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GEOTECHNICAL DESIGN REPORT WESTERN AVENUE BRIDGE NO. 5808 OVER INTERSTATE 95 MAINE DOT WIN 21672.00 AUGUSTA, MAINE

May 2023
09.0025983.01

Prepared for:
Maine Department of Transportation
Augusta, Maine

Prepared by:
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VIA EMAIL

May 19, 2023
File No. 09.0025983.01

Ms. Laura Krusinski
Maine Department of Transportation
16 State House Station
Augusta, Maine 04333-0016

Re: Geotechnical Design Report
Replacement of Western Avenue Bridge No. 5808 over Interstate 95
MaineDOT WIN 21672.00
Augusta, Maine

Dear Laura:

We are pleased to provide this Geotechnical Design Report, which includes geotechnical design recommendations for the replacement of Maine Department of Transportation (MaineDOT) Western Avenue Bridge No. 5808 in Augusta, Maine. Our work was completed under GZA GeoEnvironmental, Inc.'s (GZA's) June 3, 2020 General Consulting Agreement (GCA CTM2020060300000000709) with the MaineDOT Bridge Program, and incorporates GZA's Proposal No. 09.P000134.22, dated April 4, 2022, and the *Limitations* Included in **Appendix A** of this report.

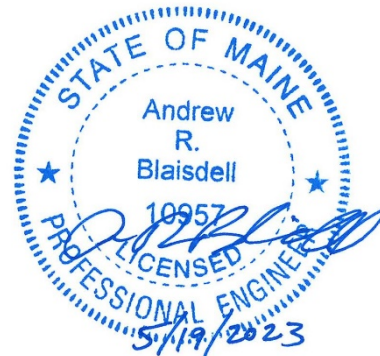
It has been a pleasure serving MaineDOT on this phase of the project, and we look forward to our continued work with you through project completion. If you have any questions regarding the report, or if we can provide further assistance, please do not hesitate to contact the undersigned.

Very truly yours,

GZA GEOENVIRONMENTAL, INC.

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Project Manager

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Attachment: Geotechnical Design Report



TABLE OF CONTENTS

	<u>Page</u>
1.0 INTRODUCTION	1
1.1 BACKGROUND	1
1.2 OBJECTIVES AND SCOPE OF SERVICES.....	2
2.0 SUBSURFACE EXPLORATIONS.....	2
2.1 PRELIMINARY DESIGN BORINGS.....	3
2.2 FINAL DESIGN PHASE BORINGS.....	3
2.3 GEOPHYSICAL TESTING.....	4
3.0 LABORATORY TESTING.....	4
4.0 SUBSURFACE CONDITIONS.....	4
4.1 SURFICIAL AND BEDROCK GEOLOGY.....	4
4.2 SUBSURFACE PROFILE	5
4.2.1 Bedrock	5
4.2.2 Highly Fractured Bedrock.....	6
4.2.3 Groundwater	7
5.0 ENGINEERING EVALUATIONS	7
5.1 GENERAL.....	7
5.2 APPROACH AND HIGHWAY EMBANKMENTS	7
5.3 EVALUATION OF FOUNDATION TYPES	8
5.3.1 Abutment and Pier Foundations	8
5.4 LOAD AND RESISTANCE FACTORS	8
5.5 SPREAD FOOTING DESIGN CONSIDERATIONS.....	9
5.5.1 Spread Footing on Bedrock	9
5.6 EXISTING ABUTMENT FOOTING EVALUATION	10
5.7 LATERAL EARTH PRESSURE.....	11
5.7.1 Geosynthetic Reinforced Soil Backfill.....	11
5.7.2 Conventional Backfill (Phase 3 wingwall portions only)	12
5.8 SEISMIC DESIGN CONSIDERATIONS.....	12
5.9 FROST PROTECTION	12
5.10 SIGNAL MAST ARM FOUNDATIONS.....	12
6.0 RECOMMENDATIONS	13
6.1 EMBANKMENT DESIGN CONSIDERATIONS	13



TABLE OF CONTENTS (*continued*)

6.2 SEISMIC DESIGN	13
6.3 GEOSYNTHETIC REINFORCED SOIL BACKFILL ABUTMENTS AND WINGWALLS	14
6.4 PHASE 3 WINGWALL DESIGN (CONVENTIONAL BACKFILL)	15
6.5 RECOMMENDATIONS FOR FOUNDATIONS	15
6.5.1 Pressure Grouting	15
6.5.2 Spread Footing Design	16
7.0 CONSTRUCTION CONSIDERATIONS	17
7.1 EXCAVATION LIMITATION BELOW EXISTING ABUTMENTS	17
7.2 EXCAVATION, TEMPORARY LATERAL SUPPORT AND DEWATERING	17
7.3 SUBGRADE PREPARATION	18
7.4 REUSE OF ON-SITE MATERIALS	18

TABLES

TABLE 1	Summary of Subsurface Explorations
TABLE 2	Summary of Rock Core

FIGURES

FIGURE 1	Locus Plan
FIGURE 2	Boring Location Plan
FIGURE 3	Interpretive Subsurface Profile 1
FIGURE 4	Interpretive Subsurface Profile 2

APPENDICES

APPENDIX A	Limitations
APPENDIX B	Test Boring Logs
APPENDIX C	Geophysical Test Reports
APPENDIX D	Laboratory Test Results
APPENDIX E	Rock Core Photographs
APPENDIX F	Engineering Calculations



1.0 INTRODUCTION

This report presents the results of the geotechnical evaluation by GZA GeoEnvironmental, Inc. (GZA) for the replacement of Maine Department of Transportation (MaineDOT) Western Avenue Bridge No. 5808 in Augusta, Maine. Our services were completed in accordance with GZA's June 3, 2020 General Consulting Agreement (GCA CTM20200603000000000709) with the MaineDOT Bridge Program, and incorporates GZA's Proposal No. 09.P000134.22, dated April 4, 2022, and the *Limitations* Included in **Appendix A** of this report. Stantec is serving as the bridge designer for MaineDOT.

1.1 BACKGROUND

The existing Western Avenue Bridge No. 5808 was constructed circa 1955 and spans east to west, carrying Western Avenue (Route 202) over Interstate 95 (I-95), as shown in **Figure 1**. It is a 257-foot-long, four-span, continuous bridge with steel beams and a reinforced concrete deck. The bridge consists of dual 45.5-foot bridges supported independently by concrete hammerhead piers and concrete stub abutments.

The 1956 as-built plans indicate that the stub abutments are supported by spread footings with bottom-of-footing elevations of El. 223.3 and 221.8 at Abutments 1 and 2, respectively. Original grades along the bridge baseline are shown to vary from about El. 230 to El. 222, indicating the footings are embedded approximately 2 feet below original grades, likely on naturally deposited soil. The three pairs of pier stems are shown to be supported by spread footings bearing on bedrock between El. 206.3 and 209.3, roughly 5 to 10 feet below the existing ground surface. Maximum footing bearing pressures are indicated to be 5.2 kips per square foot (ksf) for the abutment footings and 10.4 ksf for the pier footings on the 1956 plans. Elevations referenced in this report are in feet and refer to the North American Vertical Datum of 1988 (NAVD88). Elevations referenced from the 1956 drawings used a datum shift of approximately -0.7 feet to convert from the National Geodetic Vertical Datum of 1929 to NAVD88, as provided by Stantec.

Degradation of the substructures was documented by Stantec and MaineDOT in January and August 2018 and includes heavy deterioration, spalling and patches of the abutment backwalls and bridge seats, and isolated concrete delamination at the piers.

A full bridge replacement is planned for the project. The replacement consists of an on-alignment alternative including an approximately 157-foot-long, two-span bridge. The vertical clearance is to be increased, which will be achieved by raising the bridge approaches by about 1 foot. The superstructure is anticipated to consist either of steel plate girders with a composite deck or a Prefabricated Bridge Unit alternative. The proposed abutments will be full height, cast-in-place concrete, abutments supported by spread footings bearing directly on bedrock. The central pier will be supported on the existing spread footings bearing on bedrock with new pedestals that will be doweled into the existing footing. The new abutments will be located between the existing abutments and adjacent piers, resulting in fills up to approximately 29 feet high immediately behind the proposed abutments and wingwalls. To reduce the



abutment footprint and limit excavation near existing abutments, a backfill material limiting the lateral pressure on the backwall is planned behind the abutments.

The bridge will be constructed in three phases. A temporary detour bridge will be constructed just north of the existing bridge. During Phase 1, it is anticipated that construction of the new abutments will be completed under the existing bridge, while traffic is maintained, to limit the duration of traffic impacts of the temporary detour. Phase 2 includes moving all Western Avenue traffic to the temporary bridge, demolishing the existing bridge, and constructing the remainder of the new bridge excluding portions of the wingwalls directly adjacent to the temporary structure. Phase 3 will include moving traffic to the new bridge, removing the temporary bridge, and constructing the remaining portions of the wingwalls.

1.2 OBJECTIVES AND SCOPE OF SERVICES

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

- Conducted a site visit to observe surficial conditions and reviewed existing bridge plans, historical topography, and mapped surficial and bedrock geology of the site;
- Reviewed subsurface data collected by GZA during the preliminary subsurface exploration program, which consisted of three test borings;
- Coordinated and observed a supplemental subsurface exploration program, consisting of nine additional test borings, to further evaluate subsurface conditions for the bridge and new signal mast arm locations;
- Conducted a laboratory testing program to evaluate engineering and index properties of the site soils and bedrock;
- Conducted geotechnical engineering analyses for soil and bedrock properties; frost susceptibility; American Association of State Highway and Transportation Officials Load and Resistance Factor Design (AASHTO LRFD) load and resistance factors associated with geotechnical design elements; nominal resistance of existing and proposed footings on bedrock foundations; nominal resistance of existing footings on soil foundations; geotextile reinforced soil abutment backwall design parameters; signal mast arm foundation geotechnical design parameters; and site-specific seismic design considerations;
- Developed geotechnical engineering recommendations including foundation design recommendations for footings on bedrock; abutment backwalls, lateral earth pressures, GRS limits for backfill, excavation limitations near existing foundations, seismic design parameters; and recommended construction considerations; and
- Prepared this report summarizing our findings and design recommendations.

2.0 SUBSURFACE EXPLORATIONS

Details of the previous and current subsurface explorations are provided in the following sections.



2.1 PRELIMINARY DESIGN BORINGS

GZA completed a preliminary subsurface exploration program consisting of three (3) test borings (BB-AI95-101 through BB-AI95-103), the locations and designations of which are shown on the attached **Boring Location Plan, Figure 2**. Borings BB-AI95-101 and -103 were completed about 15 feet behind the face of each abutment, and boring BB-AI95-102 was drilled between the existing Pier 2 footings (Pier 2-N and Pier 2-S). All three borings were drilled from Western Avenue. Boring BB-AI95-102 was drilled through the bridge deck within the median of I-95. The as-drilled boring locations and elevations were surveyed by MaineDOT, provided to GZA and shown on the logs; the surveyed as-drilled locations are shown on **Figure 2**.

The borings were drilled to depths of approximately 17 to 43 feet below ground surface (bgs) and terminated approximately 10 to 11 feet into bedrock. New England Boring Contractors of Hermon, Maine provided drilling services and coordinated utility clearance. The drilling was completed from July 16 through July 18, 2018. GZA personnel monitored the drilling work and prepared logs of each boring, included in **Appendix B**.

The borings were drilled using 3-inch or 4-inch casing and drive- or spin-and-wash techniques through the overburden and coring equipment in the bedrock. Standard Penetration Testing (SPT) and split-spoon sampling were performed at 5-foot typical intervals in overburden soils, or at tighter intervals near the bearing level of existing abutment footings. SPTs were conducted according to MaineDOT requirements, using an automatic hammer system calibrated in accordance with ASTM D4633-05 and MaineDOT procedures. The drilling subcontractor backfilled the approach boreholes with cuttings or sand and topped them with asphalt cold patch upon completion, and they patched the bridge deck hole with quick-set concrete. Approximately 10 feet of rock core was taken from each boring using NX (2.0-inch diameter) coring equipment. Bedrock core photographs are presented in **Appendix E**.

2.2 FINAL DESIGN PHASE BORINGS

GZA completed a final design subsurface exploration program consisting of seven (7) bridge test borings (BB-AI95-201 through BB-AI95-206, plus boring BB-AI95-202A) and two (2) signal mast arm test borings (HB-AI95-201 and HB-AI95-202) the locations and designations of which are shown on **Figure 2**. The borings were drilled between October 30 and November 3, 2022. The BB-AI95-200 series borings were drilled at substructure locations of the proposed replacement bridge, and the HB-AI95-200 series borings were drilled near the proposed signal mast arm locations. Two BB-AI95-200 series borings were drilled at each of the proposed replacement bridge abutment and pier locations. All nine borings were drilled from Western Avenue. The seven bridge borings were drilled through the bridge deck.

The borings were drilled using a track-mounted Mobile B-53 drill rig. The borings were drilled using similar methods as described for the preliminary phase borings to depths of approximately 11 to 31 feet and terminated approximately 5 to 18 feet into bedrock. SPTs were conducted using automatic hammer NEBC No. 23, which had rated hammer efficiency factors at the time of drilling of 0.86. Rock core was taken from each boring except for HB-AI95-202 using NX (2.0-inch diameter) coring equipment. Bedrock core photographs are presented in **Appendix E**.



The as-drilled boring locations and elevations were surveyed by MaineDOT, provided to GZA and are shown on the logs; the surveyed as-drilled locations are shown on **Figure 2**. GZA personnel monitored the drilling work and prepared logs of each boring that is included in **Appendix B**.

2.3 GEOPHYSICAL TESTING

Northeast Geophysical Services (NGS) conducted acoustic and optical borehole televiewer (ATV/OTV) surveys in the cored portion of test boring BB-AI95-202A on December 28, 2022. The purpose of the survey was to evaluate the depth, orientation, aperture, dip and dip direction of apparent discontinuities in bedrock. ATV and OTV image log plots, results of the televiewer logs, and polar plots of the dip and dip direction of the 36 interpreted features for BB-AI95-202A are contained in the NGS report, which is included in **Appendix C**. *It is noted that the depth reference on the geophysical logging is at the top of casing, therefore, 1.5 feet should be subtracted from the stated depths to establish depth below ground surface.*

3.0 LABORATORY TESTING

GZA retained Thielsch Engineering's Geotechnical Laboratory in 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:

- Twenty (20) gradation analysis / MaineDOT Frost Classification / Unified Soil Classification System (USCS) assessments;
- Twenty (20) moisture content tests;
- Seven (7) hydrometer tests;
- Two (2) Atterberg limits analyses; and
- Five (5) unconfined compression/secant modulus tests on bedrock samples.

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

4.0 SUBSURFACE CONDITIONS

4.1 SURFICIAL AND BEDROCK GEOLOGY

Based on available surficial geologic mapping¹ and the results of the subsurface explorations, the surficial unit at the site is Glacial till, which consists of a stratified mixture of sand, silt and gravel-size rock debris deposited by glacial ice.

¹ Thompson, W.B., 2009. Surficial Geologic of the Augusta quadrangle, Maine: Maine Geological Survey, map 09-7, scale 1:24,000.



Bedrock in the vicinity of the site is mapped² as the Mayflower Hill formation and Hallowell granite. The Mayflower Hill formation is characterized as fine to medium grained quartz-feldspar-biotite granofels and minor schist and is part of the Vassalboro group of Silurian age. Hallowell granite is characterized primarily as medium grained, two-mica granite consisting of feldspar, quartz, biotite and muscovite of Devonian age.

4.2 SUBSURFACE PROFILE

Two soil units, Fill and Glacial Till, were encountered in the test borings underlying approximately 2.5 to 6 inches of asphalt pavement (in Western Avenue approaches) and overlying bedrock. The thicknesses and generalized descriptions of the soil units are presented in the following table, in descending order from existing ground surface. Detailed descriptions of the materials encountered at specific locations are provided in the boring logs in **Appendix B**. The subsurface conditions are also shown in relation to the bridge alignment on the **Interpretive Subsurface Profiles, Figures 3 and 4**.

Soil Unit	Approximate Encountered Thickness (ft)	Generalized Description
Fill	0.7 to 16.8	Brown to grey, dry to wet, loose to very dense, Gravelly fine to coarse SAND, trace to some silt, to Silty SAND, trace gravel, with cobbles and boulders. (USCS: SM, GM, GP-GM). Typical MaineDOT Frost Classification Range= I to III - An 8-inch apparent cobble/boulder was encountered in boring BB-AI95-101 at 8.2 to 8.9 feet bgs. - A 3-foot apparent boulder was encountered in boring HB-AI95-201 at 17.0 to 20.0 feet bgs. <i>Unit encountered in all borings.</i>
Glacial Till	7.7 to 20.2	Variable ranging <u>from</u>: Brown and grey, very stiff to hard, Sandy SILT, little fine to coarse sand, trace gravel <u>to</u>: Brown and grey, dense to very dense, fine to coarse SAND, varying silt and gravel content. (USCS: SM, SP, ML, GM). <i>MaineDOT Frost Classification = III to IV</i> Apparent cobble/boulder layer BB-AI95-101 at 19 to 23.5 feet bgs. <i>Unit encountered in all borings drilled through approaches (BB-AI95-101, BB-AI95-103, HB-AI95-201, and HB-AI95-202).</i>
Estimated Top of Bedrock		Abutment 1: Approx. El. 212.8 to 207.1 (0.8 to 5.3 feet bgs) Pier: Approx. El. 207.8 to 203.1 (7.8 to 12.9 feet bgs) Abutment 2: Approx. El. 207.6 to 205.8 (5.5 to 5.7 feet bgs)

4.2.1 Bedrock

Bedrock was cored in each test boring and was described as Granite of the Hallowell Granite Intrusion in BB-AI95-101, -204, and HB-AI95-201, and/or Granofels of the Mayflower Hill Formation in all other borings, as summarized below.

²Marvinney, Robert G., and Barker, Daniel S., 2012, Bedrock Geology of the Augusta 7.5-minute quadrangle, Kennebec County, Maine: Maine Geological Survey, Department of Conservation, Open-File No. 12-36.



Granite was generally described as very hard to hard, fresh, aphanitic to very coarse grained, grey to white, with joints that are extremely close to moderately spaced, horizontal to high angle, smooth to rough, undulating, fresh to discolored, and very tight to open. The Rock Quality Designation (RQD), defined as the sum of the length of core pieces longer than 4 inches divided by the total length of the core run, ranged from 75 to 97 percent, indicating fair to excellent rock quality.

Granofels was generally described as very hard to hard, fresh to slightly weathered, aphanitic to medium grained, and grey to black. Joints in the Granofels were generally characterized as extremely close to widely spaced, horizontal to high angle, smooth to rough, planar to undulating, fresh to decomposed, and very tight to moderately wide. Generally, with the exception of the zone described in **Section 4.2.2**, the RQD ranged from 40 to 100 percent, indicating poor to excellent rock quality.

Unconfined compressive strength and unit weight tests were conducted on four samples of fresh rock, the results of which are summarized in the following table.

SUMMARY OF BEDROCK STRENGTH TEST RESULTS							
Boring	Depth below Existing Ground	Depth below Top of Rock	Elevation	Unconfined Compressive Strength	Secant Modulus @ 50% of Failure Stress	Unit Weight (pcf)	Rock Type
	(ft bgs)	(ft bgs)	(ft NAVD 88)	(psi)	(ksi)		
BB-AI95-101	24.6	1.1	213.9	12,119	1,520	163	Granite
BB-AI95-103	33.5	0.3	199.6	12,978	4,210	174	Granofels
BB-AI95-202	6.9	1.6	210.1	7,198	3,000	174	Granofels
BB-AI95-204	8.2	0.4	206.8	8,306	1,050	159.9	Granite
BB-AI95-205	5.6	0.1	207.4	14,148	2,520	172	Granofels

4.2.2 Highly Fractured Bedrock

In test boring BB-AI95-202, a highly fractured zone of granofels bedrock was encountered between 11.5 to 18.5 feet bgs, described as hard to soft, slightly to highly weathered, and aphanitic to fine grained, with 24 to 57 percent recovery. Based on these findings, a second adjacent boring, BB-AI95-202A, was drilled to facilitate the ATV/OTV survey. Similar conditions were encountered, with the highly fractured zone encountered between approximately 9.7 and 17.0 feet bgs. The RQD in this zone ranged from 0 to 31 percent, indicating very poor rock quality. Two significant highly weathered zones with wide apertures were interpreted from the borehole geophysical data for BB-AI95-202A:

- 10.0 to 11.7 feet – Discontinuities with apertures ranging from 0.3 to 3 inches, resulting in 10 inches of total aperture (approximately 49 percent of total fracture zone length).
- 15.4 to 16.8 feet – Discontinuities with apertures ranging from 0.2 to 1.7 inches, resulting in 4.7 inches of total aperture (approximately 27 percent of total fracture zone length).



4.2.3 Groundwater

Groundwater depth was measured in all borings. Groundwater depths ranged from approximately 0.0 to 12.0 feet, corresponding to approximately El. 203.9 to El. 229.1. Groundwater levels in the borings were measured during or immediately after drilling and may have been affected by cased drilling procedures, which included introduction of water for drilling purposes.

The groundwater observations were made at the times and under the conditions stated in the boring logs. Fluctuations in groundwater level occur due to variations in season, precipitation, 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 *2020 AASHTO LRFD Bridge Design Specifications, 9th Edition* (herein designated as AASHTO) and the *MaineDOT Bridge Design Guide, 2003 Edition*, with updates through 2018 (MaineDOT BDG). The sections that follow describe the evaluations and the geotechnical basis for each element. Supporting calculations are included in **Appendix F**.

5.2 APPROACH AND HIGHWAY EMBANKMENTS

The new bridge will generally follow the existing Western Avenue alignment. The roadway profile will be raised approximately 1 foot above existing grades in the vicinity of the bridge, but the infill between the existing abutments and new abutment locations will result in a triangular fill wedge with a maximum height of 29 feet. Approach embankment side slopes will be constructed with an inclination of 2 horizontal to 1 vertical (2H:1V) or less.

The infill between the existing and new abutments will be placed over a combination of bedrock, glacial till, and fill. These materials are anticipated to have low compressibility under the weight of the fill, and most of the settlement that does occur will occur as the fill is being placed. Considering that new embankment side slopes will be comprised of engineered fill and medium dense to dense glacial till, global stability is not considered to be a concern.

Temporary embankments are proposed to be contractor-designed. Stantec provided a feasible alignment and profile for a temporary three-lane bridge located to the north of the existing bridge. As envisioned, the alignment would require approximately 60 feet of embankment widening to carry the temporary roadway. We have assumed that the fill placed for the temporary detour will be removed at the end of construction. We also assumed the embankments would be constructed per MaineDOT standard specifications and details using engineered fill placed over the existing embankment/side slopes, including temporary embankment side slope inclinations of 2H:1V or flatter. Since this is a contractor-designed temporary bridge and approach, GZA did not conduct stability evaluations for



embankment slopes associated with the temporary roadway, and these evaluations should be conducted by the contractor.

5.3 EVALUATION OF FOUNDATION TYPES

5.3.1 Abutment and Pier Foundations

Assessment of the foundation types was influenced by thickness and strength of soil, and depth to and strength of the bedrock. Considering that the as-built plans indicated spread footings bearing on bedrock at depths of 10 feet or less for all of the piers, and the new abutment foundations would be built directly behind existing piers, spread footings bearing on bedrock were selected during preliminary design as the preferred foundation alternative for abutment foundations. Pier foundations are proposed involve reuse of the existing central pier footings.

Subsurface conditions relevant to abutment foundation type considerations are summarized in the table below. These data support selection of footings bearing on bedrock as the preferred foundation type.

Boring/Hand Probe	Station	Offset	Approximate Top of Rock Depth (ft)	Approximate Top of Rock Elevation (ft)	Substructure
BB-AI95-201	18+72	39.4' L	0.7	213.3	Abutment 1
BB-AI95-202	19+13	39.6'R	5.3	211.7	Abutment 1
BB-AI95-202A	19+12	30.3'R	5.3	211.7	Abutment 1
BB-AI95-102	19+81	6.50'L	7.7	207.8	Pier
BB-AI95-203	19+55	39.5' L	12.9	201.1	Pier
BB-AI95-204	20+08	39.4' R	7.8	207.2	Pier
BB-AI95-205	20+27	39.2'L	5.5	207.5	Abutment 2
BB-AI95-206	20+75	39.5' R	5.7	206.8	Abutment 2

We anticipate that the large apertures of the joints within fractured rock zones encountered in borings BB-AI95-202 and -202A could make the bedrock more compressible at these locations. As a precaution, it is our opinion that the fractures in bedrock should be mitigated to reduce the risk for unacceptable foundation settlement. Recommendations for spread footing design and bedrock preparation are provided in **Section 6**.

5.4 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 and are presented in the following table.



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

Notes:

1. Resistance factor for footing on rock was taken as equal to footing on sand.

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

5.5 SPREAD FOOTING DESIGN CONSIDERATIONS

5.5.1 Spread Footing on Bedrock

Nominal and factored bearing resistances were calculated for bedrock-bearing footings using the Rock Mass Rating- (RMR-) based empirical correlation presented in “Foundations on Rock,” by Duncan Wyllie. RMR was evaluated in accordance with Table 10.4.6.4-1 of the *2012 AASHTO LRFD Bridge Design Specifications, 6th Edition* (AASHTO). The current (9th) Edition of the *AASHTO Design Specifications* does not include the RMR formulation included in the previous version (6th Edition). However, Articles C10.4.6.4 and 10.6.3.2.1 of the 9th Edition refer to RMR-based design procedures for footings on rock, so the 6th Edition methodology was followed.

GZA developed a nominal bearing resistance for use in design incorporating the presence of fair quality bedrock, which is considered appropriate for fractured zones in borings BB-AI95-202 and -202A after mitigation. Five samples of bedrock core were submitted for unconfined compressive strength testing. Results ranged between 7,200 and 14,150 psi. The lowest strength was measured in rock from boring BB-AI95-202. Although this strength may have been influenced by bedrock structure, it was utilized for design due to its presence above the fractured rock zone. The bedrock properties used in the bearing resistance evaluation are presented below:

DESIGN BEDROCK PROPERTIES FOR BEARING RESISTANCE EVALUATION					
Rock Type	RQD (percent)	Unconfined Compressive Strength (ksi)	Rock Mass Rating (RMR)	m	s
Granofels and Granite	25 to 50	7.2	44	0.458	0.00009

Based on these parameters, we recommend a nominal bearing resistance of 79 ksf, resulting in a factored bearing resistance of 36 ksf for the strength limit state for both new and existing footings bearing on bedrock. A bearing resistance of 36 ksf can also be used for the service limit state. Supporting calculations are provided in **Appendix F**.

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



5.6 EXISTING ABUTMENT FOOTING EVALUATION

Based on the proposed construction sequence and available subsurface data, excavation of new abutment and wingwall footings will extend to depths of approximately 17 to 20 feet below existing abutment footings. GZA evaluated the bearing resistance of the existing abutment footings to assess temporary embankment geometry and its impact on bearing resistance. The objective was to assess if under the temporary slope condition the footing will have the design bearing capacity of 5.2 ksf noted on the as-built drawings. The analyses used the theoretical method (Munfakh et al., 2001) based on SPT data, as referenced in AASHTO LRFD Articles 10.6.3.1.1 and 10.6.3.1.2. A reduction of the bearing resistance was applied considering the proximity of the temporary slope based on the methodology presented in AASHTO LRFD Section C10.6.3.1.2c.

The results showed that a factored bearing resistance of 5.2 ksf could be achieved by providing 6.75 feet of lateral embedment from the front bottom edge of the footing to the cut slope and using a 1.5H:1V inclination on the slope below the footing. Supporting calculations are provided in **Appendix F**. For practical purposes, the temporary excavation below existing footing bearing level should be sloped at an inclination of 1.5H:1V or flatter. In addition, the cut slope should be at least 7 feet laterally in front of the bottom edge of the footing, as shown in **Figure A** below. It appears that this configuration will allow excavation near, or to the top of, rock elevations at the footing locations. If this is not feasible, excavation support should be used to maintain the required lateral embedment of the existing footings.

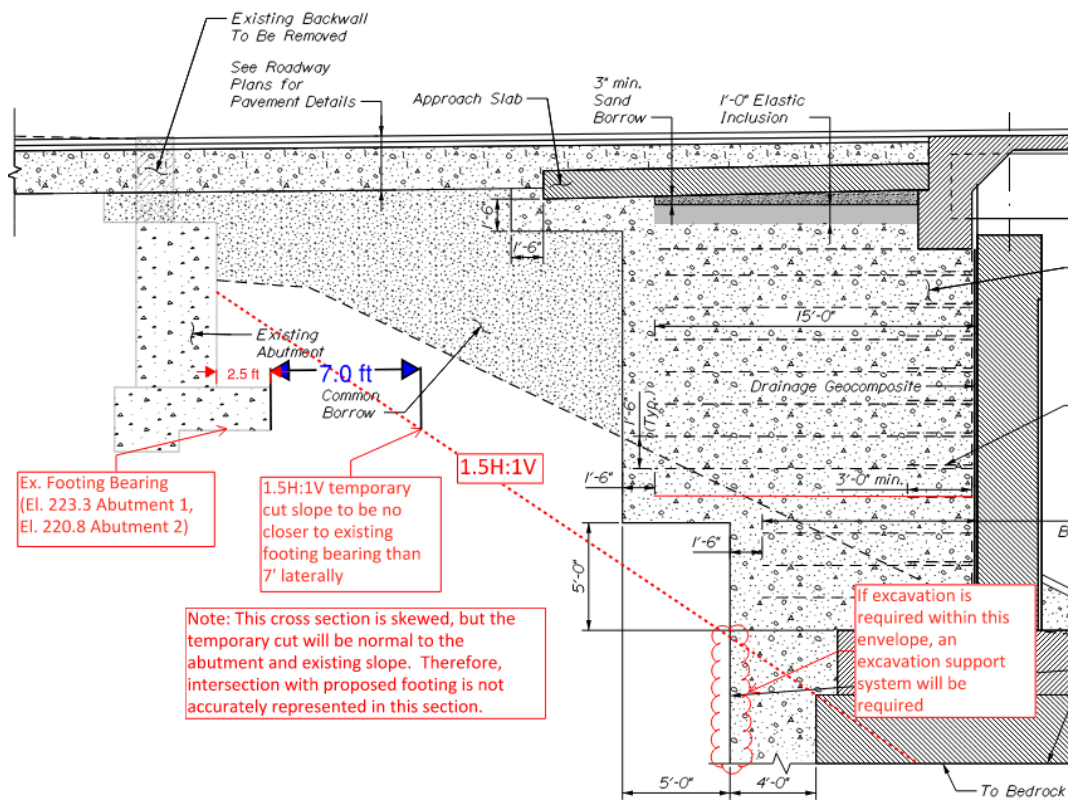


Figure A: Typical Abutment Section: Excavation Limitations



5.7 LATERAL EARTH PRESSURE

The new abutments will be constructed and backfilled underneath the active Western Avenue bridge to shorten the bridge construction schedule and reduce traffic impacts. It is desired to backfill behind the new abutments using a fill that will minimize lateral earth pressure, thereby reducing the length of the heel limiting necessary excavation and limiting the risk of undermining existing foundations, as discussed in **Section 5.6**. Three alternatives were evaluated during preliminary design including: 1) Geosynthetic Reinforced Soil (GRS) (reinforced backfill); (2) Low-Density Cellular Concrete (LDCC); and (3) Ultra-Lightweight foamed glass aggregate (ULFGA). ULFGA alone does not reduce lateral earth pressure as much as GRS and it is significantly more expensive. LDCC could provide a lower lateral earth pressure than GRS, but it is more expensive and logistically more challenging to produce and place within a relatively small work area than either of the other two alternatives. Therefore, the preferred alternative selected for final design is the GRS backfill and is discussed in the following section.

Additionally, the construction phases shown on the plans indicate that a portion of the northern wingwalls will not be constructed until after the existing footings are taken out of service. Consequently, this portion of the wingwalls may be constructed as a full-height cantilever wall with conventional backfill materials.

5.7.1 Geosynthetic Reinforced Soil Backfill

GZA conducted calculations to evaluate earth pressure on the abutments from the GRS backfill in accordance with methodology presented in "Synthesis of Geosynthetic Reinforced Soil Design Topics" (FHWA-HRT-14-094, 2015) and "Geosynthetic Reinforced Soil Integrated Bridge System Synthesis Report" (FHWA-HRT-11-027 2011).

The loading acting on the backwall includes horizontal earth pressure components resulting from the GRS backfill, a 250-psf traffic surcharge load applied at the ground surface, and the dead load of the buried approach slab and fill and pavement above it. The proposed buried approach slab is 20 feet long and 16 inches thick. Loads were factored for strength limit case conditions.

GZA conducted initial calculations that indicated the horizontal surcharge pressure resulting from the approach slab self-weight and the fill and live load above would result in undesirable lateral earth pressure even with the GRS backfill. GZA evaluated elasticized expanded polystyrene (EPS) as a compressible inclusion below the portion of the approach slab closest to the abutment to effectively move the bearing area of the approach slab behind the EPS and away from the back of the abutment, reducing the lateral surcharge on the abutment. Based on discussions with Stantec, we assumed that half of the vertical load of approach slab concrete, overlying soil/pavement and live loads would be applied directly to the abutment at the haunch for the slab, and the other half would primarily be applied to the soil subgrade behind the elasticized EPS.

The approach slab length selected to reduce the surcharge loading is 20 feet, considering the first 15 feet from the backwall being underlain by 12 inches of EPS. We evaluated an 18-inch vertical spacing between geosynthetic layers. Based on our evaluations of this configuration, a uniform horizontal earth pressure of 170 psf may be used to design the abutment and wingwall footings that utilize GRS backfill



for the strength load case. Required reinforcement length and properties and backfill type are presented in **Section 6.3**. Supporting calculations are provided in **Appendix F**.

5.7.2 Conventional Backfill (Phase 3 wingwall portions only)

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.

Based on Article 3.6.4 of the MaineDOT BDG, we conclude active earth pressure is appropriate for design of the wingwalls. Based on Figure C3.11.5.3-1 of LRFD, the walls are considered to be short-heeled walls. 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.

Design lateral earth pressure recommendations are provided in **Section 6.1.5** of this report.

5.8 SEISMIC DESIGN CONSIDERATIONS

The subsurface profile for seismic design includes the approach fills (including backfill behind abutments) and underlying glacial till overlying bedrock. The overlying soils can be ignored from the site classification since the foundations are to be placed directly on bedrock. Therefore, the bridge is assigned to Site Class B, weak rock.

Since foundations will be bearing on bedrock, liquefaction is not considered an issue for this site.

5.9 FROST PROTECTION

Fill soils are anticipated to be present at the abutments, either as existing fill or imported backfill. Based on the MaineDOT BDG, Section 5.2.1, the Freezing Index for the site is 1,615, and with fill (moisture content of approximately 20 percent) subgrade soils, the estimated depth of frost penetration is 5.9 feet. Where abutment foundations bear directly on sound rock, there is no minimum requirement for footing embedment, provided that proper drainage is in place.

5.10 SIGNAL MAST ARM FOUNDATIONS

Two new signal mast arms are proposed near the Whitten Road intersection. Borings HB-AI95-201 and HB-AI95-202 were drilled in the vicinity of the proposed foundations located at Sta. 20+85.7, 44.6 feet left and Sta. 21+68.4, 53.1 feet right, respectively. Drilled shafts are the preferred foundation type for the two supports. GZA developed soil profiles and estimated unit side friction resistances for use in shaft design by others. The recommended soil profiles and parameters are summarized for each foundation in the tables below.



SIGNAL MAST ARM FOUNDATION DESIGN PARAMETERS: Station 20+85.7, 44.6' Left (HB-AI95-201)							
Stratum	Soil Model	Top of Layer Elevation (NAVD88 ft)	Average N_{60}	k (pci)	ϕ' (deg)	Nominal Unit Side Resistance (ksf)	γ_e (pcf)
Existing Fill*	Reese Sand	237	19	102	33	0.35	120
Existing Fill (Saturated)**	Reese Sand	224.9	19	65	33	0.35	57.6
Glacial Till	Reese Sand	222.4	51	120	38	0.9	67.6
Top of Rock ⁷	Weak Rock	214.7	--	--	--	--	--

SIGNAL MAST ARM FOUNDATION DESIGN PARAMETERS: Station 21+68.4, 53.1' Right (HB-AI95-202)							
Stratum	Soil Model	Top of Layer Elevation (NAVD88 ft)	Average N_{60}	k (pci)	ϕ' (deg)	Nominal Unit Side Resistance (ksf)	γ_e (pcf)
Existing Fill*	Reese Sand	233	19	102	33	0.35	120
Existing Fill (Saturated)**	Reese Sand	220.4	19	65	33	0.35	57.6
Glacial Till	Reese Sand	213.1	51	120	38	0.9	67.6
Top of Rock ⁷	Weak Rock	< 202.1	--	--	--	--	--

Notes:

- *Indicates the top of foundation elevation per plans.
- **Indicates the top of the layer is the approximate ground water elevation based on borings.
- pci = pounds per cubic inch, deg = degrees, ksf = kips per square foot,
- γ_T = total unit weight (used above anticipated groundwater level),
- γ_e = effective unit weight (used below anticipated groundwater), pcf = pounds per cubic foot.
- Factored side resistance for strength condition may be calculated using a resistance factor $\phi = 0.55$ for compression and $\phi = 0.40$ for tension with no testing, per AASHTO Table 10.5.5.2.4-1
- For design purposes, assume Glacial Till Parameters if required length in soil is deeper than the top of rock.

6.0 RECOMMENDATIONS

6.1 EMBANKMENT DESIGN CONSIDERATIONS

Embankment side slopes should be designed with MaineDOT-typical slope angles of 2H:1V or flatter. Earth slopes should be provided with loam and seed for permanent erosion protection.

6.2 SEISMIC DESIGN

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



SITE CLASS B SEISMIC DESIGN PARAMETERS	
Parameter	Design Value
F _{pga}	1
F _a	1
F _v	1
A _s (Period = 0.0 sec)	0.08
S _{Ds} (Period = 0.2 sec)	0.16
S _{D1} (Period = 1.0 sec)	0.04

Per AASHTO Article 4.7.4.2, bridges in Seismic Zone 1 need not be analyzed for seismic loads, but for the minimum requirements for superstructure connections and support lengths as specified in AASHTO Articles 4.7.4.4 and 3.10.9.

6.3 GEOSYNTHETIC REINFORCED SOIL BACKFILL ABUTMENTS AND WINGWALLS

The GRS will consist of open-graded fill reinforced by woven polypropylene geotextile behind the cast-in-place full-height abutments and southern (Phase 1) wingwalls, and above the proposed footings bearing on bedrock. Recommendations for design and construction of GRS backfill and abutments are presented below.

- Abutments and wingwalls that are backfilled with GRS as recommended below may be designed using a uniform lateral earth pressure of 170 psf. This incorporates a 250-psf live load surcharge on the pavement and load factors associated with the strength load case.
- A 20-foot-long approach slab should be used, with the first 15 feet behind the back of the abutment underlain by a 12-inch-thick elasticized EPS layer. The elasticized EPS should have a compressive strength of 720 psf plus or minus 60 psf at 10 percent strain and should have linear-elastic stress strain behavior up to 10 percent strain. A 3-inch-minimum layer meeting the MaineDOT Specification 703.22, Underdrain Backfill Material, Type B should be placed between the approach slab and EPS.
- Geosynthetic reinforcement should be spaced vertically at a maximum of 18 inches and should consist of biaxial woven polypropylene geotextile with an ultimate reinforcement strength of 3,600 pounds per linear foot (plf) and a reinforcement strength of 540 plf at two percent strain. For Mirafi HP370 geotextile, this requires the material to be oriented in the machine direction.
- Reinforced soil placed within the reinforced zone shown on the plans shall meet the gradation requirements of MaineDOT Specification 703.02, Coarse Aggregate for Concrete, Grading A or S.
- Reinforced soil shall be placed and compacted in accordance with the requirements of SP 203, Open Graded Backfill and SP 635, Geosynthetic Reinforced Soil Wall. Hand-operated compaction equipment is required within 2 feet of the back of the wall face.
- Special Provision Section 203 – Excavation and Embankment (Elastic Inclusion for approach slab) will be included in the contract documents and contains additional material requirements as well as required submittals, independent testing, and placement.



- Geosynthetic reinforcement should be a continuous length wrapped up along the backside of the wall and overlap a minimum of 3 feet horizontally beneath the overlying layer.
- Geosynthetic reinforcement embedment lengths should be at least 10 feet for the lower 6 feet above the top of footing and 15 feet from that level to the top of the abutment.
- Abutments and wingwalls should each have full-length geosynthetic layers in accordance with these recommendations; they should not share the same geosynthetic layers at the intersection between abutment and wingwall. The abutment and wingwall geosynthetic layers should be offset by a minimum of 6 inches vertically.

6.4 PHASE 3 WINGWALL DESIGN (CONVENTIONAL BACKFILL)

- Backfill between wingwalls with conventional backfill 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, MaineDOT BDG Type 4 soil. Open graded backfill may also be acceptable as wingwall backfill material if desired by the contractor and approved by the GRS designer in order to use a single material type. 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 wingwalls without GRS)
- 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 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.

6.5 RECOMMENDATIONS FOR FOUNDATIONS

6.5.1 Pressure Grouting

We recommend mitigating the highly fractured bedrock zones beneath the northern half of Abutment 1 footings by conducting pressure grouting. The pressure grouting is intended to fill apertures in the bedrock and improve the rock quality to be consistent with the intact rock encountered through the remainder of the site.

- We recommend drilling 3-inch-minimum diameter holes to a depth of 20 feet below the top of bedrock or 1 foot into competent rock based on drill characteristics, whichever is greater, on a 5-foot by 5-foot square grid pattern within the footprint of the northern half of Abutment 1 footing and the northeastern wingwall.



- Grout should initially be placed under gravity until the grout level is stable for 5 minutes (dropping less than 1 inch within that time). The grout volume placed in the hole under gravity should be recorded. A packer should then be placed over the full rock depth of the drill hole for grouting, and the grout should be pressurized to 50 psi with the packer, and additional grout should be added if required until the grout level can be maintained under pressure. The additional grout volume with the packer should also be recorded.
- A highly flowable grout mixture is recommended, with a minimum 28-day compressive strength of 2,000 psi and a minimum water-to-cement ratio of 0.8:1.

6.5.2 Spread Footing Design

- The proposed abutments and pier should be supported on spread footing foundations bearing on sound, intact bedrock. Footings designed to bear on intact bedrock should be designed using a nominal bearing resistance, q_n , of 79 ksf. At the strength limit state, footings should be designed for a maximum factored bearing resistance of 35 ksf. A bearing resistance of 35 ksf should be used for service limit state design.
- Spread footings founded on bedrock should be checked for eccentricity with AASHTO Article 10.6.3.3. Eccentricity of the footing reaction at the strength limit state should be limited such that the resultant reaction on the base of the footing is no further than 0.45 B from the centerline of the footing, where B is the footing width perpendicular to the axis of rotation.
- For proposed foundations bearing on bedrock, we recommend that sliding resistance be assessed using a nominal sliding resistance coefficient ($C \tan \phi_i$) equal to 0.7, a resistance factor (ϕ_i) of 0.8, and a factored sliding resistance coefficient of 0.56 for the Strength Limit State.
- The bedrock surface should be cleaned of loose soil or rock at the time of concrete placement for subfooting concrete or the footing. Bearing surface preparation should be in accordance with **Section 7.3**.
- The following table summarizes the top of bedrock elevations encountered in the borings. These data are provided to assist the designer in developing bottom-of-footing elevations for the abutments.

ESTIMATED BEDROCK LEVELS FOR FOOTING DESIGN	
Foundation Element	Estimated Range in Bedrock Elevation (feet, NAVD 88)
Abutment 1	El. 207 to 213
Abutment 2	El. 205 to 208

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 estimated levels. Some construction-phase engineering should be anticipated to address the potential variability of the encountered conditions.



- If the bedrock level extends above the design bottom of footing elevation, the footing may be raised and vertical reinforcement shortened in the wall, subject to review and approval by the Designer to limit over-excavation of bedrock.
- If after cleaning the exposed bedrock surface is below the design footing bearing level, fill concrete may be placed up to the bottom of footing level.
- Anchoring, doweling, benching or other means of improving sliding resistance is recommended at locations where the prepared bedrock surface is steeper than 4H:1V in any direction.
- Rock dowels may be used to supplement the footing sliding resistance. If used, the dowels should be grouted a minimum of 2 feet into intact bedrock and embedded at least 1.5 feet into concrete. The unconfined compressive strength of the bedrock should be assumed to be 4.0 ksi for design of rock dowels.
- Dowels should be grouted with a cementitious grout on the MaineDOT Qualified Products List of Grout Materials for Keyways and Anchoring (pre-qualified for anchoring). Epoxy grout should not be used.
- Since the footings will be founded on bedrock, there is no minimum embedment required for frost protection per MaineDOT BDG Article 5.2.1, but we recommend a 2-foot minimum embedment below finish grade surrounding the footing.

7.0 CONSTRUCTION CONSIDERATIONS

This section provides guidance regarding excavation, dewatering, and foundation subgrade preparation and protection.

7.1 EXCAVATION LIMITATION BELOW EXISTING ABUTMENTS

As discussed in **Section 5.6**, the results of our evaluations indicate that a 1.5H:1V temporary cut in front of the existing abutment footings should be located at least 7 feet horizontally from the bottom front corner of the footing. This corresponds to El. 223.3 and El. 221.8 at existing Abutment 1 and Abutment 2, respectively. The plans and/or a special provision should be developed to inform the contractor of this requirement. If excavation is required within this envelope, an excavation support system will be required. The contractor should provide a submittal stipulating the details for this system. If the excavation support is required to be greater than 3 feet in height, a design should be developed that considers possible surcharge loading from existing foundations and should be stamped by a registered Maine professional engineer.

7.2 EXCAVATION, TEMPORARY LATERAL SUPPORT AND DEWATERING

It is our understanding that traffic will be carried on the existing bridge during construction of the footings and most of the abutment backwalls. The Remainder of construction will occur after traffic is transferred to a temporary detour bridge to the north. Other than the condition above, related to protection of existing bridge footings, it is anticipated that the excavations for the new abutment footings could consist of sloped, open cuts. Sheetpile support may be needed at the approaches for the



temporary bridge depending on grade change requirements. In all cases, temporary excavations should comply with Occupational Safety and Health Administration excavation safety requirements.

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

7.3 SUBGRADE PREPARATION

We anticipate it will be feasible to complete final bedrock subgrade preparation in-the-dry. The bedrock surface is known to be variable in terms of elevation, slope and localized weathering. Conventional excavation equipment is anticipated to be sufficient to complete excavations. All soil and loose, decomposed, highly weathered and fractured bedrock should be removed from the footing bearing surface using pressurized air and water prior to placement of subfootings or footings. Care should be taken to avoid removing bedrock that supports existing foundations since the bridge will still be in use.

The prepared bearing surfaces should be observed by the geotechnical engineer prior to placing concrete. If highly weathered zones are identified in the exposed bedrock surface at Abutment 1, the Geotechnical Engineer should evaluate corrective measures. The Geotechnical Engineer and Designer should also be provided cross-sections showing the prepared rock surface geometry prior to placement of concrete to evaluate whether benching, doweling, or subfooting concrete fill is needed for that foundation location. If the exposed bedrock surface is steeper than 4H:1V, then anchoring, doweling, benching or other means should be employed to improve sliding resistance.

7.4 REUSE OF ON-SITE MATERIALS

Based on the test boring results, one of the fifteen fill samples tested had less than 10 percent passing the No. 200 sieve, indicating the fill generally will not meet MaineDOT specifications for Granular Borrow and/or Granular Borrow for Underwater Backfill. The material is considered suitable for use as Common Borrow.

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



05/19/2023

GEOTECHNICAL DESIGN REPORT
WESTERN AVENUE BRIDGE NO. 5808 OVER INTERSTATE 95
Maine Department of Transportation
09.0025983.01

TABLES



TABLE 1
Summary of Subsurface Explorations
Western Ave Bridge No. 5808 Over Interstate 95
Augusta, ME
MaineDOT WIN 21672.00

Boring ID	Station	Offset	Northing, Easting	Ground Surface El. (ft)	Top of Stratum Elevation (ft)				Stratum Thickness (ft)			Depth to Top of Bedrock (ft)	Top of Bedrock Elevation (ft)	Bottom of Boring Depth (ft)	Bottom of Boring El. (ft)	Groundwater	
					Pavement	Fill	Glacial Till	Possible Bedrock	Pavement	Fill	Glacial Till					El. (ft)	Depth (ft)
BB-AI95-101	18+16.4	8.2' L	541081.7, 1131867.4	238.5	238.5	238	229.6	215.0	0.5	8.4	14.6	23.5	215.0	34.6	203.9	226.6	11.9
BB-AI95-102	19+81.4	6.50' L	541102.7, 1131703.4	215.5	NE	215.5	NE	207.8	NE	7.7	NE	7.7	207.8	17.4	198.1	211.5	4.0
BB-AI95-103	21+32.1	34.7' R	541173.2, 1131565.4	233.1	233.1	232.6	220.1	199.9	0.5	12.5	20.2	33.2	199.9	43.3	189.8	229.1	4.0
BB-AI95-201	18+71.9	39.2' L	541056.4, 1131809.1	213.5	NE	213.5	NE	212.8	NE	0.7	NE	0.8	212.7	11.0	202.5	NE	NE
BB-AI95-202	19+11.5	39.4' R	541138.7, 1131778.2	212.4	NE	212.4	NE	207.1	NE	5.3	NE	5.3	207.1	23.5	188.9	211.6	0.8
BB-AI95-202A	19+11.9	30.3' R	541129.6, 1131778.1	212.4	NE	212.4	NE	207.1	NE	5.3	NE	5.3	207.1	22.0	190.4	212.4	0.0
BB-AI95-203	19+54.0	39.2' L	541066.2, 1131726	216.0	NE	216.0	NE	203.1	NE	12.9	NE	12.9	203.1	23.0	193.0	205.9	10.1
BB-AI95-204	20+08.1	39.0' R	541151.9, 1131684.8	214.9	214.9	214.7	NE	207.1	0.2	7.6	NE	7.8	207.1	17.4	197.5	--	NM
BB-AI95-205	20+28.1	39.1' L	541078.7, 1131651.2	213.1	NE	213.1	NE	207.6	NE	5.5	NE	5.5	207.6	15.5	197.6	209.1	4.0
BB-AI95-206	20+73.3	39.3' R	541164.5, 1131622.4	211.5	NE	211.5	NE	205.8	NE	5.7	NE	5.7	205.8	16.0	195.5	208.9	2.6
HB-AI95-201	20+88.6	48.2' L	541082.4, 1131588.5	237.4	NE	237.4	222.4	214.7	NE	15.0	7.7	22.7	214.7	27.8	209.6	224.9	12.5
HB-AI95-202	21+81.0	35.4' R	541186.4, 1131519.5	233.1	233.1	232.9	213.1	NE	0.2	16.8	>11	NE	NE	31.0	202.1	NE	NE

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

Notes:

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



TABLE 2
Summary of Bedrock Data
Western Avenue Bridge No. 5808 over Interstate 95
Augusta, Maine
MaineDOT WIN 21672.00

Boring ID	Core Run	Ground Surface Elevation (ft)	Depth of Core Run below Ground Surface (ft)			Depth to Rock (ft)	Depth (ft) Below Top of Rock			Length of Core Run (ft)	Rec (in)	Rec (%)	RQD (in)	RQD %	Joint Spacing (in)	Joint Aperture (in)	Elevation (ft)		LABORATORY TESTING						Rock Type
			Top		Bottom		Top		Bottom								Top	Bottom	Depth of Sample (ft)	Depth of Sample into Rock (ft)	Elev Top of Sample (ft)	UCS (psi)	Modulus (ksi)	Unit Wt (pcf)	
BB-AI95-101	R1	238.5	24.6	-	29.6	23.5	1.1	-	6.1	5.0	50	83%	45	75%	0.75-24	0.02-0.1	213.9	208.9	24.6	1.1	213.9	12,119	1,520	163	Granite
BB-AI95-101	R2	238.5	29.6	-	34.6	23.5	6.1	-	11.1	5.0	53	88%	53	88%	24	0.01-0.1	208.9	203.9							Granite
BB-AI95-102	R1	215.5	7.7	-	12.7	7.7	0.0	-	5.0	5.0	59	98%	57	95%	2.5-80	0.01-0.1	207.8	202.8							Granofels
BB-AI95-102	R2	215.5	12.7	-	17.4	7.7	5.0	-	9.7	4.7	57	101%	51	91%	2.5-80	0.02-0.1	202.8	198.1							Granofels
BB-AI95-103	R1	233.1	33.3	-	38.3	33.2	0.1	-	5.1	5.0	60	100%	60	100%	2.5-80	0.01-0.02	199.8	194.8	33.5	0.3	199.6	12,978	4,210	174	Granofels
BB-AI95-103	R2	233.1	38.3	-	43.3	33.2	5.1	-	10.1	5.0	52	87%	35	58%	0.75-24	0.004-0.1	194.8	189.8							Granofels
BB-AI95-201	R1	213.5	1.0	-	6.0	0.7	0.3	-	5.3	5.0	60	100%	60	100%	N/A	0.1-0.4	212.5	207.5							Granofels
BB-AI95-201	R2	213.5	6.0	-	11.0	0.8	5.2	-	10.2	5.0	60	100%	60	100%	N/A	0.01-0.1	207.5	202.5							Granofels
BB-AI95-202	R1	212.4	5.5	-	10.5	5.3	0.2	-	5.2	5.0	60	100%	46	77%	<0.75-24	0.004-0.1	206.9	201.9	6.9	1.6	205.5	7,198	3,000	174	Granofels
BB-AI95-202	R2	212.4	10.5	-	13.2	5.3	5.2	-	7.9	2.7	16	48%	10	31%	8	0.004-0.01	201.9	199.2							Granofels
BB-AI95-202	R3	212.4	13.2	-	16.0	5.3	7.9	-	10.7	2.8	8	24%	0	0%	N/A	N/A	199.2	196.4							Granofels
BB-AI95-202	R4	212.4	16.0	-	18.5	5.3	10.7	-	13.2	2.5	17	57%	0	0%	<0.75-8	0.004-0.4	196.4	193.9							Granofels
BB-AI95-202	R5	212.4	18.5	-	23.5	5.3	13.2	-	18.2	5.0	45	75%	45	75%	N/A	0.004-0.01	193.9	188.9							Granofels
BB-AI95-202A	R1	212.4	6.0	-	11.0	5.3	0.7	-	5.7	5.0	52	87%	40	67%	2.5-24	0.004-0.1	206.4	201.4							Granofels
BB-AI95-202A	R2	212.4	11.0	-	14.4	5.3	5.7	-	9.1	3.4	10	24%	0	0%	<0.75-8	0.01-4	201.4	198.0							Granofels
BB-AI95-202A	R3	212.4	14.4	-	17.0	5.3	9.1	-	11.7	2.6	21	68%	6	19%	N/A	N/A	198.0	195.4							Granofels
BB-AI95-202A	R4	212.4	17.0	-	22.0	5.3	11.7	-	16.7	5.0	60	100%	55	92%	0.75-24	<0.004-0.1	195.4	190.4							Granofels
BB-AI95-203	R1	216.0	13.0	-	18.0	12.9	0.1	-	5.1	5.0	60	100%	24	40%	<0.75-24	<0.004-0.01	203.0	198.0							Granofels
BB-AI95-203	R2	216.0	18.0	-	23.0	12.9	5.1	-	10.1	5.0	60	100%	57	95%	2.5-24	<0.004	198.0	193.0							Granofels
BB-AI95-204	R1	214.9	7.8	-	12.4	7.8	0.0	-	4.6	4.6	52	95%	41	75%	<0.75-8	<0.004-0.01	207.1	202.5	8.2	0.4	206.7	8,306	1,050	159.9	Granite
BB-AI95-204	R2	214.9	12.4	-	17.4	7.8	4.6	-	9.6	5.0	60	100%	56	93%	2.5-24	<0.004-0.1	202.5	197.5							Granite
BB-AI95-205	R1	213.1	5.5	-	10.5	5.5	0.0	-	5.0	5.0	58	97%	55	91%	<0.75-24	0.004-0.1	207.6	202.6	5.6	0.1	207.5	14,148	2,520	172	Granofels
BB-AI95-205	R2	213.1	10.5	-	15.5	5.5	5.0	-	10.0	5.0	60	100%	59	99%	2.5-24	<0.004-0.1	202.6	197.6							Granofels
BB-AI95-206	R1	211.5	6.0	-	11.0	5.7	0.3	-	5.3	5.0	60	100%	54	90%	0.75-24	<0.004-0.1	205.5	200.5							Granofels
BB-AI95-206	R2	211.5	11.0	-	16.0	5.7	5.3	-	10.3	5.0	60	100%	58	97%	2.5-24	<0.004-0.1	200.5	195.5							Granofels
HB-AI95-201	R1	237.4	22.8	-	27.8	22.7	0.1	-	5.1	5.0	60	100%	58	97%	24	0.004-0.1	214.6	209.6							Granite

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.
4. Highlighted cells indicate highly fractured zones encountered in Borings BB-AI95-202/202A, refer to boring logs for additional information. Refer to Appendix C for the results of downhole geophysical testing in BB-AI95-202A.



05/19/2023

GEOTECHNICAL DESIGN REPORT
WESTERN AVENUE BRIDGE NO. 5808 OVER INTERSTATE 95
Maine Department of Transportation
09.0025983.01

FIGURES

Division: Bridge
Filename: ...\\00X BLP\\00X BLP.dgn

Date: 5/15/2023
Username: BMC

WESTERN AVENUE BRIDGE
MAINEDOT WIN 21672.00
AUGUSTA, ME

BORING LOCATION PLAN

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

PREPARED FOR:

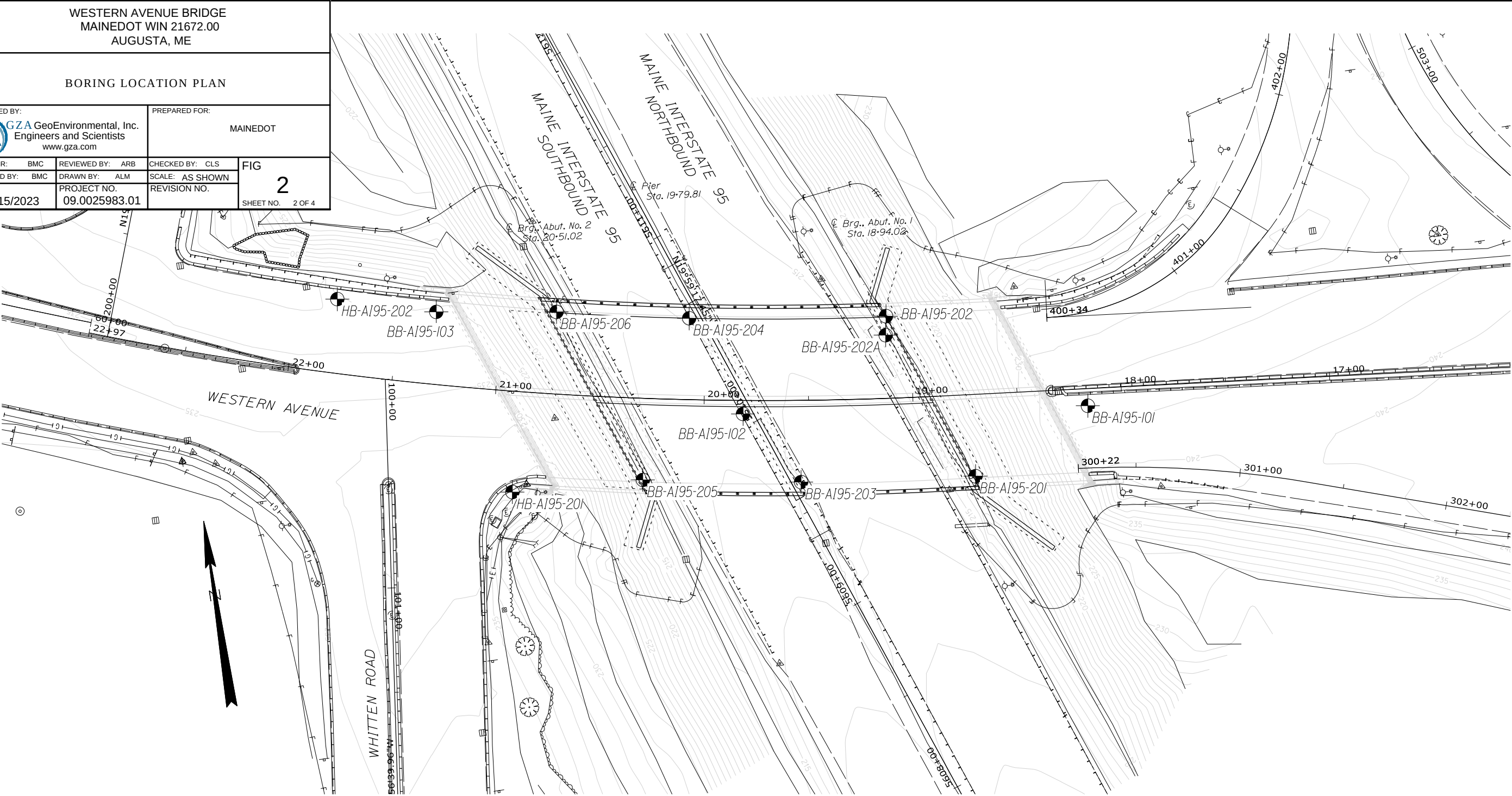
MAINEDOT

PROJ MGR: BMC
DESIGNED BY: BMC
DATE: 05/15/2023

REVIEWED BY: ARB
DRAWN BY: ALM
PROJECT NO: 09.0025983.01

CHECKED BY: CLS
SCALE: AS SHOWN
REVISION NO:

FIG
2
SHEET NO. 2 OF 4

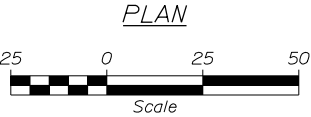


NOTES

- 1) Base map developed from electronic files (alignments.dgn, 3Dcontours.dgn, bridge.dgn, bridge_exisiting.dgn, and BDPLAN.dgn) provided by Stantec on January 31, 2023.
- 2) The as-drilled locations of the test borings were determined by survey conducted by MainedOT and supplied to GZA. This data should be considered accurate only to the degree implied by the method used.
- 3) The as-drilled locations of the BB-AI95-202A test boring was determined by field ties collected by GZA during the field exploration program. This data should be considered accurate only to the degree implied by the method used.

BORING LOCATION PLAN LEGEND

-  BB-AI95-100 Indicates 100-series bridge borings performed by New England Boring Contractors of Hermon, Maine between July 16, and 18, 2018 and observed by GZA personnel.
-  BB-AI95-200 Indicates 200-series bridge borings performed by New England Boring Contractors of Hermon, Maine between October 30, 2022 and November 2, 2022 and observed by GZA personnel.
-  HB-AI95-200 Indicates 200-series highway borings performed by New England Boring Contractors of Hermon, Maine between November 2, and 3, 2022 and observed by GZA personnel.



PREPARED BY:



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2167200

WIN
21672.00

BRIDGE NO. 5808

BRIDGE PLANS

WESTERN AVENUE BRIDGE NO.5808
OVER INTERSTATE 95
AUGUSTA, ME
KENNEBEC COUNTY

BORING LOCATION PLAN

SHEET NUMBER

14
of 178

Division: Bridge
Filename: ...\\00X ISP\\00X_ISP_south.dgn

Date: 5/15/2023
Username: BMC

WESTERN AVENUE BRIDGE
MAINEDOT WIN 21672.00
AUGUSTA, ME

INTERPRETIVE SUBSURFACE PROFILE 2

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

PREPARED FOR:

MAINEDOT

PROJ MGR: BMC
DESIGNED BY: BMC
DATE: 05/15/2023

REVIEWED BY: ARB
DRAWN BY: ALM
PROJECT NO. 09.0025983.01

CHECKED BY: CLS
SCALE: AS SHOWN
REVISION NO.

FIG
4
SHEET NO. 4 OF 4

PROFILE
Along the South Bridge Fascia
(Approximately 39.6 feet Left of Centerline)

NOTES

1) Base map developed from electronic files provided by Stantec on April 27, 2023 (File included Profile_geotech.dgn)

2) The as-drilled locations of the BB-A195-100 series test borings were determined by survey conducted by MaineDOT and supplied to GZA.

3) The as-drilled locations of the BB-A195-200 and HB-A195-200 series test borings were determined by survey conducted by MaineDOT and supplied to GZA.

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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
2167200
WIN
21672.00
Bridge NO. 5808
BRIDGE PLANS

DATE
MAY 2023
SIGNATURE
P.E. NUMBER
DATE

DATE
MAY 2023
SIGNATURE
P.E. NUMBER
DATE

PROJECT MANAGER
D. EATON
DESIGN-DETAILED
CHECKED-REVIEWED
DESIGN-DETAILED
DESIGN-DETAILED
REVISIONS 1
REVISIONS 2
REVISIONS 3
REVISIONS 4
FIELD CHANGES

WESTERN AVENUE BRIDGE NO.5808
OVER INTERSTATE 95
AUGUSTA, ME
KENNEBEC COUNTY
INTERPRETIVE SUBSURFACE PROFILE
SOUTH FASCIA

PREPARED BY:


SHEET NUMBER
16
of 178



05/19/2023

GEOTECHNICAL DESIGN REPORT
WESTERN AVENUE BRIDGE NO. 5808 OVER INTERSTATE 95
Maine Department of Transportation
09.0025983.01

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.



05/19/2023

GEOTECHNICAL DESIGN REPORT
WESTERN AVENUE BRIDGE NO. 5808 OVER INTERSTATE 95
Maine Department of Transportation
09.0025983.01

APPENDIX B – TEST BORING LOGS

January 2020

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Location: Augusta, Maine

WIN: 21672.00

Driller:	New England Boring Contractors	Elevation (ft.):	238.5	Auger ID/OD:	4.25" OD
Operator:	Mike Porter	Datum:	NAVD 88	Sampler:	Standard Splitspoon
Logged By:	Blaine Cardali	Rig Type:	Truck Rig	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	7/17/18-7/18/18	Drilling Method:	Drive & Wash	Core Barrel:	NX2
Boring Location:	Sta. 18+16.4, Offset 8.2' L	Casing ID/OD:	4"/4.5", 3"/3.5"	Water Level*:	11.9'
Hammer Efficiency Factor: 0.937		Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐			
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt		R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person S _U = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _{U(lab)} = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N _{uncorrected} = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N _{uncorrected} T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test			

Sample Information									Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)			
0	1D	24/17	0.5 - 2.5	27-24-25-25	49	77	SSA	238.0		0'-0.5': Asphalt.	G#1 A-1-b, SM WC=3.4
										Brown, dry, very dense, Gravelly fine to coarse SAND, little silt, (Fill).	
										Brown, dry, dense, fine to coarse SAND, little gravel, trace silt, (Fill).	
5	2D	24/10	4.0 - 6.0	14-14-10-6	24	37	22				
							44				
							18				
							37				
							22/2" R/C	230.3			
							AHEAD	229.6		Apparent boulder from 8.2'-8.9' bgs based on drill action.	
10	3D	24/10	10.0 - 12.0	17-16-17-15	33	52				Brown, wet, hard, Clayey SILT, some fine to coarse sand, trace gravel, (Glacial Till).	G#2 A-4, ML WC=19.4
15	4D	24/13	15.0 - 17.0	34-66-24-26	90	141				Brown, wet, very dense, Gravelly fine to coarse SAND, some silt, (Glacial Till).	
20							NX2	219.5		Increased roller bit resistance at 19.0' bgs. Advance roller bit to 19.6' and set up to core. Broke through boulder at 21.3' bgs. Roller cone encountered increased resistance at 23.5'; probable top of rock. Roll ahead to 24.6' bgs and set up to core.	
							R/C	217.2		Probable Glacial Till with cobbles to 23.5'.	
								215.0			
25	R1	60/50	24.6 - 29.6	RQD = 75%			NX2			R1: Very hard to hard, fresh, medium grained, white, GRANITE.	q _n =1745 ksf

1. 300 lb hammer 30" drop for casing blows.
2. Automatic Hammer NEBC #B-24 (Truck 83), Energy Transfer Rate = 0.937.
3. Intermitent cobbles from 12.0'-23.5' bgs.
4. Water level was measured after 15 hrs, with casing in the ground.
5. 4" casing to 15.0' bgs, broke casing and removed; spun 3" casing to 24.6' bgs.
6. Coordinates: N541081.4, E1131867.6

Boring No.: BB-AI95-101

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS					Project: Western Avenue Bridge #5808 Location: Augusta, Maine			Boring No.: BB-AI95-102 WIN: 21672.00																																																																																																																																																																																																																													
Driller: New England Boring Contractors				Elevation (ft.) 215.5				Auger ID/OD: 4.25" OD																																																																																																																																																																																																																													
Operator: Mike Porter				Datum: NAVD 88				Sampler: Standard Splitspoon																																																																																																																																																																																																																													
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Date Start/Finish: 7/18/18-7/18/18				Drilling Method: Drive & Wash				Core Barrel: NX2																																																																																																																																																																																																																													
Boring Location: Sta. 19+81.4, Offset 6.5' L				Casing ID/OD: 4"/4.5", 3"/3.5"				Water Level*: 4.0'																																																																																																																																																																																																																													
Hammer Efficiency Factor: 0.937				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐																																																																																																																																																																																																																																	
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<table><tr><td rowspan="2">Depth (ft.)</td><td colspan="8">Sample Information</td><td rowspan="2">Graphic Log</td><td rowspan="2">Visual Description and Remarks</td><td rowspan="2">Laboratory Testing Results/ AASHTO and Unified Class.</td></tr><tr><td>Sample No.</td><td>Pen./Rec. (in.)</td><td>Sample Depth (ft.)</td><td>Blows (6 in.) Shear Strength (psf) or RQD (%)</td><td>N-uncorrected</td><td>N₆₀</td><td>Casing Blows</td><td>Elevation (ft.)</td></tr><tr><td rowspan="4">0</td><td>1D</td><td>24/12</td><td>0.0 - 2.0</td><td>1-3-6-7</td><td>9</td><td>14</td><td>17</td><td rowspan="12">207.8</td><td rowspan="12"></td><td rowspan="12">Brown, dry, medium dense, fine to coarse SAND, little gravel, little silt, (Fill).</td><td rowspan="12">G#3 A-1-a, GP-GM WC=6.6</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td>44</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td>40</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td>17</td></tr><tr><td rowspan="4">5</td><td>2D</td><td>24/6</td><td>4.0 - 6.0</td><td>5-15-8-4</td><td>23</td><td>36</td><td>27</td><td rowspan="12">198.1</td><td rowspan="12"></td><td rowspan="12">Brown, wet, dense, sandy GRAVEL, little silt, (Fill).</td><td rowspan="12"></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td>70</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td>91</td></tr><tr><td>R1</td><td>60/59</td><td>7.7 - 12.7</td><td>RQD = 95%</td><td></td><td></td><td>23/7" NX2</td></tr><tr><td rowspan="4">10</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td rowspan="12">17.4</td><td rowspan="12"></td><td rowspan="12">Casing met refusal at 7.7' bgs. Roller bit to 7.7' bgs and set up to core. R1: Very hard to hard, fresh, medium grained, dark to light grey, GRANOFELS. Joints are close to widely spaced, low angle, rough, undulating, partially open to open, fresh. -MAYFLOWER HILL FORMATION- Recovery = 98% Rock Quality = Excellent Rock Core Times (min:sec): 7.7-8.7' (1:45), 8.7-9.7' (1:55), 9.7-10.7' (1:55), 10.7-11.7' (1:30), 11.7-12.7' (1:50) R2: Very hard to hard, grey, medium grained, GRANOFELS. Joints are very close to widely spaced, low angle, rough, undulating, fresh, open. -MAYFLOWER HILL FORMATION- Recovery = 100% Rock Quality = Excellent Rock Core Times (min:sec): 12.7-13.7' (2:40), 13.7-14.7' (2:10), 14.7-15.7' (1:35), 15.7-16.7' (1:55), 16.7-17.4' (1:00)</td><td rowspan="12"></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>R2</td><td>56/57</td><td>12.7 - 17.4</td><td>RQD = 91%</td><td></td><td></td><td></td></tr><tr><td rowspan="4">15</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td rowspan="12">Bottom of Exploration at 17.4 feet below ground surface.</td><td rowspan="12"></td><td rowspan="12"></td><td rowspan="12"></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td rowspan="4">20</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td rowspan="12"></td><td rowspan="12"></td><td rowspan="12"></td><td rowspan="12"></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td rowspan="4">25</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td rowspan="12"></td><td rowspan="12"></td><td rowspan="12"></td><td rowspan="12"></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr></table>												Depth (ft.)	Sample Information								Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)	0	1D	24/12	0.0 - 2.0	1-3-6-7	9	14	17	207.8		Brown, dry, medium dense, fine to coarse SAND, little gravel, little silt, (Fill).	G#3 A-1-a, GP-GM WC=6.6							44							40							17	5	2D	24/6	4.0 - 6.0	5-15-8-4	23	36	27	198.1		Brown, wet, dense, sandy GRAVEL, little silt, (Fill).								70							91	R1	60/59	7.7 - 12.7	RQD = 95%			23/7" NX2	10								17.4		Casing met refusal at 7.7' bgs. Roller bit to 7.7' bgs and set up to core. R1: Very hard to hard, fresh, medium grained, dark to light grey, GRANOFELS. Joints are close to widely spaced, low angle, rough, undulating, partially open to open, fresh. -MAYFLOWER HILL FORMATION- Recovery = 98% Rock Quality = Excellent Rock Core Times (min:sec): 7.7-8.7' (1:45), 8.7-9.7' (1:55), 9.7-10.7' (1:55), 10.7-11.7' (1:30), 11.7-12.7' (1:50) R2: Very hard to hard, grey, medium grained, GRANOFELS. Joints are very close to widely spaced, low angle, rough, undulating, fresh, open. -MAYFLOWER HILL FORMATION- Recovery = 100% Rock Quality = Excellent Rock Core Times (min:sec): 12.7-13.7' (2:40), 13.7-14.7' (2:10), 14.7-15.7' (1:35), 15.7-16.7' (1:55), 16.7-17.4' (1:00)																R2	56/57	12.7 - 17.4	RQD = 91%				15								Bottom of Exploration at 17.4 feet below ground surface.																									20																																	25																																
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Remarks: 1. 300 lb hammer 30" drop for casing blows. 2. Automatic Hammer NEBC #B-24 (Truck 83), Energy Transfer Rate = 0.937. 3. Boring was drilled through .9' of concrete bridge deck using a 4" spin shoe. 21.7' was measured from top of bridge deck to ground surface. 4. Water level was measured 15 minutes after drilling, with casing in the ground. 5. Coordinates: N541102.6, E1131703.4																																																																																																																																																																																																																																					
Stratification lines represent approximate boundaries between soil types; transitions may be gradual. * Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										Page 1 of 1 Boring No.: BB-AI95-102																																																																																																																																																																																																																											

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS					Project: Western Avenue Bridge #5808 Location: Augusta, Maine			Boring No.: BB-AI95-103 WIN: 21672.00			
Driller: New England Boring Contractors		Elevation (ft.) 233.1		Auger ID/OD: 4.25" OD							
Operator: Mike Porter		Datum: NAVD 88		Sampler: Standard Splitspoon							
Logged By: Blaine Cardali		Rig Type: Truck Rig		Hammer Wt./Fall: 140#/30"							
Date Start/Finish: 7/16/18-7/16/18		Drilling Method: Drive & Wash		Core Barrel: NX							
Boring Location: Sta. 21+32.1, Offset 34.7' R		Casing ID/OD: 4"/4.5", 3"/3.5"		Water Level*: 4.0'							
Hammer Efficiency Factor: 0.937		Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐									
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt		R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140 lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person		S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _{u(lab)} = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected							
		T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test									
Depth (ft.)	Sample Information								Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)			
25	8D	24/10	25.0 - 27.0	16-12-12-11	24	37	33		Grey, wet, dense, silty fine to medium SAND, little gravel, (Glacial Till).	q _p =1869 ksf	
						47					
						37					
						37					
						38					
30	9D	24/12	30.0 - 32.0	10-15-17-28	32	50	29				
						67					
						146					
	R1	60/60	33.3 - 38.3	RQD = 100%		78 1/2" NX2	199.9				
35											
	R2	60/52	38.3 - 43.3	RQD = 58%							
40											
50								189.8	Bottom of Exploration at 43.3 feet below ground surface.		
Remarks: 1. 300 lb hammer 30" drop for casing blows. 2. Automatic Hammer NEBC #B-24 (Truck 83), Energy Transfer Rate = 0.937. 3. Could not retrieve an 8" piece of the bottom of R2. 4. Water level was measured 30 minutes after completion of drilling, without casing in the ground. 5. Coordinates: N541173.2, E1131565.4											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual. * Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.									Page 2 of 2 Boring No.: BB-AI95-103		

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Western Avenue Bridge #5808 Location: Augusta, Maine				Boring No.: BB-AI95-202 WIN: 21672.00							
Driller: New England Boring Contractors				Elevation (ft.): 212.4				Auger ID/OD:							
Operator: B. Enos				Datum: NAVD 88				Sampler: Standard Splitspoon							
Logged By: L. Navarrete				Rig Type: B-53 Truck				Hammer Wt./Fall: 140#/30"							
Date Start/Finish: 10/31/22-10/31/22				Drilling Method: Drive & Wash				Core Barrel: NQ							
Boring Location: Sta. 19+11.5, Offset 39.4' R				Casing ID/OD: 4"/4.5", 3"/3.5"				Water Level*: 0.8'							
Hammer Efficiency Factor: 0.86				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐											
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt				R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140 lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person				S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected				T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test			
Depth (ft.)	Sample Information								Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.				
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)							
25									Rock Core Times (min:sec): 16.0-17.0' (1:18), 17.0-18.0' (1:14), 18.0-18.5' (2:13) -----18.5- R5: Very hard, fresh, aphanitic to fine grained, grey, GRANOFELS. One high angle joint is planar, smooth, discolored, tight. Second joint is horizontal, planar, smooth, fresh, very tight. - MAYFLOWER HILL FORMATION - Recovery = 75% Rock Quality = Fair Rock Core Times (min:sec): 18.5-19.5' (2:10), 19.5-20.5' (1:30), 20.5-21.5' (1:19), 21.5-22.5' (1:50), 22.5-23.5' (2:10) -----23.5- Bottom of Exploration at 23.5 feet below ground surface.						
50															
Remarks: 1. Fine Grained Soil Descriptions on this log are based on plasticity estimated using visual manual classification techniques or laboratory Atterberg Limit Tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes. 2. Automatic Hammer NEBC-23, Energy Transfer Rate = 0.86. 3. Water level measured immediately after removal of casing. 4. Cored into pavement with 5" thin wall bit. Concrete deck 9" thick. Ground level 22.5' below top of bridge deck. 5. Coordinates: N541138.7, E1131778.2															
Stratification lines represent approximate boundaries between soil types; transitions may be gradual. Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										Page 2 of 2 Boring No.: BB-AI95-202					

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Western Avenue Bridge #5808 Location: Augusta, Maine				Boring No.: BB-AI95-202A WIN: 21672.00							
Driller: New England Boring Contractors				Elevation (ft.): 212.4 (est.)				Auger ID/OD:							
Operator: B. Enos				Datum: NAVD 88				Sampler: Standard Splitspoon							
Logged By: L. Navarrete				Rig Type: B-53 Truck				Hammer Wt./Fall: 140#/30"							
Date Start/Finish: 11/1/22-11/1/22				Drilling Method: Drive & Wash				Core Barrel: NQ							
Boring Location: Sta. 19+11.9, Offset 30.3' R				Casing ID/OD: 4"/4.5", 3"/3.5"				Water Level*: 0.0'							
Hammer Efficiency Factor: 0.86				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐											
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt				R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140 lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person				S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected				T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test			
Depth (ft.)	Sample Information								Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.				
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)							
25									to moderately spaced, high angle, dipping, planar, smooth, fresh, very tight to tight. - MAYFLOWER HILL FORMATION - Recovery = 100% Rock Quality = Excellent Rock Core Times (min:sec): 17.0-17.0' (2:57), 18.0-19.0' (1:25), 19.0-20.0' (1:56), 20.0-21.0' (3:16), 21.0-22.0' (1:39) Bottom of Exploration at 22.0 feet below ground surface.						
50															
Remarks: 1. Fine Grained Soil Descriptions on this log are based on plasticity estimated using visual manual classification techniques or laboratory Atterberg Limit Tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes. 2. Automatic Hammer NEBC-23, Energy Transfer Rate = 0.86. 3. Water level measured immediately after removal of casing. 4. Cored into pavement with 5" thin wall bit. Concrete deck 9" thick. Ground level 23.0' below top of bridge deck. 5. Coordinates: N541129.6, E1131778.1															
Stratification lines represent approximate boundaries between soil types; transitions may be gradual. Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.									Page 2 of 2 Boring No.: BB-AI95-202A						

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS					Project: Western Avenue Bridge #5808 Location: Augusta, Maine			Boring No.: HB-AI95-201 WIN: 21672.00				
Driller: New England Boring Contractors				Elevation (ft.) 237.4				Auger ID/OD: 4.25"				
Operator: B. Enos				Datum: NAVD 88				Sampler: Standard Splitspoon				
Logged By: L. Navarrete				Rig Type: B-53 Truck				Hammer Wt./Fall: 140#/30"				
Date Start/Finish: 11/3/22-11/3/22				Drilling Method: Drive & Wash				Core Barrel: NQ				
Boring Location: Sta. 20+88.6, Offset 48.2' L				Casing ID/OD: 4.0/3.0"				Water Level*: 12.5'				
Hammer Efficiency Factor: 0.86				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐								
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt				R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person			Su = Peak/Remolded Field Vane Undrained Shear Strength (psf) Su(lab) = Lab Vane Undrained Shear Strength (psf) qp = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N60 = SPT N-uncorrected Corrected for Hammer Efficiency N60 = (Hammer Efficiency Factor/60%)*N-uncorrected			Ty = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test		
Depth (ft.)	Sample Information								Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.	
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)				
0	1D	24/3	0.0 - 2.0	1-4-11-5	15	22	SSA		Brown, dry, medium dense, fine to coarse SAND, some silt, little gravel, with roots, (Fill).	G# 22-S-4932 A-1-4(0), SM WC=7.8		
	2D	24/5	2.0 - 4.0	5-3-5-6	8	11					Brown, dry, medium dense, fine to coarse SAND, some silt, little gravel, (Fill).	
	3D	24/8	4.0 - 6.0	3-9-6-2	15	22					Brown, dry, medium dense, fine to coarse SAND, some silt, little gravel, (Fill).	
	4D	24/7	6.0 - 8.0	4-6-10-8	16	23					Brown, dry, medium dense, fine to coarse SAND, some gravel, some silt, (Fill).	
	5D	24/6	8.0 - 10.0	7-6-4-4	10	14					Brown, dry, medium dense, sandy GRAVEL, little silt, (Fill).	
							34					
							39					
							91					
							143					
							132					
	6D	24/12	14.0 - 16.0	7-16-30-10	46	66	18				Top 6": Brown, moist, fine SAND, trace silt, (Fill).	
15							76				Bottom 6": Black, wet, SILT, little gravel, trace fine sand, (Glacial Till).	
							300					
							158					
	7D	24/14	19.0 - 21.0	24-25-28-23	53	76	RC	Brown, dry, very dense, fine to coarse SAND, some gravel, little silt, (Glacial Till).				
20												
	R1	60/60	22.8 - 27.8	RQD = 97%				214.7				
25												
Remarks: 1. Fine Grained Soil Descriptions on this log are based on plasticity estimated using visual manual classification techniques or laboratory Atterberg Limit Tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes. 2. Automatic Hammer NEBC-23, Energy Transfer Rate = 0.86. 3. Water level measured immediately after removal of drill rods. Casing in place. 4. Coordinates: N541082.4, E1131588.5												
Stratification lines represent approximate boundaries between soil types; transitions may be gradual. * Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										Page 1 of 2 Boring No.: HB-AI95-201		

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Western Avenue Bridge #5808 Location: Augusta, Maine				Boring No.: HB-AI95-201 WIN: 21672.00			
Driller: New England Boring Contractors			Elevation (ft.): 237.4			Auger ID/OD: 4.25"					
Operator: B. Enos			Datum: NAVD 88			Sampler: Standard Splitspoon					
Logged By: L. Navarrete			Rig Type: B-53 Truck			Hammer Wt./Fall: 140#/30"					
Date Start/Finish: 11/3/22-11/3/22			Drilling Method: Drive & Wash			Core Barrel: NQ					
Boring Location: Sta. 20+88.6, Offset 48.2' L			Casing ID/OD: 4.0/3.0"			Water Level*: 12.5'					
Hammer Efficiency Factor: 0.86			Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐								
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt			R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140 lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person			S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected			T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test		
Depth (ft.)	Sample Information								Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)			
25								209.6	GRANITE. Joints are moderately spaced, horizontal, planar, smooth, fresh, tight. One moderately dipping joint. Joint is planar, smooth, fresh, tight to partially open. - HALLOWELL GRANITE INTRUSION - Recovery = 100% Rock Quality = Excellent Rock Core Times (min:sec): 22.8-23.8' (3:04), 23.8-24.8' (2:30), 24.8-25.8' (3:16), 25.8-26.8' (3:54), 26.8-27.8' (4:05) Bottom of Exploration at 27.8 feet below ground surface.		
50											
Remarks: 1. Fine Grained Soil Descriptions on this log are based on plasticity estimated using visual manual classification techniques or laboratory Atterberg Limit Tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes. 2. Automatic Hammer NEBC-23, Energy Transfer Rate = 0.86. 3. Water level measured immediately after removal of drill rods. Casing in place. 4. Coordinates: N541082.4, E1131588.5											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual. * Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										Page 2 of 2 Boring No.: HB-AI95-201	

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Location: Augusta, Maine

WIN: 21672.00

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows					
0							SSA	232.9		0'-0.2': Asphalt.	G#22-S-4934 A-1-b, SM WC=4.5	
	1D	24/16	1.0 - 3.0	32-78-31-26	109	156				Brown, dry, very dense, fine to coarse SAND, some gravel, little silt, (Fill).		
	2D	24/6	3.0 - 5.0	13-9-3-58	12	17				Brown, dry, medium dense, fine to coarse SAND, little silt, trace gravel, (Fill). Auger spoils consist of brown, dry, fine to coarse SAND, some gravel, little silt, (Fill).		
5												
	3D	24/18	9.0 - 11.0	6-11-18-14	29	42	180			Brown, moist, dense, fine to coarse SAND, some silt, trace asphalt pieces, (Fill).		
10							200					
							83					
							106			Observed casing driving resistance change at 12.2'.		
							88					
	4D	24/2	14.0 - 16.0	4-3-1-1	4	6	64			Brown, wet, very loose, GRAVEL, (Fill).		
15							91					
							63/4"					
	R1	36/22	17.0 - 20.0				NQ	216.1	Casing refusal at 17.0'. Set up to core at 17.0'. R1: Very hard, fresh, aphanitic to medium grained, grey, GRANITE BOULDER, 3"-8" long pieces recovered.			
20	5D	24/15	20.0 - 22.0	3-6-12-16	18	26	RC	213.1	Black, wet, very stiff, SILT, trace fine sand, (Glacial Till).			
	6D	24/14	22.0 - 24.0	8-18-20-54	38	54			Top 12": Grey, wet, hard, SILT & CLAY, little gravel, little fine to coarse sand.			
25									Bottom 2": Grey, wet, fine to coarse SAND, some gravel, some silt, (Glacial Till).			

1. Fine Grained Soil Descriptions on this log are based on plasticity estimated using visual manual classification techniques or laboratory Atterberg Limit Tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.
2. Automatic Hammer NEBC-23, Energy Transfer Rate = 0.86.
3. Water level measured immediately after removal of drill rods. Casing in place.
4.Coordinates: N541186.4, E1131519.5

Boring No.: HB-AI95-202

[illegible]



05/19/2023

GEOTECHNICAL DESIGN REPORT
WESTERN AVENUE BRIDGE NO. 5808 OVER INTERSTATE 95
Maine Department of Transportation
09.0025983.01

APPENDIX C – GEOPHYSICAL TEST REPORT

**GEOPHYSICAL LOG RESULTS
OF BOREHOLE BB-AI95-202A
AT THE WESTERN AVENUE
BRIDGE REPLACEMENT SITE
IN AUGUSTA, MAINE**

Northeast Geophysical Services
Bangor, Maine
December, 2022

GEOPHYSICAL LOG RESULTS OF BOREHOLE BB-AI95-202A AT THE WESTERN AVENUE BRIDGE REPLACEMENT SITE IN AUGUSTA, MAINE

Introduction

At the request of the GZA, a bedrock borehole (BB-AI95-202A) located at the Western Avenue Bridge Replacement Site in Augusta, Maine was geophysically logged. The borehole was logged on December 28, 2022 by Rudy Rawcliffe of Northeast Geophysical Services (NGS), Inc. The purpose of the geophysical logging was to determine the location and orientation of fractures, joints and cracks in the borehole. Caliper, acoustic televiewer (ATV) and optical televiewer (OTV) were run on the borehole.

Summary of Results

The caliper, OTV and ATV image log plots for the borehole are presented in the appendix. The interpreted results of the televiewer logs are summarized in a table that provides the depth and calculated strike and dip of each identified feature in the borehole and also the aperture (width) of each feature. Rose and polar plots of the dip and dip direction of all the interpreted features for the borehole is also appended.

Methods and Instrumentation

The borehole was logged with a Mount Sopris Matrix digital logger. The borehole was first logged with a caliper tool which measures the borehole diameter. Cracks and fractures often appear on the caliper log as abrupt widenings in diameter.

The borehole was then logged with an optical televiewer. The optical log provides a digital image of the borehole walls that is oriented to magnetic north. Planar features such as fractures, bedding surfaces, and joints can be identified with the optical tool and the strike, dip direction and dip angle of these features can often be determined.

The borehole was then logged with an acoustic televiewer. The ATV log provides an acoustical image of the borehole walls. The ATV works by scanning the borehole wall with an acoustic beam that is produced by a rapidly rotating piezoelectric source. Similar to the optical televiewer, planar features such as fractures, bedding surfaces and joints can be identified with the ATV tool and the strike, dip direction and dip angle of these features can often be determined. The ATV (and OTV) data are presented as “unwrapped” images of the borehole wall that are oriented to magnetic north. The dip angle and dip direction of any planar feature that intersects the borehole can be measured from this image. Figure 1, on the following page, illustrates this.

The optical and acoustical televiewer logs are somewhat duplicative in that they both can provide similar information. However, there are advantages and disadvantages to both tools. The ATV requires the borehole to be water filled and will not provide information above the water level. The OTV can work in air or water but is not effective in cloudy, turbid water whereas the ATV will work fine in cloudy water. The ATV can be better at discerning voids, cracks and fractures whereas the OTV can be better at discerning lithology. Also, sometimes water-bearing fractures

are rust stained, which can be seen by the OTV.

Borehole Televiewer Data

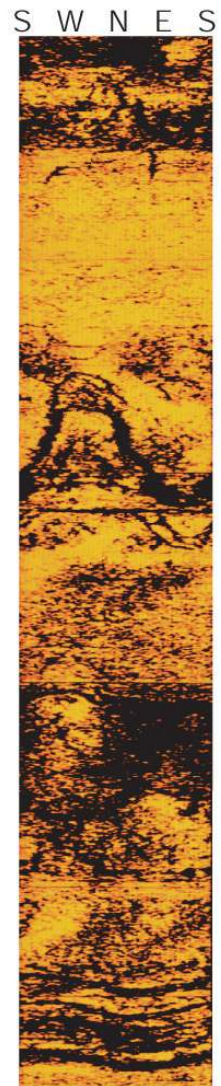
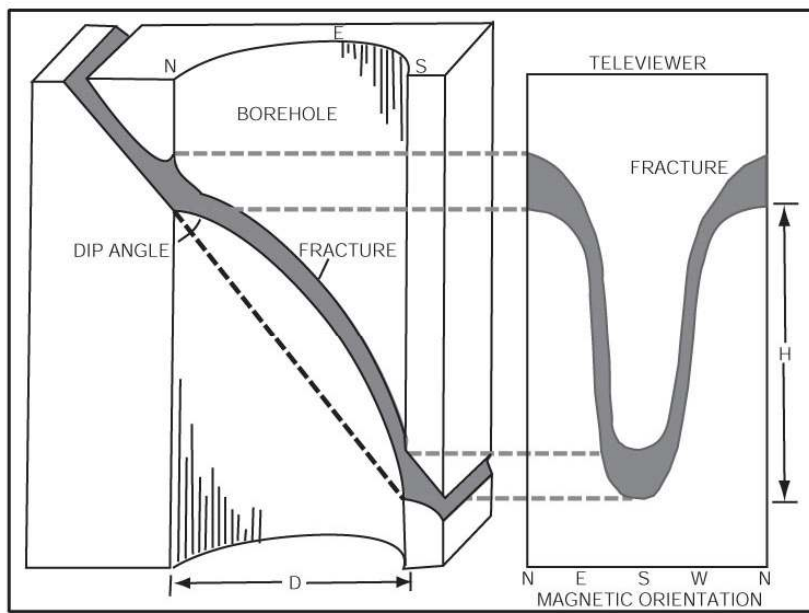


Figure 1 - ATV Unwrapped Image

Geophysical Log Results

Borehole BB-AI95-202A:

Total Depth (TOC):	23.7 feet
Casing Depth:	7.95 feet
Casing stick up:	1.51 feet
Water Level:	1.5 feet

Plate 1, in the appendix, shows the OTV and ATV logs for BB-AI95-202A. The ATV and OTV logs provide the strike and dip of planar features that intersect the borehole. These planar features may be fractures, joints, cracks or may represent cleavage or bedding planes. **All of the depths on the logs and tables are referenced from the top of the casing.**

Also shown on Plate 1 is a 3-D “virtual core” image of the borehole that is derived from the optical televiewer.

The two columns on the right side of Plate 1 present the interpreted structure of the planar features that intersect the borehole. The column titled, "Structure" is a projection of the interpreted structure in the borehole. It was obtained by digitizing the planar features from the ATV and OTV logs. The dip direction and dip amount (corrected for the deviation of the borehole) was obtained from this log and tabulated in the appendix. The relative aperture width of the fractures is indicated by the line width of the digitized planar features. The features that had aperture widths over 1 mm and under 10 mm are colored light blue and the features that are over 10mm are colored dark blue.

The log in the far right column entitled, "Tadpole Plot" presents the structural data as a tadpole plot. The head of the tadpole indicates the dip amount (from 0 to 90 degrees) and the tail of the tadpole indicates the dip direction.

Plate 2 is a Rose plot of the strike and dip direction of all the interpreted planar features in the borehole. This plot shows that there is a range of orientations but most strike northeast at around 35 degrees and dip steeply to the southeast. This probably represents the foliation or bedding of the bedrock.

Plate 3 is a polar plot (lower hemisphere) showing the dip amount and dip direction of all the interpreted planar features in the well. This graph shows that there is a wide range of dip directions of the planar features but all of the features that have aperture widths of over 10 mm dip 40 degrees or less.

Table 1 in the appendix is a tabulation of the planar features (possible fractures, joints and cracks, etc) that were identified in the borehole. This table shows that depth, strike, dip and dip direction of each identified feature. Table 1 also shows the estimated width (in millimeters) of each planar feature.

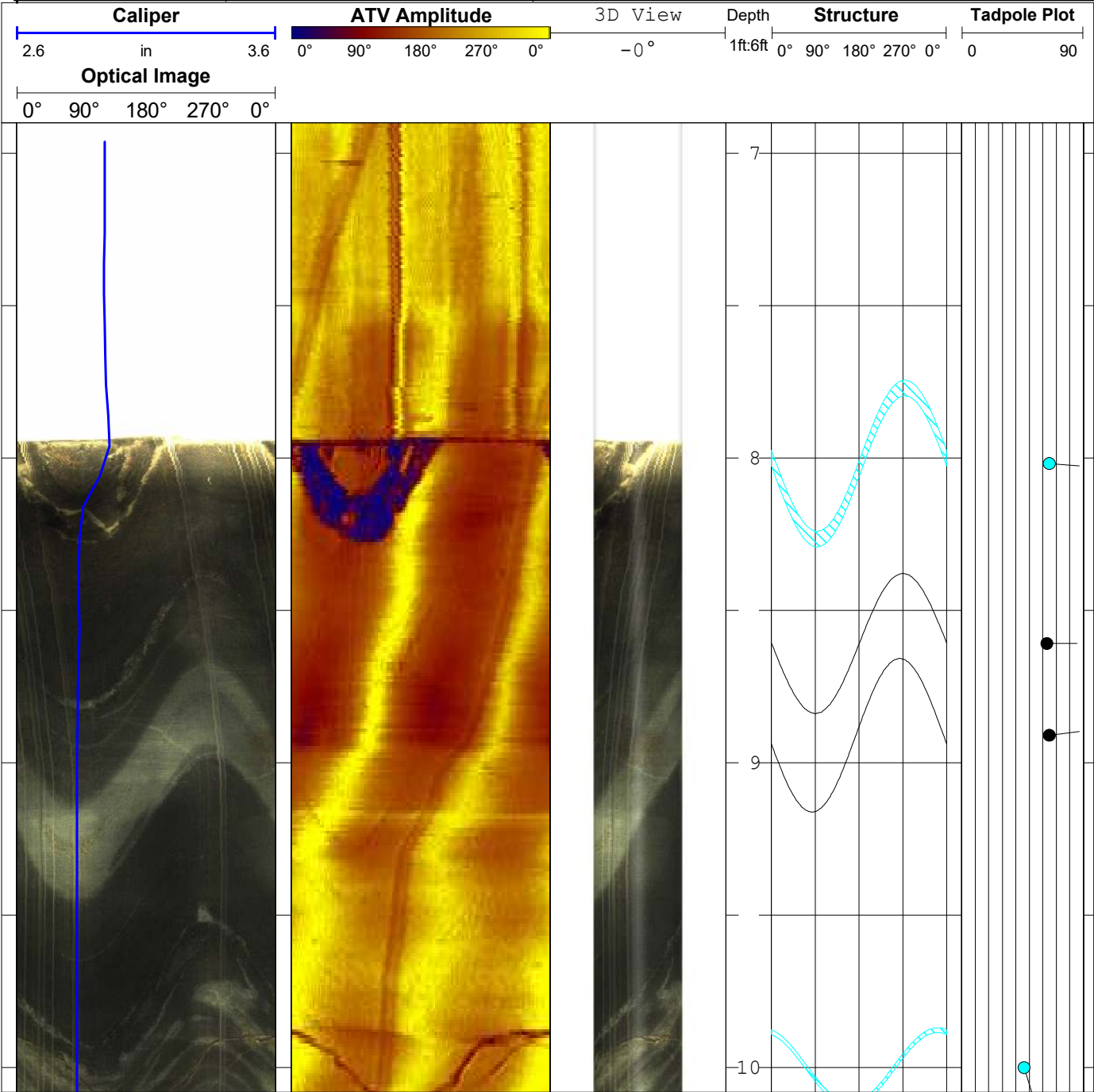
Discussion of Geophysical Log Results

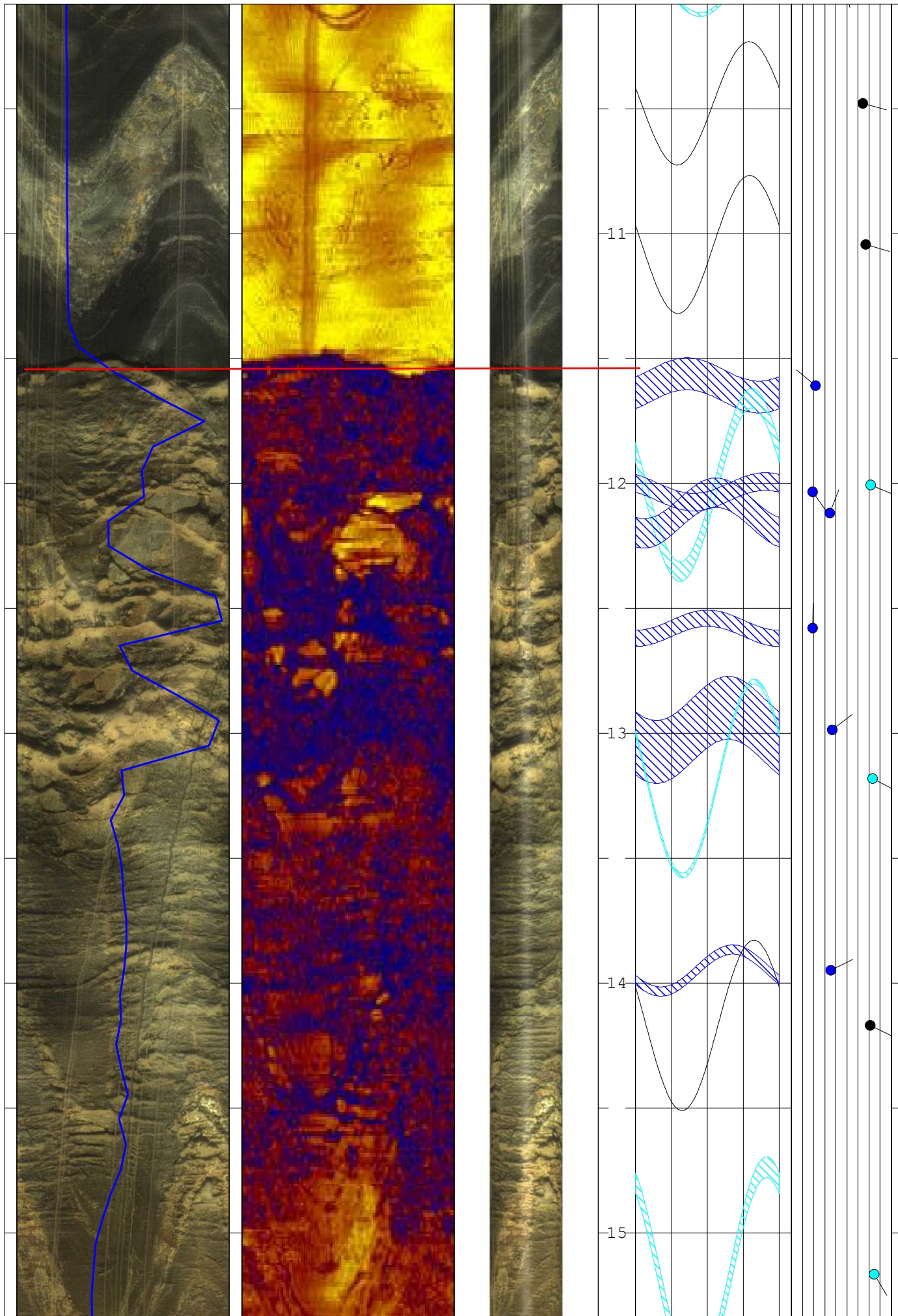
The objective of the televiewer logging was to map the discontinuities (fractures, joints and cracks) in the borehole. The acoustical televiewer provided an excellent image of the borehole walls. The optical televiewer also provided an excellent image of the borehole.

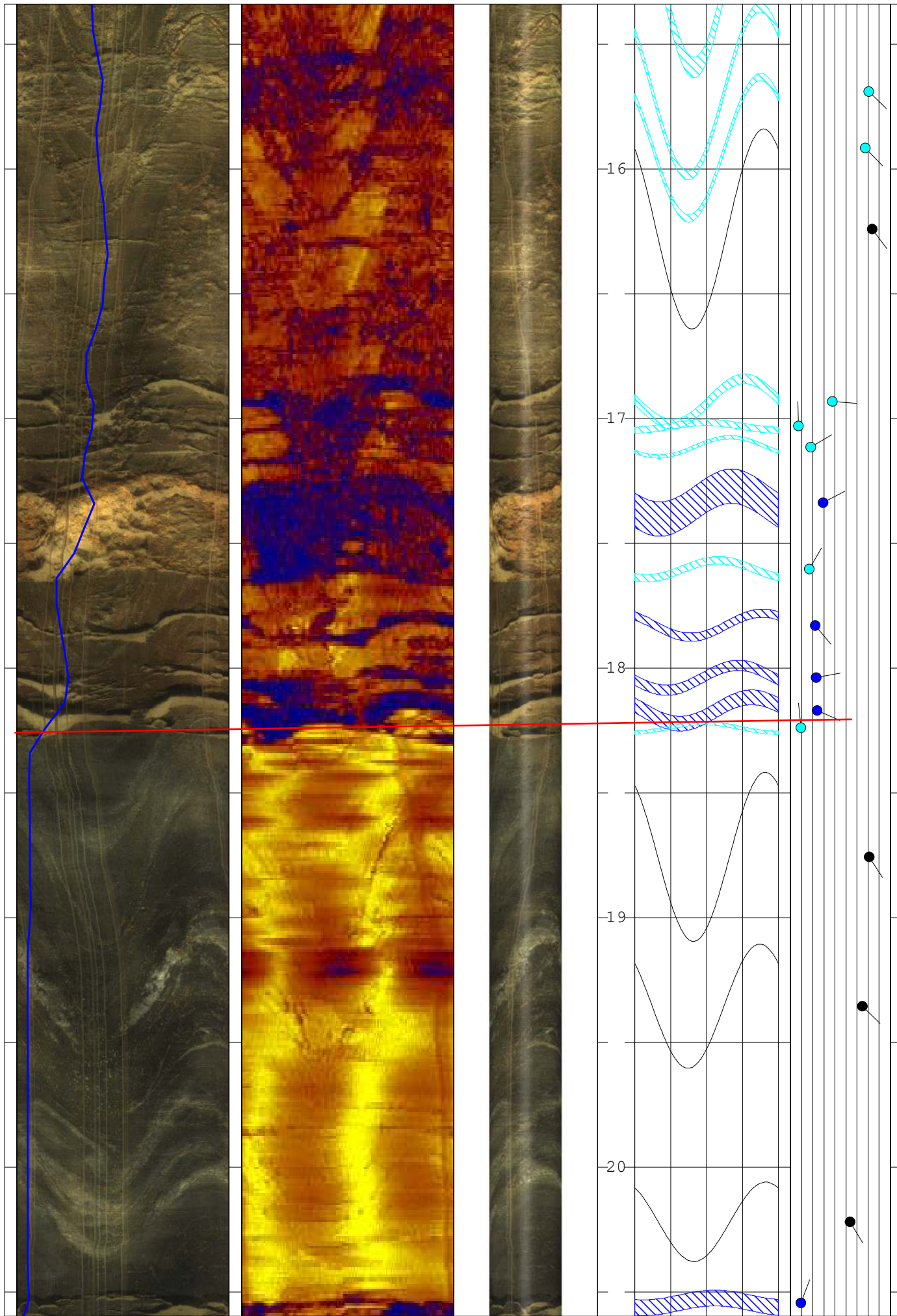
A total of 36 planar features (bedding, foliation, joints, fractures, cracks, etc.) were measured in the borehole. Of these, 12 features were estimated to have an aperture of between 1 mm and 10 mm and 11 features were estimated to have an aperture of over 10 mm. In general most of the wider fractures in the borehole are located between 11.5 and 18.5 feet. The area from about 11.5 to 13.5 feet below top of casing is very broken up and may represent a fault zone. The features that have wider apertures are more likely to represent planes of weakness in the bedrock. However, this may not always be the case as voids and cracks in the borehole can also be caused by drilling rather than representing natural features such as bedrock fractures.

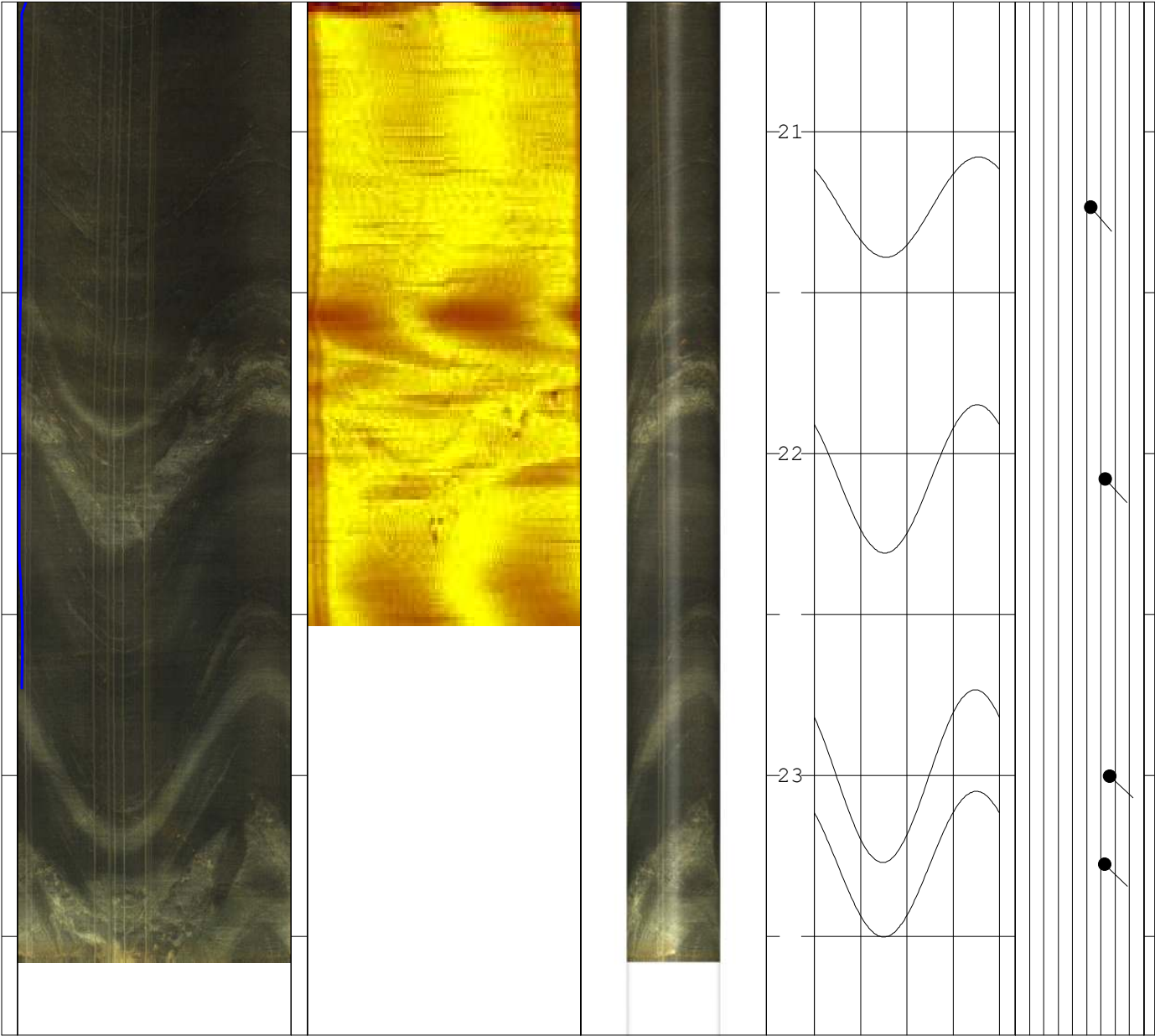
APPENDIX
BOREHOLE TELEVIEWER LOG AND GRAPHS

Northeast Geophysical Services 4 Union Street Bangor, Maine 04401 Tel. 207-942-2700 email: ngsinc@negeophysical.com		Log: Plate 1 Televiwer & Caliper Logs
		Well: BB-AI95-202A
		Site: Western Avenue Bridge Replacement
Date:	12/28/2022	Location: Augusta, Maine
Casing Depth	7.95 ft.	For: GZA
Casing Type:	3 inch PVC	Logged by: R Rawcliffe
Boring Depth:	23.7 ft.	Orientation: magnetic north
Meas. From:	top of casing	Structure Plots: black = planar feature less than 1 mm light blue = planar feature (possible fracture) 1-10 mm dark blue = planar feature (possible fracture) > 10 mm
Stickup:	1.51 ft	
Water Level:	ground surface	

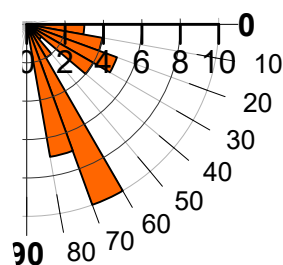








Strike and Dip Direction of all features



 Dip direction of feature
 Strike of feature
 Dip Amount (Tilt)

- Likely transmissible
- Possibly transmissible
- Aperture > 1 mm
- Aperture < 1 mm

Declination: 15.1 degrees west

TABLE 1 Planar features interpreted from acoustical and optical televiewers Borehole BB-AI95-202A (depths are from top of casing which was 1.51 feet above ground surface)									
								Logged: 12/28/2022	
Borehole	Feature # Number	Feature depth Feet	Dip Degrees	Dip Azimuth magnetic	Strike magnetic	Dip Azimuth True	Strike True	Aperture width (mm)	Category
BB-AI95-202A	1	8.0	65	94	4	79	349	7	108
BB-AI95-202A	2	8.6	63	90	0	75	345	<1 mm	100
BB-AI95-202A	3	8.9	65	84	354	68	338	<1 mm	100
BB-AI95-202A	4	10.0	46	162	72	147	57	3	108
BB-AI95-202A	5	10.5	64	104	14	89	359	<1 mm	100
BB-AI95-202A	6	11.0	67	106	16	91	1	<1 mm	100
BB-AI95-202A	7	11.6	22	309	219	294	204	36	107
BB-AI95-202A	8	12.0	71	112	22	97	7	8	108
BB-AI95-202A	9	12.0	19	145	55	130	40	20	107
BB-AI95-202A	10	12.1	35	21	291	6	276	30	107
BB-AI95-202A	11	12.6	19	1	271	346	256	19	107
BB-AI95-202A	12	13.0	37	52	322	37	307	61	107
BB-AI95-202A	13	13.2	73	118	28	103	13	2	108
BB-AI95-202A	14	14.0	35	63	333	48	318	10	107
BB-AI95-202A	15	14.2	71	117	27	102	12	<1 mm	100
BB-AI95-202A	16	15.2	75	148	58	133	43	7	108
BB-AI95-202A	17	15.7	71	134	44	119	29	3	108
BB-AI95-202A	18	15.9	67	136	46	120	30	3	108
BB-AI95-202A	19	16.2	74	143	53	128	38	<1 mm	100
BB-AI95-202A	20	16.9	38	95	5	80	350	9	108
BB-AI95-202A	21	17.0	7	359	269	344	254	8	108
BB-AI95-202A	22	17.1	18	60	330	45	315	4	108
BB-AI95-202A	23	17.3	29	63	333	48	318	37	107
BB-AI95-202A	24	17.6	17	30	300	15	285	9	108
BB-AI95-202A	25	17.8	22	140	50	125	35	10	107
BB-AI95-202A	26	18.0	23	80	350	65	335	12	107
BB-AI95-202A	27	18.2	24	113	23	98	8	16	107
BB-AI95-202A	28	18.2	9	354	264	339	249	4	108



05/19/2023

GEOTECHNICAL DESIGN REPORT
WESTERN AVENUE BRIDGE NO. 5808 OVER INTERSTATE 95
Maine Department of Transportation
09.0025983.01

APPENDIX D – LABORATORY TESTING RESULTS

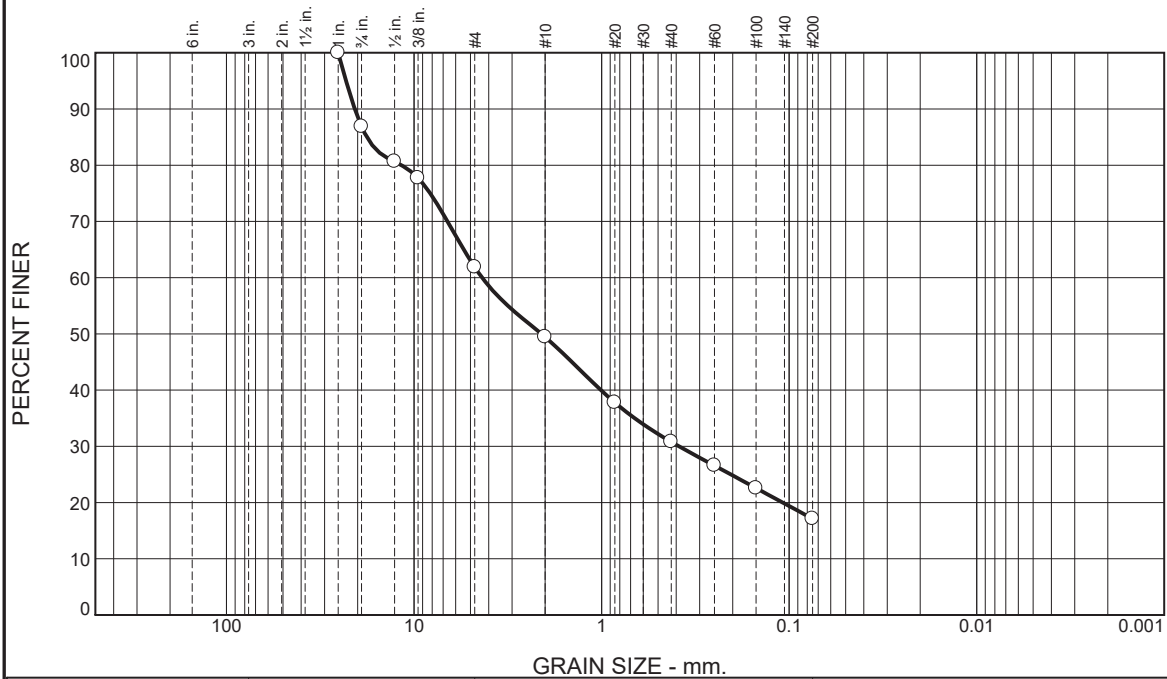
LABORATORY TESTING DATA SHEET

Boring ID	Sample No.	Depth (ft)	Laboratory No.	Identification Tests								Proctor / CBR / Permeability Tests								Laboratory Log and Soil Description
				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.)	Test Setup as % of Proctor	CBR @ 0.1"	CBR @ 0.2"	Permeability (cm/sec)	
				D2216	D4318		D6913			D2874	D854			D1557			D1883			
BB-AI95-101	1D	0.5-2.5	S-1	3.4			38.1	44.8	17.1											Brown Gravelly fine to coarse SAND, little Silt
BB-AI95-101	3D	10-12	S-2	19.4			7.0	26.8	66.2											Brown SILT & CLAY, some f-c Sand, trace fine Gravel
BB-AI95-102	2D	4-6	S-3	6.6			56.3	32.1	11.6											Brown Sandy fine to coarse GRAVEL, little Silt
BB-AI95-103	1D	0.5-2.5	S-4	2.3			37.5	48.0	14.5											Grey Gravelly fine to coarse SAND, little Silt
BB-AI95-103	2D	5-7	S-5	4.0			30.4	54.2	15.4											Brown f-c SAND, some f-c Gravel, little Silt
BB-AI95-103	5D	15-17	S-6	37.3			0.0	9.6	90.4											Grey SILT & CLAY, trace f-m Sand
BB-AI95-103	7D	20-22	S-7	19.1			6.6	18.6	74.8											Brown Grey CLAYEY SILT, little f-c Sand, trace f-c Gravel
BB-AI95-103	4D	12-14	S-8	9.7			31.2	51.6	17.2											Dark Brown f-c SAND, some f-c Gravel, little Silt

Reviewed By SKW

08.06.2018

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	13.1	25.0	12.5	18.6	13.7	17.1	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	86.9		
0.5"	80.6		
0.375"	77.7		
#4	61.9		
#10	49.4		
#20	37.8		
#40	30.8		
#60	26.6		
#100	22.6		
#200	17.1		

* (no specification provided)

Material Description

Brown Gravelly fine to coarse SAND, little Silt

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

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

Coefficients

D₉₀= 20.6995 D₈₅= 17.8667 D₆₀= 4.3336
D₅₀= 2.0928 D₃₀= 0.3865 D₁₅=
D₁₀= C_u= C_c=

Remarks

Date Received: 07.30.18 Date Tested: 08.01.18

Tested By: MN

Checked By: Rebecca Roth

Title: Laboratory Manager

Source of Sample: Borings Depth: 0.5-2.5'
Sample Number: BB-AI95-101 / 1D

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

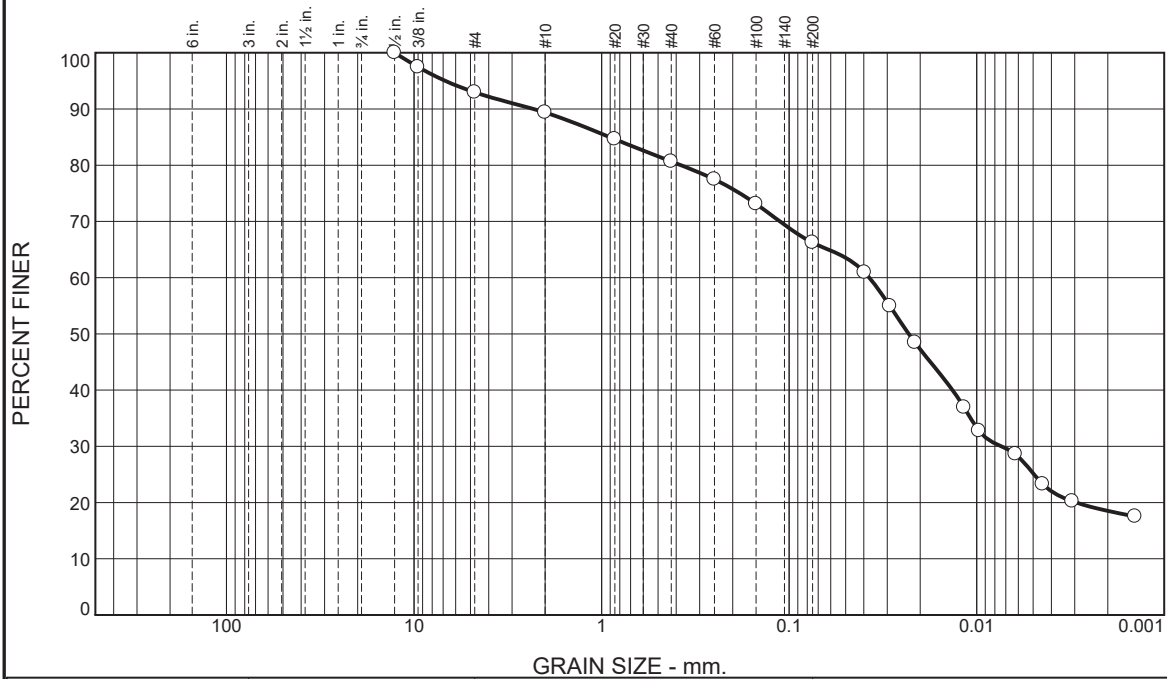
Client: GZA GeoEnvironmental

Project: Western Ave Bridge #5808, WIN 21672.00
Augusta, ME

Project No: 09.0025983.00

Figure S-1

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	7.0	3.6	8.7	14.5	47.7	18.5

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.5"	100.0		
0.375"	97.5		
#4	93.0		
#10	89.4		
#20	84.6		
#40	80.7		
#60	77.5		
#100	73.1		
#200	66.2		
0.0396 mm.	60.9		
0.0290 mm.	55.0		
0.0213 mm.	48.5		
0.0117 mm.	36.9		
0.0097 mm.	32.8		
0.0062 mm.	28.6		
0.0044 mm.	23.3		
0.0031 mm.	20.2		
0.0014 mm.	17.5		

* (no specification provided)

Material Description

Brown SILT & CLAY, some f-c Sand, trace fine Gravel

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

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

Coefficients

D₉₀= 2.3100 D₈₅= 0.9048 D₆₀= 0.0374
D₅₀= 0.0230 D₃₀= 0.0074 D₁₅=
D₁₀= C_u= C_c=

Remarks

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

Date Received: 07.31.18 Date Tested: 08.03.18

Tested By: MN

Checked By: Rebecca Roth

Title: Laboratory Manager

Source of Sample: Borings Depth: 10-12'
Sample Number: BB-AI95-101 / 3D

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

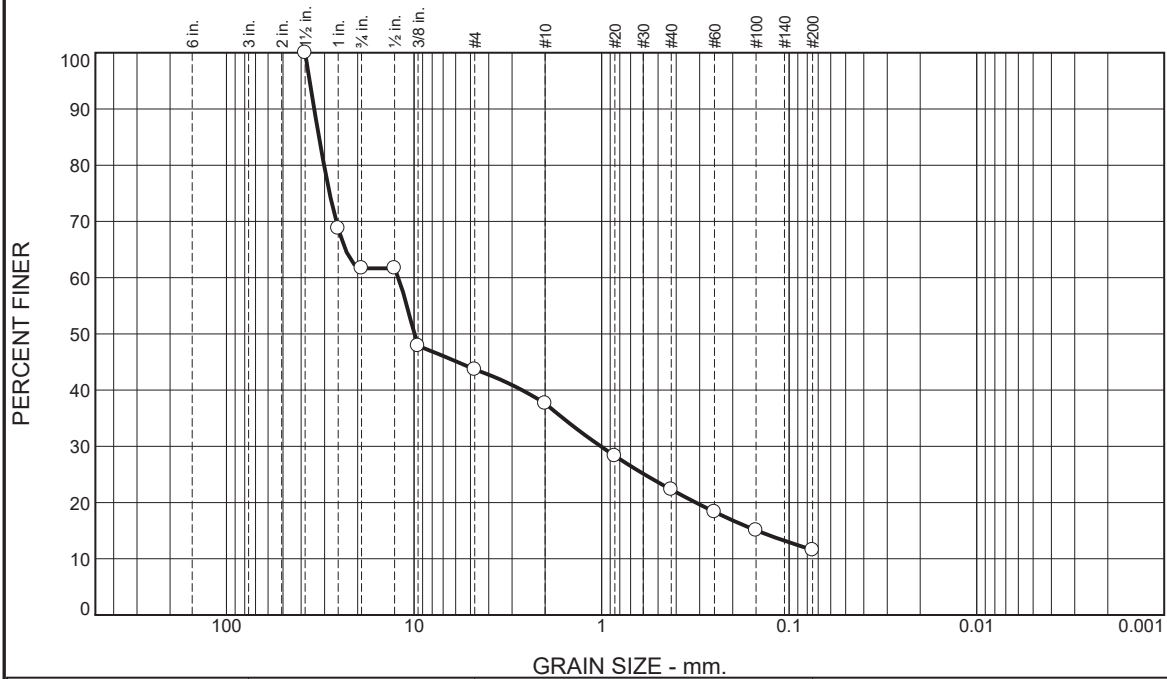
Client: GZA GeoEnvironmental

Project: Western Ave Bridge #5808, WIN 21672.00
Augusta, ME

Project No: 09.0025983.00

Figure S-2

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	38.3	18.0	6.1	15.3	10.7	11.6	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1.5"	100.0		
1"	68.8		
0.75"	61.7		
0.5"	61.7		
0.375"	47.9		
#4	43.7		
#10	37.6		
#20	28.3		
#40	22.3		
#60	18.3		
#100	15.1		
#200	11.6		

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

Coefficients

D₉₀= 34.0582 D₈₅= 32.1192 D₆₀= 12.1122
D₅₀= 9.9611 D₃₀= 1.0100 D₁₅= 0.1484
D₁₀= C_u= C_c=

Remarks

Date Received: 07.30.18 Date Tested: 08.01.18

Tested By: MN

Checked By: Rebecca Roth

Title: Laboratory Manager

Source of Sample: Borings Depth: 4-6'
Sample Number: BB-AI95-102 / 2D

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

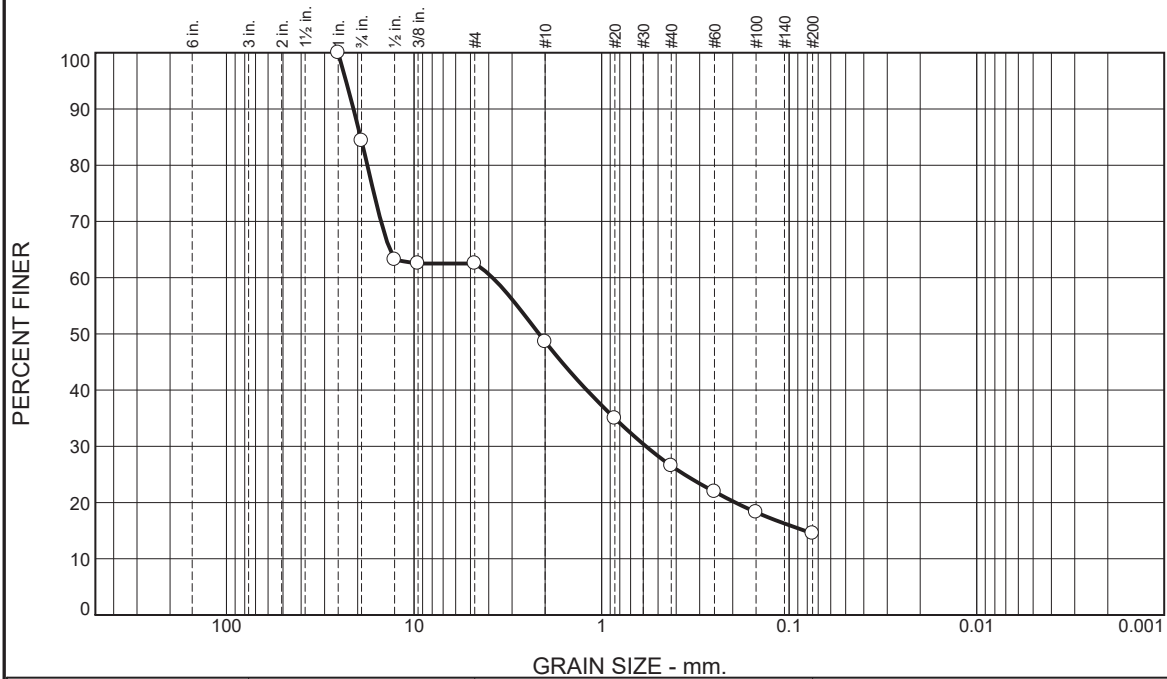
Client: GZA GeoEnvironmental

Project: Western Ave Bridge #5808, WIN 21672.00
Augusta, ME

Project No: 09.0025983.00

Figure S-3

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	15.7	21.8	13.9	22.1	12.0	14.5	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	84.3		
0.5"	63.2		
0.375"	62.5		
#4	62.5		
#10	48.6		
#20	35.0		
#40	26.5		
#60	21.9		
#100	18.3		
#200	14.5		

* (no specification provided)

Material Description

Grey Gravelly fine to coarse SAND, little Silt

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

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

Coefficients

D₉₀= 21.0202 D₈₅= 19.2616 D₆₀= 3.8159
D₅₀= 2.1556 D₃₀= 0.5802 D₁₅= 0.0826
D₁₀= C_u= C_c=

Remarks

Date Received: 07.30.18 Date Tested: 08.01.18

Tested By: MN

Checked By: Rebecca Roth

Title: Laboratory Manager

Source of Sample: Borings Depth: 0.5-2.5'
Sample Number: BB-AI95-103 / 1D

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

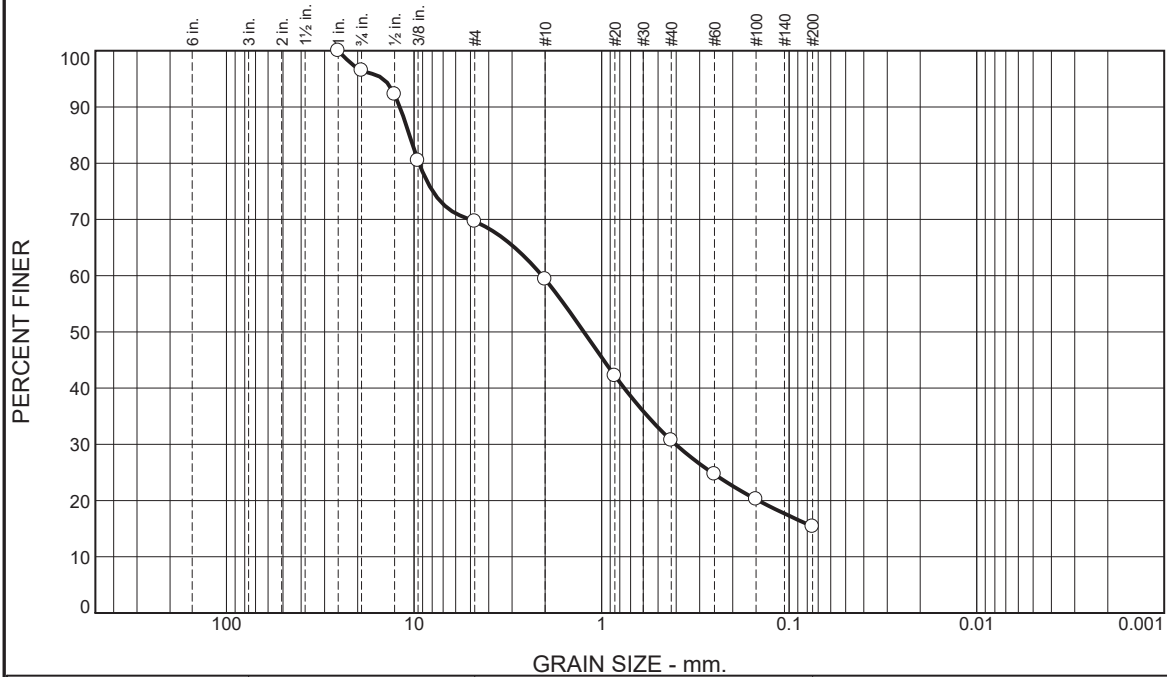
Client: GZA GeoEnvironmental

Project: Western Ave Bridge #5808, WIN 21672.00
Augusta, ME

Project No: 09.0025983.00

Figure S-4

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	3.5	26.9	10.3	28.6	15.3	15.4	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	96.5		
0.5"	92.3		
0.375"	80.5		
#4	69.6		
#10	59.3		
#20	42.2		
#40	30.7		
#60	24.7		
#100	20.2		
#200	15.4		

* (no specification provided)

Material Description

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

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

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

Coefficients

D₉₀= 11.8994 D₈₅= 10.5919 D₆₀= 2.0768
D₅₀= 1.2432 D₃₀= 0.4031 D₁₅=
D₁₀= C_u= C_c=

Remarks

Date Received: 07.30.18 Date Tested: 08.01.18

Tested By: MN

Checked By: Rebecca Roth

Title: Laboratory Manager

Source of Sample: Borings Depth: 5-7'
Sample Number: BB-AI95-103 / 2D

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

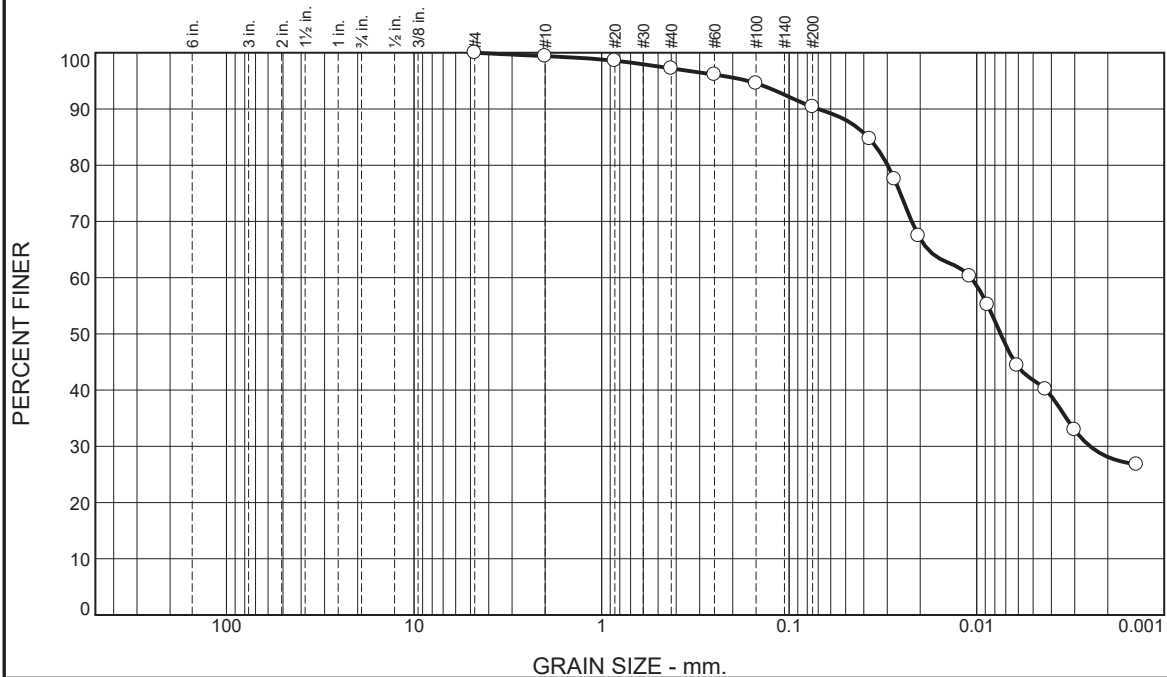
Client: GZA GeoEnvironmental

Project: Western Ave Bridge #5808, WIN 21672.00
Augusta, ME

Project No: 09.0025983.00

Figure S-5

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.6	2.2	6.8	62.3	28.1

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	100.0		
#10	99.4		
#20	98.6		
#40	97.2		
#60	96.1		
#100	94.6		
#200	90.4		
0.0372 mm.	84.7		
0.0274 mm.	77.5		
0.0204 mm.	67.4		
0.0109 mm.	60.3		
0.0088 mm.	55.2		
0.0061 mm.	44.4		
0.0043 mm.	40.1		
0.0030 mm.	33.0		
0.0014 mm.	26.8		

* (no specification provided)

Material Description

Grey SILT & CLAY, trace f-m Sand

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

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

Coefficients

D₉₀= 0.0698 D₈₅= 0.0380 D₆₀= 0.0107
D₅₀= 0.0075 D₃₀= 0.0025 D₁₅=
D₁₀= C_u= C_c=

Remarks

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

Date Received: 07.31.18 Date Tested: 08.03.18

Tested By: MN

Checked By: Rebecca Roth

Title: Laboratory Manager

Source of Sample: Borings Depth: 15-17'
Sample Number: BB-AI95-103 / 5D

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

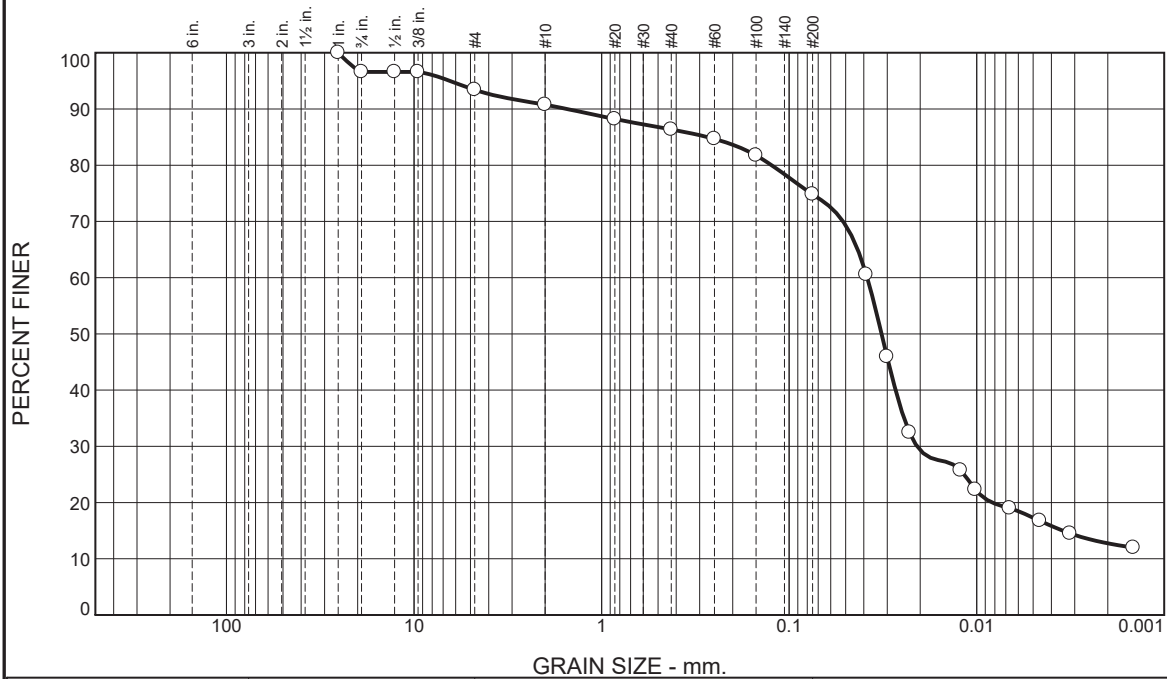
Client: GZA GeoEnvironmental

Project: Western Ave Bridge #5808, WIN 21672.00
Augusta, ME

Project No: 09.0025983.00

Figure S-6

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	3.4	3.2	2.6	4.4	11.6	62.0	12.8

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	96.6		
0.5"	96.6		
0.375"	96.6		
#4	93.4		
#10	90.8		
#20	88.2		
#40	86.4		
#60	84.7		
#100	81.7		
#200	74.8		
0.0388 mm.	60.6		
0.0301 mm.	46.0		
0.0229 mm.	32.5		
0.0122 mm.	25.7		
0.0102 mm.	22.3		
0.0067 mm.	19.0		
0.0046 mm.	16.8		
0.0032 mm.	14.5		
0.0015 mm.	12.0		

* (no specification provided)

Material Description

Brown Grey CLAYEY SILT, little f-c Sand, trace f-c Gravel

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

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

Coefficients

D₉₀= 1.5284 D₈₅= 0.2721 D₆₀= 0.0383
D₅₀= 0.0322 D₃₀= 0.0208 D₁₅= 0.0035
D₁₀= C_u= C_c=

Remarks

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

Date Received: 07.31.18 Date Tested: 08.03.18

Tested By: MN

Checked By: Rebecca Roth

Title: Laboratory Manager

Source of Sample: Borings Depth: 20-22'
Sample Number: BB-AI95-103 / 7D

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

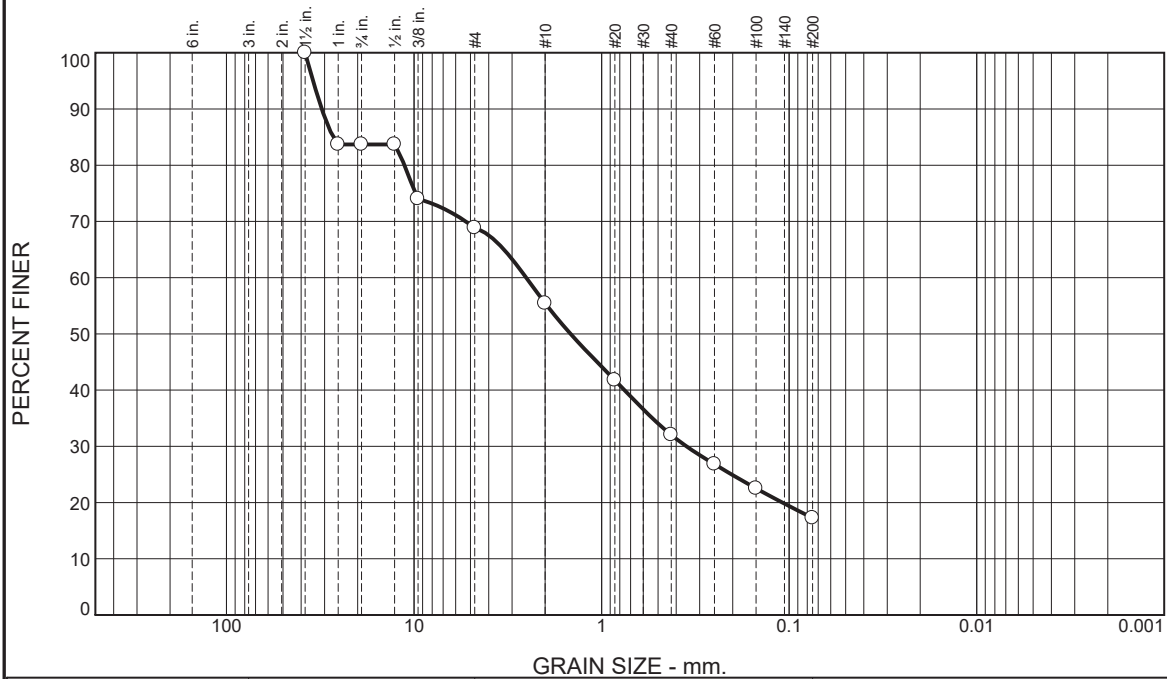
Client: GZA GeoEnvironmental

Project: Western Ave Bridge #5808, WIN 21672.00
Augusta, ME

Project No: 09.0025983.00

Figure S-7

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	16.3	14.9	13.4	23.4	14.8	17.2	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1.5"	100.0		
1"	83.7		
0.75"	83.7		
0.5"	83.7		
0.375"	74.0		
#4	68.8		
#10	55.4		
#20	41.8		
#40	32.0		
#60	26.8		
#100	22.5		
#200	17.2		

* (no specification provided)

Material Description

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

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

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

Coefficients

D₉₀= 31.0364 D₈₅= 26.9452 D₆₀= 2.5234
D₅₀= 1.4629 D₃₀= 0.3529 D₁₅=
D₁₀= C_u= C_c=

Remarks

Date Received: 07.30.18 Date Tested: 08.01.18

Tested By: MN

Checked By: Rebecca Roth

Title: Laboratory Manager

Source of Sample: Borings Depth: 12-14'
Sample Number: BB-AI95-103 / 4D

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental

Project: Western Ave Bridge #5808, WIN 21672.00
Augusta, ME

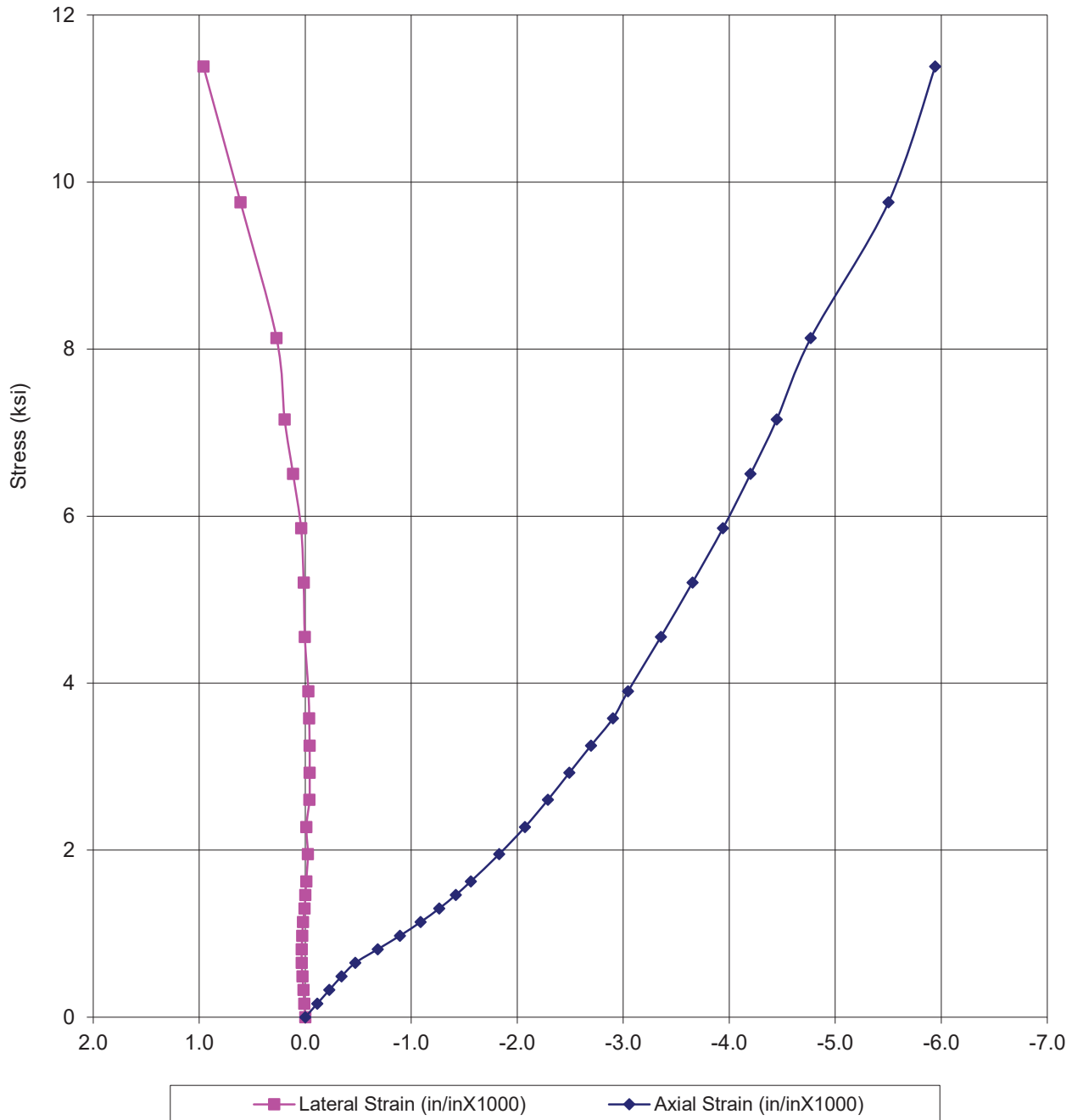
Project No: 09.0025983.00

Figure S-8

LABORATORY TESTING DATA SHEET

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-AI95-101	R-1	24.6-25.2	S-9		1.979	4.731	163.0				12119	0.59	1.52	0.02				SCHIST; fresh break
Notes: The majority of the Horizontal movement occurred at 50% of the Ultimate.																		
BB-AI95-103	R-1	33.5-34.2	S-10		1.979	4.786	174.0				12978	0.26	4.21	.20				SCHIST; fresh break
(1) Volume Determined By Measuring Dimensions					Notes	(3) PLD=Point Load (diametrical),					Notes	(5) Strain at Peak Deviator Stress						
(2) Determined by Measuring Dimensions and						PLA= Point Load (Axial) ST= Splitting Tensile						(6) Represents Secant Modulus at 50% of Total Failure Stress						
Weight of Saturated Sample						U= Unconfined Compressive Strength						(7) Represents Secant Poisson's Ratio at 50% of Total Failure Stress						
						(4) Taken at Peak Deviator Stress						(8) Estimated UCS from Table 1 of ASTM D5731 for NX cores (Is x 24)						

Western Ave Bridge
Augusta, ME



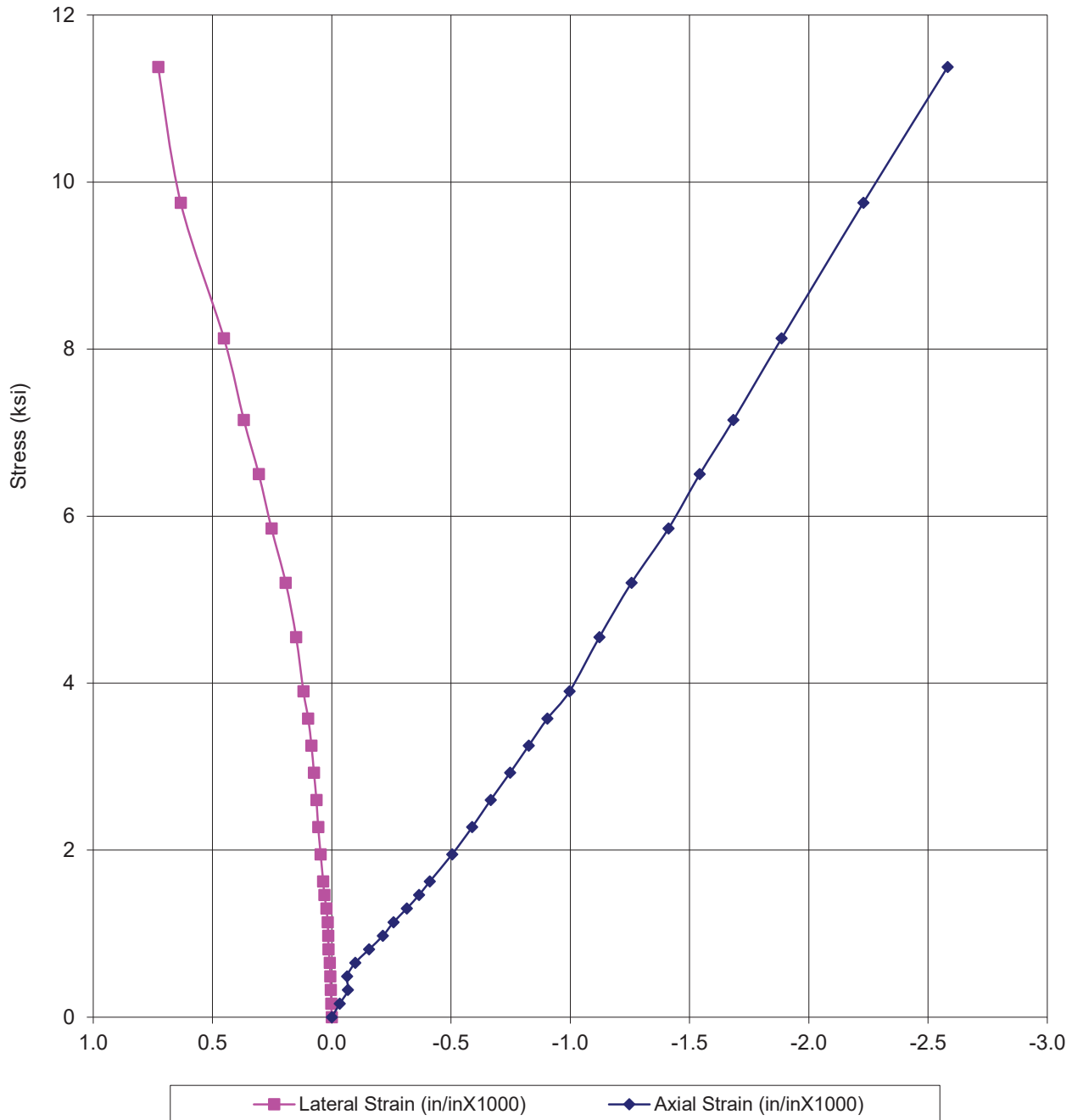
Rock Unconfined Compression Testing - ASTM D7012

Boring No. BB-AI95-101
Sample No. R1
Depth: 24.6-25.2'

File No. 09.0025983.00
Date: 7.31.18
Test No. U-9



**Western Ave Bridge
Augusta, ME**



Rock Unconfined Compression Testing - ASTM D7012

Boring No. BB-AI95-103
Sample No. R1
Depth: 33.5-34.2'

File No. 09.0025983.00
Date: 7.31.18
Test No. U-10





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: LCN
Collected By: L. Navarrete

Project Information:
Western Ave Bridge No. 5808
Augusta, ME
Project Number: 09.0025983.01
Summary Page: 1 of 1
Report Date: 12.19.22

LABORATORY TESTING DATA SHEET, Report No.: 7422-M-104

Boring	Sample No.	Depth (ft)	Laboratory No.	Identification Tests								Proctor / CBR / Permeability Tests								Laboratory Log and Soil Description
				As Rcvd Moisture Content %	LL %	PL %	Gravel %	Sand %	Fines %	Org. %	pH	Dry unit wt. (pcf)	Test Moisture Content %	γ_d MAX (pcf) W_{opt} (%)	γ_d MAX (pcf) W_{opt} (%) (Corr.)	Target Test Setup as % of Proctor	CBR @ 0.1"	CBR @ 0.2"	Permeability cm/sec	
				D2216	D4318		D6913			D2974	D4792			D1557						
BB-AI95-202	2D	2-4	22-S-4925	9.4			48.1	41.3	10.6											Brown f-c GRAVEL and f-c SAND, little Silt
BB-AI95-202	3D	4-5.3	22-S-4926	7.2			59.9	23.6	16.5											Brown f-c GRAVEL, some f-c Sand, little Silt
BB-AI95-203	4D	6-8	22-S-4927	10.4			38.5	38.7	22.8											Brown f-c SAND and f-c GRAVEL, some Silt
BB-AI95-203	5D	8-10	22-S-4928	15.0			9.1	52.1	38.8											Brown f-m SAND and SILT, trace f-c Gravel
BB-AI95-204	3D	5-7	22-S-4929	5.5			44.8	46.6	8.6											Brown f-c SAND and f-c GRAVEL, trace Silt
BB-AI95-205	2D	2.5-4	22-S-4930	5.4			73.3	23.5	3.2											Brown fine GRAVEL, some f-c Sand, trace Silt
BB-AI95-206	1D	0.4-2	22-S-4931	14.4			25.7	56.5	17.8											Brown f-c SAND, some f-c Gravel, little Silt
HB-AI95-201	3D	4-6	22-S-4932	7.7			18.0	55.8	26.2											Brown f-c SAND some Silt, little fine Gravel
HB-AI95-201	5D	8-10	22-S-4933	4.0			47.4	38.6	14.0											Brown f-c GRAVEL and f-c SAND, little Silt
HB-AI95-202	1D	1-3	22-S-4934	4.5	NV	NP	33.7	46.9	19.4											Brown f-c SAND, some f-c Gravel, little Silt
HB-AI95-202	6D	22-44	22-S-4935	18.6	25	17	19.8	15.5	64.7											Gray SILT & CLAY, little f-c Gravel, little f-c Sand
HB-AI95-202	8D	29-31	22-S-4936	7.4			49.2	35.6	15.2											Gray f-c GRAVEL and f-c SAND, little Silt

Date Received: 12.06.22

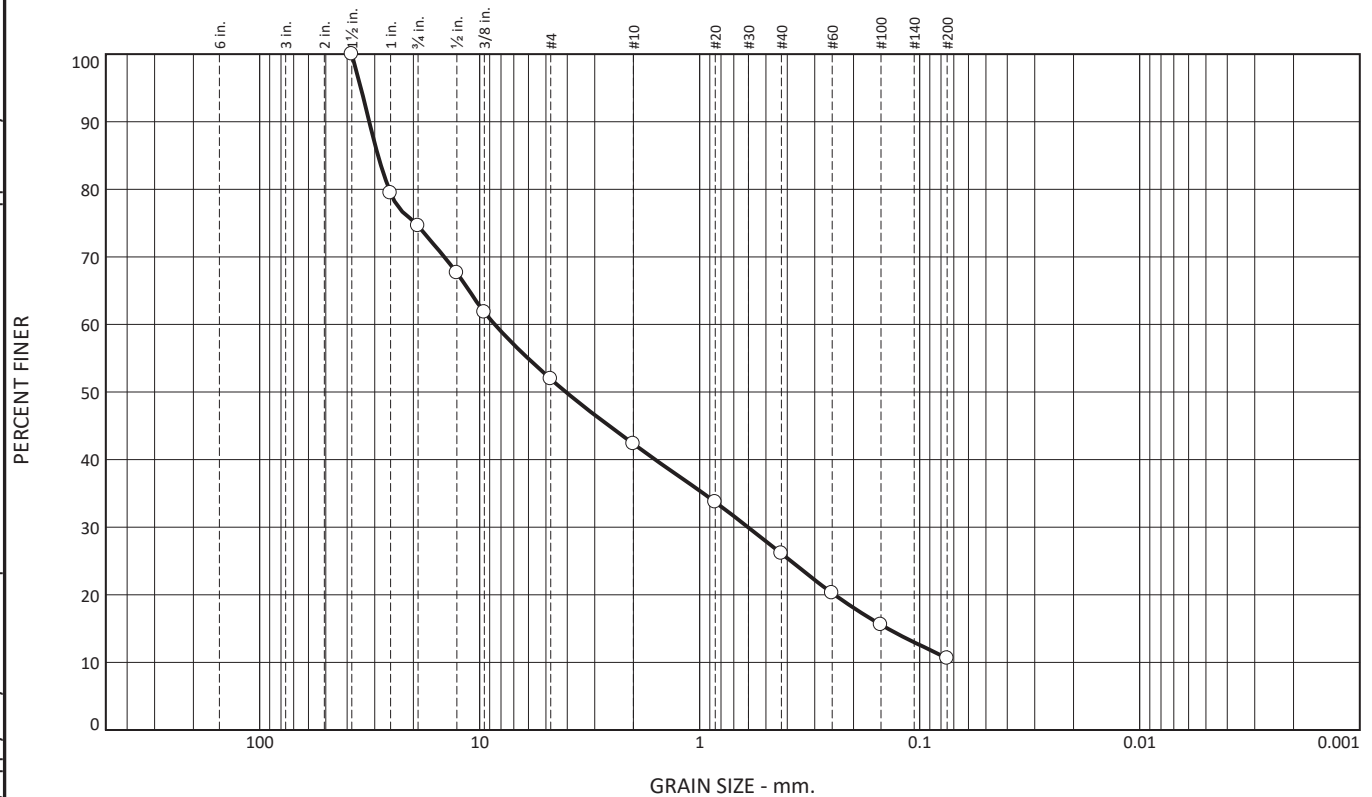
Reviewed By: 

Date Reviewed: 12.19.22

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

Particle Size Distribution Report

ASTM D6913



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	25.4	22.7	9.6	16.2	15.5	10.6	

Test Results (ASTM D6913)				
Sieve Size or Diam. (mm.)	Finer (%)	Spec. * (%)	Out of Spec. (%)	Pct. of Fines
1 1/2"	100.0			
1"	79.4			
3/4"	74.6			
1/2"	67.6			
3/8"	61.8			
#4	51.9			
#10	42.3			
#20	33.7			
#40	26.1			
#60	20.3			
#100	15.6			
#200	10.6			

* (no specification provided)

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

PL= NP **Atterberg Limits** LL= NV PI= NP

Coefficients
D₉₀= 31.7575 D₈₅= 28.9207 D₆₀= 8.5339
D₅₀= 4.0573 D₃₀= 0.6031 D₁₅= 0.1397
D₁₀= C_u= C_c=

Classification
USCS= GP-GM AASHTO= A-1-a

Test Remarks

Source of Sample: Fill Depth: 2-4'
Sample Number: BB-AI95-202 / 2D

Sample Date: 12.8.22

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Western Ave Bridge No. 5808
Augusta, ME

Project No: 09.0025983.01

Figure 22-S-4925

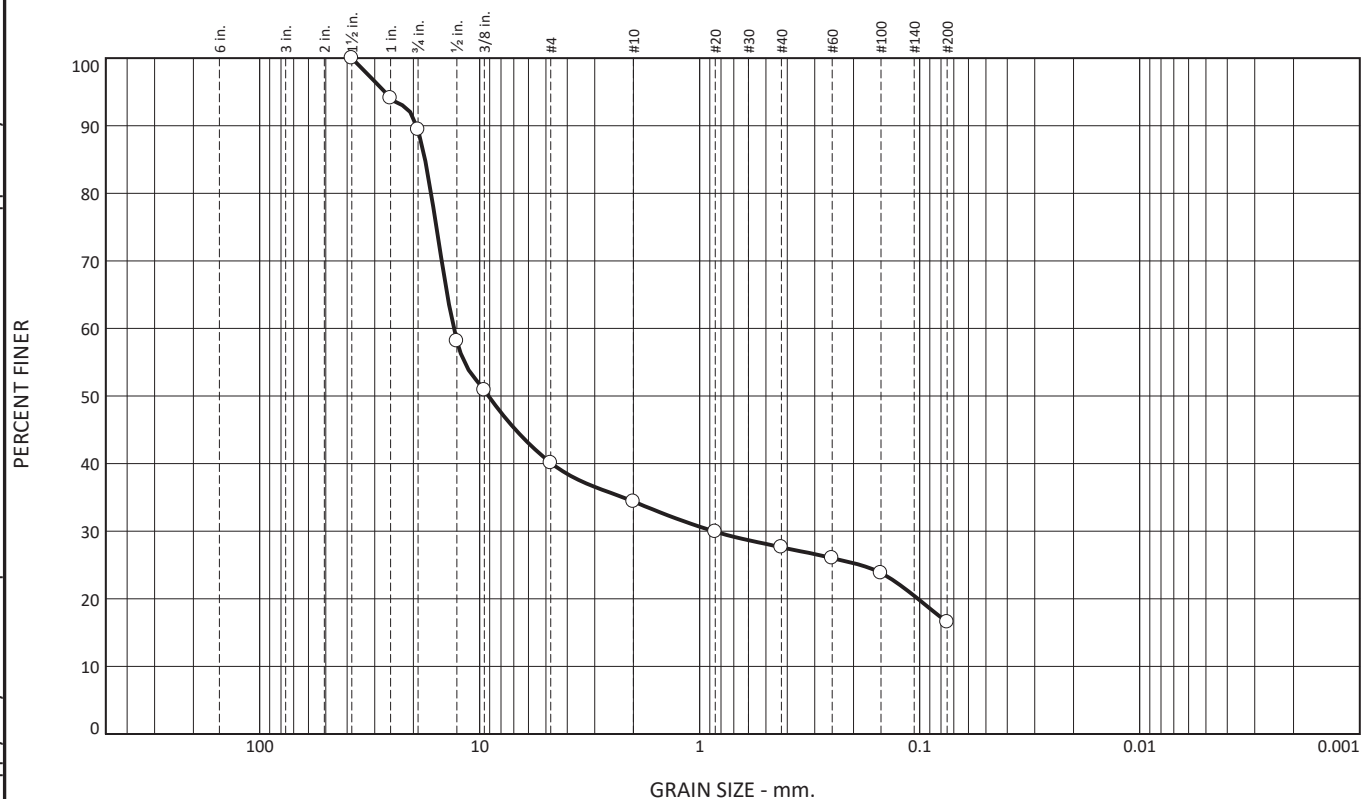
Tested By: RB

Checked By:

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

Particle Size Distribution Report

ASTM D6913



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	10.6	49.3	5.7	6.8	11.1	16.5	

Test Results (ASTM D6913)				
Sieve Size or Diam. (mm.)	Finer (%)	Spec. * (%)	Out of Spec. (%)	Pct. of Fines
1 1/2"	100.0			
1"	94.1			
3/4"	89.4			
1/2"	58.1			
3/8"	50.9			
#4	40.1			
#10	34.4			
#20	29.9			
#40	27.6			
#60	26.0			
#100	23.8			
#200	16.5			

* (no specification provided)

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

PL= NP **Atterberg Limits** LL= NV PI= NP
Coefficients
D₉₀= 19.3104 D₈₅= 17.7067 D₆₀= 13.1166
D₅₀= 9.1126 D₃₀= 0.8642 D₁₅=
D₁₀= C_u= C_c=
Classification
USCS= GM AASHTO= A-1-b
Test Remarks

Source of Sample: Fill Depth: 4-5.3'
Sample Number: BB-AI95-202 / 3D

Sample Date: 12.8.22

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Western Ave Bridge No. 5808
Augusta, ME

Project No: 09.0025983.01

Figure 22-S-4926

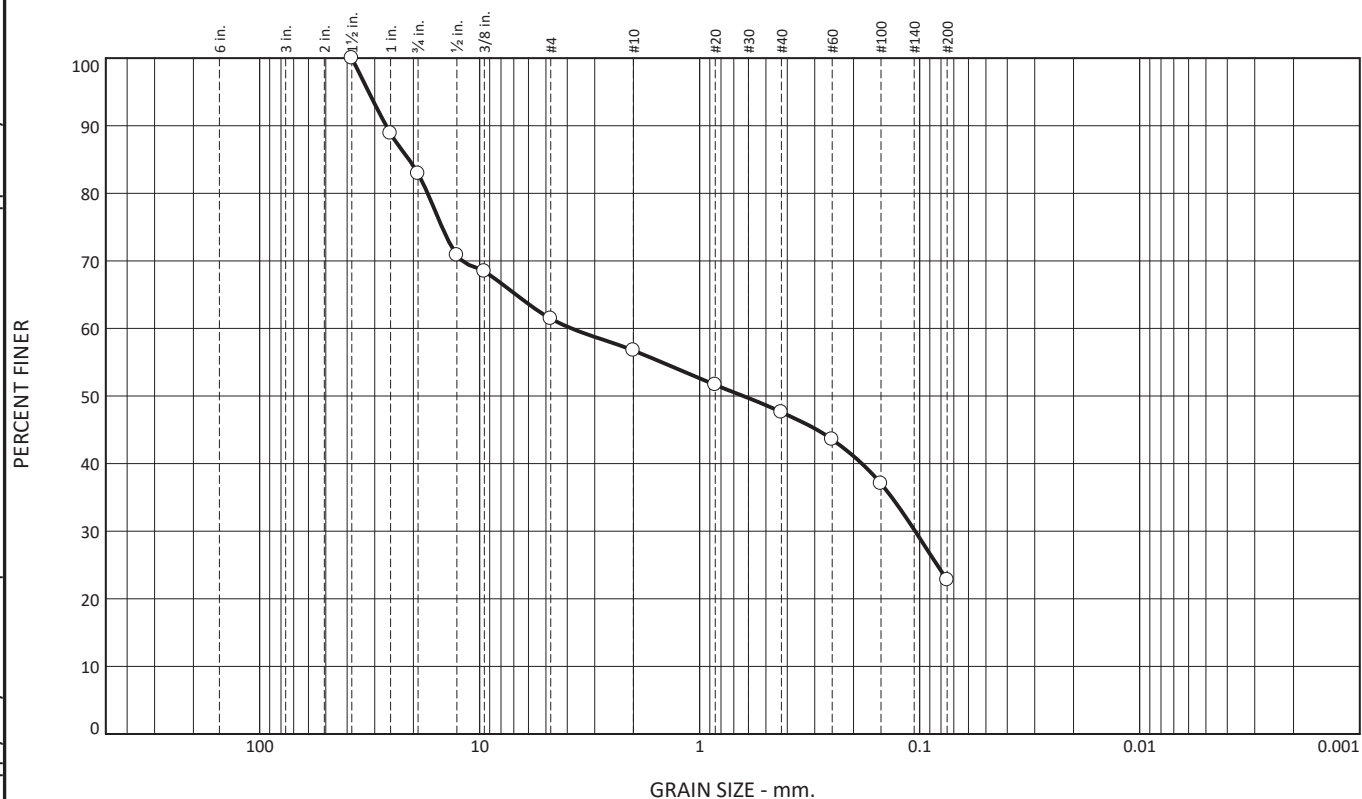
Tested By: RB

Checked By:

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

Particle Size Distribution Report

ASTM D6913



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	17.1	21.4	4.8	9.1	24.8	22.8	

Test Results (ASTM D6913)				
Sieve Size or Diam. (mm.)	Finer (%)	Spec. * (%)	Out of Spec. (%)	Pct. of Fines
1 1/2"	100.0			
1"	88.9			
3/4"	82.9			
1/2"	70.9			
3/8"	68.5			
#4	61.5			
#10	56.7			
#20	51.7			
#40	47.6			
#60	43.6			
#100	37.0			
#200	22.8			

* (no specification provided)

Material Description
Brown f-c SAND and f-c GRAVEL, some Silt

PL= NP **Atterberg Limits** LL= NV PI= NP

Coefficients

D₉₀= 26.5994 D₈₅= 20.9481 D₆₀= 3.8206
D₅₀= 0.6333 D₃₀= 0.1048 D₁₅=
D₁₀= C_u= C_c=

Classification

USCS= SM AASHTO= A-1-b

Test Remarks

Source of Sample: Fill Depth: 6-8'
Sample Number: BB-AI95-203 / 4D

Sample Date: 12.8.22

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Western Ave Bridge No. 5808
Augusta, ME

Project No: 09.0025983.01

Figure 22-S-4927

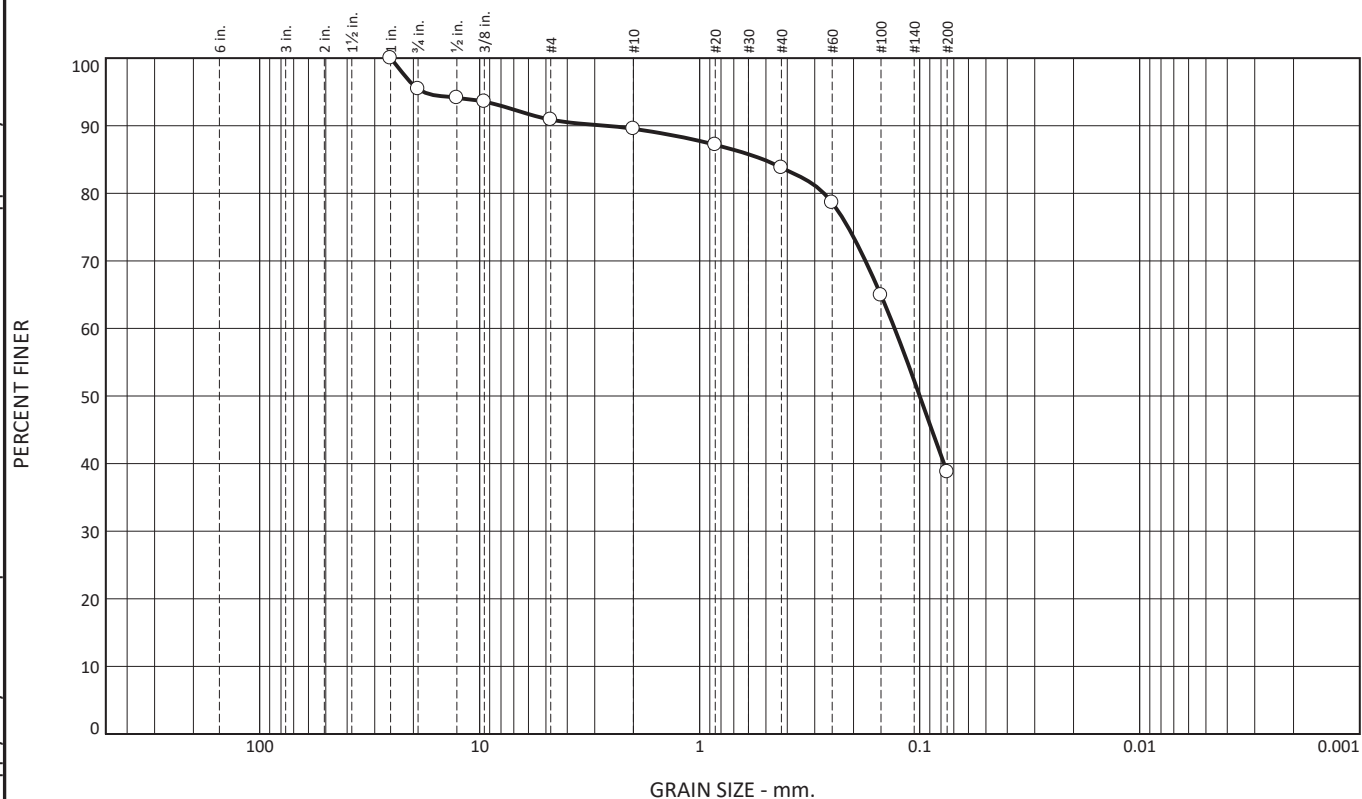
Tested By: RB

Checked By:

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

Particle Size Distribution Report

ASTM D6913



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	4.6	4.5	1.4	5.7	45.0	38.8	

Test Results (ASTM D6913)				
Sieve Size or Diam. (mm.)	Finer (%)	Spec. * (%)	Out of Spec. (%)	Pct. of Fines
1"	100.0			
3/4"	95.4			
1/2"	94.1			
3/8"	93.6			
#4	90.9			
#10	89.5			
#20	87.2			
#40	83.8			
#60	78.6			
#100	64.9			
#200	38.8			

* (no specification provided)

Material Description
Brown f-m SAND and SILT, trace f-c Gravel

PL= NP **Atterberg Limits** LL= NV PI= NP
Coefficients
D₉₀= 2.6784 D₈₅= 0.5136 D₆₀= 0.1299
D₅₀= 0.1000 D₃₀= D₁₅=
D₁₀= C_u= C_c=

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

Test Remarks
Sample visually classified as non-plastic.

Source of Sample: Fill Depth: 8-10'
Sample Number: BB-AI95-203 / 5D

Sample Date: 12.8.22

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Western Ave Bridge No. 5808
Augusta, ME

Project No: 09.0025983.01

Figure 22-S-4928

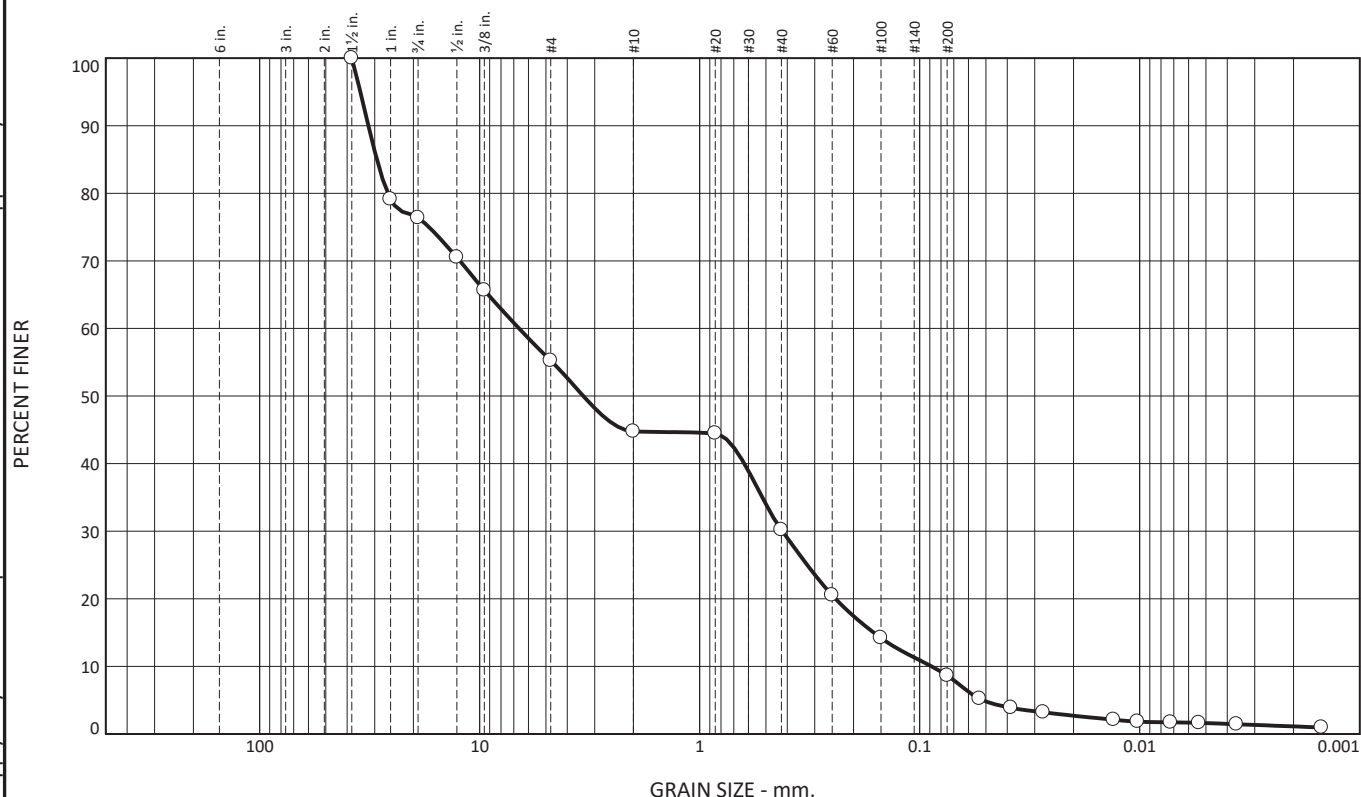
Tested By: RB

Checked By:

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

Particle Size Distribution Report

ASTM D6913 and ASTM D7928



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	23.6	21.2	10.4	14.6	21.6	7.5	1.1

Test Results (ASTM D6913 and ASTM D7928)				
Sieve Size or Diam. (mm.)	Finer (%)	Spec. * (%)	Out of Spec. (%)	Pct. of Fines
1 1/2"	100.0			
1"	79.1			
3/4"	76.4			
1/2"	70.5			
3/8"	65.6			
#4	55.2			
#10	44.8			
#20	44.5			99.3
#40	30.2			67.5
#60	20.5			45.9
#100	14.2			31.7
#200	8.6			19.3
0.0535 mm.	5.2			
0.0385 mm.	3.9			
0.0275 mm.	3.2			
0.0131 mm.	2.1			
0.0102 mm.	1.8			
0.0072 mm.	1.7			
0.0054 mm.	1.6			
0.0036 mm.	1.4			
0.0015 mm.	1.0			

* (no specification provided)

Material Description
Brown f-c SAND, and f-c GRAVEL, trace Silt

PL= NP **Atterberg Limits** LL= NV PI= NP
Coefficients
D₉₀= 32.0327 D₈₅= 29.3177 D₆₀= 6.6232
D₅₀= 3.3805 D₃₀= 0.4207 D₁₅= 0.1619
D₁₀= 0.0885 C_u= 74.85 C_c= 0.30

Classification
USCS= SP-SM AASHTO= A-1-a

Test Remarks
Sample visually classified as non-plastic.

Source of Sample: Fill Depth: 5-7'
Sample Number: BB-AI95-204 / 3D

Sample Date: 12.9.22

Thielsch Engineering Inc.

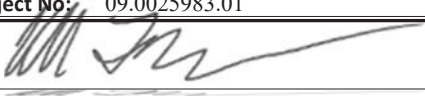
Cranston, RI

Client: GZA GeoEnvironmental
Project: Western Ave Bridge No. 5808
Augusta, ME

Project No: 09.0025983.01

Figure 22-S-4929

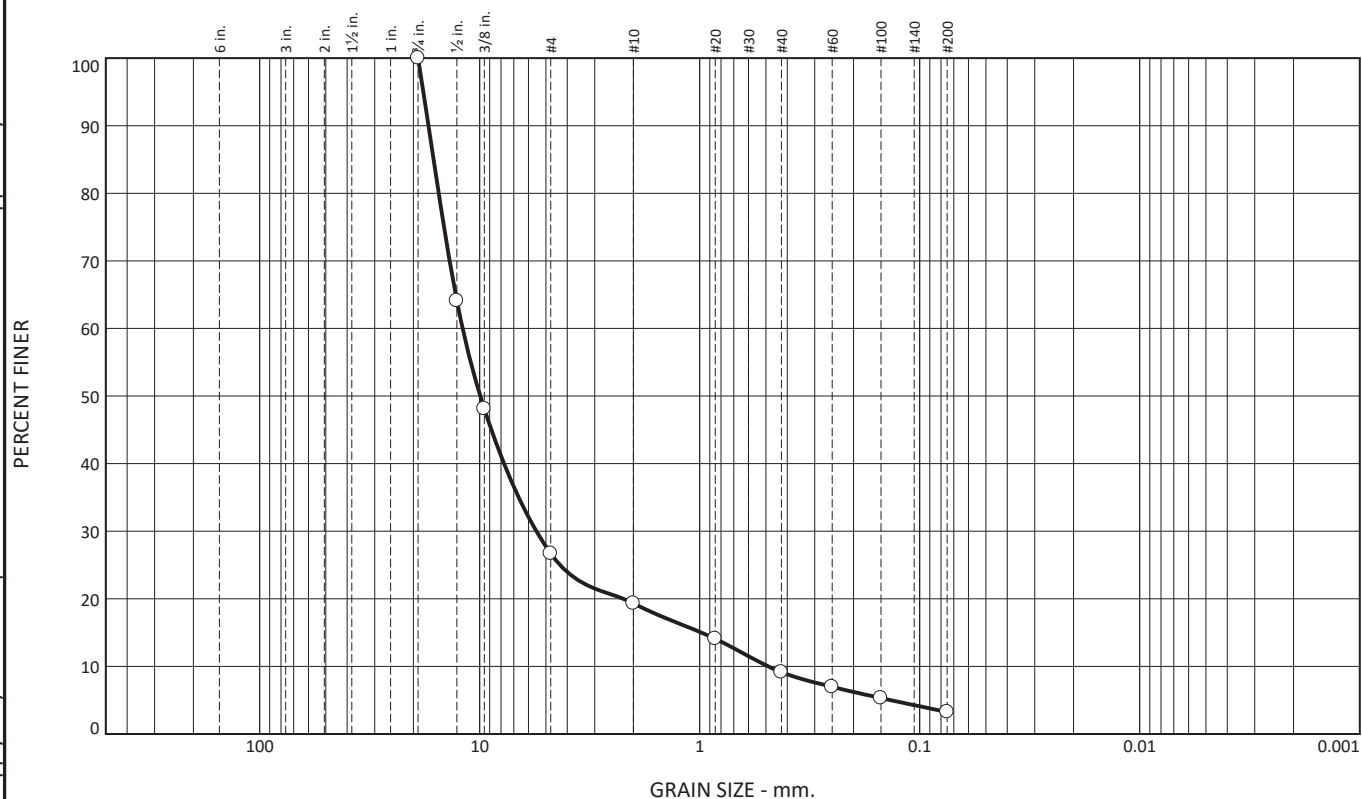
Tested By: RB / JT / RB

Checked By: 

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

Particle Size Distribution Report

ASTM D6913



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	73.3	7.4	10.2	5.9	3.2	

Test Results (ASTM D6913)				
Sieve Size or Diam. (mm.)	Finer (%)	Spec. * (%)	Out of Spec. (%)	Pct. of Fines
3/4"	100.0			
1/2"	64.1			
3/8"	48.1			
#4	26.7			
#10	19.3			
#20	14.1			
#40	9.1			
#60	7.0			
#100	5.3			
#200	3.2			

* (no specification provided)

Material Description
Brown fine GRAVEL, some f-c Sand, trace Silt

PL= NP **Atterberg Limits** LL= NV PI= NP

Coefficients
D₉₀= 17.0829 D₈₅= 16.1974 D₆₀= 11.9548
D₅₀= 9.9381 D₃₀= 5.5265 D₁₅= 0.9835
D₁₀= 0.4882 C_u= 24.49 C_c= 5.23

Classification
USCS= GP AASHTO= A-1-a

Test Remarks

Source of Sample: Fill Depth: 2.5-4'
Sample Number: BB-AI95-205 / 2D

Sample Date: 12.8.22

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Western Ave Bridge No. 5808
Augusta, ME

Project No: 09.0025983.01

Figure 22-S-4930

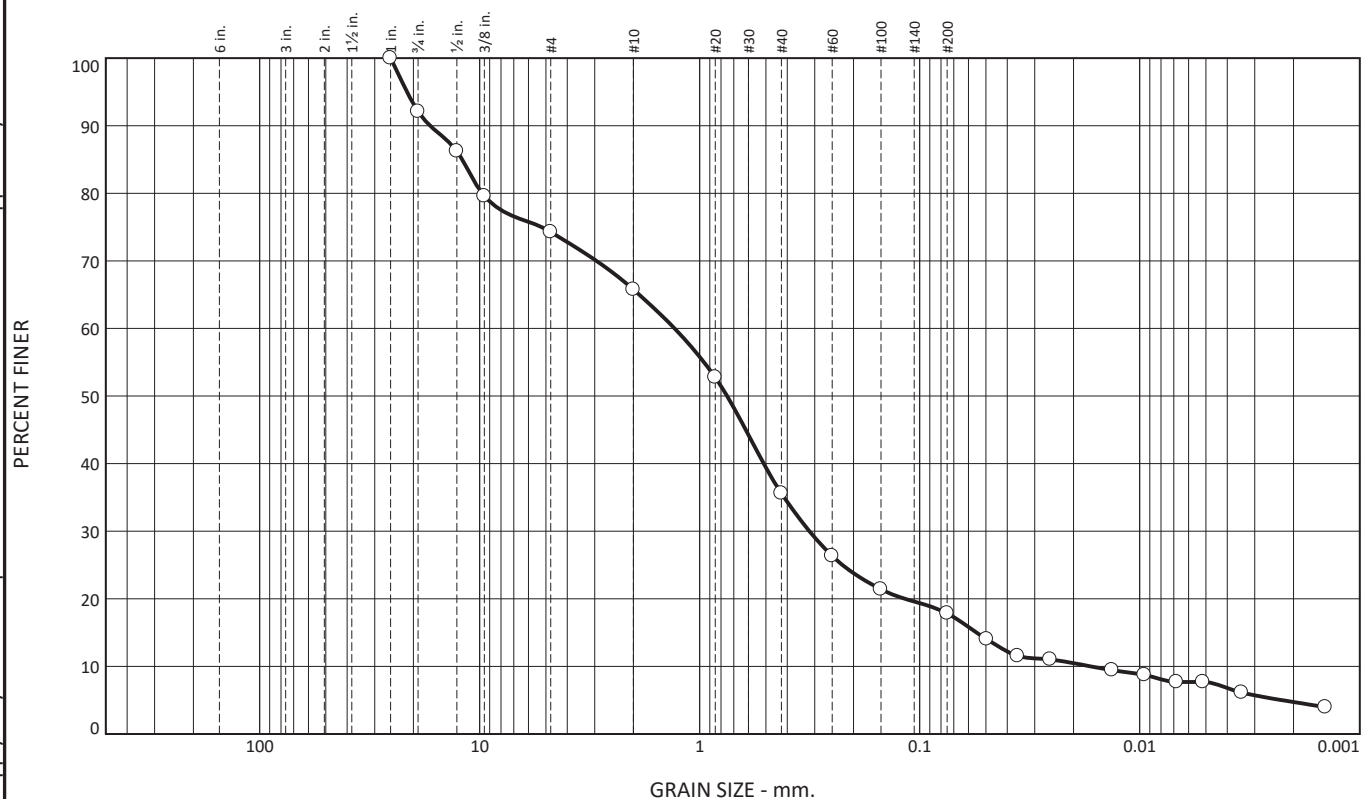
Tested By: RB

Checked By:

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

Particle Size Distribution Report

ASTM D6913 and ASTM D7928



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	7.9	17.8	8.5	30.2	17.8	13.1	4.7

Test Results (ASTM D6913 and ASTM D7928)				
Sieve Size or Diam. (mm.)	Finer (%)	Spec. * (%)	Out of Spec. (%)	Pct. of Fines
1"	100.0			
3/4"	92.1			
1/2"	86.2			
3/8"	79.6			
#4	74.3			
#10	65.8			88.5
#20	52.8			71.0
#40	35.6			47.9
#60	26.3			35.5
#100	21.4			28.8
#200	17.8			24.0
0.0495 mm.	14.0			
0.0359 mm.	11.6			
0.0255 mm.	11.0			
0.0134 mm.	9.4			
0.0095 mm.	8.7			
0.0068 mm.	7.7			
0.0052 mm.	7.7			
0.0034 mm.	6.1			
0.0014 mm.	4.0			

* (no specification provided)

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

PL= NP **Atterberg Limits** LL= NV PI= NP
Coefficients
D₉₀= 16.8119 D₈₅= 12.0030 D₆₀= 1.2994
D₅₀= 0.7504 D₃₀= 0.3172 D₁₅= 0.0552
D₁₀= 0.0167 C_u= 77.82 C_c= 4.64
Classification
USCS= SM AASHTO= A-1-b

Test Remarks

Source of Sample: Fill Depth: 0.4-2'
Sample Number: BB-AI95-206 / 1D

Sample Date: 12.12.22

Thielsch Engineering Inc.

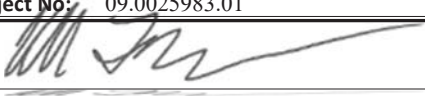
Cranston, RI

Client: GZA GeoEnvironmental
Project: Western Ave Bridge No. 5808
Augusta, ME

Project No: 09.0025983.01

Figure 22-S-4931

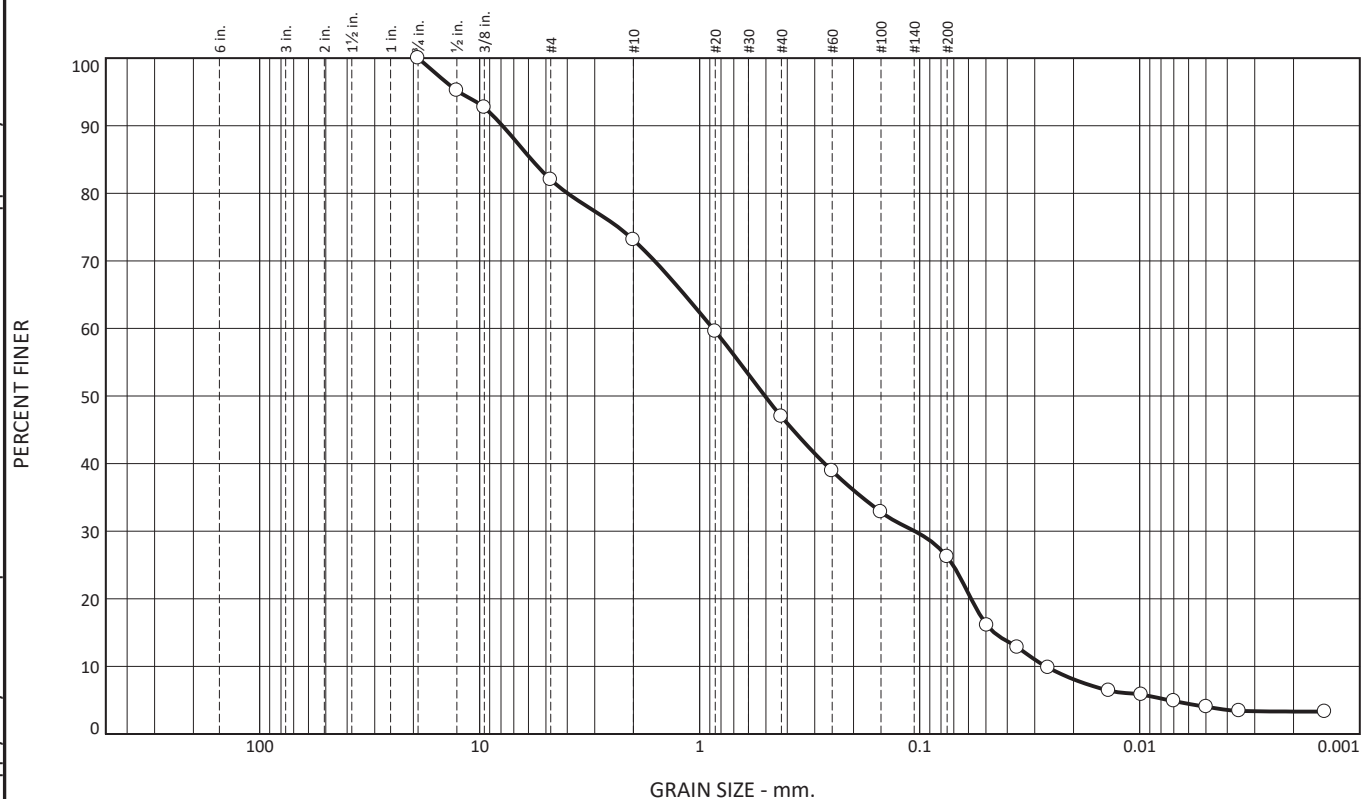
Tested By: SL / RB / JT / SL

Checked By: 

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

Particle Size Distribution Report

ASTM D6913 and ASTM D7928



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	18.0	8.9	26.1	20.8	22.9	3.3

Test Results (ASTM D6913 and ASTM D7928)				
Sieve Size or Diam. (mm.)	Finer (%)	Spec. * (%)	Out of Spec. (%)	Pct. of Fines
3/4"	100.0			
1/2"	95.2			
3/8"	92.7			
#4	82.0			
#10	73.1			89.1
#20	59.6			72.6
#40	47.0			57.3
#60	38.9			47.5
#100	32.8			40.0
#200	26.2			32.0
0.0494 mm.	16.1			
0.0359 mm.	12.8			
0.0261 mm.	9.8			
0.0138 mm.	6.4			
0.0098 mm.	5.8			
0.0070 mm.	4.9			
0.0050 mm.	4.0			
0.0035 mm.	3.4			
0.0014 mm.	3.3			

* (no specification provided)

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

PL= NP **Atterberg Limits** LL= NV PI= NP
Coefficients
D₉₀= 7.8746 D₈₅= 5.8173 D₆₀= 0.8716
D₅₀= 0.5051 D₃₀= 0.1052 D₁₅= 0.0459
D₁₀= 0.0268 C_u= 32.58 C_c= 0.47
Classification
USCS= SM AASHTO= A-2-4(0)

Test Remarks

Source of Sample: Fill Depth: 4-6'
Sample Number: HB-AI95-201 / 3D

Sample Date: 12.14.22

Thielsch Engineering Inc.

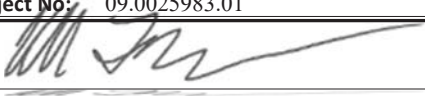
Cranston, RI

Client: GZA GeoEnvironmental
Project: Western Ave Bridge No. 5808
Augusta, ME

Project No: 09.0025983.01

Figure 22-S-4932

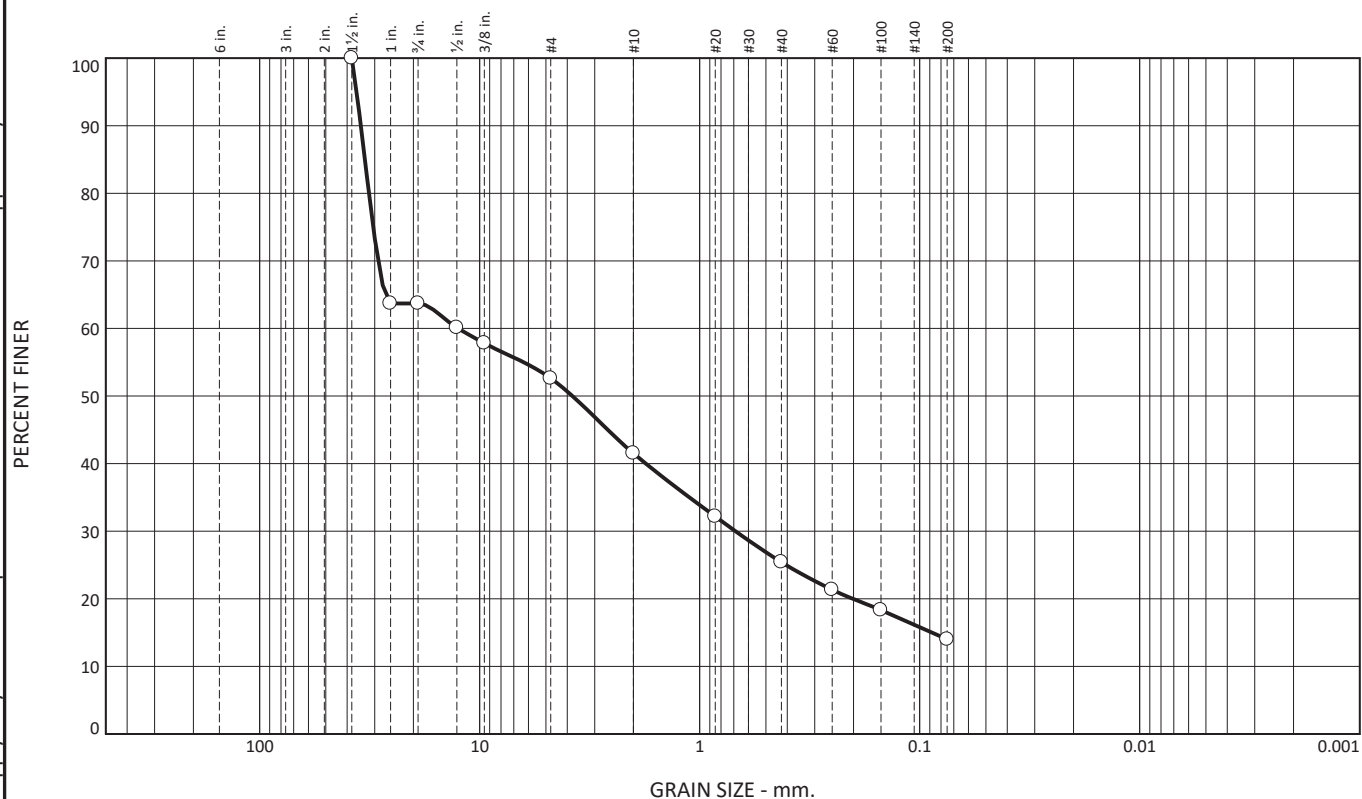
Tested By: SL / RB / JT / RB

Checked By: 

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

Particle Size Distribution Report

ASTM D6913



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	36.3	11.1	11.1	16.1	11.4	14.0	

Test Results (ASTM D6913)				
Sieve Size or Diam. (mm.)	Finer (%)	Spec. * (%)	Out of Spec. (%)	Pct. of Fines
1 1/2"	100.0			
1"	63.7			
3/4"	63.7			
1/2"	60.1			
3/8"	57.8			
#4	52.6			
#10	41.5			
#20	32.2			
#40	25.4			
#60	21.3			
#100	18.3			
#200	14.0			

* (no specification provided)

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

PL= NP **Atterberg Limits** LL= NV PI= NP

Coefficients
D₉₀= 34.6561 D₈₅= 33.2243 D₆₀= 12.5415
D₅₀= 3.7865 D₃₀= 0.6872 D₁₅= 0.0880
D₁₀= C_u= C_c=

USCS= GM **Classification** AASHTO= A-1-a

Test Remarks

Source of Sample: Fill Depth: 8-10'
Sample Number: HB-AI95-201 / 5D

Sample Date: 12.8.22

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Western Ave Bridge No. 5808
Augusta, ME

Project No: 09.0025983.01

Figure 22-S-4933

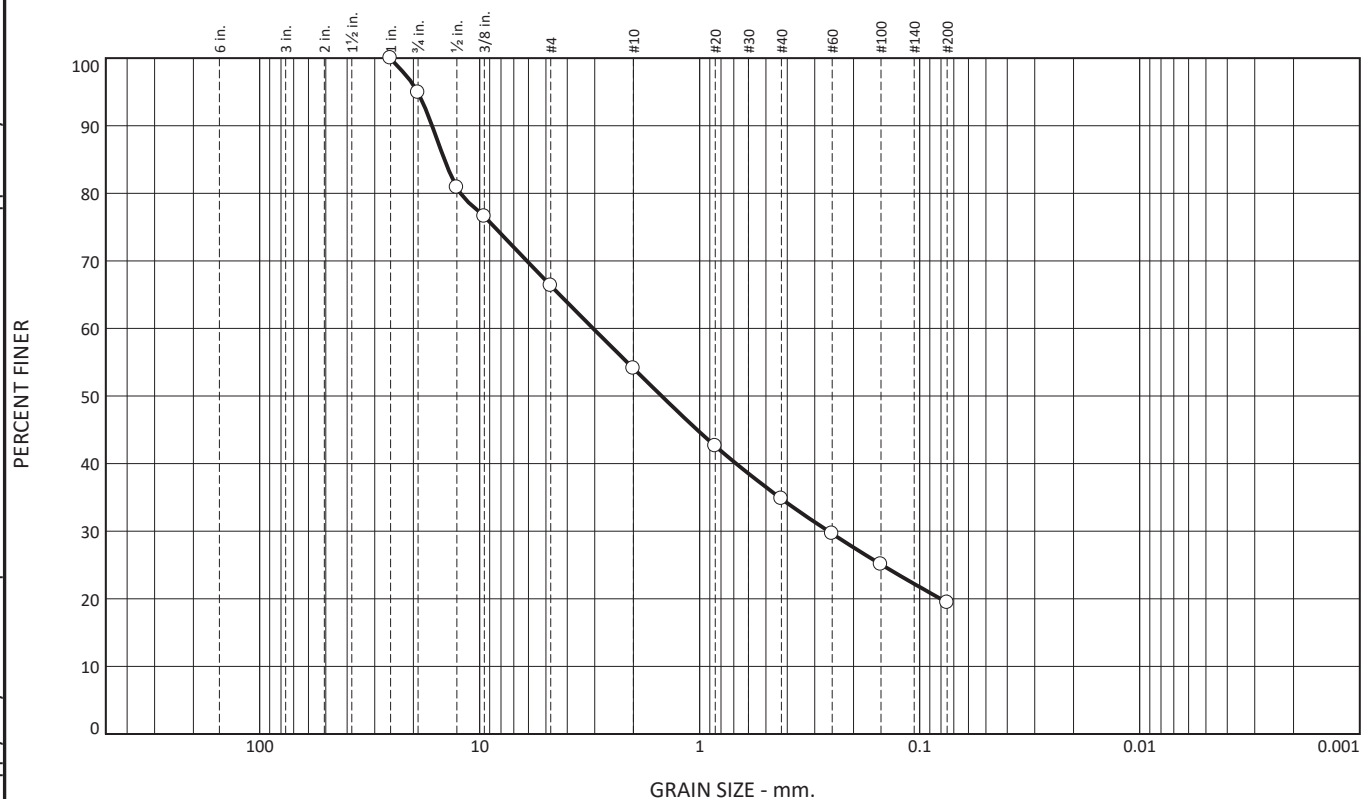
Tested By: RB

Checked By:

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

Particle Size Distribution Report

ASTM D6913



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	5.1	28.6	12.2	19.3	15.4	19.4	

Test Results (ASTM D6913)				
Sieve Size or Diam. (mm.)	Finer (%)	Spec. * (%)	Out of Spec. (%)	Pct. of Fines
1"	100.0			
3/4"	94.9			
1/2"	80.9			
3/8"	76.6			
#4	66.3			
#10	54.1			
#20	42.6			
#40	34.8			
#60	29.6			
#100	25.1			
#200	19.4			

* (no specification provided)

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

PL= NP **Atterberg Limits** LL= NV PI= NP
Coefficients
D₉₀= 16.4576 D₈₅= 14.4615 D₆₀= 3.0509
D₅₀= 1.4875 D₃₀= 0.2598 D₁₅=
D₁₀= C_u= C_c=
Classification
USCS= SM AASHTO= A-1-b
Test Remarks

Source of Sample: Fill Depth: 1-3'
Sample Number: HB-AI95-202 / 1D

Sample Date: 12.2.22

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Western Ave Bridge No. 5808
Augusta, ME

Project No: 09.0025983.01

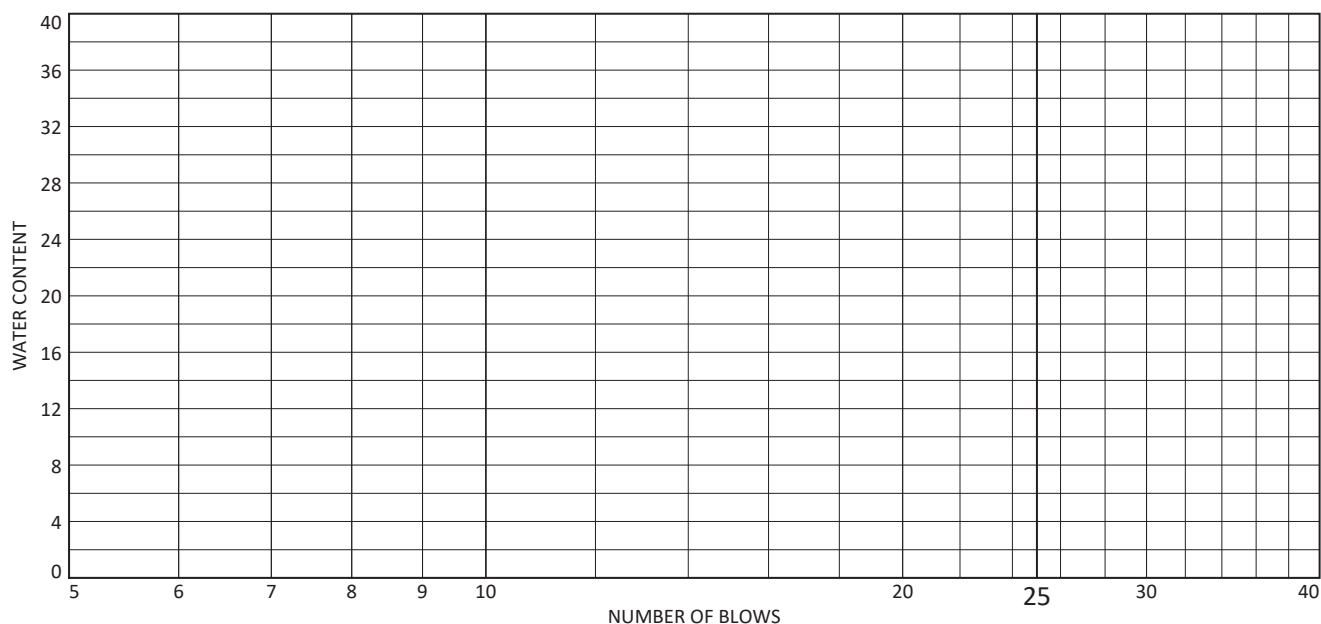
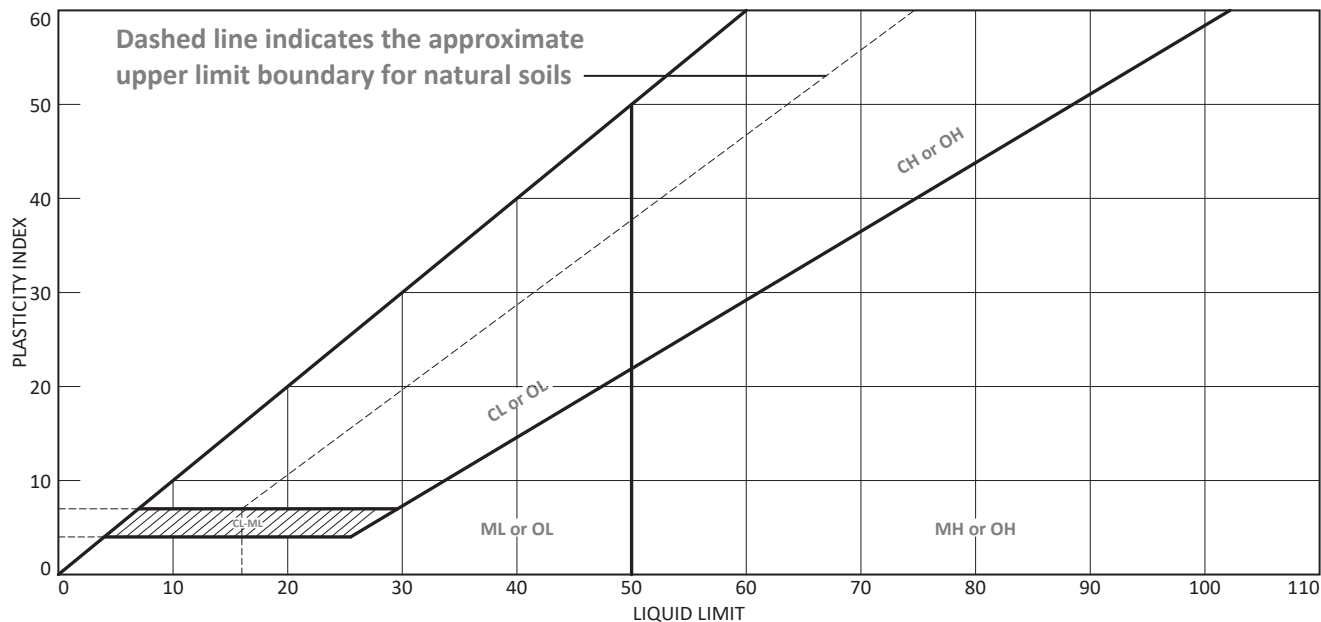
Figure 22-S-4934

Tested By: AL / AV / RB

Checked By:

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

LIQUID AND PLASTIC LIMITS TEST REPORT



MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
Brown f-c SAND, some f-c Gravel, little Silt	NV	NP	NP	34.8	19.4	SM

Project No. 09.0025983.01 **Client:** GZA GeoEnvironmental
Project: Western Ave Bridge No. 5808
Augusta, ME
Source of Sample: Fill **Depth:** 1-3'
Sample Number: HB-AI95-202 / 1D

Thielsch Engineering Inc.

Cranston, RI

Remarks:

- Sample classified as non-plastic and non-viscous. Sample could not roll past 1/4" and could not achieve more than 25 blows.

Figure 22-L-4934

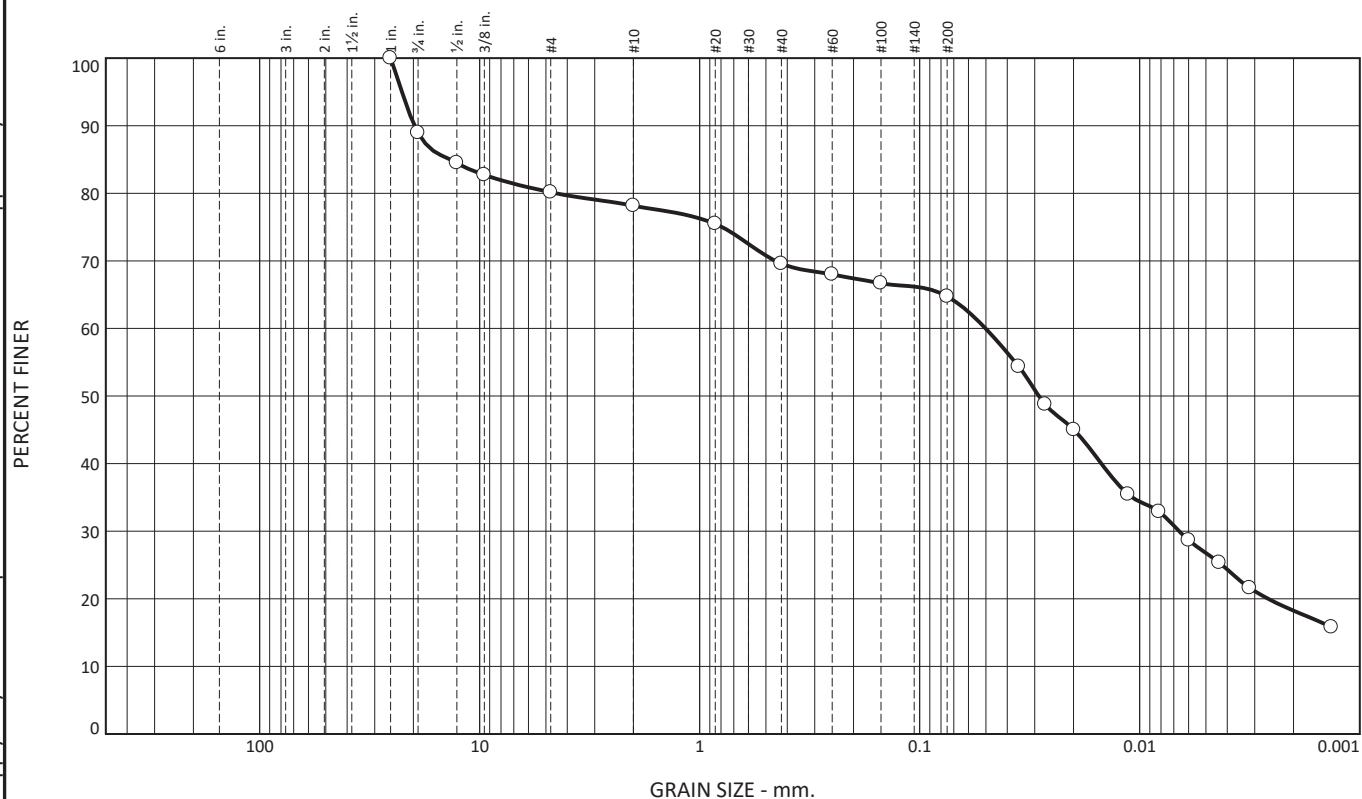
Tested By: RR

Checked By: 

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

Particle Size Distribution Report

ASTM D6913 and ASTM D7928



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	11.1	8.7	2.0	8.6	4.9	46.4	18.3

Test Results (ASTM D6913 and ASTM D7928)				
Sieve Size or Diam. (mm.)	Finer (%)	Spec. * (%)	Out of Spec. (%)	Pct. of Fines
1"	100.0			
3/4"	88.9			
1/2"	84.5			
3/8"	82.7			
#4	80.2			
#10	78.2			
#20	75.5			
#40	69.6			
#60	68.0			97.7
#100	66.7			95.8
#200	64.7			93.0
0.0354 mm.	54.4			
0.0269 mm.	48.8			
0.0199 mm.	45.0			
0.0113 mm.	35.5			
0.0082 mm.	32.9			
0.0060 mm.	28.7			
0.0043 mm.	25.3			
0.0032 mm.	21.6			
0.0013 mm.	15.8			

* (no specification provided)

Material Description
Gray SILT & CLAY, little f-c Gravel, little f-c Sand

PL= 17 **Atterberg Limits** LL= 25 PI= 8
Coefficients
D₉₀= 19.7873 D₈₅= 13.6023 D₆₀= 0.0502
D₅₀= 0.0288 D₃₀= 0.0066 D₁₅=
D₁₀= C_u= C_c=

USCS= CL **Classification** AASHTO= A-4(3)

Test Remarks

Source of Sample: Fill Depth: 22-24'
Sample Number: HB-AI95-202 / 6D

Sample Date: 12.9.22

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Western Ave Bridge No. 5808
Augusta, ME

Project No: 09.0025983.01

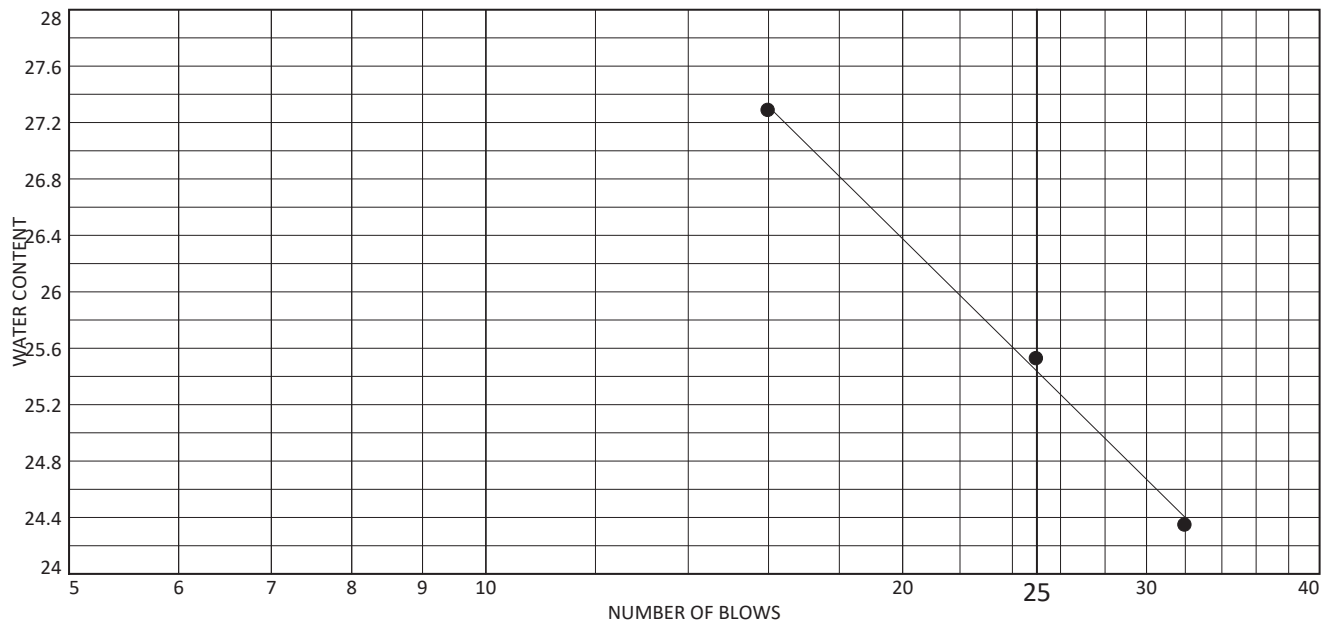
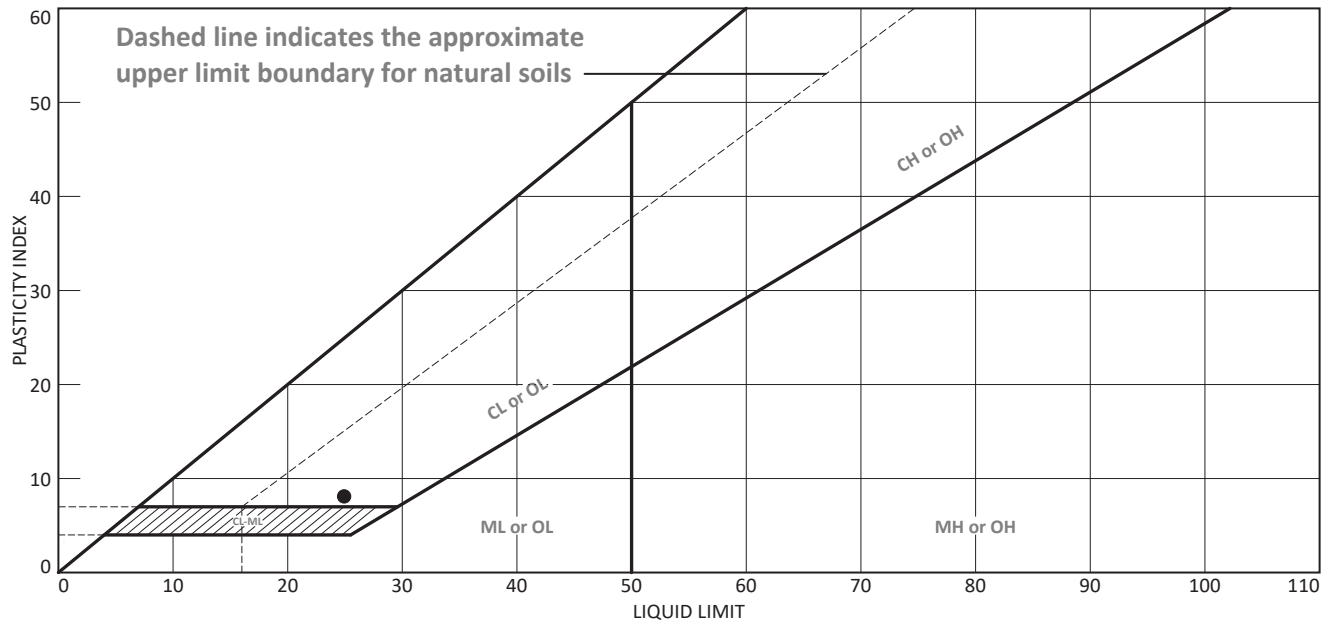
Figure 22-S-4935

Tested By: SL / RR / RB / RR

Checked By:

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

LIQUID AND PLASTIC LIMITS TEST REPORT



MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
Gray SILT & CLAY, little f-c Gravel, little f-c Sand	25	17	8	69.6	64.7	CL

Project No. 09.0025983.01 **Client:** GZA GeoEnvironmental
Project: Western Ave Bridge No. 5808
Augusta, ME
Source of Sample: Fill **Depth:** 22-24'
Sample Number: HB-AI95-202 / 6D

Thielsch Engineering Inc.

Cranston, RI

Remarks:

Figure 22-L-4935

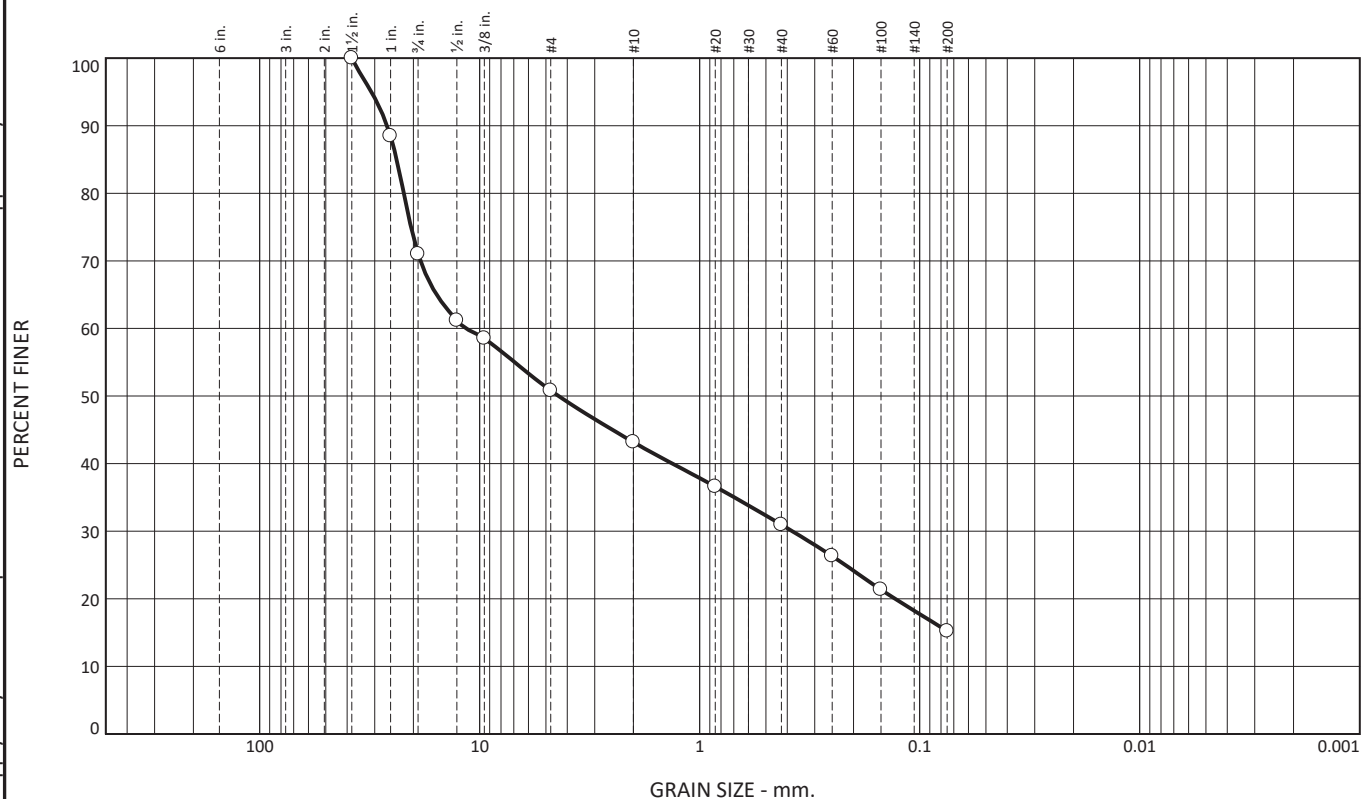
Tested By: RR

Checked By: 

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

Particle Size Distribution Report

ASTM D6913



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	29.0	20.2	7.6	12.2	15.8	15.2	

Test Results (ASTM D6913)				
Sieve Size or Diam. (mm.)	Finer (%)	Spec. * (%)	Out of Spec. (%)	Pct. of Fines
1 1/2"	100.0			
1"	88.5			
3/4"	71.0			
1/2"	61.2			
3/8"	58.5			
#4	50.8			
#10	43.2			
#20	36.6			
#40	31.0			
#60	26.3			
#100	21.4			
#200	15.2			

* (no specification provided)

Material Description
Gray f-c GRAVEL and f-c SAND, little Silt

PL= NP **Atterberg Limits** LL= NV PI= NP

Coefficients

D₉₀= 26.3494 D₈₅= 23.8235 D₆₀= 11.5403
D₅₀= 4.3740 D₃₀= 0.3794 D₁₅=
D₁₀= C_u= C_c=

Classification

USCS= GM AASHTO= A-1-b

Test Remarks

Source of Sample: Fill Depth: 29-31'
Sample Number: HB-AI95-202 / 8D

Sample Date: 12.8.22

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Western Ave Bridge No. 5808
Augusta, ME
Project No: 09.0025983.01

Figure 22-S-4936

Tested By: RB

Checked By:

	195 Frances Avenue Cranston RI, 02910 Phone: (401)-467-6454 Fax: (401)-467-2398 thielsch.com <i>Let's Build a Solid Foundation</i>	Client Information: GZA GeoEnvironmental South Portland, ME PM: Blaine Cardali Assigned By: Blaine Cardali Collected By: L. Navarrete	Project Information: Western Ave Bridge No. 5808 Augusta, ME Project Number: 09.0025983.01 Summary Page: 1 of 1 Report Date: 12.19.22
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LABORATORY TESTING DATA SHEET, Report No.: 7422-M-103

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-AI95-202	R1	6.9-7.5	22-S-4922		1.980	4.418	173.7				7198	0.224	3.00	0.05				Grey Gneiss	
Broke along foliation																			
BB-AI95-204	R1	8.2-8.5	22-S-4923		1.979	4.280	159.9				8306	0.590	1.05	0.51				Grey Granite	
Minor break at 1789psi																			
BB-AI95-205	R1	5.6 - 6.3	22-S-4924		1.979	4.621	172.0				14148	0.424	2.52	0.04				Grey Schist	
Broke along foliation																			
(1) Volume Determined By Measuring Dimensions					Notes	(3) PLD=Point Load (diametrical),						Notes	(5) Strain at Peak Deviator Stress						
(2) Determined by Measuring Dimensions and						PLA= Point Load (Axial) ST= Splitting Tensile							(6) Represents Secant Modulus at 50% of Total Failure Stress						
Weight of Saturated Sample						U= Unconfined Compressive Strength							(7) Represents Secant Poisson's Ratio at 50% of Total Failure Stress						
						(4) Taken at Peak Deviator Stress							(8) Estimated UCS from Table 1 of ASTM D5731 for NX cores (1s x 24)						

Date Received: 12.01.22

Reviewed By: [Signature]

Date Reviewed: 12.19.22

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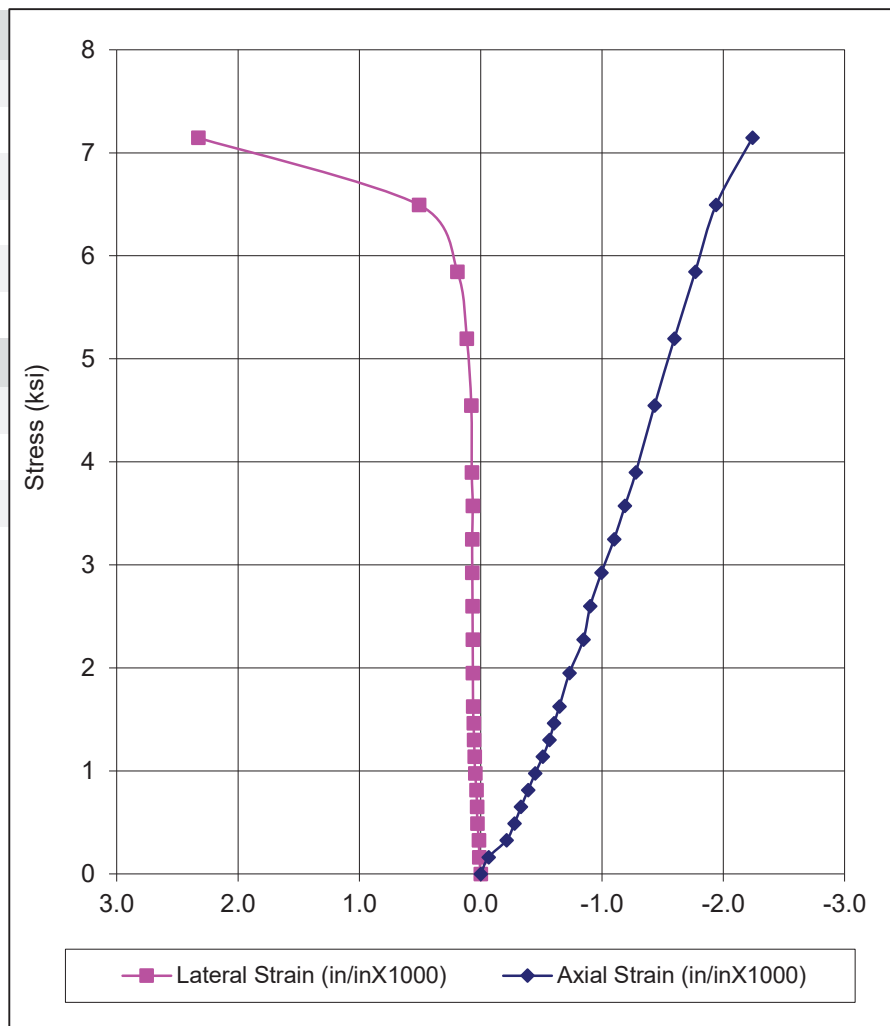
195 Frances Avenue
Cranston, Rhode Island 02910
Phone: (401) 467-6454
Fax: (401) 467-2398
www.thielsch.com
Let's Build a Solid Foundation

Client Information:
GZA GeoEnvironmental
Portland, ME
PM: Blaine Cardali
Assigned by: Blaine Cardali
Collected by: L. Navarrete

Project Information:
Western Ave Bridge Replacement
Augusta, ME
Project Number: 09.0025983.01.00
Technician: AV
Report Date: 12.19.22

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

Sample Information		Compressive Test Information	
Boring ID:	BB-AI95-202	Unit Weight (pcf):	173.7
Sample #:	R1	Failure Stress (psi):	7,198
Depth (ft):	6.9-7.5	Failure Mode:	Foliation
Tested Depth (ft):		Time to Failure (min)	2.58
Rock Type:	Grey Gneiss		
Features:	Broke along foliation		
Test Specimen Information		Elastic Moduli Test Information	
Diameter, D (in):	1.980	Poisson's Ratio @ 50%:	0.05
Length, L (in):	4.418	Strain %:	0.224
L:D Ratio:	2.23	E sec PSI @ 50%:	3.00E+06



Testing Notes:



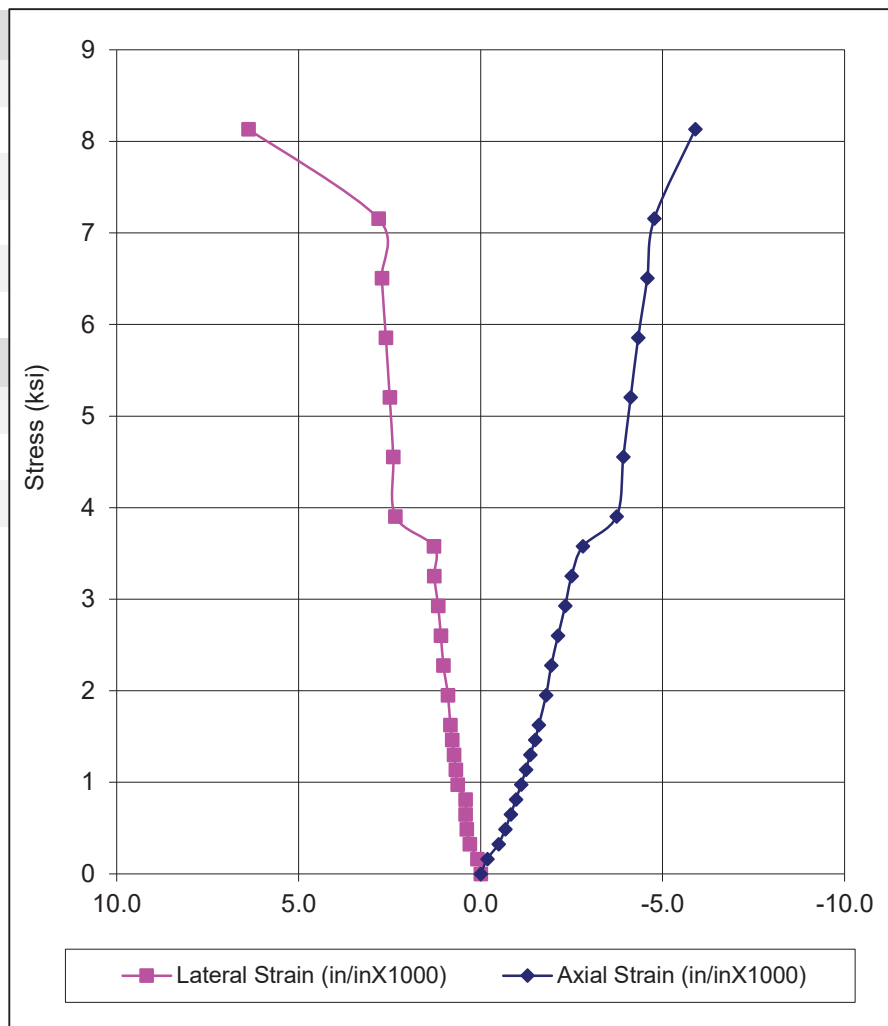
195 Frances Avenue
Cranston, Rhode Island 02910
Phone: (401) 467-6454
Fax: (401) 467-2398
www.thielsch.com
Let's Build a Solid Foundation

Client Information:
GZA GeoEnvironmental
Portland, ME
PM: Blaine Cardali
Assigned by: Blaine Cardali
Collected by: L. Navarrete

Project Information:
Western Ave Bridge Replacement
Augusta, ME
Project Number: 09.0025983.01.00
Technician: AV
Report Date: 12.19.22

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

Sample Information		Compressive Test Information	
Boring ID:	BB-AI95-204	Unit Weight (pcf):	159.9
Sample #:	R1	Failure Stress (psi):	8,306
Depth (ft):	8.2-8.5	Failure Mode:	Suture
Tested Depth (ft):		Time to Failure (min)	2.58
Rock Type:	Grey Granite		
Features:	Broke along suture		
Test Specimen Information		Elastic Moduli Test Information	
Diameter, D (in):	1.979	Poisson's Ratio @ 50%:	0.51
Length, L (in):	4.280	Strain %:	0.590
L:D Ratio:	2.16	E sec PSI @ 50%:	1.05E+06



Testing Notes: Minor break at 1789psi
Sample consists of parent rock and filled suture, break occurred along one plane of the suture



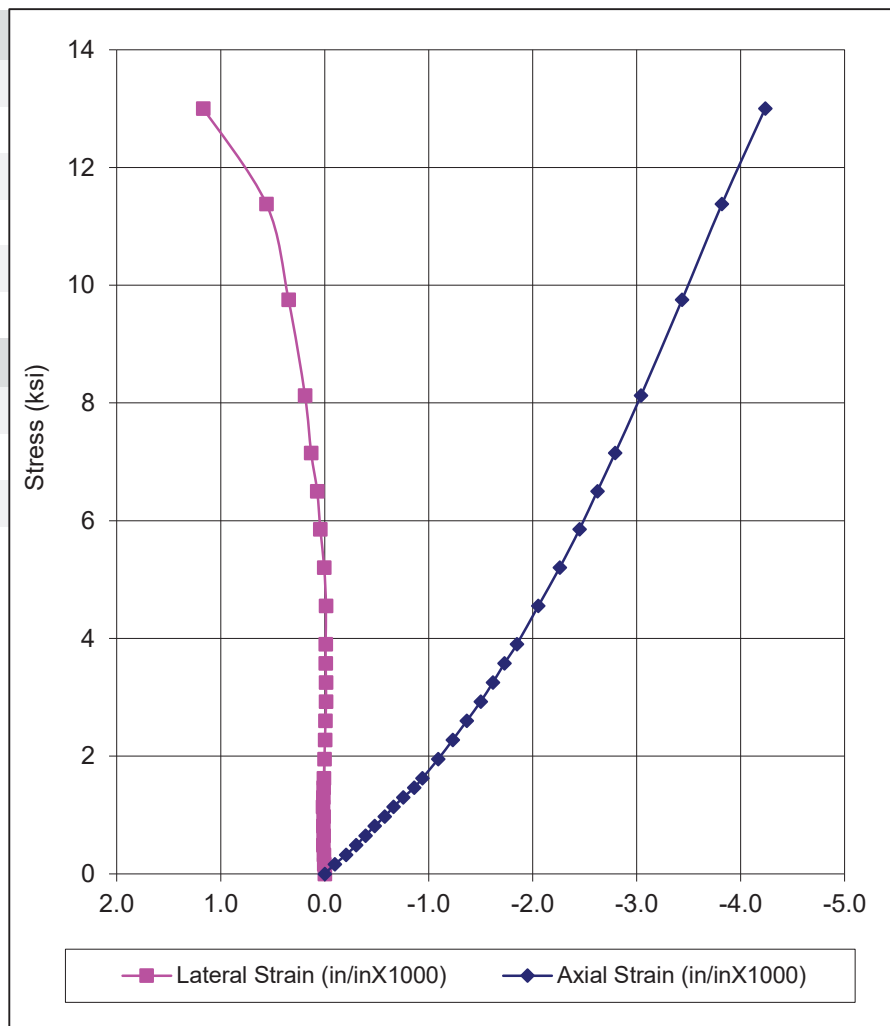
195 Frances Avenue
Cranston, Rhode Island 02910
Phone: (401) 467-6454
Fax: (401) 467-2398
www.thielsch.com
Let's Build a Solid Foundation

Client Information:
GZA GeoEnvironmental
Portland, ME
PM: Blaine Cardali
Assigned by: Blaine Cardali
Collected by: L. Navarrete

Project Information:
Western Ave Bridge Replacement
Augusta, ME
Project Number: 09.0025983.01.00
Technician: AV
Report Date: 12.19.22

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

Sample Information		Compressive Test Information	
Boring ID:	BB-AI95-205	Unit Weight (pcf):	172.0
Sample #:	R1	Failure Stress (psi):	14,148
Depth (ft):	5.6-6.3	Failure Mode:	foliation
Tested Depth (ft):		Time to Failure (min)	2.58
Rock Type:	Grey Schist		
Features:	Broke along foliation		
Test Specimen Information		Elastic Moduli Test Information	
Diameter, D (in):	1.979	Poisson's Ratio @ 50%:	0.04
Length, L (in):	4.621	Strain %:	0.424
L:D Ratio:	2.34	E sec PSI @ 50%:	2.52E+06



Testing Notes:



05/19/2023

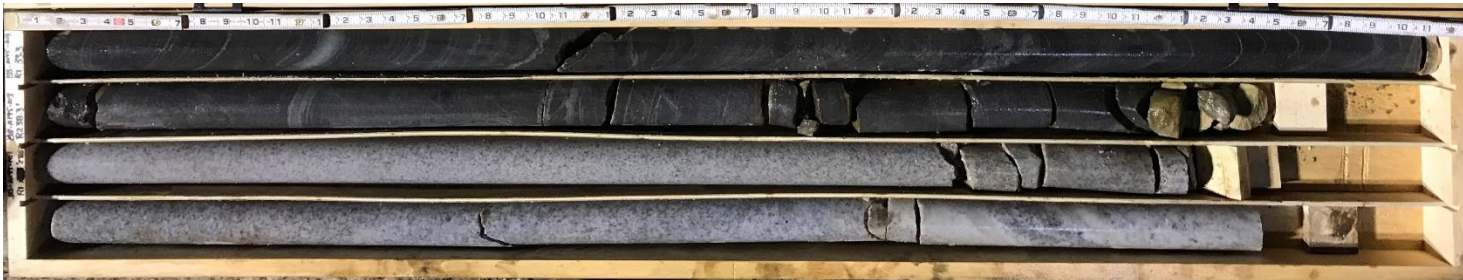
GEOTECHNICAL DESIGN REPORT
WESTERN AVENUE BRIDGE NO. 5808 OVER INTERSTATE 95
Maine Department of Transportation
09.0025983.01

APPENDIX E – ROCK CORE PHOTOGRAPHS



Western Avenue Bridge No. 5808
Augusta, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-AI95-103	R1	33.3 - 38.3	60	100	60	100	GRANOFELS	1
BB-AI95-103	R2	38.3 - 43.3	52	87	35	58	GRANOFELS	2
BB-AI95-101	R1	24.6 - 29.6	50	83	45	75	GRANITE	3
BB-AI95-101	R2	29.6 - 34.6	53	88	53	88	GRANITE	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.



Western Avenue Bridge No. 5808
Augusta, ME
 Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-AI95-102	R1	7.7 - 12.7	59	98	57	95	GRANOFELS	1
BB-AI95-102	R2	12.7 - 17.4	57	101	51	91	GRANOFELS	2



Notes:

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



Western Avenue Bridge No. 5808
Augusta, ME
 Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-AI95-201	R1	1 - 6	60	100	60	100	GRANOFELS	1
BB-AI95-201	R2	6 - 11	60	100	60	100	GRANOFELS	2
BB-AI95-202	R1	5.5 - 10.5	60	100	46	77	GRANOFELS	3
BB-AI95-202	R2	10.5 - 13.2	15.6	48	10	31	GRANOFELS	4
BB-AI95-202	R3	13.2 - 16	8	24	0	0	GRANOFELS	4
BB-AI95-202	R4	16 - 18.5	17	57	0	0	GRANOFELS	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.



Western Avenue Bridge No. 5808
Augusta, ME
Rock Core Photographs

3. Transition between core runs within a row are marked by wood or paper separators.

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-AI95-202	R5	18.5 - 23.5	45	75	45	75	GRANOFELS	1
BB-AI95-202A	R1	6 - 11	52	87	40	67	GRANOFELS	2
BB-AI95-202A	R2	11 - 14.4	10	24	0	0	GRANOFELS	2
BB-AI95-202A	R3	14.4 - 17	21	68	6	19	GRANOFELS	2,3
BB-AI95-202A	R4	17 - 22	60	100	55	92	GRANOFELS	3, 4



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



Western Avenue Bridge No. 5808
Augusta, ME
 Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-AI95-205	R1	5.5 - 10.5	58	97	55	91	GRANOFELS	1
BB-AI95-205	R2	10.5 - 15.5	60	100	60	99	GRANOFELS	2
BB-AI95-203	R1	13 - 18	60	100	24	40	GRANOFELS	3
BB-AI95-203	R2	18 - 23	60	100	57	95	GRANOFELS	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.



Western Avenue Bridge No. 5808
Augusta, ME
 Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-AI95-204	R1	7.8 - 12.4	52	95	42	75	GRANITE	1
BB-AI95-204	R2	12.4 - 17.4	60	100	56	93	GRANITE	1, 2
BB-AI95-206	R1	6 - 11	60	100	54	90	GRANOFELS	3
BB-AI95-206	R2	11 - 16	60	100	58	97	GRANOFELS	4



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



Western Avenue Bridge No. 5808
Augusta, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)		Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
HB-AI95-202	R1	17	- 20	22	61	-	-	GRANITE BOULDER	1
HB-AI95-201	R1	22.8	- 27.8	60	100	58	97	GRANITE	2



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



05/19/2023

GEOTECHNICAL DESIGN REPORT
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APPENDIX F – ENGINEERING CALCULATIONS

Bearing Resistance on
Bedrock



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Engineers and
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Western Ave Bridge, Augusta, ME

JOB: 09.0025983.00

SUBJECT: Bearing Resistance on Bedrock

SHEET: 1 OF 7

CALCULATED BY: B.Cardali 5.10.2023

REVIEWED BY: A.Blaisdell 5.10.2023

Objective

Assess nominal and factored bearing resistance of a foundation on rock based on support in GRANITE/GRANOFELS from borings BB-AI95-101, through 103, and BB-AI95-201 through -206. The fractured bedrock encountered in borings BB-AI95-202/202A are anticipated to be mitigated utilizing pressure grouting. This mitigation effectively increases the RQD in this zone to be consistent with the other borings.

Methodology

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

1. Bedrock Properties From Test Borings
2. Calculation Of Rock Mass Rating
3. Determine Rock Property Constants s and m
4. Calculate Nominal Bearing Resistance of Bedrock q_n

References

1. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 6th edition, 2012. (AASHTO LRFD).

Note: AASHTO 9th Edition is now in effect, but the coefficients used in the bedrock bearing evaluations are understood to be correlated relative to the older Hoek and Brown 1988 methodology. Therefore, RMR is used for the evaluation per LRFD 6th Edition rather than GSI per LRFD 9th Edition.

2. Wyllie, Duncan C., "Foundations on Rock", Second edition, 1992.

1. Rock Properties

Bedrock properties were obtained from rock core specimens and logs completed for the Western Ave Bridge Project in Augusta, ME. This calculation is based on the data from all borings including the mitigated fractured zones in Borings BB-AI95-202/202A.

Bedrock Strength

Boring	GS Elevation	RQD %	LAB							
			Depth of Sample (ft)	Depth of Sample into Rock (ft)	Elev Top of Sample (ft)	UCS (psi)	UCS (ksi)	UCS (ksf)	Modulus (ksi)	Unit Wt (pcf)
BB-AI95-101 R1	238.5	75%	24.6	1.1	213.9	12,119	12.12	1,745	1,520	163
BB-AI95-103 R1	233.1	100%	33.5	0.3	199.6	12,978	12.98	1,869	4,210	174
BB-AI95-202 R1	212.4	77%	6.9	1.6	210.1	7,198	7.20	1,037	3,000	174
BB-AI95-204 R1	214.9	75%	8.2	0.4	206.8	8,306	8.31	1,196	1,050	159.9
BB-AI95-205 R1	213.1	91%	5.6	0.1	207.4	14,148	14.15	2,037	2,520	172
						Average UCS	11			

Bedrock Quality

Average RQD of 71% representative of rock encountered in the borings with the exception of BB-AI95-202 and -202A.



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Engineers and
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Western Ave Bridge, Augusta, ME

JOB: 09.0025983.00

SUBJECT: Bearing Resistance on Bedrock

SHEET: 2 OF 7

CALCULATED BY: B.Cardali 5.10.2023

REVIEWED BY: A.Blaisdell 5.10.2023

Boring ID	Core Run	Length of Core Run (ft)	Rec (%)	RQD %	Joint Spacing Desc.	Joint Spacing (in)	Aperture Desc.	Joint Aperture (in)	Joint Weathering
BB-AI95-101	R1	5	83%	75%	Very Close to Moderate	0.75-24	Open	0.02-0.1	Fresh
BB-AI95-101	R2	5	88%	88%	Moderately Spaced	24	Partially Open to open	0.01-0.1	Fresh
BB-AI95-102	R1	5	98%	95%	Close to Wide	2.5-80	Partially Open to open	0.01-0.1	Fresh
BB-AI95-102	R2	4.7	101%	91%	Close to Wide	2.5-80	Open	0.02-0.1	Fresh
BB-AI95-103	R1	5	100%	100%	Close to Wide	2.5-80	Partially Open	0.01-0.02	Fresh
BB-AI95-103	R2	5	87%	58%	Very Close to Moderate	0.75-24	Tight to Partially Open	0.004-0.1	Fresh
BB-AI95-201	R1	5	100%	100%	N/A	N/A	Moderately Wide	0.1-0.4	Fresh
BB-AI95-201	R2	5	100%	100%	N/A	N/A	Partially Open to open	0.01-0.1	Fresh
BB-AI95-202	R1	5	100%	77%	Extremely Close to Moderate	<0.75-24	Tight to Partially Open	0.004-0.1	Fresh to Discolored
BB-AI95-202	R2	2.7	48%	31%	Close	8	Tight	0.004-0.01	Fresh
BB-AI95-202	R3	2.8	24%	0%	N/A	N/A	N/A	N/A	Decomposed
BB-AI95-202	R4	2.5	57%	0%	Extremely Close to Close	<0.75-8	Tight to Moderately Wide	0.004-0.4	Fresh to Decomposed
BB-AI95-202	R5	5	75%	75%	N/A	N/A	Tight	0.004-0.01	Discolored
BB-AI95-202A	R1	5	87%	67%	Close to Moderate	2.5-24	Tight to Partially Open	0.004-0.1	Fresh to Decomposed
BB-AI95-202A	R2	3.4	24%	0%	Extremely Close to Close	<0.75-8	Partially Open to Moderately Wide	0.01-0.4	Discolored to Decomposed
BB-AI95-202A	R3	2.6	68%	19%	N/A	N/A	N/A	N/A	Decomposed
BB-AI95-203	R4	5	100%	92%	Very Close to Moderate	0.75-24	Very Tight to Open	<0.004-0.1	Fresh
BB-AI95-203A	R1	5	100%	40%	Extremely Close to Moderate	<0.75-24	Very Tight to Tight	<0.004-0.01	Fresh to Discolored
BB-AI95-203	R2	5	100%	95%	Close to Moderate	2.5-24	Very Tight	<0.004	Fresh
BB-AI95-204	R1	4.6	95%	75%	Extremely Close to Close	<0.75-8	Very Tight to Tight	<0.004-0.01	Fresh to Discolored
BB-AI95-204	R2	5	100%	93%	Close to Moderate	2.5-24	Very Tight to Open	<0.004-0.1	Fresh to Discolored
BB-AI95-205	R1	5	97%	91%	Extremely Close to Moderate	<0.75-24	Tight to Partially Open	0.004-0.1	Fresh
BB-AI95-205	R2	5	100%	99%	Close to Moderate	2.5-24	Very Tight to Open	<0.004-0.1	Fresh
BB-AI95-206	R1	5	100%	90%	Very Close to Moderate	0.75-24	Very Tight to Open	<0.004-0.1	Fresh
BB-AI95-206	R2	5	100%	97%	Close to Moderate	2.5-24	Very Tight to Partially Open	<0.004-0.1	Fresh
HB-AI95-201	R1	5	100%	97%	Moderately Spaced	24	Tight to Partially Open	0.004-0.1	Fresh
			Avg RQD	71%					
			St. Dev RQD	33%					

2. Calculation of Rock Mass Rating (RMR)

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

Parameter 1- Uniaxial Compressive Strength

$$\sigma_{u,r} := 7.2 \text{ ksi} = 1037 \cdot \text{ksf}$$

Use unconfined compressive strength from boring BB-AI95-202 for rock.

From AASHTO LRFD Table 10.4.6.4-1

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

Parameter 2- Drill Core Quality

Representative RQD from table above: approximately 71%, use a reduced range of 25 to 50% for design purposes

From AASHTO LRFD Table 10.4.6.4-1

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



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*Engineers and
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Western Ave Bridge, Augusta, ME

JOB: 09.0025983.00

SUBJECT: Bearing Resistance on Bedrock

SHEET: 3 OF 7

CALCULATED BY: B.Cardali 5.10.2023

REVIEWED BY: A.Blaisdell 5.10.2023

Parameter 3- Spacing of Joints

From Boring Logs generally close to moderately spaced = 2.5 in to 2 feet

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating

$$RR_3 := 15$$

Parameter 4- Condition of Joints

From boring logs, very hard to hard walls and rough to smooth surface, generally with joint separation <0.05 in. (after improvement grouting), and described generally as fresh.

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating

$$RR_4 := 20$$

Parameter 5- Ground Water Conditions

Hydrostatic Conditions- 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 low angle to moderately dipping, with some high angle joints, and generally rough and tight. Therefore the joint orientation is considered fair.

From AASHTO LRFD Table 10.4.6.4-2

Relative Rating

$$RR_6 := -7$$

Total RMR Rating

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

$$RMR = 44$$

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



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SUBJECT: Bearing Resistance on Bedrock

SHEET: 4 OF 7

CALCULATED BY: B.Cardali 5.10.2023

REVIEWED BY: A.Blaisdell 5.10.2023

3. Determine Rock Property Constants s and m

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

Granite/Granofels is categorized as rock type E, coarse grained polyminerallic igneous & metamorphic crystalline rocks, RMR=44, using s and m values interpolated from the logarithmic trend of plotted values from AASHTO Table 10.4.6.4-4 (plots on sheet 8).

$$m := .458$$

$$s := .00009$$

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

From Wylie "Foundations on Rock"

Eq. 5.4 Pg.138

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

Where

$$C_{f1} := 1.0$$

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

$$s = 0.00009$$

For $L/B > 6$, use factor $C_{f1} = 1.0$,

$$m = 0.458$$

For $L/B = 1$, use factor $C_{f1} = 1.12$, therefore,

$$\sigma_{u,r} = 7.2 \text{ ksi}$$

For conservatism, assume long strip, lowest C_{f1} .

Nominal Bearing Resistance

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

$$q_n = 78.9 \text{ ksf}$$

Say 79 ksf

Factored Bearing Resistance for Strength Condition

Bearing Resistance Factor is specified in Table 10.5.5.2.2-1

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

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

$$q_R = 35.5 \text{ ksf}$$

Say 36 ksf



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SUBJECT: Bearing Resistance on Bedrock

SHEET: 5 OF 7

CALCULATED BY: B.Cardali 5.10.2023

REVIEWED BY: A.Blaisdell 5.10.2023

➔ Reference:I:\Mathcad\units.xmcd

10-22

AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

Table 10.4.6.4-1 Geomechanics Classification of Rock Masses.

Parameter			Ranges of Values						
1	Strength of intact rock material	Point load strength index	>175 ksf	85–175 ksf	45–85 ksf	20–45 ksf	For this low range, uniaxial compressive test is preferred		
		Uniaxial compressive strength	>4320 ksf	2160–4320 ksf	1080–2160 ksf	520–1080 ksf	215–520 ksf	70–215 ksf	20–70 ksf
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>10 ft.	3–10 ft.	1–3 ft.	2 in.–1 ft.	<2 in.		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		• 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			



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JOB: 09.0025983.00

SUBJECT: Bearing Resistance on Bedrock

SHEET: 6 OF 7

CALCULATED BY: B.Cardali 5.10.2023

REVIEWED BY: A.Blaisdell 5.10.2023

Table 10.4.6.4-2 Geomechanics Rating Adjustment for Joint Orientations.

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

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

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

10-24

AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

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

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



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Western Ave Bridge, Augusta, ME

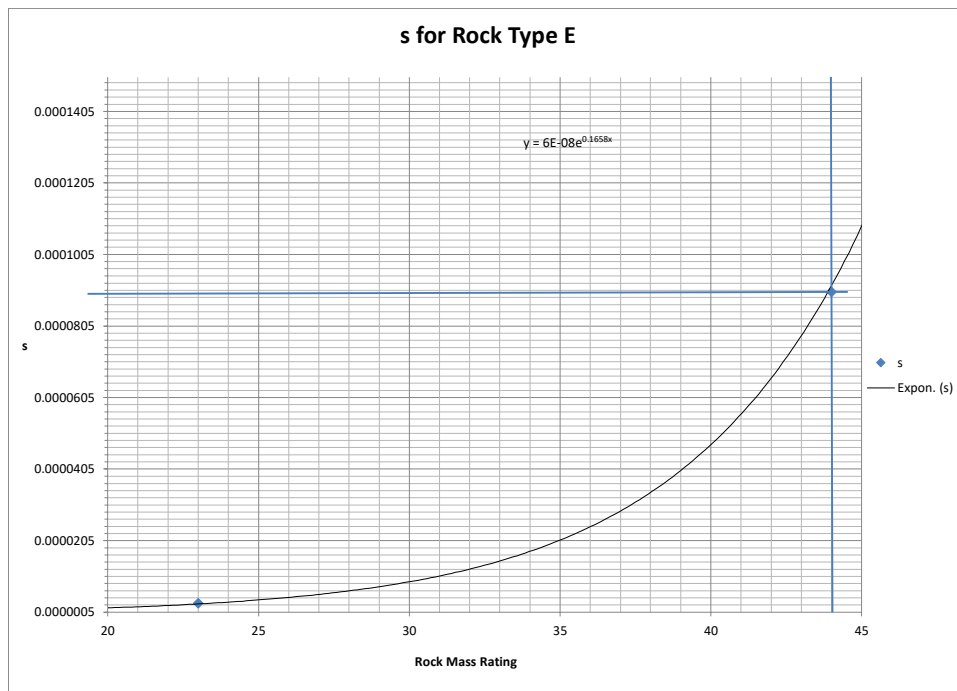
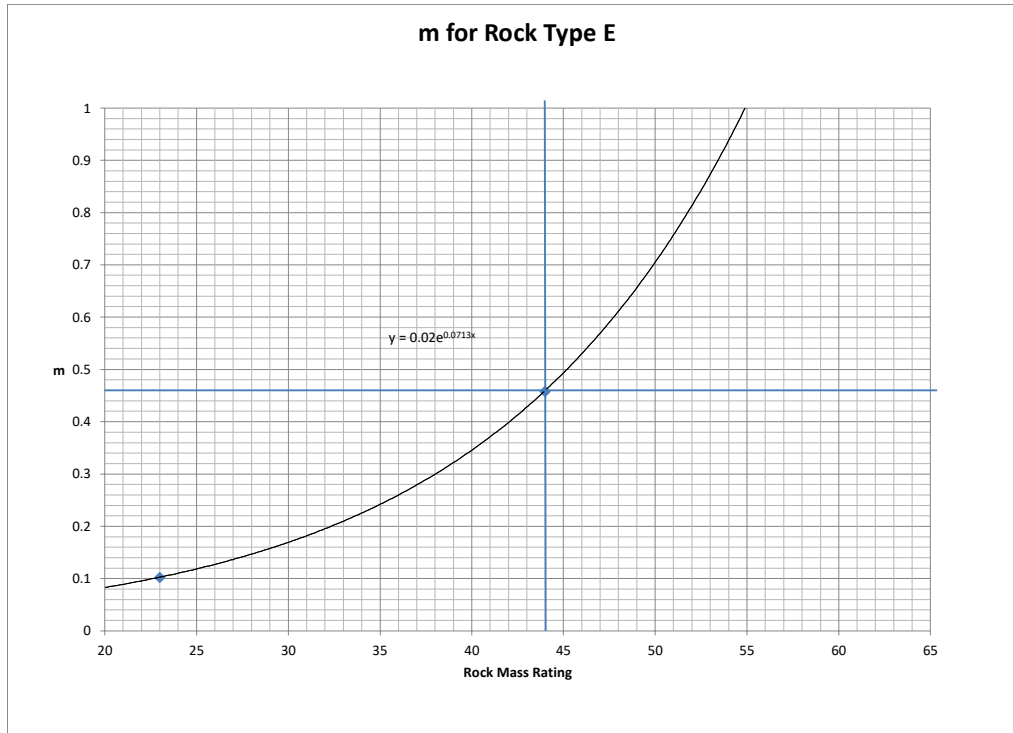
JOB: 09.0025983.00

SUBJECT: Bearing Resistance on Bedrock

SHEET: 7 OF 7

CALCULATED BY: B.Cardali 5.10.2023

REVIEWED BY: A.Blaisdell 5.10.2023



Lateral Earth Pressure
from Geosynthetic
Reinforced Soil Backfill



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*Engineers and
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JOB: 09.0025983.01
SUBJECT: GRS Wall Lateral Pressure,
Western Ave
SHEET: 1 OF 8
CALCULATED BY B.Cardali 2/8/2023
CHECKED BY A. Blaisdell 2/9/2023

Objective

Design a Geosynthetic Reinforced Soil (GRS) fill and determine the lateral pressure exerted on abutment backwalls.

References

1. Synthesis of Geosynthetic Reinforced Soil Design Topics FHWA-HRT-14-094 2015
2. Synthesis of Geosynthetic Reinforced Soil Design Topics FHWA-HRT-11-027 2011
3. STATE OF MAINE Department of Transportation (MaineDOT) Standard Specifications 2020
4. Maine DOT Bridge Design Guide (BDG)

Design Inputs

Soil and Geosynthetic Properties

$\phi_r := 34\text{-deg}$	Friction angle of the reinforced soil
$\gamma_T := 125\text{-pcf}$	Unit weight of the reinforced soil
$\gamma_{\text{conc}} := 150\text{-pcf}$	Unit weight of concrete
$\gamma_{\text{asphalt}} := 140\text{-pcf}$	Unit weight of asphalt
$K_{\text{ar}} := \frac{1 - \sin(\phi_r)}{1 + \sin(\phi_r)} = 0.283$	Rankine coefficient of Active earth pressure for reinforced soil
$q_{\text{sur}} := 250\text{-psf}$	Surcharge load from traffic

Load and Resistance Factors

Analyze Strength I Load Case	Load and Resistance Factors from AASHTO 3.4.1-1
$\gamma_{\text{EHMAX}} := 1.5$	Max earth Pressure from Retained Fill
$\gamma_{\text{ESMAX}} := 1.5$	Max earth pressure from Road Base
$\gamma_{\text{ESMIN}} := 0.75$	Min earth pressure from Road Base
$\gamma_{\text{LS}} := 1.75$	Live load - Roadway
$\gamma_{\text{LL}} := 1.75$	Live Load - Superstructure
$\gamma_{\text{EV}} := 1.35$	Max soil Fill Surcharge
$\gamma_{\text{DCMIN}} := 0.9$	Min Approach Slab dead weight
$\gamma_{\text{DCMAX}} := 1.25$	Max Approach Slab dead weight
$\Phi_{\text{cap}} := 0.45$	Resistance factor for Nominal Vertical load carrying Capacity of GRS Wall
$\Phi_T := 1.0$	Interface friction resistance factor, base of GRS wall



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 SHEET: 2 OF 8
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Geometry

Total Height from top of
 footing to final grade

Height of Reinforced Soil

Pavement thickness

Soil Thickness below
 approach slab above GRS

Thickness of EPS

Max subbase gravel above
 approach slab

Thickness of approach
 slab

Width of abutment stem

Length of approach slab (surcharge)
 beyond EPS

Length EPS (includes Approach
 slab seat)

Approach Slab Length

Approach slab surcharge offset
 from Backwall

Reinforcement vertical spacing

Height of approach slab over top
 of GRS

Height of EPS over top
 of GRS

$$H_t := 26.2 \cdot \text{ft}$$

$$H_{\text{grs}} := 21 \cdot \text{ft}$$

$$t_{\text{pave}} := 6 \cdot \text{in}$$

$$d := 4 \cdot \text{in}$$

$$d_{\text{eps}} := 12 \cdot \text{in}$$

$$d_f := 15 \cdot \text{in}$$

$$t_s := 16 \cdot \text{in}$$

$$w_{\text{stem}} := 2.75 \cdot \text{ft}$$

$$B_{\text{sur}} := 5 \cdot \text{ft}$$

$$x_s := 15 \cdot \text{ft}$$

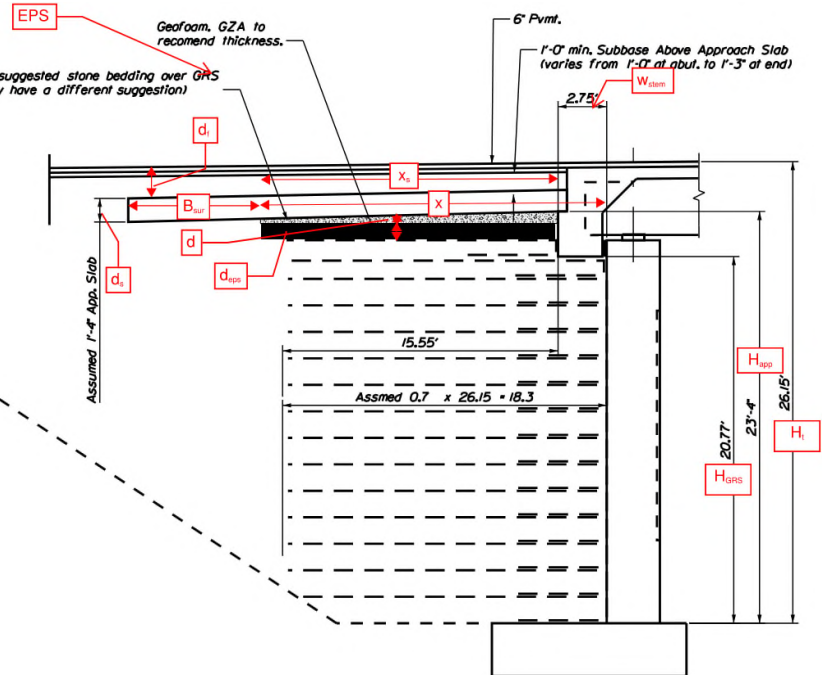
$$l_s := x_s + B_{\text{sur}} = 20 \cdot \text{ft}$$

$$x := x_s + w_{\text{stem}} = 17.75 \cdot \text{ft}$$

$$S_v := 18 \cdot \text{in}$$

$$h_{\text{slab}} := [H_t - (t_s + d_f + t_{\text{pave}})] - H_{\text{grs}} = 2.1 \cdot \text{ft}$$

$$h_{\text{eps}} := h_{\text{slab}} - (d_{\text{eps}} + d) = 0.78 \cdot \text{ft}$$



Bearing Pressures from Surcharge Loading

Lateral pressure from the traffic and fill surcharge loads must be resisted by the GRS. Develop factored bearing pressures at the applied zone of the approach slab, assuming an extended buried approach slab bridges a portion of the backfill soils. This is based on the assumption that half of the slab weight and overlying materials are supported by the abutment, and the other half is supported on the soil subgrade behind the EPS and outside of the GRS zone.

Weight of Soil and pavement on rear bearing
 length above approach slab

$$W_s := \frac{[\gamma_{\text{r}} \cdot (x_s + B_{\text{sur}}) \cdot d_f \cdot \text{ft}] + [\gamma_{\text{asphalt}} \cdot t_{\text{pave}} \cdot (x_s + B_{\text{sur}})] \cdot \text{ft}}{2} = 2.3 \cdot \text{kip}$$

Weight of concrete on rear bearing length of approach
 slab

$$W_{\text{slab}} := \frac{[\gamma_{\text{conc}} \cdot (x_s + B_{\text{sur}}) \cdot t_s \cdot \text{ft}]}{2} = 2 \cdot \text{kip}$$

Unfactored Pressure on bearing length from traffic
 live load

$$q_{\text{LL}} := \frac{[q_{\text{sur}} \cdot (x_s + B_{\text{sur}}) \cdot .5]}{B_{\text{sur}}} = 500 \cdot \text{psf}$$

Calculate factored bearing pressure on extended portion of approach slab (and bearing on back portion of EPS)

Total factored Pressure on rear bearing length
 from DL

$$Q_{\text{DL}} := \frac{(W_s \cdot \gamma_{\text{EV}}) + (W_{\text{slab}} \cdot \gamma_{\text{DCMAX}})}{(B_{\text{sur}}) \cdot \text{ft}} = 1110.9 \cdot \text{psf}$$



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 SUBJECT: GRS Wall Lateral Pressure,
Western Ave
 SHEET: 3 OF 8
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Factored Pressure on rear bearing length
 from traffic surcharge

$$Q_{LL} := \gamma_{LL} \cdot q_{LL} = 875 \cdot \text{psf}$$

Total Factored bearing pressure on bearing length
 (zone without EPS)

$$Q_b := Q_{LL} + Q_{DL} = 1985.9 \cdot \text{psf}$$

Calculate the reduced factored bearing pressure on back portion of EPS

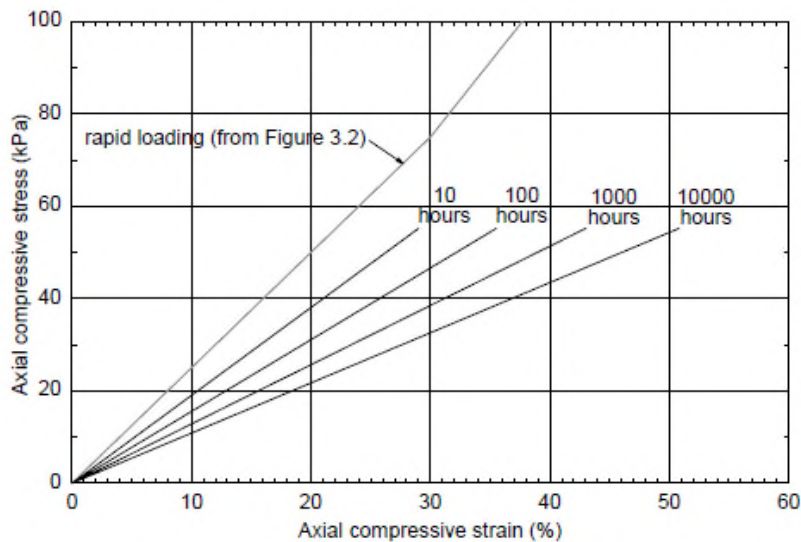


Figure 3.4. Isochronous Stress-Strain Curves for Resilient Block-Molded EPS
 (initial molded density = 12 kg/m³ (0.75 lb/ft³))
 [data courtesy of GeoTech Systems Corporation]

Modulus of EPS based on Slope of Rapid loading

$$E_{\text{eps}} := 5220 \cdot \text{psf}$$

Length of EPS with pressure (assume half
 further from abutment)

$$b_{\text{eps}} := \frac{x_s}{2} = 7.5 \text{ ft}$$

Assume maximum settlement of 0.5 inches of bearing
 zone (conservative for unfactored bearing pressure less
 than 2 ksf on structural fill)

$$S_s := 0.5 \cdot \text{in}$$

Strain of EPS

$$\epsilon_{\text{eps}} := \frac{S_s}{d_{\text{eps}}} = 0.0417$$

Bearing Pressure Below EPS

$$Q_{\text{eps}} := E_{\text{eps}} \cdot \epsilon_{\text{eps}} = 218 \cdot \text{psf}$$

Reduced factored bearing resistance on portion beyond EPS,
 removing EPS load transferred to subgrade

$$Q_{b, \text{Reduced}} := \frac{Q_b \cdot B_{\text{sur}} - Q_{\text{eps}} \cdot b_{\text{eps}}}{B_{\text{sur}}} = 1659.6 \cdot \text{psf}$$



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 SUBJECT: GRS Wall Lateral Pressure,
Western Ave
 SHEET: 4 OF 8
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Lateral Earth Pressure on Backwall from Reinforced Fill

Utilize Wu (2001) methodology for determining backwall lateral earth pressure based on reinforcement spacing, and graphically represented below in Figure 35 from "Synthesis of Geosynthetic Reinforced Soil Design Topics" (FHWA-HRT-14-094 2015)

Reinforcement vertical spacing

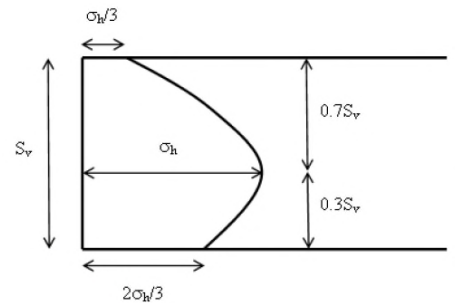
$$S_v = 18 \text{ in}$$

Total lateral thrust

$$F_{bin} := 0.72 \cdot \gamma_r \cdot K_{ar} \cdot S_v^2 = 57 \cdot \frac{\text{lb}}{\text{ft}}$$

Factored Lateral Pressure
 between reinforcement
 layers

$$Q_{bin}(z) := \frac{F_{bin} \cdot \gamma_{EHMAX}}{S_v}$$

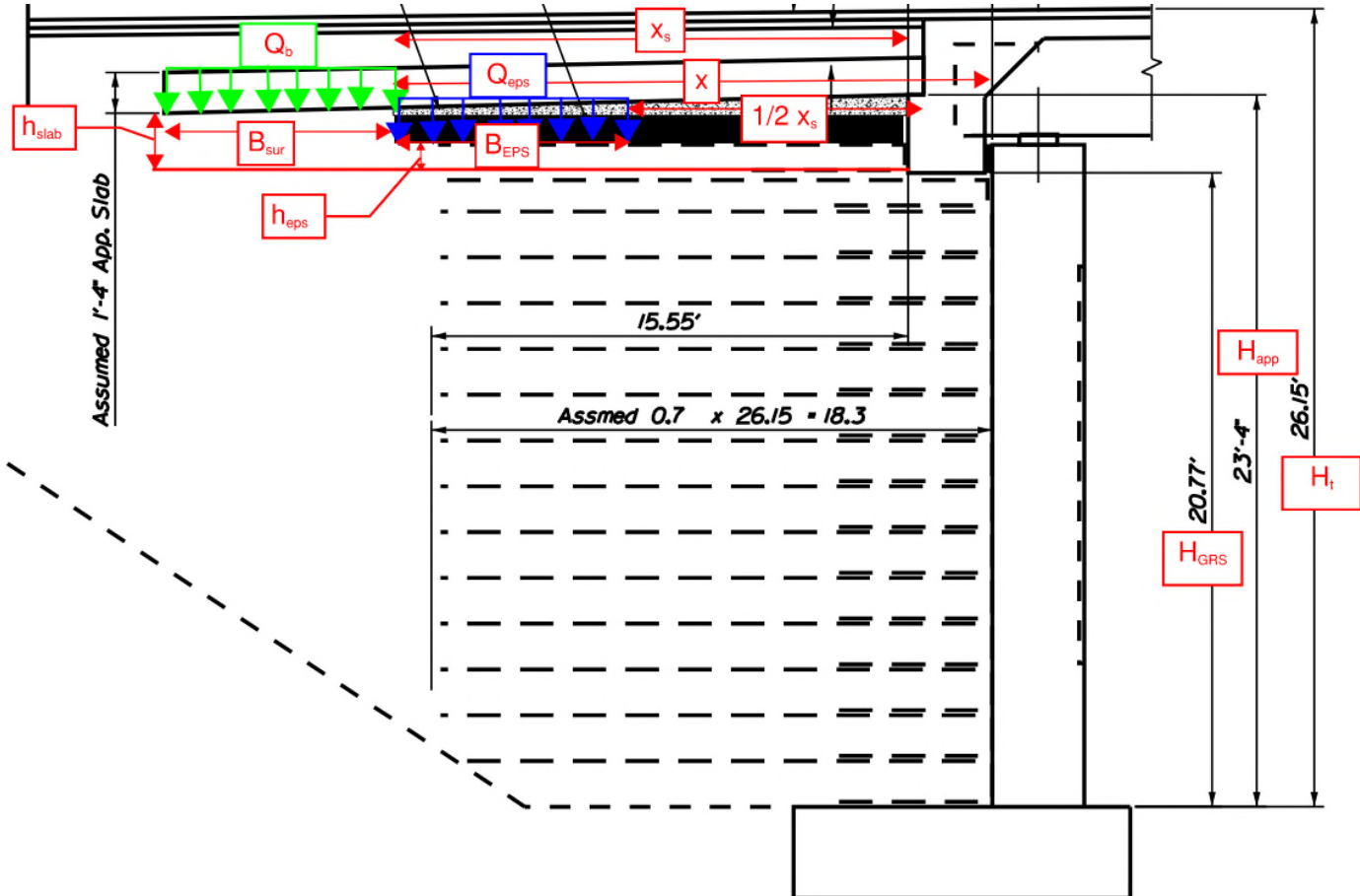


©J.T.H. Wu
 1 pcf = 16.02 kg/m³
 1 psf = 0.05 kPa

Figure 35. Diagram. Bin pressure diagram.

Lateral Earth Pressures on Backwall from Surcharge Loading

Calculate lateral pressures on abutment backwall generated from the traffic and fill surcharges





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 SUBJECT: GRS Wall Lateral Pressure,
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 SHEET: 5 OF 8
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Pressure Factors EPS bearing zone:

$$\delta_{\text{eps}}(z) := \text{atan}\left(\frac{x - b_{\text{eps}}}{z + h_{\text{eps}}}\right)$$

$$\alpha_{\text{eps}}(z) := \text{atan}\left(\frac{x}{z + h_{\text{eps}}}\right) - \text{atan}\left(\frac{x - b_{\text{eps}}}{z + h_{\text{eps}}}\right)$$

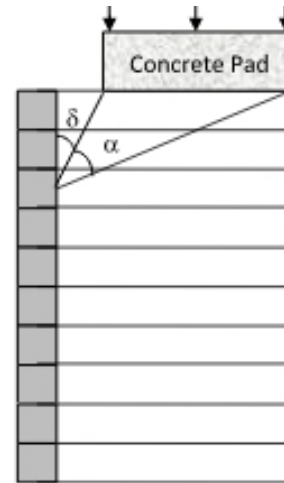
Pressure Factors bearing zone beyond EPS:

$$\delta_b(z) := \text{atan}\left(\frac{x}{z + h_{\text{slab}}}\right)$$

$$\alpha_b(z) := \text{atan}\left(\frac{x + B_{\text{sur}}}{z + h_{\text{slab}}}\right) - \text{atan}\left(\frac{x}{z + h_{\text{slab}}}\right)$$

$$\Delta\sigma_h = p/\pi [\alpha - \sin\alpha \cos(\alpha + 2\delta)]$$

Figure 42. Equation. Increase in lateral pressure $\Delta\sigma$ subscript h.



Horizontal Pressure on backwall from bearing pressure below EPS

$$Q_{H_EPS}(z) := \left(\frac{Q_{\text{eps}}}{\pi}\right) \cdot [(\alpha_{\text{eps}}(z)) - \sin(\alpha_{\text{eps}}(z)) \cdot \cos[(\alpha_{\text{eps}}(z)) + 2 \cdot \delta_{\text{eps}}(z)]]$$

Horizontal Pressure on backwall from bearing pressure beyond EPS zone

$$Q_{H_b}(z) := \left(\frac{Q_{b,\text{Reduced}}}{\pi}\right) \cdot [(\alpha_b(z)) - \sin(\alpha_b(z)) \cdot \cos[(\alpha_b(z)) + 2 \cdot \delta_b(z)]]$$

Total Factored Horizontal Pressure on backwall

$$Q_{\text{total}}(z) := Q_{\text{bin}}(z) + Q_{H_EPS}(z) + Q_{H_b}(z)$$

Figure 43. Drawing. Definition of angles α and δ .

z =	$\delta_{\text{eps}}(z) =$	$\alpha_{\text{eps}}(z) =$	$Q_{H_EPS}(z) =$	$\delta_b(z) =$	$\alpha_b(z) =$	$Q_{H_b}(z) =$	$Q_{\text{bin}}(z) =$	$Q_{\text{total}}(z) =$
0.75 ft	81.5 ·deg	3.6 ·deg	8.5 ·psf	80.8 ·deg	2 ·deg	36 ·psf	57.2 ·psf	101.8 ·psf
2.25	73.5	6.8	15.5	76.2	3	52	57.2	124.8
3.75	66.1	9.5	20.5	71.7	3.8	65	57.2	142.8
5.25	59.5	11.7	23.3	67.5	4.6	74.6	57.2	155.2
6.75	53.7	13.3	24.2	63.5	5.3	80.9	57.2	162.3
8.25	48.6	14.4	23.8	59.7	5.8	84.1	57.2	165.1
9.75	44.2	15.1	22.5	56.2	6.2	84.8	57.2	164.5
11.25	40.4	15.4	20.7	53	6.5	83.5	57.2	161.4
12.75	37.1	15.5	18.7	50.1	6.8	80.7	57.2	156.6
14.25	34.3	15.5	16.7	47.3	6.9	76.9	57.2	150.9
15.75	31.8	15.2	14.9	44.8	7	72.5	57.2	144.6
17.25	29.6	14.9	13.2	42.5	7.1	67.7	57.2	138.2
18.75	27.7	14.6	11.6	40.4	7.1	62.9	57.2	131.8
20.25	26	14.2	10.3	38.4	7.1	58.1	57.2	125.7



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JOB: 09.0025983.01
SUBJECT: GRS Wall Lateral Pressure,
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SHEET: 6 OF 8
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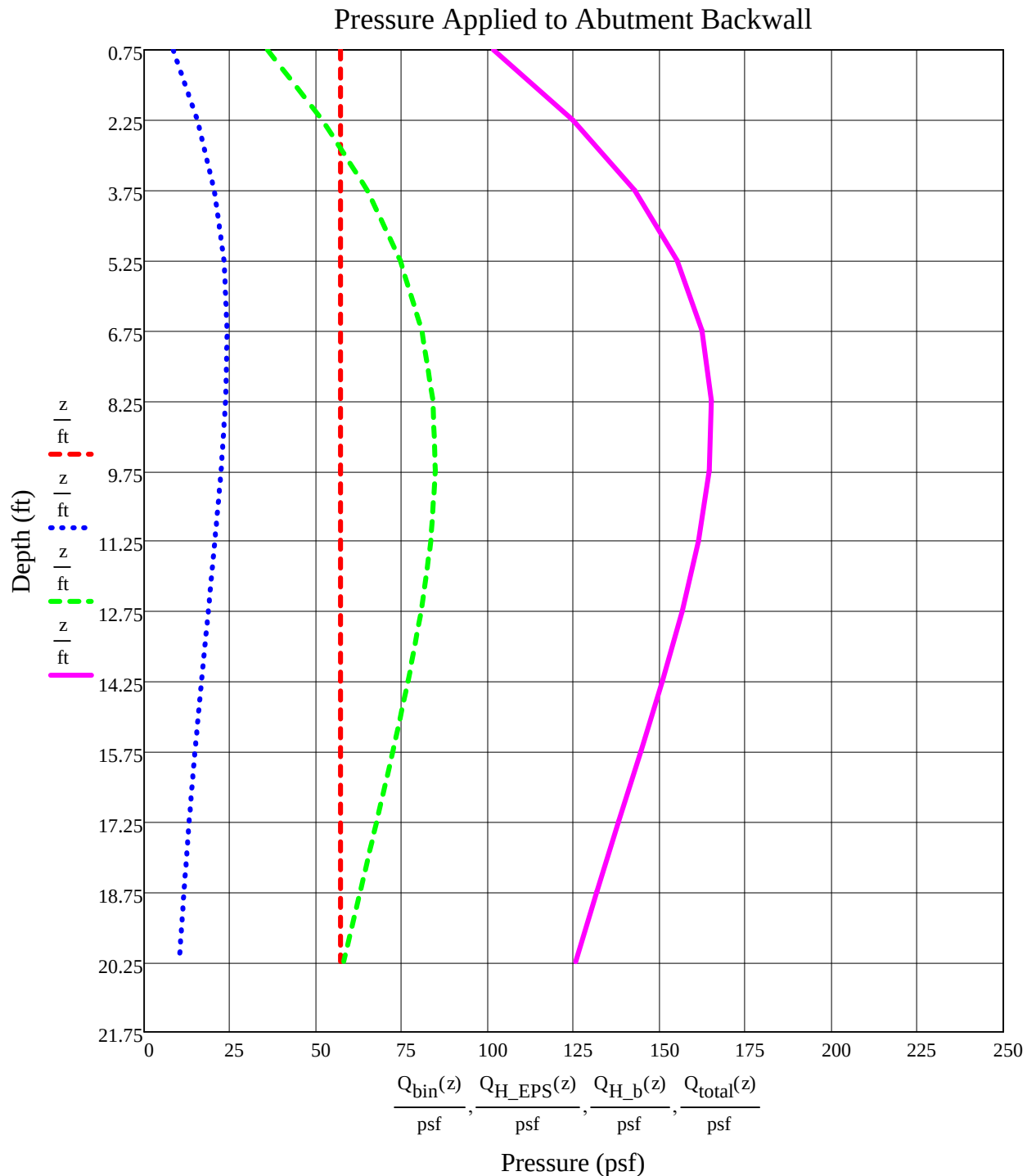
Plot Legend:

Red Line (Solid) = Lateral pressure from reinforced backfill at spacing $S_v(Q_{bin}(z))$

Blue Line (Dotted) = Lateral Pressure from EPS bearing pressure ($Q_{H_EPS}(z)$)

Green Line (Dashed) = Lateral Pressure from bearing pressure beyond EPS zone ($Q_{H_b}(z)$)

Pink Line (Dashed/Dotted) = Total Lateral Pressure on Backwall ($Q_{total}(z)$)





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 SUBJECT: GRS Wall Lateral Pressure,
Western Ave
 SHEET: 7 OF 8
 CALCULATED BY B.Cardali 2/8/2023
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GRS Internal Stability Analysis

Soil and Geosynthetic Inputs

$$S_v = 1.5 \text{ ft}$$

Reinforcement Spacing

$$d_{\max} := 2.0 \cdot \text{in}$$

Maximum grain size of the reinforced soil

$$K_{\text{pr}} := \left(\frac{1 + \sin(\phi_r)}{1 - \sin(\phi_r)} \right) = 3.537$$

Rankine Coefficient of Passive earth pressure for reinforced soil

$$K_{\text{ax}} := \frac{1 - \sin(\phi_r)}{1 + \sin(\phi_r)} = 0.283$$

Rankine Coefficient of Active earth pressure for reinforced soil

Required Reinforcement Strength (per FHWA-HRT-11-027 2011)

$$RF_{\text{global}} := 2.25$$

Global Reduction factor for long-term geotextile strength loss

$$\Phi_{\text{reinf}} := 0.9$$

Geotextile Strength reinforcement Resistance Factor

$$\sigma_{h,\max} := 164.5 \cdot \text{psf}$$

Maximum pressure on reinforcement

$$T_{\text{req},f} := \left(\frac{\frac{\sigma_{h,\max}}{S_v}}{\frac{6 \cdot d_{\max}}{0.7}} \right) \cdot S_v = 421.3 \cdot \text{plf}$$

Factored required geotextile reinforcement strength per FHWA-HRT-11-027 equation 1

Check Selected Geotextile

Mirafi HP 370 Geotextile (Woven Polypropylene)

$$T_f := 3600 \cdot \text{plf}$$

Minimum ultimate strength of the reinforcement (Machine Direction)

$$T_{\epsilon 2\%} := 540 \cdot \text{plf}$$

Minimum strength of geotextile at 2% strain (Machine Direction)

$$q_{n,\text{an}} := \left[.7 \left(\frac{S_v}{6 d_{\max}} \right) \cdot \frac{T_f}{S_v} \right] \cdot K_{\text{pr}} = 4972 \cdot \text{psf}$$

Soil-Geosynthetic composite capacity equation.
 $q_{n,\text{an}}$ = nominal ultimate load-carrying capacity per FHWA-HRT-11-027 equation 9



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JOB: 09.0025983.01
 SUBJECT: GRS Wall Lateral Pressure,
Western Ave
 SHEET: 8 OF 8
 CALCULATED BY B.Cardali 2/8/2023
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$$V_{\text{applied.f}} := \max(Q_{b,\text{Reduced}}, Q_{\text{eps}}) = 1659.6 \cdot \text{psf}$$

Max Factored applied pressure on the GRS mass. (maximum of pressure under EPS or beyond EPS)

$$\Phi_{\text{cap}} \cdot \frac{(q_{n,\text{an}})}{V_{\text{applied.f}}} = 1.3$$

Factored applied pressure must be less than the factored ultimate capacity (eq. 83). The resistance factor $\Phi_{\text{cap}} = .45$ Greater than 1, OK

$$\frac{T_{\varepsilon 2\%}}{T_{\text{req}}} = 1.3$$

The reinforcement strength at a 2% strain in geotextile must be greater than the unfactored required reinforcement strength

$$T_{f.f} := \Phi_{\text{reinf}} \cdot \frac{T_f}{\text{RF}_{\text{global}}} = 1440.0 \cdot \text{plf}$$

Factored ultimate strength of the geotextile

$$\frac{T_{f.f}}{T_{\text{req.f}}} = 3.4$$

The factored reinforcement strength must be greater than the factored required reinforcement strength. The ratio of $T_{f.f}$ to $T_{\text{req.f}}$ must be greater than 1.

Reinforcement Length

$$l_{r,\text{top}} := 0.7 \cdot H_{\text{grs}} = 14.7 \text{ ft} \quad \text{say 15 feet}$$

Minimum recommend reinforcement length per AASHTO C11.10.2.1, use 15 feet

$$l_{r,\text{bot}} := 0.4 \cdot H_{\text{grs}} = 8.4 \text{ ft} \quad \text{say 10 feet}$$

Minimum reinforcement length for lower portion of GRS. Per AASHTO C11.10.2.1, shortening of reinforcements may be below 0.7H to minimize excavation requirements provided bearing on rock or competent soil.

$$H_{\text{bot.max}} := \frac{l_{r,\text{bot}}}{\tan\left(45 - \frac{\phi_r}{2}\right)} = 9.6 \text{ ft}$$

Maximum Height above the top of footing for lower reinforcement length. Use 6 feet for design.

Use 10-foot reinforcement length for bottom four rows and remaining rows at 15-foot reinforcement length

Conclusion

The required approach slab length to reduce surcharge loading is 20 feet including the first 15 feet beyond the backwall underlain by a 12-inch thick Resilient EPS block (i.e., compressible inclusion). The resulting GRS supported backfill consists of MaineDOT Standard Specification 703.02" Coarse Aggregate for Concrete, Grading A" with a geotextile reinforcement with properties equivalent to Mirafi HP 370 Geotextile (Woven Polypropylene) spaced at 18 inches. Reinforcement lengths are 10 feet in the lower 6 feet above the footing and 15 feet thereafter. Considering the dimensions and properties herein, a uniform lateral pressure on the back wall of 170 psf may be used to design the abutment footing.

Lateral Earth Pressure Conventional Backfill



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JOB: 09.0025983.01 Western Ave. Bridge
 SUBJECT: Lateral Earth Pressures
 SHEET: 1 OF 1
 CALCULATED BY B. Cardali 5/12/23
 CHECKED BY A. Blaisdell 5/12/23

Subject:

Evaluate lateral earth pressure coefficients for wingwalls with conventional backfill.

References:

1. MaineDOT Bridge Design Guide, Chapter 3
2. AASHTO LRFD Bridge Design Specifications, 9th Edition (2020)
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
$\theta := 90\text{deg}$	Angle of back face of wall to the horizontal

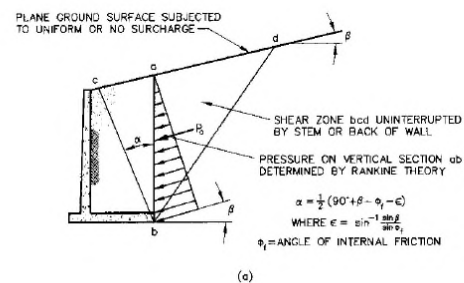
Earth Pressure Coefficients:

Active Earth Pressure (Abutments and Wingwalls)

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

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

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.



Coulomb Active Earth Pressure Coefficient (Short-Heeled Wall)

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

$$K_{ac} := \frac{(\sin(\theta + \phi))^2}{\Gamma_{ww} \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta_f)} \quad K_{ac} = 0.28$$

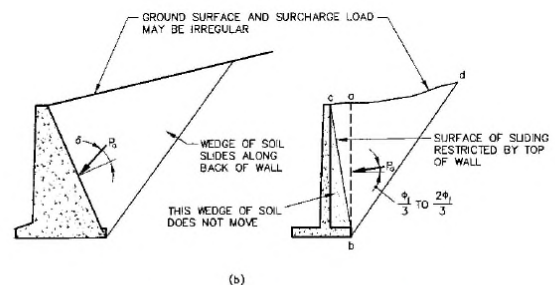
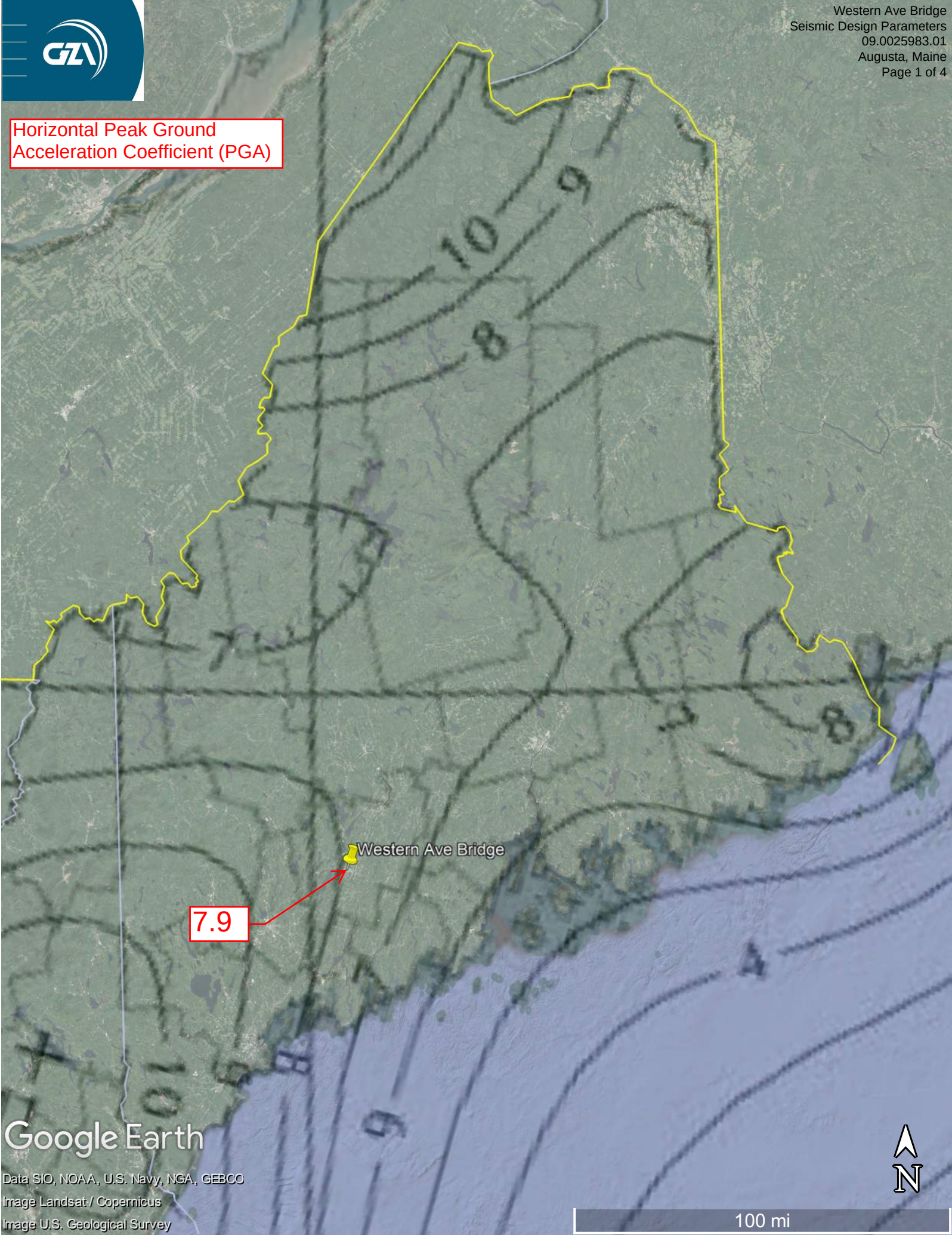


Figure C3.11.5.3-1—Application of (a) Rankine and (b) Coulomb Earth Pressure Theories in Retaining Wall Design

Seismic



Horizontal Peak Ground
Acceleration Coefficient (PGA)



Google Earth

Data SIO, NOAA, U.S. Navy, NGA, GEBCO

Image Landsat / Copernicus

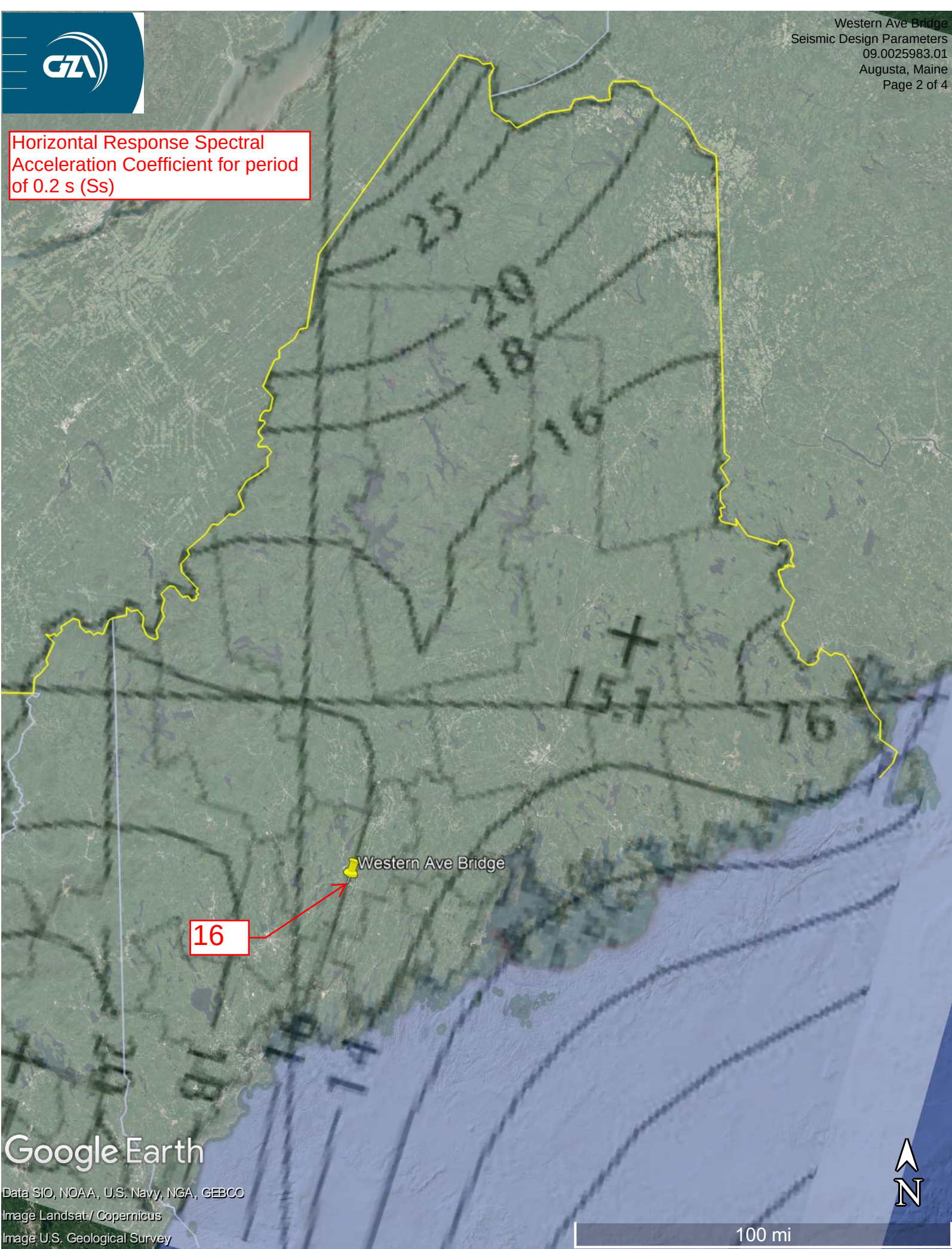
Image U.S. Geological Survey



100 mi



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



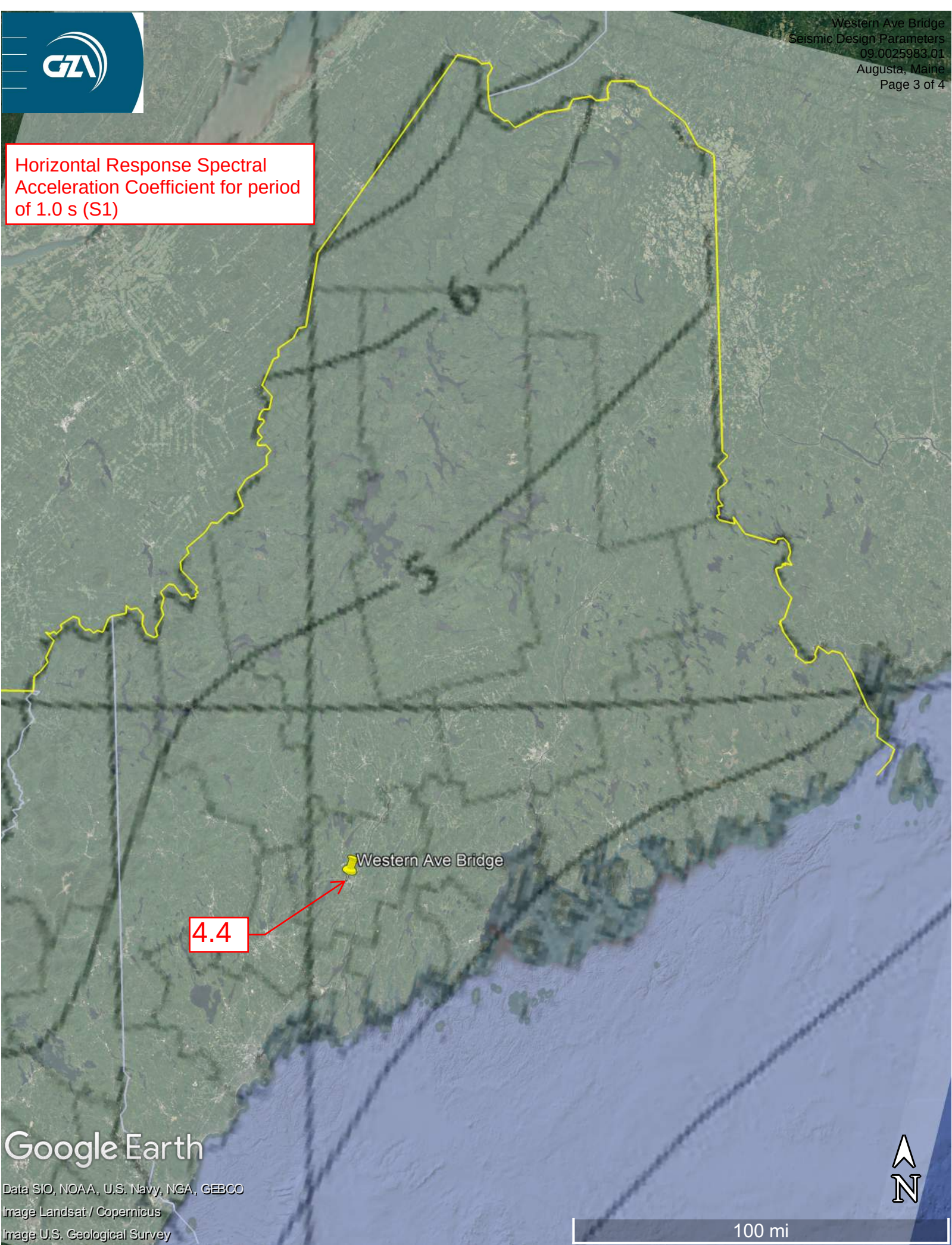
Google Earth

Data SIO, NOAA, U.S. Navy, NGA, GEBCO
Image Landsat / Copernicus
Image U.S. Geological Survey

100 mi



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



Google Earth

Data SIO, NOAA, U.S. Navy, NGA, GEBCO

Image Landsat / Copernicus

Image U.S. Geological Survey



100 mi



Site Class Seismic Design Parameters

Western Avenue/I-95 Bridge No. 5808 Over I-95

Augusta, Maine

GZA Job#: 09.0025983.01

Seismic Interpolation Coefficients		
Seismic Parameter	Interpolated Value from Maps ¹	Design Parameter
Horizontal Peak Ground Acceleration Coefficient (PGA)	7.9	0.079
Horizontal Response Spectral Acceleration Coefficient for Period of 0.2s, S _s	16.0	0.160
Horizontal Response Spectral Acceleration Coefficient for Period of 1.0s, S ₁	4.4	0.044

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 Site Class B
$F_{PGA} =$	1
$F_a =$	1
$F_v =$	1

Therefore:

$A_s = F_{PGA} \times PGA =$	0.079
$S_{DS} = F_a \times S_s =$	0.160
$S_{D1} = F_v \times S_1 =$	0.044

Summary:

SITE CLASS B SEISMIC DESIGN PARAMETERS	
Parameter	Design Value
F _{pga}	1
F _a	1
F _v	1
A _s (Period = 0.0 sec)	0.08
S _{Ds} (Period = 0.2 sec)	0.16
S _{D1} (Period = 1.0 sec)	0.04

Frost

Figure 5-1 Maine Design Freezing Index Map

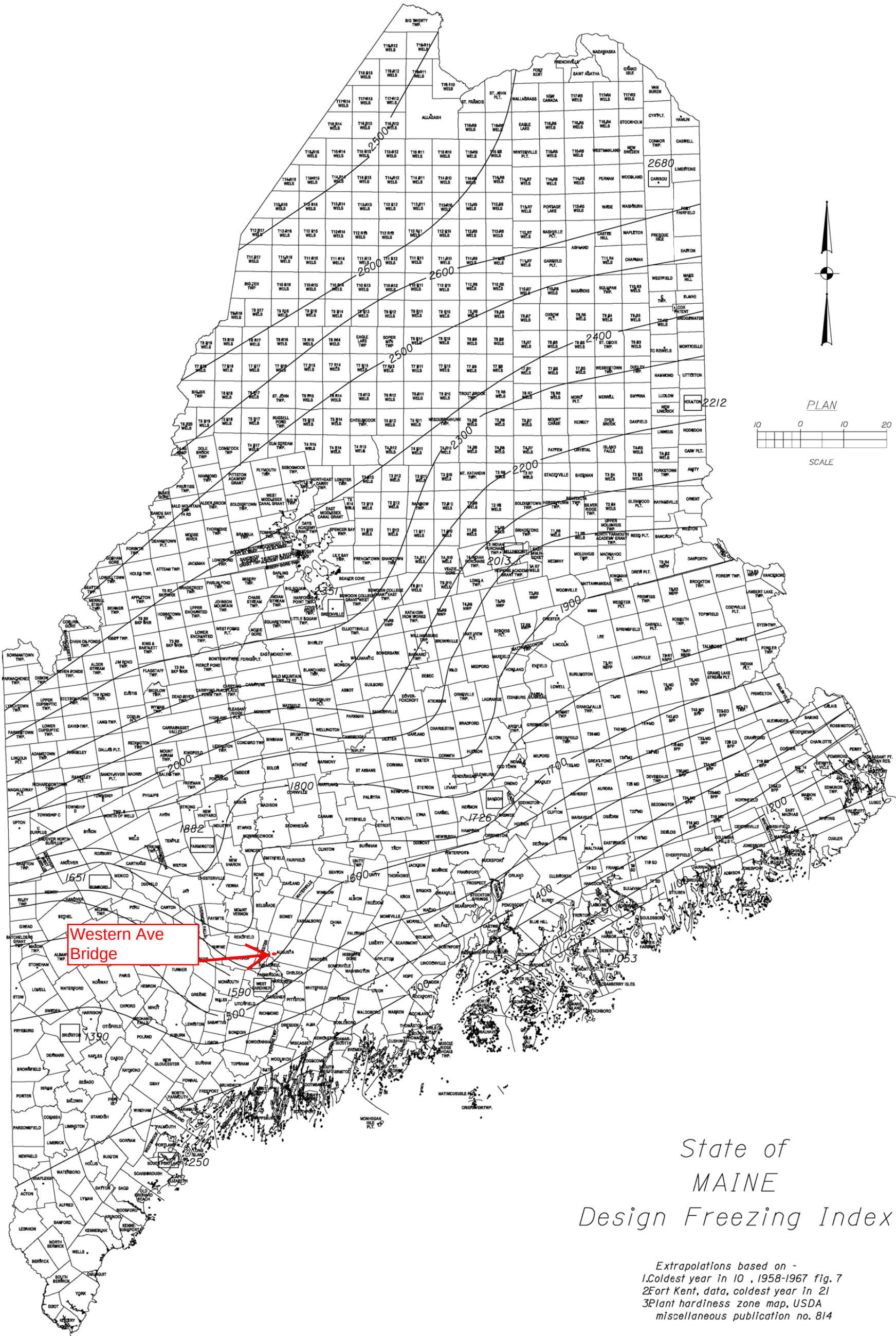


Table 5-1 Depth of Frost Penetration

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	97.5	80.7	69.2	69.3	59.7	53.8
2200	100.0	82.6	70.8	71.0	61.1	55.1
2300	102.3	84.5	72.4	72.7	62.5	56.4
2400	104.6	86.4	74.0	74.3	63.9	57.6
2500	106.9	88.2	75.6	75.9	65.2	58.8
2600	109.1	89.9	77.1	77.5	66.5	60.0

- 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.

Glacial till is anticipated to be present at the abutments near the elevation of the footings. The material is coarse-grained with water contents approximately 20%. Based on the MaineDOT BDG, Section 5.2.1 and a Freezing Index of 1,615 the estimated depth of frost penetration is 70.5 inches or 5.9 feet.

Bearing Resistance for Existing Abutments



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Near slope, Construction Case
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Objective

Calculate soil bearing resistance for the construction case for foundation bearing on soils with a friction angle equal to 38 degrees for existing fill at the site using the Theoretical method (Munfakh et al., 2001) in Sand using SPT data. Evaluate an excavation geometry that will allow the footing to maintain a factored bearing resistance for strength condition equal to the design bearing capacity from the as-built drawings for the existing footing width of 7'-3" from as-built plans.

The construction case refers to a need during construction of a new abutment foundation to excavate a steeper slope (1.5H:1V) between the foundation and I-95, as shown on Sheet 3 of this package.

References

1. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 9th edition, 2020 (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

$\phi_f := 38\text{deg}$	The recommended internal friction angle to be used for design is based on the correlation of the SPT N-value presented by Peck, Hanson and Thornburn (PHT, 1974) as mentioned in Kulhawy and Mayne (1990) and recommended in NCHRP Report 651 (below).		
$\phi_b := 0.45$	Bearing resistance factor as specified in Table 10.5.5.2.2-1 (Theoretical Method, SPT Data, Strength Limit)		
$c := 0\text{ksf}$	Cohesion, taken as undrained shear strength from correlated N values and Pocket Penetrometer data		
$\gamma := 125\text{pcf}$	Unit weight of soil above or below the bearing depth of the footing		
$N_c := 61.4$	Cohesion term bearing capacity factor as specified in Table 10.6.3.1.2a-1		
$N_q := 48.9$	Surcharge term bearing capacity factor as specified in Table 10.6.3.1.2a-1		
$N_\gamma := 78$	Total unit weight term bearing capacity factor as specified in Table 10.6.3.1.2a-1		
$C_{wq}, C_{wy} :=$	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)	$C_{wq} := 1$	$C_{wy} := .5$
$d_q :=$	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		
$S_c, S_\gamma, S_q :=$	Footing shape correction factors as specified in Table 10.6.3.1.2a-2		
$N_{60} := 20$	Average N_{60} values from SPT below footing		
	Load inclination factors are omitted considering modest embedment of footing per C10.6.3.1.2a.		



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Near slope, Construction Case
 SHEET: 2 OF 7
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Footing Dimensions

$B_1 := 7.25\text{ft}$ As-Built plans indicate 7'-3" wide footings

$L_1 := 100\text{ft}$ Length of footing

$D_f := 6.5\text{ft}$ Footing embedment depth

Strength Limit Design

$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5 \gamma B_1 N_{\gamma m} C_{w\gamma}$ Nominal Bearing Resistance Formula

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

Correction Factors

$$d_{q\text{table}} := \frac{D_f}{B_1} \quad d_q := \min \left[1 + 2 \tan(\phi_f) (1 - \sin(\phi_f))^2 \cdot \text{atan} \left(\frac{D_f}{B_1} \right), 1.4 \right] = 1.17 \quad \text{Using Eq. 10.6.3.1.2a-10}$$

$d_{q\gamma} := 1$ d_q assumed soil above footing less competent than soil below footing.

$$s_c := 1 + \left(\frac{B_1}{L_1} \right) \left(\frac{N_q}{N_c} \right) \quad s_c = 1.06$$

$$s_q := 1 + \left(\frac{B_1}{L_1} \tan(\phi_f) \right) \quad s_q = 1.06$$

$$s_\gamma := 1 - 0.4 \left(\frac{B_1}{L_1} \right) \quad s_\gamma = 0.97$$

Bearing Capacity Factors

$$N_{cm} := N_c \cdot s_c \quad N_{cm} = 64.95$$

$$N_{qm} := N_q \cdot s_q \cdot d_q \quad N_{qm} = 51.7$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \quad N_{\gamma m} = 75.7$$



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Nominal Bearing Resistance

$$q_n := \overline{(c \cdot N_{cm} + \gamma \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma \cdot B_1 \cdot N_{\gamma m} \cdot C_{w\gamma})} \quad q_n = 59.1 \cdot \text{ksf}$$

Factored Bearing Resistance - Strength Limit State

$$q_D := \phi_b \cdot q_n \quad q_D = 26.6 \cdot \text{ksf} \quad \text{for } B_1 = 7.25 \cdot \text{ft}$$

Reduction for Footing Adjacent to Slope

Used section C10.6.3.1.2c to determine the reduction coefficient for bearing resistance due to slope effects. The reduction coefficient was interpolated from Table 10.6.3.1.2c-2.

$$B := B_1 = 7.3 \cdot \text{ft} \quad \text{Footing Width}$$

$$H_s := 12.5 \cdot \text{ft} \quad \text{Height of sloping ground surface below bottom of footing}$$

$$b := 6.75 \cdot \text{ft} \quad \text{Distance from soil slope face to edge of footing}$$

$$b_s := 28 \cdot \text{ft} \quad \text{Slope width}$$

$$\beta := \text{atan}\left(\frac{H_s + D_f}{b_s}\right) = 34.2 \cdot \text{deg} \quad \text{Slope angle}$$

$$N_s := \frac{\gamma \cdot H_s}{c} = 1 \quad \text{Slope stability factor. Neglect if cohesionless and use } c = 0 \text{ column in table 10.6.3.1.2c-2.}$$

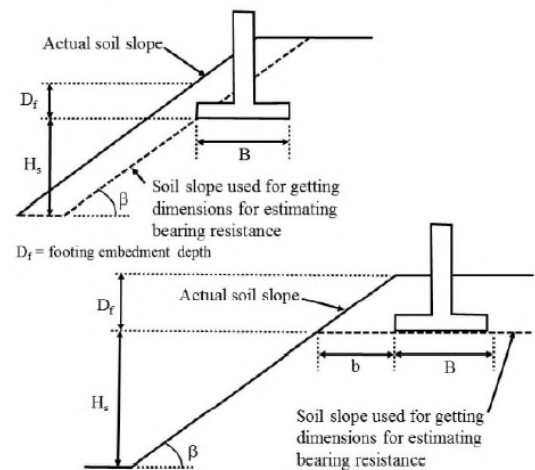
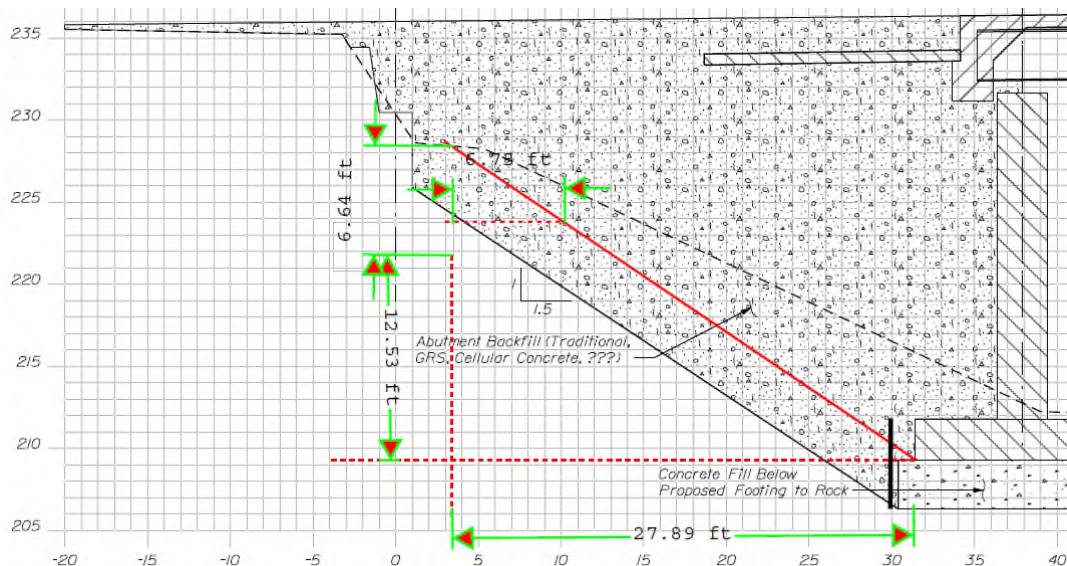


Figure 10.6.3.1.2c-1—Definition of Footing and Slope Geometric Parameters for Determination of RC_{BC}





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$$\frac{b}{B} = 0.9 \quad \frac{B}{H_s} = 0.58 \quad \text{dimensionless factors for Table 10.6.3.1.2c-2}$$

$$RB_{CB} := .2 \quad \text{Interpolated reduction coefficient for modified bearing capacity method.}$$

Modified Nominal Bearing Resistance

$$q_{n.sloping.ground} := RB_{CB} \cdot q_n = 11.8 \cdot \text{ksf} \quad \text{Modified bearing capacity}$$

Factored Bearing Resistance - Strength Limit State

$$q_{D.sloping.ground} := \phi_b \cdot q_{n.sloping.ground} = 5.3 \cdot \text{ksf}$$

The factored bearing resistance is 5.3 ksf (2.6 tsf). The design bearing pressure shown on the existing plans is 2.6 tsf; therefore a 1.5H:1V slope located 7 feet laterally outside of the bottom edge of the footing will provide suitable bearing resistance during construction.



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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
	$\phi_f > 0$	$1 + \left(\frac{B}{L} \right) \left(\frac{N_q}{N_c} \right)$	$1 - 0.4 \left(\frac{B}{L} \right)$	$1 + \left(\frac{B}{L} \tan \phi_f \right)$



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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.



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Table 10.6.3.1.2c-2 (cont.)— Reduction Coefficients (RC_{2c}) for Footings Placed Adjacent to Slopes Composed of either Purely Cohesive Soils, ($\phi = 0$); Purely Cohesionless Soils ($c'=0$); or Soils with both Cohesive and Cohesionless Strength Components

ϕ (°)	B/H	b/B	$\beta=10^\circ$				$\beta=20^\circ$				$\beta=30^\circ$				$\beta=40^\circ$			
			N_f				N_f				N_f				N_f			
			0	2	4	$c'=0$	0	2	4	$c'=0$	0	2	4	$c'=0$	0	2	4	$c'=0$
30	0.2	0	0.93	0.92	0.91	0.76	0.65	0.64	0.63	0.39	0.51	0.50	0.48	0.11	0.40	0.37	0.36	0.00
		0.5	0.74	0.81	0.80	0.75	0.70	0.66	0.65	0.50	0.57	0.52	0.49	0.21	0.47	0.42	0.39	0.00
		1.25	0.78	0.85	0.86	0.86	0.74	0.73	0.72	0.72	0.63	0.60	0.59	0.38	0.54	0.50	0.47	0.00
		2.5	0.84	0.92	0.93	0.99	0.81	0.82	0.83	0.94	0.72	0.73	0.74	0.74	0.64	0.62	0.61	0.00
		5	0.95	1.00	1.00	1.00	0.93	0.98	1.00	1.00	0.88	0.95	1.00	0.97	0.80	0.85	0.87	0.00
		10	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00
	0.5	0	0.79	0.79	0.78	0.70	0.63	0.59	0.55	0.36	0.50	0.43	0.39	0.13	0.39	0.32	0.27	0.00
		0.5	0.76	0.87	0.87	0.74	0.72	0.71	0.70	0.51	0.58	0.56	0.54	0.24	0.49	0.46	0.43	0.00
		1.25	0.79	0.85	0.92	0.87	0.75	0.73	0.76	0.72	0.63	0.62	0.61	0.45	0.54	0.52	0.50	0.00
		2.5	0.87	0.91	1.00	0.99	0.84	0.85	0.90	0.98	0.74	0.78	0.80	0.80	0.67	0.70	0.71	0.00
		5	0.97	1.00	1.00	1.00	0.95	1.00	1.00	1.00	0.90	1.00	1.00	1.00	0.85	0.94	0.98	0.00
		10	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00
	1	0	0.79	0.75	0.73	0.67	0.63	0.53	0.49	0.41	0.55	0.41	0.35	0.24	0.48	0.33	0.26	0.00
		0.5	0.78	0.87	0.89	0.74	0.75	0.74	0.74	0.51	0.64	0.62	0.60	0.35	0.59	0.56	0.54	0.00
		1.25	0.81	0.90	0.91	0.88	0.78	0.78	0.78	0.72	0.68	0.67	0.66	0.58	0.64	0.62	0.61	0.00
		2.5	0.88	0.99	1.00	0.96	0.85	0.90	0.92	0.95	0.78	0.81	0.84	0.88	0.75	0.78	0.80	0.00
		5	0.97	1.00	1.00	1.00	0.96	1.00	1.00	1.00	0.92	1.00	1.00	1.00	0.89	0.98	1.00	0.00
		10	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00
	2	0	0.88	0.88	0.87	0.65	0.87	0.85	0.83	0.48	0.85	0.82	0.80	0.38	0.83	0.80	0.76	0.00
		0.5	0.89	0.91	0.91	0.75	0.89	0.89	0.87	0.58	0.88	0.86	0.84	0.51	0.87	0.85	0.82	0.00
		1.25	0.90	0.92	0.93	0.88	0.90	0.90	0.90	0.75	0.89	0.87	0.87	0.70	0.89	0.87	0.86	0.00
		2.5	0.97	1.00	1.00	1.00	0.96	0.97	0.98	0.98	0.92	0.94	0.96	0.95	0.91	0.92	0.94	0.00
		5	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00
		10	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00
40	0.2	0	0.69	0.69	0.69	0.78	0.51	0.48	0.47	0.37	0.37	0.33	0.30	0.16	0.27	0.23	0.20	0.05
		0.5	0.65	0.73	0.71	0.74	0.60	0.55	0.53	0.38	0.64	0.38	0.35	0.25	0.34	0.29	0.25	0.13
		1.25	0.68	0.77	0.75	0.86	0.63	0.60	0.58	0.55	0.74	0.44	0.42	0.39	0.39	0.34	0.31	0.25
		2.5	0.72	0.83	0.84	1.00	0.68	0.68	0.68	0.76	0.87	0.53	0.53	0.62	0.45	0.43	0.41	0.48
		5	0.80	0.93	0.95	1.00	0.76	0.82	0.85	1.00	1.00	0.72	0.76	1.00	0.57	0.61	0.63	0.94
		10	0.94	1.00	1.00	1.00	0.91	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.76	0.93	1.00	1.00
	0.5	0	0.67	0.69	0.67	0.69	0.50	0.45	0.43	0.35	0.36	0.30	0.26	0.17	0.27	0.20	0.17	0.07
		0.5	0.68	0.81	0.81	0.73	0.63	0.62	0.61	0.46	0.47	0.44	0.41	0.25	0.39	0.35	0.32	0.09
		1.25	0.70	0.82	0.84	0.85	0.65	0.65	0.66	0.60	0.51	0.49	0.47	0.40	0.43	0.41	0.39	0.18
		2.5	0.76	0.92	0.96	1.00	0.72	0.77	0.80	0.81	0.59	0.62	0.63	0.60	0.54	0.56	0.56	0.37
		5	0.84	1.00	1.00	1.00	0.81	0.91	0.94	1.00	0.71	0.82	0.88	1.00	0.67	0.77	0.83	0.84
		10	0.96	1.00	1.00	1.00	0.94	1.00	1.00	1.00	0.89	1.00	1.00	1.00	0.86	1.00	1.00	1.00
	1	0	0.69	0.64	0.62	0.70	0.63	0.48	0.43	0.45	0.58	0.39	0.33	0.32	0.54	0.33	0.27	0.24
		0.5	0.77	0.81	0.82	0.74	0.75	0.73	0.72	0.49	0.71	0.66	0.62	0.38	0.68	0.62	0.57	0.30
		1.25	0.78	0.84	0.85	0.84	0.77	0.76	0.75	0.64	0.73	0.69	0.66	0.55	0.71	0.66	0.63	0.48
		2.5	0.83	0.92	0.95	1.00	0.81	0.85	0.87	0.85	0.76	0.78	0.79	0.76	0.75	0.76	0.77	0.72
		5	0.89	1.00	1.00	1.00	0.87	0.95	0.98	1.00	0.80	0.90	0.95	1.00	0.80	0.89	0.94	1.00
		10	0.98	1.00	1.00	1.00	0.97	1.00	1.00	1.00	0.94	1.00	1.00	1.00	0.93	1.00	1.00	1.00
	2	0	0.93	0.92	0.89	0.45	0.92	0.90	0.87	0.60	0.91	0.88	0.84	0.53	0.89	0.85	0.81	0.47
		0.5	0.93	0.95	0.93	0.76	0.93	0.92	0.90	0.65	0.92	0.89	0.87	0.64	0.92	0.89	0.86	0.60
		1.25	0.93	0.95	0.94	0.86	0.93	0.93	0.92	0.78	0.93	0.91	0.89	0.74	0.93	0.90	0.88	0.74
		2.5	0.94	0.99	1.00	1.00	0.94	0.98	0.98	0.92	0.94	0.97	0.97	0.87	0.94	0.96	0.96	0.88
		5	0.95	1.00	1.00	1.00	0.96	1.00	1.00	1.00	0.98	1.00	1.00	1.00	0.96	1.00	1.00	1.00
		10	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.99	1.00	1.00	1.00