	9.5

5.7 SEISMIC DESIGN CONSIDERATIONS

The subsurface profile for seismic design includes the approach fills (including backfill behind abutments) and Moraine Deposits overlying bedrock. Seismic site class was determined in general accordance with LRFD Table C3.10.3.1, considering the average SPT N-values in granular soils which encompasses most of the soil encountered in the borings. The average SPT N-value is generally greater than 50 blows per foot in the abutment borings, which is the criterion for Site Class C. Therefore, the bridge is assigned to Site Class C.

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

5.8 LATERAL EARTH PRESSURE

Article 3.6.4 of the MaineDOT 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. For the proposed approximately 20-foot abutment height (measured above the footing), this would correspond to about 3/8 inch of lateral movement of the top of the abutment. The structural engineer should confirm that this amount of deflection is anticipated.

Based on Article 3.6.4 of the MaineDOT BDG, we conclude active earth pressure is appropriate for design of the abutments. 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. The material

properties will be controlled by the backfill material, which is anticipated to consist of BDG Type 4 soil. Soil properties for Type 4 soil are provided in **Section 6.3** of this report.

5.9 FROST PROTECTION

Fill soils are anticipated to be present at the abutments and embankments, either as existing fill, imported backfill, or the Moraine deposit. Based on the MaineDOT BDG, Section 5.2.1, the Freezing Index for the site is 2,055, and with low-moisture content (<10 percent) soils, the estimated depth of frost penetration is approximately 8 feet.

6.0 RECOMMENDATIONS

6.1 EMBANKMENT DESIGN CONSIDERATIONS

Embankment side slopes should be designed with MaineDOT typical slope angles of 2H:1V or flatter for a loam and seed surface finish. Where a riprap surface treatment is used, a 1.75H:1V slope angle is acceptable. Riprap should also be provided for scour protection where the embankment side slopes will be near or below typical water levels in the West Branch River. The extent and nature of scour countermeasures will be evaluated by others.

6.2 SEISMIC DESIGN

The peak ground acceleration coefficient, and 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 C) for a 7 percent probability of exceedance in 75 years can be calculated using the following parameters:

SITE CLASS C SEISMIC DESIGN PARAMETERS	
Parameter	**Design Value**
Fpga	1.2
Fa	1.2
Fv	1.7
As (Period = 0.0 sec)	0.081 g
SDs (Period = 0.2 sec)	0.188 g
SD1 (Period = 1.0 sec)	0.084 g

6.3 ABUTMENT AND WINGWALL DESIGN

- Backfill between new abutments and wingwalls and a 1.5H:1V plane extending up from the bottom of the abutment to the pavement subgrade 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

- Coulomb Active Earth Pressure, K_a = 0.28 (use for design of abutment and wingwalls)

- Live load surcharge should be applied as a uniform lateral surcharge pressure using the equivalent fill height (H_{eq}) values developed in accordance with LRFD Section 3.11.6.4, based on the abutment/wingwall height and distance from the wall backface to the edge of traffic. A minimum H_{eq} of 2 feet is recommended.
- Foundation drainage should be provided in accordance with Section 5.4.1.9 of the MaineDOT BDG. We recommend the use of French drains on the uphill side of abutments and wing walls to prevent buildup of differential hydrostatic pressure. The drains should be sloped to drain by gravity and should outlet through a series of 4-inch-diameter weep holes, spaced approximately 10 feet center-to-center.

6.4 RECOMMENDATIONS FOR FOUNDATIONS

6.4.1 Spread Footing Design

Abutment 1 and Pier

- The proposed Abutment 1 and pier should be supported on spread footing foundations over tremie seals bearing on bedrock. Footings designed to bear on bedrock should be designed using a nominal bearing resistance, q_n, of 40 ksf. At the strength limit state, footings should be designed for a maximum factored bearing resistance of 18 ksf, which can also be used for service limit state design.
- For tremie seal evaluation, the top of bedrock elevation in the borings at Abutment 1 was found to be between EL. 453 and 455 and Pier 1 was found to be between El. 445 and 449. It is important to note that the top of intact rock cannot be known for the entire foundation area prior to construction. We expect that intact rock may be encountered above and below the anticipated levels. Some construction-phase engineering should be anticipated to address the potential variability of the encountered conditions.
- If the exposed surface at Abutment 1 and the pier after cleaning the bedrock is below the anticipated bottom of tremie elevation, the tremie seal may be thickened as needed.
- The Abutment 1 and pier bedrock surface should be cleaned of soil or rock prior to concrete placement for tremie seals. Bearing surface preparation should be in accordance with **Section 7.3** and Special Provision 511 - Cofferdams.
- Since the footings will be founded on bedrock, there is no minimum embedment required for frost protection per MaineDOT BDG Article 5.2.1.

Abutment 2

- The proposed Abutment 2 footing should be supported on a spread footing foundation over a tremie seal bearing on a combination of undisturbed dense Moraine gravel, fractured bedrock and/or bedrock. Tremie seal elevation may be set at El. 452 at Abutment 2. Abutment and wingwall footing bearing pressures should be checked to confirm that they are less than the resistance values presented in the table below.

ABUTMENT 2 DESIGN BEARING RESISTANCE VALUES			
Footing	**Nominal Bearing Resistance (ksf)**	**Factored Bearing Resistance, Strength Limit State (ksf)**	**Bearing Resistance, Service Limit State (ksf)**
Abutment	40	18	8.5
Wingwall	37	16.5	9.5

- The abutment subgrade surfaces should be cleaned of soil or rock that are loosened by the excavation process prior to concrete placement for tremie seals. Bearing surface preparation should be in accordance with **Section 7.3**.
- Since the Abutment 2 footing may be founded on very dense soil, we recommend that the bottom of the seal bear at or below the estimated maximum frost penetration depth of 8 feet.

All Footings

- Spread footings 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. Although this is the criteria presented for bedrock in AASHTO LRFD, it is considered acceptable for all footings for the project considering the expected very dense and rock-like nature of the bearing material.
- The recommended base resistance against sliding is based on NAVFAC DM7.02-63, Table 1, which indicates the sliding resistance coefficient (tan δ) is equal to 0.7 for cast-in-place concrete on Moraine soil, fractured rock or sound bedrock. The nominal sliding resistance between footings and anticipated subgrades is equal to the vertical force multiplied by 0.7. Including the recommended resistance factor of 0.8 for sliding, the recommended factored sliding resistance coefficient is 0.56 for Strength Limit State.
- In accordance with current MaineDOT geotechnical drawing standards, we recommend that the Abutment and Pier Geometry plan sheets show bedrock elevations only at the boring locations, rather than providing an interpreted bedrock surface. The intent of this recommendation is to limit the potential for claims associated with unanticipated conditions.
- If the bedrock level extends above the design bottom of tremie seal elevation, the footing may be raised and vertical reinforcement shortened in the wall subject to review and approval of the Designer to limit overexcavation of bedrock.
- Anchoring, doweling, benching or other means of improving sliding resistance are recommended if the prepared surface is steeper than 4H:1V in any direction.

7.0 CONSTRUCTION CONSIDERATIONS

This section provides guidance regarding quality control during excavation, dewatering, and foundation subgrade preparation and protection. These items are discussed in the paragraphs that follow.

7.1 EXCAVATION, TEMPORARY LATERAL SUPPORT AND DEWATERING

Excavations for abutment foundations are anticipated to extend approximately 22 to 25 feet below existing pavement grades and at least 5 feet below groundwater levels encountered in the borings, and approximately 6 to 8 feet below the Q1.1 river level elevation (El. 460.1). The pier excavation will be conducted in the water and will extend to bedrock approximately 12 to 15 feet below the Q1.1 river level. Therefore, we anticipate that temporary sheet pile cofferdams with tremie seals will be necessary for the construction of the abutments and piers, and excavation for the tremie seals may be conducted in the wet. However, if a soil subgrade is present that is subject to degradation in the wet, it will be necessary to remove all disturbed material or dewater and complete the subgrade preparation in the dry. This should be addressed in the cofferdam special provision. Given the lack of potential toe-in, we anticipate that the system will most likely require internal bracing until the tremie seal is placed.

Where feasible temporary dewatering from the cofferdams may be completed using a combination of sandbags and pumping from internal sumps or low points.

In areas where sufficient space is available and water conditions permit, excavation slopes may consist of sloped, open cuts. In all cases, temporary excavations should comply with OSHA excavation safety requirements.

7.2 SUBGRADE PREPARATION

We anticipate that bearing surface preparation will be conducted in the wet for abutments and pier. At Abutment 1 and the pier, preparation of a clean bedrock surface is required, and at Abutment 2, an undisturbed dense gravel or bedrock surface is suitable. A Special Provision should be prepared to define the project-specific requirements for subgrade preparation and quality assurance/quality control. Recommendations specific to abutment and pier bearing surface preparation are provided in the following sections.

7.2.1 Abutment 1 and Pier Subgrade Preparation

The bedrock surface beneath Abutment 1 and the pier will be variable in terms of elevation, slope and localized weathering. A combination of standard excavation equipment, hydraulic hoe ramming equipment, and/or air lifting may be needed to remove the overburden and fractured/weathered rock to expose a cleaned bedrock surface. All soil and loose, decomposed, highly weathered and fractured bedrock should be removed prior to placement of tremie seal at the pier location. The prepared bearing surface should be checked by depth probing in conjunction with visual means such as diver and/or remotely operated vehicle video inspection.

The Geotechnical Engineer and Designer should be provided soundings and cross-sections showing the prepared bearing surface geometry prior to placement of concrete to evaluate whether benching, doweling, or subfooting reinforcement are needed for that foundation location. Based on the test borings we do not anticipate steeply sloping bedrock conditions at the abutments. However, if the exposed bedrock surface is steeper than 4H:1V, then anchoring, doweling, benching or other means should be employed to improve sliding resistance.

We recommend that the completed seals bearing on bedrock be cored to assess the quality of the tremie seal concrete and the condition of the contact between the tremie seal and bedrock. The concrete and bedrock coring activities should be observed by the Geotechnical Engineer to assess changes in conditions based on drill action and coring penetration rate. The Geotechnical Engineer, Designer and Resident should review and evaluate the core results. If the observed conditions indicate inadequate concrete quality and/or unsuitable material at the concrete/rock interface, additional coring will be recommended, and remedial seal repairs may be required.

7.2.2 Abutment 2 Subgrade Preparation

The subgrade material beneath the tremie is anticipated to consist of undisturbed moraine gravel, fractured rock, intact rock, or a combination of these materials. If a soil subgrade is exposed that has been softened or degraded by the presence of water, additional excavation should be made to expose non-degraded material, or if possible, the excavation completed in the dry. A combination of standard excavation equipment and air lifting equipment may be needed to reach the seal elevation and remove loosened material. However, it is not necessary to expose a cleaned bedrock surface. If bedrock is encountered above the design seal depths that cannot be removed using an excavator, it is acceptable from a geotechnical standpoint to leave it in place. The Resident should consult with HNTB relative to tremie seal design and hydrostatic uplift if this condition is encountered. The Contractor should sound the completed subgrade on a grid to confirm the subgrade elevation and to assess the presence of loosened material. The cleanliness may be checked with visual means such as a diver if desired by the Resident.

TABLES

TABLE 1

Summary of Subsurface Explorations

Detective Benjamin Campbell Bridge #3666, Route 11 over West Branch Penobscot River
T3 Indian Purchase Township, ME
WIN 23236.00

Boring ID	Ground Surface El. (ft)	Top of Stratum Elevation (ft)					Stratum Thickness (ft)					Depth to Top of Bedrock (ft)	Bottom of Boring Depth (ft)	Bottom of Boring El. (ft)	Groundwater	
		Pavement	Topsoil	Fill	Moraine	Bedrock	Pavement	Topsoil	Fill	Moraine	Fractured Rock				El. (ft)	Depth (ft)
BB-IWBP-101	480.1	480.1	NE	479.3	461.6	454.3	0.8	NE	17.7	7.3	NE	25.8	37.0	443.1	NM	NM
BB-IWBP-102	461.0	NE	461.0	NE	459.0	448.5	NE	2.0	NE	10.5	NE	12.5	24.5	436.5	NM	NM
BB-IWBP-103	449.0	NE	NE	NE	NE	449.0	NE	NE	NE	NE	NE	0.0	17.8	431.2	451.0	0.0
BB-IWBP-104	461.6	NE	461.6	NE	459.6	439.0	NE	2.0	NE	20.6	9.7	22.6	40.4	421.2	NM	NM
BB-IWBP-105	474.2	474.2	NE	473.2	458.4	452.6	1	NE	14.8	5.8	16.3	21.6	47.2	427.0	NM	NM
BB-IWBP-201	480.0	480.0	NE	479.3	464.5	453.3	0.7	NE	14.8	11.2	NE	26.7	41.9	438.1	470	10.0
BB-IWBP-202	451.0	NE	NE	NE	451.0	448.3	NE	NE	NE	2.7	NE	2.7	24.2	426.8	451.0	0.0
BB-IWBP-203	449.8	NE	NE	NE	449.8	445.3	NE	NE	NE	4.5	NE	4.5	9.7	440.1	449.8	0.0
BB-IWBP-204	474.3	474.3	NE	473.8	462.3	446.7	0.5	NE	11.5	15.6	4.1	27.6	50.5	423.8	459.3	15.0

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 locations were surveyed by MaineDOT and provided to GZA by HNTB.
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 surface water of the West Branch Penobscot River was measured at El. 460, 9.5 feet above the mudline at BB-IWBP-103 on June 15, 2018.

TABLE 2
Summary of Bedrock Data
Detective Benjamin Campbell Bridge #3666, Route 2 Over West Branch Penobscot
T3 Indian Purchase Township, ME
WIN 23236.00

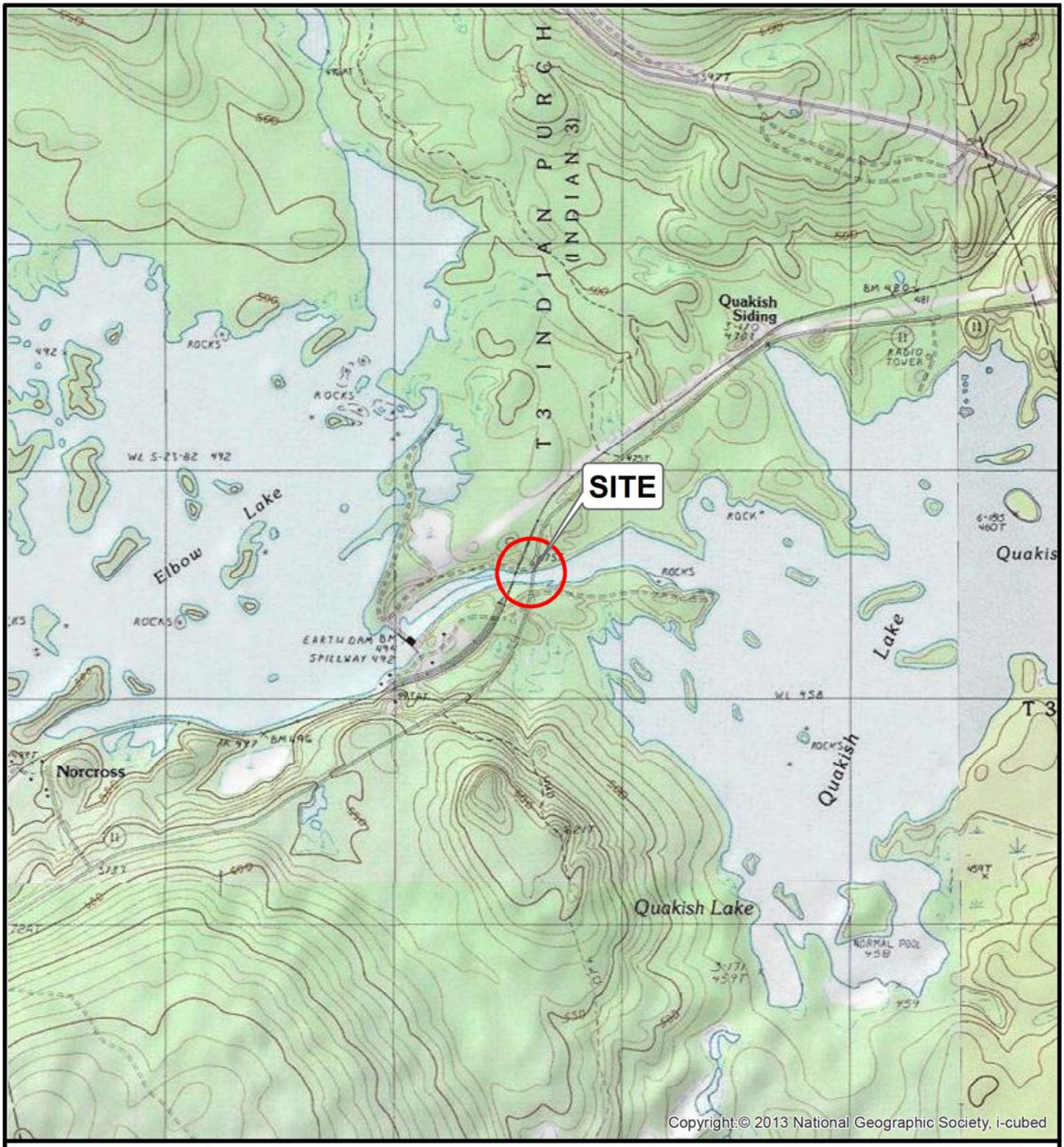
Boring ID	Core Run	Ground Surface El. (ft)	Depth of Core Run below GS (ft)			Depth to Rock (ft)	Depth Below Top of Rock (ft)			Length of Core Run (in)	Rec (in)	Rec (%)	RQD (in)	RQD %	Joint Spacing (in)	Joint Aperture (in)	Elev. (ft)		LAB						Rock Type
			Top		Bottom		Top		Bottom								Top	Bottom	Depth of Sample (ft)	Depth of Sample into Rock (ft)	Elev Top of Sample (ft)	UCS (psi)	Modulus (ksi)	Unit Wt (pcf)	
BB-IWBP-101	R1	480.1	26.9	-	32.0	25.8	1.1	-	6.2	5.1	61	100%	35	57%	<0.75-24	0.004-0.1	453.2	448.1	27.1	1.3	453.0	18,659	4,020	167.0	SCHIST
BB-IWBP-101	R2	480.1	32.0	-	34.0	25.8	6.2	-	8.2	2.0	20	83%	12	50%	8	0.004-0.01	448.1	446.1							SCHIST
BB-IWBP-101	R3	480.1	34.0	-	37.0	25.8	8.2	-	11.2	3.0	36	100%	19	53%	<0.75-24	0.004-0.4	446.1	443.1							SCHIST
BB-IWBP-102	R1	461.0	13.5	-	16.7	12.5	1.0	-	4.2	3.2	38	100%	9	24%	<0.75	0.004-0.4	447.5	444.3	*15.3*	*2.8*	*445.7*	*21,300*	--	*166.9*	SCHIST
BB-IWBP-102	R2	461.0	16.7	-	19.3	12.5	4.2	-	6.8	2.6	31	100%	14	45%	0.75-24	0.02-0.4	444.3	441.7							SCHIST
BB-IWBP-102	R3	461.0	19.3	-	22.3	12.5	6.8	-	9.8	3.0	32	89%	4	11%	<0.75-0.75	0.004-0.4	441.7	438.7							SCHIST
BB-IWBP-102	R4	461.0	22.3	-	24.5	12.5	9.8	-	12.0	2.2	26	100%	11	42%	0.75-8	0.01->0.4	438.7	436.5							SCHIST
BB-IWBP-103	R1	449.0	1.3	-	3.9	0.0	1.3	-	3.9	2.6	26	84%	0	0%	<0.75-8	0.004-0.1	447.7	445.1	*1.5*	*1.5*	*447.5*	*47,100*	--	*165.7*	SCHIST
																			1.7	*1.7*	*447.4*	*27,500*	--	*155.9*	SCHIST
BB-IWBP-103	R2	449.0	3.9	-	5.7	0.0	3.9	-	5.7	1.8	22	100%	4	19%	<0.75-8	0.004-0.1	445.1	443.3							SCHIST
BB-IWBP-103	R3	449.0	5.7	-	7.3	0.0	5.7	-	7.3	1.6	19	100%	0	0%	<0.75-8	0.004-0.1	443.3	441.7							SCHIST
BB-IWBP-103	R4	449.0	7.3	-	9.7	0.0	7.3	-	9.7	2.4	25	86%	9	31%	<0.75-8	0.02-0.4	441.7	439.3							SCHIST
BB-IWBP-103	R5	449.0	9.7	-	12.2	0.0	9.7	-	12.2	2.5	30	100%	5	17%	0.75-8	0.004-0.4	439.3	436.8							SCHIST
BB-IWBP-103	R6	449.0	12.2	-	15.2	0.0	12.2	-	15.2	3.0	33	92%	6	18%	<0.75-8	0.1-0.4	436.8	433.8							SCHIST
BB-IWBP-103	R7	449.0	15.2	-	17.5	0.0	15.2	-	17.5	2.3	22	79%	0	0%	<0.75-0.75	0.1-0.4	433.8	431.5							SCHIST
BB-IWBP-103	R8	449.0	17.5	-	17.8	0.0	17.5	-	17.8	0.3	2	50%	0	0%	<0.75	0.02-0.1	431.5	431.2							SCHIST
BB-IWBP-104	R1	461.6	23.0	-	25.5	22.6	0.4	-	2.9	2.5	30	100%	0	0%	<0.75-0.75	0.02-0.1	438.6	436.1							SCHIST
BB-IWBP-104	R2	461.6	25.5	-	28.0	22.6	2.9	-	5.4	2.5	30	100%	0	0%	<0.75-0.75	0.02-0.1	436.1	433.6							SCHIST
BB-IWBP-104	R3	461.6	28.0	-	30.0	22.6	5.4	-	7.4	2.0	18	75%	0	0%	<0.75-0.75	0.02-0.1	433.6	431.6							SCHIST
BB-IWBP-104	R4	461.6	30.0	-	34.0	22.6	7.4	-	11.4	4.0	42	88%	10	20%	0.75-8	0.004-0.01	431.6	427.6	*31.9*	*9.3*	*429.7*	*18,200*	--	*161.1*	SCHIST
BB-IWBP-104	R5	461.6	34.0	-	36.7	22.6	11.4	-	14.1	2.7	32	100%	0	0%	<0.75-8	0.02-0.1	427.6	424.9							SCHIST
BB-IWBP-104	R6	461.6	36.7	-	40.4	22.6	14.1	-	17.8	3.7	44	100%	13	30%	0.75-8	0.004-0.1	424.9	421.2							SCHIST
BB-IWBP-105	R1	474.2	22.0	-	22.8	21.6	0.4	-	1.2	0.8	1	10%	0	0%	<0.75	0.02-0.1	452.2	451.4							SCHIST
BB-IWBP-105	R2	474.2	23.0	-	24.6	21.6	1.4	-	3.0	1.6	13	68%	6	31%	<0.75	0.02-0.1	451.2	449.6	*23*	*1.4*	*451.2*	*24,700*	--	*153.6*	SCHIST
BB-IWBP-105	R3	474.2	29.8	-	30.4	21.6	8.2	-	8.8	0.6	7	100%	0	0%	<0.75	0.1-0.4	444.4	443.8							SCHIST
BB-IWBP-105	R4	474.2	30.4	-	32.7	21.6	8.8	-	11.1	2.3	11	39%	0	0%	<0.75	0.1-0.4	443.8	441.5							SCHIST
BB-IWBP-105	R5	474.2	38.0	-	41.4	21.6	16.4	-	19.8	3.4	40	98%	20	49%	<0.75-24	0.004-0.1	436.2	432.8	38	16.4	436.2	18,510	2,770	173.9	SCHIST
BB-IWBP-105	R6	474.2	41.4	-	43.9	21.6	19.8	-	22.3	2.5	30	100%	0	0%	<0.75-8	0.004-0.1	432.8	430.3							SCHIST
BB-IWBP-105	R7	474.2	43.9	-	47.2	21.6	22.3	-	25.6	3.3	39	98%	9	23%	<0.75-8	0.004-0.4	430.3	427.0							SCHIST
BB-IWBP-201	R1	480.0	27.0	-	31.4	26.7	0.3	-	4.7	4.4	52	98%	44	85%	<0.75-24	0.01-0.4	453.0	448.6	28.4	1.7	451.6	28,556	2,920	169.3	SCHIST
BB-IWBP-201	R2	480.0	31.4	-	34.2	26.7	4.7	-	7.5	2.8	33	97%	8	24%	<0.75-8	0.004-0.4	448.6	445.8							SCHIST
BB-IWBP-201	R3	480.0	34.2	-	36.9	26.7	7.5	-	10.2	2.7	30	94%	10	32%	<0.75-8	0.004-0.1	445.8	443.1							SCHIST
BB-IWBP-201	R4	480.0	36.9	-	41.9	26.7	10.2	-	15.2	5.0	60	100%	47	78%	0.75-24	0.004-0.4	443.1	438.1							SCHIST
BB-IWBP-202	R1	451.0	3.1	-	4.8	2.7	0.4	-	2.1	1.7	17	85%	0	0%	<0.75-8	0.01-0.4	447.9	446.2	*3.3*	*0.6*	*447.7*	*39,576*	--	*168.1*	SCHIST
																			3.5	*0.8*	*447.6*	*30,432*	--	*161.1*	SCHIST
BB-IWBP-202	R2	451.0	4.8	-	8.8	2.7	2.1	-	6.1	4.0	43	90%	17	39%	<0.75-8	0.004-0.4	446.2	442.2							SCHIST
BB-IWBP-202	R3	451.0	8.8	-	10.0	2.7	6.1	-	7.3	1.2	12	86%	0	0%	<0.75	0.01-0.4	442.2	441.0							SCHIST
BB-IWBP-202	R4	451.0	10.0	-	12.0	2.7	7.3	-	9.3	2.0	22	92%	4	18%	<0.75-8	0.004-0.4	441.0	439.0							SCHIST
BB-IWBP-202	R5	451.0	12.0	-	15.5	2.7	9.3	-	12.8	3.5	42	100%	9	21%	<0.75-24	0.004-0.4	439.0	435.5							SCHIST
BB-IWBP-202	R6	451.0	15.5	-	17.8	2.7	12.8	-	15.1	2.3	21	75%	8	36%	<0.75-8	0.004-0.4	435.5	433.2	16.0	13.3	435.0	17,351	1,190	166	SCHIST
BB-IWBP-202	R7	451.0	17.8	-	20.8	2.7	15.1	-	18.1	3.0	32	89%	0	0%	<0.75-8	0.004-0.4	433.2	430.2							SCHIST
BB-IWBP-202	R8	451.0	20.8	-	22.2	2.7	18.1	-	19.5	1.4	16	94%	4	25%	<0.75-8	0.004-0.4	430.2	428.8							SCHIST
BB-IWBP-202	R9	451.0	22.2	-	24.2	2.7	19.5	-	21.5	2.0	22	92%	5	23%	<0.75-8	0.004-0.4	428.8	426.8							SCHIST
BB-IWBP-203	R1	449.8	4.7	-	5.9	4.5	0.2		1.4	1.2	11	79%	0	0%	<0.75-8	0.004-0.4	445.1	443.9							SCHIST
BB-IWBP-203	R2	449.8	5.9	-	7.7	4.5	1.4		3.2	1.8	16	73%	6	34%	<0.75-8	0.004-0.4	443.9	442.1	6.7	2.2	443.1	25,992	4,530	164.8	SCHIST
BB-IWBP-203	R3	449.8	7.7	-	9.7	4.5	3.2		5.2	2.0	16	67%	0	0%	<0.75	0.01-0.4	442.1	440.1							SCHIST
BB-IWBP-204	R1	474.3	27.6	-	30.5	27.6	0.0		2.9	2.9	10	29%	0	0%	<0.75	0.004-0.4	446.7	443.8							SCHIST
BB-IWBP-204	R2	474.3	30.5	-	31.7	27.6	2.9		4.1	1.2	11	79%	0	0%	<0.75	0.01-0.4	443.8	442.6	30.8	-3.2	443.5	23,472	--	162.9	SCHIST
BB-IWBP-204	R3	474.3	31.7	-	34.0	27.6	4.1		6.4	2.3	15	54%	0	0%	<0.75	0.02-0.4	442.6	440.3							SCHIST
BB-IWBP-204	R4	474.3	34.7	-	36.6	27.6	7.1		9.0	1.9	14	63%	0	0%	<0.75	0.004-0.4	439.6	437.7							SCHIST
BB-IWBP-204	R5	474.3	36.6	-	38.0	27.6	9.0		10.4	1.4	13	76%	0	0%	<0.75	0.004-0.4	437.7	436.3							SCHIST
BB-IWBP-204	R6	474.3	38.0	-	40.2	27.6	10.4		12.6	2.2	22	85%	0	0%	<0.75-0.75	0.004-0.4	436.3	434.1							SCHIST
BB-IWBP-204	R7	474.3	40.2	-	41.6	27.6	12.6		14.0	1.4	14	82%	0	0%	<0.75	0.004-0.4	434.1	432.7							SCHIST
BB-IWBP-204	R8	474.3	41.6	-	44.2	27.6	14.0		16.6	2.6	28	90%	0	0%	<0.75-0.75	0.004-0.4	432.7	430.1							SCHIST
BB-IWBP-204	R9	474.3	44.2	-	46.2	27.6	16.6		18.6	2.0	24	100%	0	0%	<0.75	0.004-0.4	430.1	428.1							SCHIST
BB-IWBP-204	R10	474.3	46.2	-	47.4	27.6	18.6		19.8	1.2	9	64%	0	0%	<0.75	0.01-0.4	428.1	426.9							SCHIST
BB-IWBP-204	R11	474.3	47.4	-	50.5	27.6	19.8		22.9	3.1	37	100%	13	34%	<0.75-24	0.004-0.4	426.9	423.8							SCHIST

Notes: 1. BB-IWBP-105 R1 and R2 started at 22' and 23' bgs. Cored through Fractured Rock. Rolled ahead through fractured rock from R2 to R3 and from R4 to R5.

2. *Italics* lab entries indicate point load tests (BB-IWBP-102 R1, 103 R1, 104 R4, -105 R2, -202 R1, and -204 R2). Stated Unconfined Compressive Strength is based on a correlation between point load index and UCS and is the average of all point load tests for that sample (rounded to nearest 100 psi).

3. Point load test on BB-IWBP-102 R1 was Diametrical only, other samples were tested Diametrical and Axial.

FIGURES



USGS QUADRANGLE LOCATION

N
W E
S

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

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

SCALE IN FEET

Data Supplied by : esri USGS science for a changing world

GZA

PROJ. MGR.: BMC
DESIGNED BY: BMC
REVIEWED BY: ARB
OPERATOR: ADM

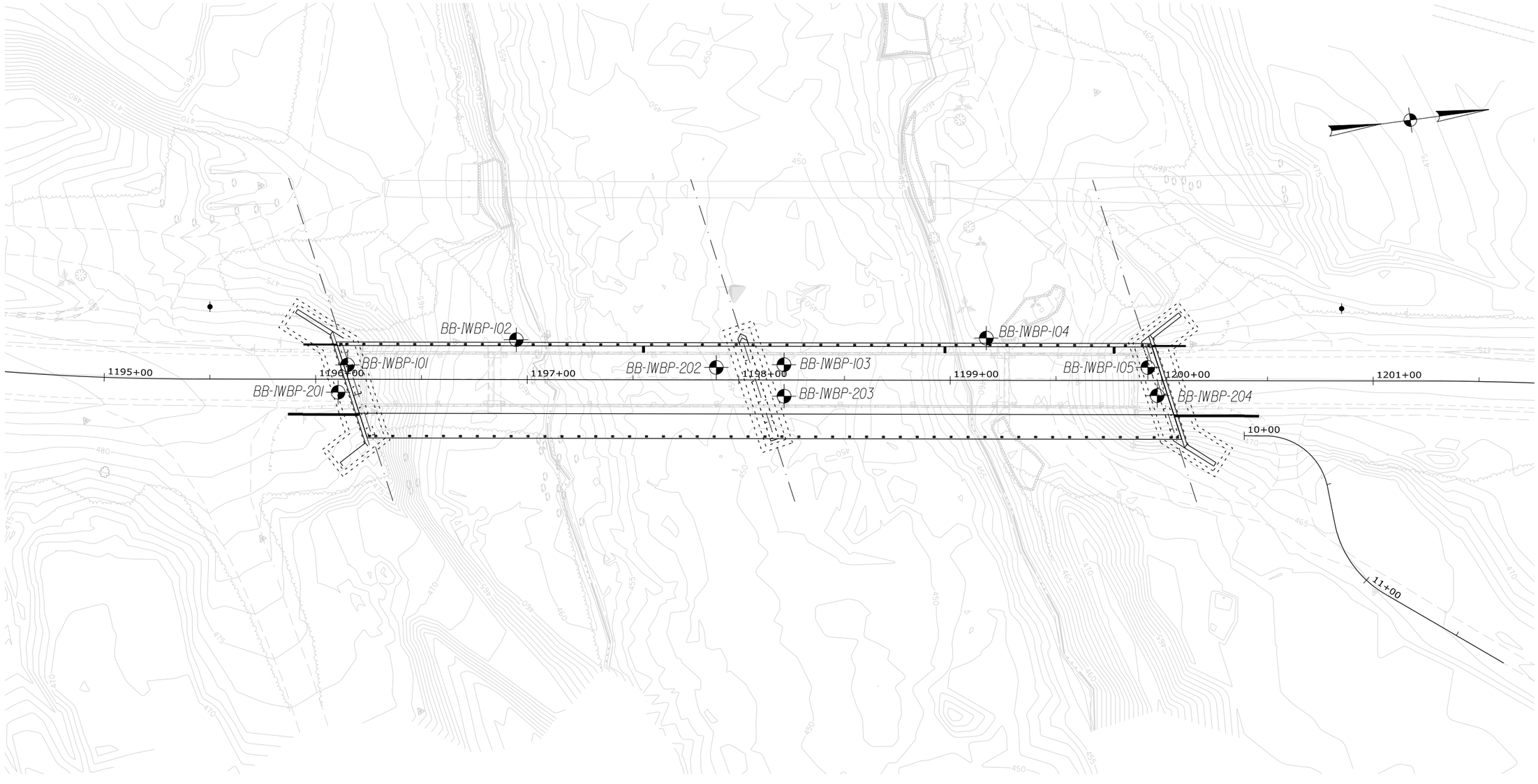
DATE: 06-16-2020

LOCUS PLAN

MAINEDOT DETECTIVE BENJAMIN CAMPBELL BRIDGE
T3 INDIAN PURCHASE TWP

JOB NO. 09.0025976.01

FIGURE NO. 1



NOTES

1) Base map developed from electronic files (3DContours.dgn, 3DTopo.dgn, Alignments.dgn, and Bridge.dgn) provided by HNTB on May 29, 2020.

2) The as-drilled locations of the test borings were surveyed by MaineDOT and provided by HNTB.

LEGEND

BB-IWBP-105 — Indicates borings performed by New England Boring Contractors of Hermon, Maine between May 29, and June 7, 2018 and observed by GZA personnel.

BB-IWBP-204 — Indicates borings performed by New England Boring Contractors of Hermon, Maine between February 25, and March 5, 2020 and observed by GZA personnel.

PLAN

25 0 25 50

Scale

Division: Bridge
Filename: ...\Microstation\BLP\BLP.dgn

Date:7/21/2020
Username: BMC

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

WIN 23236.01

Bridge No. 3666

BRIDGE PLANS

BY	DATE
--	--
--	--

SIGNATURE

P.E. NUMBER

DATE

PROJ. MANAGER	
DESIGN-DETAILED	--
CHECKED-REVIEWED	--
DESIGN2-DETAILED2	
DESIGN3-DETAILED3	
REVISIONS 1	
REVISIONS 2	
REVISIONS 3	
REVISIONS 4	
FIELD CHANGES	

DETECTIVE BENJAMIN CAMPBELL BRIDGE
WEST BRANCH PENOBSCOT RIVER
T3 INDIAN PUR. TWP PENOBSCOT COUNTY

BORING LOCATION PLAN

SHEET NUMBER

10

OF 70

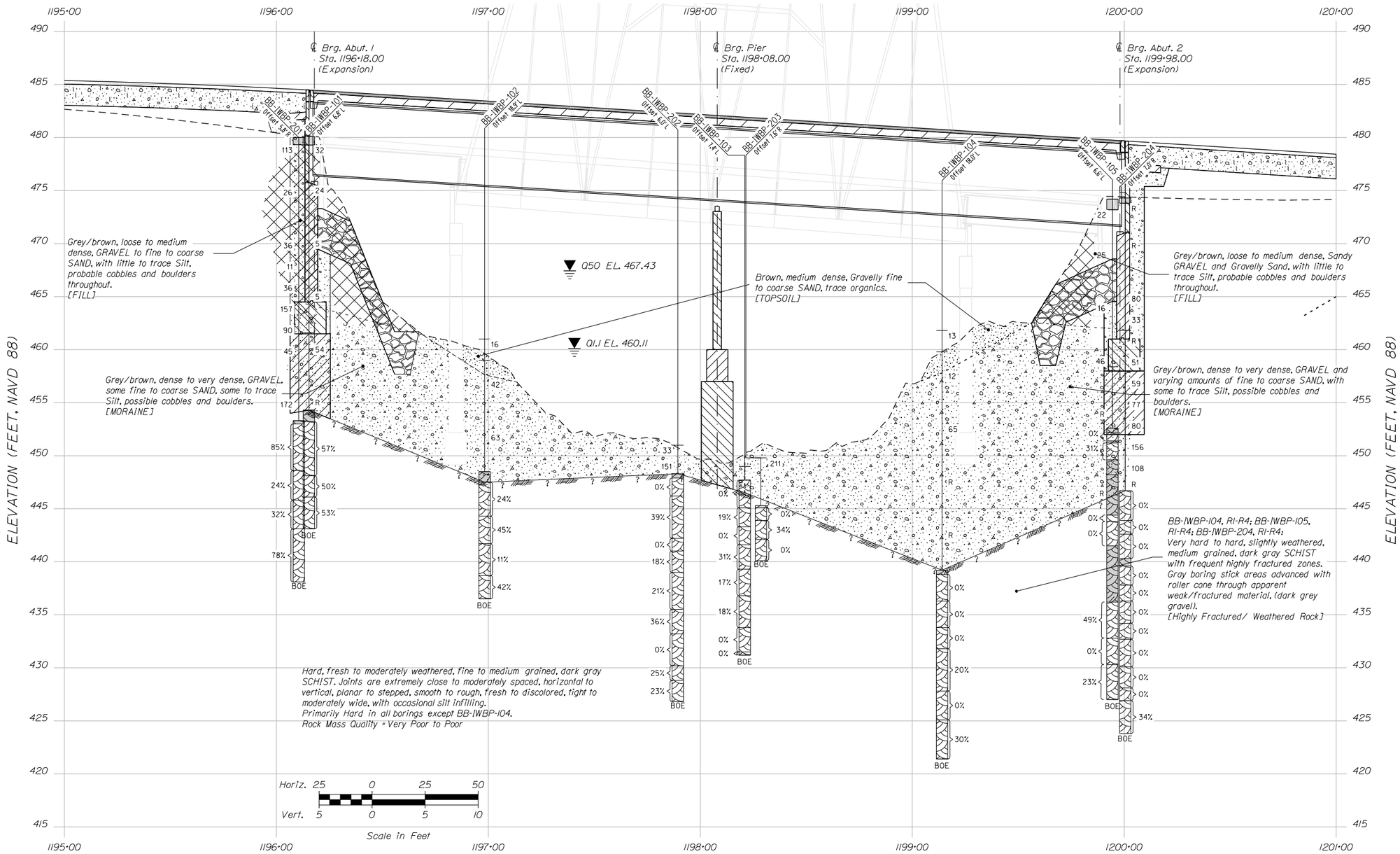
DETECTIVE BENJAMIN CAMPBELL BRIDGE NO. 3666
MAINEDOT WIN 23236.00
T3 INDIAN PURCHASE TOWNSHIP, ME

BORING LOCATION PLAN

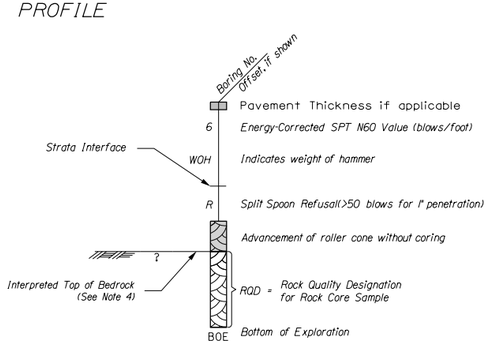
PREPARED BY:
GZA GeoEnvironmental, Inc.
Engineers and Scientists
www.gza.com

PREPARED FOR:
HNTB

PROJ MGR: BMC	REVIEWED BY: ARB	CHECKED BY: CLS	FIG 2
DESIGNED BY: BMC	DRAWN BY: BMC	SCALE: AS SHOWN	
DATE: 6/16/2020	PROJECT NO. 09.0025976.01	REVISION NO. 0	SHEET NO. 2 OF 3



PROFILE



NOTES

1) Base map developed from electronic files provided by HNTB on May 29, 2020 (File included Profile.dgn)

2) The as drilled boring locations were surveyed by MaineDOT and provided by HNTB.

3) Top of highly fractured/Weathered rock and rock is defined using either roller cone resistance prior to identification of bedrock by coring or by transition of rock quality assessed from rock core.

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

DETECTIVE BENJAMIN CAMPBELL BRIDGE NO. 3666
MAINEDOT WIN 23236.00
T3 INDIAN PURCHASE TOWNSHIP, ME

INTERPRETIVE SUBSURFACE PROFILE

PREPARED BY:		PREPARED FOR:	
GZA GeoEnvironmental, Inc. Engineers and Scientists www.gza.com		HNTB	
PROJ MGR: BMC	REVIEWED BY: ARB	CHECKED BY: CLS	FIG 3
DESIGNED BY: BMC	DRAWN BY: BMC	SCALE: AS SHOWN	
DATE: 6/16/2020	PROJECT NO. 09.0025976.01	REVISION NO. 0	SHEET NO. 3 OF 3

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

WIN 23236.01 — BRIDGE PLANS
Bridge No. 3666

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

SIGNATURE

P.E. NUMBER

DATE

DETECTIVE BENJAMIN CAMPBELL BRIDGE
WEST BRANCH PENOBSCOT RIVER
T3 INDIAN PUR. TWP PENOBSCOT COUNTY
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
11
OF 70

Division: Bridge
Filename: ...\ISP\West Branch_ISP.dgn
Date:7/21/2020
Username: BMC

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: Detective Benjamin Campbell Bridge #3666, West Branch Penobscot River
Location: T3 Indian Purchase Township, Maine

Boring No.: BB-IWBP-101

PIN: 23236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	480.1	**Auger ID/OD:**	4.25" SSA
Operator:	T. Schafer	**Datum:**	NAVD 88	**Sampler:**	Standard
Logged By:	E. Friede	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5-31-18/5-31-18	**Drilling Method:**	Drive/Spin & Wash	**Core Barrel:**	NQ
Boring Location:	N779154.0, E1730288.5	**Casing ID/OD:**	4"/4.5", 3"/3.5"	**Water Level*:**	Not Measured
Hammer Efficiency Factor: 0.6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	479.3		0'-0.8': Asphalt (0.8)	
	1D	24/11	1.0 - 3.0	12-17-15-25	32	32				Brown, moist, dense, Gravelly fine to coarse SAND, little silt, (Fill).	G#1 A-1-a SP-SM WC=4.3%
5	2D	24/13	5.5 - 7.5	5-12-12-15	24	24	29			Brown, moist, medium dense, Gravelly fine to coarse SAND, trace silt, (Fill). Increased roller cone resistance between 7.0'-8.0', possible cobbles.	G#2 A-1-a SP-SM WC=3%
							65				
							40				
							16				
							30				
10	3D	24/0	10.0 - 12.0	3-2-3-3	5	5	16			No recovery. Drove 3" OD spoon through interval. Brown, moist, loose, GRAVEL, little medium to coarse sand, trace silt, (Fill).	
							17				
							16				
							13				
							15				
15	4D	24/4	15.0 - 17.0	4-3-2-2	5	5	12			Brown, moist, loose, fine to coarse Sandy GRAVEL, trace silt, (Fill).	
							12				
							12				
							19	461.6		(18.5)	
							50				
20	5D	24/11	20.0 - 22.0	21-32-22-26	54	54	2			Brown-grey, moist, very dense, fine to coarse GRAVEL, some fine to medium sand,little silt, gravel is angular, (Moraine).	G#3 A-1-a GM WC=5.5%
							17				
							18				
							29				
25							36				

Remarks:

1. Drove 4" casing to 26.0', spun 3" casing to 26.9'.
2. No water return when spinning 3" casing.
3. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterburg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.

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: Detective Benjamin Campbell Bridge #3666, West Branch Penobscot River
Location: T3 Indian Purchase Township, Maine

Boring No.: BB-IWBP-101

PIN: 23236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	480.1	**Auger ID/OD:**	4.25" SSA
Operator:	T. Schafer	**Datum:**	NAVD 88	**Sampler:**	Standard
Logged By:	E. Friede	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5-31-18/5-31-18	**Drilling Method:**	Drive/Spin & Wash	**Core Barrel:**	NQ
Boring Location:	N779154.0, E1730288.5	**Casing ID/OD:**	4"/4.5", 3"/3.5"	**Water Level*:**	Not Measured
Hammer Efficiency Factor: 0.6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	10/8	25.0 - 25.8	23-50/4"	-		18	454.3		Brown-grey, wet, very dense, GRAVEL, some fine to coarse sand, trace silt, (Moraine).	
										25.8	
	R1	61/61	26.9 - 32.0	RQD = 57%			SPIN NQ			Increased resistance at 25.8', probable top of bedrock. Spin 3" casing to 26.9' and core. R1: Hard, fresh, medium grained, dark grey, SCHIST. Joints are extremely close to moderately spaced, low to high angle, planar to stepped, smooth to rough, fresh to slightly discolored, tight to open, no infilling. -CARRABASSETT FORMATION- Rock Quality = Fair Recovery = 100% Rock Core Times (min:sec): 26.9-28.0' (2:48), 28.0-29.0' (2:41), 29.0-30.0' (2:21), 30.0-31.0' (2:20), 31.0-32.0' (2:30)	q_p=2,687
30											
	R2	24/20	32.0 - 34.0	RQD = 50%						R2: Hard, fresh, medium grained, dark grey, SCHIST. Joints are close, low to high angle, planar, smooth, fresh, tight, grey silt infilling. Rock Quality = Poor Recovery = 83% Rock Core Times (min:sec): 32.0-33.0' (2:30), 33.0-34.0' (2:50)	
	R3	36/36	34.0 - 37.0	RQD = 53%						R3: Hard, fresh, medium grained, dark grey, SCHIST. Joints are extremely close to moderately spaced, low to high angle, planar to stepped, smooth, fresh to discolored, tight to moderately wide, no infilling. Rock Quality = Fair Recovery = 100% Rock Core Times (min:sec): 34.0-35.0' (2:45), 35.0-36.0' (2:22), 36.0-37.0' (2:32)	
35								443.1			
										37.0	
40										**Bottom of Exploration at 37.00 feet below ground surface.**	
45											
50											

Remarks:

1. Drove 4" casing to 26.0', spun 3" casing to 26.9'.
2. No water return when spinning 3" casing.
3. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterburg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.

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: Detective Benjamin Campbell Bridge #3666, West Branch Penobscot River
Location: T3 Indian Purchase Township, Maine

Boring No.: BB-IWBP-102

PIN: 23236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	461.0	**Auger ID/OD:**	4.25" SSA
Operator:	T. Schafer	**Datum:**	NAVD 88	**Sampler:**	Standard
Logged By:	E. Friede/B. Cardali	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6-6-18/6-7-18	**Drilling Method:**	Spin & Wash	**Core Barrel:**	NQ
Boring Location:	N779234.7, E1730287.3	**Casing ID/OD:**	4"/4.5", 3"/3.5"	**Water Level*:**	Not Measured
Hammer Efficiency Factor: 0.6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/8	0.0 - 2.0	2-8-8-20	16	16	SSA			Brown, moist, medium dense, medium to coarse SAND, little gravel, trace organics (roots, leaves), (Topsoil).	
								459.0		2.0	
							SPIN			Heavy grinding to 5.0' with roller cone, possible cobbles.	
5	2D	24/4	5.0 - 7.0	17-26-16-15	42	42	RC			Grey, wet, dense, fine to coarse GRAVEL, some fine to coarse sand, trace silt, (Moraine).	G#4 A-1-a GW WC=3.6%
							SPIN				
							RC				
10	3D	24/8	10.0 - 12.0	12-24-39-38	63	63				Light brown, wet, very dense, fine to coarse GRAVEL, little fine to coarse sand, little silt, gravel is angular, (Moraine).	G#5 A-1-a GP-GM WC=4.6%
								449.1		11.9	
							NQ	448.5		Increased resistance at 11.9', brown, fractured rock and quartz fragments in wash return, advance 4" casing.	
	R1	38/38	13.5 - 16.7	RQD = 24%						12.5	
15										Consistent increased roller cone resistance at 12.5' and rock chips in return, probable top of bedrock. Spin 3" casing to 13.5' and core. R1: Hard, slightly weathered, fine grained, grey, SCHIST. Joints are close to extremely close (highly fractured from 16.0'-16.7'), low to high angle, planar, smooth to rough, discolored to slightly decomposed, tight to moderately wide, grey clay infilling. -CARRABASSETT FORMATION- Rock Quality = Very Poor Recovery = 100% Rock Core Times (min:sec): 13.5-14.5' (2:22), 14.5-15.5' (2:23), 15.5-16.5' (1:54), 16.5-16.7' (0:29)	S_c=3,471, 2,653 (PLD)
	R2	31/31	16.7 - 19.3	RQD = 45%						R2: Hard, slightly weathered, fine grained, grey, SCHIST. Joints are very close to moderately spaced, low to high angle, planar, smooth to rough, discolored, open to moderately wide, no infilling. Rock Quality = Poor Recovery = 100% Rock Core Times (min:sec): 16.7-17.7' (1:54), 17.7-18.7' (1:40), 18.7-19.3' (1:06)	
20	R3	36/32	19.3 - 22.3	RQD = 11%						R3: Hard, slightly weathered, fine grained, grey, SCHIST. Joints are extremely close to very close, low to high angle, planar, smooth to rough, discolored, tight to	
	R4	26/26	22.3 - 24.5	RQD = 42%							
25								436.5			

Remarks:

1. Auger refusal at 3.8' prior to casing advancement.
2. PLD indicates diametral point load test performed. S_c indicates estimated unconfined compressive strength per ASTM D5731.
3. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterburg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.

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: Detective Benjamin Campbell Bridge #3666, West Branch Penobscot River
Location: T3 Indian Purchase Township, Maine

Boring No.: BB-IWBP-102

PIN: 23236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	461.0	**Auger ID/OD:**	4.25" SSA
Operator:	T. Schafer	**Datum:**	NAVD 88	**Sampler:**	Standard
Logged By:	E. Friede/B. Cardali	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6-6-18/6-7-18	**Drilling Method:**	Spin & Wash	**Core Barrel:**	NQ
Boring Location:	N779234.7, E1730287.3	**Casing ID/OD:**	4"/4.5", 3"/3.5"	**Water Level*:**	Not Measured
Hammer Efficiency Factor: 0.6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample Information: 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										moderately wide, with occasional silt infilling. Rock Quality = Very Poor Recovery = 89% Rock Core Times (min:sec): 19.3-20.3' (2:37), 20.3-21.3' (2:09), 21.3-22.3' (1:25) R4: Hard, slightly weathered, fine grained, grey, SCHIST with a 2" quartz seam at 23.0'-23.2'. Joints are very close to close, low angle to moderately dipping, planar to undulating, rough to smooth, discolored, with silt infilling, partially open to wide. Rock Quality = Poor Recovery = 100% Rock Core Times (min:sec): 22.3-23.3' (3:13), 23.3-24.3' (3:06), 24.3-24.5' (0:31)	
										24.5 **Bottom of Exploration at 24.50 feet below ground surface.**	
30											
35											
40											
45											
50											

Remarks:

1. Auger refusal at 3.8' prior to casing advancement.
2. PLD indicates diametral point load test performed. S_c indicates estimated unconfined compressive strength per ASTM D5731.
3. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterburg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.

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: Detective Benjamin Campbell Bridge #3666, West Branch Penobscot River
Location: T3 Indian Purchase Township, Maine

Boring No.: BB-IWBP-103

PIN: 23236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	449.0	**Auger ID/OD:**	4.25" SSA
Operator:	T. Schafer	**Datum:**	NAVD 88	**Sampler:**	Standard
Logged By:	E. Friede	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6-5-18/6-5-18	**Drilling Method:**	Spin & Wash	**Core Barrel:**	NQ
Boring Location:	N779358.6, E1730315.9	**Casing ID/OD:**	4"/4.5", 3"/3.5"	**Water Level*:**	River
Hammer Efficiency Factor: 0.6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SPIN			Spin 4" casing to 0.3' to seat. Heavy grind with roller cone to 1.2', consistent rock fragments in wash return. Telescope 3" casing to 1.3'.	
	R1	31/26	1.3 - 3.9	RQD = 0%			NQ	447.7		1.3- R1: Hard, fresh, fine grained, dark grey, SCHIST. Joints are extremely close to close, horizontal to high angle, fresh to discolored, planar, smooth to rough, tight to open, gray silt infilling. -CARRABASSETT FORMATION- Rock Quality = Very Poor Recovery = 84% Rock Core Times (min:sec): 1.3-2.3' (1:50), 2.3-3.3' (1:45), 3.3-3.9' (1:42)	
	R2	21/21	3.9 - 5.7	RQD = 19%						R2: Hard, fresh, fine grained, dark grey, SCHIST. Joints are extremely close to close, horizontal to high angle, fresh to discolored, planar, smooth to rough, tight to open, gray silt infilling. Rock Quality = Very Poor Recovery = 100% Rock Core Times (min:sec): 3.9-4.9' (2:36), 4.9-5.7' (1:58)	
5	R3	19/19	5.7 - 7.3	RQD = 0%						R3: Hard, fresh, fine grained, dark grey, SCHIST. Joints are extremely close to close, horizontal to high angle, fresh to discolored, planar, smooth to rough, tight to open, gray silt infilling. Rock Quality = Very Poor Recovery = 100% Rock Core Times (min:sec): 5.7-6.7' (2:26), 6.7-7.3' (1:20)	
	R4	29/25	7.3 - 9.7	RQD = 31%						R4: Hard to medium hard, fresh to highly weathered (at 9.1'), fine grained, dark grey, SCHIST. Joints are extremely close to close, low to high angle, planar to undulating, rough to smooth, open to moderately wide, with silt infilling approximately 1/4" thick at 9.1'. Rock Quality = Poor Recovery = 86% Rock Core Times (min:sec): 7.3-8.3' (2:36), 8.3-9.3' (1:57), 9.3-9.7' (0:48)	
	R5	30/30	9.7 - 12.2	RQD = 17%						R5: Hard, fresh to slightly weathered, fine grained, dark grey, SCHIST. Joints are very close to close, low angle to vertical, undulating to planar, rough, fresh to discolored, tight to moderately wide, trace silt infilling. Rock Quality = Very Poor Recovery = 100% Rock Core Times (min:sec): 9.7-10.7' (1:48), 10.7-11.7' (2:06), 11.7-12.2' (0:42)	
10											
	R6	36/33	12.2 - 15.2	RQD = 18%						R6: Hard, slightly weathered, medium grained, grey, SCHIST. Joints are extremely close to close, low to high angle, undulating, rough, discolored (black and orange spotted staining), moderately wide, grey clay infilling. Rock Quality = Very Poor Recovery = 92% Rock Core Times (min:sec): Not measured.	
15	R7	28/22	15.2 - 17.5	RQD = 0%						R7: Hard, moderately weathered, medium grained, grey,	
	R8	4/2	17.5 - 17.8	RQD = 0%				431.2			
20											
25											

Remarks:

1. Boring was drilled through 0.8' thick concrete bridge deck. Mudline was 27.8' below top of deck. Boring depths are referenced to the mudline. Mudline elevation calculated using the surveyed bridge deck elevation and taped distance to mudline.
2. River water level was 9.5' above mudline.
3. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterburg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.

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: Detective Benjamin Campbell Bridge #3666, West Branch Penobscot River
Location: T3 Indian Purchase Township, Maine

Boring No.: BB-IWBP-103

PIN: 23236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	449.0	**Auger ID/OD:**	4.25" SSA
Operator:	T. Schafer	**Datum:**	NAVD 88	**Sampler:**	Standard
Logged By:	E. Friede	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6-5-18/6-5-18	**Drilling Method:**	Spin & Wash	**Core Barrel:**	NQ
Boring Location:	N779358.6, E1730315.9	**Casing ID/OD:**	4"/4.5", 3"/3.5"	**Water Level*:**	River
Hammer Efficiency Factor: 0.6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample Information: 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										SCHIST. Joints are extremely close to very close, low to high angle, undulating, rough, discolored to decomposed, moderately wide, grey clay infilling. Rock Quality = Very Poor Recovery = 79% Rock Core Times (min:sec): 15.2-16.2' (3:24), 16.2-17.2' (2:26), 17.2-17.5' (0:36) R8: Hard, slightly weathered, medium grained, grey SCHIST. Joints are extremely close, highly fractured with grey clay infilling. Rock Quality = Very Poor Recovery = 50%	
30										17.8 **Bottom of Exploration at 17.80 feet below ground surface.**	
35											
40											
45											
50											

Remarks:

1. Boring was drilled through 0.8' thick concrete bridge deck. Mudline was 27.8' below top of deck. Boring depths are referenced to the mudline. Mudline elevation calculated using the surveyed bridge deck elevation and taped distance to mudline.
2. River water level was 9.5' above mudline.
3. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterburg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.

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: Detective Benjamin Campbell Bridge #3666, West Branch Penobscot River
Location: T3 Indian Purchase Township, Maine

Boring No.: BB-IWBP-104

PIN: 23236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	461.6	**Auger ID/OD:**	4.25" SSA
Operator:	T. Schafer	**Datum:**	NAVD 88	**Sampler:**	Standard
Logged By:	E. Friede	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6-4-18/6-5-18	**Drilling Method:**	Spin & Wash	**Core Barrel:**	NQ
Boring Location:	N779455.1, E1730316.2	**Casing ID/OD:**	4"/4.5", 3"/3.5"	**Water Level*:**	Not Measured
Hammer Efficiency Factor: 0.6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/8	0.0 - 2.0	4-4-9-12	13	13	SSA			Brown, moist, medium dense, Gravelly fine to coarse SAND, trace silt, trace organics (roots), (Topsoil).	G#6 A-1-a SP WC=4.7%
								459.6		2.0	
							SPIN				
5	2D	24/0	5.7 - 7.7	4-5-7-6	12	12				No recovery. Drove 3" OD spoon through interval. Grey, wet, medium dense, Sandy fine to coarse GRAVEL, trace silt, (Moraine).	G#7 A-1-a GP WC=7.7%
								452.6		9.0	
										Heavy grinding with roller cone at 7.8', no water return, possible cobble/boulder.	
10	3D	20/4	10.4 - 12.1	16-26-39-58/2"	65	65	RC			Brown, wet, very dense, GRAVEL, some fine to coarse sand, trace silt, (Moraine).	
15	4D	10/10	15.0 - 15.8	47-100/4"	R					Light brown, moist, very dense, Gravelly fine to coarse SAND, some silt, (Moraine).	G#8 A-1-b SM WC=11.6%
20	5D	4/3	20.0 - 20.3	100/4"	R					Brown, wet, very dense GRAVEL, trace silt, trace fine sand. Gravel is angular, 1" of fractured rock in spoon tip, (Moraine).	
								439.0		22.6	
	R1	30/30	23.0 - 25.5	RQD = 0%			NQ			Increased roller cone resistance and rock fragments in wash return at 22.6', apparent top of highly fractured bedrock. Spin casing to 23.0' and core. R1: Hard, slightly to moderately weathered, fine grained, dark gray to brown, SCHIST. Highly fractured. Joints are	
25											

Remarks:

1. Heavy grinding at 3.7' indicative of cobbles. Switched to Spin & Wash with 4" casing at 3.7' and 3" casing at 12.1'.
2. Heavy, consistent grinding at 7.8', no wash water return.
3. PLD, PLA indicate diametral and axial point load tests performed. S_c indicates estimated unconfined compressive strength per ASTM D5731.
4. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterburg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.

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: Detective Benjamin Campbell Bridge #3666, West Branch Penobscot River
Location: T3 Indian Purchase Township, Maine

Boring No.: BB-IWBP-104

PIN: 23236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	461.6	**Auger ID/OD:**	4.25" SSA
Operator:	T. Schafer	**Datum:**	NAVD 88	**Sampler:**	Standard
Logged By:	E. Friede	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6-4-18/6-5-18	**Drilling Method:**	Spin & Wash	**Core Barrel:**	NQ
Boring Location:	N779455.1, E1730316.2	**Casing ID/OD:**	4"/4.5", 3"/3.5"	**Water Level*:**	Not Measured
Hammer Efficiency Factor: 0.6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	R2	30/30	25.5 - 28.0	RQD = 0%						extremely close to very close, low angle to high angle, planar, rough to smooth, discolored and decomposed, open, brown silt/clay infilling. -CARRABASSETT FORMATION- (Fractured/ Weathered Rock) Rock Quality = Very Poor Recovery = 100% Rock Core Times (min:sec): 23.0-24.0' (2:24), 24.0-25.0' (2:34), 25.0-25.5' (1:03)	
	R3	24/18	28.0 - 30.0	RQD = 0%						R2: Hard, moderately weathered, fine grained, grey-brown, SCHIST (Fractured/Weathered Rock). Joints are extremely close to very close, low angle to high angle, undulating, rough, decomposed, open, with brown silt infilling. Rock Quality = Very Poor Recovery = 100% Rock Core Times (min:sec): 25.5-26.5' (4:04), 26.5-27.5' (2:02), 27.5-28.0' (1:54)	
30	R4	48/42	30.0 - 34.0	RQD = 20%				429.3		R3: Hard, moderately weathered, fine grained, grey-brown, SCHIST (Fractured/Weathered Rock). Joints are extremely close to very close, low angle to high angle, undulating, rough, decomposed, open, with brown silt infilling. Rock Quality = Very Poor Recovery = 75% Rock Core Times (min:sec): 28.0-29.0' (2:20), 29.0-30.0' (3:18)	S_c=1,609, 1,071 (PLD) S_c=4,645, 3,129 (PLA)
	R5	32/32	34.0 - 36.7	RQD = 0%						R4: Hard, fresh, fine to medium grained, grey-white banded, SCHIST. 30.0'-32.3': Fractured/Weathered Rock.	
35	R6	44/44	36.7 - 40.4	RQD = 30%						32.3 R4: 32.3'-34.0': Joints are very close to close, low angle, smooth, planar, discolored, tight, no infilling. -CARRABASSETT FORMATION- Rock Quality = Very Poor Recovery: 88% Rock Core Times (min:sec): 30.0-31.0' (1:42), 31.0-32.0' (3:20), 32.0-33.0' (2:03), 33.0-34.0' (3:01)	
40								421.2		R5: Hard, slightly weathered, fine to medium grained, grey/white banded, SCHIST. Joints are extremely close to close, low to high angle, planar to undulating, rough, discolored to decomposed, open. Trace brown silt infilling. Rock Quality = Very Poor Recovery = 100% Rock Core Times (min:sec): 34.0-35.0' (2:35), 35.0-36.0 (2:34), 36.0-36.7' (2:00)	
45										R6: Hard, fresh to slightly weathered, fine to medium grained, grey/white banded, SCHIST. 36.7'-39.4': Joints are very close to close, low to high angle, planar, smooth, discolored to decomposed, tight to open, grey clay infilling. 39.4'-40.4': Highly fractured, grey clay infilling. Rock Quality = Poor Recovery = 100%	
50											

Remarks:

1. Heavy grinding at 3.7' indicative of cobbles. Switched to Spin & Wash with 4" casing at 3.7' and 3" casing at 12.1'.
2. Heavy, consistent grinding at 7.8', no wash water return.
3. PLD, PLA indicate diametral and axial point load tests performed. S_c indicates estimated unconfined compressive strength per ASTM D5731.
4. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterburg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.

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: Detective Benjamin Campbell Bridge #3666, West Branch Penobscot River
Location: T3 Indian Purchase Township, Maine

Boring No.: BB-IWBP-104

PIN: 23236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	461.6	**Auger ID/OD:**	4.25" SSA
Operator:	T. Schafer	**Datum:**	NAVD 88	**Sampler:**	Standard
Logged By:	E. Friede	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6-4-18/6-5-18	**Drilling Method:**	Spin & Wash	**Core Barrel:**	NQ
Boring Location:	N779455.1, E1730316.2	**Casing ID/OD:**	4"/4.5", 3"/3.5"	**Water Level*:**	Not Measured
Hammer Efficiency Factor: 0.6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										Rock Core Times (min:sec): 36.7-37.7' (2:48), 37.7-38.7' (2:05), 38.7-39.7' (2:05), 39.7-40.4' (1:44)	
										40.4	
										Bottom of Exploration at 40.40 feet below ground surface.	
55											
60											
65											
70											
75											

Remarks:

1. Heavy grinding at 3.7' indicative of cobbles. Switched to Spin & Wash with 4" casing at 3.7' and 3" casing at 12.1'.
2. Heavy, consistent grinding at 7.8', no wash water return.
3. PLD, PLA indicate diametral and axial point load tests performed. S_c indicates estimated unconfined compressive strength per ASTM D5731.
4. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterburg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.

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: Detective Benjamin Campbell Bridge #3666, West Branch Penobscot River
Location: T3 Indian Purchase Township, Maine

Boring No.: BB-IWBP-105

PIN: 23236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	474.2	**Auger ID/OD:**	4.25" SSA
Operator:	T. Schafer	**Datum:**	NAVD 88	**Sampler:**	Standard
Logged By:	E. Friede	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5-29-18/5-29-18	**Drilling Method:**	Drive/Spin & Wash	**Core Barrel:**	NQ
Boring Location:	N779528.9, E1730340.1	**Casing ID/OD:**	4"/4.5", 3"/3.5"	**Water Level*:**	Not Measured
Hammer Efficiency Factor: 0.6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			0'-1.0': Asphalt. Heavy grinding, large cobbles.	
								473.2		1.0	
	1D	24/10	2.0 - 4.0	8-11-11-20	22	22				Grey-brown, dry, medium dense, Sandy fine to coarse GRAVEL, trace silt, (Fill).	G#9 A-1-a GW-GM WC=1.9%
5	2D	24/8	5.0 - 7.0	10-7-18-13	25	25	RC			Brown, moist, medium dense, Sandy fine to coarse GRAVEL, trace silt, (Fill).	G#10 A-1-a GW-GM WC=2.8%
10	3D	24/2	10.0 - 12.0	10-9-7-7	16	16				Dark grey, moist, medium dense, GRAVEL, trace fine to coarse sand, trace silt. Gravel is angular, (Fill).	
							26				
							18				
							19				
							29				
15	4D	24/12	15.5 - 17.5	11-21-25-25	46	46	RC	458.4		Top 4": Dark grey, wet, GRAVEL, trace fine to coarse sand, trace silt. Gravel is angular, (Fill).	G#11 A-1-a GM WC=6.9%
										15.8 Bottom 8": Light brown and grey mottled, wet, dense, fine to coarse GRAVEL, some fine to coarse sand, little silt, (Moraine).	
20	5D	14/1	20.0 - 21.2	10-10-50/2"	R		SPIN			Dark grey, wet, very dense, GRAVEL, trace silt, trace fine to coarse sand, (Moraine).	
								452.6		21.6	
	R1	9/1	22.0 - 22.8	RQD = 0%				452.1		Consistent increased roller cone resistance at 21.6'. Apparent top of highly fractured/weathered rock. Spin 3" casing and advance roller cone to 22.0', and set up to core.	
	R2	19/13	23.0 - 24.6	RQD = 31%				451.2		R1: Very hard, slightly weathered, medium grained, dark grey, SCHIST (Fractured/Weathered Rock). Joints are extremely close.	S_c=4,088, 4,809 (PLD)
25								450.1			S_c=3,899,

Remarks:

1. Hole cave at 5.0'; advance HSA to 5.0'; switch to Drive & Wash; advance 4" casing to 21.3'; advance 3" casing to 38.0'.
2. PLD, PLA indicate diametral and axial point load tests performed. S_c indicates estimated unconfined compressive strength per ASTM D5731.
3. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterburg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.

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: Detective Benjamin Campbell Bridge #3666, West Branch Penobscot River
Location: T3 Indian Purchase Township, Maine

Boring No.: BB-IWBP-105

PIN: 23236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	474.2	**Auger ID/OD:**	4.25" SSA
Operator:	T. Schafer	**Datum:**	NAVD 88	**Sampler:**	Standard
Logged By:	E. Friede	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5-29-18/5-29-18	**Drilling Method:**	Drive/Spin & Wash	**Core Barrel:**	NQ
Boring Location:	N779528.9, E1730340.1	**Casing ID/OD:**	4"/4.5", 3"/3.5"	**Water Level*:**	Not Measured
Hammer Efficiency Factor: 0.6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										Recovery = 11% Rock Core Times (min:sec): 22.0-22.8' (3:58)	1,421 (PLA)
	6D	16/2	26.0 - 27.3	24-39-50/4"	R					22.1 Hole caved-in. Advanced 3" casing to 23.0', no wash water return. Possible highly fractured/weathered rock.	
										23.0 R2: Very hard, slightly weathered, medium grained, dark grey, SCHIST (Fractured/Weathered Rock). Joints are close, low to high angle, undulating, smooth to rough, discolored, open, brown silt infilling at bottom.	
	R3	7/7	29.8 - 30.4	RQD = 0%			NQ			Rock Quality = Poor Recovery = 68%	
30	R4	28/11	30.4 - 32.7	RQD = 0%			SPIN	444.4		Rock Core Times (min:sec): 23.0-24.0' (2:20), 24.0-24.4' (0:39)	
								442.9		24.1 Advanced roller cone through apparent fractured/ weathered rock to 26.0'.	
										Dark grey, wet, very dense, GRAVEL, trace clay, trace fine to coarse sand, (Fractured/Weathered Rock).	
										Advance roller cone through apparent fractured/ weathered rock to 29.8', set up to core.	
35										29.8 R3: Hard, slightly weathered, medium grained, dark grey, SCHIST (Fractured/ Weathered Rock). Joints are extremely close, low angle to vertical, undulating, rough, discolored, moderately wide, moderate grey-brown clay infilling.	
										Rock Quality = Very Poor Recovery = 100%	
								436.3		Rock Core Times (min:sec): 29.8-30.4' (4:11)	q_p=2,665
	R5	41/40	38.0 - 41.4	RQD = 49%			NQ			R4 (top): Hard, slightly weathered, medium grained, dark grey, SCHIST (Fractured/Weathered Rock). Joints are extremely close, low to high angle, undulating, rough, discolored, moderately wide, brown clay infilling (>1/ 4"). Residual soil plug at bottom.	
40										Rock Quality = Very Poor Recovery = 39%	
	R6	30/30	41.4 - 43.9	RQD = 0%						Rock Core Times (min:sec): 30.4-31.4' (2:15), 31.4-32.4' (1:56), 32.4-32.7' (1:50)	
										Sample consistent with R3. Recovery sample assumed to be from top of core.	
	R7	39/39	43.9 - 47.2	RQD = 23%						31.3 No recovery from bottom of R4 Core.	
45										Advance roller cone after R4 with low resistance. Clay and rock fragments observed in wash return. Possible highly fractured/weathered rock.	
										37.9 Consistent increased roller cone resistance at 37.9'. Spin 3" casing and advance roller cone to 38.0' and set up to core.	
								427.0		R5: Hard, slightly weathered, fine grained, dark grey, SCHIST. Joints are extremely close to moderately spaced, low angle to moderately dipping, planar, smooth, discolored, tight to open, moderate clay infilling.	
50										-CARRABASSETT FORMATION-	

Remarks:

1. Hole cave at 5.0'; advance HSA to 5.0'; switch to Drive & Wash; advance 4" casing to 21.3'; advance 3" casing to 38.0'.
2. PLD, PLA indicate diametral and axial point load tests performed. S_c indicates estimated unconfined compressive strength per ASTM D5731.
3. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterburg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.

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: Detective Benjamin Campbell Bridge #3666, West Branch Penobscot River
Location: T3 Indian Purchase Township, Maine

Boring No.: BB-IWBP-105

PIN: 23236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	474.2	**Auger ID/OD:**	4.25" SSA
Operator:	T. Schafer	**Datum:**	NAVD 88	**Sampler:**	Standard
Logged By:	E. Friede	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5-29-18/5-29-18	**Drilling Method:**	Drive/Spin & Wash	**Core Barrel:**	NQ
Boring Location:	N779528.9, E1730340.1	**Casing ID/OD:**	4"/4.5", 3"/3.5"	**Water Level*:**	Not Measured
Hammer Efficiency Factor: 0.6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										Rock Quality = Poor Recovery = 98% Rock Core Times (min:sec): 38.0-39.0' (2:41), 39.0-40.0' (2:06), 40.0-41.0' (2:45), 41.0-41.4' (0:34) R6: Hard, slightly weathered, fine grained, dark grey and brown, SCHIST. Joints are extremely close to close, low to high angle, planar and smooth to undulating and rough, discolored to decomposed, tight to open, rusty brown, silt infilling. Rock Quality = Very Poor Recovery = 100% Rock Core Times (min:sec): 41.4-42.4' (1:50), 42.4-43.4' (1:48), 43.4-43.9' (1:18) R7: Hard, slightly weathered, medium grained, dark grey and white banded SCHIST. Joints are extremely close to close, low to high angle, planar to undulating, smooth to rough, discolored, tight to moderately wide, trace brown silt infilling. Rock Quality = Poor Recovery = 100% Rock Core Times (min:sec): 43.9-44.9' (1:43), 44.9-45.9' (1:38), 45.9-46.9' (1:24), 46.9-47.2' (0:40)	
55											
60										47.2 **Bottom of Exploration at 47.20 feet below ground surface.**	
65											
70											
75											

Remarks:

1. Hole cave at 5.0'; advance HSA to 5.0'; switch to Drive & Wash; advance 4" casing to 21.3'; advance 3" casing to 38.0'.
2. PLD, PLA indicate diametral and axial point load tests performed. S_c indicates estimated unconfined compressive strength per ASTM D5731.
3. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterburg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.

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: Detective Benjamin Campbell Bridge #3666, West Branch Penobscot River
Location: T3 Indian Purchase Township, Maine

Boring No.: BB-IWBP-201

PIN: 23236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	480.0	**Auger ID/OD:**	2.5 SSA
Operator:	Eric Baron	**Datum:**	NAVD88	**Sampler:**	Standard
Logged By:	M. Johnescu	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	2-25-20 / 2-25-20	**Drilling Method:**	Mobile Drill B53	**Core Barrel:**	NX
Boring Location:	N779147.9, E1730300.7	**Casing ID/OD:**	4/4.5", 3/3.5"	**Water Level*:**	10.0'
Hammer Efficiency Factor: 0.904		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	2/2	0.7 - 0.9				SSA	479.3		0'-0.7': Asphalt	
										0.7	
	2D	24/18	1.0 - 3.0	71-45-30-33	75	113				Brown, dry, very dense, gravelly fine to coarse SAND, little silt, (Fill). Brown/light brown, dry, very dense, fine to coarse SAND, some gravel, little silt, (Fill).	
5	3D	24/7	5.0 - 7.0	8-10-7-12	17	26	51			Brown, dry, medium dense, fine to coarse SAND, some gravel, trace silt, (Fill).	
							76				
							65				
							138				
							90				
10	4D	24/5	10.0 - 12.0	29-18-6-4	24	36	21			Brown, wet, dense, fine to coarse G RAVEL, some fine to medium sand, little silt, (Fill).	
							30				
	5D	24/0	12.0 - 14.0	1-2-5-6	7	11	12			No recovery. One coarse, angular gravel piece in spoon tip.	
							35				
	6D	24/3	14.0 - 16.0	30-19-5-14	24	36	200			Brown, wet, dense, fine to coarse G RAVEL, some fine to coarse sand, little silt, angular gravel, (Fill).	
15								464.5		15.5	
	7D	24/6	16.0 - 18.0	11-57-47-66	104	157				Brown, wet, very dense, fine to coarse GRAVEL, some fine to coarse sand, little silt, angular gravel, (Moraine).	
							28				
	8D	24/9	18.0 - 20.0	29-35-25-21	60	90	43			Brown, wet, very dense, fine to coarse GRAVEL, some fine to coarse sand, little silt, angular gravel, (Moraine).	
							65				
20	9D	24/6	20.0 - 22.0	12-11-19-18	30	45	34			Brown, wet, dense, fine to coarse GRAVEL, some fine to coarse sand, little silt, (Moraine).	
							42				
							38				
							68				
25							72				

Remarks:

1. Automatic hammer NEBC #D24 Energy Transfer Ratio = 0.904.
2. Cobble/gravel stuck in spoon at 1D; refusal; drilled down to 1.0' and sampled again.
3. Switched to drive and wash method at 5.0'; heavy rig chatter and loss of water at 12.0' bgs. Bent splitspoon on sample 6D.
4. Loss of water return at 14.5' bgs, heavy rig chatter, probable cobble; broke through at 15.7' and regained water return.
5. From 14.5'-17.0' drilled ahead of casing; started using 3/3.5" case inside 4/4.5" case.

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: Detective Benjamin Campbell Bridge #3666, West Branch Penobscot River
Location: T3 Indian Purchase Township, Maine

Boring No.: BB-IWBP-201

PIN: 23236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	480.0	**Auger ID/OD:**	2.5 SSA
Operator:	Eric Baron	**Datum:**	NAVD88	**Sampler:**	Standard
Logged By:	M. Johnescu	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	2-25-20 / 2-25-20	**Drilling Method:**	Mobile Drill B53	**Core Barrel:**	NX
Boring Location:	N779147.9, E1730300.7	**Casing ID/OD:**	4/4.5", 3/3.5"	**Water Level*:**	10.0'
Hammer Efficiency Factor: 0.904		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	10D	24/16	25.0 - 27.0	41-53-61-50/4"	114	172	50			Brown, wet, very dense, gravelly fine to coarse SAND, little silt, angular gravel, (Moraine).	
							125	453.3		26.7	
	R1	53/52	27.0 - 31.4	RQD = 85%			85/2"			Splitspoon refusal at 26.7'. Advanced roller bit to 27.0' and set up to core.	q_p=4112
							NX			R1: Hard, slightly weathered, fine grained, extremely grey, SCHIST. Joints are close to moderately spaced, moderately dipping to high angle, planar, smooth to rough, discolored, partially open to moderately wide, highly fractured from 30.9'-31.4'. -CARRABASSETT FORMATION- Rock Quality = Good Recovery = 98% Rock Core Times (min:sec): 27.0-28.0' (1:47), 28.0-29.0' (1:24), 29.0-30.0' (1:19), 30.0-31.0' (1:34), 31.0'-31.4' (1:38)	
30											
	R2	34/33	31.4 - 34.2	RQD = 24%						R2: Hard, slightly weathered, fine grained, grey, SCHIST. Joints are extremely close to closely spaced, planar, rough to smooth, discolored, tight to moderately wide, with silt infilling, heavily fractured from 32.3'-33.7'. Fractured into angular gravel fragments from 32.3'-33.7'. Rock Quality = Very Poor Recovery = 97% Rock Core Times (min:sec): 31.4-32.4' (1:40), 32.4-33.4' (1:20), 33.4-34.2' (2:36)	
35	R3	32/30	34.2 - 36.9	RQD = 32%						R3: Hard, slightly weathered, fine grained, grey, SCHIST. Joints are extremely close to closely spaced, low to high angle, planar, rough to smooth, discolored, tight to open, with some silt infilling. Rock Quality = Poor Recovery = 94% Rock Core Times (min:sec): 34.2-35.2' (1:56), 35.2-36.2' (1:36), 36.2-36.9' (2:03)	
	R4	60/60	36.9 - 41.9	RQD = 78%				438.1		R4: Hard, slightly weathered, fine grained, grey, SCHIST. Joints are very close to moderately spaced, moderately dipping to vertical, planar, smooth to rough, discolored, tight to moderately wide. Rock Quality = Good Recovery = 100% Rock Core Times (min:sec): 36.9-37.9' (1:52), 37.9-38.9' (1:30), 38.9-39.9' (1:32), 39.9-40.9' (1:28), 40.9-41.9' (1:56)	
40										41.9	
45										**Bottom of Exploration at 41.90 feet below ground surface.**	
50											

Remarks:

1. Automatic hammer NEBC #D24 Energy Transfer Ratio = 0.904.
2. Cobble/gravel stuck in spoon at 1D; refusal; drilled down to 1.0' and sampled again.
3. Switched to drive and wash method at 5.0'; heavy rig chatter and loss of water at 12.0' bgs. Bent splitspoon on sample 6D.
4. Loss of water return at 14.5' bgs, heavy rig chatter, probable cobble; broke through at 15.7' and regained water return.
5. From 14.5'-17.0' drilled ahead of casing; started using 3/3.5" case inside 4/4.5" case.

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: Detective Benjamin Campbell Bridge #3666, West Branch Penobscot River
Location: T3 Indian Purchase Township, Maine

Boring No.: BB-IWBP-202

PIN: 23236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	451.0	**Auger ID/OD:**	2.5 SSA
Operator:	Brad Enos	**Datum:**	NAVD88	**Sampler:**	Standard
Logged By:	M. Johnescu	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	2-26-20 / 2-26-20	**Drilling Method:**	Mobile Drill B53	**Core Barrel:**	NX
Boring Location:	N779326.7, E1730312.9	**Casing ID/OD:**	4/4.5", 3/3.5"	**Water Level*:**	River
Hammer Efficiency Factor: 0.904		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	19/5	0.0 - 1.6	2-8-14-50/1"	22	33	24			Brown, wet, medium dense, GRAVEL, little sand, trace silt, (Moraine).	
							155				
	2D	8/1	2.0 - 2.7	31-100/3"	100	151	NX	448.3		Brown, wet, dense, GRAVEL, little sand, trace silt, one angular gravel piece in spoon tip, (Moraine). 2.7-	
	R1	20/17	3.1 - 4.8	RQD = 0%						Splitspoon refusal at 2.7'; advanced roller bit to 3.1' and set up to core. R1: Hard, slightly weathered, fine grained, grey, SCHIST. Joints are extremely close to close, moderately dipping to high angle, planar, rough to smooth, discolored, partially open to moderately wide, with silt infilling. -CARRABASSETT FORMATION- Rock Quality = Very Poor Recovery = 85% Rock Core Times (min:sec): 3.1-4.1' (2:40), 4.1-4.8' (2:27)	
	R2	48/43	4.8 - 8.8	RQD = 39%						R2: Hard, slightly weathered, fine grained, grey, SCHIST. Joints are extremely close to closely spaced, low angle to vertical, planar, rough to smooth, discolored, tight to moderately wide, with silt infilling. Rock Quality = Poor Recovery = 90% Rock Core Times (min:sec): 4.8-5.1' (0:41), 5.1-6.1' (2:06), 6.1-7.1' (2:23), 7.1-8.1' (1:49), 8.1-8.8' (2:33)	
5											
	R3	14/12	8.8 - 10.0	RQD = 0%						R3: Hard, slightly weathered, fine grained, grey, SCHIST. Joints are extremely close, moderately dipping, planar, smooth to rough, discolored, p artially open to moderately wide, with silt infilling. Fractured into angular gravel fragments. Rock Quality = Very Poor Recovery = 86% Rock Core Times (min:sec): 8.8-9.8' (2:03), 9.8-10.0' (1:27)	
10	R4	24/22	10.0 - 12.0	RQD = 18%						R4: Hard, slightly weathered, fine grained, grey, SCHIST. Joints are e xtremely close to closely spaced, m o derately dipping to vertical, planar, rough to smooth, discolored, tight to moderately wide, with silt infilling. Fractured into angular gravel fragments from 10.0'-10.8'. Rock Quality = Very Poor Recovery = 92% Rock Core Times (min:sec): 10.0-10.8' (1:14), 10.8-11.8' (1:24), 11.8-12.0' (0:46)	
	R5	42/42	12.0 - 15.5	RQD = 21%						R5: Hard, moderately weathered, fine to medium grained, grey, SCHIST. Joints are extremely close to moderately spaced, low to high angle, planar, rough to smooth, discolored to decomposed (14.6'-15.0'), tight to moderately wide, with silt and sand infilling. Rock Quality = Very Poor Recovery = 100% Rock Core Times (min:sec): 12.0-12.8' (1:44), 12.8-13.8' (1:13), 13.8-14.8' (1:48), 14.8-15.5' (1:29)	
15	R6	28/21	15.5 - 17.8	RQD = 36%							
	R7	36/32	17.8 - 20.8	RQD = 0%							
20	R8	17/16	20.8 - 22.2	RQD = 25%							
	R9	24/22	22.2 - 24.2	RQD = 23%							
25								426.8			

Remarks:

1. Mudline 28.0' below bridge deck.
2. Automatic hammer NEBC #D24 Energy Transfer Ratio = 0.904.

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: Detective Benjamin Campbell Bridge #3666, West Branch Penobscot River
Location: T3 Indian Purchase Township, Maine

Boring No.: BB-IWBP-202

PIN: 23236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	451.0	**Auger ID/OD:**	2.5 SSA
Operator:	Brad Enos	**Datum:**	NAVD88	**Sampler:**	Standard
Logged By:	M. Johnescu	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	2-26-20 / 2-26-20	**Drilling Method:**	Mobile Drill B53	**Core Barrel:**	NX
Boring Location:	N779326.7, E1730312.9	**Casing ID/OD:**	4/4.5", 3/3.5"	**Water Level*:**	River
Hammer Efficiency Factor: 0.904		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample Information: 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										R6: Hard, slightly weathered, fine grained, grey, SCHIST. Joints are extremely close to close, low to high angle, planar, rough to smooth, discolored, tight to moderately wide, with silt/sand infilling. Rock Quality = Poor Recovery = 75% Rock Core Times (min:sec): 15.5-16.8' (2:00), 16.8-17.8' (2:10)	
										R7: Hard, slightly weathered, fine grained, grey, SCHIST. Joints are e xtremely close to closely spaced, m oderately dipping to vertical, stepped to planar, rough to smooth, discolored, tight to moderately wide, with silt infilling. Rock Quality = Very Poor Recovery = 89% Rock Core Times (min:sec): 17.8-18.8' (1:23), 18.8-19.8' (1:54), 19.8-20.8' (1:32)	
30										R8: Hard, slightly weathered, fine grained, grey, SCHIST. Joints are extremely close to closely spaced, low angle to high angle, planar, rough to smooth, discolored, tight to moderately wide. Rock Quality = Poor Recovery = 94% Rock Core Times (min:sec): 20.8-21.8' (1:23), 21.8-22.2 (1:39)	
										R9: Hard, slightly weathered, fine grained, grey, SCHIST. Joints are extremely close to closely spaced, low angle to high angle, planar, rough to smooth, discolored, tight to moderately wide, with some silt infilling. Rock Quality = Very Poor Recovery = 92% Rock Core Times (min:sec): 22.2-22.8' (1:04), 22.8-23.8' (2:46), 23.8-24.2' (1:13)	
35										24.2 **Bottom of Exploration at 24.20 feet below ground surface.**	
40											
45											
50											

Remarks:

1. Mudline 28.0' below bridge deck.
2. Automatic hammer NEBC #D24 Energy Transfer Ratio = 0.904.

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: Detective Benjamin Campbell Bridge #3666, West Branch Penobscot River
Location: T3 Indian Purchase Township, Maine

Boring No.: BB-IWBP-203

PIN: 23236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	449.8	**Auger ID/OD:**	2.5 SSA
Operator:	Mike Porter	**Datum:**	NAVD88	**Sampler:**	Standard
Logged By:	M. Johnescu	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3-2-20 / 3-2-20	**Drilling Method:**	Mobile Drill B53	**Core Barrel:**	NX
Boring Location:	N779356.6, E1730330.6	**Casing ID/OD:**	4/4.5"	**Water Level*:**	River
Hammer Efficiency Factor: 0.904		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/14	0.0 - 2.0	8-72-68-57	140	211				Brown, wet, very dense, GRAVEL, some fine to coarse sand, some silt, (Moraine).	
	R1	14/11	4.7 - 5.9	RQD = 0%				445.3		4.5	
5	R2	22/16	5.9 - 7.7	RQD = 34%						Increased roller cone resistance at 4.5'; apparent top of rock. Advance roller cone to 4.7' and start to core. R1: Hard, slightly weathered, fine grained, grey, SCHIST. Joints are e xtremely close to closely spaced, m o derately dipping to vertical, planar, rough to smooth, discolored, tight to moderately wide. -CARRABASSETT FORMATION- Rock Quality = Very Poor Recovery = 79% Rock Core Times (min:sec): 4.7-5.7' (2:34), 5.7-5.9' (2:14)	
	R3	24/16	7.7 - 9.7	RQD = 0%						R2: Hard, slightly weathered, fine grained, grey, SCHIST. Joints are extremely close to closely spaced, low to high angle, planar, rough to smooth, discolored, tight to moderately wide. Rock Quality = Poor Recovery = 73% Rock Core Times (min:sec): 5.9-6.7' (2:07), 6.7-7.7' (2:48)	
10								440.1		R3: Hard, slightly weathered, fine grained, grey, SCHIST. Joints are extremely close, low to high angle, planar, rough to smooth, discolored, partially open to moderately wide. Rock Quality = Very Poor Recovery = 67% Rock Core Times (min:sec): 7.7-8.7' (1:45)	
										9.7	
15										**Bottom of Exploration at 9.70 feet below ground surface.**	
20											
25											

Remarks:

1. Mudline 28.3' below bridge deck.
2. Automatic hammer NEBC #D24 Energy Transfer Ratio = 0.904.

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: Detective Benjamin Campbell Bridge #3666, West Branch Penobscot River
Location: T3 Indian Purchase Township, Maine

Boring No.: BB-IWBP-204-1
PIN: 23236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	474.3	**Auger ID/OD:**	2.5 SSA
Operator:	Brad Enos	**Datum:**	NAVD88	**Sampler:**	Standard
Logged By:	M. Johnescu	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	2-26-20 / 3-5-20	**Drilling Method:**	Mobile Drill B53	**Core Barrel:**	NX
Boring Location:	N779531.5, E1730353.9	**Casing ID/OD:**	4/4.5"	**Water Level*:**	15.0'
Hammer Efficiency Factor: 0.904		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	17/17	0.5 - 1.9	60-72-79/5"			SSA	473.8		0'-0.5': Asphalt	
										0.5	
										Brown, moist, very dense, sandy GRAVEL, little silt, angular gravel pieces, (Fill).	
	2D	9/4	4.0 - 4.8	7-100/3"	R		RC			Brown, moist, very dense, fine to coarse SAND, some gravel, little silt, (Fill).	
5							38				
							67				
							82				
							62				
	3D	24/6	9.0 - 11.0	10-26-27-10	53	80	47			Grey, wet, very dense, fine to coarse GRAVEL, little fine to coarse sand, (Fill).	
10							123				
	4D	24/10	11.0 - 13.0	14-12-10-21	22	33	105			Grey, wet, dense, gravelly fine to coarse SAND, trace silt, (Fill).	
							132	462.3		12.0	
	5D	16/4	13.0 - 14.3	62-1-50/4"	R		RC			Brown, wet, hard, Silty CLAY, little fine to medium sand, trace silt, medium to low plasticity (Moraine).	
15											
	6D	18/6	15.0 - 16.5	71-18-16	34	51				Brown and grey, moist, hard, gravelly CLAY, trace silt, low plasticity, (Moraine).	
								457.8		16.5	
								457.3		See BB-IWBP-204-2 for remainder of boring log from 17.0'-50.5' bgs.	
										17.0	
										Bottom of Exploration at 17.00 feet below ground surface.	
20											
25											

Remarks:

1. Automatic hammer NEBC #D24 Energy Transfer Ratio = 0.904.
2. Possible cobble at 4.8' bgs, heavy grinding and rig chatter.
3. Heavy rig chatter and on/off loss of drilling returns starting at 12.5'.
4. Drill rig changed to NEBC drill rig B23 on 3-2-2020 at 17.0'.

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: Detective Benjamin Campbell Bridge #3666, West Branch Penobscot River
Location: T3 Indian Purchase Township, Maine

Boring No.: BB-IWBP-204-2
PIN: 23236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	474.3	**Auger ID/OD:**	2.5 SSA
Operator:	Brad Enos	**Datum:**	NAVD88	**Sampler:**	Standard
Logged By:	M. Johnescu	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	2-26-20 / 3-5-20	**Drilling Method:**	Mobile Drill B53	**Core Barrel:**	NX
Boring Location:	N779531.5, E1730353.9	**Casing ID/OD:**	4/4.5"	**Water Level*:**	15.0'
Hammer Efficiency Factor: 0.842		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0										See BB-IWBP-204-1 for material descriptions from 0.0'-17.0' bgs.	
5											
10											
15											
								457.3		17.0	
	7D	24/12	17.0 - 19.0	51-22-20-22	42	59	RC			Brown, moist, very dense, fine to coarse SAND, some gravel, some silt, (Moraine).	
20	8D	24/7	19.0 - 21.0	28-26-29-38	55	77				Brown, moist, very dense, sandy GRAVEL, trace silt, (Moraine).	
	9D	24/9	21.0 - 23.0	40-32-25-27	57	80				Brown, wet, very dense, fine to coarse SAND, little gravel, trace silt, (Moraine).	
25	10D	24/11	23.0 - 25.0	28-81-30-39	111	156				Brown and grey, wet, very dense, GRAVEL, little sand, trace silt, gravel is subangular, (Moraine).	

Remarks:

1. Automatic hammer NEBC #D23 Energy Transfer Ratio = 0.842.
2. Possible cobble at 4.8' bgs, heavy grinding and rig chatter.
3. Heavy rig chatter and on/off loss of drilling returns starting at 12.5'.
4. Drill rig changed to NEBC drill rig B23 on 3-2-2020 at 17.0'.
5. After coring sample R3 to 34.0' bgs, the roller bit was advanced to 34.7', and then coring was continued.

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: Detective Benjamin Campbell Bridge #3666, West Branch Penobscot River
Location: T3 Indian Purchase Township, Maine

Boring No.: BB-IWBP-204-2

PIN: 23236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	474.3	**Auger ID/OD:**	2.5 SSA
Operator:	Brad Enos	**Datum:**	NAVD88	**Sampler:**	Standard
Logged By:	M. Johnescu	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	2-26-20 / 3-5-20	**Drilling Method:**	Mobile Drill B53	**Core Barrel:**	NX
Boring Location:	N779531.5, E1730353.9	**Casing ID/OD:**	4/4.5"	**Water Level*:**	15.0'
Hammer Efficiency Factor: 0.842		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	11D	24/10	25.0 - 27.0	37-52-25-28	77	108				Brown and grey, wet, very dense, GRAVEL, little sand, trace silt, gravel is subangular, (Moraine).	
	12D	7/7	27.0 - 27.6	66-50/2"	R		NX	446.7		27.6'	s_c=3380 (PLD)
	R1	35/10	27.6 - 30.5	RQD = 0%						Splitspoon refusal at 27.6', probable bedrock. Advance roller cone to 27.6' and start to core. Probable top of rock at 27.6'. R1: Hard, slightly weathered, fine grained, grey, SCHIST (Fractured/Weathered Rock). Joints are extremely close, moderately dipping, stepped to planar, rough to smooth, discolored, tight to moderately wide. Highly fractured. -CARRABASSETT FORMATION- Rock Quality = Very Poor Recovery = 29% Rock Core Times (min:sec): 27.6-28.6' (3:06), 28.6-29.6' (2:28), 29.6-30.5' (3:03)	
30	R2	14/11	30.5 - 31.7	RQD = 0%						R2: Hard, slightly weathered, fine grained, grey, SCHIST (Fractured/Weathered Rock). Joints are extremely close, moderately dipping to vertical, stepped to planar, rough to smooth, discolored, partially open to moderately wide. Fractured into angular gravel fragments. Rock Quality = Very Poor Recovery = 79% Rock Core Times (min:sec): 30.5-31.6' (1:35), 31.6-31.7' (0:35)	
	R3	28/15	31.7 - 34.0	RQD = 0%						R3: Hard, slightly weathered, fine grained, grey, SCHIST (Fractured/Weathered Rock). Joints are extremely close, stepped to planar, rough, discolored, partially open to moderately wide. Rock Quality = Very Poor Recovery = 54% Rock Core Times (min:sec): 31.7-32.6' (1:23), 32.6-33.6' (1:41)	
	R4	23/14	34.7 - 36.6	RQD = 0%			RC			R4: Hard, slightly weathered, fine grained, grey, SCHIST (Fractured/Weathered Rock). Joints are extremely close, low angle, stepped to planar, rough to smooth, discolored, tight to moderately wide. Rock Quality = Very Poor Recovery = 63% Rock Core Times (min:sec): Not recorded	
35							NX				
	R5	17/13	36.6 - 38.0	RQD = 0%				437.8		36.5' R5: Hard, slightly weathered, fine grained, grey, SCHIST. Joints are extremely close, low angle to vertical, s tepped to planar, rough to smooth, discolored, tight to moderately wide. -CARRABASSETT FORMATION- Rock Quality = Very Poor Recovery = 76% Rock Core Times (min:sec): 36.6-37.6' (5:03), 37.6-38.0' (3:16)	
	R6	26/22	38.0 - 40.2	RQD = 0%							
40	R7	17/14	40.2 - 41.6	RQD = 0%							
	R8	31/28	41.6 - 44.2	RQD = 0%							
	R9	24/24	44.2 - 46.2	RQD = 0%							
45											
	R10	14/9	46.2 - 47.4	RQD = 0%							
	R11	37/37	47.4 - 50.5	RQD = 34%							
50										R6: Hard, slightly weathered, fine grained, grey, SCHIST. Joints are extremely close to very close, low	

Remarks:

1. Automatic hammer NEBC #D23 Energy Transfer Ratio = 0.842.
2. Possible cobble at 4.8' bgs, heavy grinding and rig chatter.
3. Heavy rig chatter and on/off loss of drilling returns starting at 12.5'.
4. Drill rig changed to NEBC drill rig B23 on 3-2-2020 at 17.0'.
5. After coring sample R3 to 34.0' bgs, the roller bit was advanced to 34.7', and then coring was continued.

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: Detective Benjamin Campbell Bridge #3666, West Branch Penobscot River
Location: T3 Indian Purchase Township, Maine

Boring No.: BB-IWBP-204-2

PIN: 23236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	474.3	**Auger ID/OD:**	2.5 SSA
Operator:	Brad Enos	**Datum:**	NAVD88	**Sampler:**	Standard
Logged By:	M. Johnescu	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	2-26-20 / 3-5-20	**Drilling Method:**	Mobile Drill B53	**Core Barrel:**	NX
Boring Location:	N779531.5, E1730353.9	**Casing ID/OD:**	4/4.5"	**Water Level*:**	15.0'
Hammer Efficiency Factor:	0.842	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50								423.8		angle to high angle, planar, rough to smooth, discolored, tight to moderately wide. Rock Quality = Very Poor Recovery = 85% Rock Core Times (min:sec): 38.0-38.6' (1:24), 38.6-39.6' (4:08), 39.6-40.2' (3:09) R7: Hard, slightly weathered, fine grained, grey, SCHIST. Joints are extremely close, low angle to moderately dipping, planar, rough to smooth, discolored, tight to moderately wide. Rock Quality = Very Poor Recovery = 82% Rock Core Times (min:sec): 40.2-41.2' (3:19), 41.2-41.6' (1:52) R8: Hard, slightly weathered, fine grained, grey, SCHIST. Joints are extremely close to very close, low angle to high angle, planar, rough to smooth, discolored, tight to moderately wide. Rock Quality = Very Poor Recovery = 90% Rock Core Times (min:sec): 41.6-42.2' (1:25), 42.2-43.2' (2:14), 43.2-44.2' (2:45) R9: Hard, slightly weathered, fine grained, grey, SCHIST. Joints are e xtremely close, low angle to vertical, s tepped to planar, rough to smooth, discolored, tight to moderately wide. Rock Quality = Very Poor Recovery = 100% Rock Core Times (min:sec): 44.2-45.2' (1:58), 45.2-46.2' (2:06) R10: Hard, slightly weathered, fine grained, grey, SCHIST. Joints are extremely close, low angle to high angle, planar, rough to smooth, discolored, partially open to moderately wide. Fractured into angular gravel fragments. Rock Quality = Very Poor Recovery = 64% Rock Core Times (min:sec): 46.2-47.2' (3:15), 47.2-47.4' (1:16) R11: Hard, slightly weathered, fine grained, grey, SCHIST. Joints are e xtremely close to moderately spaced, low angle to high angle, stepped to planar, rough to smooth, discolored, tight to moderately wide. Rock Quality = Poor Recovery = 100% Rock Core Times (min:sec): 47.4-48.2' (1:54), 48.2-49.2' (2:09), 49.2-50.2' (2:47), 50.2-50.5' (0:37)	
55											
60											
65											
70											
75										50.5 **Bottom of Exploration at 50.50 feet below ground surface.**	

Remarks:

1. Automatic hammer NEBC #D23 Energy Transfer Ratio = 0.842.
2. Possible cobble at 4.8' bgs, heavy grinding and rig chatter.
3. Heavy rig chatter and on/off loss of drilling returns starting at 12.5'.
4. Drill rig changed to NEBC drill rig B23 on 3-2-2020 at 17.0'.
5. After coring sample R3 to 34.0' bgs, the roller bit was advanced to 34.7', and then coring was continued.

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

APPENDIX C – LABORATORY TEST RESULTS

THIELSCH ENGINEERING	195 Frances Avenue Cranston RI, 02910 Phone: (401)-467-6454 Fax: (401)-467-2398 http://www.thielsch.com *Let's Build a Solid Foundation*	Client Information: GZA GeoEnvironmental, Inc Portland, ME PM: Theodore Baire Assigned By: EDF Collected By: EDF	Project Information: **West Branch Bridge #3666** **T3 Indian Purchase Twp, ME** GZA Project Number: 09.0025976.00 Summary Page: 1 of 2 Report Date: 06.26.18

LABORATORY TESTING DATA SHEET

Boring ID	Sample No.	Depth (ft)	Laboratory No.	Identification Tests								Proctor / Direct Shear Tests								Laboratory Log and Soil Description
				Water Content %	LL %	PL %	Gravel %	Sand %	Fines %	Org. %	G_s	Dry unit wt. pcf	γ_d MAX (pcf) / W_{opt} (%)	γ_d MAX (pcf) / W_{opt} (%) (Corr.)	Strength (psi) @ 250 psf	Strength (psi) @ 500 psf	Strength (psi) @ 750 psf	Strength (psi) @ 1000 psf	EST. Shear Angle	
BB-IWBP-101	1D	1-3	S-1	4.3			38.0	51.0	11.0											Brown Gravelly fine to coarse SAND, little Silt
BB-IWBP-101	2D	5.5-7.5	S-2	3.0			43.9	47.7	8.4											Brown Gravelly fine to coarse SAND, trace Silt
BB-IWBP-101	5D	20-22	S-3	5.5			54.4	31.2	14.4											Brown f-c GRAVEL, some f-m Sand, little Silt
BB-IWBP-102	2D	5-7	S-4	3.6			74.6	21.7	3.7											Grey f-c GRAVEL, some f-c Sand, trace Silt
BB-IWBP-102	3D	10-12	S-5	4.6			72.3	17.0	10.7											Light Brown f-c GRAVEL, little f-c Sand, little Silt
BB-IWBP-104	1D	0-2	S-6	4.7			45.4	51.6	3.0											Brown Gravelly fine to coarse SAND, trace Silt
BB-IWBP-104	2D	5.7-7.7	S-7	7.7			63.2	36.6	0.2											Grey Sandy fine to coarse GRAVEL, trace Silt
BB-IWBP-104	4D	15-15.8	S-8	11.6			32.0	43.7	24.3											Light Brown Gravelly fine to coarse SAND, some Silt
BB-IWBP-105	1D	2-4	S-9	1.9			47.1	45.9	7.0											Brown Sandy fine to coarse GRAVEL, trace Silt
BB-IWBP-105	2D	5-7	S-10	2.8			46.5	45.5	8.0											Brown Sandy fine to coarse GRAVEL, trace Silt
BB-IWBP-105	4D	16.2-17.5	S-11	6.9			54.0	32.7	13.3											Light Brown f-c GRAVEL, some f-c Sand, little Silt

Reviewed By _____ Date Reviewed 06.27.2018

THIELSCH ENGINEERING

State of Maine - Department of Transportation
Laboratory Testing Summary Sheet

West Branch Bridge #3666

MDOT Project Number: WIN 023236.00

GZA Project Number: 09.0025976.00

Town(s): T3 Indian Purchase Twp, ME

Boring & Sample Identification Number	Station (Feet)	Sample No.	Depth (Feet)	Lab Number	Organic %	W.C.	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-IWBP-101		1D	1-3	S-1		4.3			SP-SM	A-1-a	II
BB-IWBP-101		2D	5.5-7.5	S-2		3.0			SP-SM	A-1-a	II
BB-IWBP-101		5D	20-22	S-3		5.5			GM	A-1-a	I
BB-IWBP-102		2D	5-7	S-4		3.6			GW	A-1-a	0
BB-IWBP-102		3D	10-12	S-5		4.6			GP-GM	A-1-a	I
BB-IWBP-104		1D	0-2	S-6		4.7			SP	A-1-a	0
BB-IWBP-104		2D	5.7-7.7	S-7		7.7			GP	A-1-a	0
BB-IWBP-104		4D	15-15.8	S-8		11.6			SM	A-1-b	III
BB-IWBP-105		1D	2-4	S-9		1.9			GW-GM	A-1-a	I
BB-IWBP-105		2D	5-7	S-10		2.8			GW-GM	A-1-a	I
BB-IWBP-105		4D	16.2-17.5	S-11		6.9			GM	A-1-a	I

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

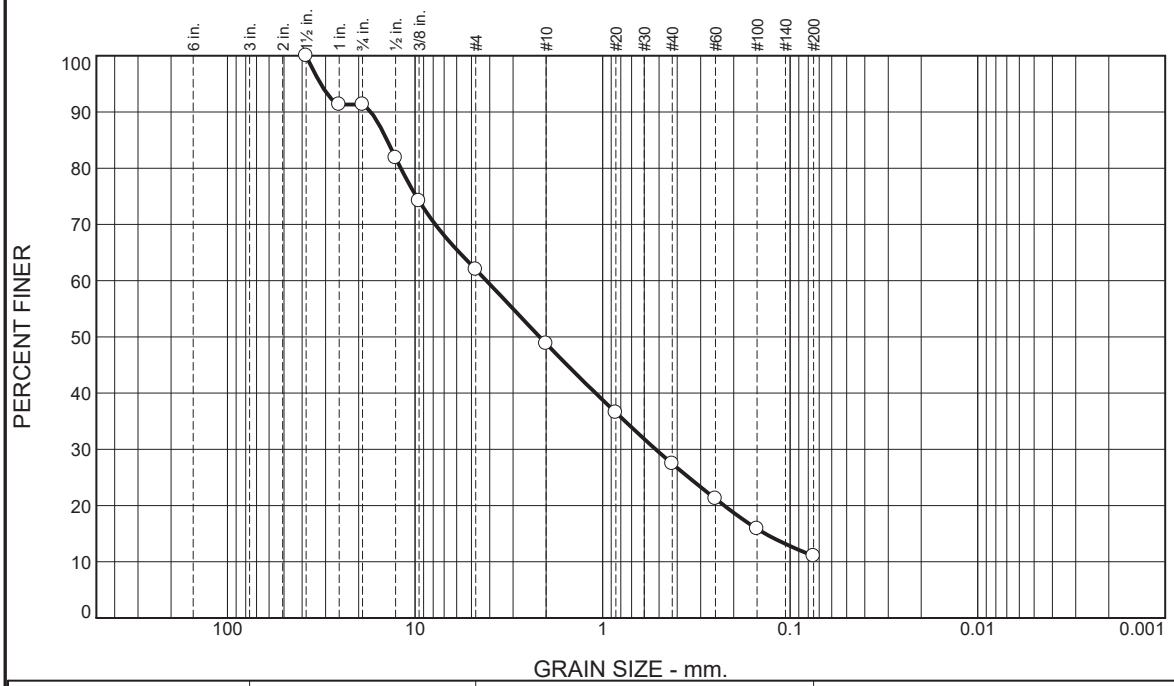
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	8.7	29.3	13.2	21.4	16.4	11.0	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1.5"	100.0		
1"	91.3		
0.75"	91.3		
0.5"	81.8		
0.375"	74.2		
#4	62.0		
#10	48.8		
#20	36.5		
#40	27.4		
#60	21.2		
#100	15.9		
#200	11.0		

* (no specification provided)

Material Description

Brown Gravelly fine to coarse SAND, little Silt

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SP-SM **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 17.1899 D_{85}= 14.1081 D_{60}= 4.1612
D_{50}= 2.1646 D_{30}= 0.5208 D_{15}= 0.1356
D_{10}= C_u= C_c=

Remarks

Date Received: 06.13.18 **Date Tested:** 06.20.18

Tested By: RR

Checked By: Steven Accetta

Title: Laboratory Manager

Source of Sample: Borings **Depth:** 1-3' **Date Sampled:**
Sample Number: BB-IWBP-101 / 1D

Thielsch Engineering Inc.

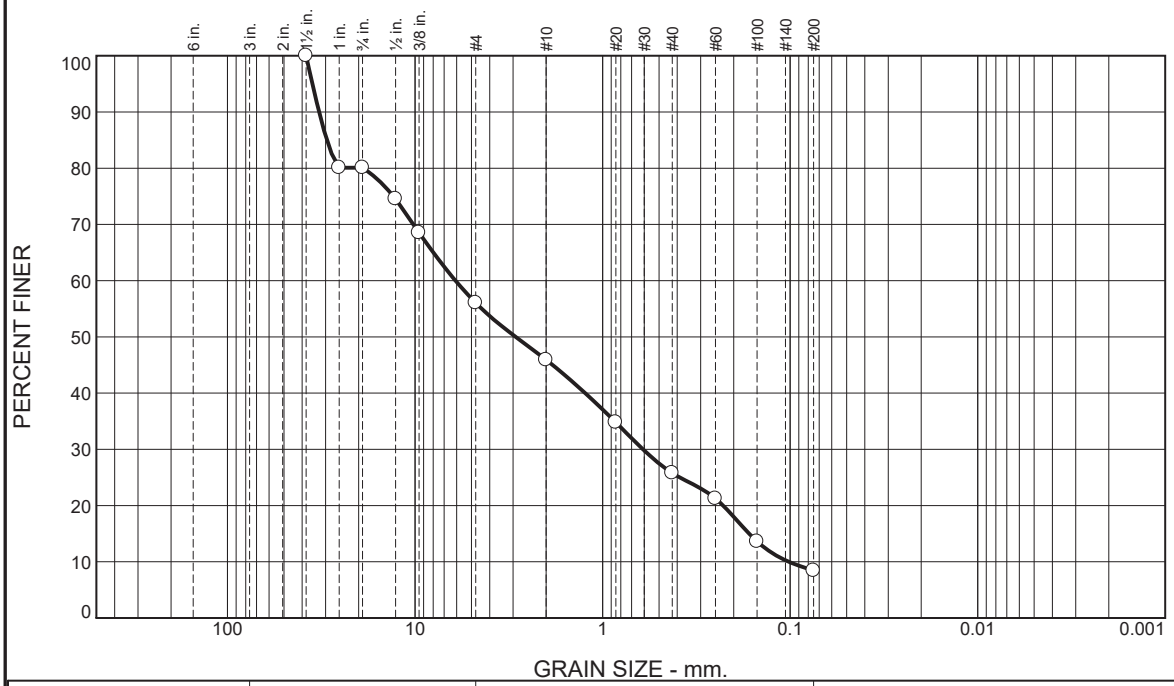
Cranston, RI

Client: GZA GeoEnvironmental

Project: West Branch Bridge #3666
T3 Indian Purchase Twp, ME

Project No: 09.0025976.00 **Figure** S-1

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	19.9	24.0	10.2	20.1	17.4	8.4	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1.5"	100.0		
1"	80.1		
0.75"	80.1		
0.5"	74.6		
0.375"	68.5		
#4	56.1		
#10	45.9		
#20	34.8		
#40	25.8		
#60	21.2		
#100	13.6		
#200	8.4		

* (no specification provided)

Material Description

Brown Gravelly fine to coarse SAND, trace Silt

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SP-SM **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 32.4664 D_{85}= 29.5029 D_{60}= 6.0837
D_{50}= 2.9100 D_{30}= 0.6088 D_{15}= 0.1654
D_{10}= 0.1016 C_u= 59.91 C_c= 0.60

Remarks

Date Received: 06.13.18 **Date Tested:** 06.20.18

Tested By: RR

Checked By: Steven Accetta

Title: Laboratory Manager

Source of Sample: Borings **Depth:** 5.5-7.5' **Date Sampled:**
Sample Number: BB-IWBP-101 / 2D

Thielsch Engineering Inc.

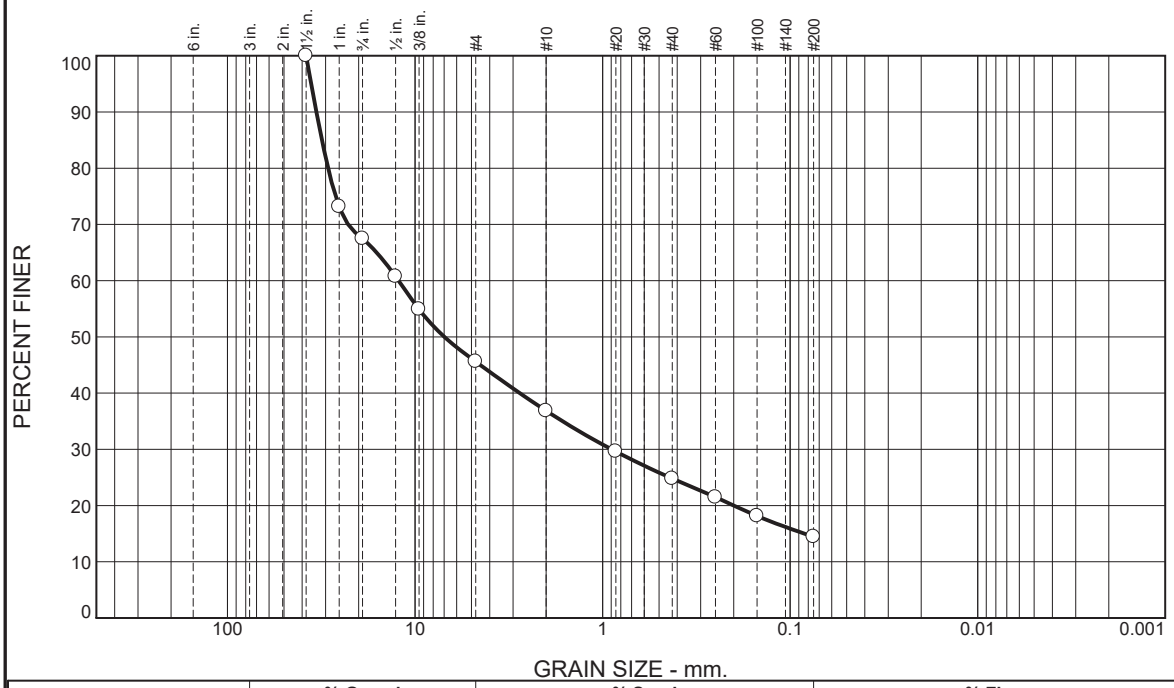
Cranston, RI

Client: GZA GeoEnvironmental

Project: West Branch Bridge #3666
T3 Indian Purchase Twp, ME

Project No: 09.0025976.00 **Figure** S-2

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	32.5	21.9	8.7	12.1	10.4	14.4	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1.5"	100.0		
1"	73.1		
0.75"	67.5		
0.5"	60.7		
0.375"	54.9		
#4	45.6		
#10	36.9		
#20	29.6		
#40	24.8		
#60	21.5		
#100	18.2		
#200	14.4		

* (no specification provided)

Material Description

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

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

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

Coefficients

D_{90}= 33.5600 D_{85}= 31.3526 D_{60}= 12.2419
D_{50}= 6.9695 D_{30}= 0.8952 D_{15}= 0.0838
D_{10}= C_u= C_c=

Remarks

Date Received: 06.13.18 **Date Tested:** 06.20.18

Tested By: RR

Checked By: Steven Accetta

Title: Laboratory Manager

Source of Sample: Borings **Depth:** 20-22' **Date Sampled:**
Sample Number: BB-IWBP-101 / 5D

Thielsch Engineering Inc.

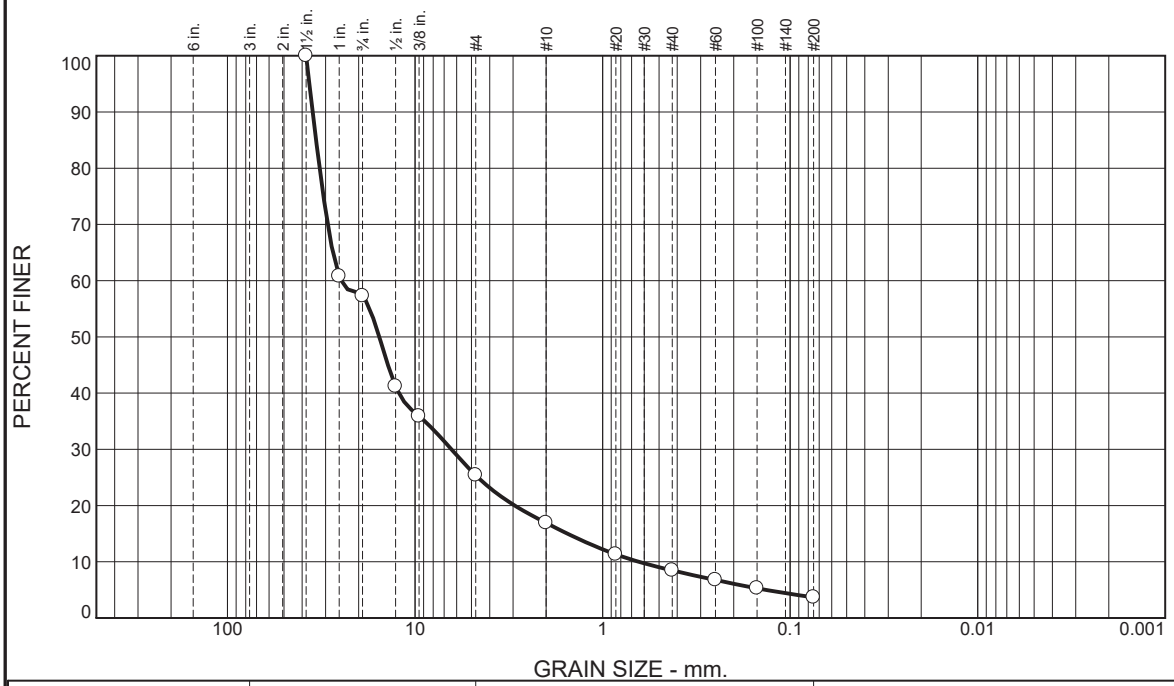
Cranston, RI

Client: GZA GeoEnvironmental

Project: West Branch Bridge #3666
T3 Indian Purchase Twp, ME

Project No: 09.0025976.00 **Figure** S-3

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	42.7	31.9	8.5	8.4	4.8	3.7	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1.5"	100.0		
1"	60.8		
0.75"	57.3		
0.5"	41.2		
0.375"	35.9		
#4	25.4		
#10	16.9		
#20	11.3		
#40	8.5		
#60	6.8		
#100	5.3		
#200	3.7		

* (no specification provided)

Material Description

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

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= GW **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 35.1443 D_{85}= 33.7048 D_{60}= 24.8320
D_{50}= 15.5152 D_{30}= 6.3941 D_{15}= 1.5378
D_{10}= 0.6405 C_u= 38.77 C_c= 2.57

Remarks

Date Received: 06.13.18 **Date Tested:** 06.21.18

Tested By: JS

Checked By: Steven Accetta

Title: Laboratory Manager

Source of Sample: Borings **Depth:** 5-7'
Sample Number: BB-IWBP-102 / 2D

Date Sampled:

Thielsch Engineering Inc.

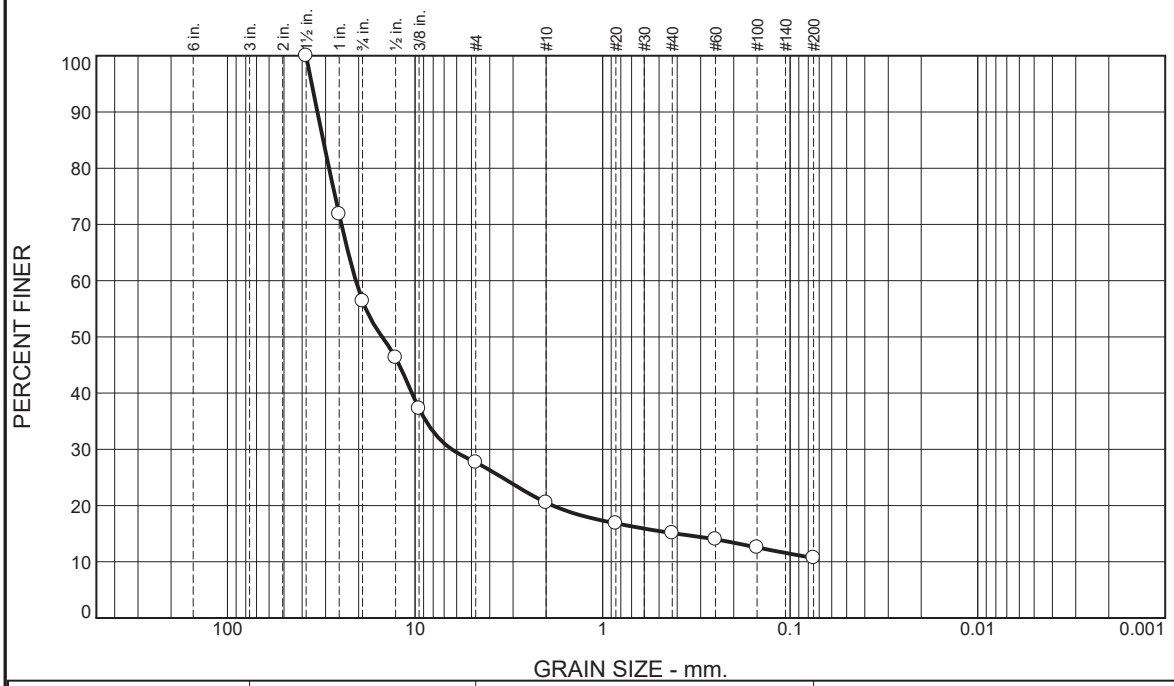
Cranston, RI

Client: GZA GeoEnvironmental

Project: West Branch Bridge #3666
T3 Indian Purchase Twp, ME

Project No: 09.0025976.00 **Figure** S-4

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	43.6	28.7	7.2	5.4	4.4	10.7	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1.5"	100.0		
1"	71.8		
0.75"	56.4		
0.5"	46.3		
0.375"	37.2		
#4	27.7		
#10	20.5		
#20	16.8		
#40	15.1		
#60	14.0		
#100	12.5		
#200	10.7		

* (no specification provided)

Material Description

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

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= GP-GM **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 33.1425 D_{85}= 30.8755 D_{60}= 20.6877
D_{50}= 14.9851 D_{30}= 6.3686 D_{15}= 0.4058
D_{10}= C_u= C_c=

Remarks

Date Received: 06.1308 **Date Tested:** 06.21.18

Tested By: JS

Checked By: Steven Accetta

Title: Laboratory Manager

Source of Sample: Borings **Depth:** 20-22 **Date Sampled:**
Sample Number: BB-IWBP-102 / 3D

Thielsch Engineering Inc.

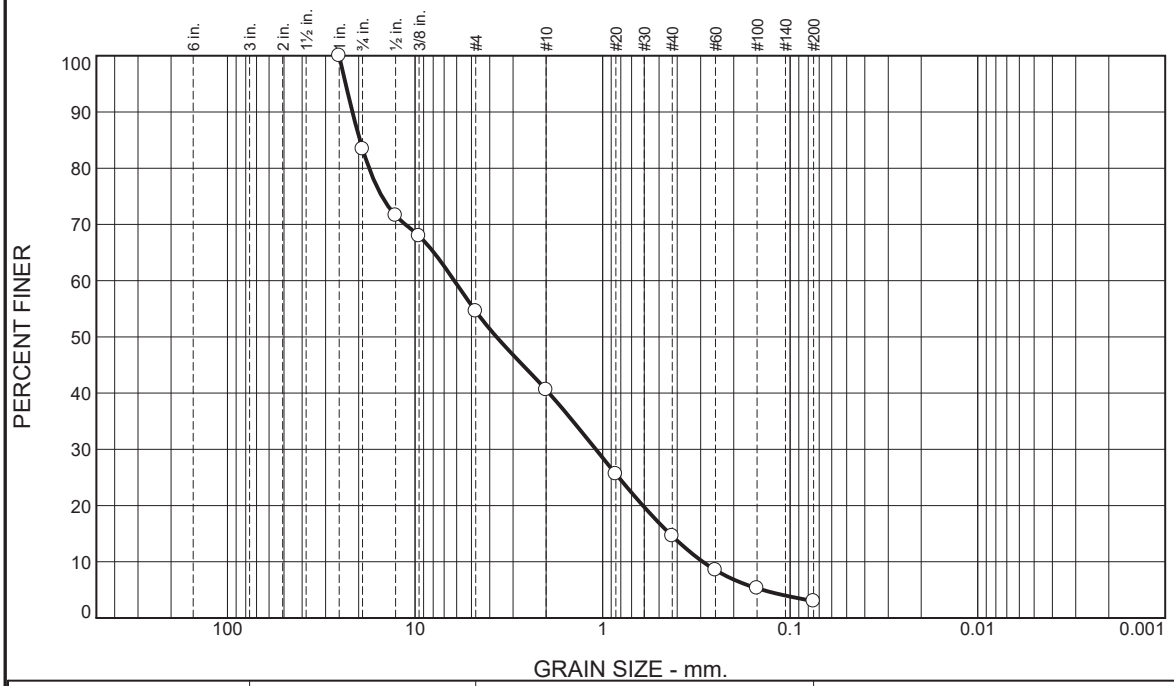
Cranston, RI

Client: GZA GeoEnvironmental

Project: West Branch Bridge #3666
T3 Indian Purchase Twp, ME

Project No: 09.0025976.00 **Figure** S-5

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	16.6	28.8	14.0	26.0	11.6	3.0	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	83.4		
0.5"	71.6		
0.375"	68.0		
#4	54.6		
#10	40.6		
#20	25.6		
#40	14.6		
#60	8.5		
#100	5.3		
#200	3.0		

* (no specification provided)

Material Description

Brown Gravelly fine to coarse SAND, trace Silt

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SP **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 21.5695 D_{85}= 19.6748 D_{60}= 6.1753
D_{50}= 3.6783 D_{30}= 1.0871 D_{15}= 0.4384
D_{10}= 0.2920 C_u= 21.15 C_c= 0.66

Remarks

Date Received: 06.13018 **Date Tested:** 06.21.18

Tested By: JS

Checked By: Steven Accetta

Title: Laboratory Manager

Source of Sample: Borings **Depth:** 0-2' **Date Sampled:**
Sample Number: BB-IWBP-104 / 1D

Thielsch Engineering Inc.

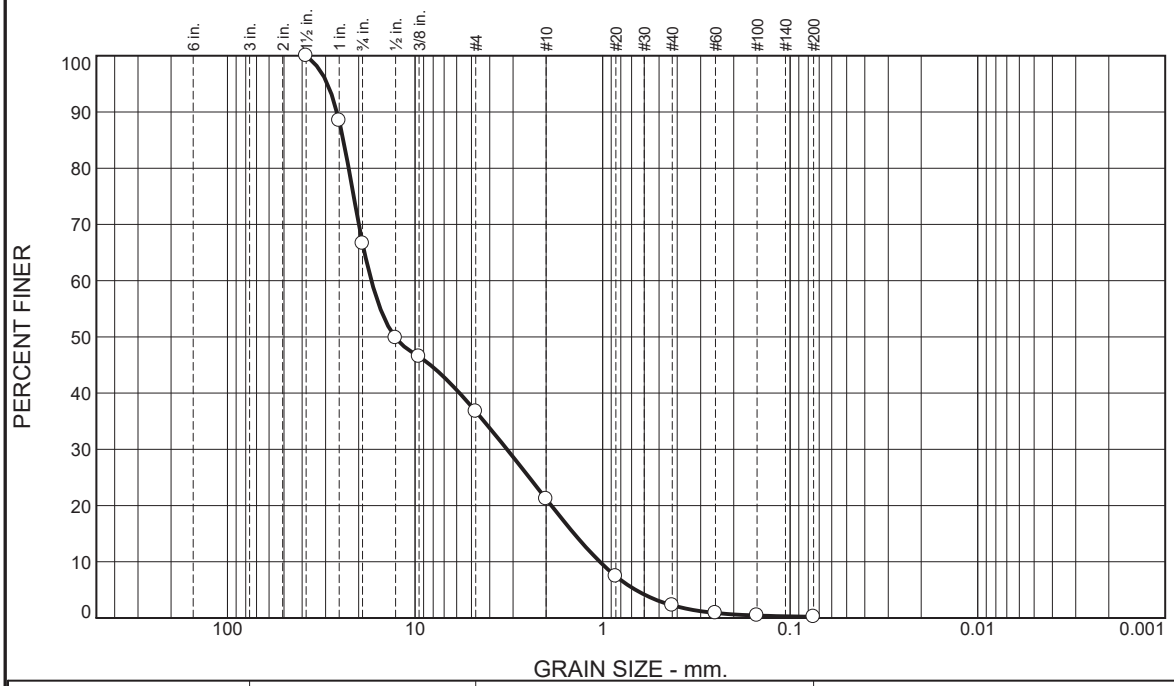
Cranston, RI

Client: GZA GeoEnvironmental

Project: West Branch Bridge #3666
T3 Indian Purchase Twp, ME

Project No: 09.0025976.00 **Figure** S-6

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	33.4	29.8	15.6	18.9	2.1	0.2	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1.5"	100.0		
1"	88.5		
0.75"	66.6		
0.5"	49.8		
0.375"	46.5		
#4	36.8		
#10	21.2		
#20	7.4		
#40	2.3		
#60	0.9		
#100	0.4		
#200	0.2		

* (no specification provided)

Material Description

Grey Sandy fine to coarse GRAVEL, trace Silt

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= GP **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 26.0786 D_{85}= 24.1157 D_{60}= 17.1135
D_{50}= 12.8046 D_{30}= 3.2351 D_{15}= 1.4171
D_{10}= 1.0344 C_u= 16.54 C_c= 0.59

Remarks

Date Received: 06.13.08 **Date Tested:** 06.21.18

Tested By: JS

Checked By: Steven Accetta

Title: Laboratory Manager

Source of Sample: Borings **Depth:** 5.7-7.7' **Date Sampled:**
Sample Number: BB-IWBP-104 / 2D

Thielsch Engineering Inc.

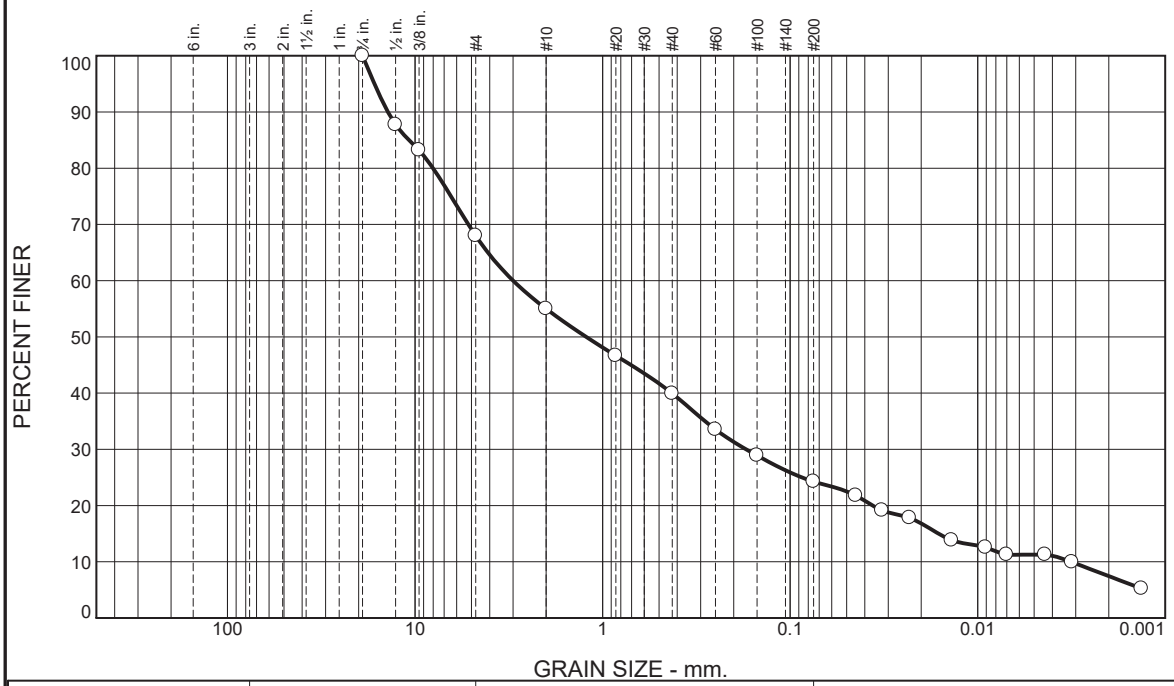
Cranston, RI

Client: GZA GeoEnvironmental

Project: West Branch Bridge #3666
T3 Indian Purchase Twp, ME

Project No: 09.0025976.00 **Figure** S-7

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	32.0	13.0	15.1	15.6	16.8	7.5

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.75"	100.0		
0.5"	87.7		
0.375"	83.2		
#4	68.0		
#10	55.0		
#20	46.7		
#40	39.9		
#60	33.5		
#100	28.9		
#200	24.3		
0.0446 mm.	21.8		
0.0323 mm.	19.1		
0.0231 mm.	17.8		
0.0138 mm.	13.8		
0.0091 mm.	12.5		
0.0070 mm.	11.3		
0.0044 mm.	11.3		
0.0031 mm.	9.9		
0.0013 mm.	5.2		

* (no specification provided)

Material Description

Light Brown Gravelly fine to coarse SAND, some Silt

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

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

Coefficients

D_{90}= 13.9607 D_{85}= 10.7748 D_{60}= 3.0016
D_{50}= 1.2233 D_{30}= 0.1718 D_{15}= 0.0162
D_{10}= 0.0032 C_u= 942.67 C_c= 3.09

Remarks

Date Received: 06.13.18 **Date Tested:** 06.21.18

Tested By: MN / JS

Checked By: Steven Accetta

Title: Laboratory Manager

Source of Sample: Borings **Depth:** 15-15.8' **Date Sampled:**
Sample Number: BB-IWBP-104 / 4D

Thielsch Engineering Inc.

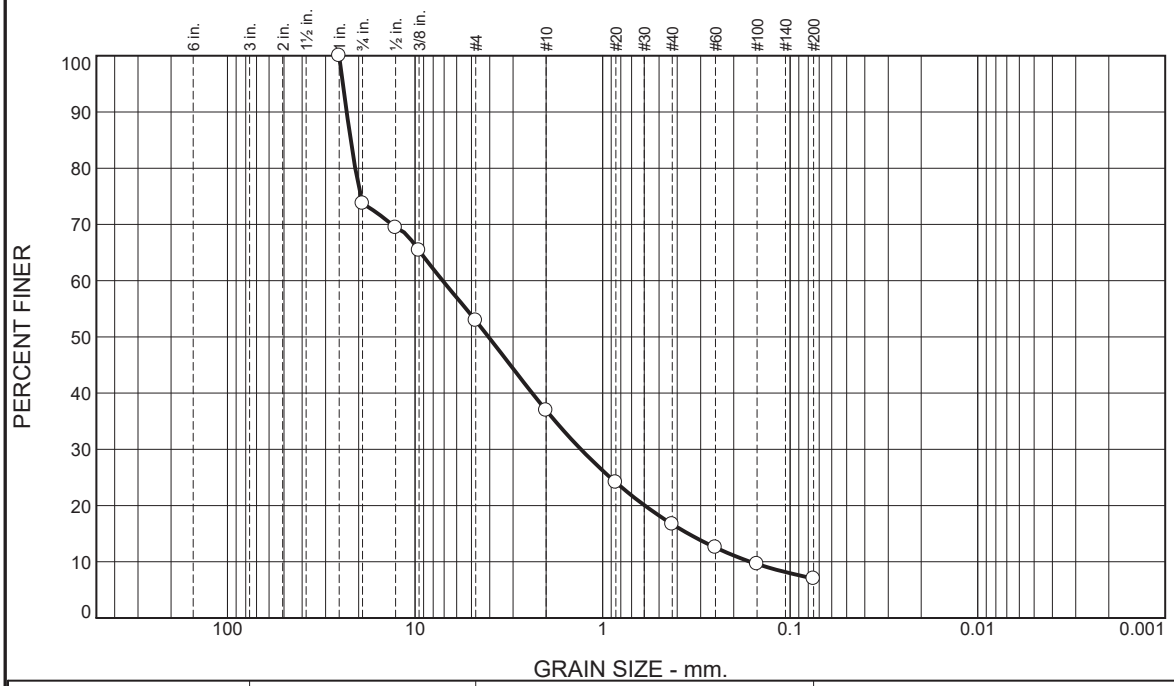
Cranston, RI

Client: GZA GeoEnvironmental

Project: West Branch Bridge #3666
T3 Indian Purchase Twp, ME

Project No: 09.0025976.00 **Figure** S-8

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	26.3	20.8	16.0	20.2	9.7	7.0	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	73.7		
0.5"	69.4		
0.375"	65.4		
#4	52.9		
#10	36.9		
#20	24.1		
#40	16.7		
#60	12.5		
#100	9.6		
#200	7.0		

* (no specification provided)

Material Description

Brown Sandy fine to coarse GRAVEL, trace Silt

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= GW-GM **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 23.1545 D_{85}= 22.0310 D_{60}= 7.1212
D_{50}= 4.0542 D_{30}= 1.3037 D_{15}= 0.3487
D_{10}= 0.1630 C_u= 43.68 C_c= 1.46

Remarks

Date Received: 06.13.18 **Date Tested:** 06.21.18

Tested By: MN / JS

Checked By: Steven Accetta

Title: Laboratory Manager

Source of Sample: Borings **Depth:** 2-4'
Sample Number: BB-IWBP-105 / 1D

Date Sampled:

Thielsch Engineering Inc.

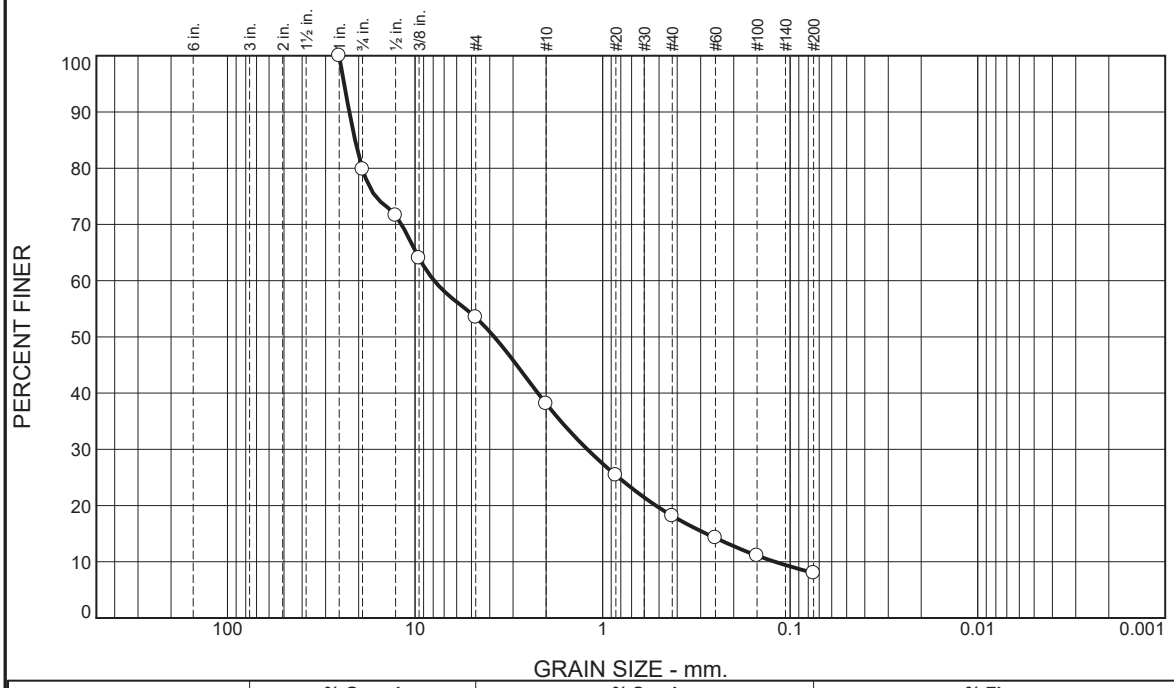
Cranston, RI

Client: GZA GeoEnvironmental

Project: West Branch Bridge #3666
T3 Indian Purchase Twp, ME

Project No: 09.0025976.00 **Figure** S-9

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	20.2	26.3	15.4	19.9	10.2	8.0	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	79.8		
0.5"	71.6		
0.375"	64.0		
#4	53.5		
#10	38.1		
#20	25.4		
#40	18.2		
#60	14.2		
#100	11.1		
#200	8.0		

* (no specification provided)

Material Description

Brown Sandy fine to coarse GRAVEL, trace Silt

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= GW-GM **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 22.4126 D_{85}= 20.8856 D_{60}= 7.9012
D_{50}= 3.7796 D_{30}= 1.2051 D_{15}= 0.2799
D_{10}= 0.1204 C_u= 65.65 C_c= 1.53

Remarks

Date Received: 06.13.18 **Date Tested:** 06.21.18

Tested By: MN / JS

Checked By: Steven Accetta

Title: Laboratory Manager

Source of Sample: Borings **Depth:** 5-7' **Date Sampled:**
Sample Number: BB-IWBP-105 / 2D

Thielsch Engineering Inc.

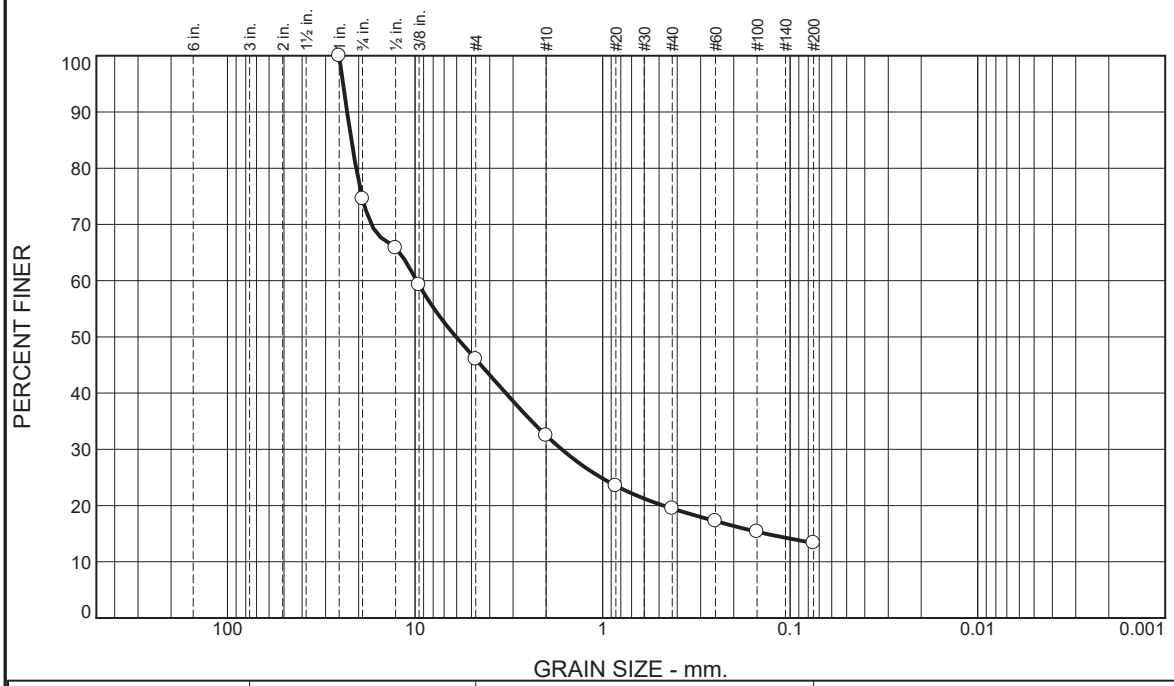
Cranston, RI

Client: GZA GeoEnvironmental

Project: West Branch Bridge #3666
T3 Indian Purchase Twp, ME

Project No: 09.0025976.00 **Figure** S-10

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	25.4	28.6	13.5	13.0	6.2	13.3	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	74.6		
0.5"	65.8		
0.375"	59.2		
#4	46.0		
#10	32.5		
#20	23.5		
#40	19.5		
#60	17.2		
#100	15.3		
#200	13.3		

* (no specification provided)

Material Description

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

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

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

Coefficients

D_{90}= 23.0273 D_{85}= 21.8470 D_{60}= 9.8100
D_{50}= 6.0446 D_{30}= 1.6519 D_{15}= 0.1351
D_{10}= C_u= C_c=

Remarks

Date Received: 06.13.18 **Date Tested:** 06.21.18

Tested By: MN / JS

Checked By: Steven Accetta

Title: Laboratory Manager

Source of Sample: Borings **Depth:** 16.2-17.5 **Date Sampled:**
Sample Number: BB-IWBP-105 / 4D

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental

Project: West Branch Bridge #3666
T3 Indian Purchase Twp, ME

Project No: 09.0025976.00 **Figure** S-11

THIELSCH ENGINEERING

195 Frances Avenue
Cranston RI, 02910
Phone: (401)-467-6454
Fax: (401)-467-2398
thielsch.com
Let's Build a Solid Foundation

Client Information:
GZA GeoEnvironmental
Portland, ME
PM: Blaine Cardali
Assigned By: Andrew Blaisdell
Collected By: EDF

Project Information:
West Branch Bridge #3666
T3 Indian Purchase Twp, ME
GZA Project Number: 09.0025976.01
Summary Page: 1 of 1
Report Date: 04.10.2020

LABORATORY TESTING DATA SHEET, Report No.: 7420-D-113

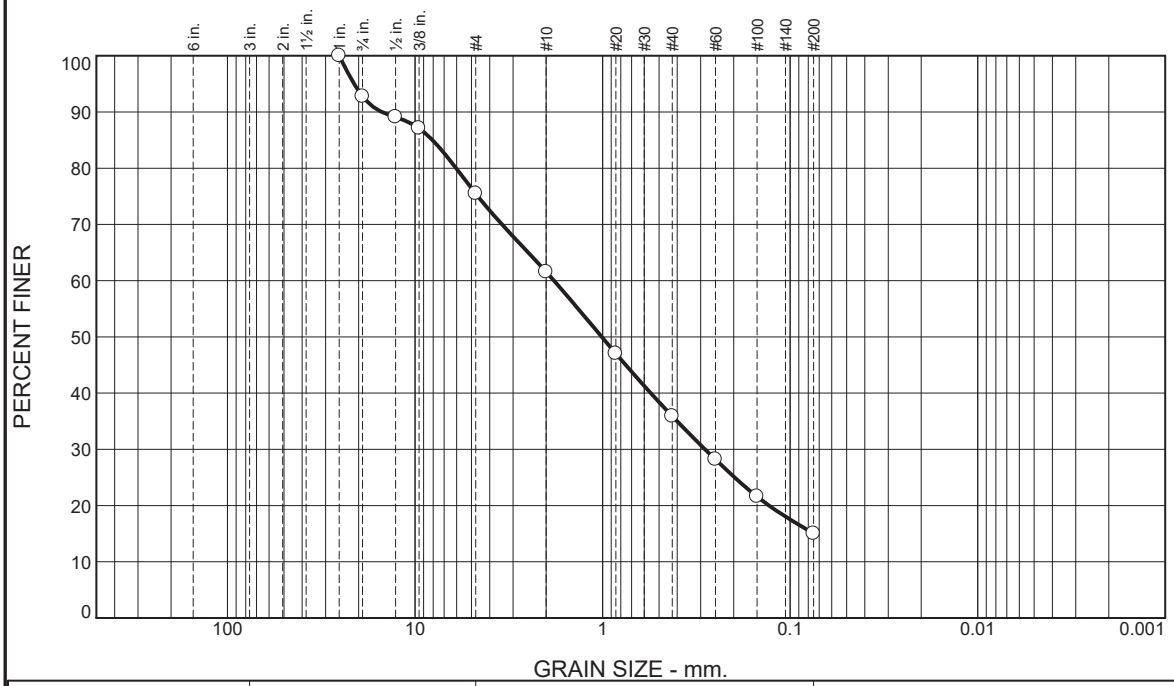
Boring	Sample No.	Depth (Ft)	Laboratory No.	Identification Tests								Proctor / CBR / Permeability Tests								Laboratory Log and Soil Description
				As Received Water Content %	LL %	PL %	Gravel %	Sand %	Fines %	Org. %	G_s	Dry unit wt. pcf	Test Water Content %	γ_d MAX (pcf) W_{opt} (%)	γ_d MAX (pcf) W_{opt} (%) (Corr.)	Target Test Setup as % of Proctor	CBR @ 0.1"	CBR @ 0.2"	Permeability cm/sec	
				D2216	D4318		D6913			D2974	D854			D1557						
BB-IWBP-201	3D	5-7	20-S-885	2.8			24.5	60.5	15.0											Brown f-c SAND, some f-c Gravel, little Silt
BB-IWBP-201	4D	10-12	20-S-886	8.5			62.0	29.1	8.9											Brown f-c GRAVEL, some f-c Sand, trace Silt
BB-IWBP-201	7D	16-18	20-S-887	8.0			46.5	40.6	12.9											Brown SANDY fine to coarse GRAVEL, little Silt
BB-IWBP-201	9D	20-22	20-S-888	8.8			43.9	46.6	9.5											Brown GRAVELLY fine to coarse SAND, trace Silt
BB-IWBP-204	2D	4-4.8	20-S-889	7.8			16.6	59.2	24.2											Brown f-c SAND, some Silt, little fine Gravel
BB-IWBP-204	4D	11-13	20-S-890	7.2			66.3	27.6	6.1											Dark Brown f-c GRAVEL, some f-c Sand, trace Silt
BB-IWBP-204	5D	13-14.3	20-S-891	30.9			18.2	40.3	41.5											Dark Brown SANDY, CLAYEY SILT, little fine Gravel
BB-IWBP-204	7D	17-19	20-S-892	7.1			39.2	35.3	25.5											Brown SANDY fine to coarse GRAVEL, some Silt & Clay

Date Received: *04.03.2020*

Reviewed By:

Date Reviewed: *04.10.2020*

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	7.2	17.3	13.9	25.7	20.9	15.0	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	92.8		
0.5"	89.1		
0.375"	87.1		
#4	75.5		
#10	61.6		
#20	47.0		
#40	35.9		
#60	28.2		
#100	21.6		
#200	15.0		

* (no specification provided)

Material Description

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

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

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

Coefficients

D_{90}= 15.0395 D_{85}= 8.0678 D_{60}= 1.8165
D_{50}= 1.0107 D_{30}= 0.2841 D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 04.03.2020 **Date Tested:** 04.07.2020

Tested By: RR / JM

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: BB-IWBP-201 **Depth:** 5-7' **Date Sampled:**
Sample Number: 3D

Thielsch Engineering Inc.

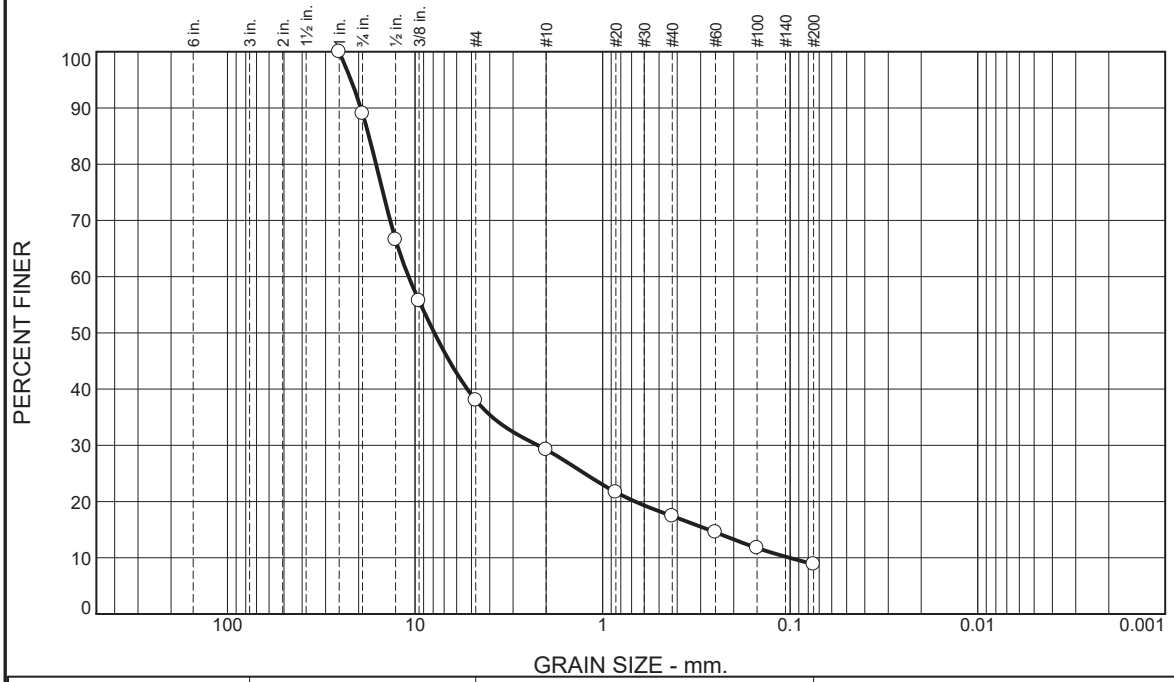
Cranston, RI

Client: GZA GeoEnvironmental

Project: West Branch Bridge #3666
T3 Indian Purchase Twp, ME

Project No: 09.0025976.01 **Figure** 20-S-885

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	11.0	51.0	8.8	11.8	8.5	8.9	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	89.0		
0.5"	66.6		
0.375"	55.7		
#4	38.0		
#10	29.2		
#20	21.7		
#40	17.4		
#60	14.5		
#100	11.7		
#200	8.9		

* (no specification provided)

Material Description

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

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= GP-GM **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 19.4477 D_{85}= 17.6267 D_{60}= 10.8399
D_{50}= 7.8859 D_{30}= 2.2116 D_{15}= 0.2723
D_{10}= 0.1009 C_u= 107.40 C_c= 4.47

Remarks

Date Received: 04.03.2020 **Date Tested:** 04.07.2020

Tested By: RR / JM

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: BB-IWBP-201 **Depth:** 10-12' **Date Sampled:**
Sample Number: 4D

Thielsch Engineering Inc.

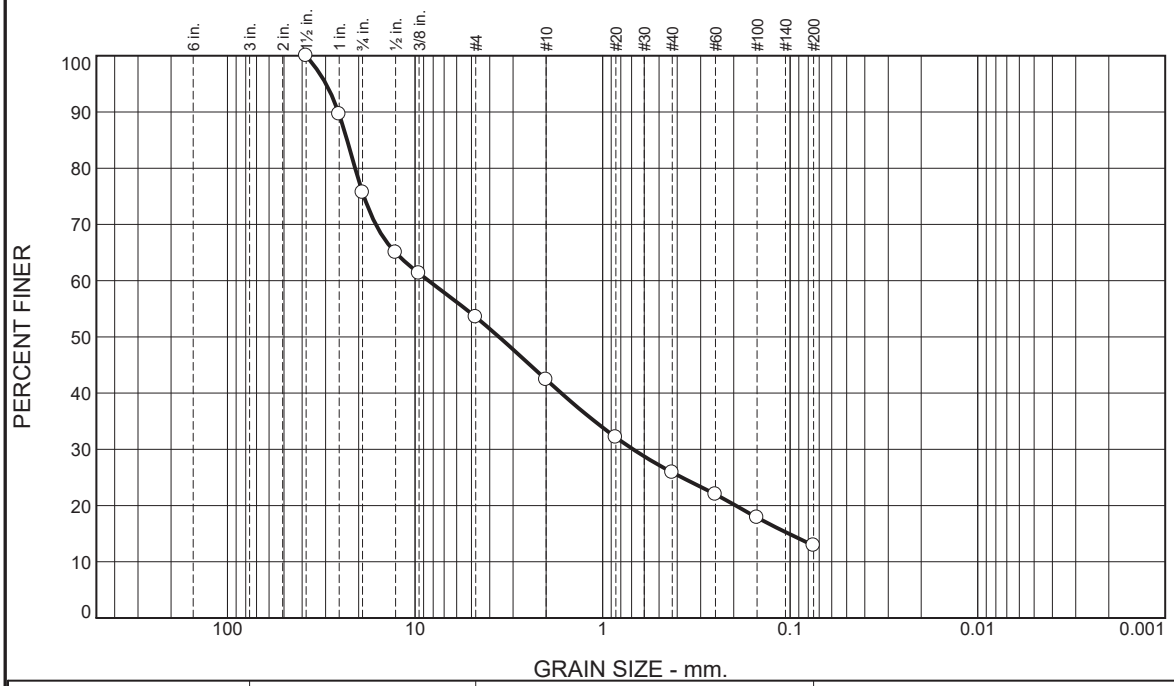
Cranston, RI

Client: GZA GeoEnvironmental

Project: West Branch Bridge #3666
T3 Indian Purchase Twp, ME

Project No: 09.0025976.01 **Figure** 20-S-886

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	24.3	22.2	11.1	16.5	13.0	12.9	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1-1/2"	100.0		
1"	89.6		
3/4"	75.7		
1/2"	65.0		
3/8"	61.3		
#4	53.5		
#10	42.4		
#20	32.1		
#40	25.9		
#60	22.0		
#100	17.9		
#200	12.9		

* (no specification provided)

Material Description

Brown SANDY fine to coarse GRAVEL, little Silt

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

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

Coefficients

D_{90}= 25.6338 D_{85}= 23.0176 D_{60}= 8.4711
D_{50}= 3.5700 D_{30}= 0.6870 D_{15}= 0.1016
D_{10}= C_u= C_c=

Remarks

Date Received: 04.03.2020 **Date Tested:** 04.07.2020

Tested By: RR / JM

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: BB-IWBP-201 **Depth:** 16-18' **Date Sampled:**

Sample Number: 7D

Thielsch Engineering Inc.

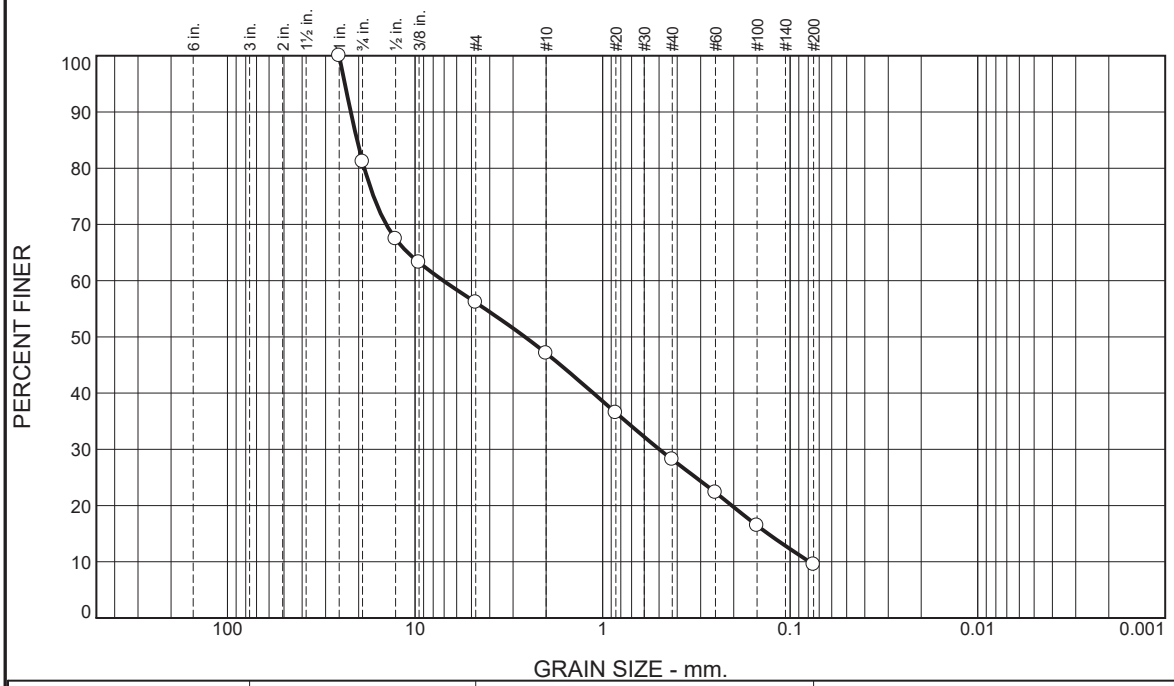
Cranston, RI

Client: GZA GeoEnvironmental

Project: West Branch Bridge #3666
T3 Indian Purchase Twp, ME

Project No: 09.0025976.01 **Figure** 20-S-887

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	18.8	25.1	9.0	18.9	18.7	9.5	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	81.2		
0.5"	67.4		
0.375"	63.2		
#4	56.1		
#10	47.1		
#20	36.5		
#40	28.2		
#60	22.3		
#100	16.4		
#200	9.5		

* (no specification provided)

Material Description

Brown GRAVELLY fine to coarse SAND, trace Silt

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SP-SM **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 22.0169 D_{85}= 20.3677 D_{60}= 7.0629
D_{50}= 2.5974 D_{30}= 0.4978 D_{15}= 0.1311
D_{10}= 0.0788 C_u= 89.60 C_c= 0.45

Remarks

Date Received: 04.03.2020 **Date Tested:** 04.07.2020

Tested By: RR / JM

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: BB-IWBP-201 **Depth:** 20-22' **Date Sampled:**

Sample Number: 9D

Thielsch Engineering Inc.

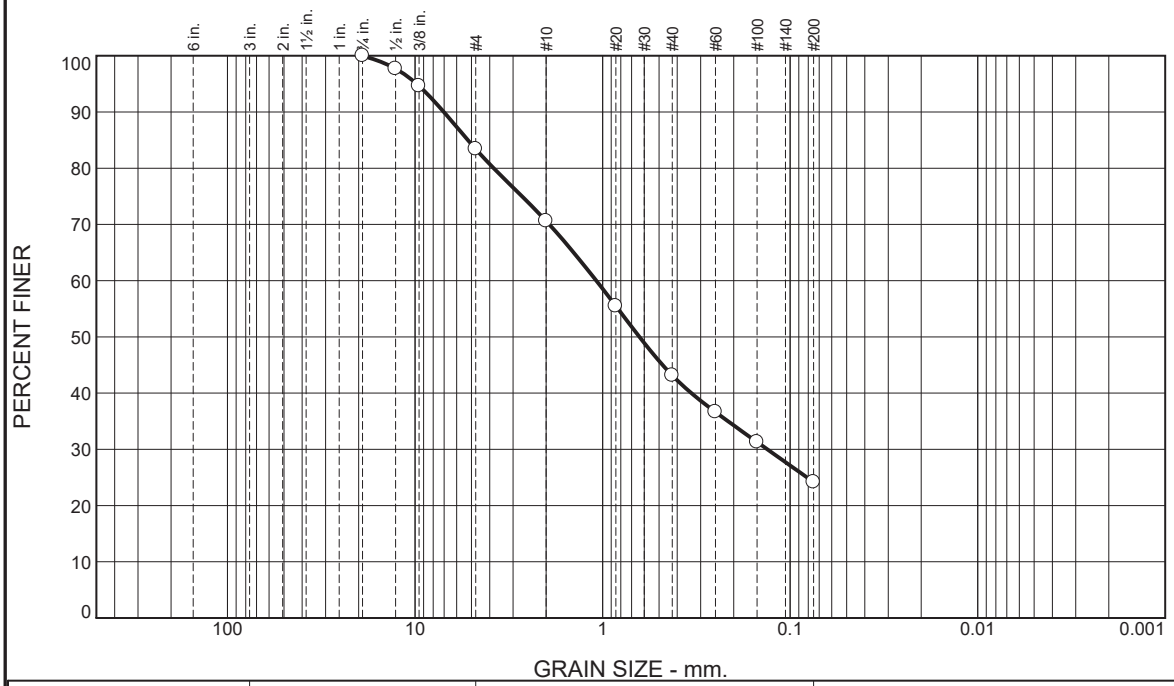
Cranston, RI

Client: GZA GeoEnvironmental

Project: West Branch Bridge #3666
T3 Indian Purchase Twp, ME

Project No: 09.0025976.01 **Figure** 20-S-888

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	16.6	12.8	27.5	18.9	24.2	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.75"	100.0		
0.5"	97.7		
0.375"	94.6		
#4	83.4		
#10	70.6		
#20	55.5		
#40	43.1		
#60	36.6		
#100	31.3		
#200	24.2		

* (no specification provided)

Material Description

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

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

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

Coefficients

D_{90}= 7.0195 D_{85}= 5.2278 D_{60}= 1.0807
D_{50}= 0.6363 D_{30}= 0.1323 D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 04.03.2020 **Date Tested:** 04.07.2020

Tested By: RR / JM

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: BB-IWBP-204 **Depth:** 4-4.8' **Date Sampled:**
Sample Number: 2D

Thielsch Engineering Inc.

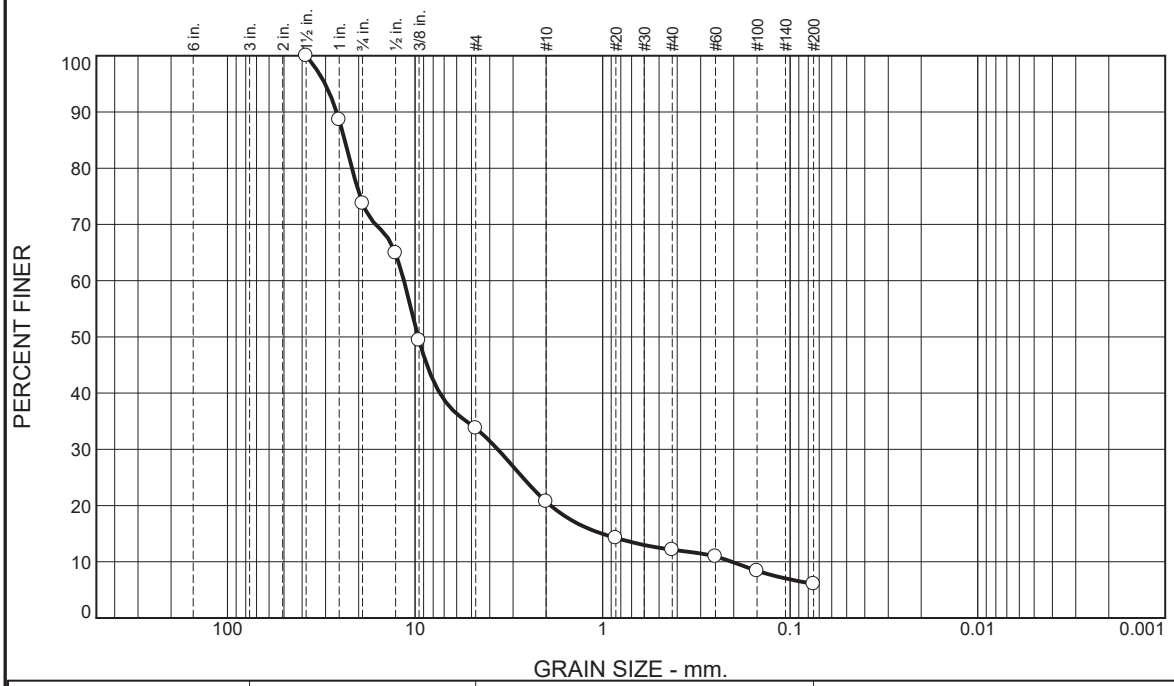
Cranston, RI

Client: GZA GeoEnvironmental

Project: West Branch Bridge #3666
T3 Indian Purchase Twp, ME

Project No: 09.0025976.01 **Figure** 20-S-889

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	26.3	40.0	13.0	8.6	6.0	6.1	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1-1/2"	100.0		
1"	88.6		
3/4"	73.7		
1/2"	64.9		
3/8"	49.4		
#4	33.7		
#10	20.7		
#20	14.2		
#40	12.1		
#60	10.9		
#100	8.4		
#200	6.1		

* (no specification provided)

Material Description

Dark Brown f-c GRAVEL, some f-c Sand, trace Silt

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= GP-GM **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 26.1696 D_{85}= 23.7045 D_{60}= 11.4619
D_{50}= 9.6344 D_{30}= 3.6230 D_{15}= 1.0087
D_{10}= 0.2040 C_u= 56.18 C_c= 5.61

Remarks

Date Received: 04.03.2020 **Date Tested:** 04.07.2020

Tested By: RR / JM

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: BB-IWBP-204 **Depth:** 11-13' **Date Sampled:**
Sample Number: 4D

Thielsch Engineering Inc.

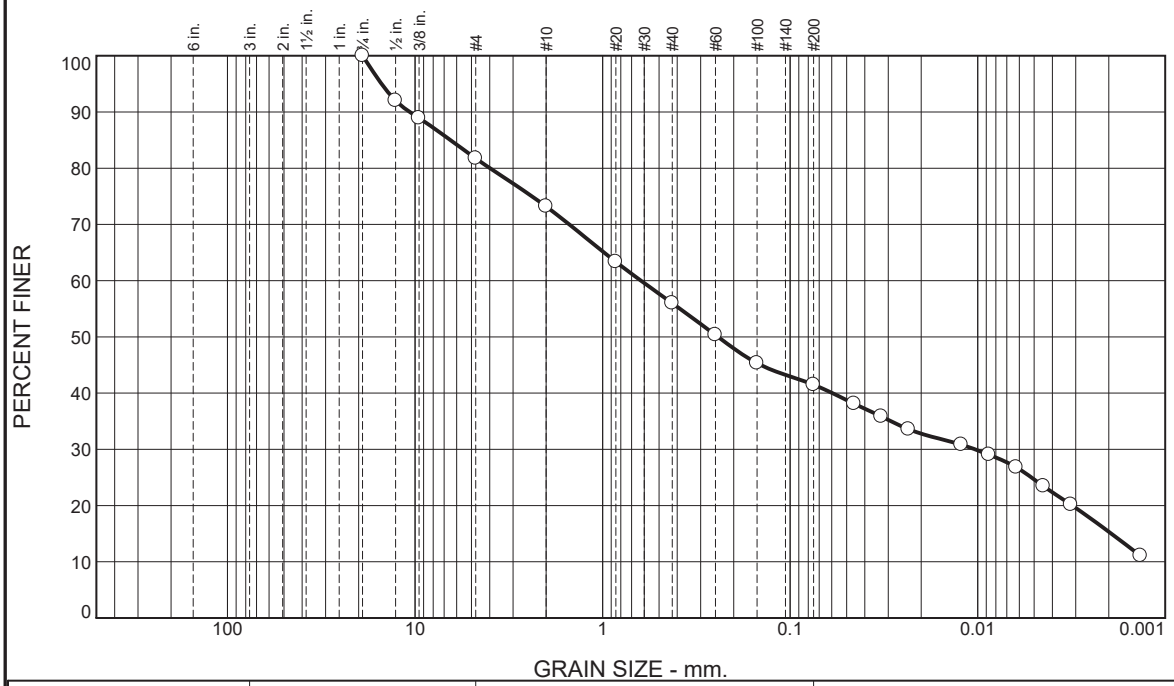
Cranston, RI

Client: GZA GeoEnvironmental

Project: West Branch Bridge #3666
T3 Indian Purchase Twp, ME

Project No: 09.0025976.01 **Figure** 20-S-890

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	18.2	8.6	17.2	14.5	26.2	15.3

Test Results (D7928 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.75"	100.0		
0.5"	92.0		
0.375"	88.9		
#4	81.8		
#10	73.2		
#20	63.3		
#40	56.0		
#60	50.3		
#100	45.3		
#200	41.5		
0.0456 mm.	38.1		
0.0326 mm.	35.8		
0.0233 mm.	33.6		
0.0122 mm.	30.8		
0.0087 mm.	29.0		
0.0062 mm.	26.8		
0.0045 mm.	23.5		
0.0032 mm.	20.2		
0.0014 mm.	11.1		

* (no specification provided)

Material Description

Dark Brown SANDY, CLAYEY SILT, little fine Gravel

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= SC AASHTO (M 145)= A-6

Coefficients

D_{90}= 10.6837 D_{85}= 6.4733 D_{60}= 0.6242
D_{50}= 0.2424 D_{30}= 0.0103 D_{15}= 0.0019
D_{10}= C_u= C_c=

Remarks

Sample visually classified as plastic. Sample rolled to 1/16".

Date Received: 04.03.2020 **Date Tested:** 04.08.2020

Tested By: RR / JM

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: BB-IWBP-204 **Depth:** 13-14.3' **Date Sampled:**
Sample Number: 5D

Thielsch Engineering Inc.

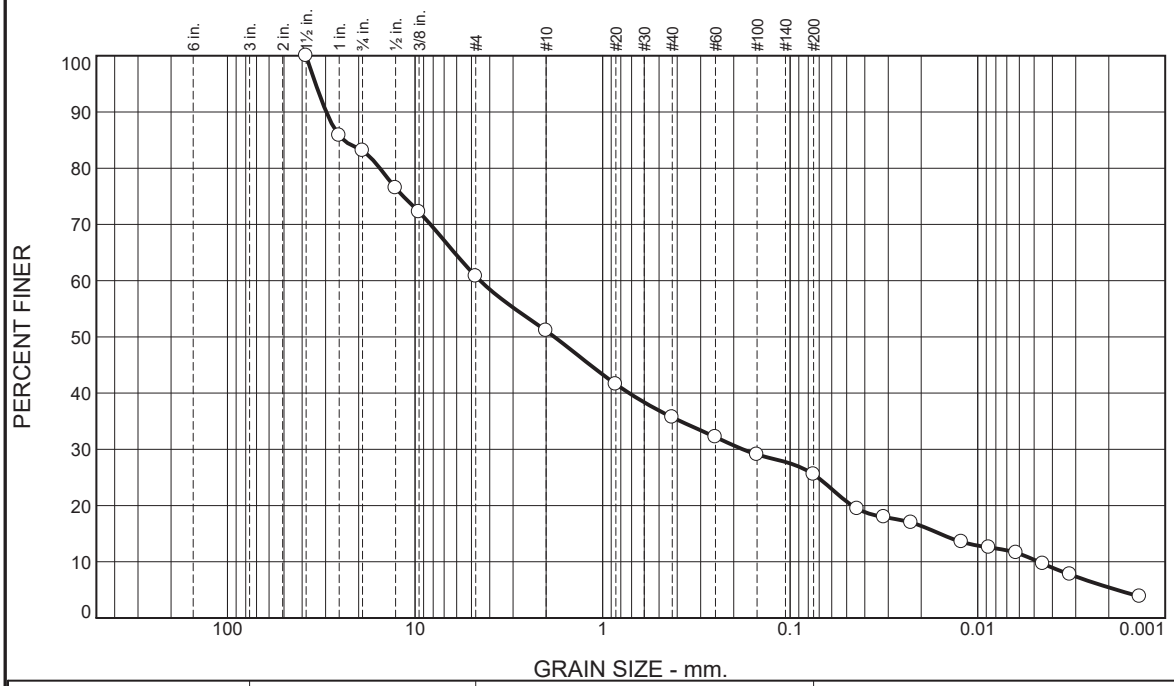
Cranston, RI

Client: GZA GeoEnvironmental

Project: West Branch Bridge #3666
T3 Indian Purchase Twp, ME

Project No: 09.0025976.01 **Figure** 20-S-891

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	16.9	22.3	9.7	15.4	10.2	20.1	5.4

Test Results (D7928 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1-1/2"	100.0		
1"	85.8		
3/4"	83.1		
1/2"	76.5		
3/8"	72.2		
#4	60.8		
#10	51.1		
#20	41.6		
#40	35.7		
#60	32.2		
#100	29.1		
#200	25.5		
0.0439 mm.	19.4		
0.0316 mm.	18.0		
0.0227 mm.	17.0		
0.0122 mm.	13.5		
0.0087 mm.	12.6		
0.0062 mm.	11.6		
0.0045 mm.	9.7		
0.0032 mm.	7.7		
0.0014 mm.	3.8		

* (no specification provided)

Material Description

Brown SANDY fine to coarse GRAVEL, some Silt & Clay

Atterberg Limits (ASTM D 4318)

PL= **LL=** **PI=**

Classification

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

Coefficients

D_{90}= 29.7128 D_{85}= 24.0745 D_{60}= 4.4940
D_{50}= 1.8062 D_{30}= 0.1787 D_{15}= 0.0159
D_{10}= 0.0047 C_u= 949.82 C_c= 1.50

Remarks

Sample visually classified as plastic. Sample rolled to 1/8".

Date Received: 04.03.2020 **Date Tested:** 04.08.2020

Tested By: RR / JM

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: BB-IWBP-204 **Depth:** 17-19' **Date Sampled:**
Sample Number: 7D

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental

Project: West Branch Bridge #3666
T3 Indian Purchase Twp, ME

Project No: 09.0025976.01 **Figure** 20-S-892

THIELSCH ENGINEERING

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

West Branch Bridge #366

MDOT Project Number:

GZA Project Number: 09.0025976.01

Town(s): T3 Indian Purchase TWP, ME

Boring & Sample Identification Number	Station (Feet)	Sample No.	Depth (Feet)	Lab Number	Organic %	W.C.	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-IWBP-201		3D	5-7	20-S-885		2.8			SM	A-1-b	II
BB-IWBP-201		4D	10-12	20-S-886		8.5			GP-GM	A-1-a	0
BB-IWBP-201		7D	16-18	20-S-887		8.0			GM	A-1-a	I
BB-IWBP-201		9D	20-22	20-S-888		8.8			SP-SM	A-1-a	0
BB-IWBP-204		2D	4-4.8	20-S-889		7.8			SM	A-1-b	II
BB-IWBP-204		4D	11-13	20-S-890		7.2			GP-GM	A-1-a	0
BB-IWBP-204		5D	13-14.3	20-S-891		30.9			SC	A-6	III
BB-IWBP-204		7D	17-19	20-S-892		7.1			GC	A-2-4	III

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

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

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

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

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

THIELSCH ENGINEERING
195 Frances Avenue
Cranston RI, 02910
Phone: (401)-467-6454
Fax: (401)-467-2398
http://www.thielsch.com
Let's Build a Solid Foundation

Client Information:
GZA GeoEnvironmental, Inc
Portland, ME
PM: Theodore Baire
Assigned By: EDF
Collected By: EDF

Project Information:
West Branch Bridge #3666
T3 Indian Purchase Twp, ME
GZA Project Number: 09.0025976.00
Summary Page: 1 of 3
Report Date: 07.11.2018

LABORATORY TESTING DATA SHEET

Boring No.	Sample No.	Depth (ft)	Laboratory No.	Specimen Data: Mohs Hard-ness	Diameter (in)	Length (in)	(1) Unit Weight (PCF)	(2) Wet Density (PCF)	Bulk G_s	Compressive Strength Tests: (3) Other Tests	(4) Strength PSI	(5) Strain %	(6) E sec PSI EE+06	(7) Poisson's Ratio	$\sigma\tau$ PSI	Is_{50} PSI	(8) s_c PSI	Rock Formation or Description or Remarks
BB-IWBP-101	R1	27.1-27.9	S-12		1.981	4.702	167.0				18659	0.384	4.02	0.08				SCHIST; fresh break
Note:																		
BB-IWBP-105	R5	38-38.8	S-16		1.977	4.716	173.9				18510	0.581	2.77	0.20				SCHIST; fresh break
Note: Minor breaks occurred around 1.8, 4.0, 7.3, and 9.5 kpsi. Break at 4 kpsi had the biggest impact and did affect poisson's ratio and the secant modulus.																		

(1) Volume Determined By Measuring Dimensions

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

Notes

(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

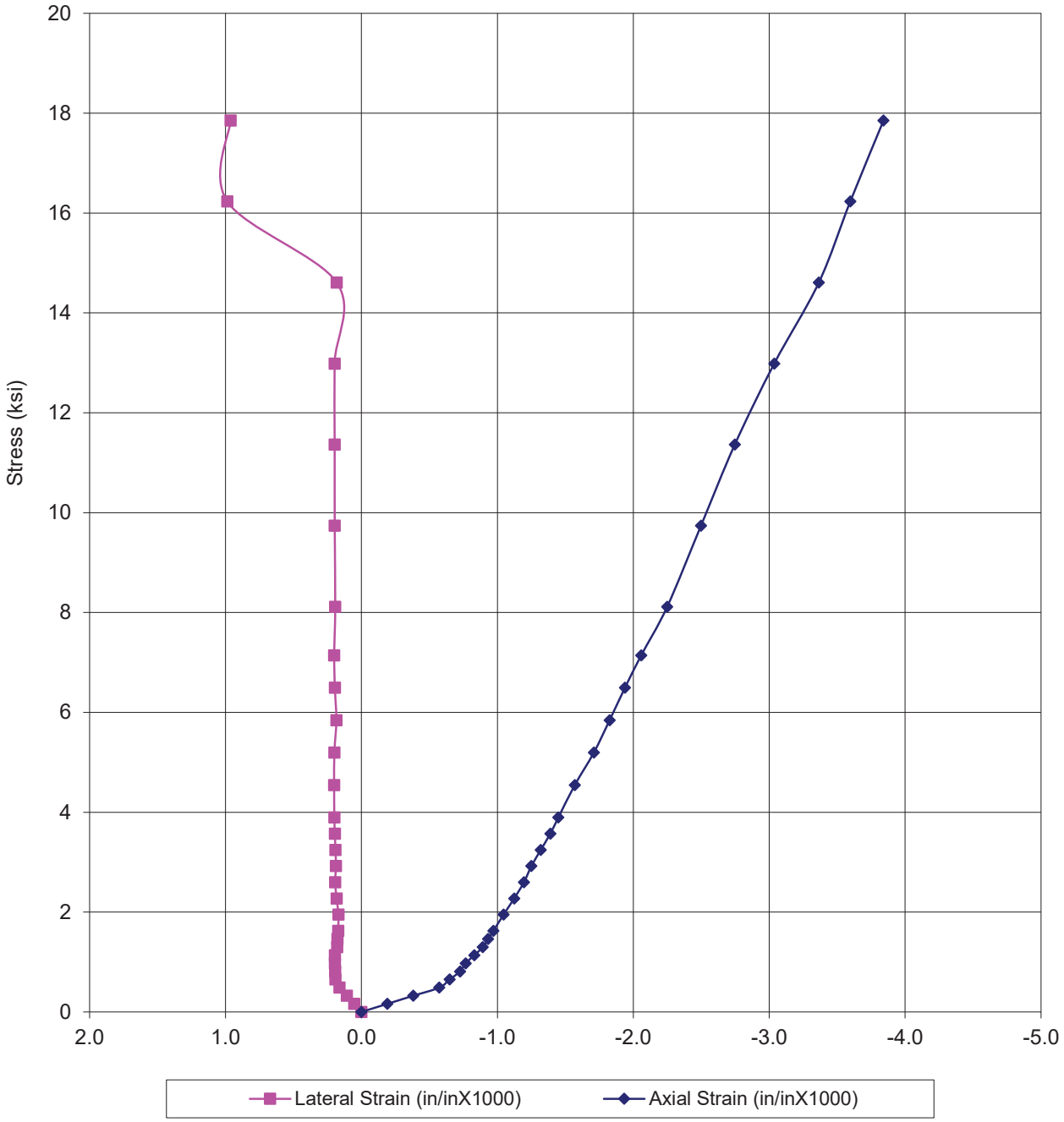
(7) Represents Secant Poisson's Ratio at 50% of Total Failure Stress

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

Reviewed By 

Date Reviewed *07.12.2018*

West Branch Bridge #3666
T3 Indian Purchase Twp, ME

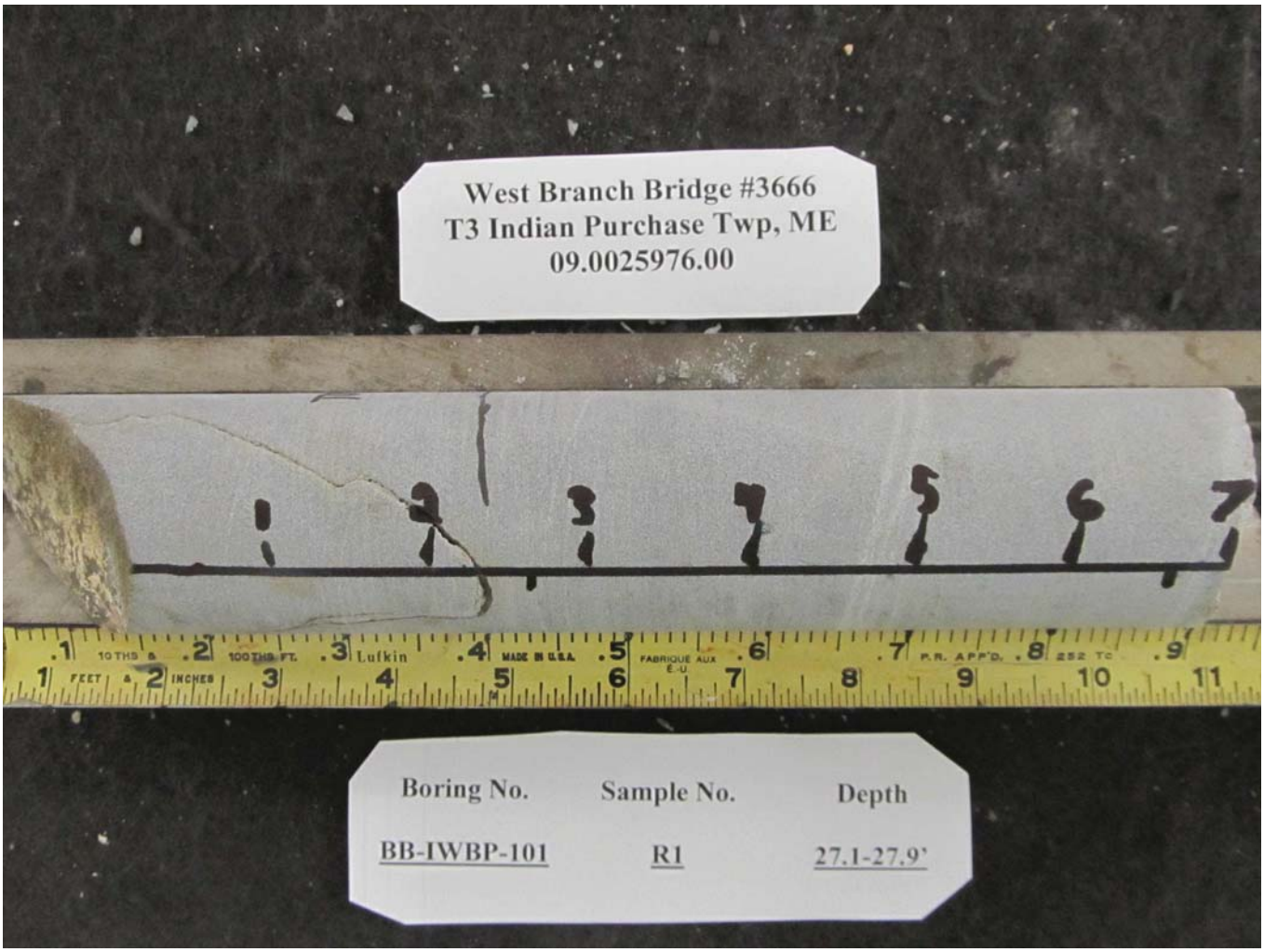


Rock Unconfined Compression Testing - ASTM D7012

Boring No. BB-IWBP-101
Sample No. R1
Depth: 27.1-27.9

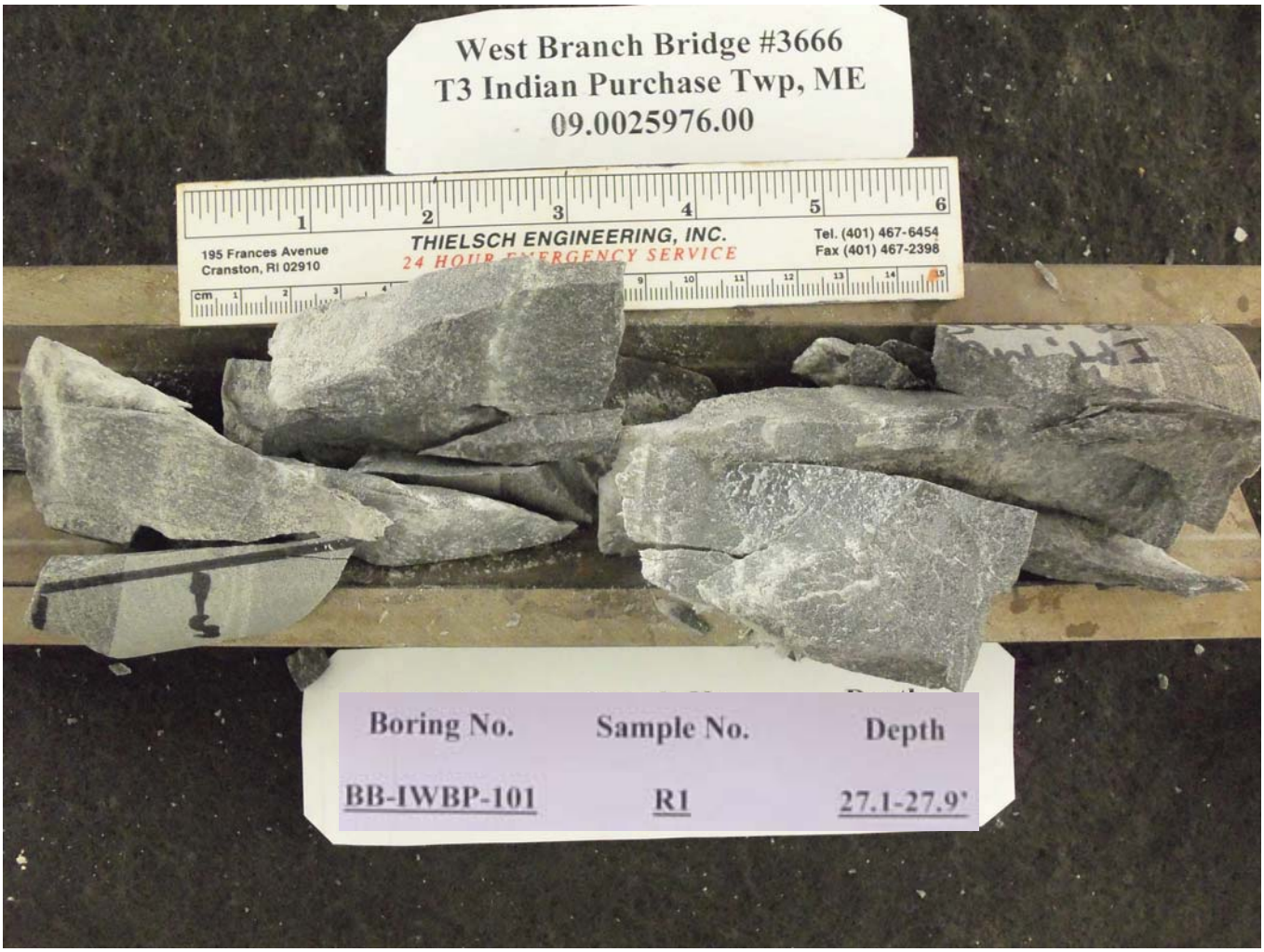
File No. 09.0025976.00
Date: 07.10.2018
Test No. U-12

THIELSCH ENGINEERING

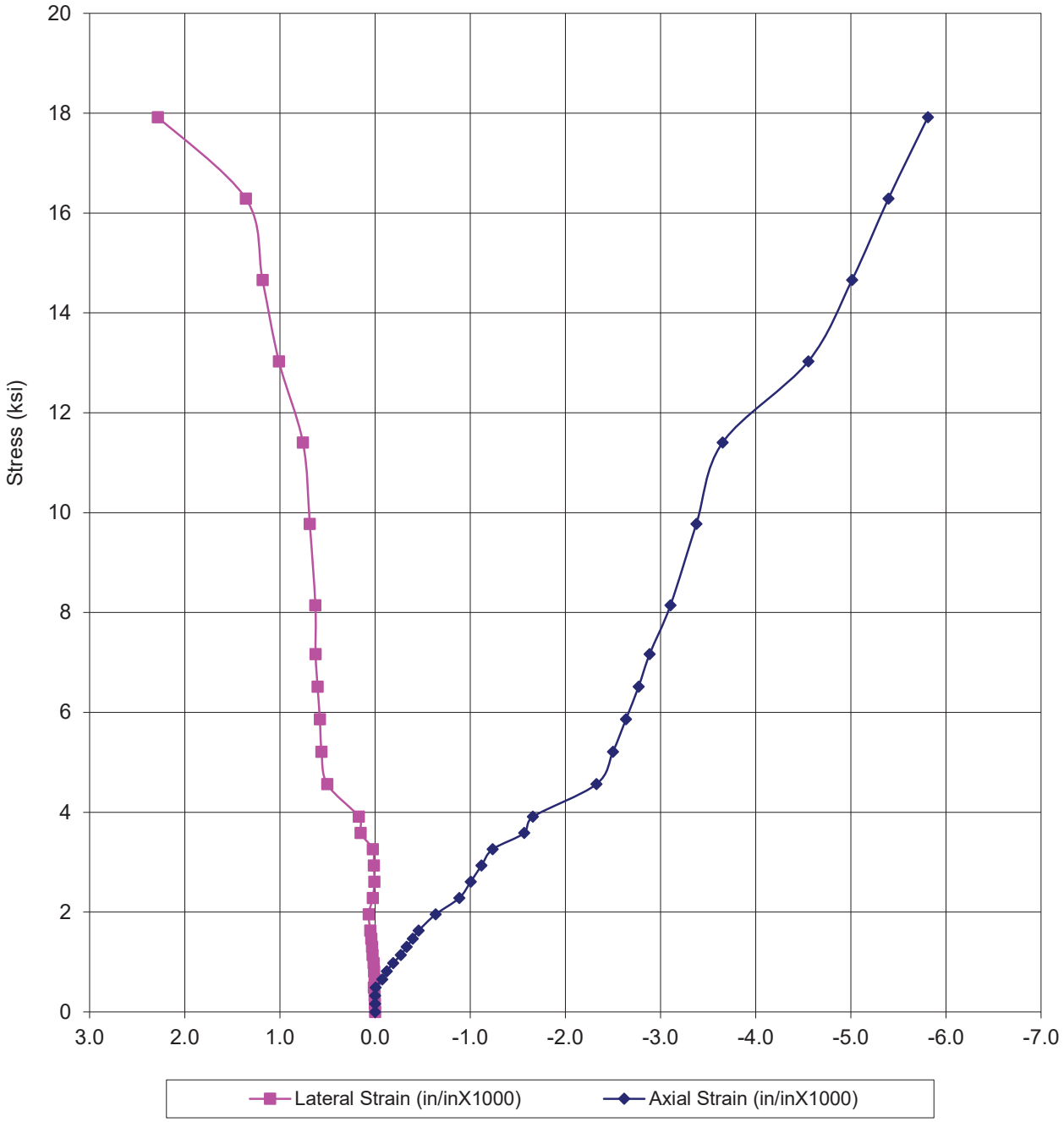

West Branch Bridge #3666
T3 Indian Purchase Twp, ME
09.0025976.00
Lufkin
MADE IN U.S.A.
FABRIQUE AUX E.-U.
FEET & INCHES
Boring No.
Sample No.
Depth
BB-IWBP-101
R1
27.1-27.9'


West Branch Bridge #3666
T3 Indian Purchase Twp, ME
09.0025976.00
Lufkin
MADE IN U.S.A.
FABRIQUE AUX E.-U.
P.R. APP'D.
Boring No.
Sample No.
Depth
BB-IWBP-101
R1
27.1-27.9'


West Branch Bridge #3666
T3 Indian Purchase Twp, ME
09.0025976.00
10THS & 100THS FT.
Lufkin
MADE IN U.S.A.
FEET & INCHES
Boring No.
Sample No.
Depth
BB-IWBP-101
R1
27.1-27.9'


West Branch Bridge #3666
T3 Indian Purchase Twp, ME
09.0025976.00
THIELSCH ENGINEERING, INC.
195 Frances Avenue
Cranston, RI 02910
Tel. (401) 467-6454
Fax (401) 467-2398
Boring No.
Sample No.
Depth
BB-IWBP-101
R1
27.1-27.9'

Est Branch Bridge #3666
T3 Indian Purchase Twp, ME



Rock Unconfined Compression Testing - ASTM D7012

Boring No. BB-IWBP-105
Sample No. R5
Depth: 38-38.8

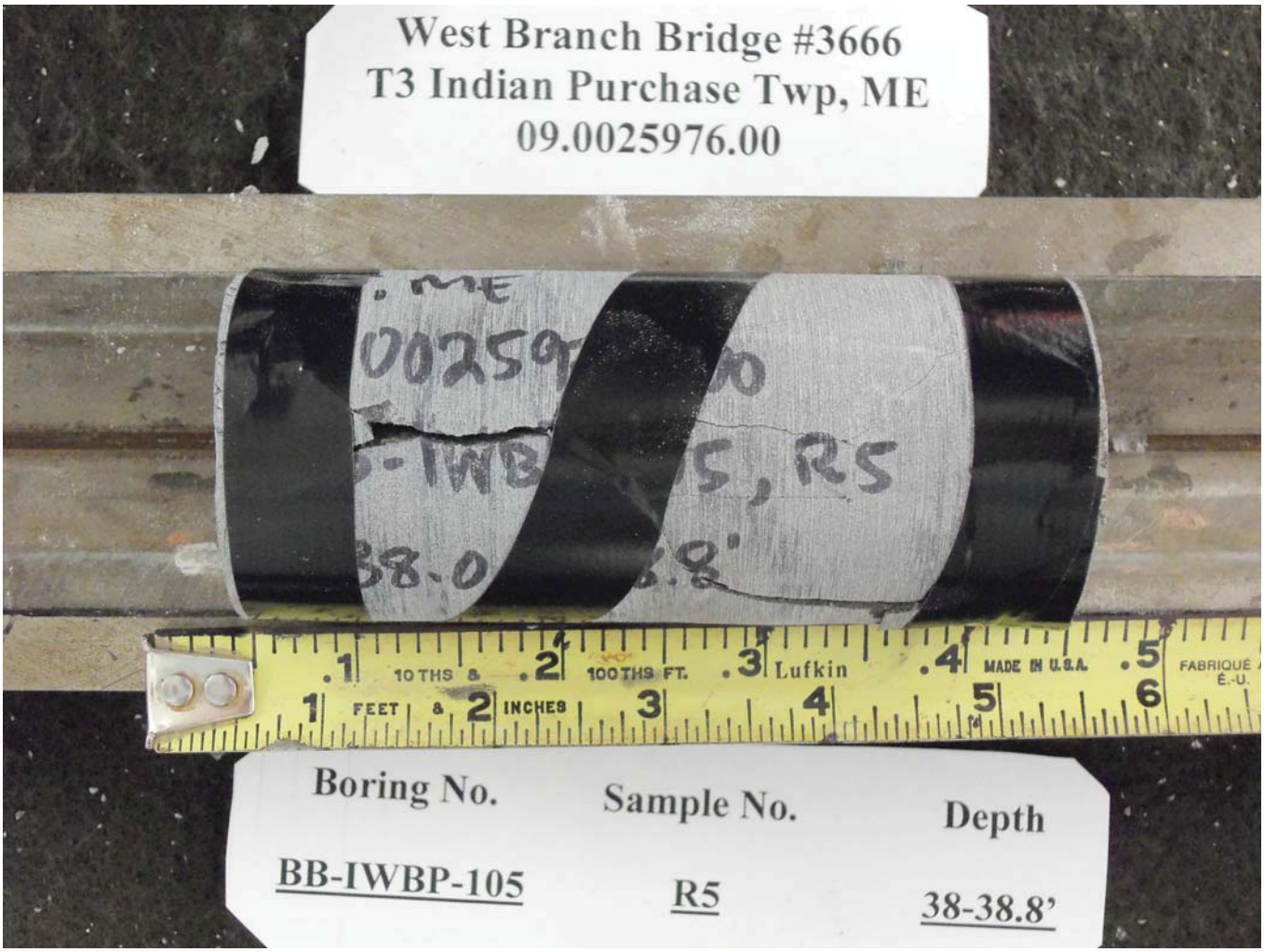
File No. 09.0025976.00
Date: 07.11.2018
Test No. U-16

THIELSCH ENGINEERING


West Branch Bridge #3666
T3 Indian Purchase Twp, ME
09.0025976.00
1 2 3 4 5 6
Lufkin
MADE IN U.S.A.
Boring No.
Sample No.
Depth
BB-IWBP-105
R5
38-38.8'


West Branch Bridge #3666
T3 Indian Purchase Twp, ME
09.0025976.00
Boring No.
Sample No.
Depth
BB-IWBP-105
R5
38-38.8'


West Branch Bridge #3666
T3 Indian Purchase Twp, ME
09.0025976.00
105, R
Lufkin
MADE IN U.S.A.
FABRIQUE AUX E.-U.
10THS & FEET & 2 INCHES
100THS FT.
Boring No.
Sample No.
Depth
BB-IWBP-105
R5
38-38.8'


West Branch Bridge #3666
T3 Indian Purchase Twp, ME
09.0025976.00
Boring No.
BB-IWBP-105
Sample No.
R5
Depth
38-38.8'

THIELSCH ENGINEERING
195 Frances Avenue
Cranston RI, 02910
Phone: (401)-467-6454
Fax: (401)-467-2398
http://www.thielsch.com
Let's Build a Solid Foundation

Client Information:
GZA GeoEnvironmental, Inc
Portland, ME
PM: Theodore Baire
Assigned By: EDF
Collected By: EDF

Project Information:
West Branch Bridge #3666
T3 Indian Purchase Twp, ME
GZA Project Number: 09.0025976.00
Summary Page: 2 of 3
Report Date: 07.11.2018

LABORATORY TESTING DATA SHEET

Boring No.	Sample No.	Depth (ft)	Laboratory No.	Specimen Data: Mohs Hardness	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	Poisson's Ratio	στ PSI	(7) Is PSI	(8) s_c PSI	Rock Formation or Description or Remarks
BB-IWBP-102	R1	15.3-15.9	13D		1.924	1.788	170.8			PLD						1005	24109	SCHIST; irregular break, shattered
			13D		1.977	2.122	163.0			PLD						768	18421	
			13A															Sample had an irregular break and axial point load test could not be performed
			13A															
BB-IWBP-104	R4	31.9-32.3	14D		1.966	2.108	164.0			PLD						466	11177	SCHIST; irregular break
			14D		1.968	2.186	166.3			PLD						310	7436	
			14A		1.967	1.8	144.4*			PLA						1344	32256	
			14A		1.966	1.2	169.8*			PLA						906	21733	

Notes

(1) Volume Determined By Measuring Dimensions

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

(3) PLD=Point Load (diametrical), PLA= Point Load (Axial) ST= Splitting Tensile U= Unconfined Compressive Strength

(4) Taken at Peak Deviator Stress

(5) Strain at Peak Deviator Stress

(6) Represents Secant Modulus at 50% of Total Failure Stress

(7) The Point Load Index is represented as Is

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

Reviewed By SAH

Date Reviewed 07.12.2018

THIELSCH ENGINEERING
195 Frances Avenue
Cranston RI, 02910
Phone: (401)-467-6454
Fax: (401)-467-2398
http://www.thielsch.com
Let's Build a Solid Foundation

Client Information:
GZA GeoEnvironmental, Inc
Portland, ME
PM: Theodore Baire
Assigned By: EDF
Collected By: EDF

Project Information:
West Branch Bridge #3666
T3 Indian Purchase Twp, ME
GZA Project Number: 09.0025976.00
Summary Page: 3 of 3
Report Date: 07.11.2018

LABORATORY TESTING DATA SHEET

Boring No.	Sample No.	Depth (ft)	Laboratory No.	Specimen Data: Mohs Hardness	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	Poisson's Ratio	$\sigma\tau$ PSI	(7) Is PSI	(8) s_c PSI	Rock Formation or Description or Remarks
BB-IWBP-105	R2	23-23.3	15D		1.972	2.421	172.7			PLD						1183	28389	SCHIST; Irregular break
			15D		1.97	2.100	157.0			PLD						1391	33394	
			15A		1.975	1.240	161.8*			PLA						1128	27073	
			15A		1.973	1.219	122.8*			PLA						411	9865	

Note: (*) Axial point load test was done with the remains from the diametral point load test, so the unit weights for the axial are estimates.

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) The Point Load Index is represented as Is

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

Reviewed By _____ Date Reviewed 07.12.2018

THIELSCH ENGINEERING
195 Frances Avenue
Cranston RI, 02910
Phone: (401)-467-6454
Fax: (401)-467-2398
thielsch.com
Let's Build a Solid Foundation

Client Information:
GZA GeoEnvironmental
South Portland, ME
PM: Blaine Cardali
Assigned By: ARB
Collected By: MRJ

Project Information:
West Branch Bridge WIN 23236.00
T3 Indian Purchase Twp, ME
GZA Project Number: 09.0025976.01

Report Date: 03.26.2020

LABORATORY TESTING DATA SHEET, Report No.: 7420-C-151

Boring No.	Sample No.	Depth (ft)	Laboratory No.	Specimen Data						Compressive Strength Tests								Rock Formation or Description or Remarks
				Mohs Hard-ness	Diameter (in)	Length (in)	(1) Unit Weight (PCF)	(2) Wet Density (PCF)	Bulk G_s	(3) Other Tests	(4) Strength PSI	(5) Strain %	(6) E sec PSI EE+06	(7) Poisson's Ratio	$\sigma\tau$ PSI	Is_{50} PSI	(8) s_c PSI	
BB-IWBP-103	R1	1.5-1.8	20-S-761D		1.980	1.354	165.7			PLD						1964	47136	Slate
Break was fresh.																		
BB-IWBP-103	R1	1.5-1.8	20-S-761A		1.994	1.613	155.9			PLA						1148	27552	Slate
Break was fresh.																		
BB-IWBP-202	R1	3.3-3.6	20-S-762D		1.985	1.390	168.1			PLD						1649	39576	Slate
Load was applied normal to foliation. Break was fresh.																		
BB-IWBP-202	R1	3.3-3.6	20-S-762A		2.011	1.849	161.1			PLA						1268	30432	Slate
Load was applied normal to foliation. Break was fresh.																		

Notes

(1) Volume Determined By Measuring Dimensions

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

Notes

(3) PLD=Point Load (diametrical), PLA= Point Load (Axial) ST= Splitting Tensile U= Unconfined Compressive Strength

(4) Taken at Peak Deviator Stress

(5) Strain at Peak Deviator Stress

(6) Represents Secant Modulus at 50% of Total Failure Stress

(7) Represents Secant Poisson's Ratio at 50% of Total Failure Stress

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

Date Received: *03.24.2020*

Reviewed By:

Date Reviewed: *03.30.2020*

THIELSCH ENGINEERING
195 Frances Avenue
Cranston RI, 02910
Phone: (401)-467-6454
Fax: (401)-467-2398
thielsch.com
Let's Build a Solid Foundation

Client Information:
GZA GeoEnvironmental
South Portland, ME
PM: Blaine Cardali
Assigned By: ARB
Collected By: MRJ

Project Information:
West Branch Bridge WIN 23236.00
T3 Indian Purchase Twp, ME
GZA Project Number: 09.0025976.01
Summary Page: 2 of 2
Report Date: 03.26.2020

LABORATORY TESTING DATA SHEET, Report No.: 7420-C-151

Boring No.	Sample No.	Depth (ft)	Laboratory No.	Specimen Data						Compressive Strength Tests									Rock Formation or Description or Remarks
				Mohs Hard-ness	Diameter (in)	Length (in)	(1) Unit Weight (PCF)	(2) Wet Density (PCF)	Bulk G_s	(3) Other Tests	(4) Strength PSI	(5) Strain %	(6) E sec PSI EE+06	(7) Poisson's Ratio	στ PSI	Is_{50} PSI	(8) s_c PSI		
BB-IWBP-202	R6	16-16.7	20-S-763		1.991	4.675	166.0				17351	1.071	1.19	0.32					Slate
Minor break at 7066psi along exisiting fault that did not affect Poisson's Ratio or Elastic Moduli. Major break was fresh.																			
BB-IWBP-203	R2	6.7-7.2	20-S-764		2.003	4.585	164.8				25992	0.481	4.53	0.33					Slate
Minor break at 19041psi that did not affect Poisson's Ratio or Elastic Moduli. All breaks were fresh.																			

Notes

(1) Volume Determined By Measuring Dimensions

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

(3) PLD=Point Load (diametrical), PLA= Point Load (Axial) ST= Splitting Tensile U= Unconfined Compressive Strength

(4) Taken at Peak Deviator Stress

(5) Strain at Peak Deviator Stress

(6) Represents Secant Modulus at 50% of Total Failure Stress

(7) Represents Secant Poisson's Ratio at 50% of Total Failure Stress

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

Date Received: *03.24.2020*

Reviewed By:

Date Reviewed: *03.30.2020*

THIELSCH ENGINEERING
195 Frances Avenue
Cranston, Rhode Island 02910
Phone: (401) 467-6454
Fax: (401) 467-2398
www.thielsch.com
Let's Build a Solid Foundation

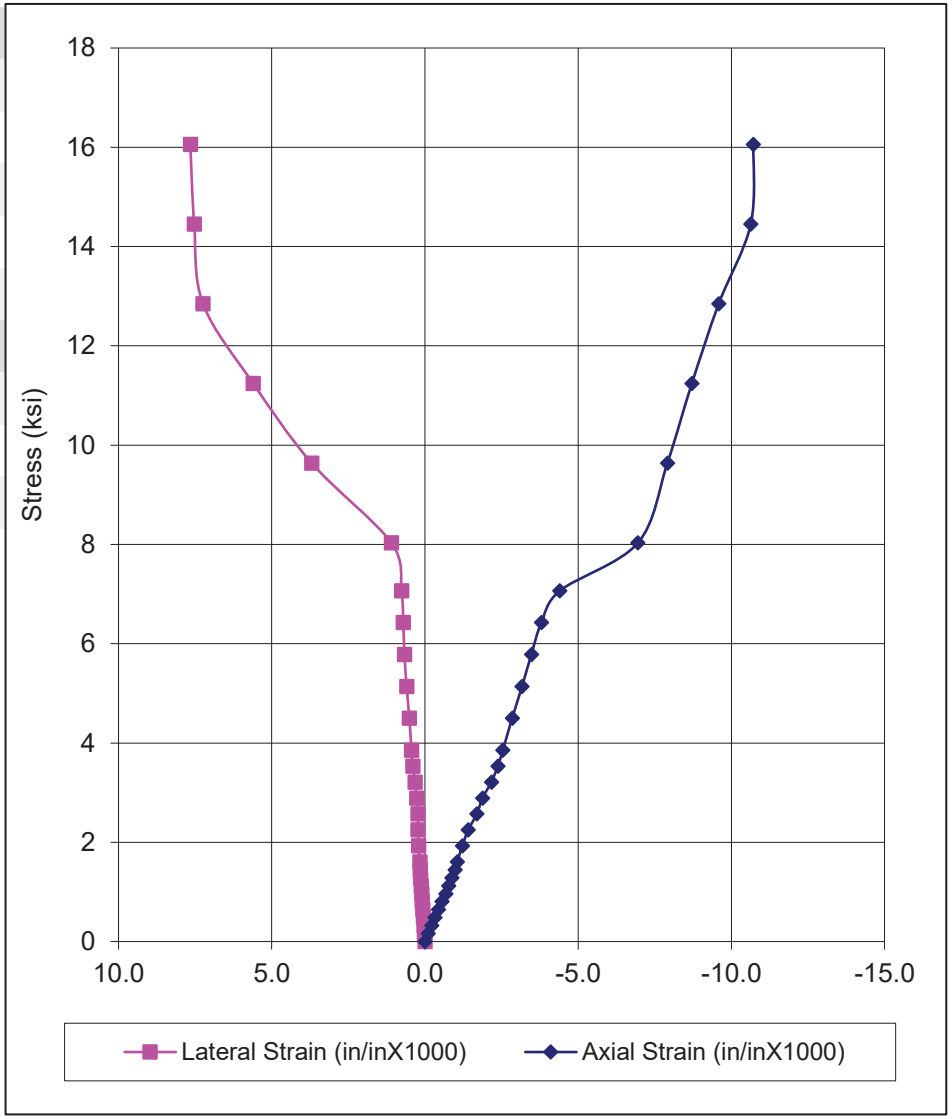
Client Information:
GZA GeoEnvironmental
South Portland, ME
PM: Blaine Cardali
Assigned by: ARB
Collected by: MRJ

Project Information:
West Branch Bridge WIN 23236.00
T3 Indian Purchase Twp, ME
Project Number: 09.0025976.01
Technician: JM
Report Date: 03.25.2020

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

Rock Information			
Boring ID:	BB-IWBP-202	**Diameter, D (in):**	1.991
Sample #:	R6	**Length, L (in):**	4.675
Depth (ft):	16.0-16.7	**L:D Ratio:**	2.35
Tested Depth (ft):	16.2-16.6	**Type:**	Slate
Features:	Existing fault, low fissile		

Compressive Test Information		Elastic Moduli Test Information	
Unit Weight (pcf):	166.0	**Poisson's Ratio @ 50%:**	0.32
Failure Stress (psi):	17,351	**Strain %:**	1.071
Failure Mode:	Fresh	**E sec PSI @ 50%:**	1.19E+06
Time to Failure (min)	3.25		



THIELSCH ENGINEERING

195 Frances Avenue
Cranston, Rhode Island 02910
Phone: (401) 467-6454
Fax: (401) 467-2398
www.thielsch.com
Let's Build a Solid Foundation

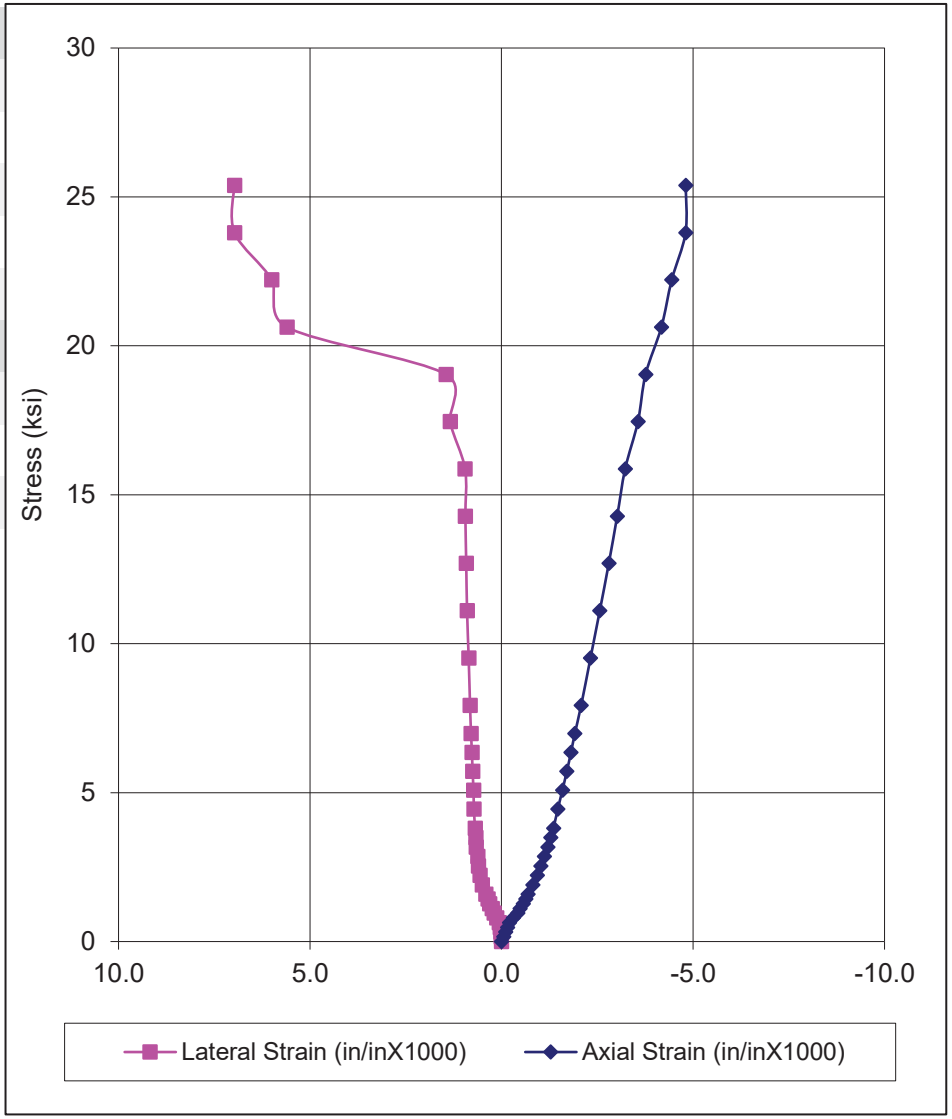
Client Information:
GZA GeoEnvironmental
South Portland, ME
PM: Blaine Cardali
Assigned by: ARB
Collected by: MRJ

Project Information:
West Branch Bridge WIN 23236.00
T3 Indian Purchase Twp, ME
Project Number: 09.0025976.01
Technician: JM
Report Date: 03.26.2020

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

Rock Information			
Boring ID:	BB-IWBP-203	**Diameter, D (in):**	2.003
Sample #:	R2	**Length, L (in):**	4.585
Depth (ft):	6.7-7.2	**L:D Ratio:**	2.29
Tested Depth (ft):	6.7-7.1	**Type:**	Slate
Features:	Existing fault, low fissile		

Compressive Test Information		Elastic Moduli Test Information	
Unit Weight (pcf):	164.8	**Poisson's Ratio @ 50%:**	0.33
Failure Stress (psi):	25,992	**Strain %:**	0.481
Failure Mode:	Fresh	**E sec PSI @ 50%:**	4.53E+06
Time to Failure (min)	5.77		



THIELSCH ENGINEERING
195 Frances Avenue
Cranston RI, 02910
Phone: (401)-467-6454
Fax: (401)-467-2398
thielsch.com
Let's Build a Solid Foundation

Client Information:
GZA GeoEnvironmental
South Portland, ME
PM: Blaine Cardali
Assigned By: Andrew Blaisdell
Collected By: MRJ

Project Information:
West Branch Bridge WIN 23236.00
T3 Indian Purchase Twp, ME
GZA Project Number: 09.0025976.01
Summary Page: 1 of 1
Report Date: 04.08.200

LABORATORY TESTING DATA SHEET, Report No.: 7420-D-112

Boring No.	Sample No.	Depth (ft)	Laboratory No.	Specimen Data						Compressive Strength Tests									Rock Formation or Description or Remarks
				Mohs Hard-ness	Diameter (in)	Length (in)	(1) Unit Weight (PCF)	(2) Wet Density (PCF)	Bulk G_s	(3) Other Tests	(4) Strength PSI	(5) Strain %	(6) E sec PSI EE+06	(7) Poisson's Ratio	$\sigma\tau$ PSI	Is_{50} PSI	(8) s_c PSI		
BB-IWBP-201	R1	28.4-29.1	20-S-883		1.993	4.542	169.3				28556	0.735	2.92	0.04					Slate
Minor break at 1604 and 22450 psi. All breaks were fresh.																			
BB-IWBP-204	R2	30.8-31.2	20-S-884D		1.977	0.937	162.9			PLD						978	23472		Slate
Break was fresh.																			

(1) Volume Determined By Measuring Dimensions

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

Notes

(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: *04.03.2020*

Reviewed By:

Date Reviewed: *04.09.2020*

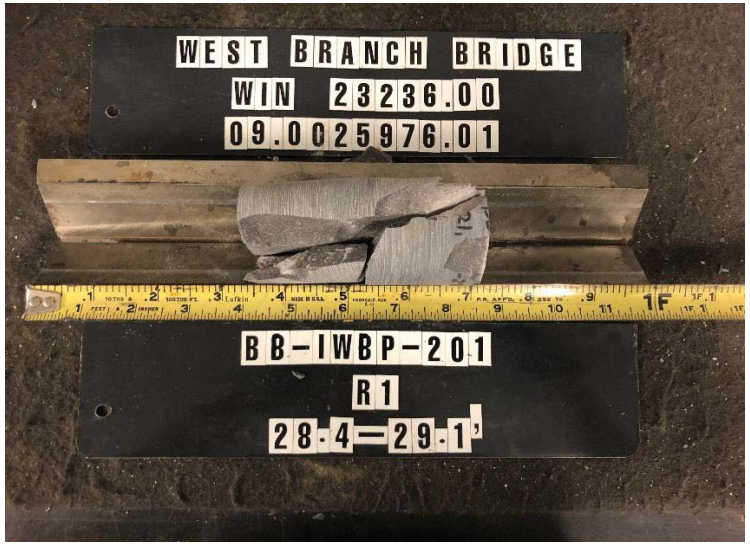
THIELSCH ENGINEERING
195 Frances Avenue
Cranston, Rhode Island 02910
Phone: (401) 467-6454
Fax: (401) 467-2398
www.thielsch.com
Let's Build a Solid Foundation

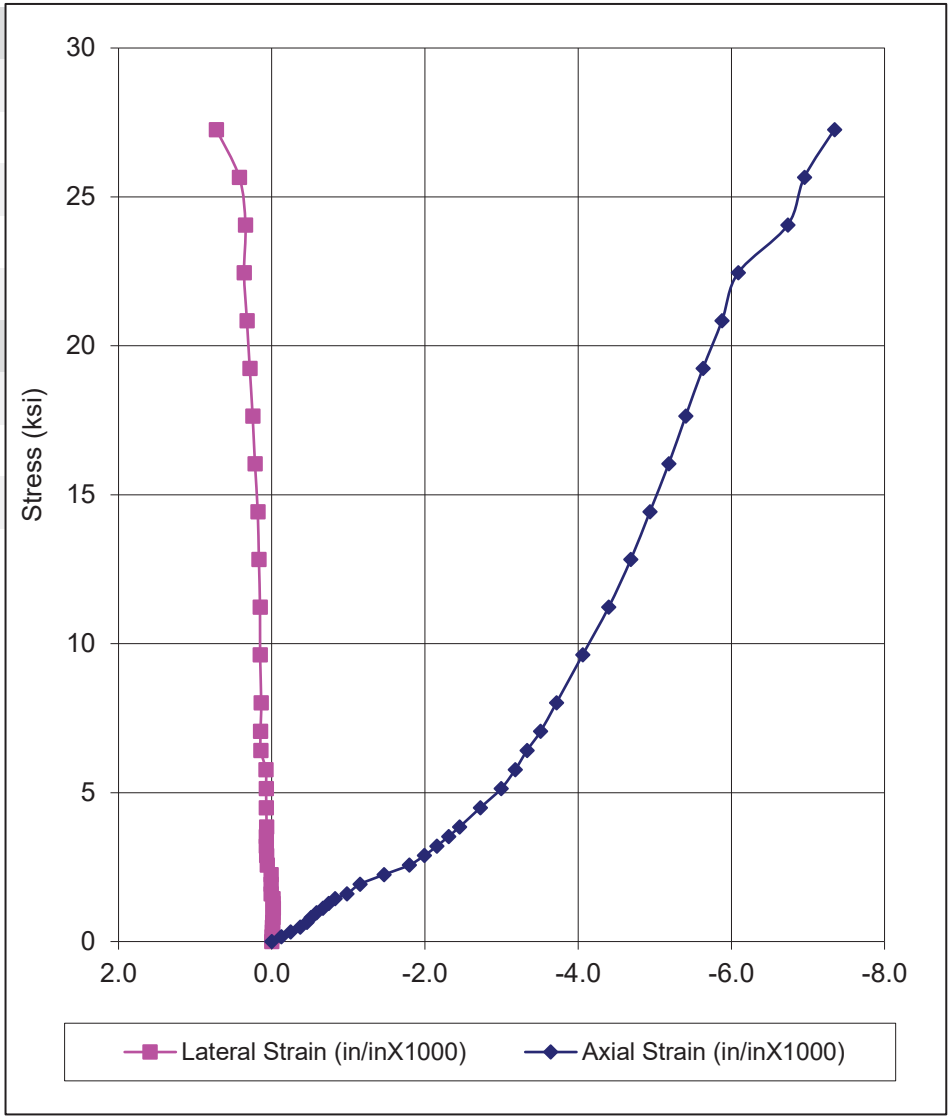
Client Information:
GZA GeoEnvironmental
Portland, ME
PM: Blaine Cardali
Assigned by: Andrew Blaidell
Collected by: MRJ

Project Information:
West Branch Bridge WIN 23236.00
T3 Indian Purchase Twp, ME
Project Number: 09.0025976.01
Technician: JM
Report Date: 04.06.2020

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

Rock Information			
Boring ID:	BB-IWBP-201	**Diameter, D (in):**	1.993
Sample #:	R1	**Length, L (in):**	4.542
Depth (ft):	28.4-29.1	**L:D Ratio:**	2.28
Tested Depth (ft):	28.5-28.9	**Type:**	Slate
Features:	Non weathered		
Compressive Test Information		**Elastic Moduli Test Information**	
Unit Weight (pcf):	169.3	**Poisson's Ratio @ 50%:**	0.04
Failure Stress (psi):	28,556	**Strain %:**	0.735
Failure Mode:	Fresh	**E sec PSI @ 50%:**	2.92E+06
Time to Failure (min)	6.083		





APPENDIX D – BEDROCK CORE PHOTOGRAPHS

MaineDOT Detective Benjamin Campbell Bridge #3666
Carries US Route 11 over West Branch Penobscot River
T3 Indian Purchase Township, ME

Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-IWBP-105	R1	22.0 - 22.8	1	11	0	0	SCHIST	1
BB-IWBP-105	R2	23.0 - 24.6	13	68	6	31	SCHIST	1
BB-IWBP-105	R3	29.8 - 30.4	7	100	0	0	SCHIST	1
BB-IWBP-105	R4	30.4 - 32.7	11	39	0	0	SCHIST	1
BB-IWBP-105	R5	38.0 - 41.4	40	98	20	49	SCHIST	1,2
BB-IWBP-105	R6	41.4 - 43.9	30	100	0	0	SCHIST	2
BB-IWBP-105	R7	43.9 - 47.2	39	100	9	23	SCHIST	4
BB-IWBP-101	R1	26.9 - 32.0	61	100	35	57	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.
3. Transition between core runs within a row are marked by wood separators.

MaineDOT Detective Benjamin Campbell Bridge #3666
Carries US Route 11 over West Branch Penobscot River
T3 Indian Purchase Township, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-IWBP-101	R2	32.0 - 34.0	20	83	12	50	SCHIST	1
BB-IWBP-101	R3	34.0 - 37.0	36	100	19	53	SCHIST	1
BB-IWBP-104	R1	23.0 - 25.5	30	100	0	0	SCHIST	2
BB-IWBP-104	R2	25.5 - 28.0	30	100	0	0	SCHIST	2
BB-IWBP-104	R3	28.0 - 30.0	18	75	0	0	SCHIST	3
BB-IWBP-104	R4	30.0 - 34.0	42	88	10	20	SCHIST	3
BB-IWBP-104	R5	34.0 - 36.7	32	100	0	0	SCHIST	4
BB-IWBP-104	R6 (Top 23”)	36.7 - 40.4	44	100	13	30	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.
3. Transition between core runs within a row are marked by wood separators.

MaineDOT Detective Benjamin Campbell Bridge #3666
Carries US Route 11 over West Branch Penobscot River
T3 Indian Purchase Township, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-IWBP-104	R6 (Bot 21")	36.7 - 40.4	44	100	13	30	SCHIST	1
BB-IWBP-102	R1	13.5 - 16.7	38	100	9	24	SCHIST	2
BB-IWBP-102	R2	16.7 - 19.3	31	100	14	45	SCHIST	2,3
BB-IWBP-102	R3	19.3 - 22.3	32	89	4	11	SCHIST	3
BB-IWBP-102	R4	22.3 - 24.5	26	100	11	42	SCHIST	3,4

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

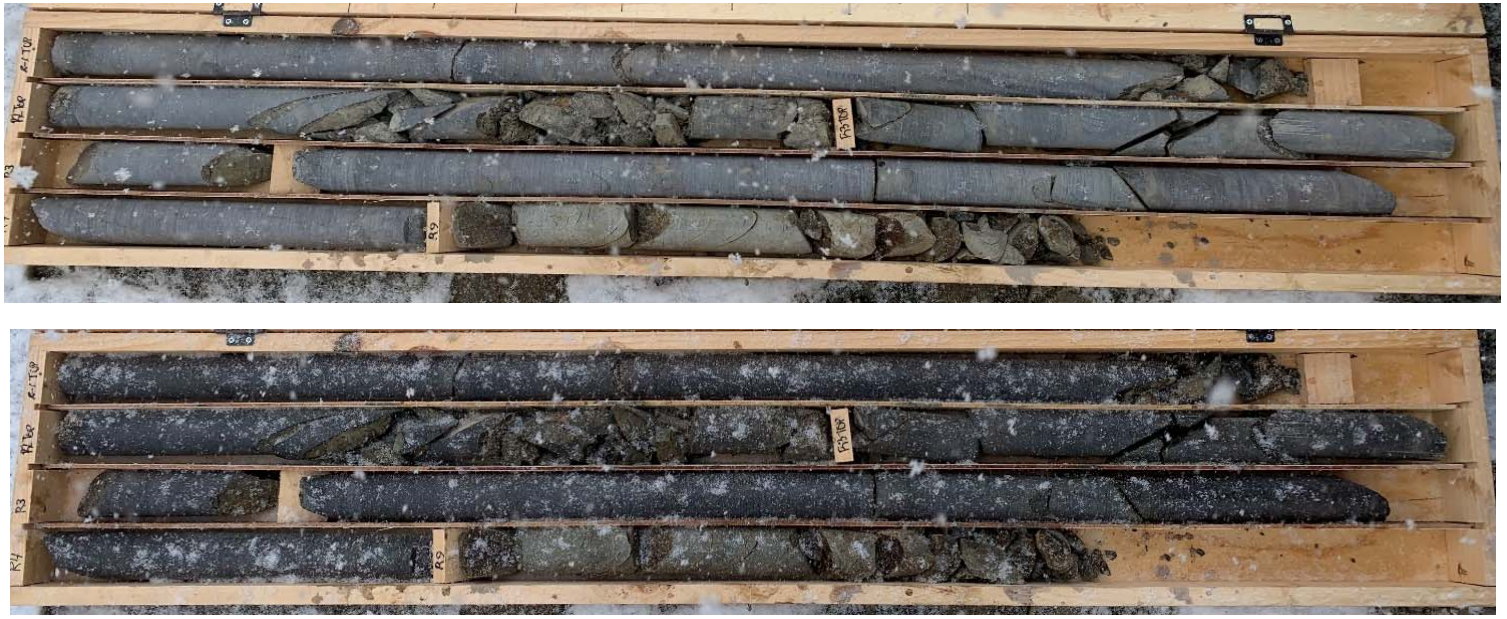
MaineDOT Detective Benjamin Campbell Bridge #3666
Carries US Route 11 over West Branch Penobscot River
T3 Indian Purchase Township, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-IWBP-103	R1	1.3 - 3.9	26	84	0	0	SCHIST	1
BB-IWBP-103	R2	3.9 - 5.7	21	100	4	19	SCHIST	1
BB-IWBP-103	R3	5.7 - 7.3	20	100	0	0	SCHIST	2
BB-IWBP-103	R4	7.3 - 9.7	25	86	9	31	SCHIST	2,3
BB-IWBP-103	R5	9.7 - 12.2	30	100	5	17	SCHIST	3
BB-IWBP-103	R6	12.2 - 15.2	33	92	0	0	SCHIST	3,4
BB-IWBP-103	R7	15.2 - 17.5	22	79	0	0	SCHIST	4
BB-IWBP-103	R8	17.5 - 17.8	2	50	0	0	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.
3. Transition between core runs within a row are marked by wood separators.

MaineDOT Detective Benjamin Campbell Bridge #3666
Carries US Route 11 over West Branch Penobscot River
T3 Indian Purchase Township, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-IWBP-201	R1	27.0 - 31.4	52	98	44	85	SCHIST	1
BB-IWBP-201	R2	31.4 - 34.2	33	97	8	24	SCHIST	2
BB-IWBP-201	R3	34.2 - 36.9	30	94	10	32	SCHIST	2,3
BB-IWBP-201	R4	36.9 - 41.9	60	100	47	78	SCHIST	3,4
BB-IWBP-202	R9	22.2 - 24.2	22	92	5	23	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.
3. Transition between core runs within a row are marked by wood separators.

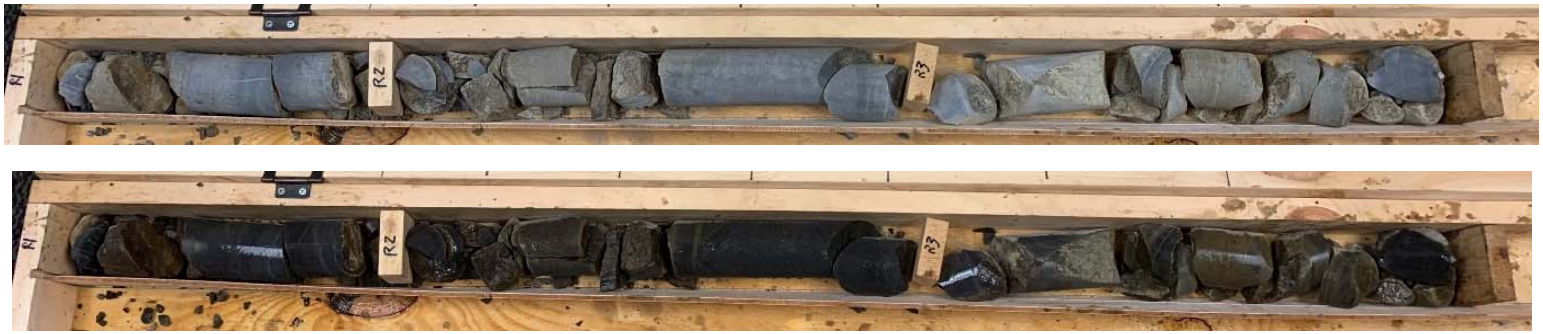
MaineDOT Detective Benjamin Campbell Bridge #3666
Carries US Route 11 over West Branch Penobscot River
T3 Indian Purchase Township, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-IWBP-202	R1	3.1 - 4.8	17	85	0	0	SCHIST	1
BB-IWBP-202	R2	4.8 - 8.8	43	90	17	39	SCHIST	1,2
BB-IWBP-202	R3	8.8 - 10.0	12	86	0	0	SCHIST	2
BB-IWBP-202	R4	10.0 - 12.0	22	92	4	18	SCHIST	2
BB-IWBP-202	R5	12.0 - 15.5	42	100	9	21	SCHIST	2,3
BB-IWBP-202	R6	15.5 17.8	21	75	8	36	SCHIST	3
BB-IWBP-202	R7	17.8 20.8	32	89	0	0	SCHIST	4
BB-IWBP-202	R8	20.8 22.2	16	94	4	25	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.
3. Transition between core runs within a row are marked by wood separators.

MaineDOT Detective Benjamin Campbell Bridge #3666
Carries US Route 11 over West Branch Penobscot River
T3 Indian Purchase Township, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-IWBP-203	R1	4.7 - 5.9	11	79	0	0	SCHIST	1
BB-IWBP-203	R2	5.9 - 7.7	16	73	6	34	SCHIST	1
BB-IWBP-203	R3	7.7 - 9.7	16	67	0	0	SCHIST	1



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

MaineDOT Detective Benjamin Campbell Bridge #3666
Carries US Route 11 over West Branch Penobscot River
T3 Indian Purchase Township, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-IWBP-204	R1	27.6 - 30.5	10	29	0	0	SCHIST	1
BB-IWBP-204	R2	30.5 - 31.7	11	79	0	0	SCHIST	1
BB-IWBP-204	R3	31.7 - 34.0	15	54	0	0	SCHIST	1
BB-IWBP-204	R4	34.7 - 36.6	14	63	0	0	SCHIST	1
BB-IWBP-204	R5	36.6 - 38.0	13	76	0	0	SCHIST	2
BB-IWBP-204	R6	38.0 - 40.2	22	85	0	0	SCHIST	2
BB-IWBP-204	R7	40.2 - 41.6	14	82	0	0	SCHIST	2
BB-IWBP-204	R8	41.6 - 44.2	28	90	0	0	SCHIST	3
BB-IWBP-204	R9	44.2 - 46.2	24	100	0	0	SCHIST	3
BB-IWBP-204	R10	46.2 - 47.4	9	64	0	0	SCHIST	4
BB-IWBP-204	R11	47.4 - 50.5	37	100	13	34	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.
3. Transition between core runs within a row are marked by wood separators.

APPENDIX E – CALCULATIONS

Frost Calculation

March 2014

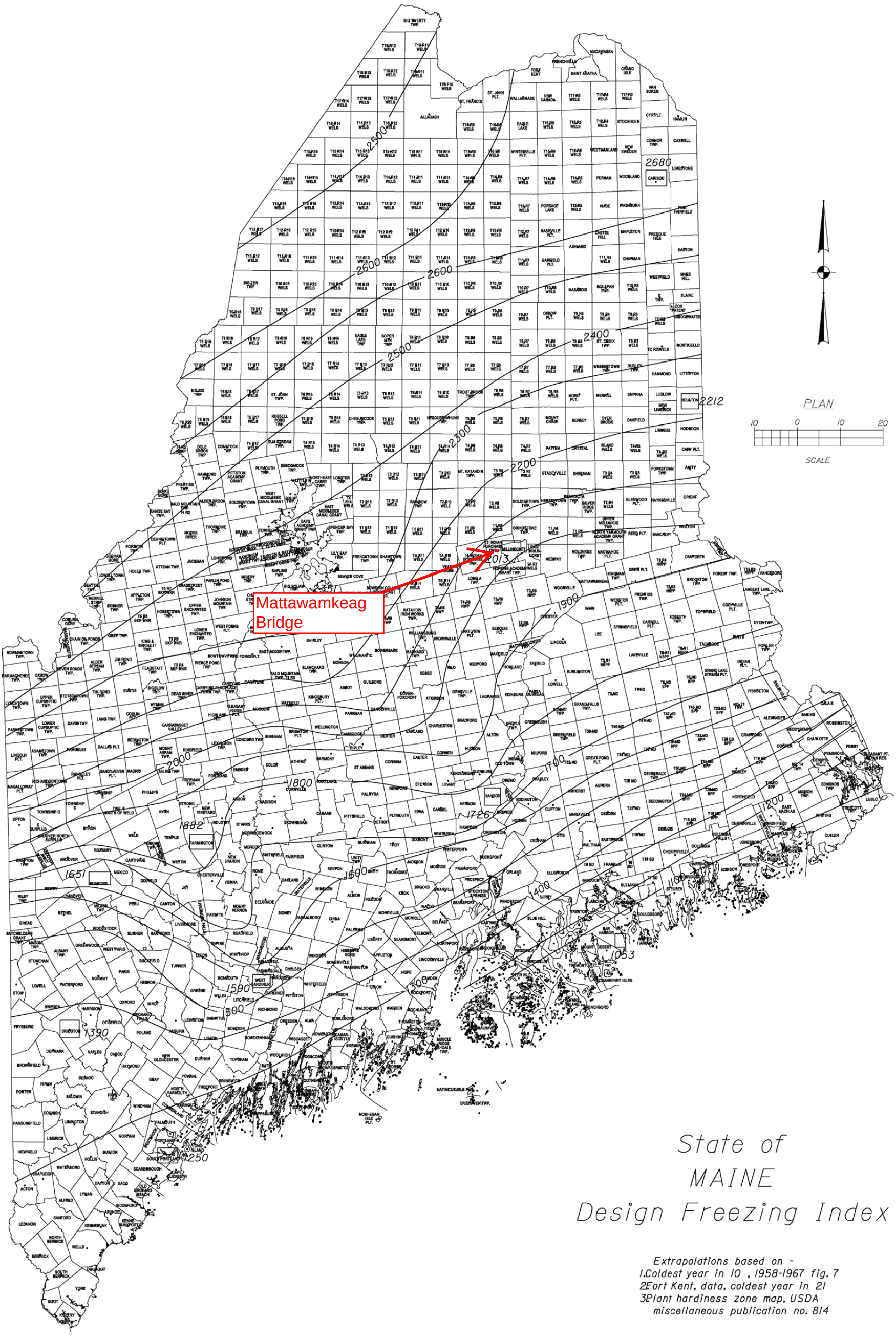


Figure 5-1 Maine Design Freezing Index Map

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	56.4	56.3	48.5	43.9
1500	82.1	67.9	58.4	58.3	50.2	45.4
1600	84.8	70.2	60.3	60.2	51.9	46.9
1700	87.5	72.4	62.2	62.2	53.5	48.4
1800	90.1	74.5	64.0	64.0	55.1	49.8
1900	92.6	76.6	65.7	65.8	56.7	51.1
2000	95.1	78.7	67.5	67.6	58.2	52.5
2100	97.6	80.7	69.2	69.3	59.7	53.8
2200	100.0	82.6	70.8	71.0	61.1	55.1
2300	102.3	84.5	72.4	72.7	62.5	56.4
	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

2055

96" = 8'

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.

Moraine soils and fractured bedrock are anticipated to be present at the abutments near the elevation of the footings. The material is coarse-grained with water contents less than 10%. Based on the MaineDOT BDG, Section 5.2.1 and a Freezing Index of 2,055 the estimated depth of frost penetration is 96 inches.

Seismic Calculation

Seismic Site Class Calculation Summary

Project: Detective Benjamin Campbell Bridge Bridge **Project No.:** 09.0025976.01
Location: T3 Indian Purchase TWP, ME
Evaluated By/Date: E. Tome **Date** 6/16/2020
Checked By/Date: B.Cardali **Date** 6/18/2020

Objective:
Determine seismic site class by performing calculations in accordance with the MassDOT LRFD Bridge Manual 2013 Edition, which references the AASHTO LRFD Seismic Bridge Design Specifications, 2nd Edition (2011) and Interim.

Subsurface Data: Borings BB-IWBP-101 through -105 were drilled by NEBC of Hermon, Maine between May 29 and June 7, 2018. Borings BB-IWBP-201 through -204 were drilled my NEBC of Hermon, Maine between February 25 and March 5, 2020.

Assumptions: Soil borings extended to depths between 17 and 51 feet below the roadway level and bedrock was encountered encountered in the soil borings.

Approach:
1) Evaluate if the procedure in AASHTO LRFD Seismic Section 3.4.2.1 for classifying a site is appropriate for the site. Sites with highly variable subsurface conditions or very large sites may require multiple site class determinations or a site-specific seismic response analysis. Furthermore, classifying a site based on the 100 feet of soil and rock beneath the ground surface may be inappropriate if deep deposits of weak soils are present below 100 feet, or if foundation structures are supported on firm soil or rock below soft soils which can be justified as having little effect on the structure's seismic response.

2) Evaluate if soil properties are known in sufficient detail to determine site class. If data is not known in sufficient detail, AASHTO permits the use of Site Class D, unless conditions for Site Class E or Site Class F are likely to be present.

3) Check for the four categories of Site Class F requiring site-specific evaluation:
- Soils vulnerable to potential failure (liquefiable soils, sensitive clays, weakly cemented soils)
- Peats or highly organic clays greater than 10 feet in thickness
- Thick layers (greater than 25 feet) of highly plastic clay (PI > 75)
- Very thick soft/medium stiff clays (greater than 125 feet)

4) Check for existence of greater than 10 feet of soft clay (where s_u < 500 psf, w > 40%, and PI > 20). If these conditions are met, classify as Site Class E.

5) Categorize the site using one of the following three methods in AASHTO 3.4.2.2:
- $\overline{v}_s$ (Method A) - $\overline{N}$ (Method B) - $\overline{N}_{ch}$ and $\overline{s}_u$ (Method C)

If shear wave velocity data are available, they should be used to classify the site. The N and s_u methods should only be used if shear wave velocity data is not available, as the correlation between site amplification and these geotechnical parameters is more uncertain (and therefore more conservative) than the correlation with v_s.

Results: Calculations of the Seismic Site Class based on Method B as described in section 3.4.2.2 of the LRFD Seismic Bridge Design Specifications are attached. Calculations results are summarized in the table below.

Boring ID	BB-IWBP-101	BB-IWBP-102	BB-IWBP-104	BB-IWBP-105	BB-IWBP-201	BB-IWBP-204	Average
N-Value	26	56	62	41	92.1	52.7	55.0

Conclusions: Based on the procedure outlined in section 3.4.2.2 and table 3.4.2.1-1 of the LRFD Seismic Bridge Design Specifications, we recommend that Site Class "C" be used for preliminary design.

Detective Benjamin Campbell Bridge Bridge
T3 Indian Purchase TWP, ME

Calculated By: E. Tome Date: 6/16/2020
Checked By: B.Cardali Date: 6/18/2020

INPUT

Exploration ID: BB-IWBP-101 Ground Surface Elevation: 480.1 ft Depth of Boring: 37.0 ft

Depth to Bedrock: 25.8 ft

EQUATIONS

$$\overline{N}_{ch} = \frac{\sum_{i=1}^{m} d_i}{\sum_{i=1}^{m} \frac{d_i}{N_i}}$$

where: m = number of layers
d_i = the thickness of all layers between 0 and 100 feet.
d_c = the thickness of any clay layers between 0 and 100 feet.
N_i = the Standard Penetration Resistance (ASTM D 1586) of cohesionless soil layers not to exceed 100 blows/ft, corrected for hammer energy for calibrated auto hammers (i.e., N_{60}).
Note: d_i calculated assuming breaks between sub-layers occur at the midpoint between SPT sample intervals (unless noted otherwise)

CALCULATION

$\overline{N}$ = 26.5

Soil Strata	SPT Interval Depth Top, ft	SPT Interval Depth Bottom, ft	SPT Elevation (mid-interval)	SPT N-value	d_i	d_i / N_i	Comment
Fill	1.0	3.0	478.1	32	4.3	0.13	
	5.5	7.5	473.6	24	4.5	0.19	
	10.0	12.0	469.1	5	4.8	0.95	
	15.0	17.0	464.1	5	5.0	1.00	
Moraine	20.0	22.0	459.1	54	81.5	1.51	
Top of Rock	25.8						

Detective Benjamin Campbell Bridge Bridge
T3 Indian Purchase TWP, ME

Calculated By: E. Tome Date: 6/16/2020
Checked By: B.Cardali Date: 6/18/2020

INPUT

Exploration ID: BB-IWBP-102 Ground Surface Elevation: 461.0 ft Depth of Boring: 24.5 ft

Depth to Bedrock: 12.5 ft

EQUATIONS

$$\overline{N}_{ch} = \frac{\sum_{i=1}^{m} d_i}{\sum_{i=1}^{m} \frac{d_i}{N_i}}$$

where: m = number of layers
d_i = the thickness of all layers between 0 and 100 feet.
d_c = the thickness of any clay layers between 0 and 100 feet.
N_i = the Standard Penetration Resistance (ASTM D 1586) of cohesionless soil layers not to exceed 100 blows/ft, corrected for hammer energy for calibrated auto hammers (i.e., N_{60}).
Note: d_i calculated assuming breaks between sub-layers occur at the midpoint between SPT sample intervals (unless noted otherwise)

CALCULATION

$\overline{N}$ = 55.9

Soil Strata	SPT Interval Depth Top, ft	SPT Interval Depth Bottom, ft	SPT Elevation (mid-interval)	SPT N-value	d_i	d_i / N_i	Comment
moraine	0.0	2.0	460.0	16	3.5	0.22	
	5.0	7.0	455.0	42	5.0	0.12	
	10.0	12.0	450.0	63	91.5	1.45	
Top of Rock							

Detective Benjamin Campbell Bridge Bridge
T3 Indian Purchase TWP, ME

Calculated By: E. Tome Date: 6/16/2020
Checked By: B.Cardali Date: 6/18/2020

INPUT

Exploration ID: **BB-IWBP-104** Ground Surface Elevation: 474.3 ft Depth of Boring: 40.4 ft

Depth to Bedrock: 32.3 ft

EQUATIONS

$$\overline{N}_{ch} = \frac{\sum_{i=1}^{m} d_i}{\sum_{i=1}^{m} \frac{d_i}{N_i}}$$

where: m = number of layers
d_i = the thickness of all layers between 0 and 100 feet.
d_c = the thickness of any clay layers between 0 and 100 feet.
N_i = the Standard Penetration Resistance (ASTM D 1586) of cohesionless soil layers not to exceed 100 blows/ft, corrected for hammer energy for calibrated auto hammers (i.e., N_{60}).
Note: d_i calculated assuming breaks between sub-layers occur at the midpoint between SPT sample intervals (unless noted otherwise)

CALCULATION

$\overline{N}$ = 61.6

Soil Strata	SPT Interval Depth Top, ft	Bottom, ft	SPT Elevation (mid-interval)	SPT N-value	d_i	d_i / N_i	Comment
Fill	1.0	3.0	472.3	13	4.0	0.31	
	5.0	7.0	468.3	12	4.5	0.38	
	10.0	12.0	463.3	65	5.0	0.08	
Alluvium	15.0	17.0	458.3	100	5.0	0.05	
	20.0	22.0	453.3	100	81.5	0.82	
Top of Rock	32.3						

Detective Benjamin Campbell Bridge Bridge
T3 Indian Purchase TWP, ME

Calculated By: E. Tome Date: 6/16/2020
Checked By: B.Cardali Date: 6/18/2020

INPUT

Exploration ID: BB-IWBP-105 Ground Surface Elevation: 474.2 ft Depth of Boring: 47.2 ft

Depth to Bedrock: 21.6 ft

EQUATIONS

$$\overline{N}_{ch} = \frac{\sum_{i=1}^{m} d_i}{\sum_{i=1}^{m} \frac{d_i}{N_i}}$$

where: m = number of layers
d_i = the thickness of all layers between 0 and 100 feet.
d_c = the thickness of any clay layers between 0 and 100 feet.
N_i = the Standard Penetration Resistance (ASTM D 1586) of cohesionless soil layers not to exceed 100 blows/ft, corrected for hammer energy for calibrated auto hammers (i.e., N_{60}).
Note: d_i calculated assuming breaks between sub-layers occur at the midpoint between SPT sample intervals (unless noted otherwise)

CALCULATION

$\overline{N}$ = 41.4

Soil Strata	SPT Interval Depth Top, ft	SPT Interval Depth Bottom, ft	SPT Elevation (mid-interval)	SPT N-value	d_i	d_i / N_i	Comment
Fill	2.0	4.0	471.2	22	4.5	0.20	
	5.0	7.0	468.2	25	4.0	0.16	
	10.0	12.0	463.2	16	5.0	0.31	
Moraine	15.0	17.0	458.2	46	5.0	0.11	
	20.0	22.0	453.2	50	81.5	1.63	
Top of Rock	21.6						

Detective Benjamin Campbell Bridge Bridge
T3 Indian Purchase TWP, ME

Calculated By: E. Tome Date: 6/16/2020
Checked By: B.Cardali Date: 6/18/2020

INPUT

Exploration ID: BB-IWBP-201 Ground Surface Elevation: 480.0 ft Depth of Boring: 41.9 ft

Depth to Bedrock: 26.7 ft

EQUATIONS

$$\overline{N}_{ch} = \frac{\sum_{i=1}^{m} d_i}{\sum_{i=1}^{m} \frac{d_i}{N_i}}$$

where: m = number of layers
d_i = the thickness of all layers between 0 and 100 feet.
d_c = the thickness of any clay layers between 0 and 100 feet.
N_i = the Standard Penetration Resistance (ASTM D 1586) of cohesionless soil layers not to exceed 100 blows/ft, corrected for hammer energy for calibrated auto hammers (i.e., N_{60}).
Note: d_i calculated assuming breaks between sub-layers occur at the midpoint between SPT sample intervals (unless noted otherwise)

CALCULATION

$\overline{N}$ = 92.1

Soil Strata	SPT Interval Depth Top, ft	Bottom, ft	SPT Elevation (mid-interval)	SPT N-value	d_i	d_i / N_i	Comment
Fill	2.0	4.0	477.0	113	4.5	0.04	
	5.0	7.0	474.0	26	4.0	0.15	
	10.0	12.0	469.0	36	3.5	0.10	
	12.0	14.0	467.0	11	2.0	0.18	
Moraine	14.0	16.0	465.0	36	2.0	0.06	
	16.0	18.0	463.0	157	2.0	0.01	
	18.0	20.0	461.0	90	2.0	0.02	
	20.0	22.0	459.0	45	3.5	0.08	
	25.0	27.0	454.0	172	76.5	0.44	
Top of Rock	26.7						

Detective Benjamin Campbell Bridge Bridge
T3 Indian Purchase TWP, ME

Calculated By: E. Tome Date: 6/16/2020
Checked By: B.Cardali Date: 6/18/2020

INPUT

Exploration ID: BB-IWBP-204 Ground Surface Elevation: 474.3 ft Depth of Boring: 50.5 ft

Depth to Bedrock: 27.6 ft

EQUATIONS

$$\overline{N}_{ch} = \frac{\sum_{i=1}^{m} d_i}{\sum_{i=1}^{m} \frac{d_i}{N_i}}$$

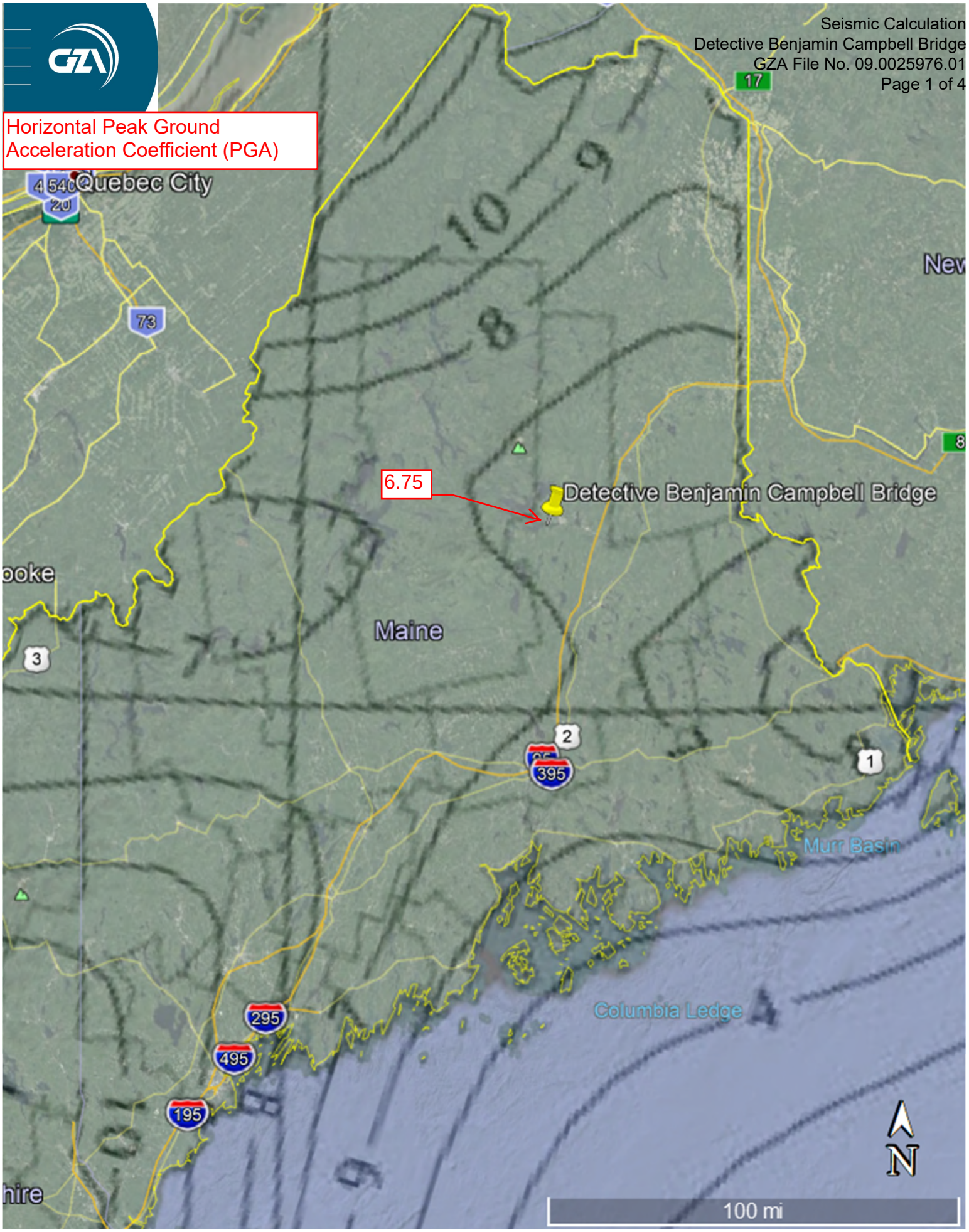
where: m = number of layers
d_i = the thickness of all layers between 0 and 100 feet.
d_c = the thickness of any clay layers between 0 and 100 feet.
N_i = the Standard Penetration Resistance (ASTM D 1586) of cohesionless soil layers not to exceed 100 blows/ft, corrected for hammer energy for calibrated auto hammers (i.e., N_{60}).
Note: d_i calculated assuming breaks between sub-layers occur at the midpoint between SPT sample intervals (unless noted otherwise)

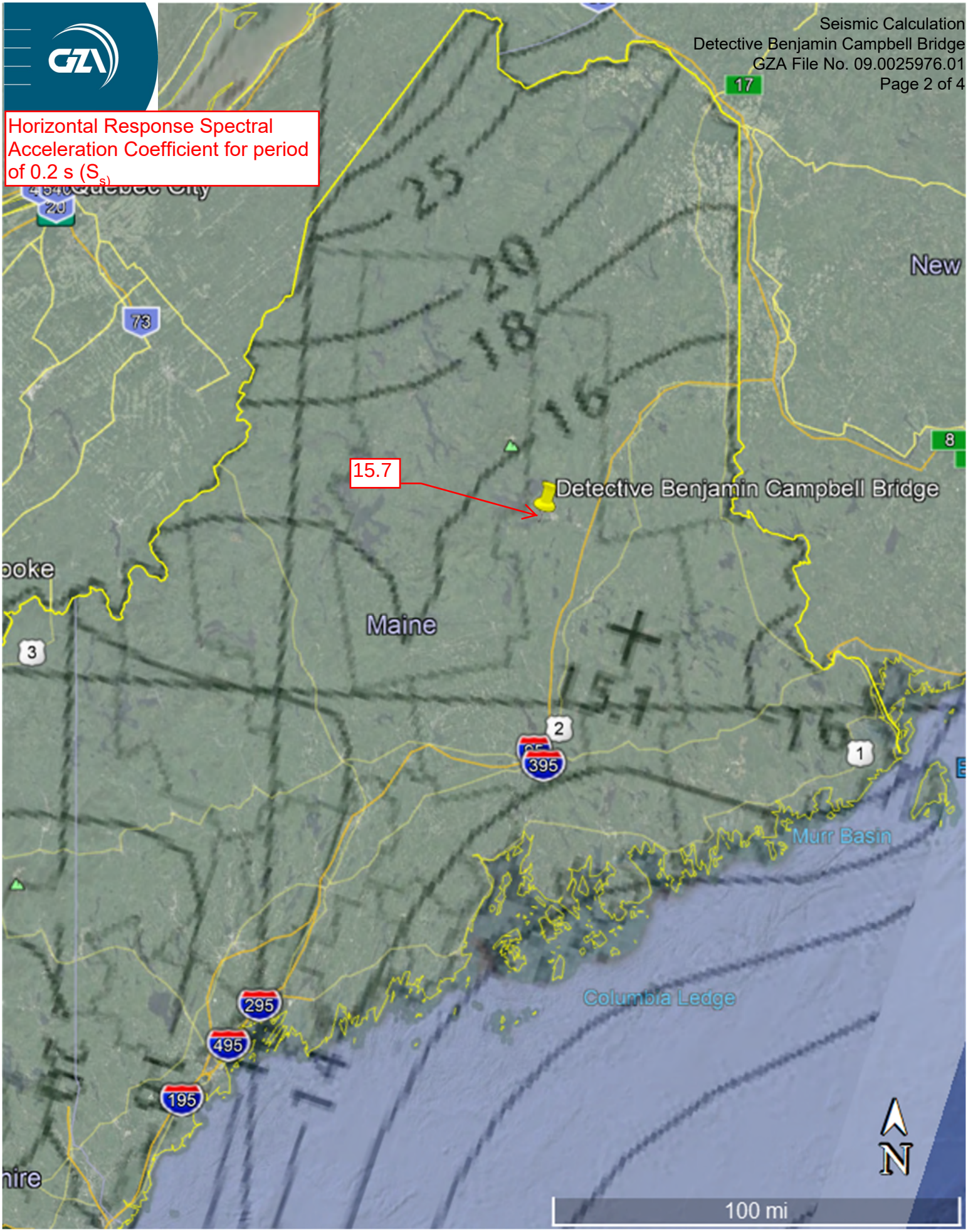
CALCULATION

$\overline{N}$ = 52.7

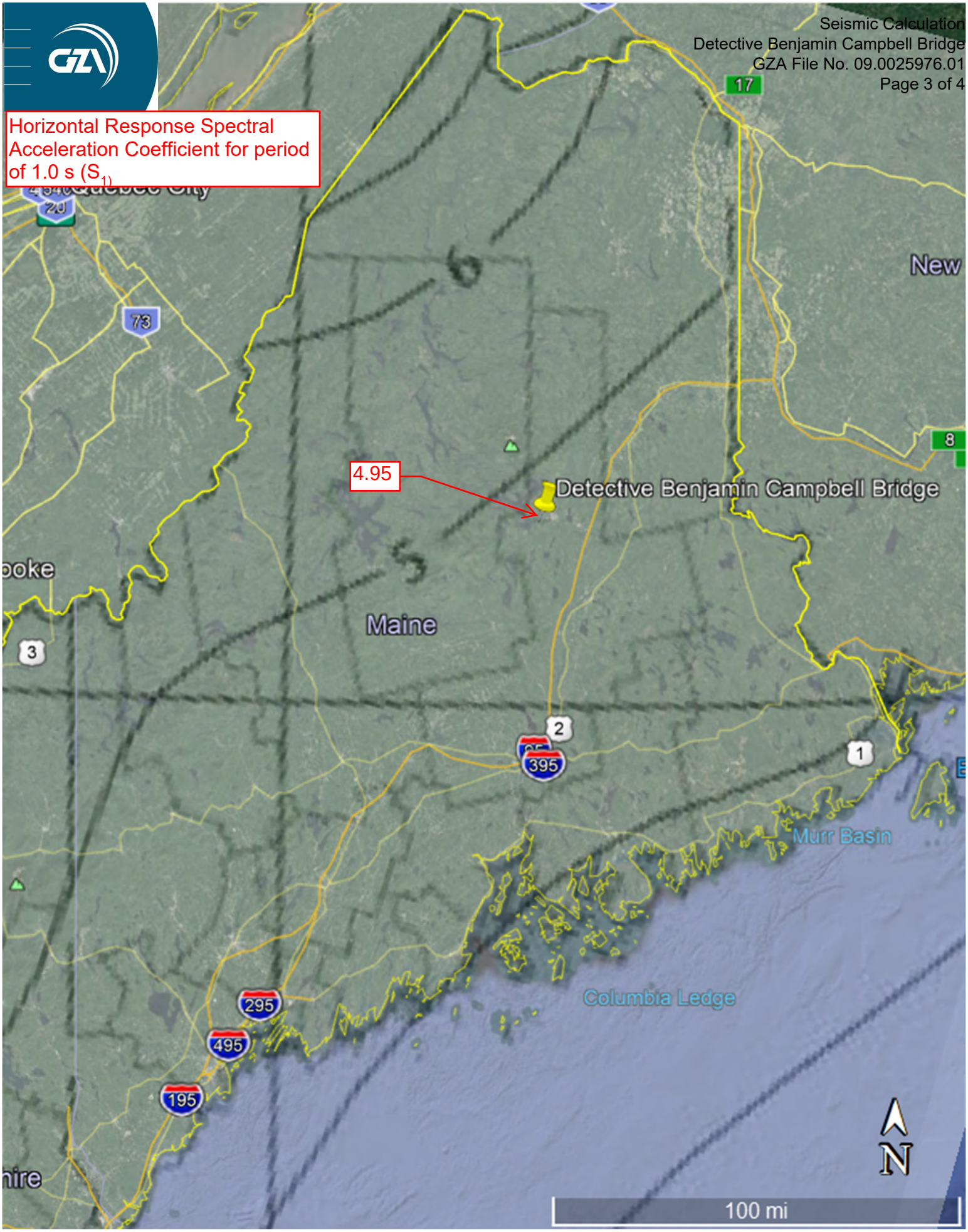
Soil Strata	SPT Interval Depth Top, ft	SPT Interval Depth Bottom, ft	SPT Elevation (mid-interval)	SPT N-value	d_i	d_i / N_i	Comment
Fill	4.0	4.8	469.9	50	6.9	0.14	
	9.0	11.0	464.3	80	4.1	0.05	
Moraine	11.0	13.0	462.3	33	2.0	0.06	
	13.0	14.3	460.7	50	1.7	0.03	
	15.0	16.5	458.6	51	2.1	0.04	
	17.0	19.0	456.3	63	2.3	0.04	
	19.0	21.0	454.3	83	2.0	0.02	
	21.0	23.0	452.3	86	2.0	0.02	
	23.0	25.0	450.3	167	2.0	0.01	
	25.0	27.0	448.3	116	2.0	0.02	
	27.0	27.6	447.0	50	73.0	1.46	
Top of Rock	35.0						

Seismic Design Parameters


Horizontal Peak Ground Acceleration Coefficient (PGA)
Quebec City
10
9
8
7
6.75
Detective Benjamin Campbell Bridge
Maine
Murr Basin
Columbia Ledge
N
100 mi


Horizontal Response Spectral Acceleration Coefficient for period of 0.2 s (S_s)
15.7
Detective Benjamin Campbell Bridge
Maine
New
Murr Basin
Columbia Ledge
25
20
18
16
15.1
17
8
73
3
2
1
295
495
195
395
N
100 mi

Horizontal Response Spectral Acceleration Coefficient for period of 1.0 s ($S_{1)}$



Benjamin Campbell Bridge Seismic Interpolation for Coefficients		
Seismic Parameter	**Interpolated Value from Maps[1]**	**Design Parameter**
Horizontal Peak ground Acceleration Coefficient	6.75	$PGA = .0675$
Horizontal Response Spectral Acceleration Coefficient for Period of 0.2s	15.70	$S_s = 0.157$
Horizontal Response Spectral Acceleration Coefficient for Period of 1.0s	4.95	$S_1 = .0495$

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 C, values of F_{PGA} and F_a = 1.2, and F_v = 1.7

Therefore:

$$A_s = F_{PGA} \times PGA = 1.2 \times 0.0675 = 0.081\ g$$

$$S_{DS} = F_a \times S_s = 1.2 \times 0.157 = 0.188\ g$$

$$S_{D1} = F_v \times S_1 = 1.7 \times 0.0495 = 0.084\ g$$

Summary:

SITE CLASS C SEISMIC DESIGN PARAMETERS	
Parameter	**Design Value**
Fpga	1.2
Fa	1.2
Fv	1.7
As (Period = 0.0 sec)	0.081 g
SDs (Period = 0.2 sec)	0.188 g
SD1 (Period = 1.0 sec)	0.084 g

Bedrock Bearing Resistance Calculation

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

Engineers and Scientists

JOB: 09.0025976.01 Dectective Benjamin Campbell Bridge
SUBJECT: Footing Bearing Resistance on Rock
SHEET: 1 OF 10
CALCULATED BY: BMC 06/16/20
REVIEWED BY: ARB 06/17/20

Objective

Evaluate nominal and factored preliminary bearing resistance of a spread footing foundation on rock for the proposed bridge alignment using Unconfined Compressive Strength (UCS) and Rock Quality Designation (RQD) data.

Methodology

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

1. Use Bedrock Properties From Test Borings BB-IWBP-101 through BB-IWBP-105 and BB-IWBP-201 through -204.
2. Calculate Rock Mass Rating (RMR)
3. Determine Rock Property Constants s and m
4. Calculate Nominal and Factored Bearing Resistance of Bedrock (q_n and q_r)

References

1. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 6th Edition 2012. (AASHTO LRFD)
2. Wyllie, Duncan C., "Foundations on Rock", Second edition, 1992.

Evaluation

The boring data at Abutment 2, and to a lesser extent the pier, show the presence of fractured rock before encountering higher quality rock. However, the strength appears similar in the fractured rock. Calculate bearing based on joint characteristics in fractured rock

1. Rock Mass Properties

A. Sources of Data

Bedrock properties were obtained from rock core specimens and logs completed for the subject project, and strength data was obtained from laboratory rock testing for the subject project.

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

Engineers and Scientists

JOB: 09.0025976.01 Dectective Benjamin Campbell Bridge
SUBJECT: Footing Bearing Resistance on Rock
SHEET: 2 OF 10
CALCULATED BY: BMC 06/16/20
REVIEWED BY: ARB 06/17/20

B. Unconfined Compressive Strength

Boring	Run	GS Elevation	LAB						
			Depth of Sample (ft)	Depth of Sample into Rock (ft)	Elev Top of Sample (ft)	UCS (psi)	Modulus (ksi)	Unit Wt (pcf)	Rock Type
BB-IWBP-101	R1	480.1	27.1	1.3	453.0	18,659	4,020	167	SCHIST
BB-IWBP-102	R1	461.0	15.3	2.8	445.7	21265*		166.9	SCHIST
BB-IWBP-104	R4	461.6	31.9	9.3	429.7	18150.5*		161.1	SCHIST
BB-IWBP-105	R2	474.2	23	6.8	451.2	24680.25*		153.6	SCHIST
BB-IWBP-105	R5	474.2	38	8.2	436.2	18,510	2,770	173.9	SCHIST
BB-IWBP-103	R1	449.0	1.5	1.5	447.5	47,136		165.7	SCHIST
BB-IWBP-103	R1	449.0	1.7	1.7	447.4	27,552		155.9	SCHIST
BB-IWBP-202	R1	451.0	3.3	0.6	447.7	39,576		168.1	SCHIST
BB-IWBP-202	R1	451.0	3.5	0.8	447.6	30,432		161.1	SCHIST
BB-IWBP-202	R6	451.0	16.0	13.3	435.0	17,351	1,190	166	SCHIST
BB-IWBP-203	R2	449.8	6.7	2.2	443.1	25,992	4,530	164.8	SCHIST
BB-IWBP-201	R1	480.0	28.4	1.7	451.6	28,556	2,920	169.3	SCHIST
BB-IWBP-204	R2	474.3	30.8	3.2	443.5	23,472		162.9	SCHIST

* Average from Point Load Testing provided in the table below

Note: 1. BB-IWBP-105 R1 and R2 started at 22' and 23' bgs. Cored through Fractured Rock

2. Point load tests were performed on BB-IWBP-102 R1, 103R1, 104 R4, 105 R2, 202 R1, and 202 R4

3. Point load test on BB-IWBP-102 R1 and -204 R2 were Diametrical only, other samples were tested Diametrical and Axial

Choose design unconfined compressive strength as lowest test value (BB-IWBP-202, R6: 17.4 ksi).

$$\sigma_{u.r} := 17.4\text{ksi}$$

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

Engineers and Scientists

JOB: 09.0025976.01 Dectective Benjamin Campbell Bridge
SUBJECT: Footing Bearing Resistance on Rock
SHEET: 3 OF 10
CALCULATED BY: BMC 06/16/20
REVIEWED BY: ARB 06/17/20

C. Rock Quality Designation

Select representative RQD based on data collected in the in borings.

Boring ID	Core Run	Ground Surface El. (ft)	Depth Below Top of Rock (ft)			Length of Core Run (in)	Rec (%)	RQD %	Joint Spacing (in)	Joint Aperture (in)	Elev. (ft)		Rock Type
			Top		Bottom						Top	Bottom	
BB-IWBP-101	R1	480.1	1.1	-	6.2	5.1	100%	57%	<0.75-24	0.004-0.1	453.2	448.1	SCHIST
BB-IWBP-101	R2	480.1	6.2	-	8.2	2.0	83%	50%	8	0.004-0.01	448.1	446.1	SCHIST
BB-IWBP-101	R3	480.1	8.2	-	11.2	3.0	100%	53%	<0.75-24	0.004-0.4	446.1	443.1	SCHIST
BB-IWBP-102	R1	461.0	1.0	-	4.2	3.2	100%	24%	<0.75	0.004-0.4	447.5	444.3	SCHIST
BB-IWBP-102	R2	461.0	4.2	-	6.8	2.6	100%	45%	0.75-24	0.02-0.4	444.3	441.7	SCHIST
BB-IWBP-102	R3	461.0	6.8	-	9.8	3.0	89%	11%	<0.75-0.75	0.004-0.4	441.7	438.7	SCHIST
BB-IWBP-102	R4	461.0	9.8	-	12.0	2.2	100%	42%	0.75-8	0.01->0.4	438.7	436.5	SCHIST
BB-IWBP-103	R1	449.0	1.3	-	3.9	2.6	84%	0%	<0.75-8	0.004-0.1	447.7	445.1	SCHIST
BB-IWBP-103	R2	449.0	3.9	-	5.7	1.8	100%	19%	<0.75-8	0.004-0.1	445.1	443.3	SCHIST
BB-IWBP-103	R3	449.0	5.7	-	7.3	1.6	100%	0%	<0.75-8	0.004-0.1	443.3	441.7	SCHIST
BB-IWBP-103	R4	449.0	7.3	-	9.7	2.4	86%	31%	<0.75-8	0.02-0.4	441.7	439.3	SCHIST
BB-IWBP-103	R5	449.0	9.7	-	12.2	2.5	100%	17%	0.75-8	0.004-0.4	439.3	436.8	SCHIST
BB-IWBP-103	R6	449.0	12.2	-	15.2	3.0	92%	18%	<0.75-8	0.1-0.4	436.8	433.8	SCHIST
BB-IWBP-103	R7	449.0	15.2	-	17.5	2.3	79%	0%	<0.75-0.75	0.1-0.4	433.8	431.5	SCHIST
BB-IWBP-103	R8	449.0	17.5	-	17.8	0.3	50%	0%	<0.75	0.02-0.1	431.5	431.2	SCHIST
BB-IWBP-104	R1	461.6	-9.3	-	-6.8	2.5	100%	0%	<0.75-0.75	0.02-0.1	438.6	436.1	SCHIST
BB-IWBP-104	R2	461.6	-6.8	-	-4.3	2.5	100%	0%	<0.75-0.75	0.02-0.1	436.1	433.6	SCHIST
BB-IWBP-104	R3	461.6	-4.3	-	-2.3	2.0	75%	0%	<0.75-0.75	0.02-0.1	433.6	431.6	SCHIST
BB-IWBP-104	R4	461.6	-2.3	-	1.7	4.0	88%	20%	0.75-8	0.004-0.01	431.6	427.6	SCHIST
BB-IWBP-104	R5	461.6	1.7	-	4.4	2.7	100%	0%	<0.75-8	0.02-0.1	427.6	424.9	SCHIST
BB-IWBP-104	R6	461.6	4.4	-	8.1	3.7	100%	30%	0.75-8	0.004-0.1	424.9	421.2	SCHIST
BB-IWBP-105	R1	474.2	-9.3	-	-8.5	0.8	10%	0%	<0.75	0.02-0.1	452.2	451.4	SCHIST
BB-IWBP-105	R2	474.2	-8.3	-	-6.7	1.6	68%	31%	<0.75	0.02-0.1	451.2	449.6	SCHIST
BB-IWBP-105	R3	474.2	-1.5	-	-0.9	0.6	100%	0%	<0.75	0.1-0.4	444.4	443.8	SCHIST
BB-IWBP-105	R4	474.2	-0.9	-	1.4	2.3	39%	0%	<0.75	0.1-0.4	443.8	441.5	SCHIST
BB-IWBP-105	R5	474.2	6.7	-	10.1	3.4	98%	49%	<0.75-24	0.004-0.1	436.2	432.8	SCHIST
BB-IWBP-105	R6	474.2	10.1	-	12.6	2.5	100%	0%	<0.75-8	0.004-0.1	432.8	430.3	SCHIST
BB-IWBP-105	R7	474.2	12.6	-	15.9	3.3	98%	23%	<0.75-8	0.004-0.4	430.3	427.0	SCHIST
BB-IWBP-201	R1	480.0	0.3	-	4.7	4.4	98%	85%	<0.75-24	0.01-0.4	453.0	448.6	SCHIST
BB-IWBP-201	R2	480.0	4.7	-	7.5	2.8	97%	24%	<0.75-8	0.004-0.4	448.6	445.8	SCHIST
BB-IWBP-201	R3	480.0	7.5	-	10.2	2.7	94%	32%	<0.75-8	0.004-0.1	445.8	443.1	SCHIST
BB-IWBP-201	R4	480.0	10.2	-	15.2	5.0	100%	78%	0.75-24	0.004-0.4	443.1	438.1	SCHIST
BB-IWBP-202	R1	451.0	0.4	-	2.1	1.7	85%	0%	<0.75-8	0.01-0.4	447.9	446.2	SCHIST
BB-IWBP-202	R2	451.0	2.1	-	6.1	4.0	90%	39%	<0.75-8	0.004-0.4	446.2	442.2	SCHIST
BB-IWBP-202	R3	451.0	6.1	-	7.3	1.2	86%	0%	<0.75	0.01-0.4	442.2	441.0	SCHIST
BB-IWBP-202	R4	451.0	7.3	-	9.3	2.0	92%	18%	<0.75-8	0.004-0.4	441.0	439.0	SCHIST
BB-IWBP-202	R5	451.0	9.3	-	12.8	3.5	100%	21%	<0.75-24	0.004-0.4	439.0	435.5	SCHIST
BB-IWBP-202	R6	451.0	12.8	-	15.1	2.3	75%	36%	<0.75-8	0.004-0.4	435.5	433.2	SCHIST
BB-IWBP-202	R7	451.0	15.1	-	18.1	3.0	89%	0%	<0.75-8	0.004-0.4	433.2	430.2	SCHIST
BB-IWBP-202	R8	451.0	18.1	-	19.5	1.4	94%	25%	<0.75-8	0.004-0.4	430.2	428.8	SCHIST
BB-IWBP-202	R9	451.0	19.5	-	21.5	2.0	92%	23%	<0.75-8	0.004-0.4	428.8	426.8	SCHIST
BB-IWBP-203	R1	449.8	0.2		1.4	1.2	79%	0%	<0.75-8	0.004-0.4	445.1	443.9	SCHIST
BB-IWBP-203	R2	449.8	1.4		3.2	1.8	73%	34%	<0.75-8	0.004-0.4	443.9	442.1	SCHIST
BB-IWBP-203	R3	449.8	3.2		5.2	2.0	67%	0%	<0.75	0.01-0.4	442.1	440.1	SCHIST
BB-IWBP-204	R1	474.3	-8.9		-6.0	2.9	29%	0%	<0.75	0.004-0.4	446.7	443.8	SCHIST
BB-IWBP-204	R2	474.3	-6.0		-4.8	1.2	79%	0%	<0.75	0.01-0.4	443.8	442.6	SCHIST
BB-IWBP-204	R3	474.3	-4.8		-2.5	2.3	54%	0%	<0.75	0.02-0.4	442.6	440.3	SCHIST
BB-IWBP-204	R4	474.3	-1.8		0.1	1.9	63%	0%	<0.75	0.004-0.4	439.6	437.7	SCHIST
BB-IWBP-204	R5	474.3	0.1		1.5	1.4	76%	0%	<0.75	0.004-0.4	437.7	436.3	SCHIST
BB-IWBP-204	R6	474.3	1.5		3.7	2.2	85%	0%	<0.75-0.75	0.004-0.4	436.3	434.1	SCHIST
BB-IWBP-204	R7	474.3	3.7		5.1	1.4	82%	0%	<0.75	0.004-0.4	434.1	432.7	SCHIST
BB-IWBP-204	R8	474.3	5.1		7.7	2.6	90%	0%	<0.75-0.75	0.004-0.4	432.7	430.1	SCHIST
BB-IWBP-204	R9	474.3	7.7		9.7	2.0	100%	0%	<0.75	0.004-0.4	430.1	428.1	SCHIST
BB-IWBP-204	R10	474.3	9.7		10.9	1.2	64%	0%	<0.75	0.01-0.4	428.1	426.9	SCHIST
BB-IWBP-204	R11	474.3	10.9		14.0	3.1	100%	34%	<0.75-24	0.004-0.4	426.9	423.8	SCHIST

Average RQD for all borings is 23%. However, footings may be supported on rock with RQD=0%.

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

Engineers and Scientists

JOB: 09.0025976.01 Dectective Benjamin Campbell Bridge
SUBJECT: Footing Bearing Resistance on Rock
SHEET: 4 OF 10
CALCULATED BY: BMC 06/16/20
REVIEWED BY: ARB 06/17/20

2. Calculation of Rock Mass Rating (RMR)

From AASHTO LRFD Tables 10.4.6.4-1 and 10.4.6.4-2, determine the RMR (see sheets 7 and 8 for reference tables)

Rock Mass Rating

Parameter 1- Uniaxial Compressive Strength

$\sigma_{u.r} = 17.4 \cdot ksi$ $\sigma_{u.r} = 2506 \cdot ksf$

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_1 := 12$

Parameter 2- Drill Core Quality

Footings may be supported on rock with RQD=0%.

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_2 := 3$ for RQD = 0 to 25%

Parameter 3- Spacing of Joints

From boring logs, generally extremely close to close = < 0.75 in to 8 inches. Assume <2 in to account for fractured rock.

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating

$RR_3 := 5$

Parameter 4- Condition of Joints

From boring logs, aperture 0.004-0.4 inches and slightly rough surfaces.

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_4 := 12$

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

Engineers and Scientists

JOB: 09.0025976.01 Dectective Benjamin Campbell Bridge
SUBJECT: Footing Bearing Resistance on Rock
SHEET: 5 OF 10
CALCULATED BY: BMC 06/16/20
REVIEWED BY: ARB 06/17/20

Parameter 5- Ground Water Conditions

Hydrostatic Conditions- water under moderate pressure.

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating

$$RR_5 := 4$$

Parameter 6- Adjustment for joint orientation

The joint sets are generally high angle to low angle and generally rough and tight to moderately wide. Considering rock will remain embedded below foundation bearing level, joint orientation is considered fair.

From AASHTO LRFD Table 10.4.6.4-2

Relative Rating $RR_6 := -7$

Total RMR Rating

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

$$RMR = 29$$

From AASHTO LRFD Table 10.4.6.4-3 RMR= 29 is indicative of Poor Rock Quality

3. Determine Rock Property Constants s and m

From AASHTO LRFD Table 10.4.6.4-4 for Fair Quality Rock Mass

Categorized as rock type B, Schist, using s and m values interpolated from the logarithmic trend of plotted values from AASHTO Table 10.4.6.4-4 on Sheet 9, data plotted in range of RMR=29 on Sheet 10).

$m_1 := .064$ $s_1 := .0000072$ for RMR=29

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

From Wyllie "Foundations on Rock" Eq. 5.4 Pg.138

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

Where $C_{fl} := 1$ 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

Assume long strip, lowest $C_{fl.}$

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

Engineers and Scientists

JOB: 09.0025976.01 Dectective Benjamin Campbell Bridge
SUBJECT: Footing Bearing Resistance on Rock
SHEET: 6 OF 10
CALCULATED BY: BMC 06/16/20
REVIEWED BY: ARB 06/17/20

Nominal Bearing Resistance

$$q_n := C_{f1}\cdot\sqrt{s_1}\cdot\sigma_{u.r}\cdot\left[1+\sqrt{m_1\cdot\left(s_1^{-\frac{1}{2}}\right)+1}\right] = 40\cdot ksf$$

Say 40 ksf, based on RMR=29

Factored Bearing Resistance

Bearing Resistance Factor is specified in Table 10.5.5.2.2-1

$\phi_b := 0.45$ Footing on rock (Strength Limit State)

$q_R := \phi_b\cdot q_n$ $q_R = 18\cdot ksf$

Say 18 ksf

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

Engineers and Scientists

JOB: 09.0025976.01 Dectective Benjamin Campbell Bridge
SUBJECT: Footing Bearing Resistance on Rock
SHEET: 7 OF 10
CALCULATED BY: BMC 06/16/20
REVIEWED BY: ARB 06/17/20

Reference:I:\Mathcad\units.xmcd

Table 10.4.6.4-1 Geomechanics Classification of Rock Masses.

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

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

Engineers and Scientists

Table 10.4.6.4-2 Geomechanics Rating Adjustment for Joint Orientations.

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

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

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

Table 10.5.5.2.2-1—Resistance Factors for Geotechnical Resistance of Shallow Foundations at the Strength Limit State

Method/Soil/Condition			Resistance Factor
Bearing Resistance	φ_b	Theoretical method (Munfakh et al., 2001), in clay	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using *CPT*	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using *SPT*	0.45
		Semi-empirical methods (Meyerhof, 1957), all soils	0.45
		Footings on rock	0.45
		Plate Load Test	0.55
Sliding	φ_τ	Precast concrete placed on sand	0.90
		Cast-in-Place Concrete on sand	0.80
		Cast-in-Place or precast Concrete on Clay	0.85
		Soil on soil	0.90
	φ_{ep}	Passive earth pressure component of sliding resistance	0.50

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

Engineers and Scientists

JOB: 09.0025976.01 Dectective Benjamin Campbell Bridge
SUBJECT: Footing Bearing Resistance on Rock
SHEET: 9 OF 10
CALCULATED BY: BMC 06/16/20
REVIEWED BY: ARB 06/17/20

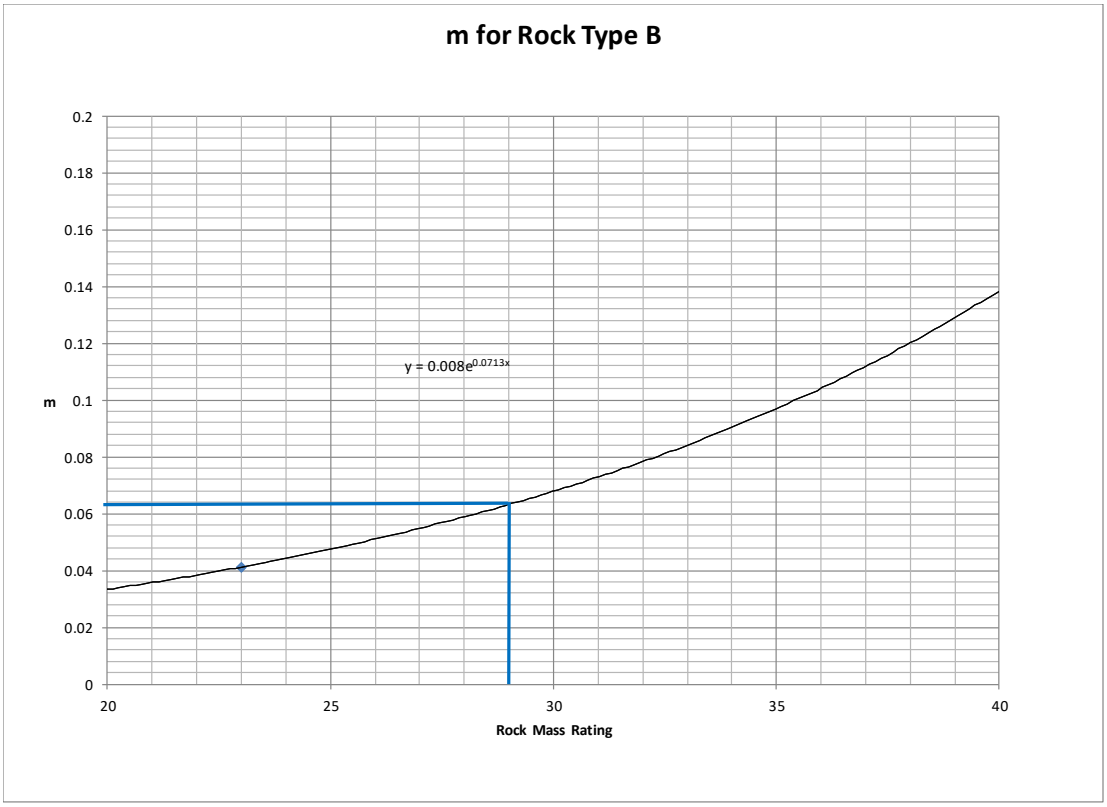
Table 10.4.6.4-4 Approximate relationship between rock-mass quality and material constants used in defining nonlinear strength (*Hoek and Brown, 1988*)

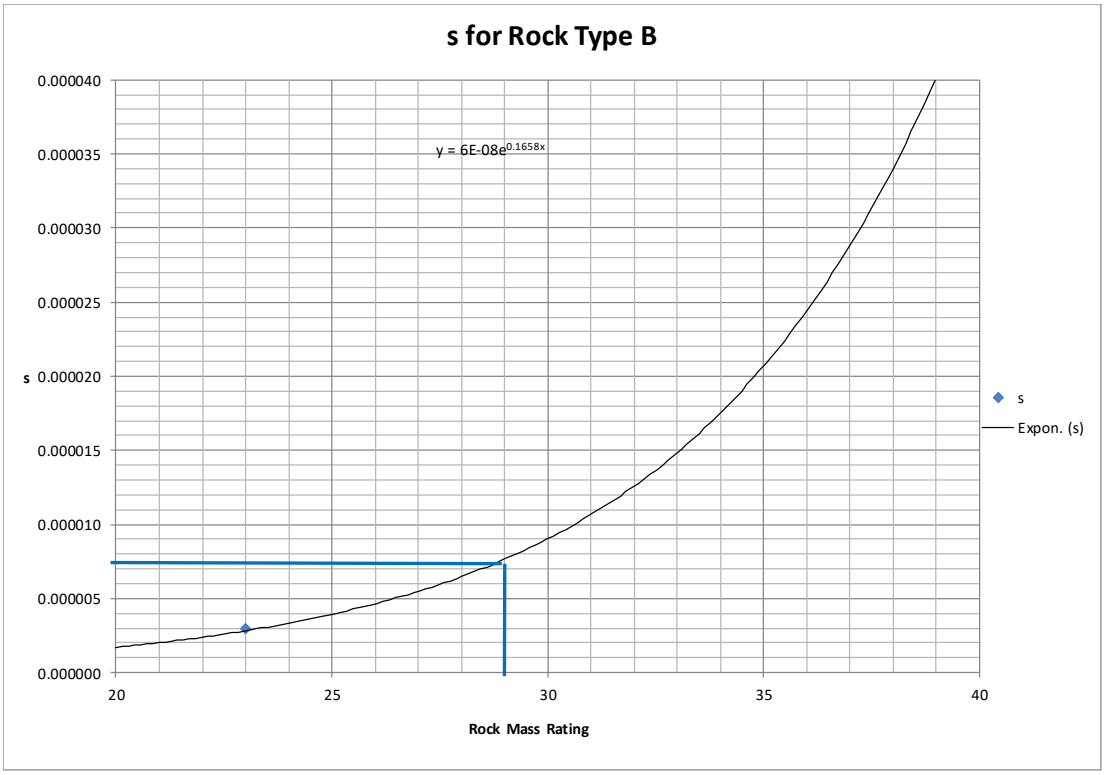
Rock Quality	Constants	Rock Type A = Carbonate rocks with well developed crystal cleavage—*dolomite, limestone and marble* B = Lithified argrillaceous rocks—*mudstone, siltstone, shale and slate (normal to cleavage)* C = Arenaceous rocks with strong crystals and poorly developed crystal cleavage—*sandstone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

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

Engineers and Scientists

JOB: 09.0025976.01 Dectective Benjamin Campbell Bridge
SUBJECT: Footing Bearing Resistance on Rock
SHEET: 10 OF 10
CALCULATED BY: BMC 06/16/20
REVIEWED BY: ARB 06/17/20





Soil Bearing Resistance Calculation

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

Engineers and Scientists

JOB: 09.0025976.01 Det. Benjamin Campbell Bridge #3666
SUBJECT: Abutment Bearing on Moraine (Abutment 2)
SHEET: 1 OF 7
CALCULATED BY B. Cardali 4/9/2020
CHECKED BY A. Blaisdell 4/22/2020

Objective

Calculate soil bearing resistance for foundation bearing on dense Moraine sand with a friction angle equal to 36 degrees using the Theoretical method (Munfakh et al., 2001) in Sand using SPT data. Evaluate strength and service limit bearing resistance for a range of effective footing widths.

References

1. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 5th edition, 2010 (AASHTO LRFD), Articles 10.5.5.2.2 and 10.6.3.1.
2. Terzaghi, Peck & Mesri, Soil Mechanics in Engineering Practice, Third Edition, 1996.

Soil Properties and Geotechnical Inputs

$\phi_f := 36deg$ Friction angle of soil (Dense Gravel - Moraine OR Fractured Rock)

$\phi_b := 0.45$ Bearing resistance factor as specified in Table 10.5.5.2.2-1 (Theoretical Method, SPT Data, Strength Limit)

$c := 0ksf$ Cohesion, taken as undrained shear strength from correlated N values and Pocket Penetrometer data

$\gamma := 130pcf$ Unit weight of soil above or below the bearing depth of the footing

$N_c := 50.6$ Cohesion term bearing capacity factor as specified in Table 10.6.3.1.2a-1

$N_q := 37.8$ Surcharge term bearing capacity factor as specified in Table 10.6.3.1.2a-1

$N_\gamma := 56.3$ Total unit weight term bearing capacity factor as specified in Table 10.6.3.1.2a-1

C_{wq}, $C_{w\gamma}$:= Correction factors to account for the location of the groundwater table as specified in Table 10.6.3.1.2a-2

Depth to water table at or above depth of footing (D_f) $C_{wq} := .5$ $C_{w\gamma} := .5$

d_q:= Correction factor to account for the shearing resistance along the failure surface passing through cohesionless material above the bearing elevation as specified in Table 10.6.3.1.2a-4

s_c, s_γ, s_q:= Footing shape correction factors as specified in Table 10.6.3.1.2a-2

$S_c := \frac{1}{4}in$ Allowable settlement *(keep very low to allow some immediate compression of material immediately below seal that is not directly considered)*

q_s= Service limit bearing resistance for allowable settlement, Resistance Factor = 1.0

$N_{60} := 80$ Lowest N_{60} values from SPT near bottom of seal, 100+ below seal

Load inclination factors are omitted considering modest embedment offooting per C10.6.3.1.2a.

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

Engineers and Scientists

JOB: 09.0025976.01 Det. Benjamin Campbell Bridge #3666
SUBJECT: Abutment Bearing on Moraine (Abutment 2)
SHEET: 2 OF 7
CALCULATED BY B. Cardali 4/9/2020
CHECKED BY A. Blaisdell 4/22/2020

Footing Dimensions

$B_1 := \begin{pmatrix} 6 \\ 9 \\ 12 \\ 15 \\ 16 \end{pmatrix} ft$ Range of effective footing widths considered (includes eccentricity)

$L_1 := 55ft$ Length of footing

$D_f := 6ft$ Footing embedment depth *(considers footing only and excludes riprap - conservative)*

Strength Limit Design

$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\gamma B N_{\gamma m} C_{w\gamma}$ Nominal Bearing Resistance Formula

$q.D = \varphi_b \times q_n$ Factored Bearing Resistance Formula

Correction Factors

$d_{qtable} := \frac{D_f}{B_1}$ $d_{qtable} = \begin{pmatrix} 1 \\ 0.67 \\ 0.5 \\ 0.4 \\ 0.38 \end{pmatrix}$ Using Table 10.6.3.1.2a-4

$d_q := 1$ dq assumed soil above footing less competent than soil below footing.

$s_c := 1 + \left(\frac{B_1}{L_1}\right)\left(\frac{N_q}{N_c}\right)$ $s_c = \begin{pmatrix} 1.08 \\ 1.12 \\ 1.16 \\ 1.2 \\ 1.22 \end{pmatrix}$

$s_q := 1 + \left(\frac{B_1}{L_1}\tan(\phi_f)\right)$ $s_q = \begin{pmatrix} 1.08 \\ 1.12 \\ 1.16 \\ 1.2 \\ 1.21 \end{pmatrix}$

$s_\gamma := 1 - 0.4\left(\frac{B_1}{L_1}\right)$ $s_\gamma = \begin{pmatrix} 0.96 \\ 0.93 \\ 0.91 \\ 0.89 \\ 0.88 \end{pmatrix}$

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

Engineers and Scientists

JOB: 09.0025976.01 Det. Benjamin Campbell Bridge #3666
SUBJECT: Abutment Bearing on Moraine (Abutment 2)
SHEET: 3 OF 7
CALCULATED BY B. Cardali 4/9/2020
CHECKED BY A. Blaisdell 4/22/2020

Bearing Capacity Factors

$$N_{cm} := N_c \cdot s_c \qquad N_{cm} = \begin{pmatrix} 54.72 \\ 56.79 \\ 58.85 \\ 60.91 \\ 61.6 \end{pmatrix}$$

$$N_{qm} := N_q \cdot s_q \cdot d_q \qquad N_{qm} = \begin{pmatrix} 40.8 \\ 42.3 \\ 43.8 \\ 45.3 \\ 45.8 \end{pmatrix}$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \qquad N_{\gamma m} = \begin{pmatrix} 53.8 \\ 52.6 \\ 51.4 \\ 50.2 \\ 49.7 \end{pmatrix}$$

Nominal Bearing Resistance

$$q_n := \overrightarrow{\left(c \cdot N_{cm} + \gamma \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma \cdot B_1 \cdot N_{\gamma m} \cdot C_{w\gamma}\right)} \qquad q_n = \begin{pmatrix} 26.4 \\ 31.9 \\ 37.1 \\ 42.1 \\ 43.7 \end{pmatrix} \cdot \text{ksf}$$

Factored Bearing Resistance - Strength Limit State

$$q_D := \phi_b \cdot q_n \qquad q_D = \begin{pmatrix} 11.9 \\ 14.3 \\ 16.7 \\ 19 \\ 19.7 \end{pmatrix} \cdot \text{ksf} \qquad \text{for} \quad B_1 = \begin{pmatrix} 6 \\ 9 \\ 12 \\ 15 \\ 16 \end{pmatrix} \cdot \text{ft}$$

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

Engineers and Scientists

JOB: 09.0025976.01 Det. Benjamin Campbell Bridge #3666
SUBJECT: Abutment Bearing on Moraine (Abutment 2)
SHEET: 4 OF 7
CALCULATED BY B. Cardali 4/9/2020
CHECKED BY A. Blaisdell 4/22/2020

Service Limit Design

Evaluate service limit bearing resistance for the specified allowable settlement using the semi-empirical SPT Method by Burland and Burbidge (1985) provided in Terzaghi, Peck & Mesri, 96.

$S_{cm} := S_c \cdot \frac{1}{1mm}$ $\quad S_{cm} = 6.4$ Allowable settlement in millimeters and unitless

$B_{1m} := B_1 \cdot \frac{1}{1m}$ $\quad B_{1m} = \begin{pmatrix} 1.8 \\ 2.7 \\ 3.7 \\ 4.6 \\ 4.9 \end{pmatrix}$ Effective footing width in meters and unitless

$S_{cmr} := S_{cm} \cdot \left[\frac{\left[1.25 \cdot \left(\frac{L_1}{B_1} \right) \right]}{\left(\frac{L_1}{B_1} \right) + 0.25} \right]^2$ $\quad S_{cmr} = \begin{pmatrix} 9.4 \\ 9.2 \\ 8.9 \\ 8.7 \\ 8.6 \end{pmatrix}$ Correction formula for rectangular footings (Terzaghi EQ 50.14)

$EQ_1 := S_{cm} \cdot \left(\frac{S_{cm}}{S_{cmr}} \right)$ $\quad EQ_1 = \begin{pmatrix} 4.29 \\ 4.4 \\ 4.52 \\ 4.64 \\ 4.68 \end{pmatrix}$

$EQ_2 := \frac{N_{60}^{1.4}}{1.7 \cdot B_{1m}^{0.75}}$ $\quad EQ_2 = \begin{pmatrix} 173 \\ 127 \\ 103 \\ 87 \\ 83 \end{pmatrix}$

$q_{snc} := \overrightarrow{\left(EQ_1 \cdot EQ_2 \right)}$ $\quad q_{snc} =$

	0
0	740.6
1	561.0
2	464.0
3	402.8
4	387.0

Formula results are in kPa (Terzaghi EQ 50.28)

Results represent normally consolidated soil.

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

Engineers and Scientists

JOB: 09.0025976.01 Det. Benjamin Campbell Bridge #3666
SUBJECT: Abutment Bearing on Moraine (Abutment 2)
SHEET: 5 OF 7
CALCULATED BY B. Cardali 4/9/2020
CHECKED BY A. Blaisdell 4/22/2020

$q_s := q_{snc}$ $\quad q_s = \begin{pmatrix} 741 \\ 561 \\ 464 \\ 403 \\ 387 \end{pmatrix}$

Assumes sand is normally consolidated at current effective stress (likely conservative)

$q_{sm} := q_s \cdot 1\,kPa$ $\quad q_{sm} = \begin{pmatrix} 741 \\ 561 \\ 464 \\ 403 \\ 387 \end{pmatrix} \cdot kPa$

Service limit bearing resistance for allowable settlement (metric units)

$q_{se} := q_{sm}$ $\quad q_{se} = \begin{pmatrix} 15.5 \\ 11.7 \\ 9.7 \\ 8.4 \\ 8.1 \end{pmatrix} \cdot ksf$

Service limit bearing resistance for allowable settlement (English units)

for $\quad B_1 = \begin{pmatrix} 6 \\ 9 \\ 12 \\ 15 \\ 16 \end{pmatrix} \cdot ft$

Bearing Chart

Abutment 2 Footing with a given effective width should be sized to keep Fatored Bearing Pressure (Strength condition) below the Blue curve and Service Bearing Pressure below the Red Curve.

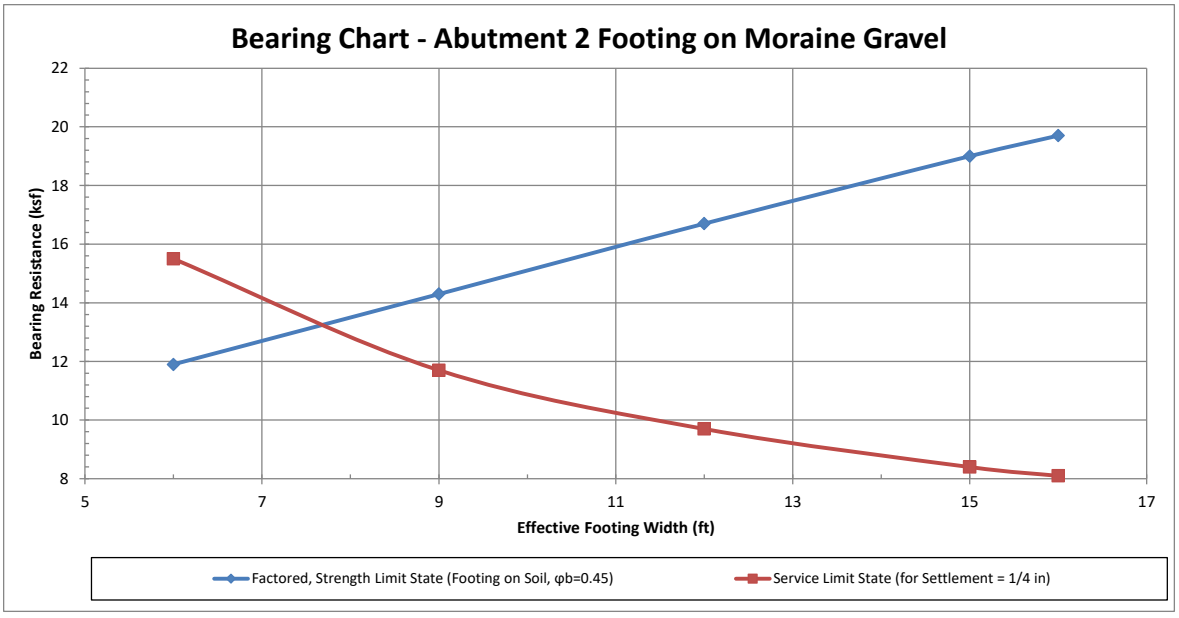


Table 10.6.3.1.2a-1—Bearing Capacity Factors N_c (Prandtl, 1921), N_q (Reissner, 1924), and N_γ (Vesic, 1975)

ϕ_f	N_c	N_q	N_γ	ϕ_f	N_c	N_q	N_γ
0	5.14	1.0	0.0	23	18.1	8.7	8.2
1	5.4	1.1	0.1	24	19.3	9.6	9.4
2	5.6	1.2	0.2	25	20.7	10.7	10.9
3	5.9	1.3	0.2	26	22.3	11.9	12.5
4	6.2	1.4	0.3	27	23.9	13.2	14.5
5	6.5	1.6	0.5	28	25.8	14.7	16.7
6	6.8	1.7	0.6	29	27.9	16.4	19.3
7	7.2	1.9	0.7	30	30.1	18.4	22.4
8	7.5	2.1	0.9	31	32.7	20.6	26.0
9	7.9	2.3	1.0	32	35.5	23.2	30.2
10	8.4	2.5	1.2	33	38.6	26.1	35.2
11	8.8	2.7	1.4	34	42.2	29.4	41.1
12	9.3	3.0	1.7	35	46.1	33.3	48.0
13	9.8	3.3	2.0	36	50.6	37.8	56.3
14	10.4	3.6	2.3	37	55.6	42.9	66.2
15	11.0	3.9	2.7	38	61.4	48.9	78.0
16	11.6	4.3	3.1	39	67.9	56.0	92.3
17	12.3	4.8	3.5	40	75.3	64.2	109.4
18	13.1	5.3	4.1	41	83.9	73.9	130.2
19	13.9	5.8	4.7	42	93.7	85.4	155.6
20	14.8	6.4	5.4	43	105.1	99.0	186.5
21	15.8	7.1	6.2	44	118.4	115.3	224.6
22	16.9	7.8	7.1	45	133.9	134.9	271.8

Table 10.6.3.1.2a-2—Coefficients C_{wq} and $C_{w\gamma}$ for Various Groundwater Depths

D_w	C_{wq}	$C_{w\gamma}$
0.0	0.5	0.5
D_f	1.0	0.5
$>1.5B + D_f$	1.0	1.0

Where the position of groundwater is at a depth less than 1.5 times the footing width below the footing base, the bearing resistance is affected. The highest anticipated groundwater level should be used in design.

Table 10.6.3.1.2a-3—Shape Correction Factors s_c, s_γ, s_q

Factor	Friction Angle	Cohesion Term (s_c)	Unit Weight Term (s_γ)	Surcharge Term (s_q)
Shape Factors s_c, s_γ, s_q	$\phi_f = 0$	$1+\left(\frac{B}{5L}\right)$	1.0	1.0
	$\phi_f > 0$	$1+\left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)$	$1-0.4\left(\frac{B}{L}\right)$	$1+\left(\frac{B}{L}\tan\phi_f\right)$

Table 10.6.3.1.2a-4—Depth Correction Factor d_q

Friction Angle, ϕ_f (degrees)	D_f/B	d_q
32	1	1.20
	2	1.30
	4	1.35
	8	1.40
37	1	1.20
	2	1.25
	4	1.30
	8	1.35
42	1	1.15
	2	1.20
	4	1.25
	8	1.30

The depth correction factor should be used only when the soils above the footing bearing elevation are as competent as the soils beneath the footing level; otherwise, the depth correction factor should be taken as 1.0.

Linear interpolations may be made for friction angles in between those values shown in Table 10.6.3.1.2a-4.

Earth Pressure Calculation

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

Engineers and Scientists

JOB: 09.0025976.01 Detective Benjamin Campbell Bridge
SUBJECT: Lateral Earth Pressures
SHEET: 1 OF 1
CALCULATED BY B. Cardali 6/18/20
CHECKED BY A. Blaisdell 6/18/20

Subject:

Evaluate lateral earth pressure coefficients for Abutments

References:

1. MaineDOT Bridge Design Guide, Chapter 3
2. AASHTO LRFD Bridge Design Specifications, 8th Edition (2017)
3. U.S. Army Corps of Engineers Engineer Manual 1110-2-2502, Retaining and Flood Walls

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)*

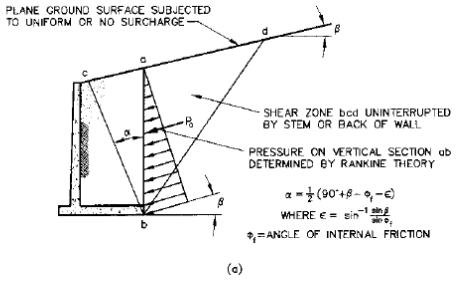
$\beta := 0\text{deg}$ Angle of backfill to the horizontal

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

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.

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

Based on Figure C3.11.5.3-1 of LRFD, the abutment is considered to be a short-heeled wall. See page two of this calculation for Abutment section. Therefore, Coulomb theory should be used to calculate active earth pressures.



Coloumb Active Earth Pressure Coefficient (Short-Heeled Wall)

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

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

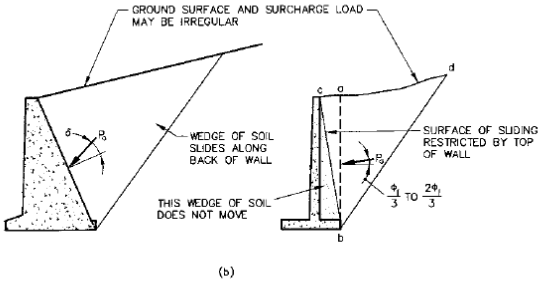


Figure C3.11.5.3-1—Application of (a) Rankine and (b) Coulomb Earth Pressure Theories in Retaining Wall Design

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

Engineers and Scientists

JOB: 09.0025976.01 Detective Benjamin Campbell Bridge
SUBJECT: Lateral Earth Pressures
SHEET: 2 OF 1
CALCULATED BY B. Cardali 6/18/20
CHECKED BY A. Blaisdell 6/18/20