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GEOTECHNICAL DESIGN REPORT

RINES HILL BRIDGE NO. 3528 OVER OLD MAINE

CENTRAL RAILROAD

MAINE DOT WIN 021872.00
AUGUSTA, MAINE

Prepared for:
Stantec
Portland, Maine

October 2022
09.0025943.01

Prepared by:
GZA GeoEnvironmental, Inc.
707 Sable Oaks Drive | Suite 150 | South Portland, Maine 04106
207.358.5126

31 Offices Nationwide



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



VIA EMAIL

October 18, 2022
File No. 09.0025943.01

Mr. Tim Merritt, P.E.
Stantec
2211 Congress Street, Suite 380
Portland, Maine 04102

Re: Geotechnical Design Report
Rines Hill Bridge No. 3528
Water Street over Old Maine Central Railroad
Maine Department of Transportation WIN 021872.00
Augusta, Maine

Dear Laura:

We are pleased to provide this Geotechnical Design Report for Maine Department of Transportation (MaineDOT) Bridge No. 3528 over Old Maine Central Railroad in Augusta, Maine. Our services were provided in accordance with Master Services Agreement 40839 between Stantec and GZA, and Stantec Task Order JN 195311775, dated August 13, 2020, which incorporates GZA's proposal No. 09.P000048.20, dated June 21, 2019, and the *Limitations* included in **Appendix A** of this report.

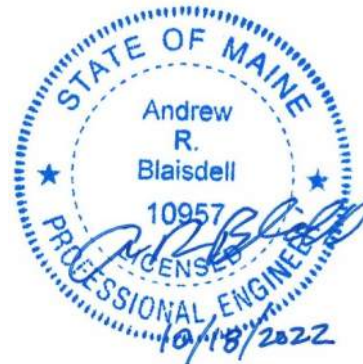
It has been a pleasure serving Stantec on this project. If you have any questions regarding the report, or if we can provide further assistance, please do not hesitate to contact the undersigned.

Very truly yours,

GZA GEOENVIRONMENTAL, INC.

Nicholas Williams, P.E.
Project Manager

Christopher L. Snow, P.E.
Consultant Reviewer



Andrew R. Blaisdell, P.E.
Associate Principal

NVW/ARB/CLS:erc

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Attachment: Geotechnical Design Report



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1.0 INTRODUCTION

This report presents the results of GZA GeoEnvironmental, Inc.'s (GZA's) geotechnical evaluation for the proposed replacement of Maine Department of Transportation (MaineDOT) Rines Hill Bridge No. 3528 over the Old Maine Central Railroad in Augusta, Maine. Our services were provided in accordance with Master Services Agreement 40839 between Stantec and GZA, and Stantec Task Order JN 195311775, dated August 13, 2020, which incorporates GZA's proposal No. 09.P000048.20, dated June 21, 2019, and the *Limitations* included in **Appendix A** of this report.

1.1 BACKGROUND

The existing Rines Hill Bridge No. 3528 was constructed in approximately 1939 and spans north to south, carrying Water Street over the Old Maine Central Railroad (MCRR). The 1939 superstructure was supported on reconfigured abutments and wingwalls of the older MCRR Bridge OH62.4. The bridge has a length of 68 feet and a width of 61 feet. The bridge is skewed approximately 56 degrees from the alignment of Water Street, and the deck slopes down from south to north at an inclination of approximately 11 percent. The superstructure consists of concrete-encased rolled steel beams with a composite concrete deck. The top of the bridge deck is approximately 20 to 35 feet above the ground surface where it passes over the MCRR. The MCRR at this location carries local bus traffic from a park-and-ride lot immediately southeast of the bridge. The MCRR surface consists of processed gravel on the west side and asphalt pavement along the east side.

The plans (all references to plans allude to the 1939 plans) indicate a reinforced concrete cap was doweled into and poured over the top of pre-existing abutments to support the current bridge. The available plans do not include design details for the abutments. Based on our review of the photographs provided by Stantec, our site visits, and the geophysical field test results, we have not observed signs that would indicate the abutments consist of masonry walls with concrete facing.

Sheet 37-13 of the existing plans shows roadway section details as summarized in the table below:

Location	North Approach	South Approach East	South Approach West
Bituminous Thickness	2"	2"	2"
Leveling Mix Thickness	1" +/-	--	--
Bituminous Base Thickness	--	6"	--
Reinforced Concrete Slab Thickness	--	--	7"
Bottom of Pavement Section	Existing Cobblestone Pavement	Existing Cobblestone Pavement	12" Gravel Base

The plans do not indicate that any subgrade preparation was done in areas where existing cobblestone pavement was present, while gravel base and reinforced concrete were used in areas that may not have contained cobblestone pavement. Sheets 37-37 and 37-39 of the plans provide additional information on the plan limits and details for the reinforced concrete slab, which was placed in approximately 10-foot by 25-foot typical sections. The plans indicate that approximately the western half of the approach roadway is underlain by the concrete slab, and the slab width increases to underly all of Grove Street farther south from the bridge.

Existing retaining walls extend approximately 250 to 300 feet or more in both directions past the ends of the south abutment. The wall height increases from northwest to southeast, with a maximum height of about



30 feet southeast of the bridge. The area above the south wall slopes steeply toward the wall and is wooded in some areas. The walls are shorter along both sides of the north abutment, retaining embankment slopes that are inclined down toward the MCRR elevation on both sides. The construction of the walls appears to be similar to the abutments, and it is considered likely that the walls were constructed at the same time as the abutments, during construction of the MCRR line.

GZA reviewed the history of the railroad in the area. The *Maine Central Railroad Bridge Inventory* indicates that the nearest upstream bridge crossing the Kennebec River was replaced in approximately 1914. The piers are still visible in the river and indicate the bridge was relocated north and west of the original. The United States Geological Survey (USGS) topographic maps show a similar westward realignment of the railroad at the Rines Hill Underpass between 1892 and 1943. Based on this information and the bridge inventory, we conclude the railroad was relocated and the Rines Hill Bridge substructures were probably completely reconstructed within the 1914 timeframe.

Deterioration of the abutments was documented in the bridge inspection reports and includes cracking and scaling of the concrete with areas of seepage. The northerly bearing has moderate bridge seat spalling around the bearing masonry plate. At the south abutment, large delamination and spall areas are present on the abutments and retaining walls. At the time of GZA's site visits in March 2017 and July 2017, it was not apparent that the abutment performance had negatively affected the serviceability of the bridge.

We understand that the preferred alternative will consist of a 63.5-foot-long single span superstructure replacement supported by the existing abutments, resulting in minimal grade changes, and no bridge widening. Reuse of the existing abutment will require staged construction to maintain one lane of traffic to complete the bearing seat replacement and necessary backwall work.

1.2 OBJECTIVES AND SCOPE OF SERVICES

The objectives of our work were to evaluate subsurface conditions and provide final geotechnical design recommendations and construction considerations for bridge superstructure replacement. To meet these objectives, GZA completed the following Scope of Services:

- Conducted site visits to observe surficial conditions and reviewed mapped surficial geology of the site;
- Coordinated and observed a preliminary series of borings and probes to evaluate subsurface conditions and interpolate the existing abutment geometries;
- Coordinated a preliminary ground-penetrating radar (GPR) survey to characterize the geometry of the abutments and retaining walls;
- Coordinated and observed a supplemental series of borings and probes to further evaluate the existing abutment geometries, materials, and bearing conditions;
- Coordinated a supplemental GPR survey to characterize the Water Street pavement section, underlying materials, and their lateral extents;
- Conducted a laboratory testing program to evaluate engineering properties of the site soils and bedrock;
- Completed geotechnical evaluations for soil and bedrock properties; AASHTO load and resistance factors associated with geotechnical design elements; existing abutment geometry; existing spread footing bearing and lateral resistance; and seismic design considerations;



- Developed recommended geotechnical engineering design parameters for use in evaluating existing substructures, including bearing resistance for spread footings; lateral earth pressures and resistance; and seismic design parameters; and
- Prepared this report summarizing our findings and design recommendations.

2.0 SUBSURFACE EXPLORATIONS

2.1 TEST BORINGS

GZA completed a preliminary design exploration program in 2017 consisting of seven (7) test borings and five (5) probes, and a supplemental exploration program in 2019 consisting of two (2) test borings and four (4) groups of probes and cores. The approximate as-drilled exploration locations of the preliminary explorations were surveyed by MaineDOT, and the supplemental exploration locations were estimated by tape measurement from existing bridge components, and are shown on **Figure 2**. Ground surface elevations for 2019 borings and probes were estimated from available contour data. Elevations referenced in this report are in feet and refer to the National American Vertical Datum of 1988 (NAVD88).

New England Boring Contractors (NEBC) of Hermon, Maine provided drilling services and coordinated utility clearance. GZA personnel monitored the drilling work and prepared logs of each boring and of the probes that were cored. The logs are included in **Appendix B**.

The borings and probes were drilled using a track-mounted drill rig, a truck-mounted drill rig, and a trailer-mounted drill rig. The borings and probes were backfilled with cuttings, and roadway locations were patched with asphalt cold patch.

The borings were drilled using 3- and 4-inch casing, using drive-and-wash and/or spin-and-wash methods, or hollow stem auger drilling techniques. Measured energy transfer ratios for automatic hammers are noted on the boring logs in **Appendix B**.

2.1.1 Preliminary Borings

GZA and NEBC completed a subsurface exploration program consisting of seven (7) test borings (BB-ARH-101 through -105, and RB-ARH-101 and -102) and five (5) probes (BB-ARH-101A through -101C, BB-ARH-104A and -104B) between July 10 and July 27, 2017. Three borings were drilled behind the abutments (BB-ARH-101, -104, and -105), two were drilled between the abutments under the existing bridge deck (BB-ARH-102 and -103), and two were drilled along the rail trail north and south of the alignment (RB-ARH-101 and -102).

The borings were drilled to depths of approximately 32 to 91 feet below ground surface (BGS). Four borings (BB-ARH-101, -102, -103 and -105) were terminated approximately 10 feet into bedrock. Boring BB-ARH-104 was terminated at 33 feet BGS after the drill casing was broken, and RB-ARH-101 and -102 were terminated in medium dense to dense granular soil.

Standard penetration testing (SPT) and split-spoon sampling were performed at 5-foot typical intervals in the overburden using a 24-inch-long, 1-3/8-inch inside-diameter sampler, driven with a calibrated automatic hammer or a rope and cathead. An in-situ field vane shear test was taken within a softer portion of a clay layer,



and pocket penetrometer testing was conducted on split-spoon samples of stiffer clay. Bedrock cores were obtained using NQ2 wire-line coring equipment in four of the test borings

The BB-ARH-104 group of probes (A-B) was conducted behind the south abutment and the BB-ARH-101 group of probes (A-C) was conducted behind the north abutment. Each probe group was conducted in a line roughly perpendicular to the respective abutment face to interpret the approximate top of abutment concrete. The probes were conducted using solid stem augers and were terminated upon practical refusal, described herein as continuous augering without achieving downward advancement, or at a depth approximately 10 feet below rail trail grade if refusal was not encountered.

2.1.2 Supplemental Borings

GZA and NEBC completed a supplemental subsurface exploration program between April 12 and April 18, 2019 consisting of two (2) test borings (BB-ARH-301 and -302), and four (4) groups of probes (BP-ARH-201 through -204). The borings were drilled behind the front face of the abutments and the probes were drilled from the MCRR in front of the abutments.

The borings were drilled to depths of approximately 48 to 58 feet BGS. The borings were drilled through existing abutment concrete and were terminated approximately 20 feet below the bottom of the abutment in medium dense to dense granular soil.

SPT and split-spoon sampling were performed continuously through the overburden material above the concrete abutments in both borings and either continuously or at 5-foot intervals beneath the abutments using a 24-inch-long, 1-3/8-inch inside-diameter sampler, driven with a calibrated automatic hammer. Concrete cores were obtained from the abutment using HQ2 wire-line coring equipment in the two test borings.

Each group of probes (4 to 5 probes per group) was conducted in front of the base of the abutment at offsets between 4 and 24 inches away from the face of the wall. Each group was conducted in a line with varying distances from the abutment, approximately perpendicular to the abutment face. One core was drilled through the abutment footing in BP-ARH-201 and -204 to determine the thickness of the concrete toe, and the remaining probes were conducted using solid stem augers and were terminated upon practical refusal or continued to depths below the encountered top of concrete. We interpreted this latter condition to indicate that the probe was beyond the horizontal limit of the footing.

2.2 GEOPHYSICAL SURVEY

2.2.1 Initial Ground-Penetrating Radar Survey

Northeast Geophysical Services, Inc. (NGS) of Bangor, Maine provided GPR survey services to attempt to characterize the geometry of the abutments and retaining walls. GPR data were acquired on the face of the abutments and retaining walls along several vertically- and horizontally-oriented lines of coverage and on the ground surface on the MCRR using a GSSI S3000 GPR system with a 400 MHz antenna.

The GPR lines are summarized as follows:

- Four vertical lines on the face of each abutment, extending from ground surface up to 20 feet above existing ground surface (designated V1 through V8);



- One vertical line on the face of the retaining wall beyond the south bridge abutment, extending from ground surface up to 15 feet above existing ground surface (designated V9);
- One horizontal line along the face of each abutment, just above the existing ground surface (designated L1 and L2); and
- Seven lines on the MCRR ground surface between the abutments, perpendicular to the abutments (aligned with the vertical wall profiles (designated P1 through P5).

The results of the GPR survey are documented in a report prepared by NGS dated June 2017, which is presented in **Appendix C**. The report provides additional details on the GPR line locations and interpretations.

A MaineDOT crew surveyed the coordinates and ground surface elevation at the ends of the survey lines, where accessible, and provided this information to GZA as an electronic data file. A GZA engineer was on site to provide oversight and assist our geophysical subconsultant during the field work.

2.2.2 Supplemental Ground-Penetrating Radar Survey

NDT Corporation (NDT) of Sterling, Massachusetts provided GPR survey services to characterize the Water Street pavement section and underlying materials and their and lateral extents. GPR data were acquired along the roadway and sidewalk of the existing approaches along longitudinal lines (parallel to the alignment) at 5-foot spacing, and along transverse lines at 20-foot spacing extending approximately 100 to 160 feet behind each abutment. GPR data were acquired via a digital system coupled with 400 MHz, 900 MHz, and 1500 MHz antennas. The locations of the GPR lines were estimated by NDT by taping from the end of the bridge.

The results of the GPR survey are documented in a report prepared by NDT dated May 1, 2019, which is presented in **Appendix D**. The report provides additional details on the GPR line locations and interpretations.

A GZA engineer was on site to provide oversight and assist our geophysical subconsultant during the field work.

3.0 LABORATORY TESTING

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

- 30 gradation analysis / AASHTO Classification / Unified Soil Classification System / Frost Classification assessments;
- Two sets of Atterberg Limits; and
- 33 moisture content tests.

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



4.0 SUBSURFACE CONDITIONS

4.1 SURFICIAL AND BEDROCK GEOLOGY

Based on available surficial geologic mapping¹, the soil unit along the river is Stream Alluvium consisting of sand, gravel and silt deposited on flood plains of the Kennebec River. Farther inland, the unit transitions to Stream Terraces consisting of sand and gravel deposited by the Kennebec River at elevations higher than the most recent flood plains; the unit is mapped as Presumpscot Formation consisting of glaciomarine silt, clay, and sand deposited on the late-glacial sea floor. Glacial till is also mapped in the vicinity of the site. It consists of a stratified mixture of sand, silt and gravel-size rock debris deposited by glacial ice. Glacial till is typically the base deposit beneath those described previously.

Bedrock in the vicinity of the site is mapped² as the Mayflower Hill formation, the Waterville formation and as Hallowell granite. The Mayflower Hill formation is characterized as fine to medium grained quartz-feldspar-biotite granofels and minor schist. The Waterville formation is characterized as thinly-bedded granofels and schist. Both formations are 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

Three soil units were encountered in the test borings overlying bedrock: Fill, Marine Clay (Presumpscot Formation), and Glacial Till. Approximately 10 to 11 inches of asphalt pavement overlying 4 to 11 inches of cobblestones and/or concrete was encountered in the borings drilled through the Water Street roadway. This indicates the presence of the cobblestone and concrete slab layers shown on the plans, but during auger advancement, only grey granular material was returned, so the difference between these materials could not be determined in the BB-ARH-100 series borings and probes. Concrete was visually confirmed beneath the pavement in borings BB-ARH-301 and BB-ARH-302. Concrete was also confirmed in these borings when drilling through the back of the abutments, confirming the abutments are concrete and is not a granite block gravity wall, as described further herein.

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 **Figure 3** and **Appendix B**.

¹ Thompson, Woodrow B., 2009, Surficial geology of the Augusta 7.5-minute quadrangle, Kennebec County, Maine: Maine Geological Survey, Open-File No. 09-7

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



GENERALIZED SUBSURFACE CONDITIONS		
Soil Unit	Approximate Encountered Thickness (ft)	Generalized Description
Fill	2 to 32	Light tan/brown, loose to very dense, fine to coarse SAND, trace to some Gravel, trace to some Silt (USCS: SM, SP-SM). Primarily loose behind existing abutments and medium dense to very dense on the MCRR at toe of abutments/retaining walls. Locally included approximately 15 to 70 percent Silty CLAY (USCS: CL, SP-SC) in boring BB-ARH-104 at 1 to 8 feet BGS. Also included about 2 feet of Gravelly Clay (USCS: CL) in boring BB-ARH-301 at 28 to 29.5', immediately below the abutment footing. • MaineDOT Frost Classification = II-III (IV from 1 to 5 feet in BB-ARH-104 and 28 to 29.5 feet in BB-ARH-301) • Encountered in all borings
Marine Clay	24.5	Olive/Brown/Grey, stiff to medium stiff, Silty CLAY (USCS: CL). • Encountered in boring BB-ARH-101 only • MaineDOT Frost Classification = IV
Glacial Till	23.5 to 65	Variable ranging <u>from</u> : Brown, medium dense to very dense, fine to coarse SAND, trace to little Silt, varying amounts of Gravel <u>to</u> Dark brown, very dense, sandy GRAVEL, little Silt (USCS: SM, SP, SP-SM, GP-GM). • Encountered in all borings that penetrated Fill (all but BB-ARH-104)
Top of Bedrock Elevation	<u>Encountered Top of Rock:</u> Approx. El. 2 to El. -11	

4.2.1 Bedrock

Bedrock was cored in four (4) test borings and was described as Granite and Schist as summarized below.

Granite was found in two borings (BB-ARH-101 and -105), located above the Schist. It was typically described as very hard, fresh, coarse grained, light grey, GRANITE. Joints are very close to widely spaced, horizontal to moderately dipping, undulating, rough, fresh to discolored, tight to open, trace Silt infilling. The Rock Quality Designation (RQD) ranged from 75 to 82 percent, with an average of 79 percent.

Schist was the most prevalent rock type, found in all cored borings (BB-ARH-101 through -103, and BB-ARH-105). It was typically described as fresh, aphanitic to fine grained, dark grey banded, SCHIST. Joints are very close to moderately spaced, horizontal to high angle, planar to undulating, rough to smooth, discolored to decomposed (weathered mica infill), very tight, trace Silt infilling. The RQD ranged from 11 to 87 percent, with an average of 56 percent.

4.2.2 Groundwater

Groundwater was not encountered during the test borings. Moisture contents indicate soil is dry to moist throughout the test borings, therefore groundwater is anticipated to be 20 feet or more below the MCRR level.

The groundwater observations were made at the times and under the conditions stated in the boring logs. Fluctuations in groundwater levels will occur due to variations in seasonal influences, tide, precipitation amounts, and other factors. Consequently, water levels during and after construction are likely to vary from those encountered at the time the observations were made.



4.2.3 Concrete Pavement

NDT's results identified the presence of an apparent 12- to 14-inch-thick concrete pavement underlying 8 to 10 inches of asphalt pavement at the south approach. The geophysical survey indicates that concrete pavement extends from approximately the west curb (edge of sidewalk) to approximately 5 feet west of the east curb (Figure 3 of **NDT report, Appendix D**), extending beyond the southern survey limits 100 to 160 feet behind the abutment. The concrete appears to transition to a "change in material," possibly cobblestone, under the east side of the road. Approximately 8 to 10 inches of asphalt pavement over fill was shown beneath the sidewalk outside the limits of the concrete pavement and possible cobblestones. By comparison, the plans show concrete pavement extending from the west curb to approximately 2 feet across the roadway center line, where the material beneath the pavement transitions back to pre-existing cobblestones, extending beyond the limits of the geophysical survey to the south. Therefore, the results interpreted from the GPR are generally consistent with the concrete limits shown on the plans for the south approach, except that the location of the east edge of the concrete varies from that shown on the plans. A utility trench cut was also interpreted in the slab roughly 15 to 20 feet behind the abutment.

NDT's results also identified the presence of 12- to 14-inch-thick concrete pavement underlying 8 to 10 inches of asphalt pavement at the north approach, extending beyond the survey limits 100 to 140 feet behind the abutments. The plans do not show the concrete section beneath the pavement at the north approach. The geophysical results indicate the concrete extends from approximately the west curb to approximately 30 feet across the roadway (8 to 10 feet from the edge of the east curb). Approximately 8 to 10 inches of asphalt pavement over fill was shown beneath the sidewalk outside the limits of the concrete.

It is noted that GZA's borings and probes completed in Water Street all encountered either concrete or cobblestones beneath the asphalt pavement. In the 100-series borings, the nature of concrete versus cobblestones was not confirmed, but closer observation in the 300-series borings confirmed concrete at both approaches at the locations of BB-ARH-301 and -302.

5.0 ENGINEERING EVALUATIONS

Evaluations were completed based on the assumption that the preferred alternative will consist of a 63.5-foot-long single span superstructure replacement supported by the existing abutments, resulting in minimal grade changes, and no bridge widening.

5.1 GENERAL

GZA conducted geotechnical engineering evaluations based on the *2020 AASHTO LRFD Bridge Design Specifications, 9th Edition with Interims*, (AASHTO), and the *MaineDOT Bridge Design Guide, 2003 Edition*, with updates through 2018 (MaineDOT BDG, BDG). Supporting calculations developed by GZA for the project are attached in **Appendix F** of this report.



5.2 APPROACH EMBANKMENTS

The approach embankment grades will be maintained at the same approximate elevation and no widening of the slopes is planned. As a result, negligible post-construction settlement is anticipated at the approaches, and it is our opinion that the potential for global instability of the approach embankments is low.

5.3 LOAD AND RESISTANCE FACTORS

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

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

RESISTANCE FACTORS – STRENGTH LIMIT STATE		
Foundation Resistance Type	Method/Condition	Resistance Factor (ϕ)
Bearing	Theoretical method, in sand	$\phi_b = 0.45$
Sliding	Cast-in-Place Concrete on soil	$\phi_\tau = 0.80$
	Passive Earth Pressure Component of Sliding Resistance	$\phi_{ep} = 0.5$

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

5.4 SUBSTRUCTURE EVALUATIONS

5.4.1 Interpreted Abutment Geometry

Understanding the approximately 1914 vintage of the retaining walls that support the bridge, it is our belief that a concrete gravity structure would be historically appropriate at this site. GZA reviewed available literature and found that a widely-employed method of proportioning retaining walls in that era was the 1890 Trautwine empirical approach³. That method provided a wall base thickness to height (T:H) ratio of approximately 0.66. For a 30-foot wall height, the base thickness would be on the order of 20 feet, and for a 20-foot wall height, the base thickness would be on the order of 13 feet.

The existing abutment geometry was evaluated based on the information described above; review of data from borings, probes, geophysical testing; and existing bridge plans to develop interpretive cross-sections through the abutments. The estimated thickness of the abutment from the geophysical survey ranged from approximately 3.5 feet at the upper survey locations on the abutment face to as much as 8.5 feet at the trail level, which indicated a narrower wall thickness than would be interpreted from the probe data or anticipated based on typical wall-proportioning described above. NGS noted in their report that the GPR signal likely did not penetrate to the back of the retaining wall due to a combination of reinforcing steel reflections in the signal and lack of a dielectric contrast at the back of the wall. Therefore, the geophysical test results were inconclusive with regard to interpreting the full abutment profile.

³ Eschenasy, Dan, P.E., Inspection of Old Retaining Walls, NYC Buildings Presentation, Page 40, October 2015.

Borings BB-ARH-301 and BB-ARH-302 were cored through the concrete abutments to confirm the abutment material type and the depth of the top of the abutment backwall, and to assess the elevation of the base of the abutment. The cored abutment material appeared to be good quality and relatively fresh, with large rounded aggregate typical of early 1900s concrete. The cores did not encounter any reinforcing steel. Representative core photographs showing the concrete quality are presented in the photo below.



Cored Concrete, North Abutment (BB-ARH-302)

The BB-ARH-100 series probes and borings BB-ARH-301 and BB-ARH-302 were interpreted to estimate the depth to abutment concrete below Water Street pavement level at each of the locations, which are plotted on **Figure 4**. The probe and boring results indicate an average backwall batter that is consistent with the T:H ratio of 0.66 noted previously. Therefore, interpreted cross-sections were selected with a batter generally matching the probe and boring results. Borings BB-ARH-301 and BB-ARH-302 and cores BP-ARH-201 and -204 were interpreted to assess the elevation of the base of the abutment, which was estimated to be approximately El. 41.5 at the South Abutment and El. 42.5 at the North Abutment. The data indicate a relatively consistent bottom of concrete elevation at each abutment, which is shown on **Figure 4**.

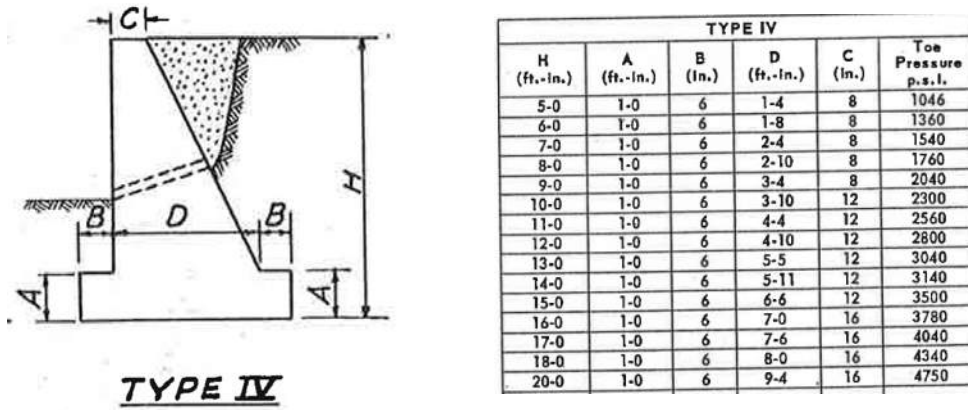
The footing configuration and embedment depth of the abutment toe were evaluated based on the 200-series probes. The probe results were reviewed to evaluate the toe extension beyond the face of the abutment and the embedment depth of the toe, and the cores were used to evaluate the thickness. The estimated toe footing dimensions are summarized in the following table.

SUMMARY OF ABUTMENT TOE DIMENSIONS		
Toe Footing Dimension	South Abutment (BP-ARH-203, -204)	North Abutment (BP-ARH-201, -202)
Minimum Extension beyond Face of Abutment (feet)	1.5	1
Range in Depth of Top of Footing below Existing Grade (feet)	3.8 to 4.2	3.9 to 4.9
Thickness (feet)	3	3.5
Depth of Bottom of Footing below Existing Grade (feet)	7.2	7.4

The interpreted footing widths from **Figure 4** at the analyzed sections are 19 feet at the south abutment, and 16 feet at the north abutment. The interpreted footing widths are measured between the edge of the toe evaluated with the 200-series probes and the back of the heel approximated based on the Type IV footing from



Seelye⁴ for cantilever- and gravity-type retaining walls, which shows equal toe and heel lengths for the typical detail, as shown below.



Typical Type IV Abutment Geometry

GZA recommends that the abutment profiles shown on **Figure 4** be used for Stantec's structural design calculations.

5.4.2 Spread Footing on Soil Design Considerations

Bearing resistance values have been developed for the abutment footings using the theoretical method (Munfakh et al., 2001) in Sand using SPT data. Bearing resistances were evaluated in accordance with Articles 10.6.3.1.1 and 10.6.3.1.2a of AASHTO LRFD. Based on available data described previously, the existing abutments are interpreted to be supported on spread footing foundations bearing on Glacial Till. It should be noted that directly beneath the footing in BB-ARH-301 from approximately 27.6 to 29.5 feet, a Gravelly CLAY sample was encountered, which appears to be Fill. All other borings encountered Glacial Till at that elevation, including boring BB-ARH-302 drilled through the south abutment. Therefore, it is our opinion that the material is likely a localized pocket of re-worked material that was left during the construction process, and it is not representative of the typical bearing soil.

Service and factored bearing resistance values recommended for the existing foundations for design are presented in **Figure 5**. Service limit design is based on a settlement of 1 inch for a footing with the average bearing pressure shown on the plot. However, it is anticipated that the bearing pressure of the abutment is largely incurred by the retaining walls, and the new bridge weight will likely be similar to existing weight. Therefore, settlement has already occurred under the existing unfactored loads, service limit state design may not be relevant to the performance of the foundations supporting the new bridge, and it is not necessary to limit the ability to reuse the abutments based on serviceability criteria for design.

5.5 SEISMIC DESIGN CONSIDERATIONS

Seismic site class was evaluated in general accordance with LRFD Table C3.10.3.1, considering the average SPT N-values in granular soils which encompasses most of the soil encountered in the borings. Considering the

⁴ Seelye, Elwyn. Data Book for Civil Engineers – DESIGN, 3rd Ed. Wiley & Sons. 1960.



presence of loose to medium dense fill behind the abutments and a relatively thick layer of medium dense to very dense glacial till below the abutments, the bridge should be assigned to Site Class D.

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

5.6 LATERAL EARTH PRESSURES AND RESISTANCE

Existing soil fill material retained by the existing abutments is generally loose sand and is interpreted to have an internal friction angle of 29 degrees. The average friction angle for dissimilar materials, δ , between formed concrete and a clean sand/silty sand mixture, is 20 degrees (AASHTO LRFD Table C3.11.5.3-1). The design earth pressure coefficients provided in **Section 6.2** of this report are based on the geometry of the south abutment, which is considered to be a short-heeled wall in accordance with AASHTO LRFD C3.11.5.3, and the fill material properties presented above.

Passive resistance is typically neglected for resistance to sliding in consideration that soil may be removed in the future and that the soils will be subject to freeze-thaw weakening in accordance with BDG Article 5.3.8. However, considering the presence of the former rail line buried beneath the ground, and the fact that the soil has been there for approximately 100 years and SPT-N values indicate a medium dense to dense material, it may be appropriate to consider passive resistance on the abutment toe side for some part of the embedded portion of the wall. BDG Article 5.3.8 indicates that the available (nominal) passive resistance should be taken as 50 percent of the maximum Rankine Passive Earth Pressure coefficient. Recommended passive resistance parameters are presented in **Section 6.3**.

6.0 RECOMMENDATIONS

6.1 SEISMIC DESIGN

The USGS online *Design Maps Tool* was used to develop parameters for bridge design. Based on the site coordinates, the software provided the recommended AASHTO Response Spectra (Site Class D) for a 7 percent probability of exceedance in 75 years. These results are summarized for the site as follows:

SITE CLASS D SEISMIC DESIGN PARAMETERS	
Parameter	Design Value
Fpga	1.6
Fa	1.6
Fv	2.4
As (Period = 0.0 sec)	0.13 g
SDs (Period = 0.2 sec)	0.26 g
SD1 (Period = 1.0 sec)	0.11 g

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



6.2 EXISTING ABUTMENT AND WINGWALL EARTH PRESSURES

- Abutment backfill is based on the soil encountered; any soil replaced during construction should consist of MaineDOT 703.19 Granular Borrow for Underwater Backfill, MaineDOT BDG Type 4 soil. Recommended soil properties for the soil encountered behind the abutment are as follows:
 - Internal Friction Angle of Soil = 29° ;
 - Soil Total Unit Weight = 125 pcf;
 - Coulomb Coefficient of Active Earth Pressure:
 - $K_{ac} = 0.50$ (Active Earth Pressure);
 - $K_{ach} = 0.40$ (Active Earth Pressure – Horizontal Component);
 - $K_{acv} = 0.31$ (Active Earth Pressure – Vertical Component);
- 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.

6.3 RECOMMENDED DESIGN PARAMETERS FOR EXISTING FOUNDATIONS

- Factored strength limit bearing resistance and service limit bearing resistance are provided as a function of effective footing widths on the bearing charts included in **Figure 5**. The structural designer should check the effective footing widths in their design against the bearing resistance plots in **Figure 5** to determine the appropriate resistance values.
- The base resistance against sliding was evaluated in accordance with AASHTO Article 10.6.3.4 using $\phi_s' = \delta = 33$ degrees. Nominal sliding resistance for footings on glacial till is equal to the vertical force multiplied by the sliding resistance coefficient ($\tan \delta$), which is equal to 0.65. The factored sliding resistance coefficient is 0.52 for the strength condition.
- Footings founded on soil should be checked for overturning per 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 $B/3$ from the centerline of the footing, where B is the principal dimension of the footing perpendicular to the axis of rotation.
- Passive resistance may be calculated for the embedded portion of the abutment as 50 percent of the Rankine Passive Earth Pressure coefficient. The Rankine Passive Earth Pressure coefficient is as follows:
 - $K_p = 1.7$ (Reduced Nominal Passive Earth Pressure, 50 percent of maximum value)
 - Toe Soil Total Unit Weight = 125 pcf

7.0 CONSTRUCTION CONSIDERATIONS

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



7.1 EXCAVATION, TEMPORARY LATERAL SUPPORT AND DEWATERING

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

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 and river water should comply with all local, State, and federal regulations.

Considering that the backwall reconstruction and superstructure replacement will require staged construction and a single-lane closure, temporary lateral earth support consisting of driven sheet piles will be needed. At a minimum, temporary lateral support will be required to retain the soil on the inboard side of the excavation. The extent of lateral earth support on other sides of the excavation will depend on the contractor's means and methods. Care should be taken while installing sheetpiles near the abutments to avoid damaging the existing battered concrete abutment backwalls.

7.2 REUSE OF ON-SITE MATERIALS

Based on the test boring results, fill samples tested within the upper 15 feet behind the abutments generally had more than 10 percent passing the No. 200 sieve, indicating the fill will not meet MaineDOT Specification requirements for Granular Borrow for Underwater Backfill and would not be suitable for use as structural backfill. The results of the same tested samples generally had 10 percent to 20 percent passing the No. 200 sieve, indicating the fill could potentially be used as Granular Borrow for embankment construction. 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.

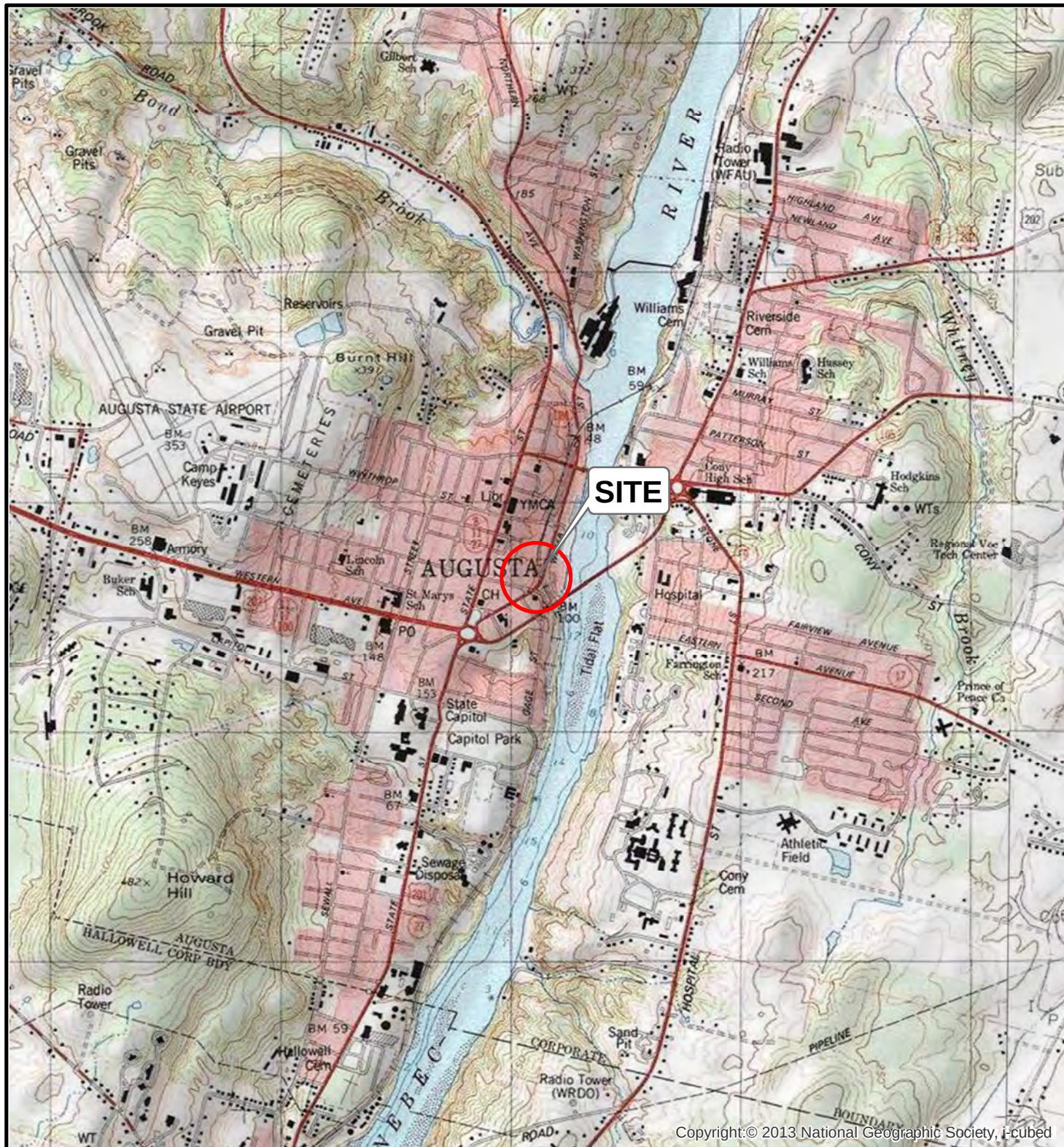


10/18/2022

**RINES HILL BRIDGE NO. 3528 OVER OLD MAINE CENTRAL RAILROAD
GEOTECHNICAL DESIGN REPORT**

09.0025943.01

FIGURES



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

Data Supplied by :



PROJ. MGR.: NVW
DESIGNED BY: NVW
REVIEWED BY: ARB
OPERATOR: ADM
DATE: 11-29-2018

LOCUS PLAN

RINES HILL BRIDGE
AUGUSTA, MAINE

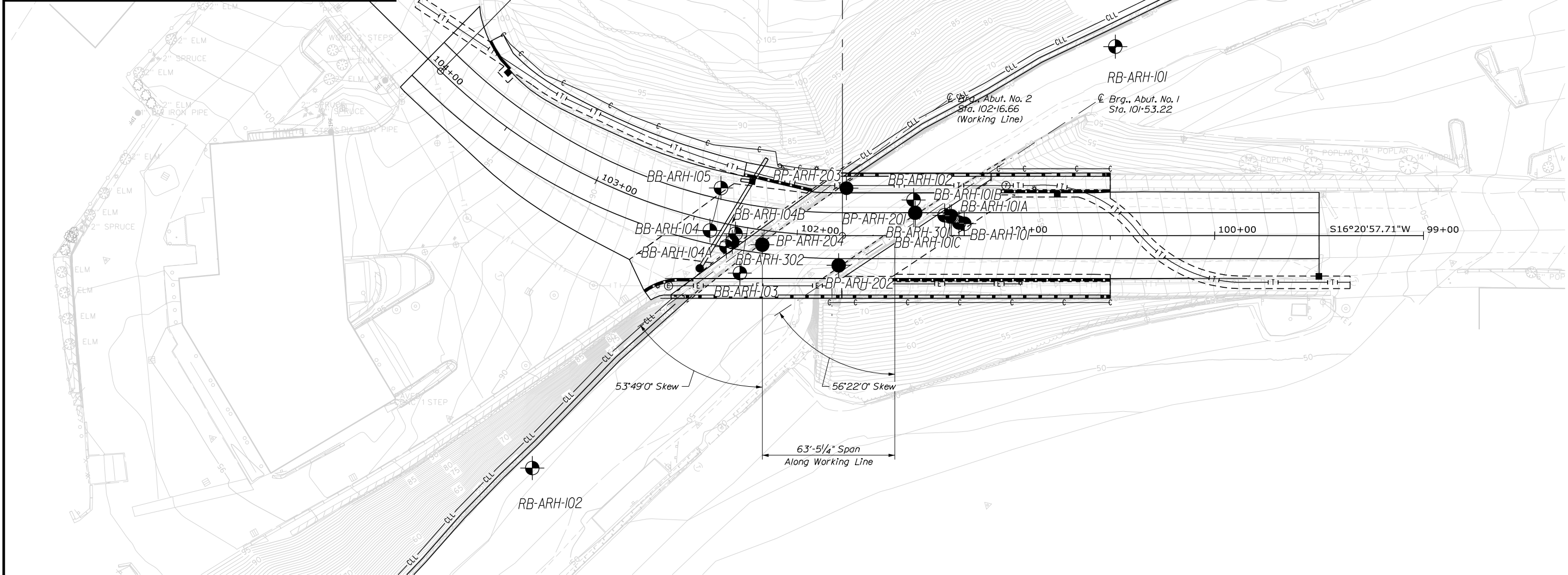
JOB NO.
09.0025943.01
FIGURE NO.
1

PREPARED BY:

 GZA GeoEnvironmental, Inc.
Engineers and Scientists
www.gza.com

PROJ MGR: NVW	REVIEWED BY: ARB	CHECKED BY: CLS	 FIG 2
DESIGNED BY: NVW	DRAWN BY: NVW	SCALE: AS SHOWN	
DATE: 10/18/22	PROJECT NO. 09.0025943.01	REVISION NO.	

SHEET NO. 2 OF 5



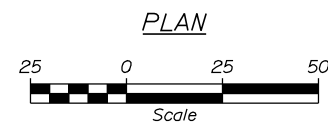
1) Base map developed from electronic files (bridge.dgn, Contours.dgn, Text.dgn, Topo.dgn, and Alignments.dgn) provided by Stantec on September 13, 2022.

2) The as-drilled locations of the preliminary test borings and probes (100 series) were surveyed by MEDOT and provided by Stantec in an electronic file (021872.00 BORINGS 8 AUG 17.csv) on December 13, 2017. The supplementary test borings and probes (200 & 300 series) were measured from existing features.

3) BB-ARH-100 and RB-ARH-100 series borings were performed by New England Boring Contractors and observed by GZA personnel between July 10 and 27, 2017.

4) BP-ARH-200 series probes and BB-ARH-300 series borings were performed by New England Boring Contractors and observed by GZA personnel between April 15 and 17, 2019.

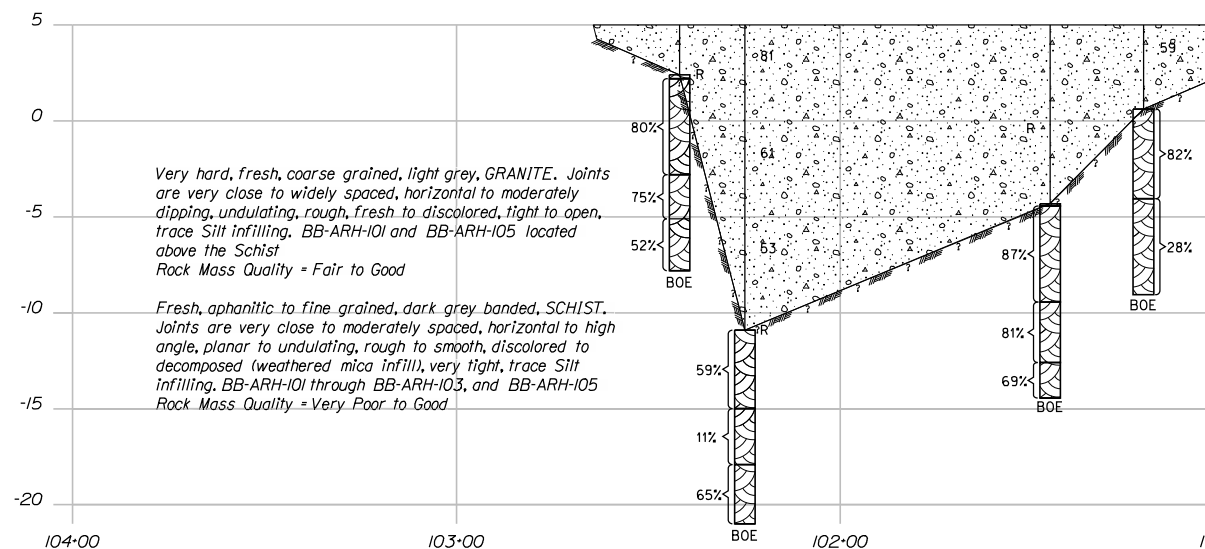
	BB-ARH-302	As-drilled location and designation of 300 series borings/abutment cores.
	BP-ARH-204	As-drilled location and designation of 200 series probes/footing cores
	BB-ARH-105	As-drilled location and designation of 100 series bridge boring or probe (explorations with labels A through C are auger probes, others are cased wash borings)
	RB-ARH-102	As-drilled location and designation of 100 series retaining wall boring



DESIGN-DETAILS	N.WILLIAMS	N.WILLIAMS	OCT 2022
CHECKED-REVIEWED	ABLASDELL	C.SNOW	OCT 2022
DESIGN3-DETAILD3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			
SIGNATURE			P.E. NUMBER
DATE			

KENNEBEC COUNTY
OVER OLD MAINE CENTRAL RAILROAD
AUGUSTA

SHEET NUMBER



6) Abutment Outlines are oriented along the alignment. The profile of each abutment will vary as a function of height of the abutments, and should be considered approximate.

Boring No. _____
Offset, ft shown _____

Roadway Pavement/Concrete Thickness if applicable _____

6 _____

Energy-Corrected SPT N60 Value (blows/foot) _____

WOH _____

R _____

Split Spoon Refusal (>50 blows for 1' penetration)

3' Concrete Core _____

BOE _____

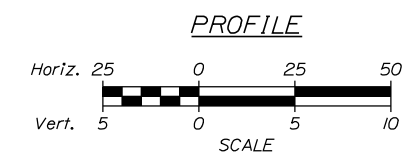
RQD=Rock Quality Designation for Rock Core Sample

BOE Bottom of Exploration

Interpreted Top of Bedrock (See Note 5)

BOE Bottom of Exploration, Refusal

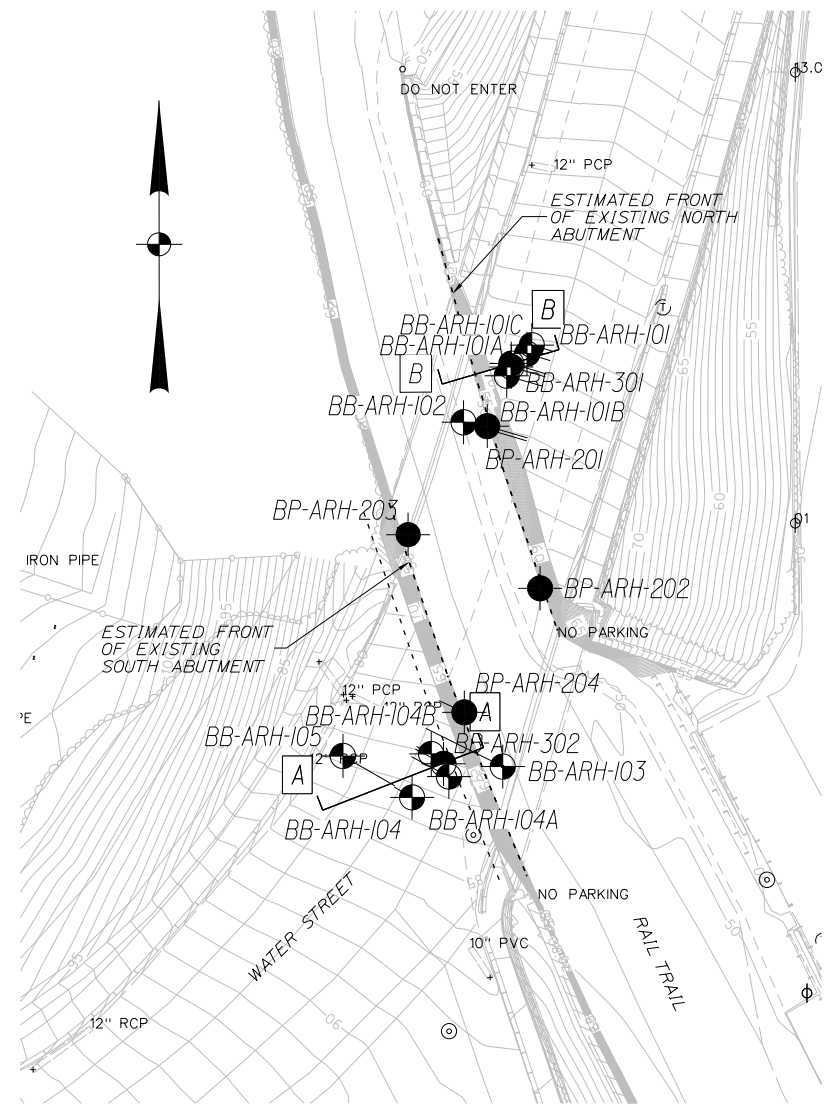
BOE Bottom of Exploration, No Refusal



RINES HILL BRIDGE MAINEDOT WIN 21872.00 AUGUSTA, ME			
INTERPRETIVE SUBSURFACE PROFILE			
PREPARED BY:  GZA GeoEnvironmental, Inc. Engineers and Scientists www.gza.com		PREPARED FOR: MAINEDOT	
PROJ MGR: NVW	REVIEWED BY: ARB	CHECKED BY: CLS	3
DESIGNED BY: NVW	DRAWN BY: NVW	SCALE: AS SHOWN	
DATE: 10/18/22	PROJECT NO. 09.0025943.01	REVISION NO.	
			SHEET NO. 3 OF 5

PREPARED BY:

The logo for GTA (Gulf Technical Association) is located in the bottom right corner of the page. It features the letters "GTA" in a bold, blue, sans-serif font. To the right of the letters is a stylized blue arc that curves upwards and to the right, resembling a swoosh or a partial circle.



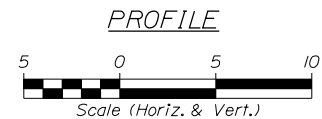
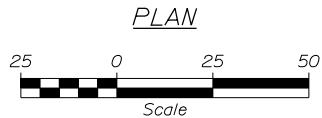
BORING LOCATION PLAN LEGEND

- BB-ARH-105 As-drilled location and designation of 100 series boring or probe (explorations with labels A through C are auger probes, others are cased wash borings)
- BP-ARH-204 As-drilled location and designation of 200 series probes/footing cores
- BB-ARH-302 As-drilled location and designation of 300 series borings/abutment cores

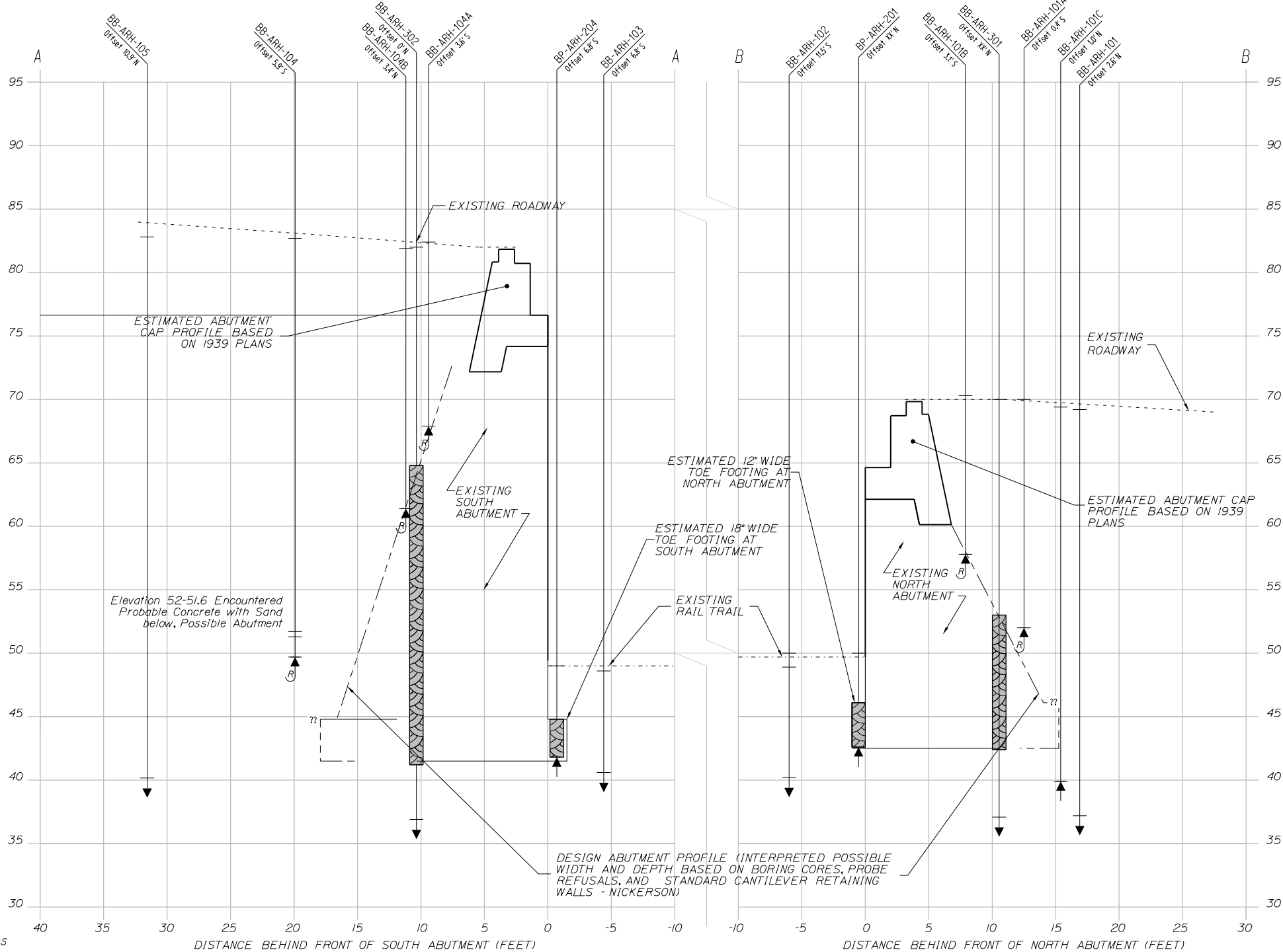
A A Location and designation of interpreted existing abutment profile

PROFILE LEGEND

- Bottom of Exploration, Refusal (Apparent/Possible Concrete)
- Bottom of Exploration, No Refusal
- Indicates exploration extends below depth shown
- Design Abutment Profile
- Concrete Core



ELEVATION (FT. NAVD88)



NOTES

1. Interpreted abutment profiles are oriented normal to the face of abutments, resulting in a skew from the baseline.
2. The south abutment profile (Section A-A) is on the higher (southeast) half of the south abutment, and the north abutment profile (Section B-B) is on the lower (northwest) half of the north abutment. The profile of each abutment will vary as a function of height of the abutments. Abutment height varies by roughly 9 feet across the width of the bridge at each abutment.
3. The base map was developed from electronic files (3DContours.dgn, 3DText.dgn, 3DTopo_02aug16.dgn, and Alignments.dgn) provided by Stantec on September 13, 2022. Existing ground profiles were interpreted from topographic data shown on these plans.
4. Existing abutment cap profiles were developed through a combination of the original 1939 plan details and surveyed bridge seat data near the profile locations.
5. BB-ARH-100 series probes were advanced using solid stem augers to significantly increased resistance on possible concrete. Refusal was interpreted to consist of possible abutment concrete.
6. BB-ARH-200 series probes were advanced at possible abutment toe locations using solid stem augers to significantly increased resistance on possible concrete and cores were conducted through the apparent toe.
7. BB-ARH-300 series borings were advanced through the back of the concrete abutments and the underlying soil.
8. Boring and probe offsets reference the normal distance from Sections A-A and B-B and are not related to the project baseline.
9. The limits are intended to convey the interpreted geometry of the abutments at the locations explored and should be used for retaining wall evaluations. The abutment profile limits are approximate and idealized.

PREPARED BY:



STATE OF MAINE	
DEPARTMENT OF TRANSPORTATION	
BRIDGE NO. 3528	
WIN 21872.00	
BRIDGE PLANS	

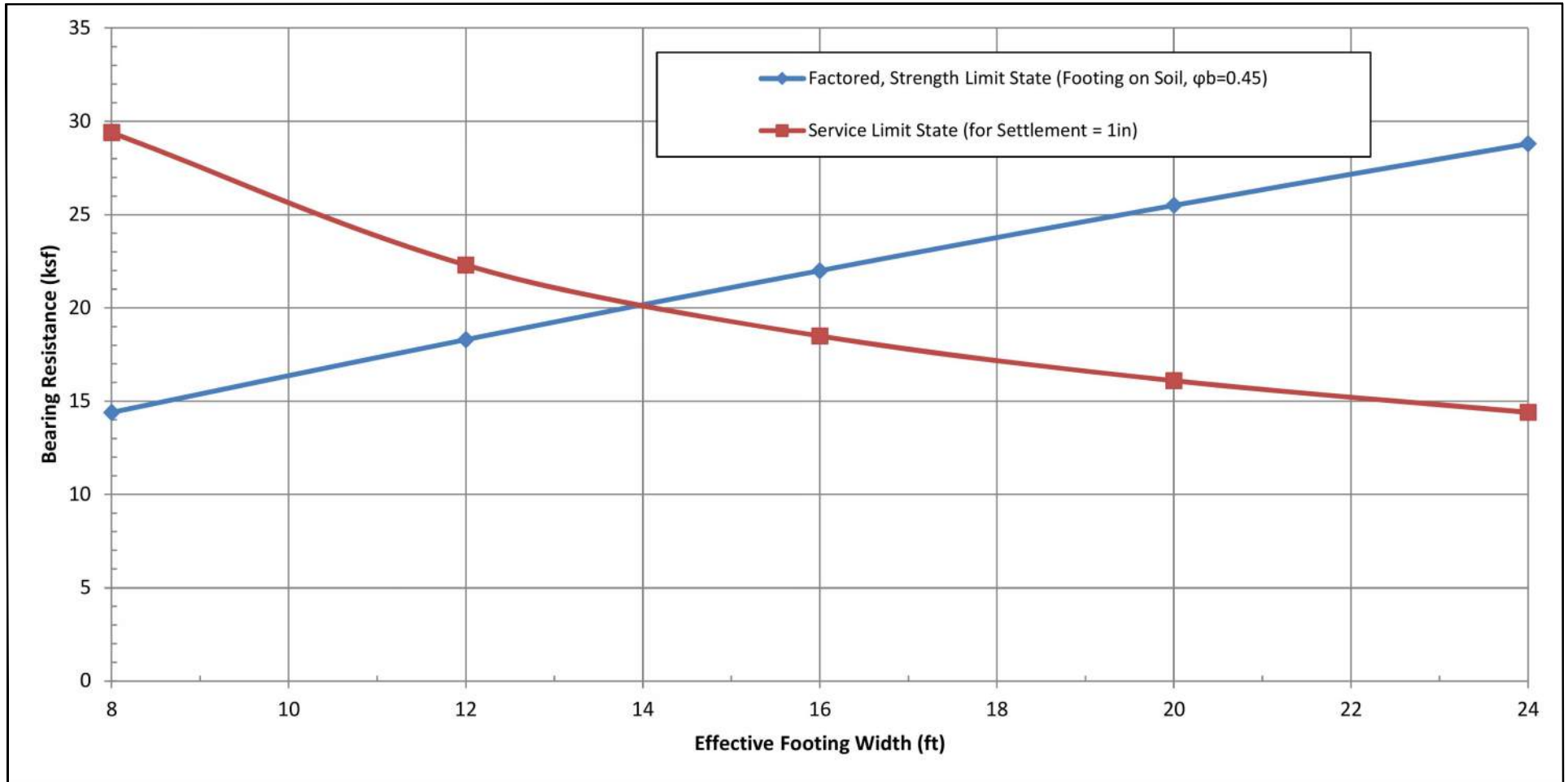
PROJ. MANAGER	DATE	BY	DATE	SIGNATURE	P.E. NUMBER	DATE
DESIGN-DETAILED	OCT 2022	N. WILLIAMS	OCT 2022			
CHECKED-REVIEWED		A. B. ASSELL				
DESIGN-DETAILED						
REVISIONS 1						
REVISIONS 2						
REVISIONS 3						
REVISIONS 4						
FIELD CHANGES						

RINES HILL BRIDGE	
OVER OLD MAINE CENTRAL RAILROAD	
KENNEBEC COUNTY	
AUGUSTA	
ABUTMENT PROFILE	

FIGURE

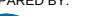
4

OF 5



Notes:

1. Service limit bearing resistance is based on a footing with the stated dimensions and the average applied bearing stress resulting in 1 inch of settlement.
2. The service limit settlement of 1 inch is based on full application of the load. Where the load is already in place, service limit state does not control the design.
3. Bearing resistance values are based on Glacial Till encountered at approximately elevation 41 and deeper, in borings BB-ARH-102 and BB-ARH-103.

				RINES HILL BRIDGE #3528 AUGUSTA, MAINE				PREPARED BY:				PREPARED FOR:							
								 GZA GeoEnvironmental, Inc. Engineers and Scientists www.gza.com				STANTEC 482 PAYNE ROAD SCARBOROUGH COURT SCARBOROUGH, MAINE 04074							
1	REVISED BASED ON UPDATED FOOTING WIDTHS			NW	5/14/19			BEARING RESISTANCE CHART GLACIAL TILL				PROJ MGR: NW		REVIEWED BY: ARB		CHECKED BY: CLS		FIGURE	
NO.	ISSUE/DESCRIPTION			BY	DATE							DESIGNED BY: NW		DRAWN BY: NW		SCALE: AS SHOWN		5 OF 5	
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							OCT 2022					09.0025943.01		1					



10/18/2022

**RINES HILL BRIDGE NO. 3528 OVER OLD MAINE CENTRAL RAILROAD
GEOTECHNICAL DESIGN REPORT**

09.0025943.01

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.



10/18/2022

**RINES HILL BRIDGE NO. 3528 OVER OLD MAINE CENTRAL RAILROAD
GEOTECHNICAL DESIGN REPORT**

09.0025943.01

APPENDIX B – GZA BORING LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM				MODIFIED BURMISTER SYSTEM			
MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES			
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.			
		(little or no fines)	GP	Poorly-graded gravels, gravel sand mixtures, little or no fines.			
		GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.			
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	CLEAN SANDS	SW	Well-graded sands, Gravelly sands, little or no fines			
		(little or no fines)	SP	Poorly-graded sands, Gravelly sand, little or no fines.			
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures			
FINE-GRAINED SOILS (more than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS (liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, Silty or Clayey fine sands, or Clayey silts with slight plasticity.			
			CL	Inorganic clays of low to medium plasticity, Gravelly clays, Sandy clays, Silty clays, lean clays.			
			OL	Organic silts and organic Silty clays of low plasticity.			
	SILTS AND CLAYS (liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine Sandy or Silty soils, elastic silts.			
			CH	Inorganic clays of high plasticity, fat clays.			
			OH	Organic clays of medium to high plasticity, organic silts.			
	HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.			
Desired Soil Observations (in this order, if applicable): Color (Munsell color chart) Moisture (dry, damp, moist, wet) Density/Consistency (from above right hand side) Texture (fine, medium, coarse, etc.) Name (Sand, Silty Sand, Clay, etc., including portions - trace, little, etc.) Gradation (well-graded, poorly-graded, uniform, etc.) Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic) Structure (layering, fractures, cracks, etc.) Bonding (well, moderately, loosely, etc.,) Cementation (weak, moderate, or strong) Geologic Origin (till, marine clay, alluvium, etc.) Groundwater level				Desired Rock Observations (in this order, if applicable): Color (Munsell color chart) Texture (aphanitic, fine-grained, etc.) Rock Type (granite, schist, sandstone, etc.) Hardness (very hard, hard, mod. hard, etc.) Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.) Geologic discontinuities/jointing: -dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.) -spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet) -tightness (tight, open, or healed) -infilling (grain size, color, etc.) Formation (Waterville, Ellsworth, Cape Elizabeth, etc.) RQD and correlation to rock quality (very poor, poor, etc.) ref: ASTM D6032 and FHWA NHI-16-072 GEC 5 - Geotechnical Site Characterization, Table 4-12 Recovery (inch/inch and percentage) Rock Core Rate (X.X ft - Y.Y ft (min:sec))			
Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information				Sample Container Labeling Requirements: WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth			

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS						Project: Rines Hill Bridge #3528, Water Street 09.0025943.00 Location: Augusta, Maine			Boring No.: BB-ARH-101 WIN: 21872.00																																																																																				
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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Rines Hill Bridge #3528, Water Street 09.0025943.00 Location: Augusta, Maine				Boring No.: BB-ARH-101 WIN: 21872.00																																																																																																																																																																																	
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Joints are moderately spaced, moderately dipping to high angle, undulating, rough to smooth, fresh to discolored, partially open to open, trace silt infilling. -VASSALBORO FORMATION- Rock Mass Quality = Good Recovery = 92% Rock Core Times (min:ft): 54.1-55.1' (2:51), 55.1-56.1' (2:48), 56.1-57.1' (2:54), 57.1-58.1' (2:51), 58.1-59.1' (2:37) R3: Very hard, fresh, aphanitic to fine grained, dark grey to grey, SCHIST. Joints are very close to moderately spaced, horizontal to high angle, undulating to planar, rough to slickensided, fresh to discolored, partially open to open, trace silt infilling. Rock Mass Quality = Fair Recovery = 100% Rock Core Times (min:ft): 59.1-60.1' (3:23), 60.1-61.1' (2:50), 61.1-62.1' (3:03), 62.1-62.3' (2:12) R4: Very hard, fresh, aphanitic to fine grained, dark grey to grey, SCHIST. Joints are close to moderately spaced, low angle, undulating, rough, fresh to discolored, partially open, trace silt infilling. 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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 3 of 3 Boring No.: BB-ARH-102																																																																																																																																																																																																																																																			

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS					Project: Rines Hill Bridge #3528, Water Street 09.0025943.00 Location: Augusta, Maine			Boring No.: BB-ARH-104 WIN: 21872.00							
Driller: New England Boring Contractors		Elevation (ft.) 82.7'		Auger ID/OD: 2.5"											
Operator: Mike Porter		Datum: NAVD 88		Sampler: Split Spoon											
Logged By: B. Woodman		Rig Type: Mobile B-53 Truck		Hammer Wt./Fall: 140/30											
Date Start/Finish: 7/12/17-7/12/17		Drilling Method: Cased		Core Barrel: NQ2											
Boring Location: 102+41.2, 4.8' RT		Casing ID/OD: 3"/3.5"		Water Level*: NE											
Hammer Efficiency Factor: 0.869		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.) 25 30 35 40 45 50	Sample Information								Graphic Log	Visual Description and Remarks Brown, moist, loose, fine to coarse SAND, little Gravel, trace Silt. (Taken with 3" OD spoon.) -FILL- (SP-SM) Casing refusal at 27.8', 28.6', 29.5', and 29.9'. Advance roller cone past casing refusals to 31.0'. Brown, moist, loose, fine to coarse SAND, little Silt, trace wood, trace organics. -FILL- (SM) Casing broken at 30.0', boring terminated. Bottom of Exploration at 33.0 feet below ground surface.	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.)							
	6D	24/4	25.0 - 27.0	10-5-4-4	9	13	3								
							9								
							RC								
	7D	24/7	31.0 - 33.0	4-2-2-3	5	7									
	Remarks:														
	1. Automatic Hammer: NEBC #3, rated hammer efficiency factor = 0.869. 2. Sample 4D consisted of material in spoon tip; balance of recovery was wash material. 3. Bottom 5.0' of casing was not recovered. 4. 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. 5. The as-drilled location and elevation was surveyed by MaineDOT (N539720.3, E1141203.9)														
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-ARH-104					

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Boring No.: BB-ARH-105

WIN: 21872.00

Driller:	New England Boring Contractors	Elevation (ft.)	82.8'	Auger ID/OD:	2.5"
Operator:	Mike Porter	Datum:	NAVD 88	Sampler:	Split Spoon
Logged By:	B. Woodman	Rig Type:	Mobile B-59 Truck	Hammer Wt./Fall:	140/30
Date Start/Finish:	7/24/17-7/27/17	Drilling Method:	Cased	Core Barrel:	NQ2
Boring Location:	102+40.7, 16.2' LT	Casing ID/OD:	3"/3.5"	Water Level*:	NE
Hammer Efficiency Factor: 0.869		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_p -uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N_{60} = SPT N-corrected 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									Elevation (ft.)	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	N ₆₀	Casing Blows					
0								82.1		-BITUMINOUS PAVEMENT-	G A-1-a, GP-GM WC=3.1	
	1D	24/11	1.5 - 3.5	13-5-9-6	14	20		81.6		-POSSIBLE CONCRETE/COBBLE STONES-		
								80.7		Tan, dry, medium dense, sandy GRAVEL, little Silt. -FILL- (GP-GM)		
										Apparent round gravel at 2.1'.		
								78.7		Possible concrete at 4.1'.		
5	2D	24/9	5.0 - 7.0	3-5-6-7	11	16		77.8		Tan, dry, medium dense, fine to coarse SAND, trace Gravel, trace Silt. -FILL- (SP-SM)	G A-1-b, SP-SM WC=11.7	
10	3D	24/8	10.0 - 12.0	6-12-12-13	24	35	25			Brown, moist, dense, fine to coarse SAND, little Silt, trace Gravel. -GLACIAL TILL- (SM)		
15	4D	24/10	15.0 - 17.0	7-11-12-11	23	33	30			Brown, moist, dense, fine to coarse SAND, little Silt, trace Gravel. -GLACIAL TILL- (SM)		
20	5D	24/8	20.0 - 22.0	10-6-12-13	18	26	39			Brown, moist, medium dense, fine to coarse SAND, little Silt, trace Gravel. -GLACIAL TILL- (SP-SM)		
25							17					

1. Automatic Hammer: NEBC# 3, rated hammer efficiency factor = 0.869.
2. Fine-Grained soil descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterberg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.
3. The as-drilled location and elevation was surveyed by MaineDOT (N539731.1, E1141185.9)

Page 1 of 4

Boring No.: BB-ARH-105

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS					Project: Rines Hill Bridge #3528, Water Street 09.0025943.00 Location: Augusta, Maine			Boring No.: BB-ARH-105 WIN: 21872.00			
Driller: New England Boring Contractors		Elevation (ft.): 82.8'		Auger ID/OD: 2.5"							
Operator: Mike Porter		Datum: NAVD 88		Sampler: Split Spoon							
Logged By: B. Woodman		Rig Type: Mobile B-59 Truck		Hammer Wt./Fall: 140/30							
Date Start/Finish: 7/24/17-7/27/17		Drilling Method: Cased		Core Barrel: NQ2							
Boring Location: 102+40.7, 16.2' LT		Casing ID/OD: 3"/3.5"		Water Level*: NE							
Hammer Efficiency Factor: 0.869		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	6D	24/11	25.0 - 27.0	13-18-19-20	37	54	57		Brown, moist, very dense, fine to coarse SAND, some Gravel, trace Silt. -GLACIAL TILL- (SP-SM)	G A-1-a, SM WC=8.4	
							91				
							120				
							114				
							144				
30	7D	24/4	30.0 - 32.0	19-22-22-29	44	64	68		Brown, moist, very dense, fine to coarse SAND, some Gravel, little Silt. -GLACIAL TILL- (SM)		
	8D	24/11	32.0 - 34.0	19-25-23-26	188	272	68				
	9D	24/11	34.0 - 36.0	14-18-21-47	39	56	62				
35	10D	24/2	36.0 - 38.0	24-31-19-19	50	72	115		Light brown, moist, very dense, silty GRAVEL, some Sand. -GLACIAL TILL- (GP-GM)		
	11D	24/4	38.0 - 40.0	7-3-9-7	12	17	23				
	12D	24/3	40.0 - 42.0	N/A-N/A-4-5	N/A		50				
40							61		Brown, moist, sandy GRAVEL, little Silt. From 40.0'-41.0', rods mistakenly dropped into soil. -GLACIAL TILL- (GP-GM)		
							78				
							82				
							70				
	13D	24/2	45.0 - 47.0	8-6-1-1	7	10	RC				
45								Brown, moist, loose, medium coarse SAND, little Gravel, trace Silt. Rock in tip of spoon. -GLACIAL TILL- (SP-SM)			
							168				
50							77				
Remarks: 1. Automatic Hammer: NEBC# 3, rated hammer efficiency factor = 0.869. 2. Fine-Grained soil descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterberg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes. 3. The as-drilled location and elevation was surveyed by MaineDOT (N539731.1, E1141185.9)											
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 4 Boring No.: BB-ARH-105		

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Rines Hill Bridge #3528, Water Street 09.0025943.00 Location: Augusta, Maine				Boring No.: BB-ARH-105 WIN: 21872.00																																																																																																																																																																																																																																							
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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 3 of 4 Boring No.: BB-ARH-105																																																																																																																																																																																																																																				

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Boring No.:	BB-ARH-105
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WIN:	21872.00
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Hammer Efficiency Factor: 0.869	Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐
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Definitions:	R = Rock Core Sample	S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf)	T _v = Pocket Torvane Shear Strength (psf)
D = Split Spoon Sample	SSA = Solid Stem Auger	S _{u(lab)} = Lab Vane Undrained Shear Strength (psf)	WC = Water Content, percent
MD = Unsuccessful Split Spoon Sample Attempt	HSA = Hollow Stem Auger	q _p = Unconfined Compressive Strength (ksf)	LL = Liquid Limit
U = Thin Wall Tube Sample	RC = Roller Cone	N _{uncorrected} = Raw Field SPT N-value	PL = Plastic Limit
MU = Unsuccessful Thin Wall Tube Sample Attempt	WOH = Weight of 140 lb. Hammer	Hammer Efficiency Factor = Rig Specific Annual Calibration Value	PI = Plasticity Index
V = Field Vane Shear Test, PP = Pocket Penetrometer	WOR/C = Weight of Rods or Casing	N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency	G = Grain Size Analysis
MV = Unsuccessful Field Vane Shear Test Attempt	WO1P = Weight of One Person	N ₆₀ = (Hammer Efficiency Factor/60%)N _{uncorrected}	C = Consolidation Test

Remarks:
1. Automatic Hammer: NEBC# 3, rated hammer efficiency factor = 0.869. 2. Fine-Grained soil descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterberg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes. 3. The as-drilled location and elevation was surveyed by MaineDOT (N539731.1, E1141185.9)

1. Automatic Hammer: NEBC# 3, rated hammer efficiency factor = 0.869.
2. Fine-Grained soil descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterberg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.
3. The as-drilled location and elevation was surveyed by MaineDOT (N539731.1, E1141185.9)

* 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.

[illegible]

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Boring No.:	<u>RB-ARH-102</u>
WIN:	<u>21872.00</u>

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

[illegible]

Remarks:

1. Automatic Hammer: NEBC# 3, rated hammer efficiency factor = 0.869.
2. Geotextile fabric at 1.0'.
3. No recovery from 15.0'-17.0' - spoon went approximately 3" after 60 blows and no recovery - stem auger grinding - rubber returned on auger at approximately 16.0'.
4. 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.
5. The as-drilled location and elevation was surveyed by MaineDOT (N539606.7, E1141289.1)

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Rines Hill Bridge #3528, Water Street 09.0025943.00 Location: Augusta, Maine				Boring No.: RB-ARH-102 WIN: 21872.00																																																																																																																																																																																																																																																																																																																																																																																																										
Driller: New England Boring Contractors				Elevation (ft.): 48.9'				Auger ID/OD:																																																																																																																																																																																																																																																																																																																																																																																																										
Operator: Mike Porter				Datum: NAVD 88				Sampler: Split Spoon																																																																																																																																																																																																																																																																																																																																																																																																										
Logged By: B. Woodman				Rig Type: Mobile B-59 Truck				Hammer Wt./Fall: 140/30																																																																																																																																																																																																																																																																																																																																																																																																										
Date Start/Finish: 7/27/17-7/27/17				Drilling Method: Hollow Stem Auger				Core Barrel: --																																																																																																																																																																																																																																																																																																																																																																																																										
Boring Location: 102+85.0, 90.5' LT				Casing ID/OD: 4.5" I.D.				Water Level*: Dry																																																																																																																																																																																																																																																																																																																																																																																																										
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25	6D	24/4	25.0 - 27.0	26-4-4-5	8	12			Dark brown, moist, medium dense, sandy GRAVEL, trace Silt. -GLACIAL TILL- (GP-GM)	G A-1-a, GP-GM WC=4.1																																																																																																																																																																																																																																																																																																																																																																																																								
30	7D	24/1	30.0 - 32.0	5-7-8-5	15	22																																																																																																																																																																																																																																																																																																																																																																																																												
								Granite fragments.																																																																																																																																																																																																																																																																																																																																																																																																										
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50																																																																																																																																																																																																																																																																																																																																																																																																																		
Remarks: 1. Automatic Hammer: NEBC# 3, rated hammer efficiency factor = 0.869. 2. Geotextile fabric at 1.0'. 3. No recovery from 15.0-17.0' - spoon went approximately 3" after 60 blows and no recovery - stem auger grinding - rubber returned on auger at approximately 16.0'. 4. 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. 5. The as-drilled location and elevation was surveyed by MaineDOT (N539606.7, E1141289.1)																																																																																																																																																																																																																																																																																																																																																																																																																		
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.: RB-ARH-102																																																																																																																																																																																																																																																																																																																																																																																																								

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Boring No.:	BP-ARH-201
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WIN: 21872.00

Hammer Efficiency Factor: N/A		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	

[illegible]

1. Set up probe 4" off the face of Abutment 2.
2. Advanced SSA to practical refusal at 3.9'. Fill material was observed in auger flights.
3. 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.
4. The as-drilled location was measured from existing features and the elevation was estimated based on existing ground surface elevation contours (N539817, E1141223).

Page 1 of 1

Boring No.: BP-ARH-201

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Boring No.:	BP-ARH-204
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WIN:	<u>21872.00</u>
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Hammer Efficiency Factor: --		Hammer Type: Automatic ☐ Hydraulic ☐ Rope & Cathead ☐	
Definitions:			
D = Split Spoon Sample	R = Rock Core Sample	S _y = Peak/Remolded Field Vane Undrained Shear Strength (psf)	T _v = Pocket Torvane Shear Strength (psf)
MD = Unsuccessful Split Spoon Sample Attempt	SSA = Solid Stem Auger	S _{u(lab)} = Lab Vane Undrained Shear Strength (psf)	WC = Water Content, percent
U = Thin Wall Tube Sample	HSA = Hollow Stem Auger	q _p = Unconfined Compressive Strength (ksf)	LL = Liquid Limit
MU = Unsuccessful Thin Wall Tube Sample Attempt	RC = Roller Cone	N _{uncorrected} = Raw Field SPT N-value	PL = Plastic Limit
V = Field Vane Shear Test	WOH = Weight of 140lb. Hammer	Hammer Efficiency Factor = Rig Specific Annual Calibration Value	PI = Plasticity Index
PP = Pocket Penetrometer	WOR/C = Weight of Rods or Casing	N ₆₀ = SPT N-corrected Corrected for Hammer Efficiency	G = Grain Size Analysis
MV = Unsuccessful Field Vane Shear Test Attempt	WO1P = Weight of One Person	N ₆₀ = (Hammer Efficiency Factor/60%)*N _{uncorrected}	C = Consolidation Test

[illegible]

1. Set up probe 8" off the face of Abutment 1.
2. Advanced SSA to practical refusal at 4.2'. Fill material was observed in auger flights.
3. 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.
4. The as-drilled location was measured from existing features and the elevation was estimated based on existing ground surface elevation contours (N539742, E1141218).

Page 1 of 1

Boring No.: BP-ARH-204

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Rines Hill Bridge #3528, Water Street 09.0025943.00 Location: Augusta, Maine				Boring No.: BB-ARH-301 WIN: 21872.00																																																																																																																																																																																																																																																																																																																																																										
Driller: New England Boring Contractors		Elevation (ft.): 70' (est)		Auger ID/OD: 2.5"																																																																																																																																																																																																																																																																																																																																																														
Operator: Brad Enos		Datum: NAVD 88		Sampler: Split Spoon																																																																																																																																																																																																																																																																																																																																																														
Logged By: B. Woodman		Rig Type: Mobile B-53 Truck		Hammer Wt./Fall: 140/30																																																																																																																																																																																																																																																																																																																																																														
Date Start/Finish: 4/17/19 - 4/17/19		Drilling Method: SSA, Drive & Wash		Core Barrel: HQ																																																																																																																																																																																																																																																																																																																																																														
Boring Location: 101+27, 10' LT		Casing ID/OD: 3/3.5", 4/4.5"		Water Level*: NE																																																																																																																																																																																																																																																																																																																																																														
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																																																																																																																																																																																																																																																																																																																																																																
<table><tr><th rowspan="2">Depth (ft.)</th><th colspan="8">Sample Information</th><th rowspan="2">Graphic Log</th><th rowspan="2">Visual Description and Remarks</th><th rowspan="2">Laboratory Testing Results/ AASHTO and Unified Class.</th></tr><tr><th>Sample No.</th><th>Pen./Rec. (in.)</th><th>Sample Depth (ft.)</th><th>Blows (6 in.) Shear Strength (psf) or RQD (%)</th><th>N-uncorrected</th><th>N₆₀</th><th>Casing Blows</th><th>Elevation (ft.)</th></tr><tr><td>25</td><td></td><td></td><td></td><td></td><td></td><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><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td>R3</td><td>7/7</td><td>27.0 - 27.6</td><td></td><td></td><td></td><td></td><td></td><td>42.4</td><td></td><td>R3: CONCRETE. Recovery = 100% Coring resistance lost at 27.6', apparent bottom of concrete.</td><td></td></tr><tr><td></td><td>9D</td><td>24/14</td><td>28.0 - 30.0</td><td>WOH-3-7-25</td><td>10</td><td>16</td><td></td><td></td><td></td><td></td><td>Olive/brown, moist, very stiff, Gravelly CLAY, little fine to coarse Sand. -FILL- (CL)</td><td></td></tr><tr><td>30</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>40.5</td><td></td><td></td><td></td></tr><tr><td></td><td>10D</td><td>24/13</td><td>30.0 - 32.0</td><td>27-25-29-29</td><td>54</td><td>84</td><td></td><td></td><td></td><td></td><td>Brown, dry, very dense, fine to coarse SAND, little Gravel, little Silt. Oxidation throughout. -GLACIAL TILL- (SM)</td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td>11D</td><td>24/16</td><td>32.0 - 34.0</td><td>25-25-24-31</td><td>49</td><td>77</td><td></td><td></td><td></td><td></td><td>Brown, dry, very dense, fine to coarse SAND, little Silt, little Gravel. -GLACIAL TILL- (SM)</td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td>12D</td><td>24/7</td><td>34.0 - 36.0</td><td>31-73-71-73</td><td>144</td><td>225</td><td></td><td></td><td></td><td></td><td>Brown, dry, very dense, fine to coarse SAND, little Silt, little Gravel. -GLACIAL TILL- (SM)</td><td></td></tr><tr><td>35</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td>13D</td><td>24/17</td><td>36.0 - 38.0</td><td>35-30-42-45</td><td>72</td><td>112</td><td></td><td></td><td></td><td></td><td>Brown, dry, very dense, fine to coarse SAND, little Silt, little Gravel. -GLACIAL TILL- (SM)</td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><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><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><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>40</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td>14D</td><td>24/15</td><td>41.0 - 43.0</td><td>17-18-18-22</td><td>36</td><td>56</td><td></td><td></td><td></td><td></td><td>Brown, dry, very dense, fine to coarse SAND, little Silt, little Gravel. -GLACIAL TILL- (SM)</td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><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><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><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>45</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td>15D</td><td>24/14</td><td>46.0 - 48.0</td><td>16-18-26-53</td><td>44</td><td>69</td><td></td><td></td><td></td><td></td><td>Brown, dry, very dense, fine to coarse SAND, little Silt, little Gravel. -GLACIAL TILL- (SM)</td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><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><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>50</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>22.0</td><td></td><td>Bottom of Exploration at 48.0 feet below ground surface.</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.)	25																											R3	7/7	27.0 - 27.6						42.4		R3: CONCRETE. Recovery = 100% Coring resistance lost at 27.6', apparent bottom of concrete.			9D	24/14	28.0 - 30.0	WOH-3-7-25	10	16					Olive/brown, moist, very stiff, Gravelly CLAY, little fine to coarse Sand. -FILL- (CL)		30									40.5					10D	24/13	30.0 - 32.0	27-25-29-29	54	84					Brown, dry, very dense, fine to coarse SAND, little Gravel, little Silt. Oxidation throughout. -GLACIAL TILL- (SM)																11D	24/16	32.0 - 34.0	25-25-24-31	49	77					Brown, dry, very dense, fine to coarse SAND, little Silt, little Gravel. -GLACIAL TILL- (SM)																12D	24/7	34.0 - 36.0	31-73-71-73	144	225					Brown, dry, very dense, fine to coarse SAND, little Silt, little Gravel. -GLACIAL TILL- (SM)		35														13D	24/17	36.0 - 38.0	35-30-42-45	72	112					Brown, dry, very dense, fine to coarse SAND, little Silt, little Gravel. -GLACIAL TILL- (SM)																																									40														14D	24/15	41.0 - 43.0	17-18-18-22	36	56					Brown, dry, very dense, fine to coarse SAND, little Silt, little Gravel. -GLACIAL TILL- (SM)																																									45														15D	24/14	46.0 - 48.0	16-18-26-53	44	69					Brown, dry, very dense, fine to coarse SAND, little Silt, little Gravel. -GLACIAL TILL- (SM)																												50									22.0		Bottom of Exploration at 48.0 feet below ground surface.	
Depth (ft.)	Sample Information								Graphic Log		Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.																																																																																																																																																																																																																																																																																																																																																						
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Remarks: 1. Automatic Hammer: NEBC Drill Rig No. B-24, rated hammer efficiency factor = 0.937 2. Advanced augers through approximately 8" of asphalt, underlain by 13" of concrete. 3. Telescoped 3" casing through abutment after core run R3. Washed ahead prior to advancement of 3" casing 28'-46'. 4. 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. 5. The as-drilled location was measured from existing features and the elevation was estimated based on existing ground surface elevation contours (N539832, E1141230).																																																																																																																																																																																																																																																																																																																																																																		
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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Rines Hill Bridge #3528, Water Street 09.0025943.00 Location: Augusta, Maine				Boring No.: BB-ARH-302 WIN: 21872.00																																																																																																																																																																																																																																																																																																																																											
Driller: New England Boring Contractors		Elevation (ft.): 82' (est)		Auger ID/OD: 2.5"																																																																																																																																																																																																																																																																																																																																															
Operator: Brad Enos		Datum: NAVD 88		Sampler: Split Spoon																																																																																																																																																																																																																																																																																																																																															
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Boring Location: 102+30, 8' RT		Casing ID/OD: 3/3.5", 4/4.5"		Water Level*: NE																																																																																																																																																																																																																																																																																																																																															
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Remarks: 1. Automatic Hammer: NEBC Drill Rig No. B-24, rated hammer efficiency factor = 0.937 2. Advanced augers through approximately 9" of asphalt, underlain by 10" of concrete. 3. 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. 4. The as-drilled location was measured from existing features and the elevation was estimated based on existing ground surface elevation contours (N539729, E1141212).																																																																																																																																																																																																																																																																																																																																																			
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 3 Boring No.: BB-ARH-302																																																																																																																																																																																																																																																																																																																																									

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Rines Hill Bridge #3528, Water Street 09.0025943.00 Location: Augusta, Maine				Boring No.: BB-ARH-302 WIN: 21872.00																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
Driller: New England Boring Contractors		Elevation (ft.): 82' (est)		Auger ID/OD: 2.5"																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
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Logged By: B. Woodman		Rig Type: Mobile B-53 Truck		Hammer Wt./Fall: 140/30																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
Date Start/Finish: 4/18/19 - 4/18/19		Drilling Method: SSA, Drive & Wash		Core Barrel: HQ																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
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58.0</td><td>12-10-12-15</td><td>22</td><td>34</td><td>101</td></tr><tr><td></td><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><td></td></tr><tr><td></td><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><td></td></tr><tr><td></td><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><td></td></tr><tr><td>60</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td rowspan="12">Bottom of Exploration at 58.0 feet below ground surface.</td><td 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Remarks: 1. Automatic Hammer: NEBC Drill Rig No. B-24, rated hammer efficiency factor = 0.937 2. Advanced augers through approximately 9" of asphalt, underlain by 10" of concrete. 3. 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. 4. The as-drilled location was measured from existing features and the elevation was estimated based on existing ground surface elevation contours (N539729, E1141212).																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
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 3 of 3 Boring No.: BB-ARH-302																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														



10/18/2022

**RINES HILL BRIDGE NO. 3528 OVER OLD MAINE CENTRAL RAILROAD
GEOTECHNICAL DESIGN REPORT**

09.0025943.01

APPENDIX C – NGS GEOPHYSICAL REPORT

Northeast Geophysical Services

4 Union Street, Suite 3, Bangor, ME 04401

**GROUND PENETRATING RADAR SURVEYS AT
THE RINES HILL BRIDGE SITE
AUGUSTA, MAINE**

**For:
GZA GeoEnvironmental, Inc.**

June, 2017

GROUND PENETRATING RADAR SURVEYS AT THE RINES HILL BRIDGE SITE, AUGUSTA, MAINE

INTRODUCTION

At the request of GZA GeoEnvironmental, Inc. (GZA) ground penetrating radar (GPR) surveys were conducted at the Rines Hill Bridge site in Augusta, Maine. The objective of the surveys was to determine the thickness of the abutment concrete walls under the bridge and, if possible, the dimensions of the footings under the walls. The distribution of reinforcing steel in the concrete walls and footings was also of interest. The field surveys were conducted on June 6, 2017 by Mike Scully and Alexander Audet of Northeast Geophysical Services (NGS) with the assistance of Erik Friede of GZA. This report summarizes site conditions, methods used, and the results of the geophysical surveys.

SITE LOCATION

The Rines Hill Bridge is located on Water Street in Augusta, Maine where Water Street passes over an old railroad bed that is now used as an access road to a parking lot and city bus stop. No rails are visible in this portion of the old bed. The GPR profiles were all collected under the bridge on the abutment and adjacent retaining walls and on the old railroad bed surface between the two abutments. The exposed abutment walls are vertical and the westerly abutment is the taller of the two rising over 25 feet above the rail bed while the easterly abutment wall is about 15 feet high. The abutment walls are approximately 30 feet apart under the bridge.

METHODS AND INSTRUMENTATION

Ground Penetrating Radar (GPR): Ground penetrating radar utilizes high frequency radio waves to probe the subsurface. Radar waves are transmitted into the substrate from an antenna that is pulled across the surface. In the subsurface, radar waves are reflected at interfaces of materials with contrasting dielectric properties. The returning signal is intercepted by a receiver and converted to a digital graphic image. The horizontal axis of the image is distance along the traverse. The vertical axis is two-way travel time of the radar pulses in nanoseconds (ns), which can be converted to depth using certain assumptions about the materials that the signal is traveling through.

The GPR graphic images are examined and features noted on the images can then be transferred to a map or section. Tanks, pipelines and other objects with rounded tops (boulders, tree roots, or segments of old foundations, for example) may show up on the profiles as hyperbola-shaped reflections. Metal reflects 100% of the radar signal so metal objects like tanks, metal pipes and reinforcing steel rods normally cause very strong reflections on the radar profiles. Tanks and pipelines usually appear on more than one survey line as hyperbolic reflectors on lines perpendicular to the tank or pipe axis and as horizontal reflectors on lines along the axis of the feature. The GPR instrument used here was a GSSI, SIR-3000. A 500 MHz antenna was used for the vertical profiles on the abutments and retaining walls, while a 400 MHz antenna was used on the horizontal profiles done on the ground surface.

Field Procedures: Nine vertical profiles were conducted on the abutment walls under the bridge

Northeast Geophysical Services

and adjacent retaining walls (See attached profiles V1 through V9). These profiles were recorded using an extension ladder that was set up in a normal fashion against the walls. Five foot distance marks were made on the wall using paint at each location to allow later distance calibration of the profile records. One person would climb the ladder to the top of the profile (15-20 Feet) and then pull up the antenna and control cable using a rope. That person would then slowly slide the antenna down the wall while the second person ran the GPR machine and held the ladder.

Nine additional GPR profiles were recorded on the ground surface using a 400 MHz antenna setup on the GSSI survey cart. Seven profiles (P1 through P7) were recorded on the surface of the old rail bed perpendicular to the abutment walls and retaining walls. The antenna was mounted on the front forks of the cart so that it could be pushed right up against each of the walls. Two profiles (L1 & L2) were also recorded on the ground surface parallel to the walls approximately three feet out from each of the walls.

SURVEY RESULTS

The attached 18 profiles show the results of the GPR surveys at this site. Following the field survey the raw GPR data was processed using GSSI's RADAN software. Preliminary profiles were prepared and then interpreted by visual inspection. A set of draft interpretive profiles were then sent to GZA for discussion. Following those discussions some additional alternative interpretations of the data were added to the profiles based upon their knowledge of the structures. The attached annotated final profiles were produced using Golden Software's Surfer program and then printed in pdf format.

Vertical Profiles (V1-V9): The main purpose of the vertical profiles was to attempt to determine the shape and thickness of the concrete abutments and adjacent retaining walls. A secondary purpose was to determine the distribution of reinforcing steel in the structures. For these profiles the results of the surveys show that the GPR conducted was not very successful at confidently determining the shape and thickness of the abutments or retaining walls. The main reasons that the GPR did not work well for this purpose were likely: 1) the radar signal did not penetrate the concrete sufficiently because of the amount of reinforcing steel in the structure, and 2) there was not enough of a dielectric contrast between the back surface of the wall and the fill behind it. Subsequent surface borings performed by GZA found that the abutment walls were significantly thicker than either of the interpretations provided here.

The GPR profiles were somewhat successful at showing the distribution of the near-surface horizontal reinforcing steel in the structures. The steel in the retaining wall south of the bridge (V1) showed up the best and appears to be at 1 foot spacing in the horizontal direction. The steel reflections in the abutment structures were not as clear but appear to be at about 1 to 1.5 foot spacing.

Ground Surface Profiles (P1-P7 & L1-L2): The main purpose of these profiles was to determine the width and thickness of a footing under the abutment structures – assuming that there would be a footing that extended out beyond the exposed face of the abutments. As seen on the attached profiles the main features detected in this data were remnants of the old railroad that passed under the bridge. It appears that the current vehicle road was built on top of the old

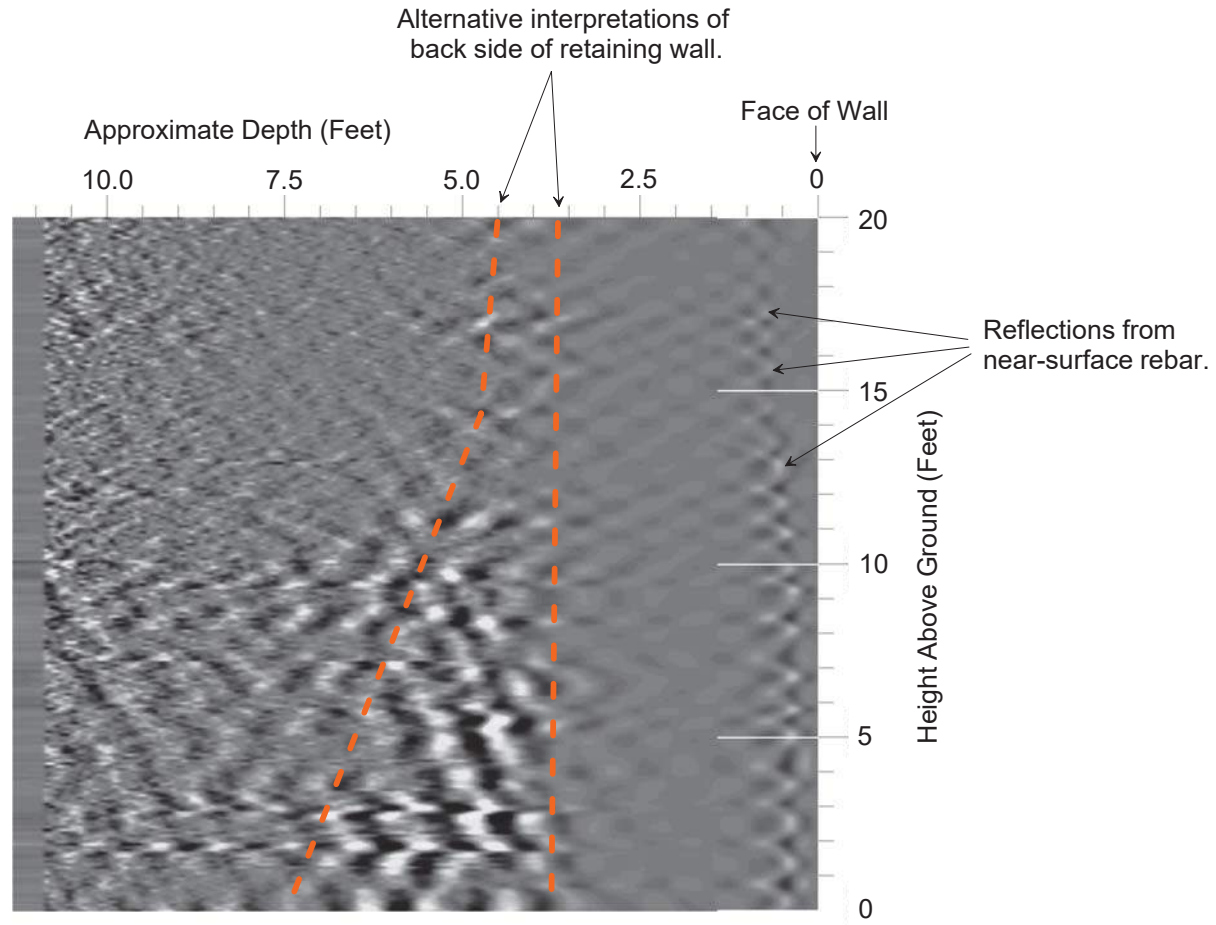
Northeast Geophysical Services

railroad without removing any of the rails, ties or ballast there. These features caused very strong reflections within the top 2 feet that make it impossible to discern any more subtle features below. It was hoped that we could possibly see the top and perhaps the bottom edges of the face of a structural footing under the abutments but that was not possible. It appears from the pattern of the rails detected that there were two separate train tracks north of the bridge that eventually merge on the south side of the bridge.

LIMITATIONS OF THE SURVEYS

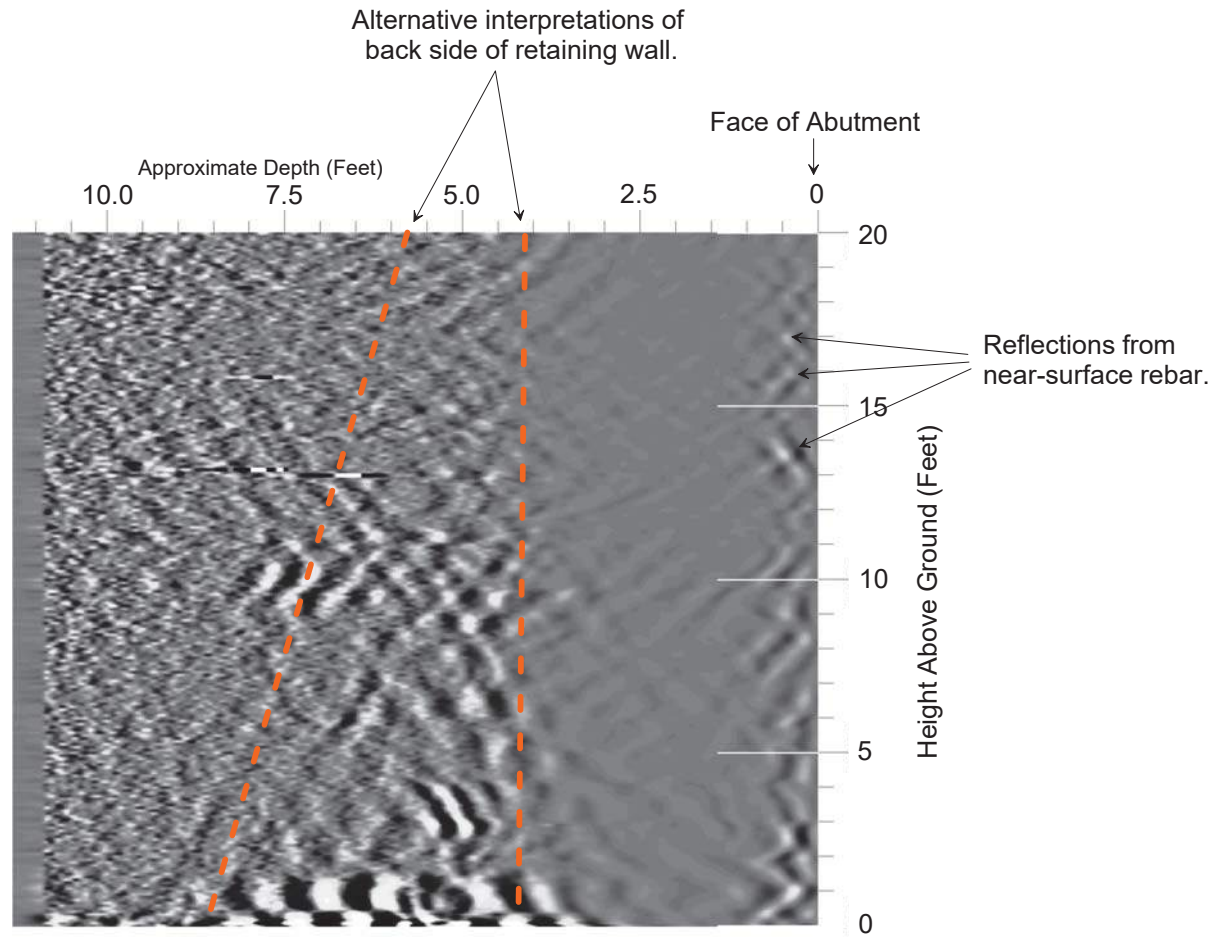
The GPR survey produces reflectors at interfaces of materials with contrasting dielectric properties. The instrument provide indirect measurements of subsurface conditions. The actual cause of the features depicted on the figures can only be conclusively determined by direct observation.

RINES HILL BRIDGE, AUGUSTA, ME



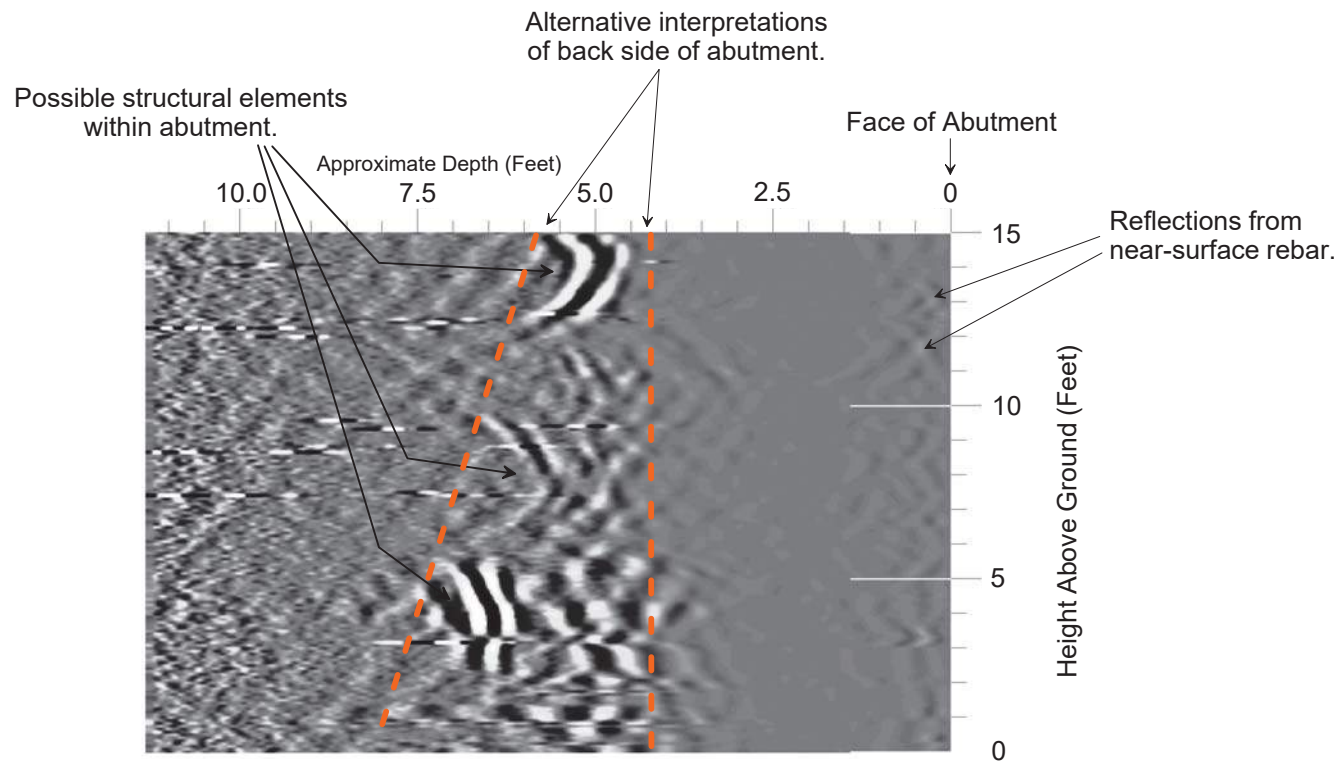
GPR Profile V1: Vertical profile at 4+00' on west retaining wall south of bridge.

RINES HILL BRIDGE, AUGUSTA, ME



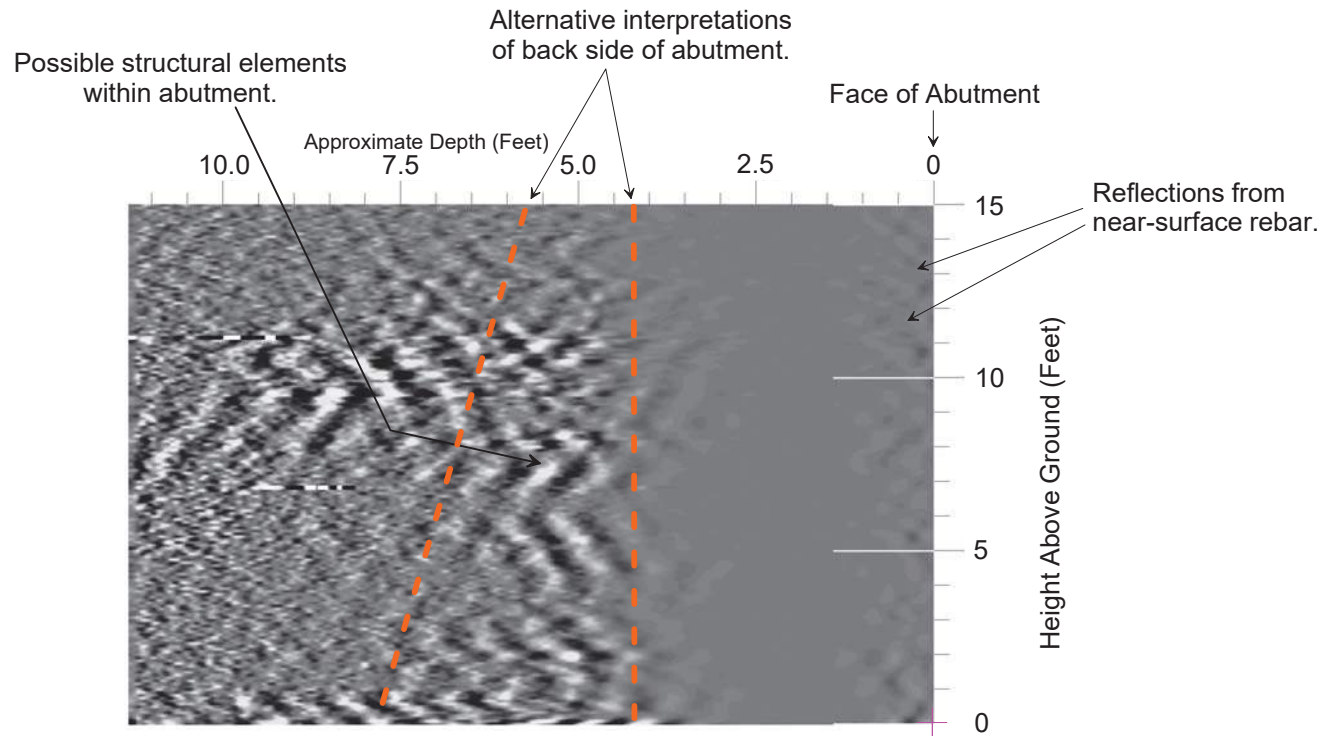
GPR Profile V2: Vertical profile at 2+88' on west abutment wall under south side of bridge.

RINES HILL BRIDGE, AUGUSTA, ME



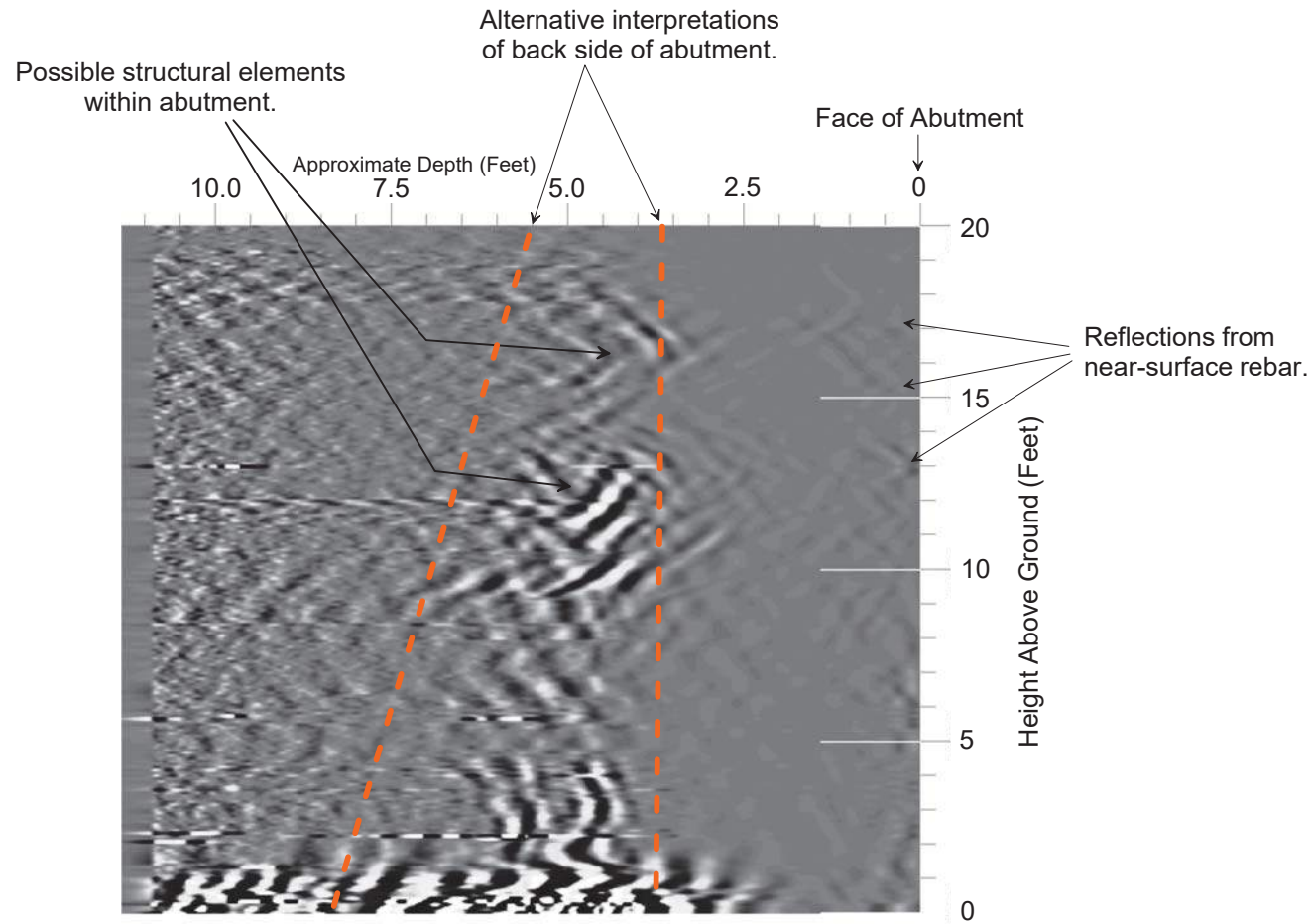
GPR Profile V3: Vertical profile at 2+60' on west abutment wall under bridge.

RINES HILL BRIDGE, AUGUSTA, ME



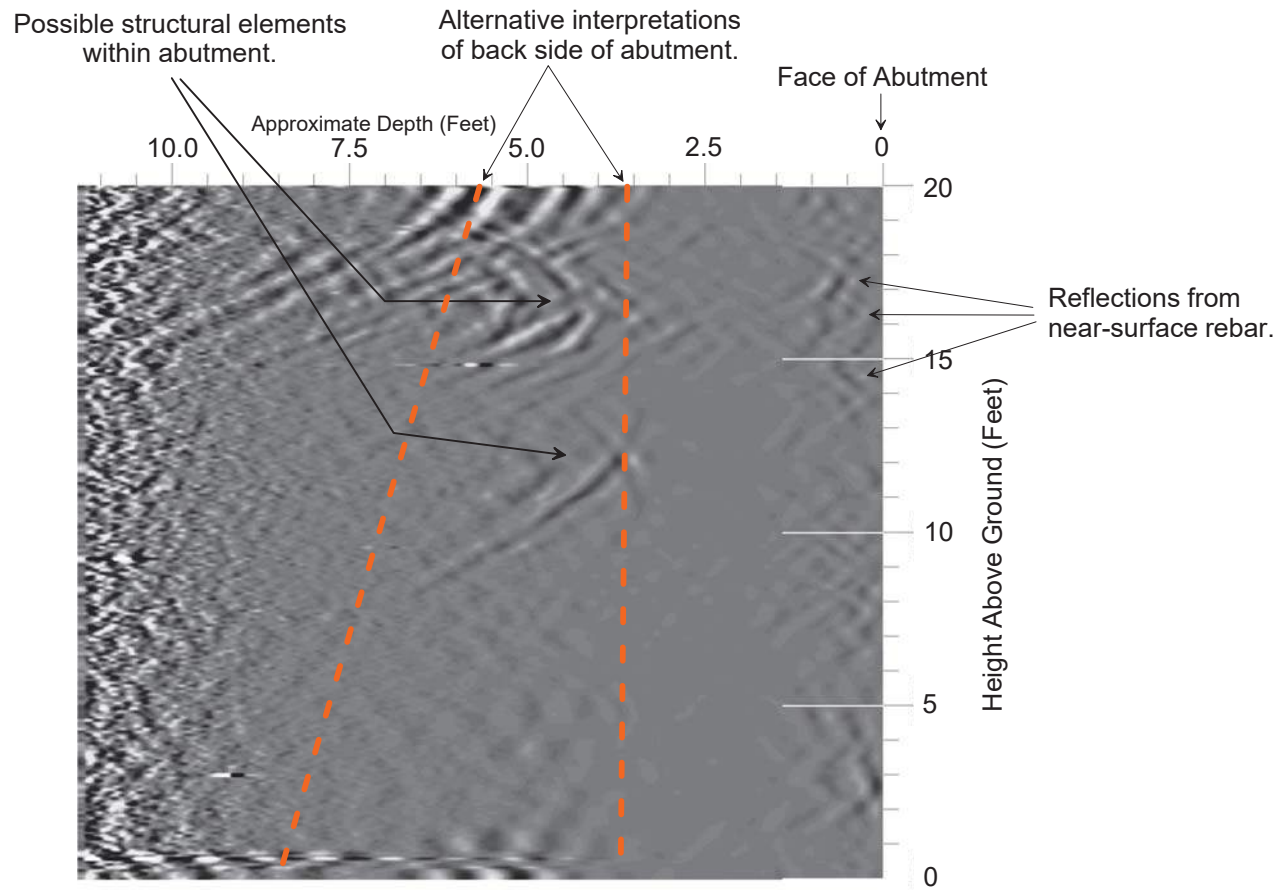
GPR Profile V4: Vertical profile at 2+32' on west abutment wall under bridge.

RINES HILL BRIDGE, AUGUSTA, ME



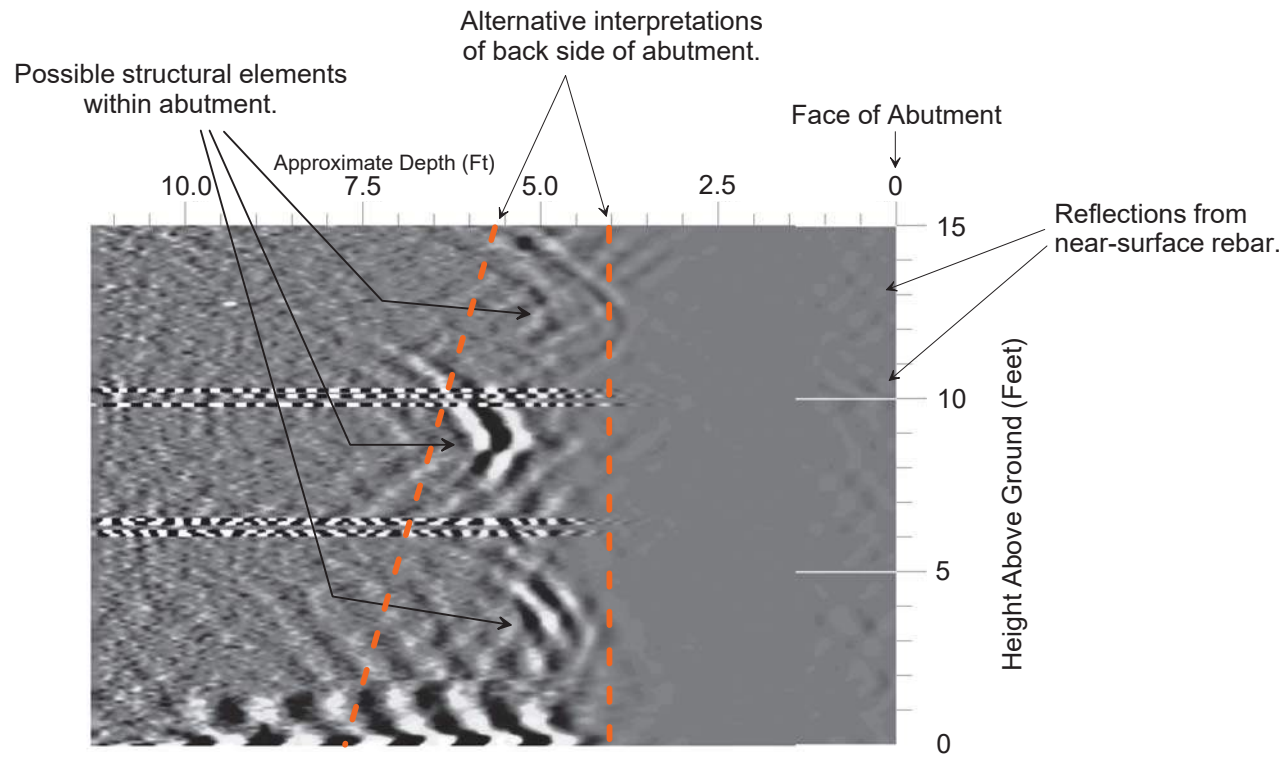
GPR Profile V5: Vertical profile at 2+24' on east abutment wall under south side of bridge.

RINES HILL BRIDGE, AUGUSTA, ME



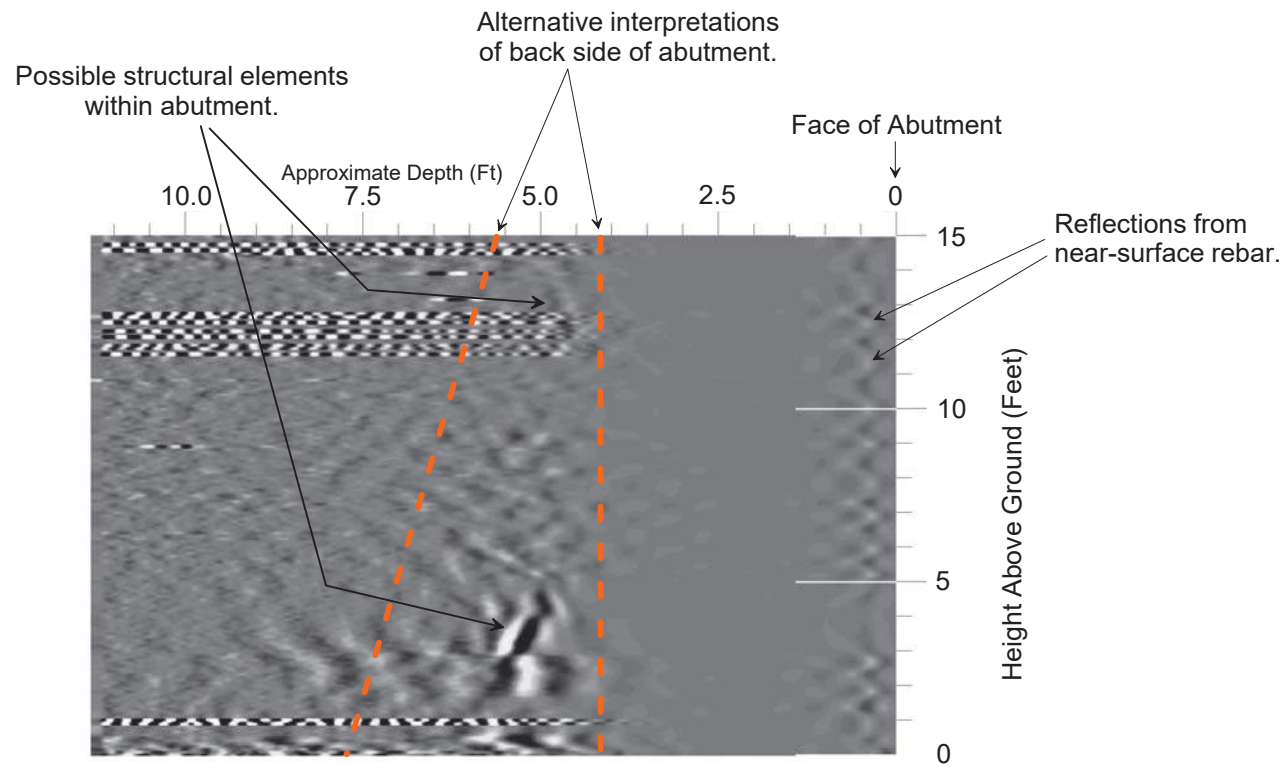
GPR Profile V6: Vertical profile at 1+85' on west abutment wall under north side of bridge.

RINES HILL BRIDGE, AUGUSTA, ME



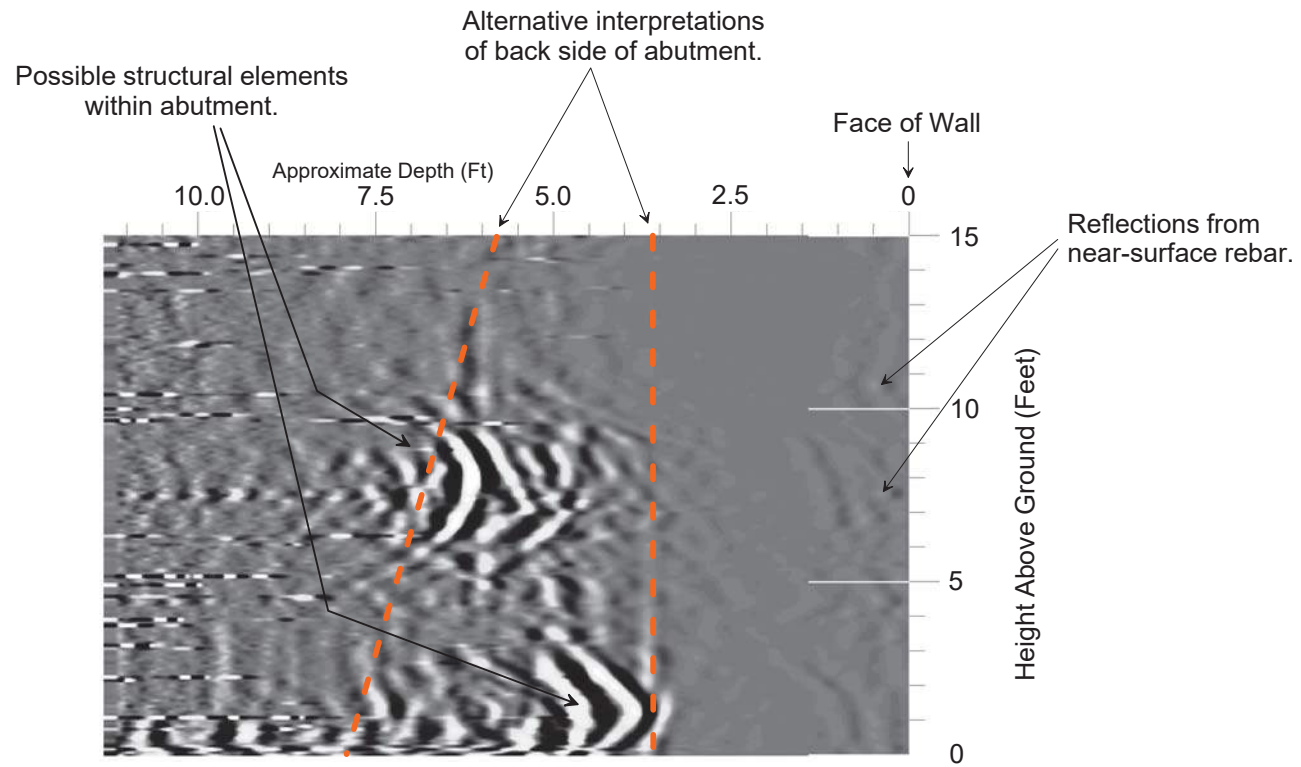
GPR Profile V7: Vertical profile at 1+85' on east abutment wall under bridge.

RINES HILL BRIDGE, AUGUSTA, ME



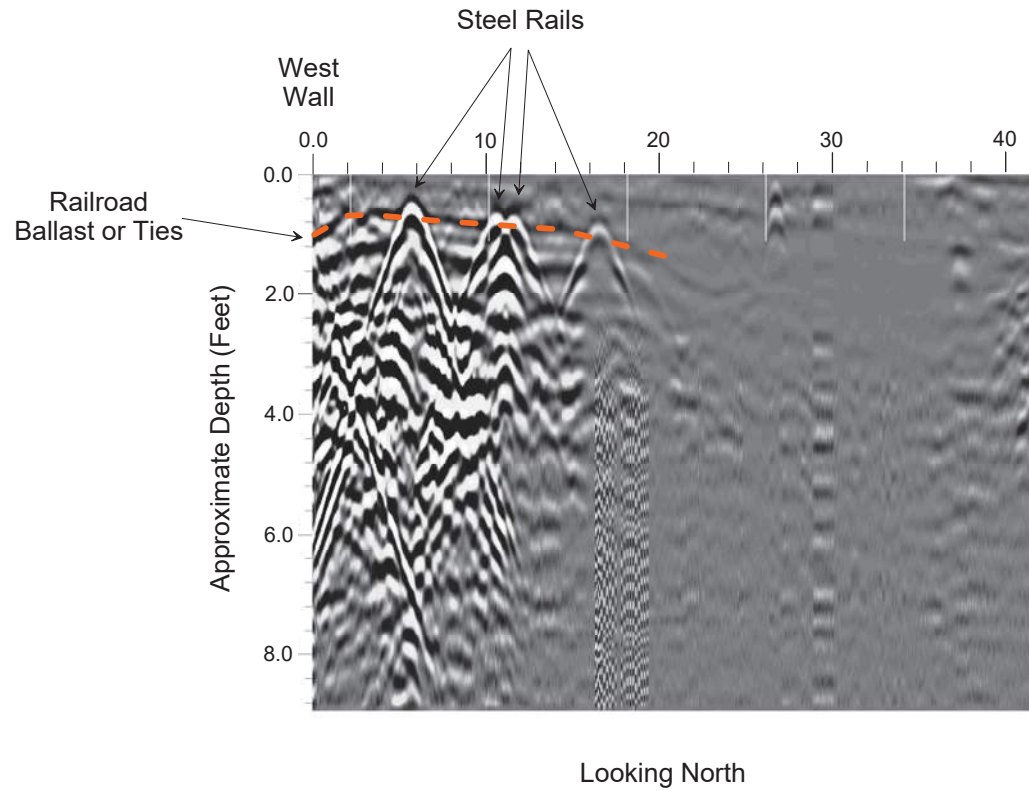
GPR Profile V8: Vertical profile at 1+50' on east abutment wall under north side of bridge.

RINES HILL BRIDGE, AUGUSTA, ME



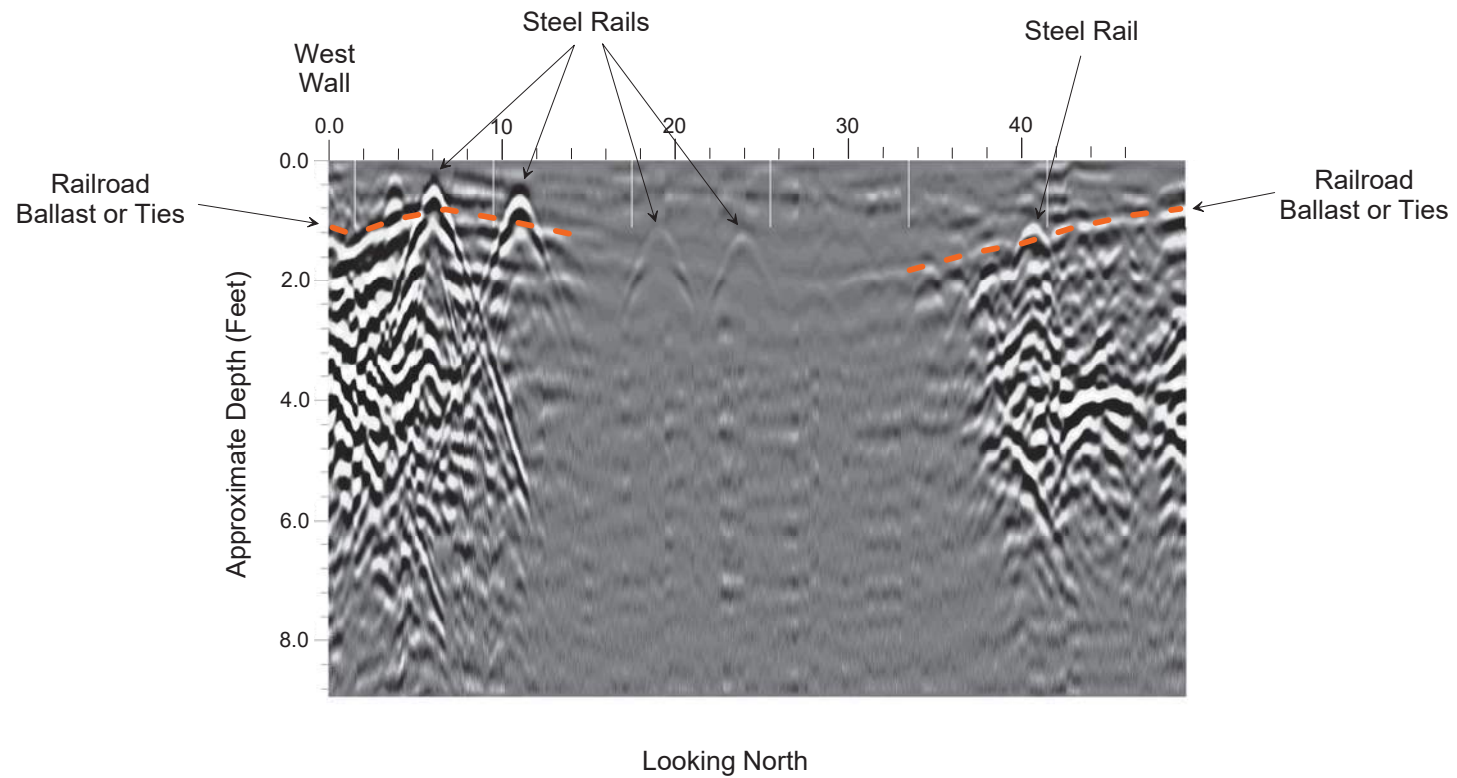
GPR Profile V9: Vertical profile at 1+20' on east abutment wing wall on north side of bridge.

RINES HILL BRIDGE, AUGUSTA, ME



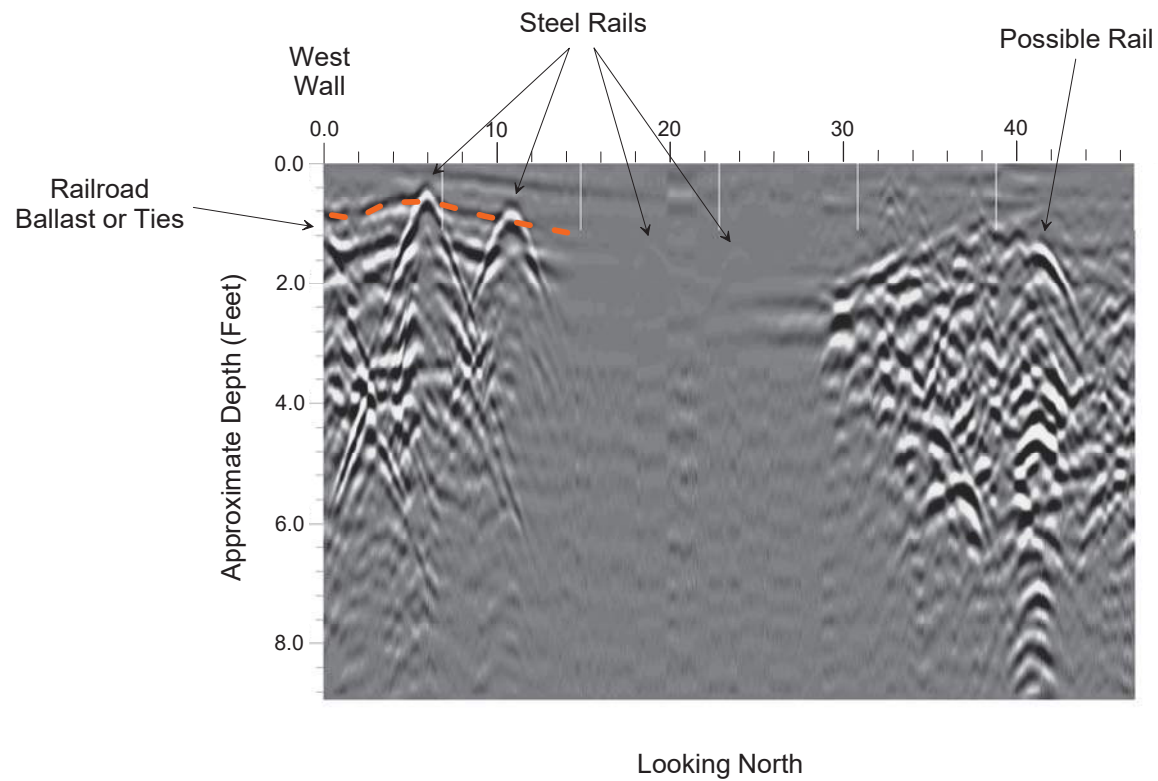
GPR Profile P1: Profile along ground from retaining wall at 4+00', west to east.

RINES HILL BRIDGE, AUGUSTA, ME



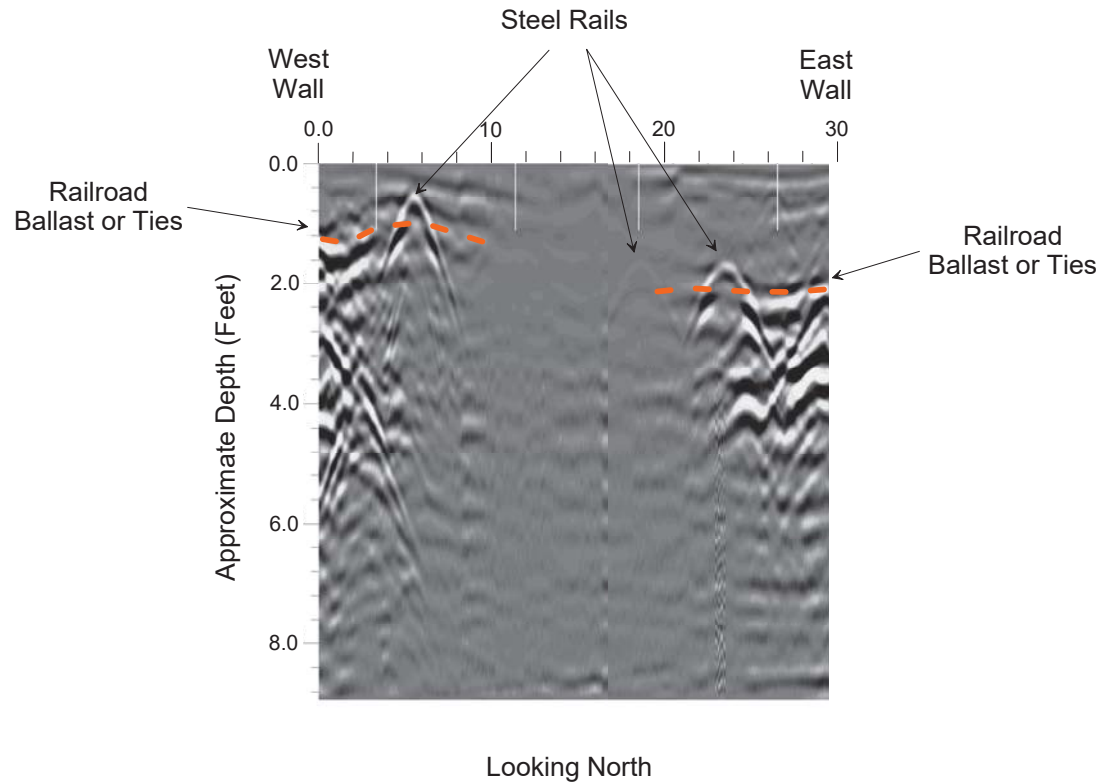
GPR Profile P2: Profile along ground from west abutment at 2+88', west to east.
Line passes to the south of the east abutment wing wall.

RINES HILL BRIDGE, AUGUSTA, ME



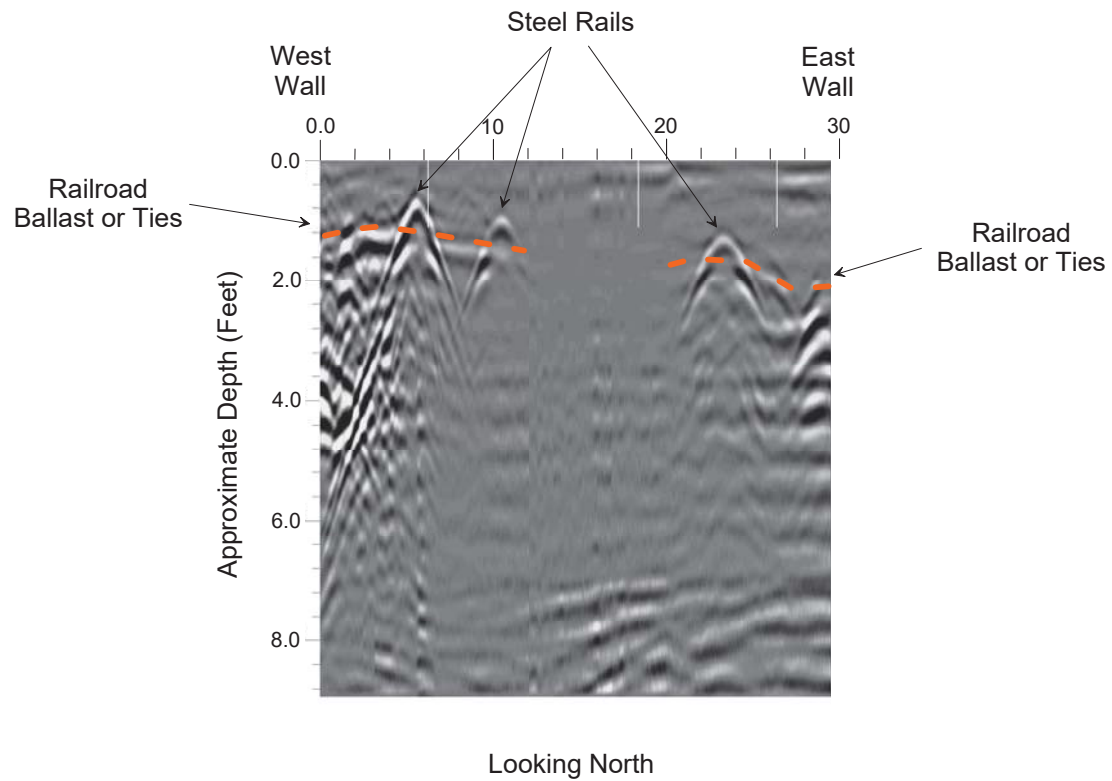
GPR Profile P3: Profile along ground from west abutment at 2+60', west to east.
Line passes to the south of the east abutment wing wall.

RINES HILL BRIDGE, AUGUSTA, ME



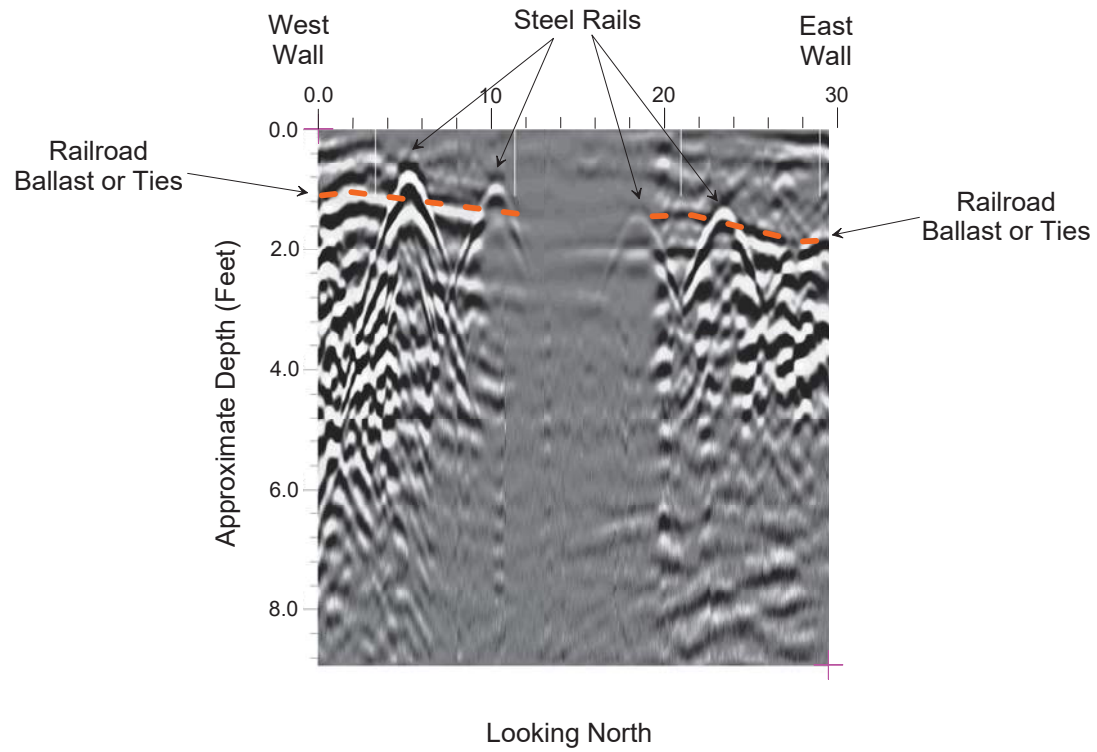
GPR Profile P4: Profile along ground between abutments at 2+00' from west to east.

RINES HILL BRIDGE, AUGUSTA, ME



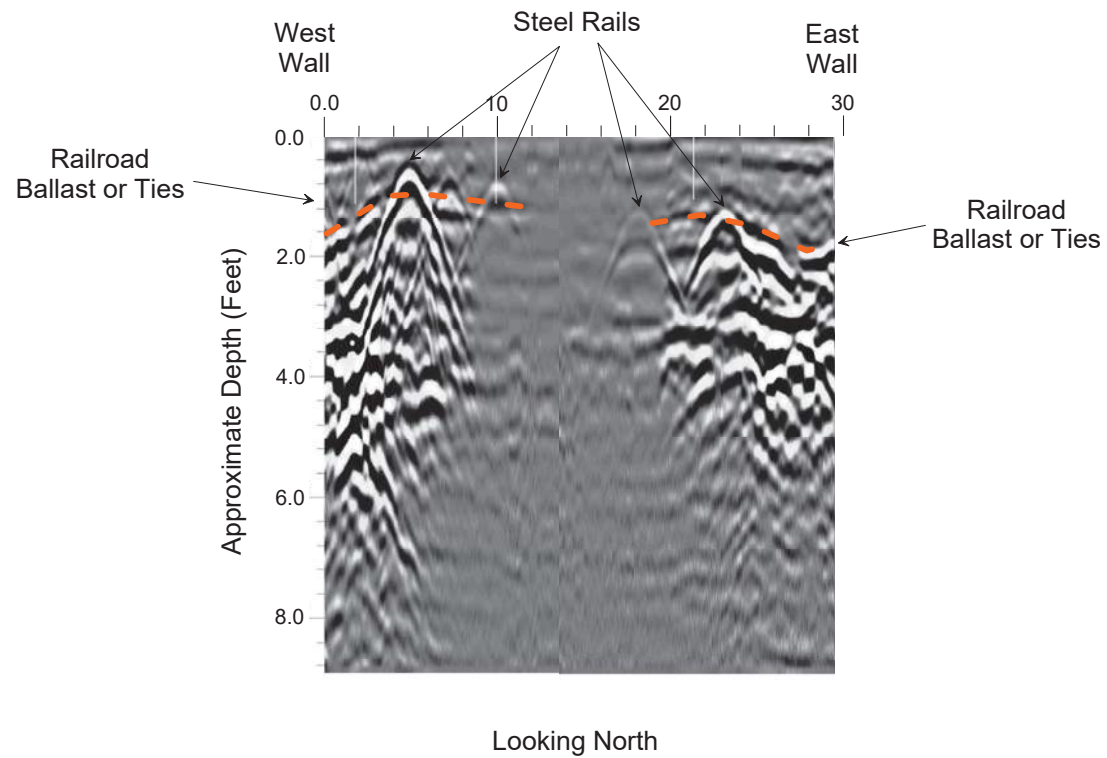
GPR Profile P5: Profile along ground between abutments at 1+85' from west to east.

RINES HILL BRIDGE, AUGUSTA, ME



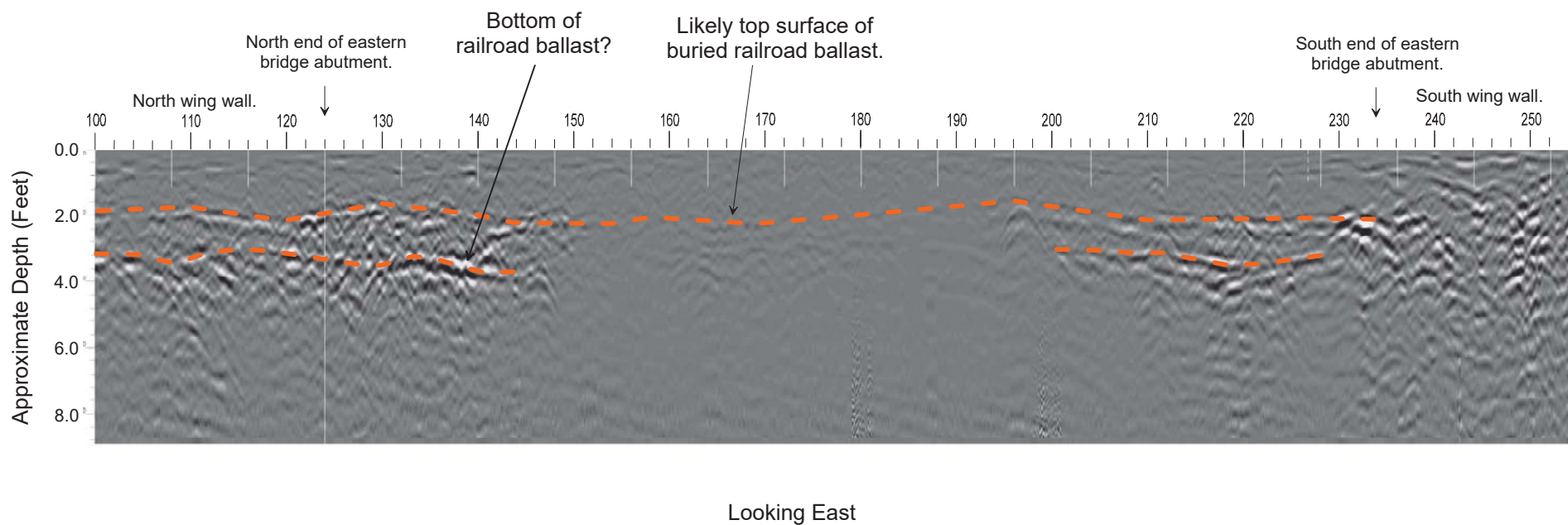
GPR Profile P6: Profile along ground between abutments at 1+50' from west to east.

RINES HILL BRIDGE, AUGUSTA, ME



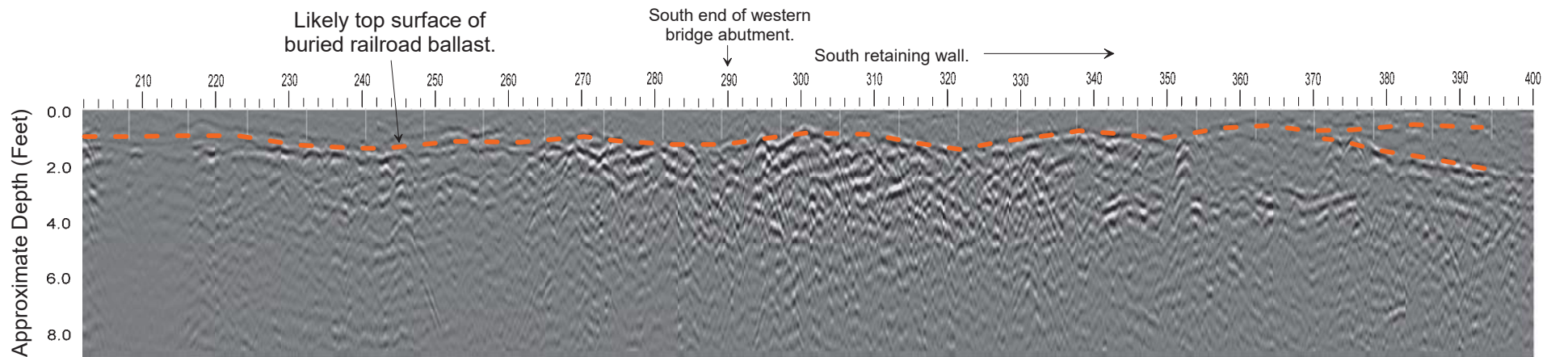
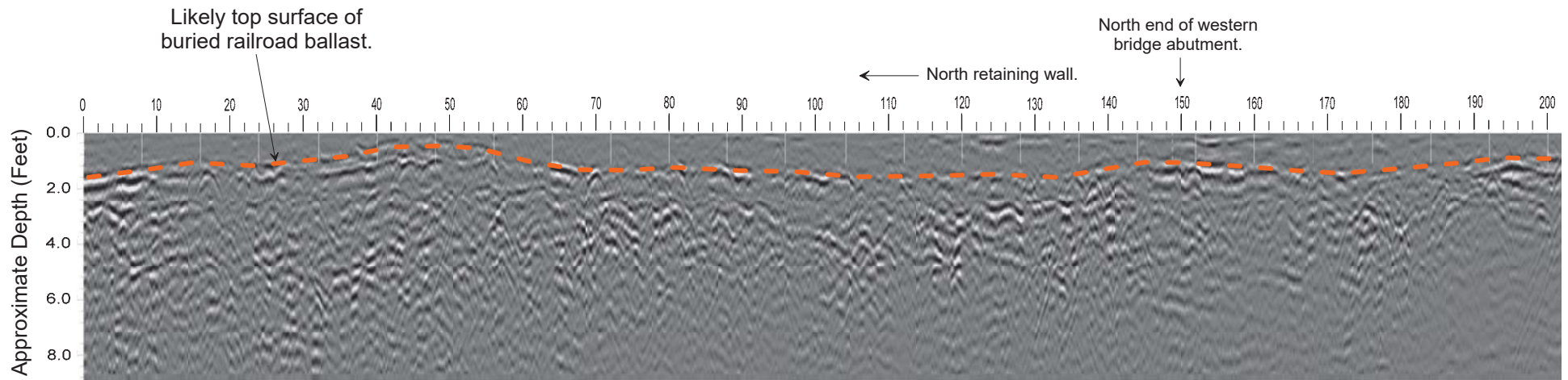
GPR Profile P7: Profile along ground between abutments at 1+20' from west to east.

RINES HILL BRIDGE, AUGUSTA, ME



GPR Profile L1: Profile along ground adjacent to east side bridge abutment from 1+00' to 2+40', north to south.

RINES HILL BRIDGE, AUGUSTA, ME



Looking East

GPR Profile L2: Profile along ground adjacent to west side bridge abutment from 0+00' to 4+00', north to south.



10/18/2022

**RINES HILL BRIDGE NO. 3528 OVER OLD MAINE CENTRAL RAILROAD
GEOTECHNICAL DESIGN REPORT**

09.0025943.01

APPENDIX D – NDT GEOPHYSICAL REPORT

Ground Penetrating Radar Survey
Rines Hill Bridge – Bridge No. 3528

Water Street Over Kennebec River Rail Trail

Augusta, Maine

Prepared for

GZA

May, 2019



We Save Structures™

May 1, 2019

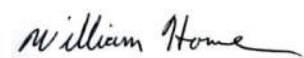
Mr. Andrew R. Blaisdell, P.E., G.E.*
Associate Principal / Geotechnical Engineer
GZA
1477 Congress Street, Suite 700
Portland, Maine 04101

Dear Mr. Blaisdell:

NDT Corporation conducted ground penetrating radar (GPR) measurements on the North and South approaches of Bridge No. 3528, Water Street over the Kennebec River Rail Trail in Augusta, Maine. Field work was conducted on April 12th, 2019. The objective of this survey is to determine the subsurface conditions/layering by locating areas where the reflection patterns are changing to determine the length, width, and depth dimensions of the layers below the roadway surface. This report presents the results of the GPR measurements to aid GZA engineers with their bridge renovation evaluation.

We thank you for the opportunity to perform this work and look forward to being of service to you in the future. If you have any questions or comments, contact the undersigned at 978-563-1327.

Sincerely,
NDT Corporation



William Horne

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1.0	INTRODUCTION AND PURPOSE	Page 1
2.0	LOCATION AND SURVEY CONTROL	Page 1
3.0	NONDESTRUCTIVE TESTING METHODS	Page 1
4.0	RESULTS	Page 2

FIGURES

APPENDIX 1	GPR Method of Investigation
APPENDIX 2	Cross-sections and GPR Data for cross-lines along the North Approach
APPENDIX 3	Cross-sections and GPR Data for cross-lines along the South Approach

1.0 INTRODUCTION AND PURPOSE

Based on preliminary information provided to NDT by GZA, the subsurface materials found at the north and south approaches of Bridge No. 3528, Water Street over the Kennebec River Rail Trail in Augusta, Maine includes a variety of asphalt, buried concrete slabs, cobblestones, sub-grade and gravel layers. NDT Corporation conducted a GPR survey on April 12th, 2019 along the north and south approaches of Water Street (approximately 100 feet north and south of each abutment) to evaluate subsurface layering conditions, thicknesses, and lateral extents.

2.0 LOCATION AND SURVEY CONTROL

The location of Bridge No. 3528, Water Street over the Kennebec River Rail Trail in Augusta, Maine is shown on Figure 1. Figure 2 is an aerial photograph of the bridge. The stationing used by NDT is shown as white lines overlain on the photograph. Due to the skew of the bridge abutments and traffic control considerations Station 1+00 was located at the north-west corner of the side walk and the bridge. From this location a survey wheel was used to locate Station 0+00. Station 0+00 was marked on the west curb/street and the east curb/street and the survey wheel was used to mark 20 foot stations along both the west and east curb lines.

GPR data was collected along longitudinal (north to south) lines at 5 foot spacing (5' – middle sidewalk, 10' - west curb, ... 30' - centerline ... 50' - east curb, and 55' – middle of sidewalk).

GPR data was collected along transverse/cross-lines (west to east) at 20 foot spacing. These lines were begun at the west curb (sta. 10) and ended at the east curb (sta. 50). Between station 2+60 and 3+40 the transverse lines were extended into the fire station parking area.

3.0 NONDESTRUCTIVE TESTING METHODS

3.1 Ground Penetrating Radar (GPR)

The GPR method uses a pulsed electromagnetic signal that is transmitted to and reflected by a target back to the point of transmission. The electromagnetic wave transmission and reflection is dependent on the dielectric constant and conductivity (electrical) properties of the material(s) being investigated. These electrical properties are highly dependent on moisture content, saturated or concentrated moist conditions provide both strong reflections and high attenuation.

GPR data were acquired using a digital system coupled with a 400 MHz, 900 MHz and 1500 MHz antennas. The 400 MHz antenna has an approximate depth of investigation from the surface to depths of 5 to 10 feet; the 900 MHz antenna has an approximate depth of investigation of 3 to 4 feet; and the higher resolution 1,600 MHz antenna has an approximate depth of investigation of 1-2 feet. The actual depth of investigation is dependent on the soil types and moisture conditions. Depths of investigation are usually deeper in dry sands and gravels than in moist silts and clays.

Changes in materials: asphalt pavement, concrete, cobblestone and naturally deposited soils or fill soils placed in lifts during construction retain moisture between material interfaces and typically have horizontal or near horizontal bedding planes. These conditions cause a change in

conductivity which shows as continuous reflective layers on GPR data. Filled utility trenches and old excavations may have similar characteristics. Appendix 1 has a more detailed description of the GPR method of investigation.

4.0 RESULTS

GPR results are summarized in Figure 3, a plan view of the north and south approaches and cross-sections with GPR data are shown in Appendix 2 (north approach) and Appendix 3 (south approach). In general the whole area tested was covered with asphalt/pavement with a variable thickness. The shade areas show the limits of changes in materials below the asphalt/pavement layer.

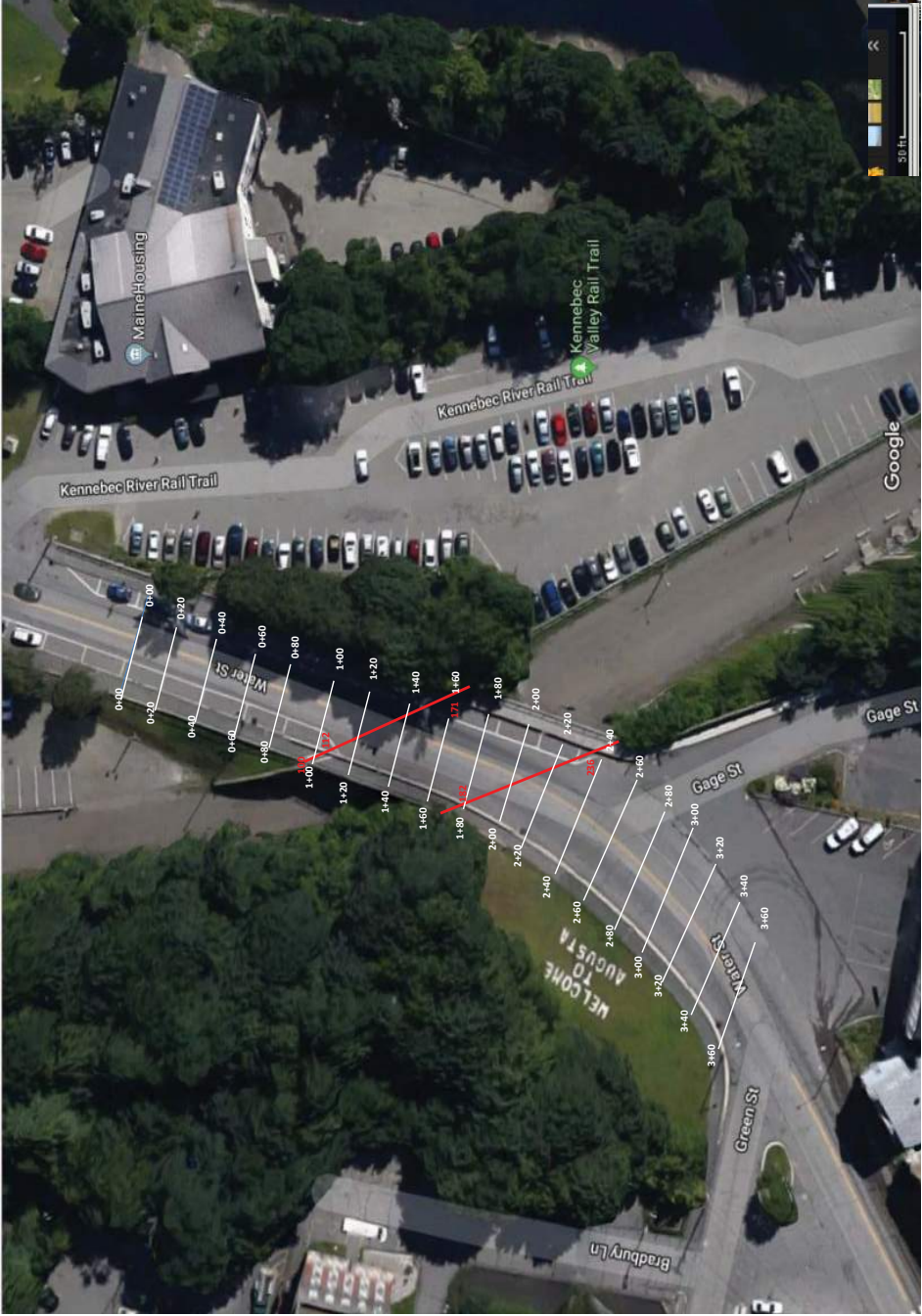
- | | |
|--------------|--|
| 1) Green: | Roadway – typically 8-10 inches of asphalt over soil/fill materials |
| 2) Pink | Sidewalk – typically 4-6 inches of asphalt over soil/fill materials |
| 3) Dark Grey | 12-14” thick concrete roadway slab |
| 4) Orange | Change in materials approximately 10-15 feet wide extending north and south from the bridge deck joints (it is expected that this area is a combination of concrete, soil, asphalt associated with the abutments). |
| 5) Purple | Bridge deck slab – concrete deck |
| 6) Blue | Change in materials along west side of the concrete slab of the south approach. This change in materials may be indicative of cobblestone pavement shown on historic plans. |

FIGURES

Rines Hill Bridge - Bridge No. 3528
Water Street over Kennebec River Rail Trail
Augusta, Maine

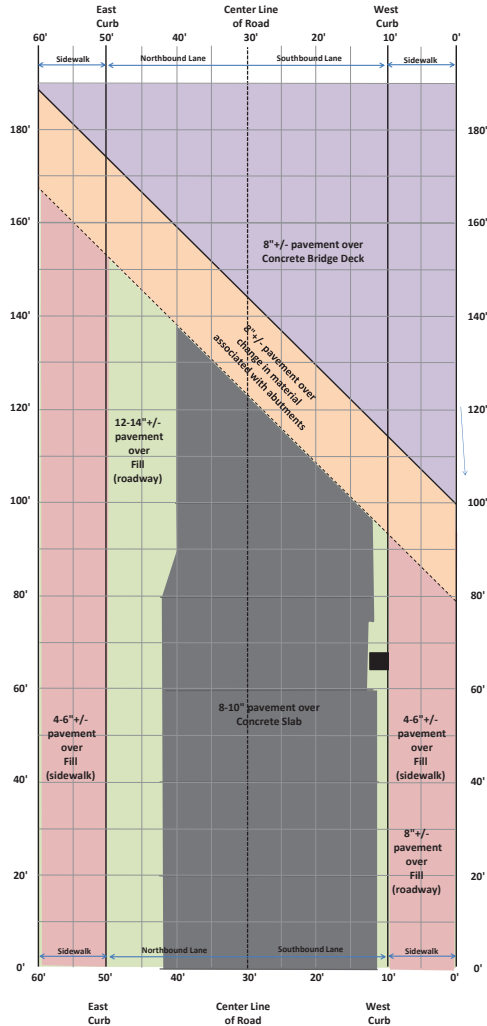


Ground Penetrating Radar Survey Rines Hill Bridge – Bridge No. 3528 Water Street Over Kennebec River Rail Trail Augusta, Maine Prepared for GZA by NDT Corporation		Area of Investigation	
		May 2019	Figure 1

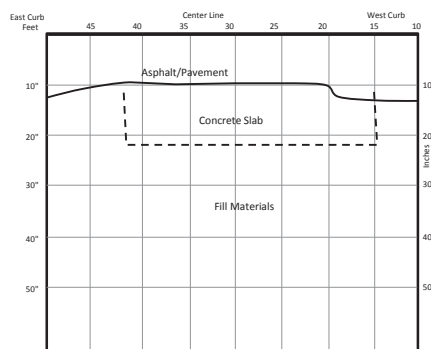


Ground Penetrating Radar Survey Rines Hill Bridge – Bridge No. 3528 Water Street Over Kennebec River Rail Trail Augusta, Maine Prepared for GZA by NDT Corporation	Location and Stationing	
	May 2019	Figure 2

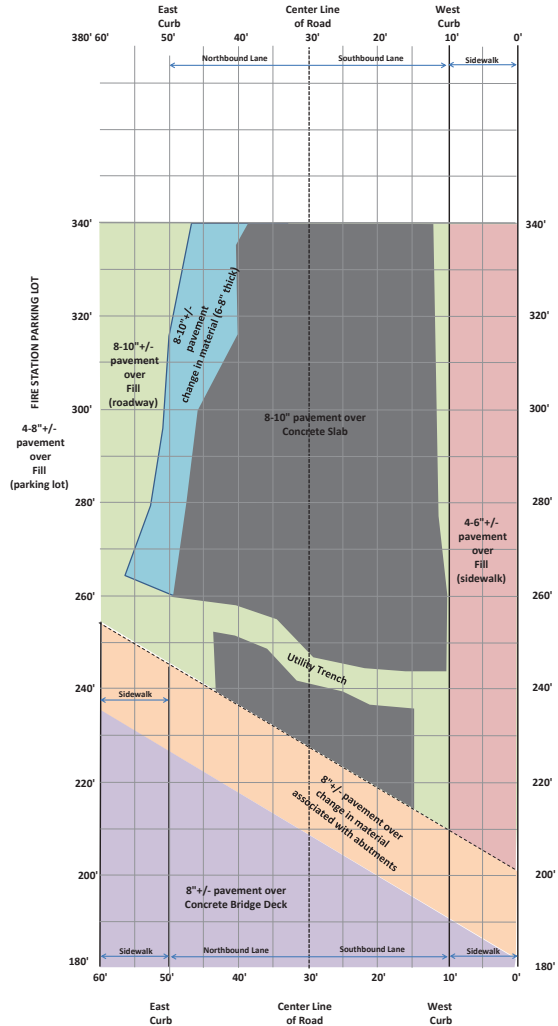
GPR Results for North Approach



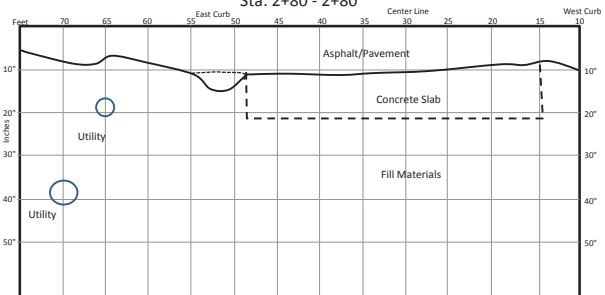
Cross-Section At



GPR Results for South Approach (sketch plan - does not show curve)



Cross-Section At Sta: 2+80 - 2+80



Ground Penetrating Radar Survey
Rines Hill Bridge - Bridge No. 3528
Water Street Over Kennebec River Rail Trail
Augusta, Maine
Prepared for
GEA
by
NDT Corporation

GPR Results
Plan View

April 2019 Figure 3

APPENDIX 1

GPR METHOD OF INVESTIGATION

APPENDIX: GROUND PENETRATING RADAR

Ground Penetrating Radar (GPR) is an electrical geophysical method for evaluating subsurface conditions by transmitting high frequency electromagnetic waves into the ground and detecting the energy reflected back to the surface. Electromagnetic signals are transmitted from the antenna (transmitter and receiver) at ground surface and reflected back to the antenna from interfaces with differing electrical (dielectric constant and conductivity) properties. The greater the contrast in the electrical properties between two materials, the more energy that is reflected to the surface and the more defined results are.

GPR SYSTEM:

GPR systems consist of: Control unit (pulse transmitter, digital recorder, data storage, monitor); and an antenna(s) and survey wheel.



The GPR control unit is a computer which controls data acquisition parameters, such as sampling rate, range, gain control, filtering, etc. The Control Unit also visually displays the data, digitally archives the data, and allows for play back of the data.

Coaxial cable connects the control unit to the antenna. The antenna(s) are sealed and shielded in fiberglass housing.. Selection of the antenna is dictated by the requirements of the survey. For high resolution, near-surface data, a high frequency antenna is used; for deeper penetration investigation, a lower frequency antenna is used. Typically the 100 to 400 MHz antennas are used for geologic surveys; 400 to 900MHz are used for utility, near surface voiding settlement, foundation, etc surveys while the 900 to 1500 MHz are used for concrete reinforcing assessment.

APPLICATIONS

Ground Penetrating Radar (GPR) can be used to locate underground pipes, buried drums, foundations, voids in rock and concrete, soil settlement, determine stratigraphy, depth to water table, buried artifacts, filled excavations, and locate voids/settlement behind walls and under floor slabs, etc. GPR is also a good tool for evaluating concrete structures such as

bridges, walls, beams, ceilings, etc where the GPR can locate rebar and conduits, quantify rebar spacing, cover variability over reinforcing, and concrete thickness.

GPR reflections typically occur at subsurface discontinuities such as:

- Buried metal objects (utilities, tanks, reinforcing)
- Open and water filled voids
- Water table
- Soil stratification
- Seepage paths
- Bedrock fractures

DEPTH OF PENETRATION AND LIMITATIONS

The depth of penetration of GPR is site specific, limited by the attenuation of the electromagnetic energy. Signal attenuation is controlled by four different mechanisms:

- Scattering: energy losses due to scattering occur when signals are dispersed in random directions, away from the receiving antenna, by closely spaced rebar or large irregular shaped objects, such as boulders or tree stumps.
- High conductivity layers: the greater the conductivity values of materials at a site, the more signal attenuation or less penetration. (Mineral content, high moisture content, water table, metal plates, etc.)

Signal penetration is also dependent on the frequency of the antenna. High frequency antennas have shallow penetration and high resolution. Low frequency antennas have greater depths of penetration, but the resolution of small and near surface targets is reduced. Listed below are antenna frequency, approximate depths of penetration and typical application. (Depths of penetration are in ideal conditions if a highly conductive layer, such as a brackish water table, steel plate, etc., is present all antennas will be limited to the depth of this layer.)

1500 and 1600 MHz	+/-2 feet	Asphalt/Concrete thickness Wire mesh/rebar/conduit location Voiding within and behind structures
900 MHz	3-5 feet	Concrete thickness Rebar and utility location Voiding within and behind structures
400 MHz	10-15 feet	Concrete/Masonry thickness Utility location Soil settlement/sinkhole development Geologic and Environmental mapping Archaeological Surveys
200 MHz	25-30	Soil settlement/sinkhole development

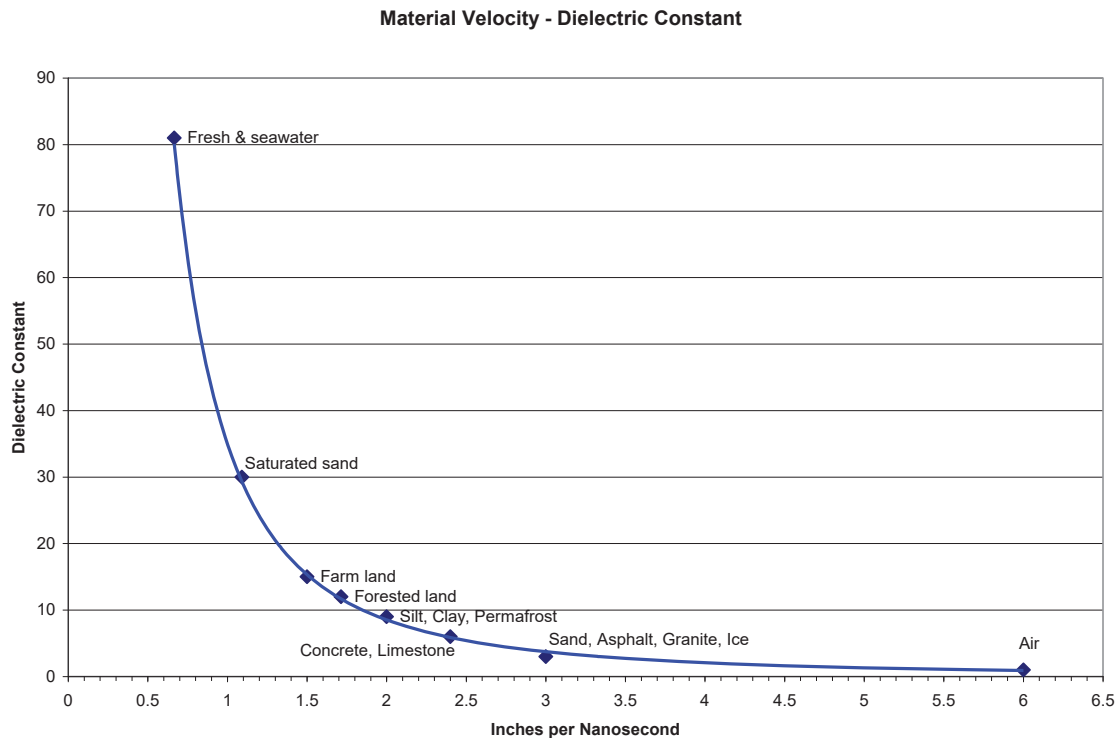
Geologic and Environmental mapping
Archaeological Surveys

100 MHz

+/-50

Soil settlement/sinkhole development
Geologic and Environmental mapping
Archaeological Surveys

Depth of investigation can be estimated using material dielectric constants and the diagram shown below. Typically 2 inches per nanosecond can be used as an average signal velocity for most materials and sites. When available an onsite depth calibration can be conducted to determine the electrical properties (speed of the signal) of the materials at the site. Depth calibrations typically consist of collecting GPR data over a metal target with a known depth. Known utilities, and buried metal plates are good targets for calibrations. GPR surveys can be very effective when coupled with other geophysical surveys and/or ground truth methods to verify, correlate and extrapolate GPR results. GPR surveys are a fast and cost effective method to collect data over large or obstructed sites, and isolate anomalies and areas where borings or other methods can be focused for the best interest of a project.

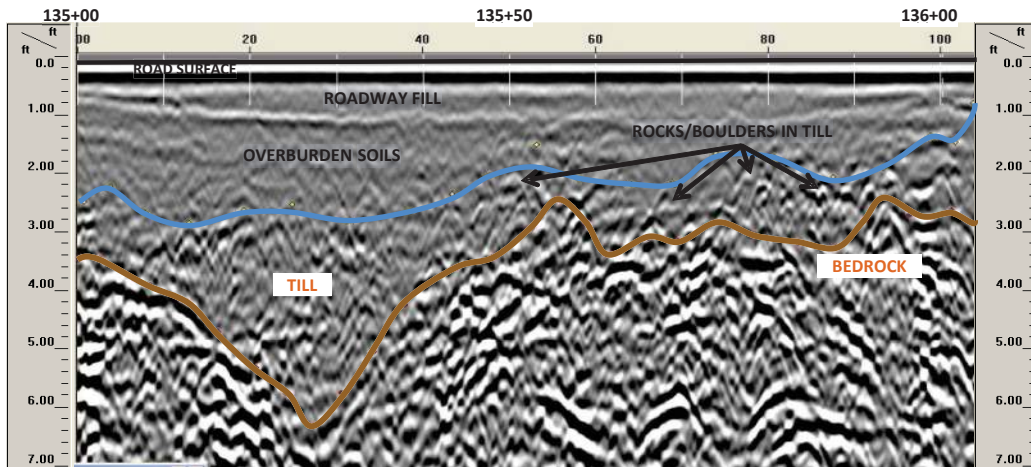


ACQUISITION AND INTERPRETATION:

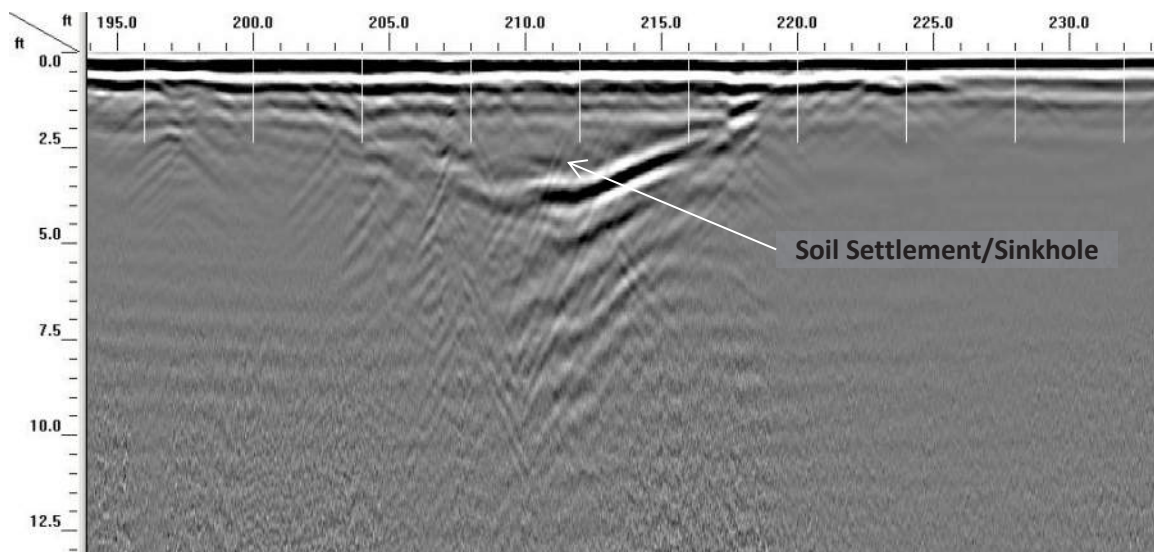
Radar data are typically acquired at a slow walking speed along a grid pattern of survey lines or a series of parallel lines. Data is displayed on LCD screen for field verification and quality control of results and digitally saved. Calibrated measuring wheels are used to automatically added footage/station markers to the digital data. The saved data can be printed or post processed.

Interpretation of GPR data is subjective. GPR results should be verified with borings or test pits. GPR lines indicate a cross-section in time/depth along a survey line.

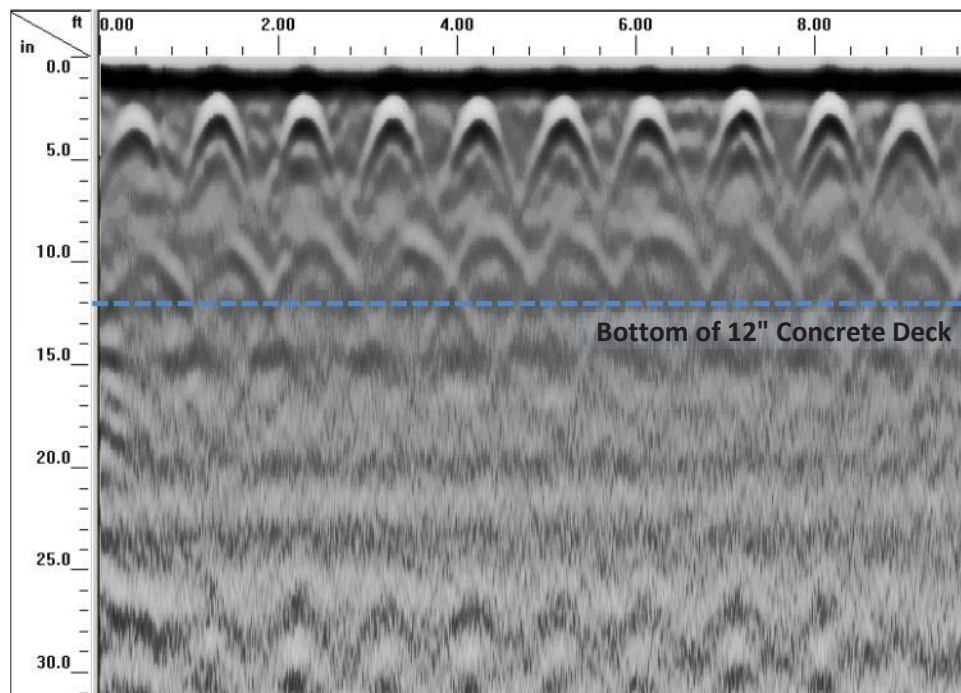
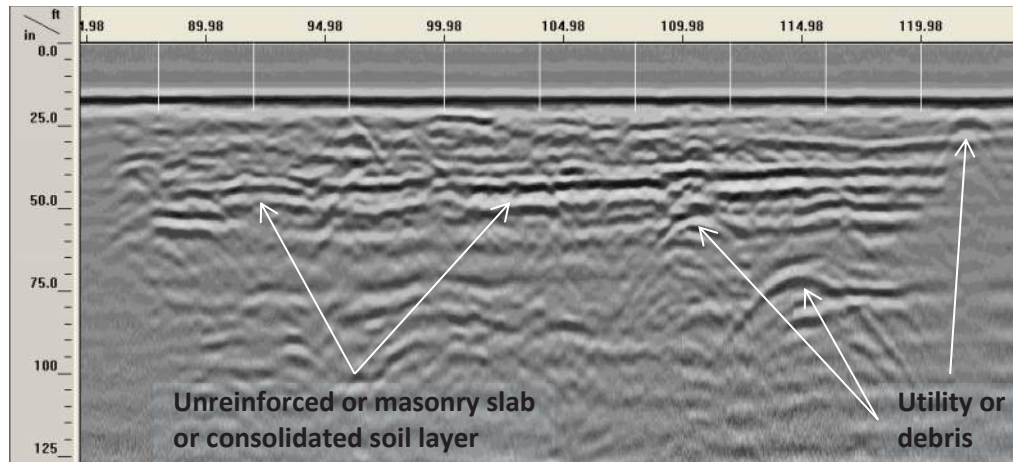
Natural soils or fill placed in lifts during construction retain moisture between material interfaces and typically have horizontal or near horizontal bedding planes. These conditions cause a change in conductivity which shows as continuous reflective layers on GPR data. The strength of a reflected signal and/or the continuity of the reflector across the record may be indicative of a stratigraphic contact, water table, top of rock, back of wall/slab.



Locations where GPR data indicates these horizontal bedding planes/layers are sloping, draped or disturbed can be indication of soil settlement, trenching and/or voiding. Areas where GPR data is less reflective, indicating fine soil materials (clays and silts) have been washed or eroded away or areas that are more reflective, indicating loose soil conditions where moisture has accumulated are also indicative of and associated with settlement, sinkholes, and voiding.



Often point targets, such as reinforcing, buried utilities, boulders, create a distinctive parabolic feature on GPR records. Point targets trending perpendicular to the direction of the line of coverage are detected, therefore to detect longitudinal reinforcing a transverse line of data would be collected and to detect transverse reinforcing, a longitudinal line of data would be collected. Plotting point targets of similar signal strength, depth, and shape located along the grid of GPR lines give the trend and location of individual utilities and/or reinforcing.



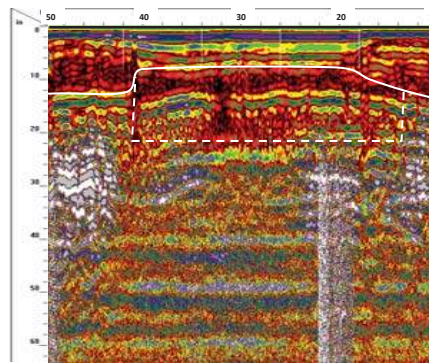
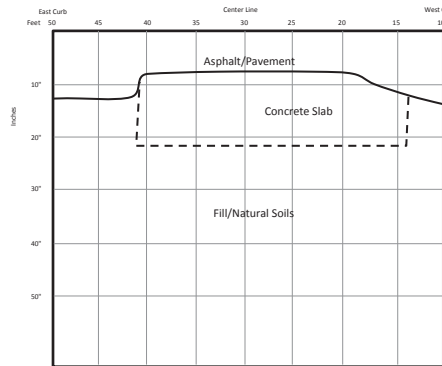
Top Reinforcing
12" spacing
2+/- inches cover

Bottom Reinforcing
12" spacing
8-9 inches cover

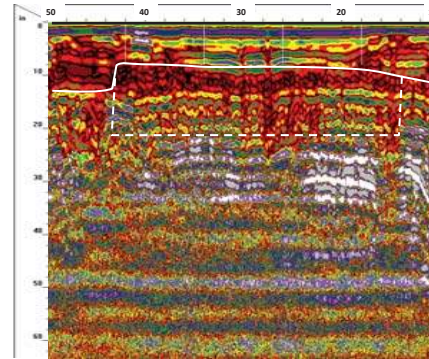
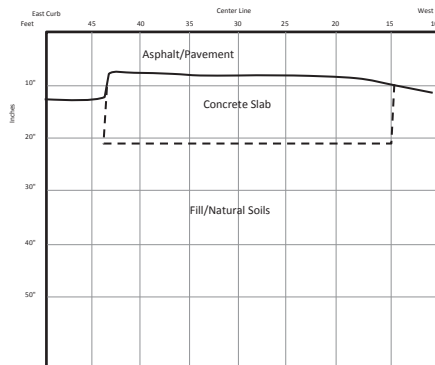
APPENDIX 2

Cross-sections and GPR Data
for cross-lines along the North Approach

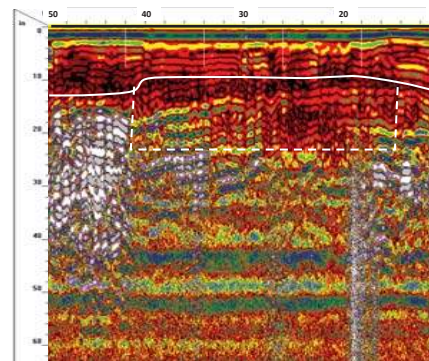
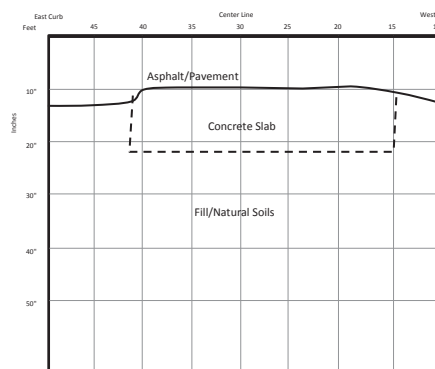
Cross- Section At
Sta: 0+00



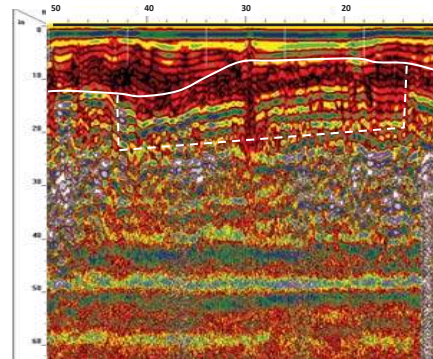
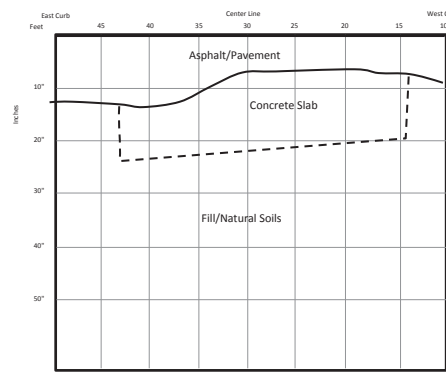
Cross- Section At
Sta: 0+20



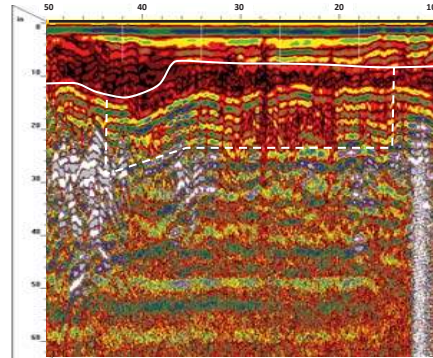
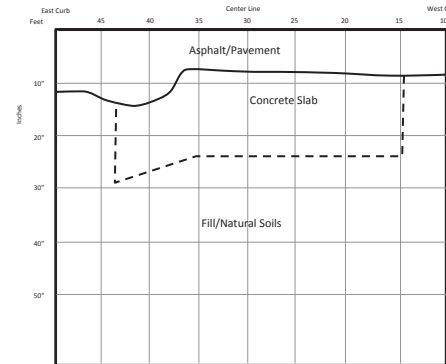
Cross- Section At
Sta: 0+40



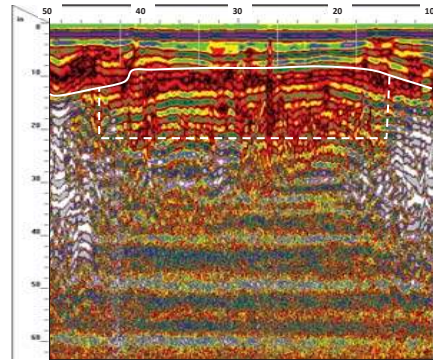
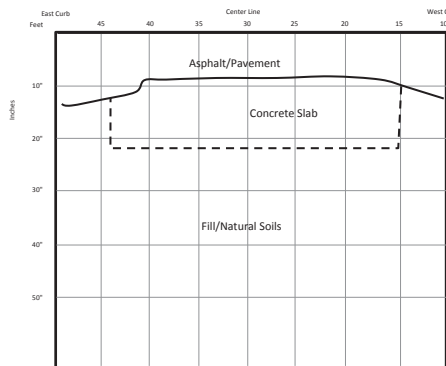
Cross- Section At
Sta: 0+60



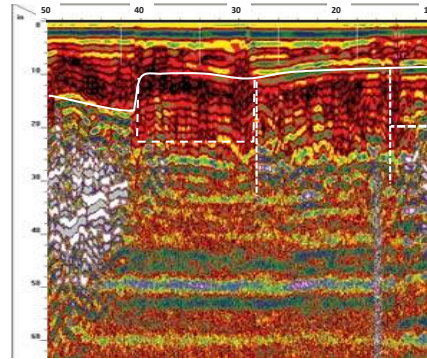
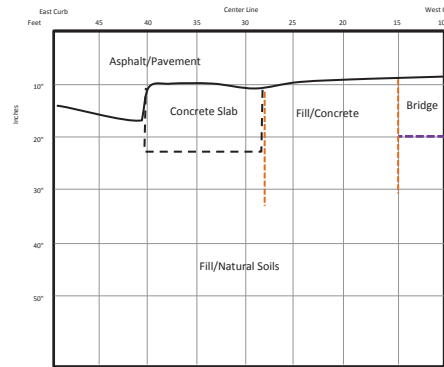
Cross- Section At
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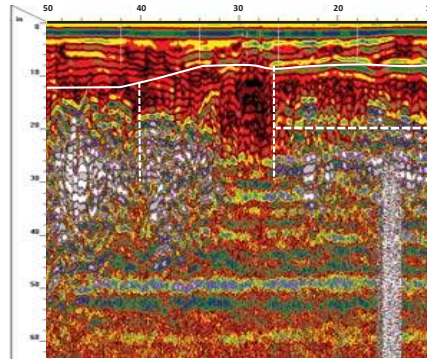
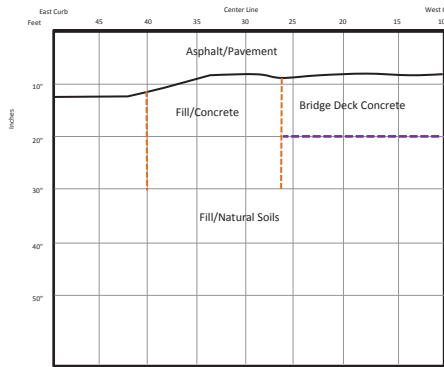
Cross- Section At
Sta: 1+00



Cross- Section At
Sta: 1+20



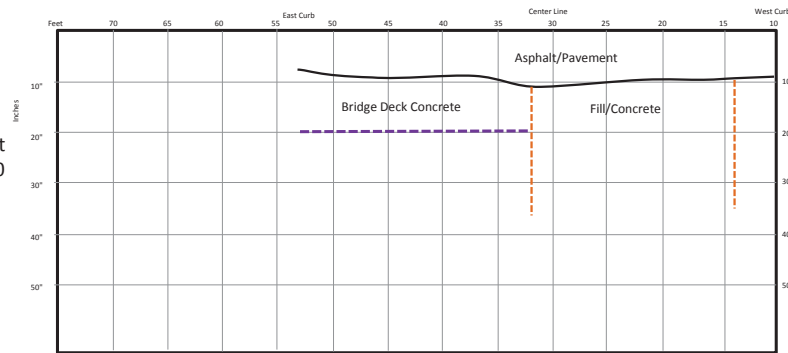
Cross- Section At
Sta: 1+40



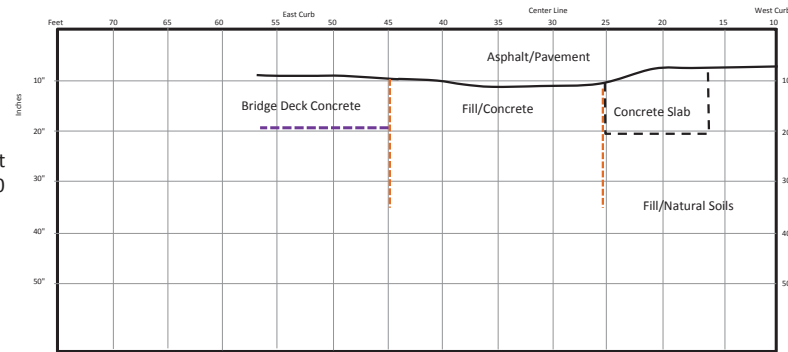
APPENDIX 3

Cross-sections and GPR Data
for cross-lines along the South Approach

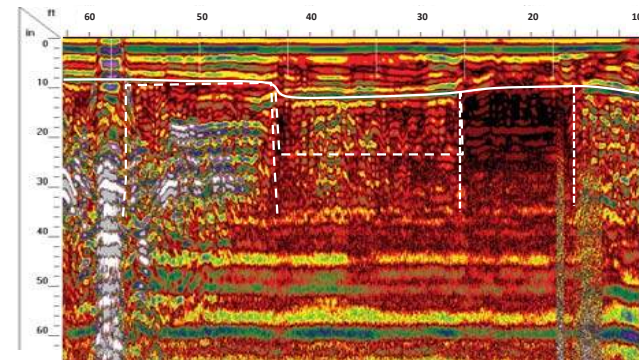
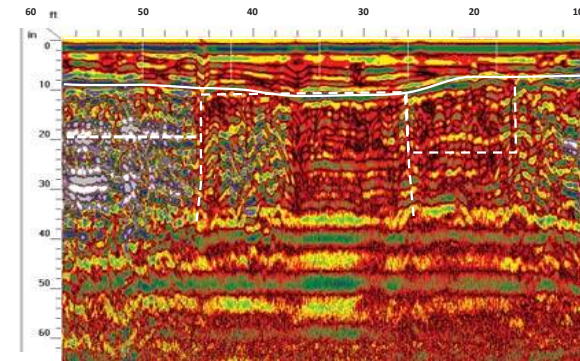
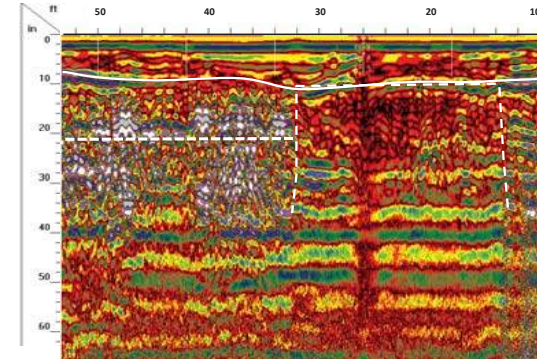
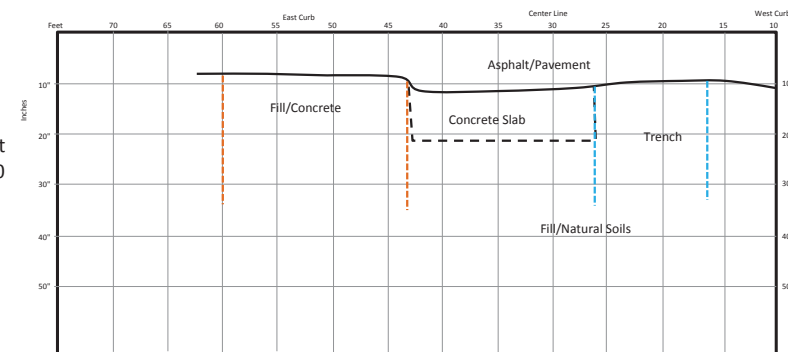
Cross- Section At
Sta: 2+00 - 2+00



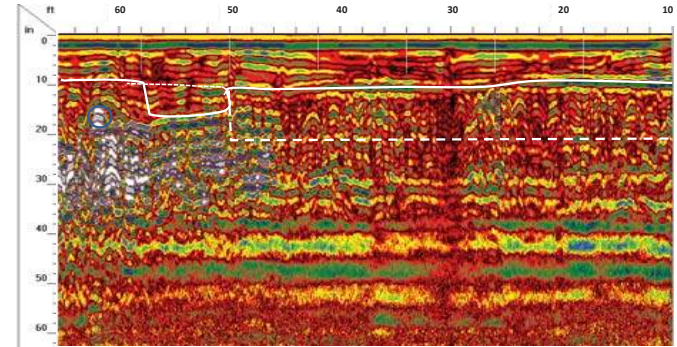
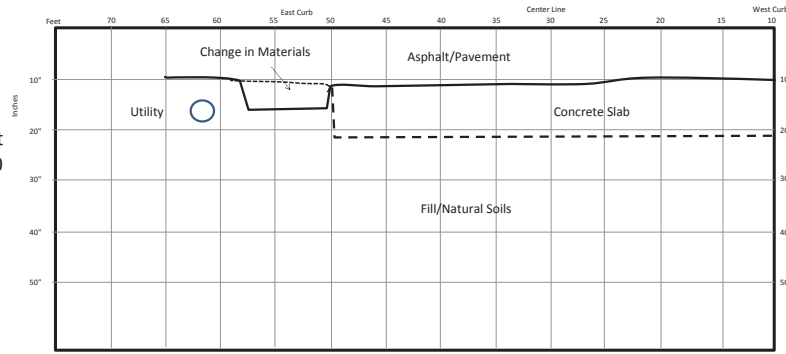
Cross- Section At
Sta: 2+20 - 2+20



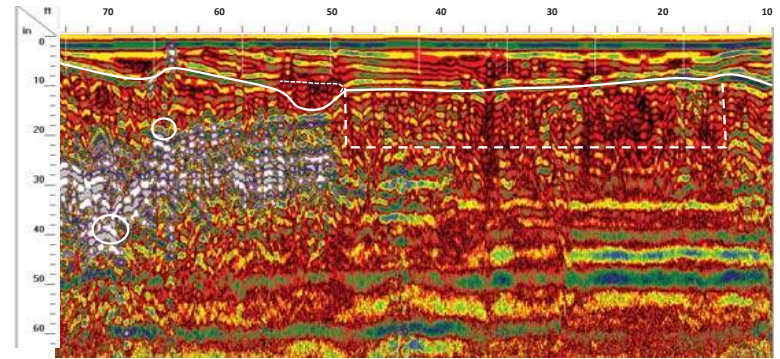
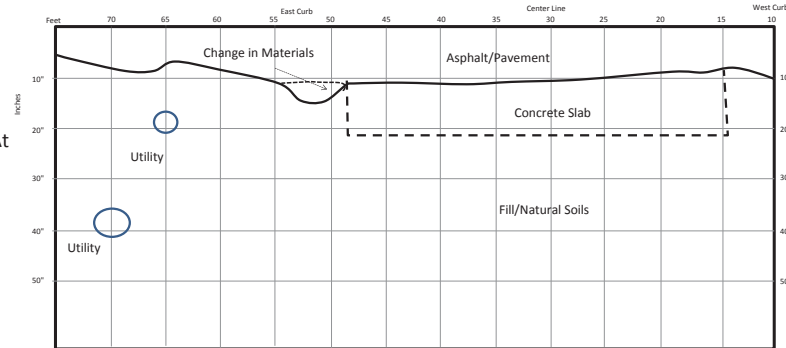
Cross- Section At
Sta: 2+40 - 2+40



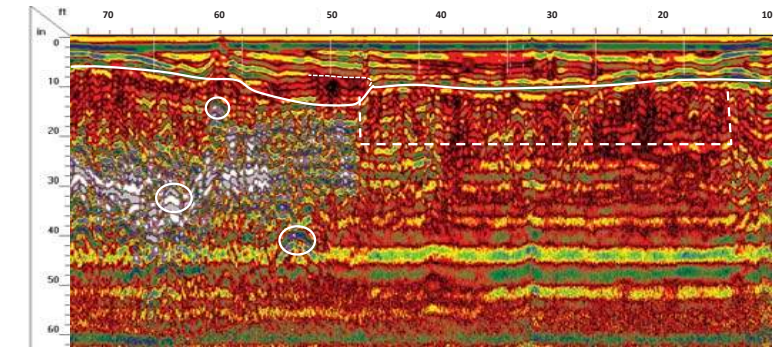
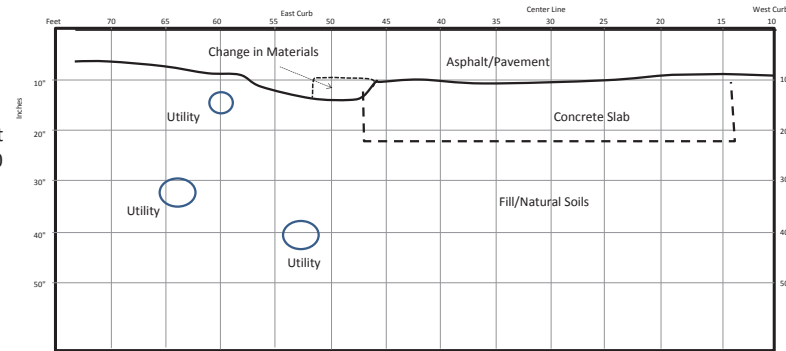
Cross- Section At
Sta: 2+60 - 2+60



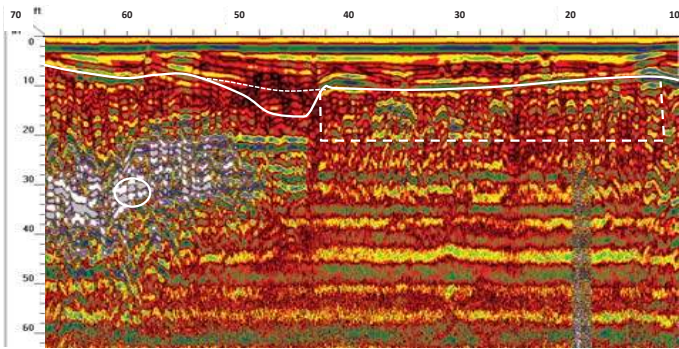
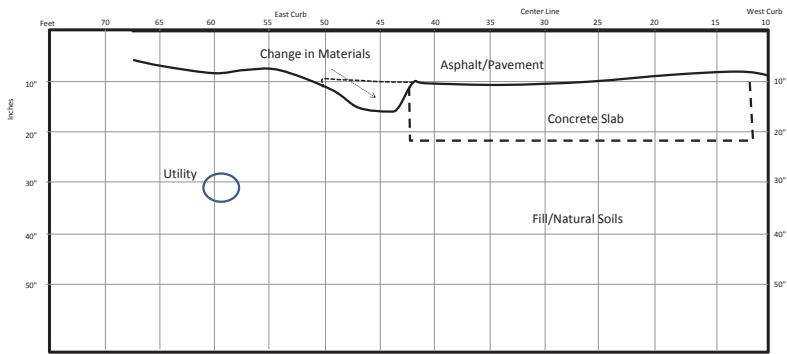
Cross- Section At



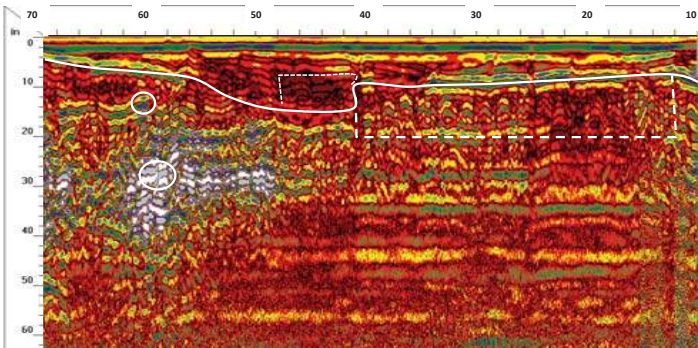
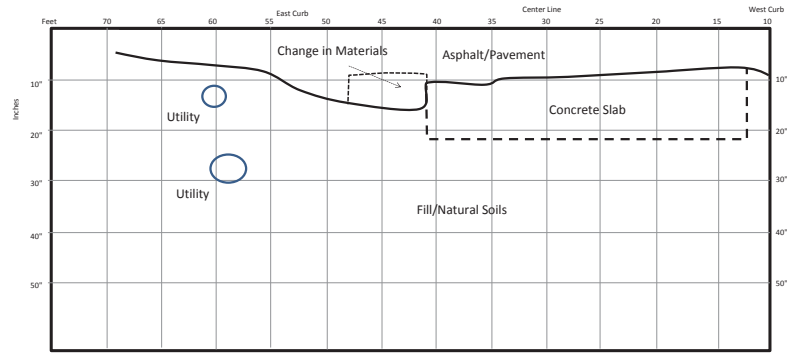
Cross- Section At
Sta: 2+80 - 3+00



Cross- Section At
Sta: 3+00 - 3+20



Cross- Section At
Sta: 3+20 - 3+40





10/18/2022

**RINES HILL BRIDGE NO. 3528 OVER OLD MAINE CENTRAL RAILROAD
GEOTECHNICAL DESIGN REPORT**

09.0025943.01

APPENDIX E – LABORATORY TEST RESULTS



Technologies to manage risk
for infrastructure

Boston
Atlanta
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Los Angeles
New York

www.geotesting.com

Transmittal

TO:

Andrew Blaisdell

GZA GeoEnvironmental, Inc.

477 Congress Street, Suite 700,

Portland, ME 04101

DATE: 10/5/2017

GTX NO: 306898

RE: Rines Hill Bridge Replacement

COPIES	DATE	DESCRIPTION
	10/5/2017	September 2017 Laboratory Test Report

REMARKS:

CC:

SIGNED:

Ethan Marro, Assistant Laboratory Manager

APPROVED BY:

Joe Tomei, Laboratory Manager

October 5, 2017

Andrew Blaisdell
GZA GeoEnvironmental, Inc.
477 Congress Street, Suite 700
Portland, ME 04101

RE: Rines Hill Bridge Replacement, Augusta, ME (GTX-306898)

Dear Andrew:

Enclosed are the test results you requested for the above referenced project. GeoTesting Express, Inc. (GTX) received 29 samples from you on 8/22/2017. These samples were labeled as follows:

Boring	Sample	Depth
BB-ARH-101	1D	1.7-3.7 ft
BB-ARH-101	2D	4.5-6.5 ft
BB-ARH-101	4D	14.5-16.5 ft
BB-ARH-101	5D	19.5-21.5 ft
BB-ARH-101	7D	23.5-25.5 ft
BB-ARH-101	10D	29.5-31.5 ft
BB-ARH-101	12D	39.5-41.5 ft
BB-ARH-101	16D	59.5-61.5 ft
BB-ARH-101	17D	64.5-66.5 ft
BB-ARH-102	7D	15-17 ft
BB-ARH-102	8D	20-22 ft
BB-ARH-102	10D	30-32 ft
BB-ARH-103	5D	8-10 ft
BB-ARH-103	9D	25-27 ft
BB-ARH-103	11D	35-37 ft
BB-ARH-103	12D	40-42 ft
BB-ARH-103	14D	50-52 ft
BB-ARH-104	1D	1.3-3.3 ft
BB-ARH-105	1D	1.5-3.5 ft
BB-ARH-105	5D	20-22 ft
BB-ARH-105	9D	34-36 ft
BB-ARH-105	11D	38-40 ft
BB-ARH-105	13D	50-52 ft
BB-ARH-105	17D	65-67 ft
BB-ARH-105	18D	70-72 ft
RB-ARH-101	5D	8-10 ft
RB-ARH-101	6D	10-12 ft
RB-ARH-101	9D	25-27 ft
RB-ARH-102	6D	25-27 ft

GTX performed the following tests on these samples:

- 27 ASTM D2216 - Moisture Contents
- 1 ASTM D2487 - Soil Classification
- 4 AASHTO M 145 - Soil Classifications
- 24 ASTM D422 - Grain Size Analyses - Sieve Only
- 2 ASTM D4318 - Atterberg Limits

A copy of your test request is attached.

The results presented in this report apply only to the items tested. This report shall not be reproduced except in full, without written approval from GeoTesting Express. The remainder of these samples will be retained for a period of sixty (60) days and will then be discarded unless otherwise notified by you. Please call me if you have any questions or require additional information. Thank you for allowing GeoTesting Express the opportunity of providing you with testing services. We look forward to working with you again in the future.

Respectfully yours,



Ethan Marro
Assistant Laboratory Manager



*Technologies to manage risk
for infrastructure*

Boston
Atlanta
Chicago
Los Angeles
New York

www.geotesting.com

Geotechnical Test Report

10/5/2017

GTX-306898

Rines Hill Bridge Replacement

Augusta, ME

Client Project No.: 09.0025943.00

Prepared for:

GZA GeoEnvironmental, Inc.



Client:	GZA GeoEnvironmental, Inc.		
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME	Project No:	GTX-306898
Boring ID:	BB-ARH-101	Sample Type:	bag
Sample ID:	7D	Test Date:	09/15/17
Depth :	23.5-25.5 ft	Test Id:	421619
Test Comment:	---		
Visual Description:	Moist, olive clay		
Sample Comment:	---		

USCS Classification - ASTM D2487

Boring ID	Sample ID	Depth	Group Name	Group Symbol	Gravel, %	Sand, %	Fines, %
BB-ARH-101	7D	23.5-25.5 ft	Lean clay	CL	0.0	1.9	98.1

Remarks: Grain Size analysis performed by ASTM D422 results enclosed
Atterberg Limits performed by ASTM D4318, results enclosed



Client: GZA GeoEnvironmental, Inc.
Project Name: Rines Hill Bridge Replacement
Project Location: Augusta, ME
GTX #: 306898
Date: 09/14/17

Soil Classification - AASHTO M 145

Boring ID	Sample ID	Depth, ft	Group Name	Group Symbol	Gravel, %	Sand, %	Fines, %
BB-ARH-101	1D	1.7-3.7	Stone Fragments, Gravel and Sand	A-1-b (0)	54	27	19
BB-ARH-103	5D	8-10	Stone fragments, Gravel and Sand	A-1-b (0)	50	36	14
BB-ARH-104	1D	1.3-3.3	Fine Sand	A-3 (1)	2	29	69
BB-ARH-105	1D	1.5-3.5	Silty Gravel and Sand	A-2-4 (0)	60	28	12

Remarks: Grain Size Analysis performed by ASTM D422, results enclosed
Atterberg Limits performed by ASTM D4318, results enclosed



Client:	GZA GeoEnvironmental, Inc.		
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME	Project No:	GTX-306898
Boring ID:	---	Sample Type:	---
Sample ID:	---	Test Date:	08/30/17
Depth :	---	Test Id:	421556
		Tested By:	jbr
		Checked By:	emm

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content, %
BB-ARH-101	1D	1.7-3.7 ft	Moist, dark grayish brown sandy gravel, little silt	4.4
BB-ARH-101	2D	4.5-6.5 ft	Moist, grayish brown clay	15.4
BB-ARH-101	4D	14.5-16.5 ft	Moist, olive clay	29.8
BB-ARH-101	5D	19.5-21.5 ft	Moist, olive clay	19.2
BB-ARH-101	7D	23.5-25.5 ft	Moist, olive clay	35.3
BB-ARH-101	10D	29.5-31.5 ft	Moist, olive gray silt, some sand, trace gravel	23.0
BB-ARH-101	12D	39.5-41.5 ft	Moist, dark brown sand, some gravel, little silt	8.9
BB-ARH-101	16D	59.5-61.5 ft	Moist, dark brown sand, little silt, trace gravel	14.6
BB-ARH-101	17D	64.5-66.5 ft	Moist, dark gray sand, little silt, little gravel	12.3

Notes: Temperature of Drying : 110° Celsius



Client:	GZA GeoEnvironmental, Inc.		
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME	Project No:	GTX-306898
Boring ID:	---	Sample Type:	---
Sample ID:	---	Test Date:	08/30/17
Depth :	---	Test Id:	421565
		Tested By:	jbr
		Checked By:	emm

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content, %
BB-ARH-102	7D	15-17 ft	Moist, dark brown sand, little silt, little gravel	10.4
BB-ARH-102	8D	20-22 ft	Moist, dark grayish brown sand, some gravel, little silt	9.6
BB-ARH-102	10D	30-32 ft	Moist, dark gray sandy gravel, little silt	7.0
BB-ARH-103	5D	8-10 ft	Moist, dark gray sand, some gravel, little silt	8.0
BB-ARH-103	9D	25-27 ft	Moist, dark yellowish brown sand, some silt, trace gravel	10.9
BB-ARH-103	11D	35-37 ft	Moist, dark gray gravelly sand, little silt	9.4
BB-ARH-103	14D	50-52 ft	Moist, dark brown gravelly sand, little silt	9.1
BB-ARH-104	1D	1.3-3.3 ft	Moist, light brownish gray clay, some sand, trace gravel	12.1

Notes: Temperature of Drying : 110° Celsius



Client:	GZA GeoEnvironmental, Inc.		
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME	Project No:	GTX-306898
Boring ID:	---	Sample Type:	---
Sample ID:	---	Test Date:	08/30/17
Depth :	---	Test Id:	421576
		Tested By:	jbr
		Checked By:	emm

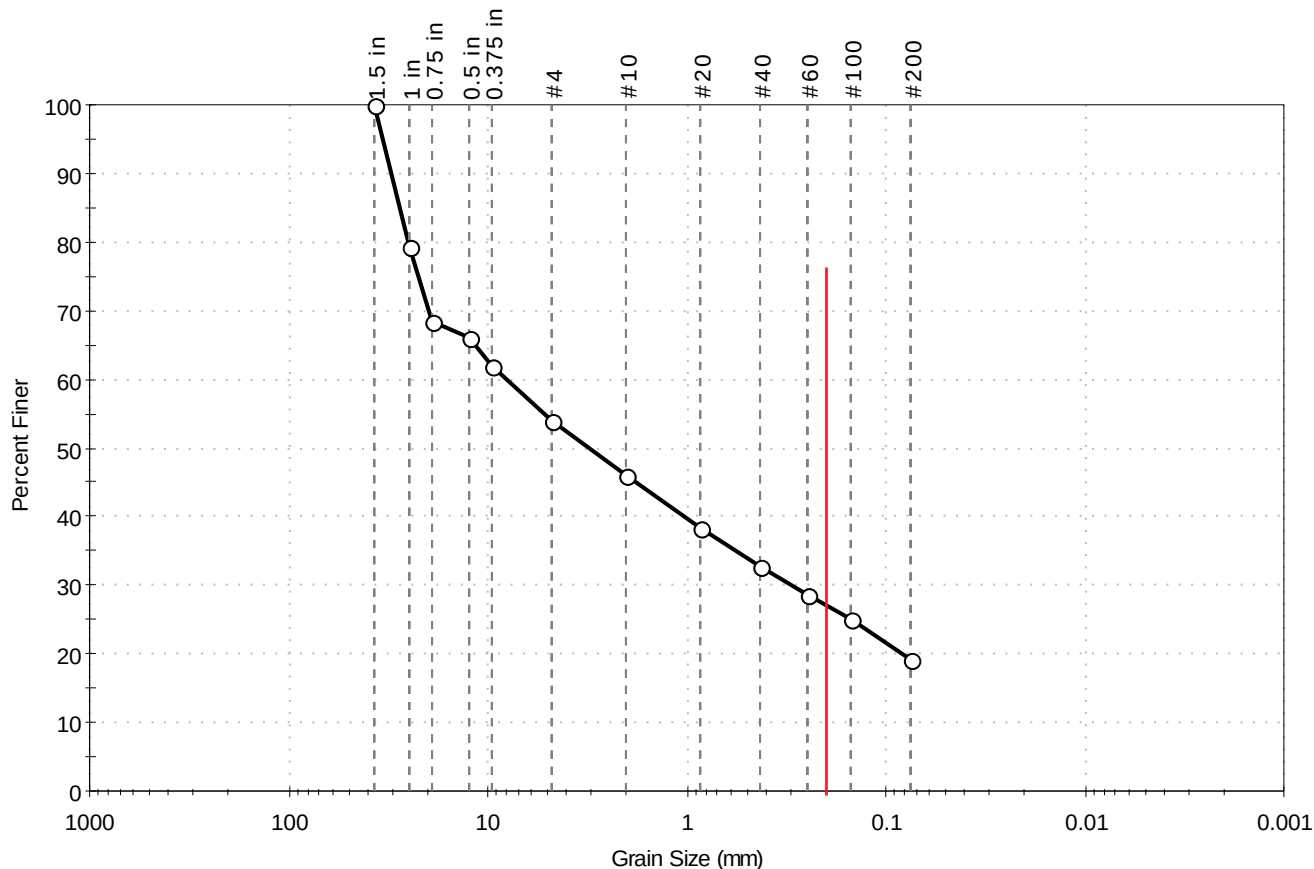
Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content, %
BB-ARH-105	1D	1.5-3.5 ft	Moist, dark grayish brown sandy gravel, little silt	3.1
BB-ARH-105	5D	20-22 ft	Moist, dark brown sand, some silt, trace gravel	11.7
BB-ARH-105	9D	34-36 ft	Moist, dark gray sand, some gravel, little silt	8.4
BB-ARH-105	11D	38-40 ft	Moist, dark brown gravel some sand, trace silt	4.7
BB-ARH-105	17D	65-67 ft	Moist, dark brownish gray sand, some silt	13.2
BB-ARH-105	18D	70-72 ft	Moist, olive brown sand, some silt	23.6
RB-ARH-101	5D	8-10 ft	Moist, olive clay	30.3
RB-ARH-101	6D	10-12 ft	Moist, yellowish brown gravelly sand, trace silt	12.3
RB-ARH-101	9D	25-27 ft	Moist, olive brown sand, some silt, some gravel	6.4
RB-ARH-102	6D	25-27 ft	Moist, gray sandy gravel, trace silt	4.1

Notes: Temperature of Drying : 110° Celsius

Client:	GZA GeoEnvironmental, Inc.		
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME	Project No:	GTX-306898
Boring ID:	BB-ARH-101	Sample Type:	bag
Sample ID:	1D	Test Date:	09/07/17
Depth:	1.7-3.7 ft	Test Id:	421584
Test Comment:	---		
Visual Description:	Moist, dark grayish brown sandy gravel, little silt		
Sample Comment:	MEDOT Modified Burmister description as requested		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	46.0	34.7	19.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	79		
0.75 in	19.00	68		
0.5 in	12.50	66		
0.375 in	9.50	62		
#4	4.75	54		
#10	2.00	46		
#20	0.85	38		
#40	0.42	33		
#60	0.25	29		
#100	0.15	25		
#200	0.075	19		

Coefficients

D ₈₅ = 27.9484 mm	D ₃₀ = 0.2960 mm
D ₆₀ = 7.9780 mm	D ₁₅ = N/A
D ₅₀ = 3.0977 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

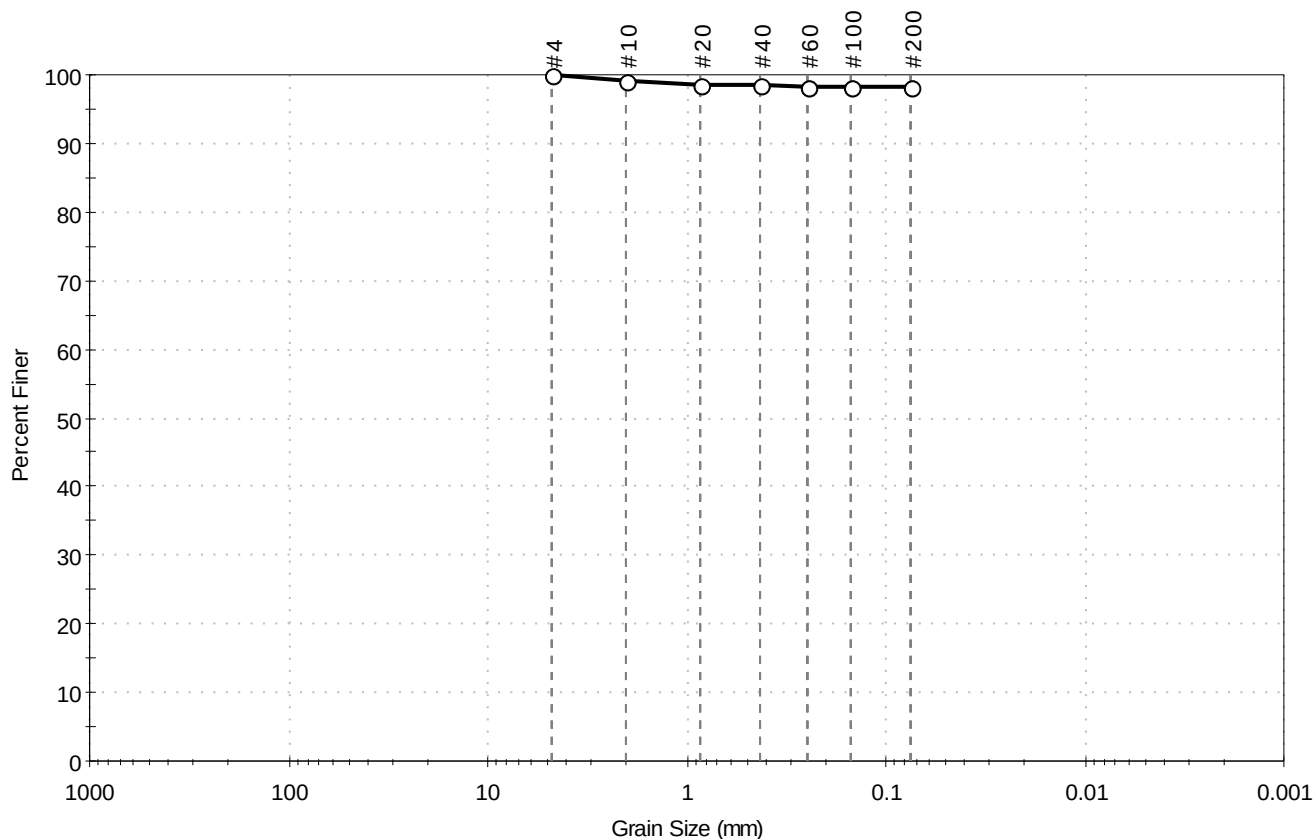
Sample / Test Description

Sand/Gravel Particle Shape : ANGULAR
Sand/Gravel Hardness : HARD



Client:	GZA GeoEnvironmental, Inc.		
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME	Project No:	GTX-306898
Boring ID:	BB-ARH-101	Sample Type:	bag
Sample ID:	7D	Test Date:	09/08/17
Depth :	23.5-25.5 ft	Test Id:	421621
Test Comment:	---		
Visual Description:	Moist, olive clay		
Sample Comment:	MEDOT Modified Burmister description as requested		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	0.0	1.9	98.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	99		
#20	0.85	99		
#40	0.42	98		
#60	0.25	98		
#100	0.15	98		
#200	0.075	98		

Coefficients

D ₈₅ = N/A	D ₃₀ = N/A
D ₆₀ = N/A	D ₁₅ = N/A
D ₅₀ = N/A	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM Lean clay (CL)

AASHTO Clayey Soils (A-6 (22))

Sample / Test Description

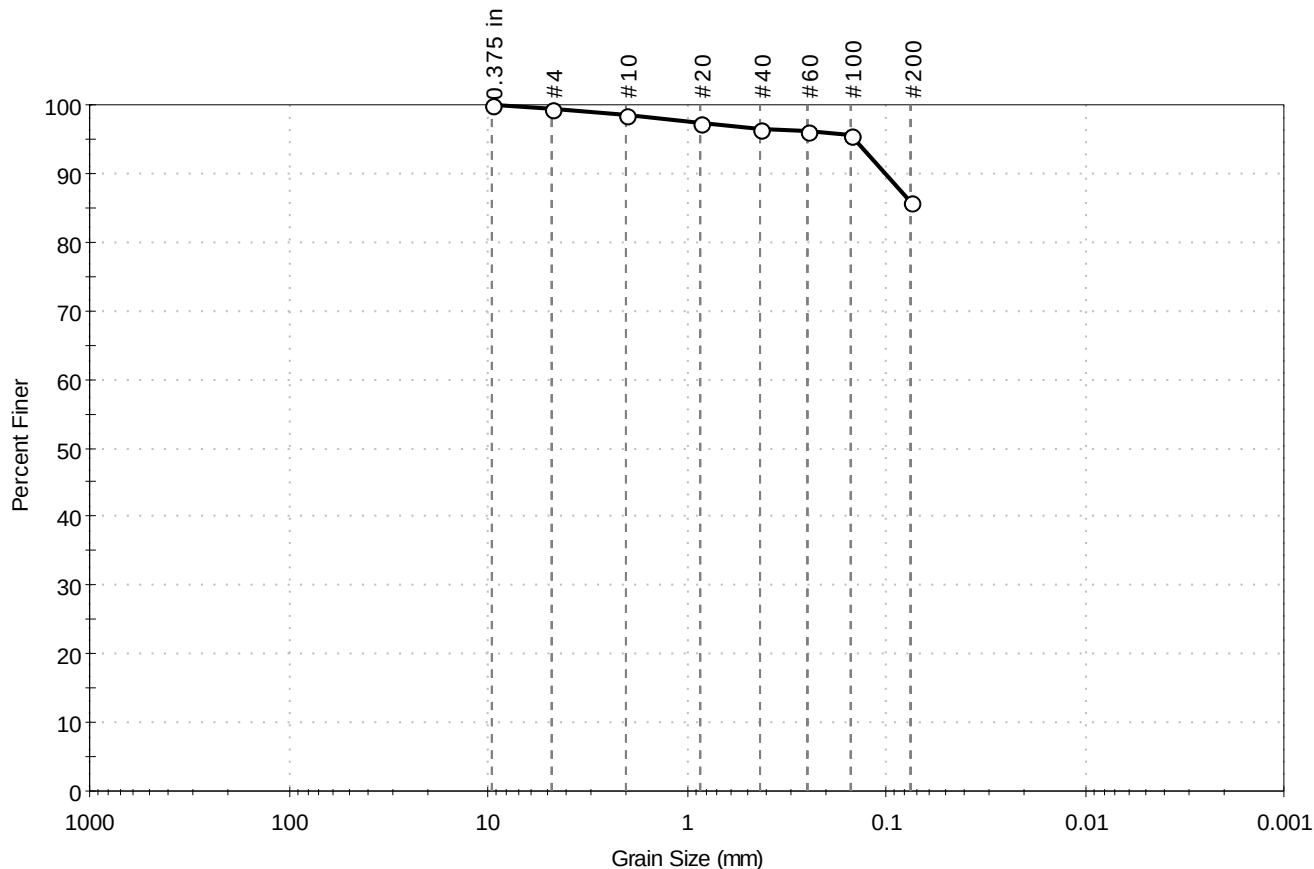
Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---



Client:	GZA GeoEnvironmental, Inc.	Project No:	GTX-306898
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME		
Boring ID:	BB-ARH-101	Sample Type:	bag
Sample ID:	10D	Test Date:	09/07/17
Depth :	29.5-31.5 ft	Test Id:	421585
Test Comment:	---		
Visual Description:	Moist, olive gray silt, some sand, trace gravel		
Sample Comment:	MEDOT Modified Burmister description as requested		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	0.5	13.6	85.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	99		
#10	2.00	98		
#20	0.85	97		
#40	0.42	97		
#60	0.25	96		
#100	0.15	96		
#200	0.075	86		

Coefficients

D ₈₅ = N/A	D ₃₀ = N/A
D ₆₀ = N/A	D ₁₅ = N/A
D ₅₀ = N/A	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample / Test Description

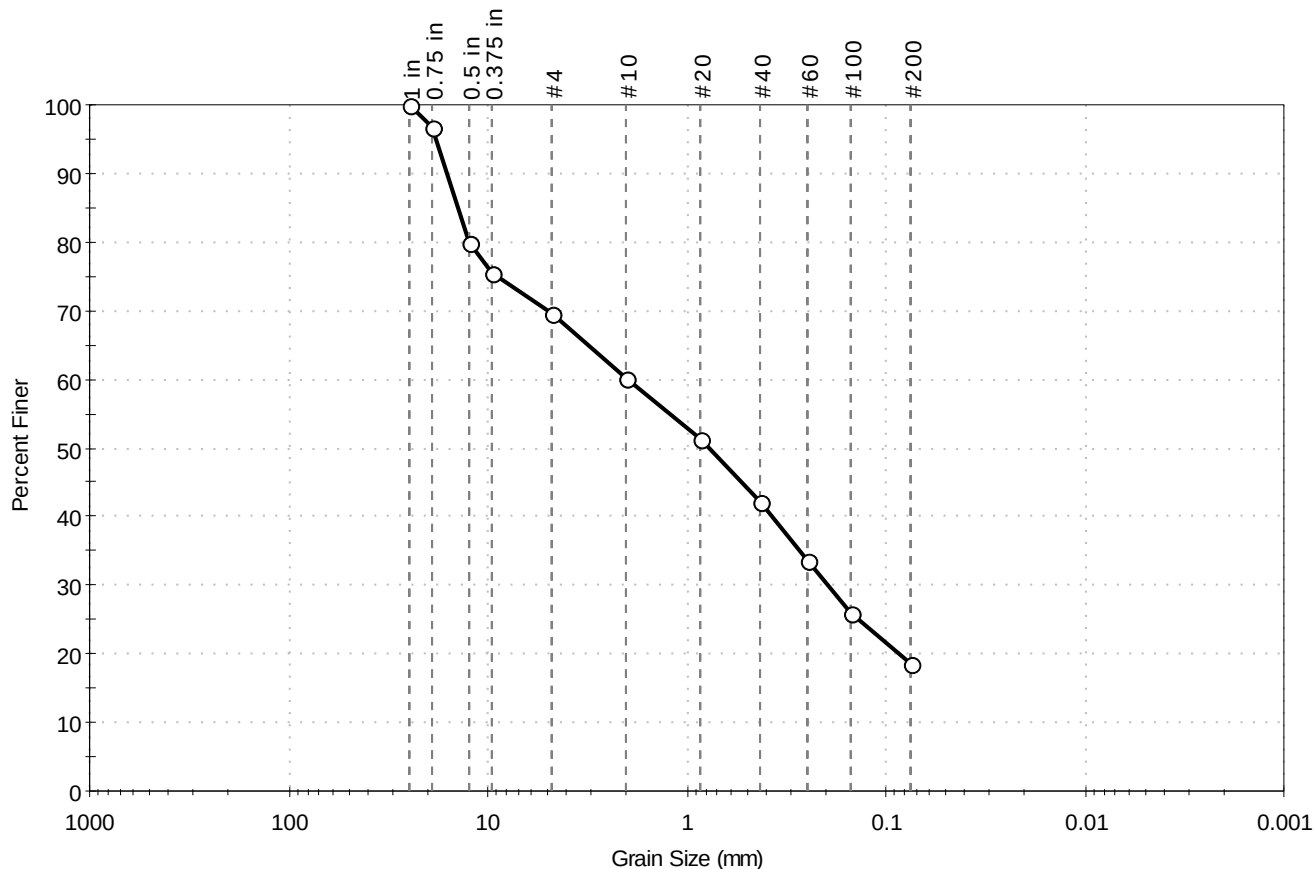
Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---



Client:	GZA GeoEnvironmental, Inc.		
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME	Project No:	GTX-306898
Boring ID:	BB-ARH-101	Sample Type:	bag
Sample ID:	12D	Test Date:	09/08/17
Depth :	39.5-41.5 ft	Test Id:	421586
Test Comment:	---		
Visual Description:	Moist, dark brown sand, some gravel, little silt		
Sample Comment:	MEDOT Modified Burmister description as requested		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	30.3	51.2	18.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	97		
0.5 in	12.50	80		
0.375 in	9.50	75		
#4	4.75	70		
#10	2.00	60		
#20	0.85	51		
#40	0.42	42		
#60	0.25	34		
#100	0.15	26		
#200	0.075	18		

Coefficients

D ₈₅ = 14.1460 mm	D ₃₀ = 0.1974 mm
D ₆₀ = 1.9488 mm	D ₁₅ = N/A
D ₅₀ = 0.7741 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

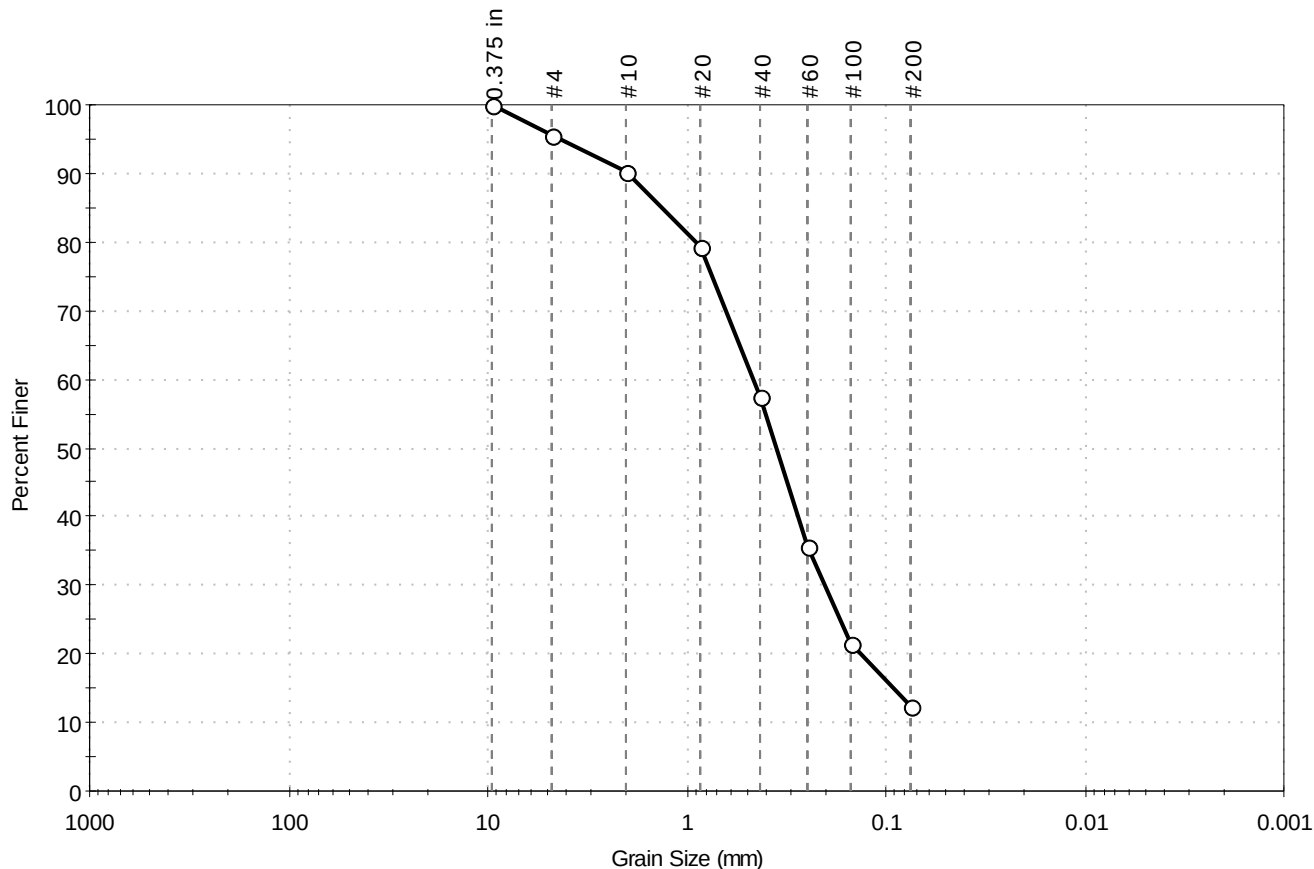
Sample / Test Description

Sand/Gravel Particle Shape : ANGULAR
Sand/Gravel Hardness : HARD



Client:	GZA GeoEnvironmental, Inc.		
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME	Project No:	GTX-306898
Boring ID:	BB-ARH-101	Sample Type:	bag
Sample ID:	16D	Test Date:	09/08/17
Depth :	59.5-61.5 ft	Test Id:	421587
Test Comment:	---		
Visual Description:	Moist, dark brown sand, little silt, trace gravel		
Sample Comment:	MEDOT Modified Burmister description as requested		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	4.6	83.0	12.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	95		
#10	2.00	90		
#20	0.85	79		
#40	0.42	58		
#60	0.25	36		
#100	0.15	22		
#200	0.075	12		

Coefficients

D ₈₅ = 1.3239 mm	D ₃₀ = 0.2030 mm
D ₆₀ = 0.4598 mm	D ₁₅ = 0.0914 mm
D ₅₀ = 0.3535 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

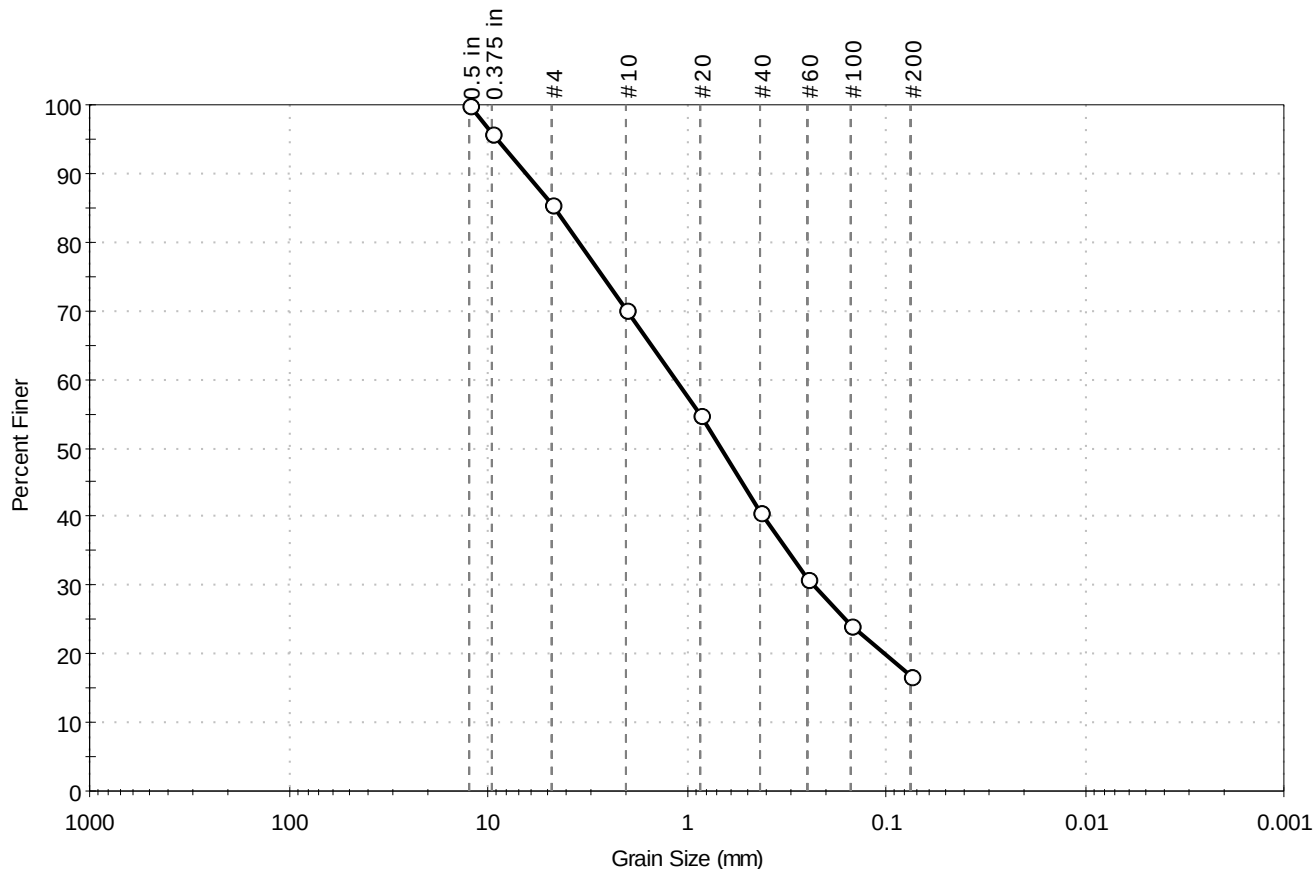
Sample / Test Description

Sand/Gravel Particle Shape : ANGULAR
Sand/Gravel Hardness : HARD



Client:	GZA GeoEnvironmental, Inc.		
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME	Project No:	GTX-306898
Boring ID:	BB-ARH-101	Sample Type:	bag
Sample ID:	17D	Test Date:	09/08/17
Depth :	64.5-66.5 ft	Test Id:	421588
Test Comment:	---		
Visual Description:	Moist, dark gray sand, little silt, little gravel		
Sample Comment:	MEDOT Modified Burmister description as requested		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	14.4	68.7	16.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	96		
#4	4.75	86		
#10	2.00	70		
#20	0.85	55		
#40	0.42	41		
#60	0.25	31		
#100	0.15	24		
#200	0.075	17		

Coefficients

D ₈₅ = 4.6019 mm	D ₃₀ = 0.2321 mm
D ₆₀ = 1.1328 mm	D ₁₅ = N/A
D ₅₀ = 0.6677 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

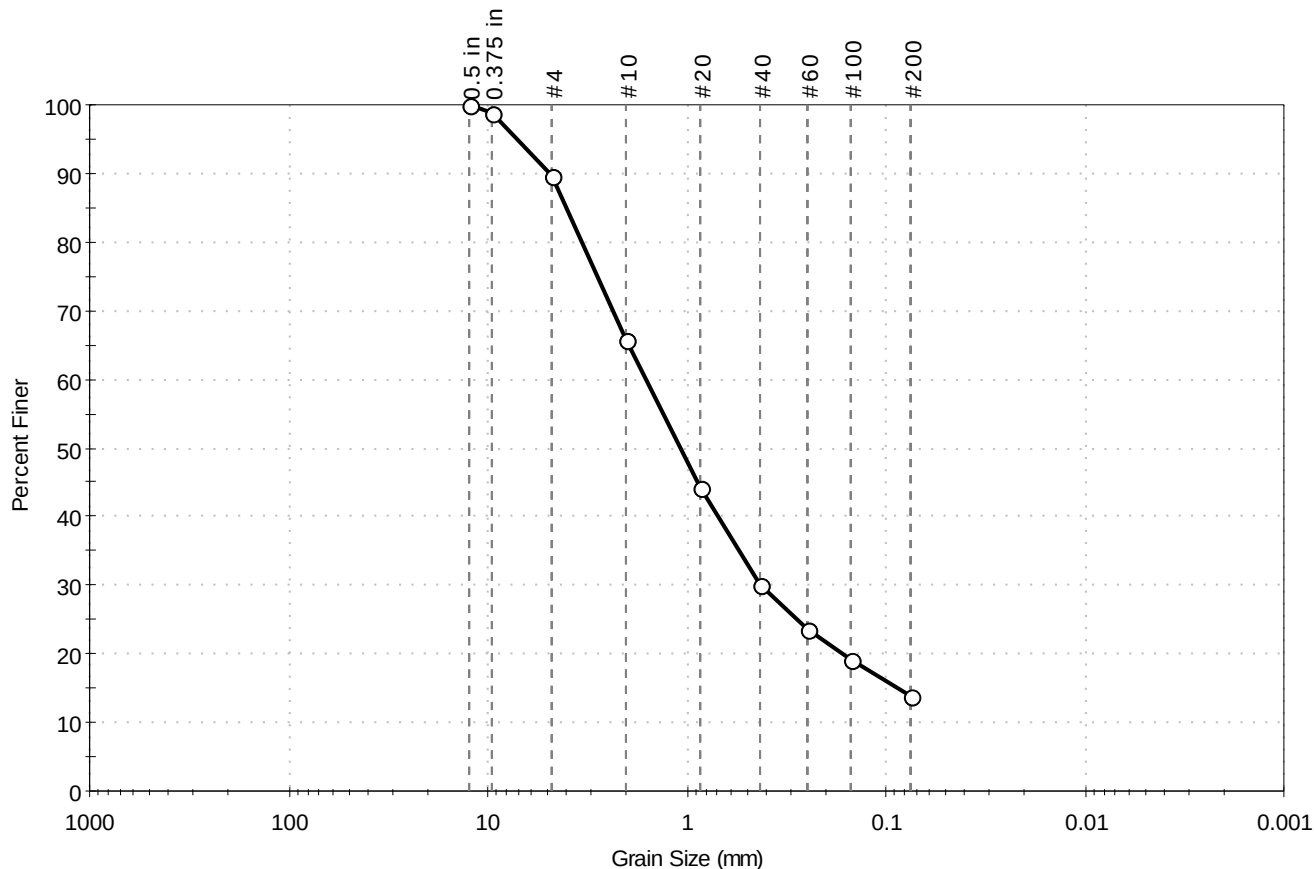
Sample / Test Description

Sand/Gravel Particle Shape : ANGULAR
Sand/Gravel Hardness : HARD



Client:	GZA GeoEnvironmental, Inc.		
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME	Project No:	GTX-306898
Boring ID:	BB-ARH-102	Sample Type:	bag
Sample ID:	7D	Test Date:	09/07/17
Depth :	15-17 ft	Test Id:	421589
Test Comment:	---		
Visual Description:	Moist, dark brown sand, little silt, little gravel		
Sample Comment:	MEDOT Modified Burmister description as requested		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	10.4	75.8	13.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	99		
#4	4.75	90		
#10	2.00	66		
#20	0.85	44		
#40	0.42	30		
#60	0.25	24		
#100	0.15	19		
#200	0.075	14		

Coefficients

D ₈₅ = 4.0223 mm	D ₃₀ = 0.4202 mm
D ₆₀ = 1.5853 mm	D ₁₅ = 0.0878 mm
D ₅₀ = 1.0660 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample / Test Description

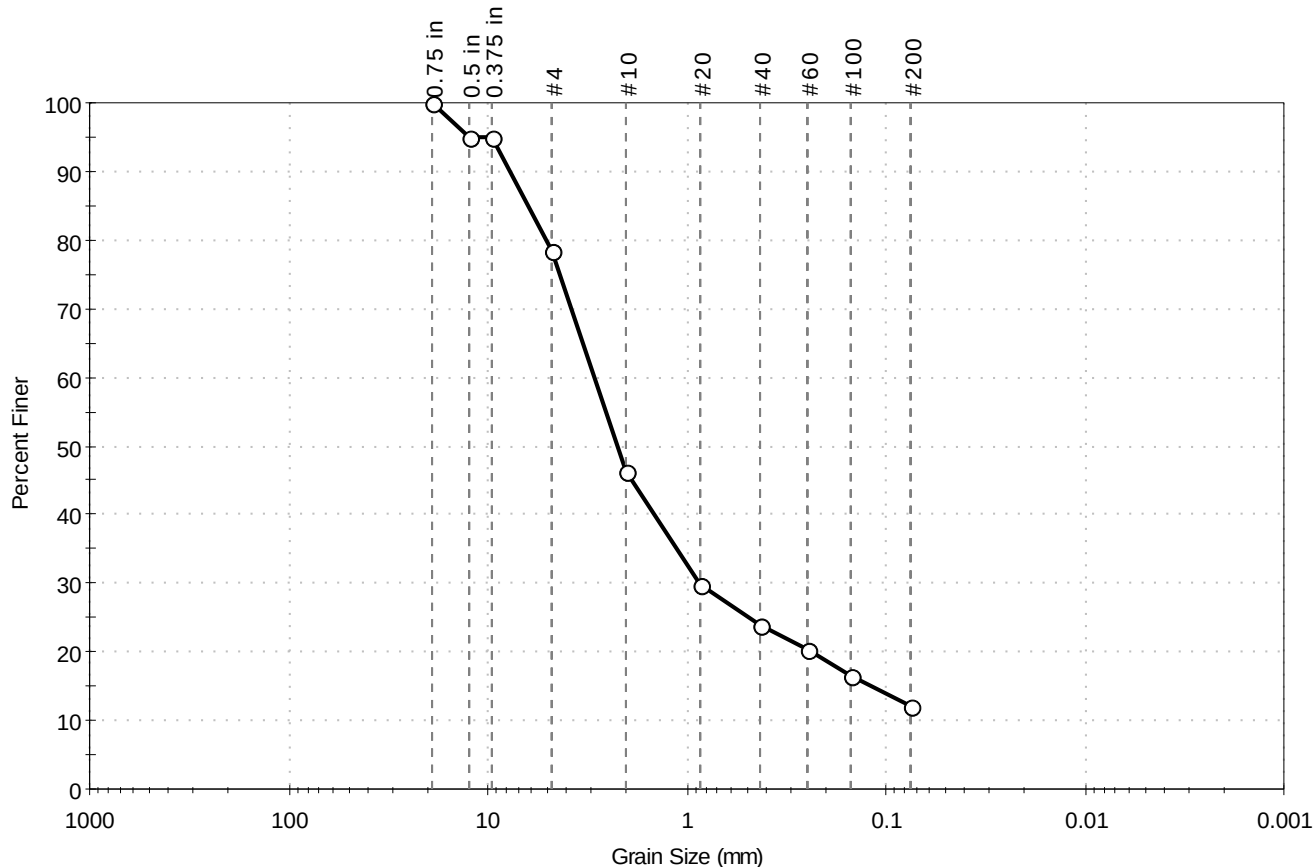
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD



Client:	GZA GeoEnvironmental, Inc.		
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME	Project No:	GTX-306898
Boring ID:	BB-ARH-102	Sample Type:	bag
Sample ID:	8D	Test Date:	09/06/17
Depth :	20-22 ft	Test Id:	421590
Test Comment:	---		
Visual Description:	Moist, dark grayish brown sand, some gravel, little silt		
Sample Comment:	MEDOT Modified Burmister description as requested		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	21.6	66.3	12.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	95		
0.375 in	9.50	95		
#4	4.75	78		
#10	2.00	46		
#20	0.85	30		
#40	0.42	24		
#60	0.25	20		
#100	0.15	17		
#200	0.075	12		

Coefficients

D ₈₅ = 6.2605 mm	D ₃₀ = 0.8650 mm
D ₆₀ = 2.8883 mm	D ₁₅ = 0.1164 mm
D ₅₀ = 2.2045 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

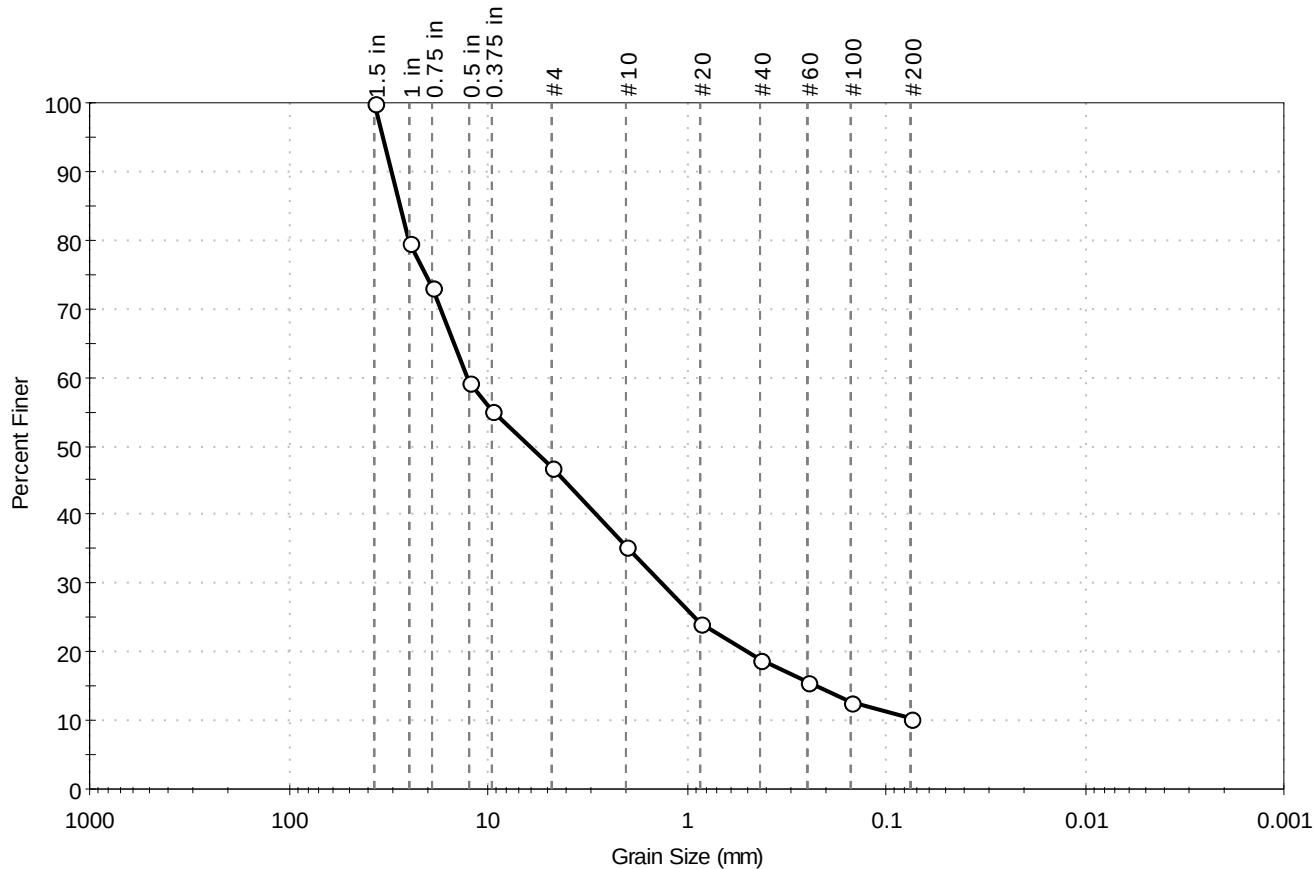
AASHTO Stone Fragments, Gravel and Sand (A-1-a (0))

Sample / Test Description

Sand/Gravel Particle Shape : ANGULAR
Sand/Gravel Hardness : HARD

Client:	GZA GeoEnvironmental, Inc.		
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME	Project No:	GTX-306898
Boring ID:	BB-ARH-102	Sample Type:	bag
Sample ID:	10D	Test Date:	09/08/17
Depth :	30-32 ft	Test Id:	421591
Test Comment:	---		
Visual Description:	Moist, dark gray sandy gravel, little silt		
Sample Comment:	MEDOT Modified Burmister description as requested		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	53.3	36.5	10.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	80		
0.75 in	19.00	73		
0.5 in	12.50	59		
0.375 in	9.50	55		
#4	4.75	47		
#10	2.00	35		
#20	0.85	24		
#40	0.42	19		
#60	0.25	16		
#100	0.15	13		
#200	0.075	10		

Coefficients

D ₈₅ = 27.8020 mm	D ₃₀ = 1.3215 mm
D ₆₀ = 12.7853 mm	D ₁₅ = 0.2274 mm
D ₅₀ = 6.1809 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-a (0))

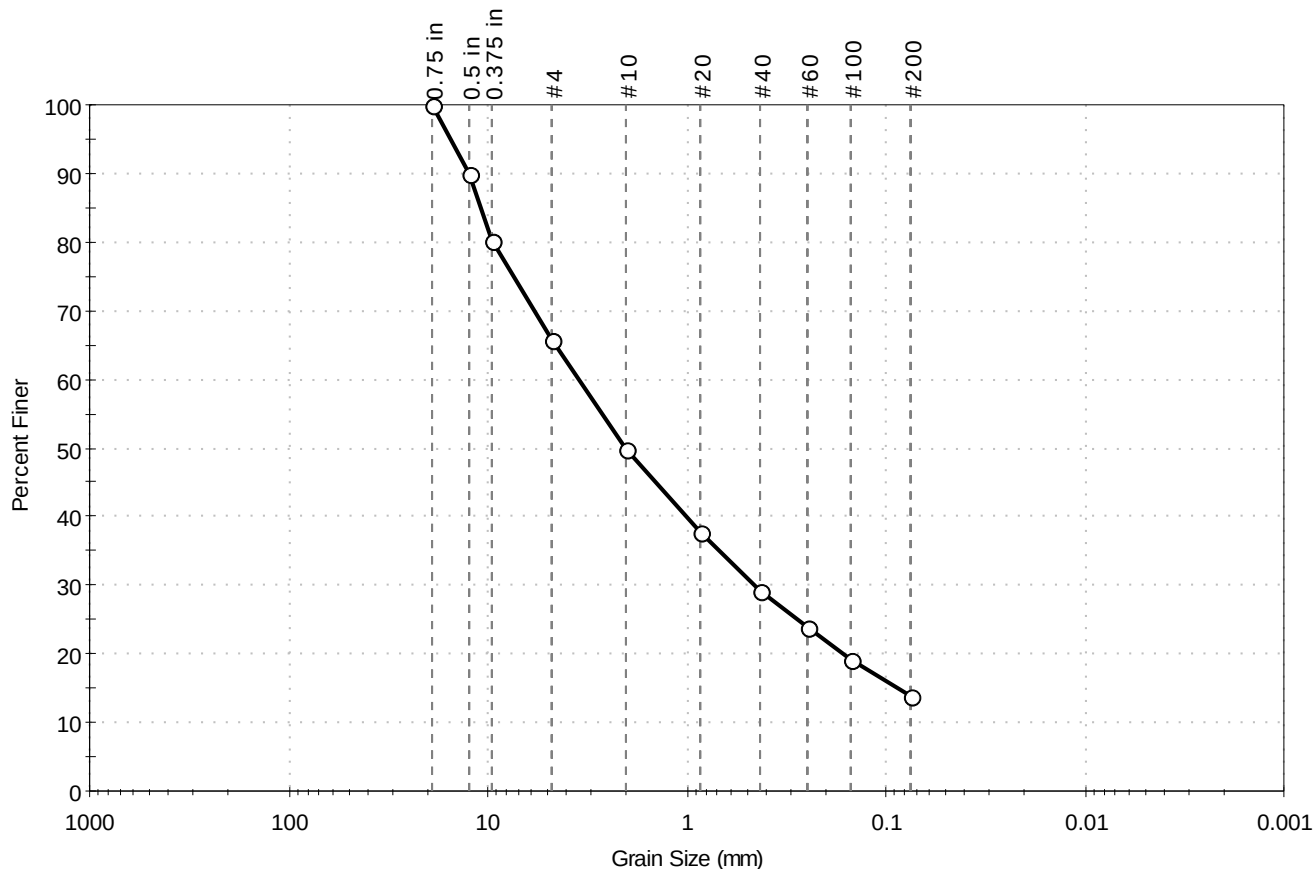
Sample / Test Description

Sand/Gravel Particle Shape : ANGULAR
Sand/Gravel Hardness : HARD



Client:	GZA GeoEnvironmental, Inc.	Project No:	GTX-306898
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME		
Boring ID:	BB-ARH-103	Sample Type:	bag
Sample ID:	5D	Test Date:	09/08/17
Depth :	8-10 ft	Test Id:	421592
Test Comment:	---		
Visual Description:	Moist, dark gray sand, some gravel, little silt		
Sample Comment:	MEDOT Modified Burmister description as requested		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	34.1	52.0	13.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	90		
0.375 in	9.50	80		
#4	4.75	66		
#10	2.00	50		
#20	0.85	38		
#40	0.42	29		
#60	0.25	24		
#100	0.15	19		
#200	0.075	14		

Coefficients

D ₈₅ = 10.8686 mm	D ₃₀ = 0.4570 mm
D ₆₀ = 3.4636 mm	D ₁₅ = 0.0865 mm
D ₅₀ = 2.0215 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-a (0))

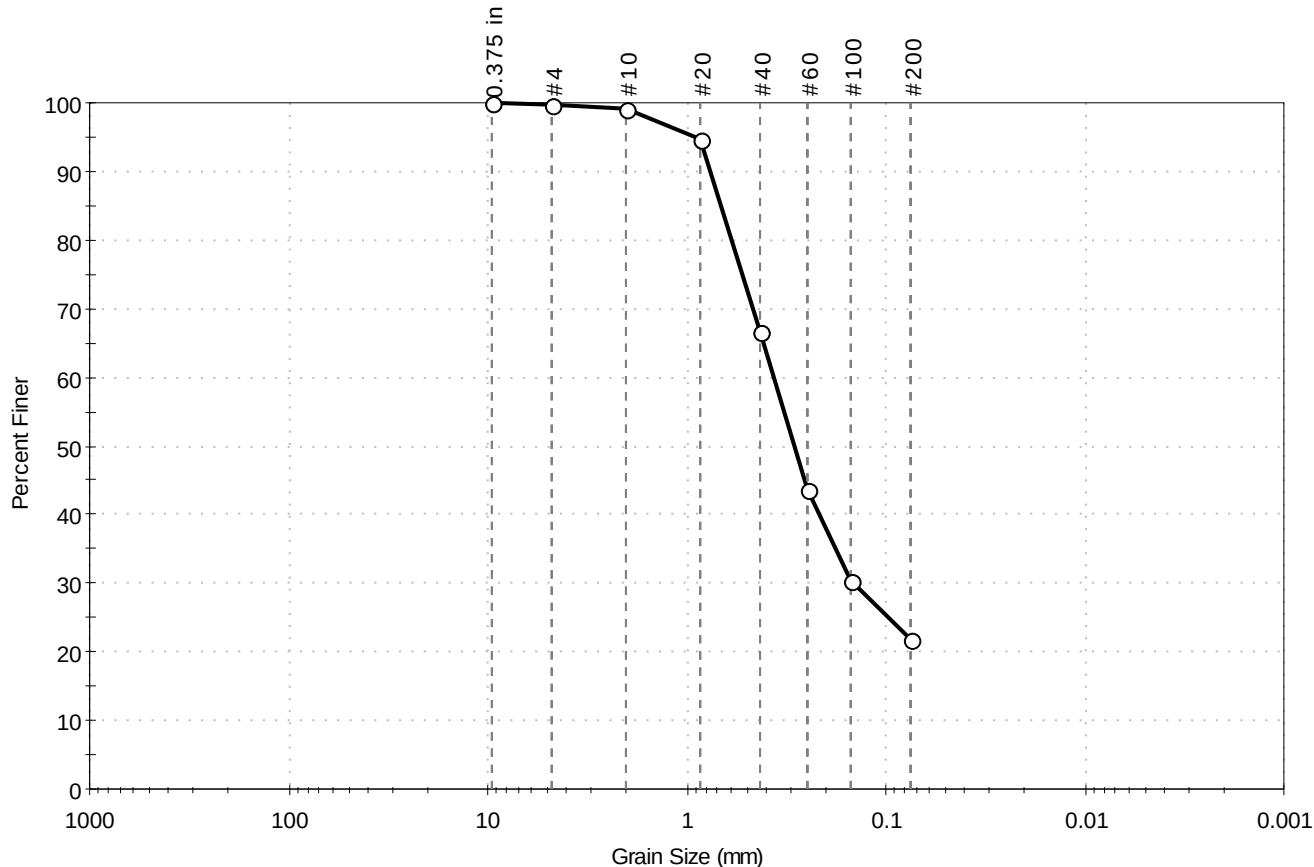
Sample / Test Description

Sand/Gravel Particle Shape : ANGULAR
Sand/Gravel Hardness : HARD



Client:	GZA GeoEnvironmental, Inc.	Project No:	GTX-306898
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME		
Boring ID:	BB-ARH-103	Sample Type:	bag
Sample ID:	9D	Test Date:	09/06/17
Depth :	25-27 ft	Test Id:	421593
Test Comment:	---		
Visual Description:	Moist, dark yellowish brown sand, some silt, trace gravel		
Sample Comment:	MEDOT Modified Burmister description as requested		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	0.3	77.9	21.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	100		
#10	2.00	99		
#20	0.85	95		
#40	0.42	67		
#60	0.25	44		
#100	0.15	30		
#200	0.075	22		

Coefficients

D ₈₅ = 0.6705 mm	D ₃₀ = 0.1450 mm
D ₆₀ = 0.3645 mm	D ₁₅ = N/A
D ₅₀ = 0.2893 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

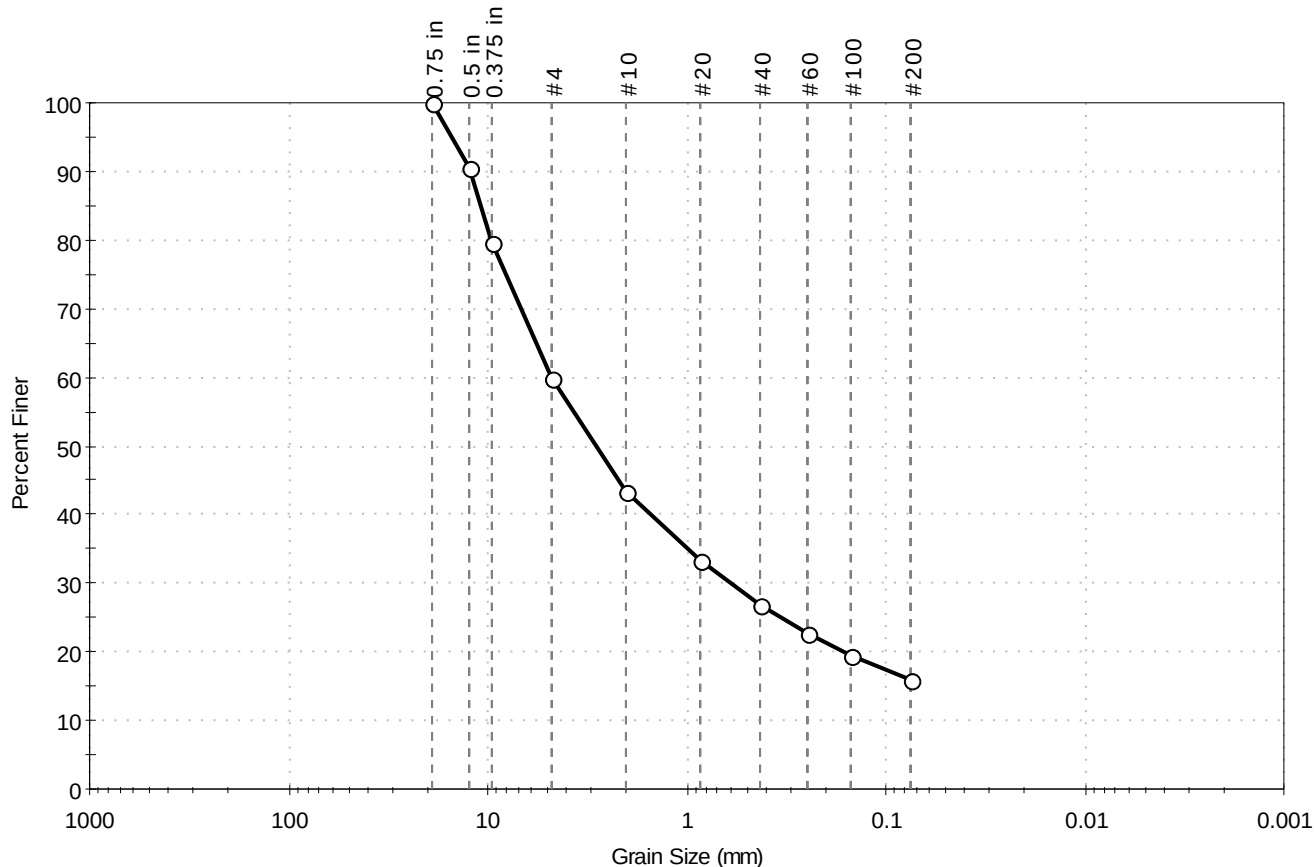
Sample / Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Client:	GZA GeoEnvironmental, Inc.		
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME	Project No:	GTX-306898
Boring ID:	BB-ARH-103	Sample Type:	bag
Sample ID:	11D	Test Date:	09/08/17
Depth :	35-37 ft	Test Id:	421594
Test Comment:	---		
Visual Description:	Moist, dark gray gravelly sand, little silt		
Sample Comment:	MEDOT Modified Burmister description as requested		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	40.3	43.7	16.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	90		
0.375 in	9.50	80		
#4	4.75	60		
#10	2.00	43		
#20	0.85	33		
#40	0.42	27		
#60	0.25	23		
#100	0.15	19		
#200	0.075	16		

Coefficients

D ₈₅ = 10.8795 mm	D ₃₀ = 0.6003 mm
D ₆₀ = 4.7937 mm	D ₁₅ = N/A
D ₅₀ = 2.8374 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

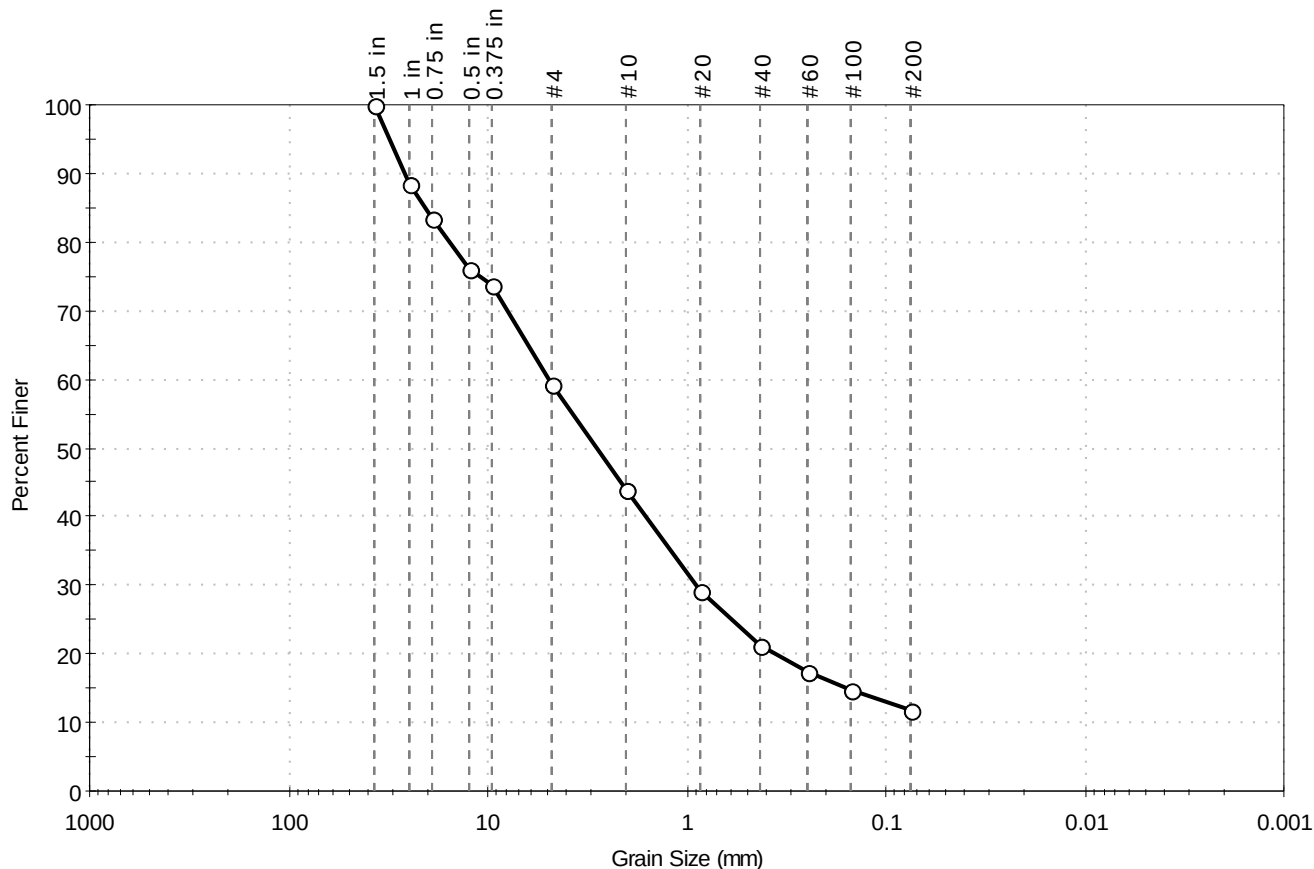
Sample / Test Description

Sand/Gravel Particle Shape : ANGULAR
Sand/Gravel Hardness : HARD



Client:	GZA GeoEnvironmental, Inc.		
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME	Project No:	GTX-306898
Boring ID:	BB-ARH-103	Sample Type:	bag
Sample ID:	14D	Test Date:	09/08/17
Depth :	50-52 ft	Test Id:	421596
Test Comment:	---		
Visual Description:	Moist, dark brown gravelly sand, little silt		
Sample Comment:	MEDOT Modified Burmister description as requested		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	40.6	47.6	11.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	88		
0.75 in	19.00	84		
0.5 in	12.50	76		
0.375 in	9.50	74		
#4	4.75	59		
#10	2.00	44		
#20	0.85	29		
#40	0.42	21		
#60	0.25	18		
#100	0.15	15		
#200	0.075	12		

Coefficients

D ₈₅ = 20.5439 mm	D ₃₀ = 0.8867 mm
D ₆₀ = 4.8892 mm	D ₁₅ = 0.1593 mm
D ₅₀ = 2.8151 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-a (0))

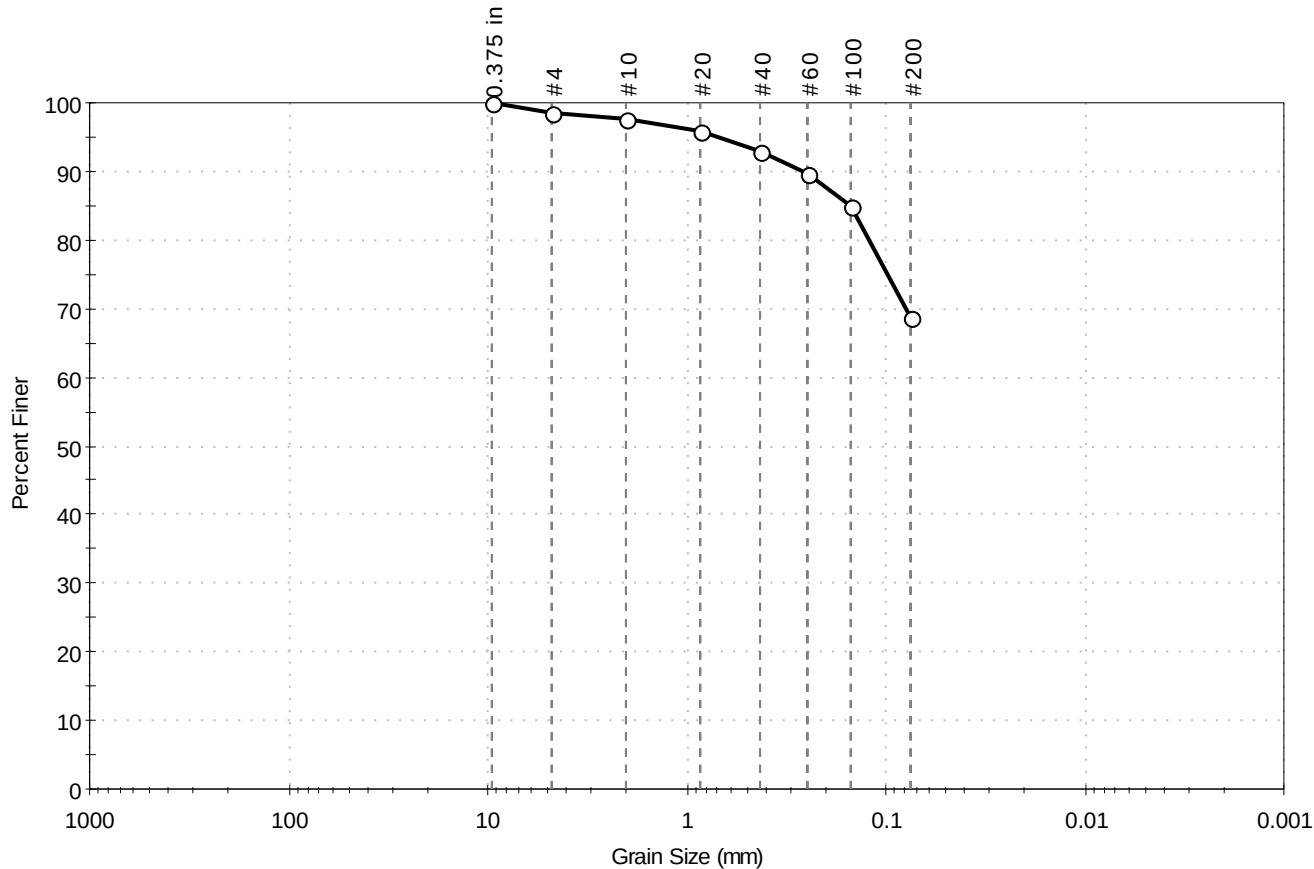
Sample / Test Description

Sand/Gravel Particle Shape : ANGULAR
Sand/Gravel Hardness : HARD



Client:	GZA GeoEnvironmental, Inc.	Project No:	GTX-306898
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME		
Boring ID:	BB-ARH-104	Sample Type:	bag
Sample ID:	1D	Test Date:	09/08/17
Depth :	1.3-3.3 ft	Test Id:	421597
Test Comment:	---		
Visual Description:	Moist, light brownish gray clay, some sand, trace gravel		
Sample Comment:	MEDOT Modified Burmister description as requested		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	1.5	29.8	68.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	99		
#10	2.00	98		
#20	0.85	96		
#40	0.42	93		
#60	0.25	90		
#100	0.15	85		
#200	0.075	69		

Coefficients

D ₈₅ = 0.1527 mm	D ₃₀ = N/A
D ₆₀ = N/A	D ₁₅ = N/A
D ₅₀ = N/A	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample / Test Description

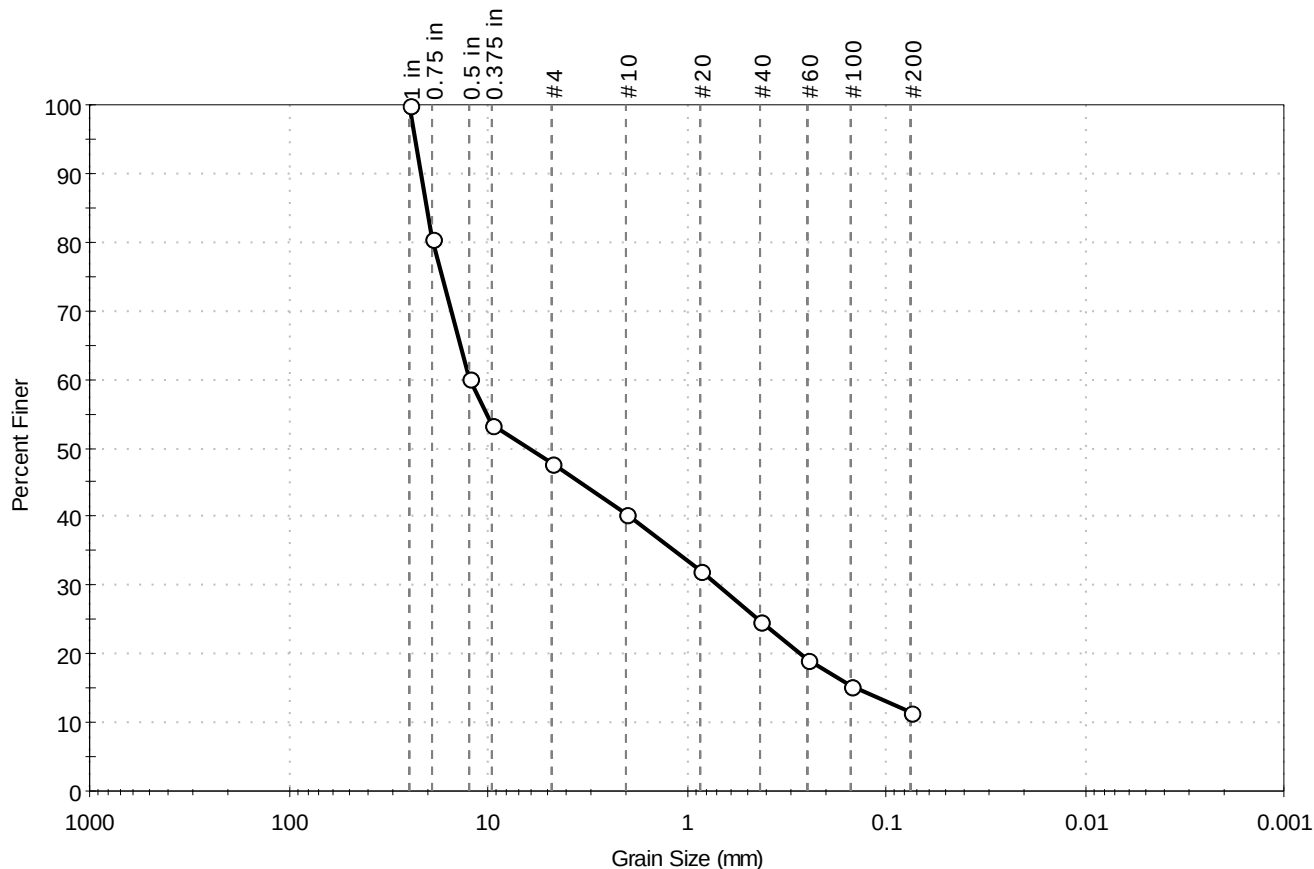
Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---



Client:	GZA GeoEnvironmental, Inc.		
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME	Project No:	GTX-306898
Boring ID:	BB-ARH-105	Sample Type:	bag
Sample ID:	1D	Test Date:	09/08/17
Depth :	1.5-3.5 ft	Test Id:	421598
Test Comment:	---		
Visual Description:	Moist, dark grayish brown sandy gravel, little silt		
Sample Comment:	MEDOT Modified Burmister description as requested		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	52.2	36.2	11.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	80		
0.5 in	12.50	60		
0.375 in	9.50	53		
#4	4.75	48		
#10	2.00	40		
#20	0.85	32		
#40	0.42	25		
#60	0.25	19		
#100	0.15	15		
#200	0.075	12		

Coefficients

D ₈₅ = 20.2522 mm	D ₃₀ = 0.6927 mm
D ₆₀ = 12.3468 mm	D ₁₅ = 0.1429 mm
D ₅₀ = 6.2056 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-a (0))

Sample / Test Description

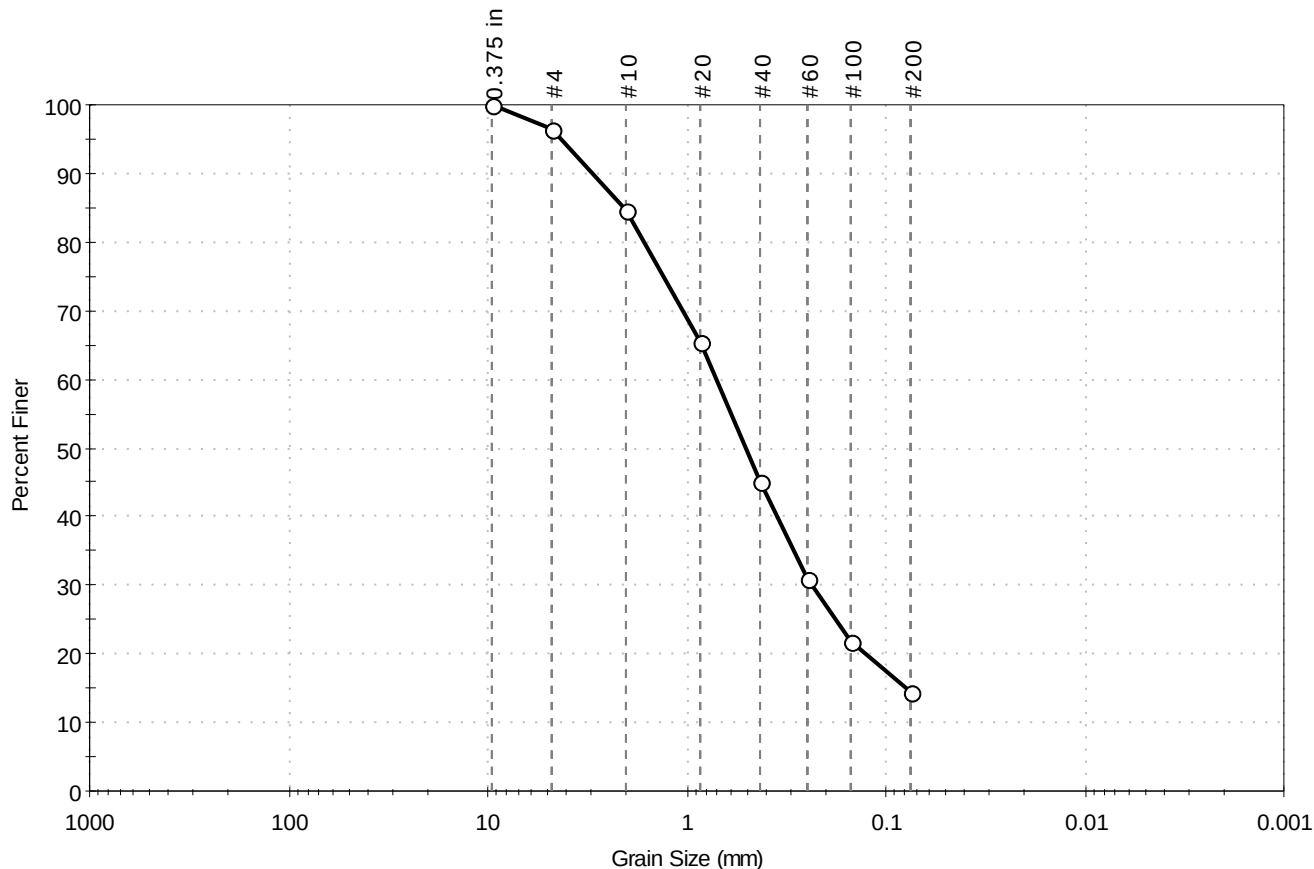
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD



Client:	GZA GeoEnvironmental, Inc.		
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME	Project No:	GTX-306898
Boring ID:	BB-ARH-105	Sample Type:	bag
Sample ID:	5D	Test Date:	09/08/17
Depth :	20-22 ft	Test Id:	421599
Test Comment:	---		
Visual Description:	Moist, dark brown sand, some silt, trace gravel		
Sample Comment:	MEDOT Modified Burmister description as requested		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	3.6	81.9	14.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	96		
#10	2.00	85		
#20	0.85	65		
#40	0.42	45		
#60	0.25	31		
#100	0.15	22		
#200	0.075	15		

Coefficients

D ₈₅ = 2.0499 mm	D ₃₀ = 0.2354 mm
D ₆₀ = 0.7082 mm	D ₁₅ = 0.0783 mm
D ₅₀ = 0.5039 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

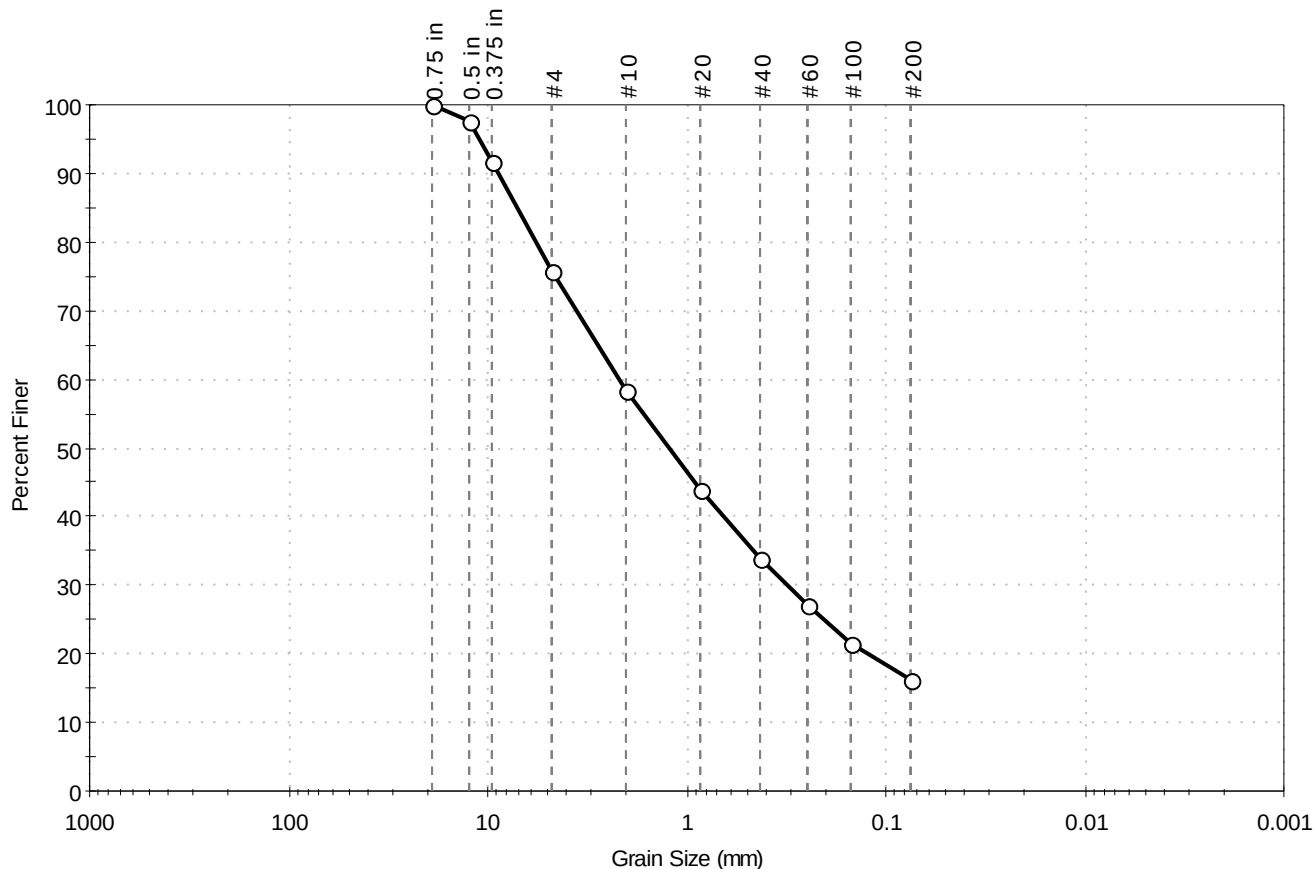
Sample / Test Description

Sand/Gravel Particle Shape : ANGULAR
Sand/Gravel Hardness : HARD



Client:	GZA GeoEnvironmental, Inc.		
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME	Project No:	GTX-306898
Boring ID:	BB-ARH-105	Sample Type:	bag
Sample ID:	9D	Test Date:	09/06/17
Depth :	34-36 ft	Test Id:	421600
Test Comment:	---		
Visual Description:	Moist, dark gray sand, some gravel, little silt		
Sample Comment:	MEDOT Modified Burmister description as requested		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	24.2	59.5	16.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	98		
0.375 in	9.50	92		
#4	4.75	76		
#10	2.00	58		
#20	0.85	44		
#40	0.42	34		
#60	0.25	27		
#100	0.15	22		
#200	0.075	16		

Coefficients

D ₈₅ = 7.0723 mm	D ₃₀ = 0.3148 mm
D ₆₀ = 2.1699 mm	D ₁₅ = N/A
D ₅₀ = 1.2123 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

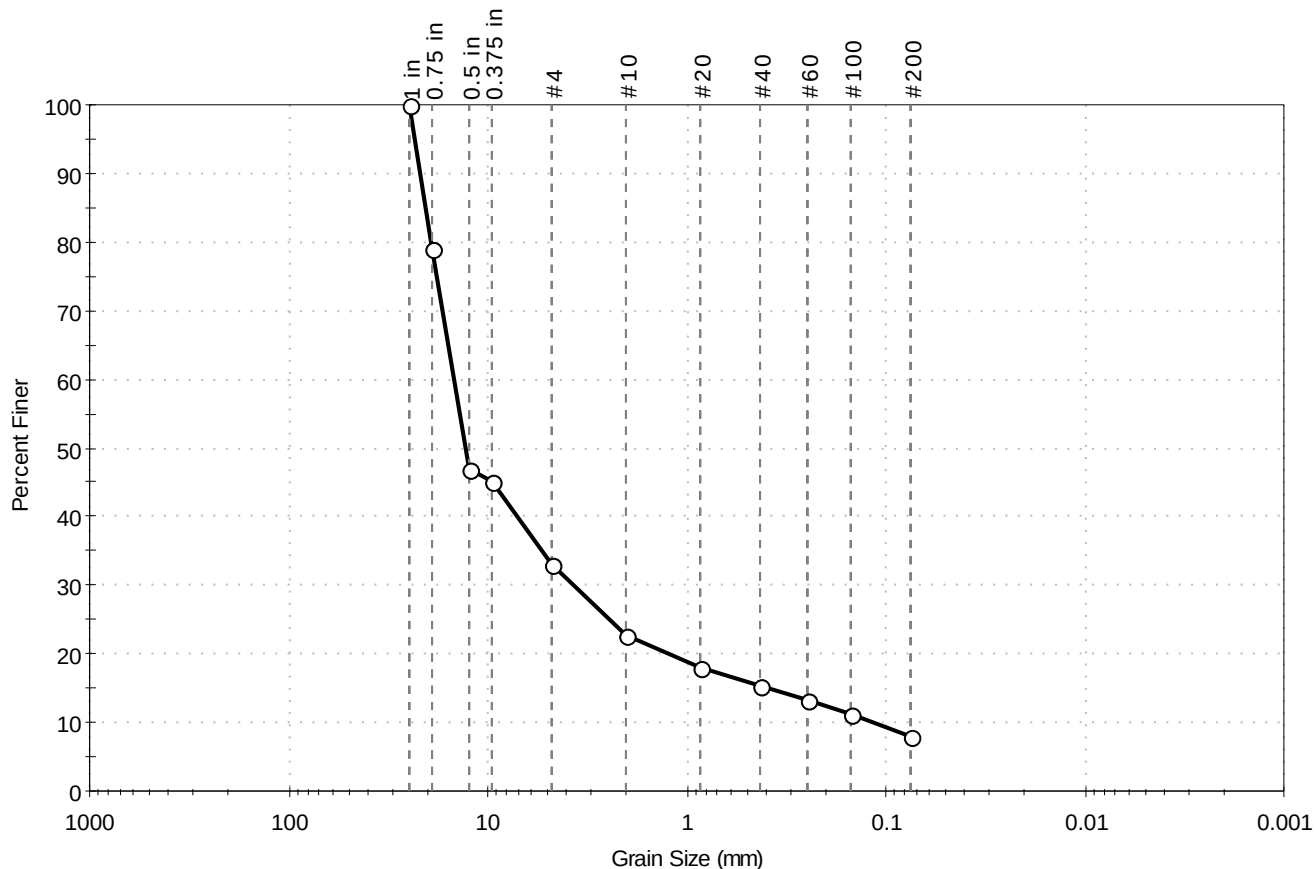
AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample / Test Description

Sand/Gravel Particle Shape : ANGULAR
Sand/Gravel Hardness : HARD

Client:	GZA GeoEnvironmental, Inc.		
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME	Project No:	GTX-306898
Boring ID:	BB-ARH-105	Sample Type:	bag
Sample ID:	11D	Test Date:	09/08/17
Depth :	38-40 ft	Test Id:	421601
Test Comment:	---		
Visual Description:	Moist, dark brown gravel some sand, trace silt		
Sample Comment:	MEDOT Modified Burmister description as requested		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	66.9	25.0	8.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	79		
0.5 in	12.50	47		
0.375 in	9.50	45		
#4	4.75	33		
#10	2.00	23		
#20	0.85	18		
#40	0.42	15		
#60	0.25	13		
#100	0.15	11		
#200	0.075	8.1		

Coefficients

D ₈₅ = 20.5654 mm	D ₃₀ = 3.6968 mm
D ₆₀ = 14.8487 mm	D ₁₅ = 0.3830 mm
D ₅₀ = 13.0345 mm	D ₁₀ = 0.1146 mm
C _u = 129.570	C _c = 8.031

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-a (1))

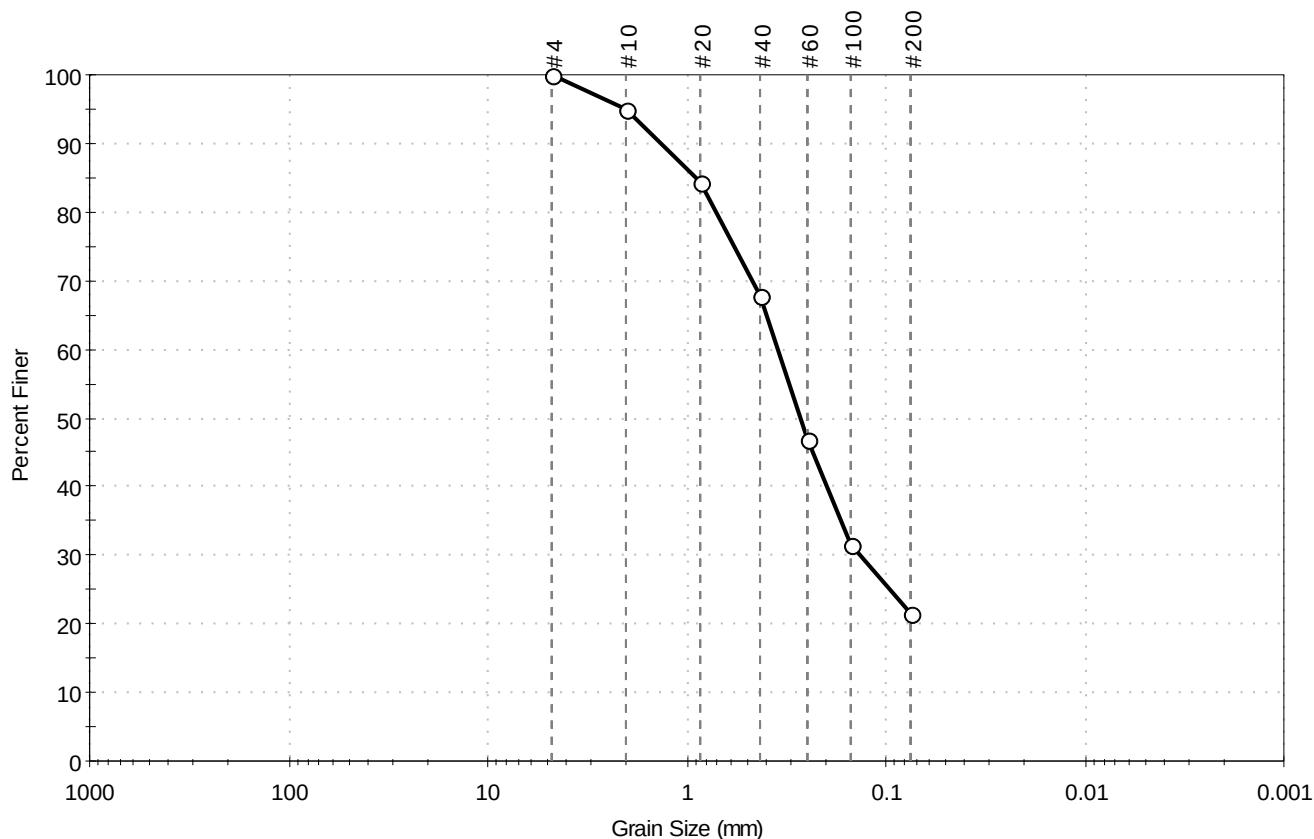
Sample / Test Description

Sand/Gravel Particle Shape : ANGULAR
Sand/Gravel Hardness : HARD



Client:	GZA GeoEnvironmental, Inc.		
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME	Project No:	GTX-306898
Boring ID:	BB-ARH-105	Sample Type:	bag
Sample ID:	17D	Test Date:	09/08/17
Depth :	65-67 ft	Test Id:	421603
Test Comment:	---		
Visual Description:	Moist, dark brownish gray sand, some silt		
Sample Comment:	MEDOT Modified Burmister description as requested		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	0.0	78.4	21.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	95		
#20	0.85	84		
#40	0.42	68		
#60	0.25	47		
#100	0.15	32		
#200	0.075	22		

Coefficients

D ₈₅ = 0.8874 mm	D ₃₀ = 0.1337 mm
D ₆₀ = 0.3483 mm	D ₁₅ = N/A
D ₅₀ = 0.2707 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

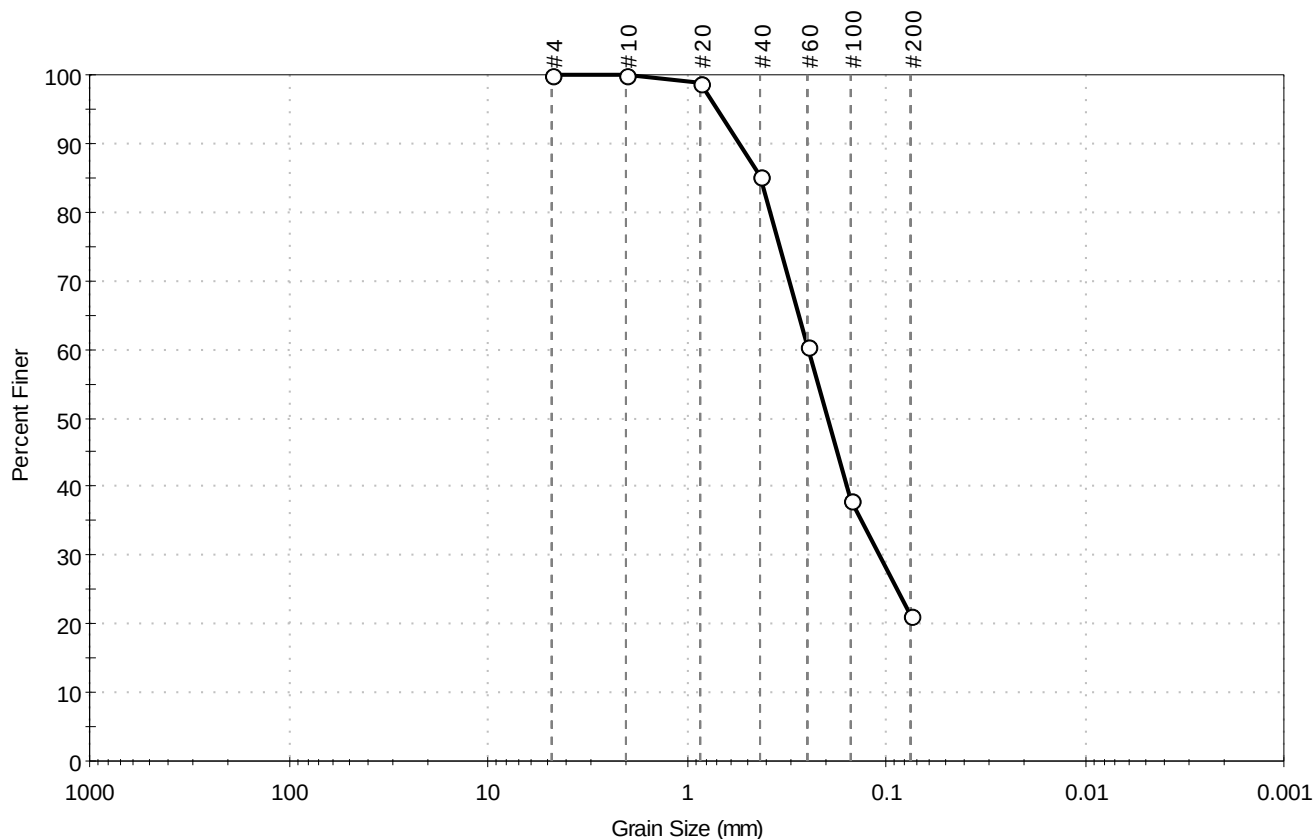
Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---



Client:	GZA GeoEnvironmental, Inc.				
Project:	Rines Hill Bridge Replacement				
Location:	Augusta, ME			Project No:	GTX-306898
Boring ID:	BB-ARH-105	Sample Type:	bag	Tested By:	jbr
Sample ID:	18D	Test Date:	09/07/17	Checked By:	emm
Depth :	70-72 ft	Test Id:	421604		
Test Comment:	---				
Visual Description:	Moist, olive brown sand, some silt				
Sample Comment:	MEDOT Modified Burmister description as requested				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	0.1	78.6	21.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	99		
#40	0.42	85		
#60	0.25	60		
#100	0.15	38		
#200	0.075	21		

Coefficients

D ₈₅ = 0.4240 mm	D ₃₀ = 0.1074 mm
D ₆₀ = 0.2478 mm	D ₁₅ = N/A
D ₅₀ = 0.1971 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample / Test Description

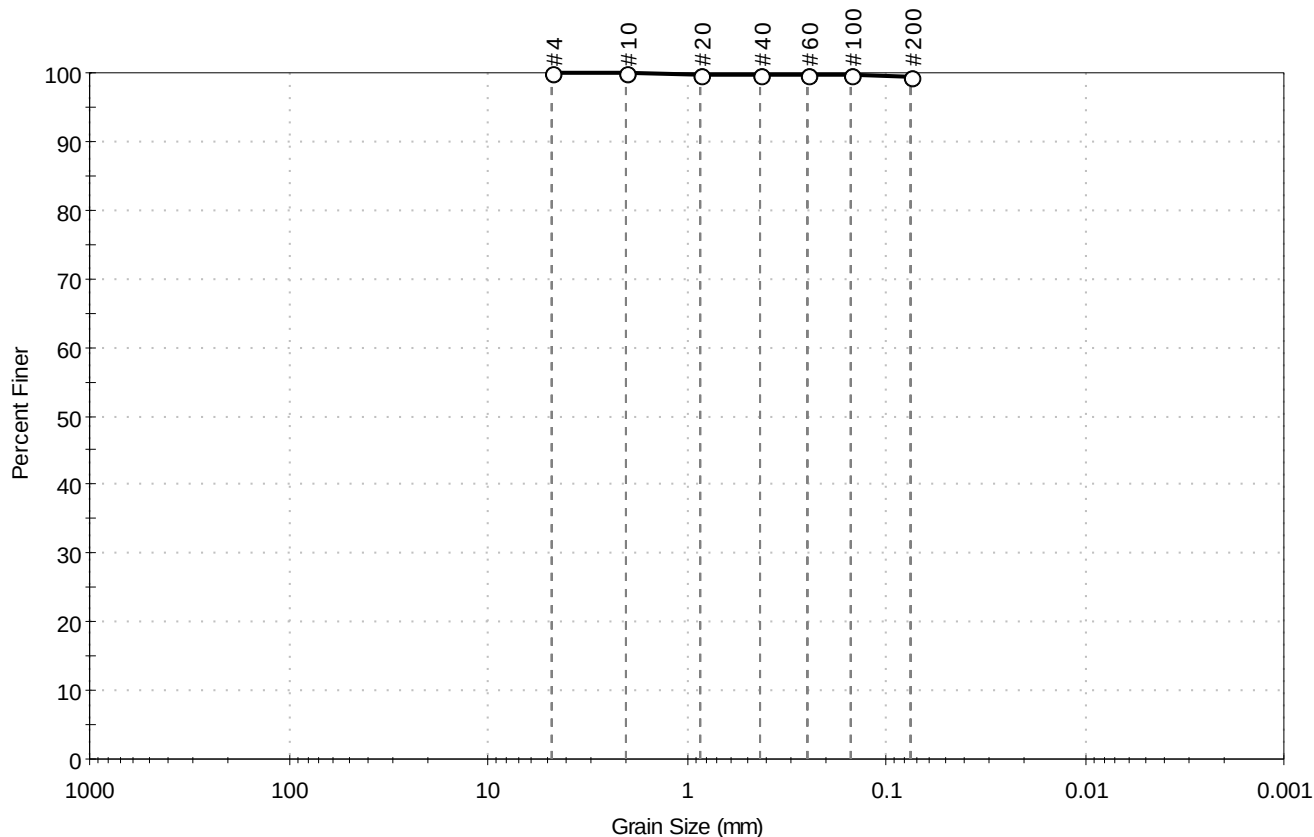
Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---



Client:	GZA GeoEnvironmental, Inc.		
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME	Project No:	GTX-306898
Boring ID:	RB-ARH-101	Sample Type:	bag
Sample ID:	5D	Test Date:	09/08/17
Depth :	8-10 ft	Test Id:	421605
Test Comment:	---		
Visual Description:	Moist, olive clay		
Sample Comment:	MEDOT Modified Burmister description as requested		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	0.0	0.5	99.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	100		
#100	0.15	100		
#200	0.075	100		

Coefficients

D ₈₅ = N/A	D ₃₀ = N/A
D ₆₀ = N/A	D ₁₅ = N/A
D ₅₀ = N/A	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample / Test Description

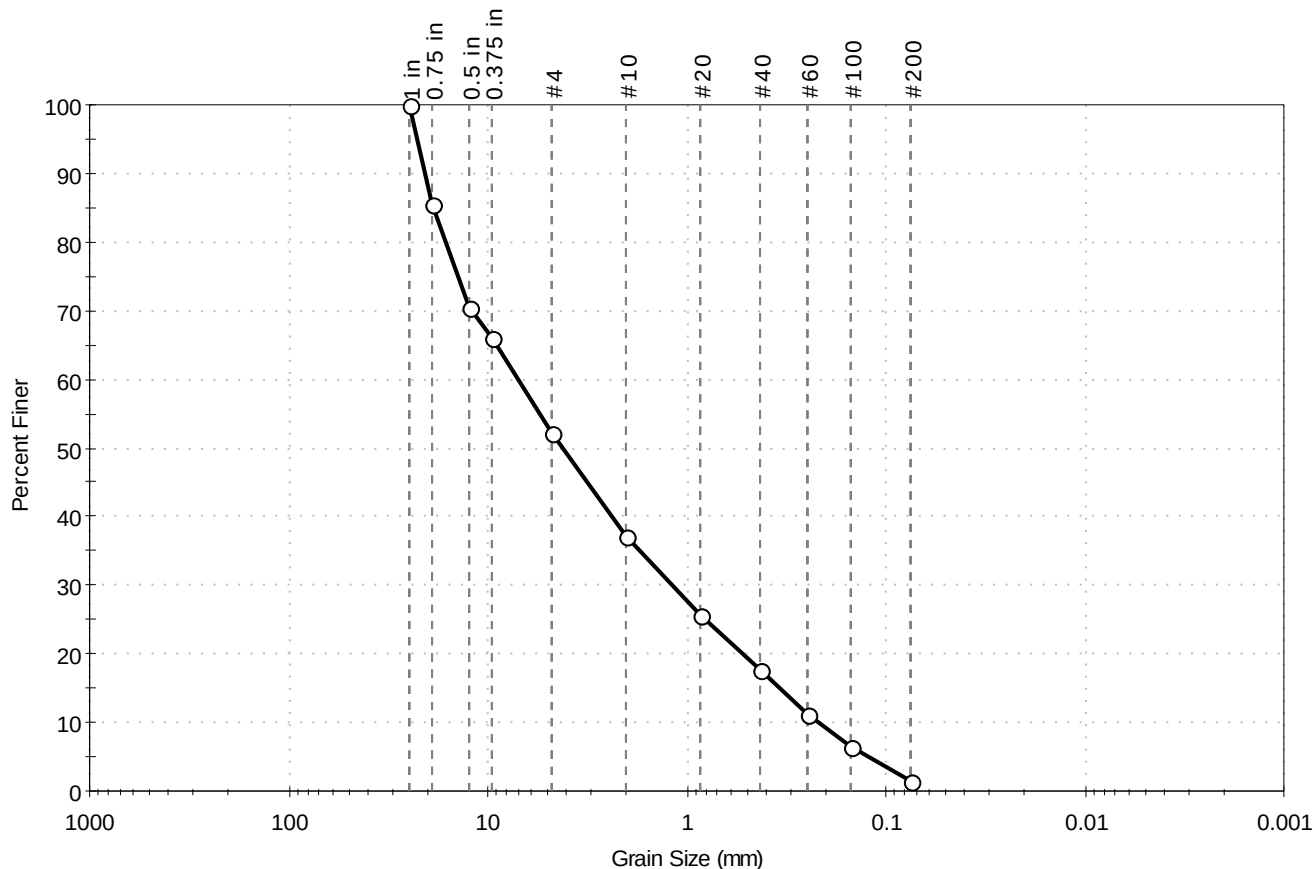
Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---



Client:	GZA GeoEnvironmental, Inc.		
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME	Project No:	GTX-306898
Boring ID:	RB-ARH-101	Sample Type:	bag
Sample ID:	6D	Test Date:	09/08/17
Depth :	10-12 ft	Test Id:	421606
Test Comment:	---		
Visual Description:	Moist, yellowish brown gravelly sand, trace silt		
Sample Comment:	MEDOT Modified Burmister description as requested		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	47.7	50.8	1.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	85		
0.5 in	12.50	70		
0.375 in	9.50	66		
#4	4.75	52		
#10	2.00	37		
#20	0.85	26		
#40	0.42	18		
#60	0.25	11		
#100	0.15	6		
#200	0.075	1.5		

Coefficients

D ₈₅ = 18.7798 mm	D ₃₀ = 1.1702 mm
D ₆₀ = 7.0214 mm	D ₁₅ = 0.3423 mm
D ₅₀ = 4.1629 mm	D ₁₀ = 0.2189 mm
C _u = 32.076	C _c = 0.891

Classification

ASTM Poorly graded sand with gravel (SP)

AASHTO Stone Fragments, Gravel and Sand (A-1-a (1))

Sample / Test Description

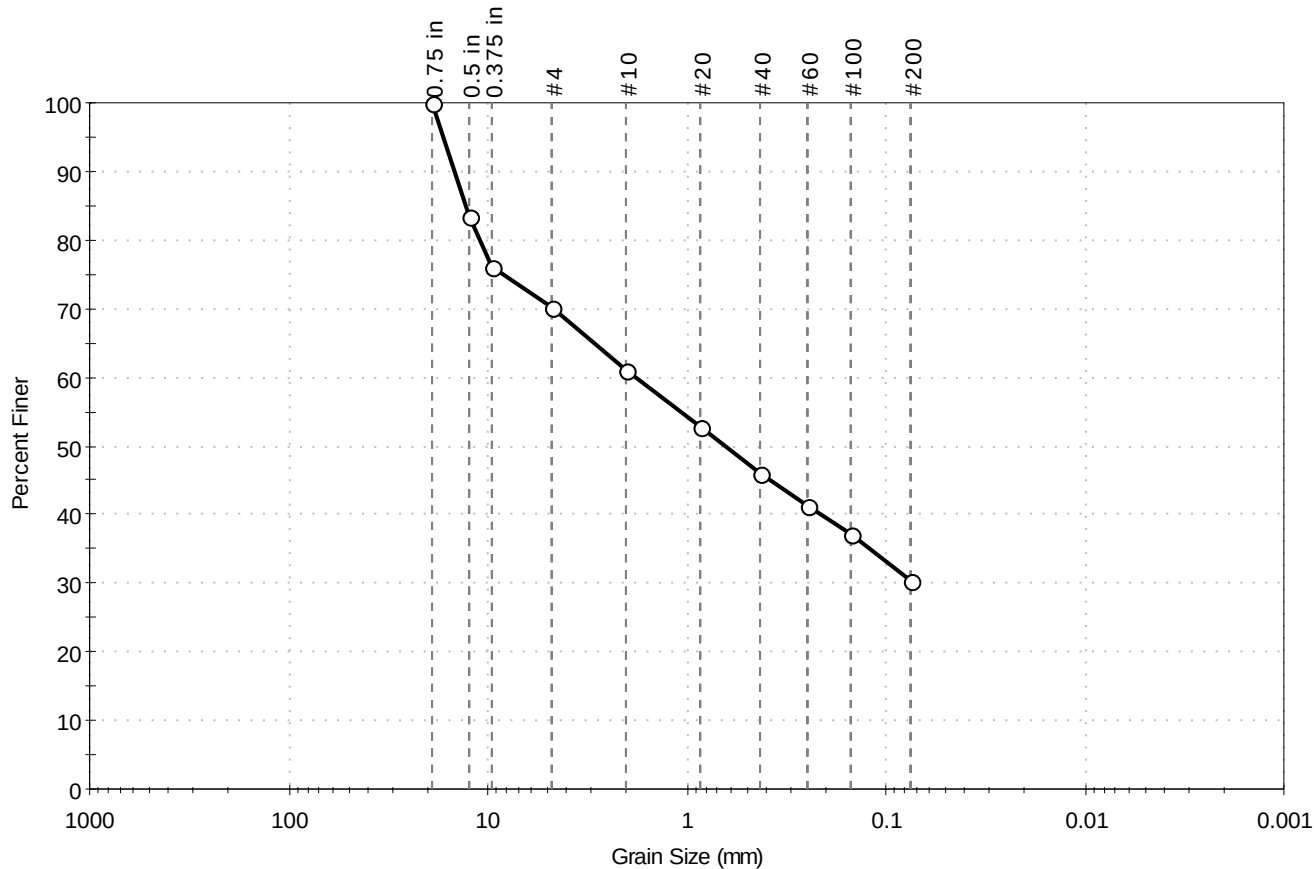
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD



Client:	GZA GeoEnvironmental, Inc.		
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME	Project No:	GTX-306898
Boring ID:	RB-ARH-101	Sample Type:	bag
Sample ID:	9D	Test Date:	09/06/17
Depth :	25-27 ft	Test Id:	421607
Test Comment:	---		
Visual Description:	Moist, olive brown sand, some silt, some gravel		
Sample Comment:	MEDOT Modified Burmister description as requested		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	29.7	40.0	30.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	84		
0.375 in	9.50	76		
#4	4.75	70		
#10	2.00	61		
#20	0.85	53		
#40	0.42	46		
#60	0.25	41		
#100	0.15	37		
#200	0.075	30		

Coefficients

D ₈₅ = 12.9694 mm	D ₃₀ = N/A
D ₆₀ = 1.7831 mm	D ₁₅ = N/A
D ₅₀ = 0.6338 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

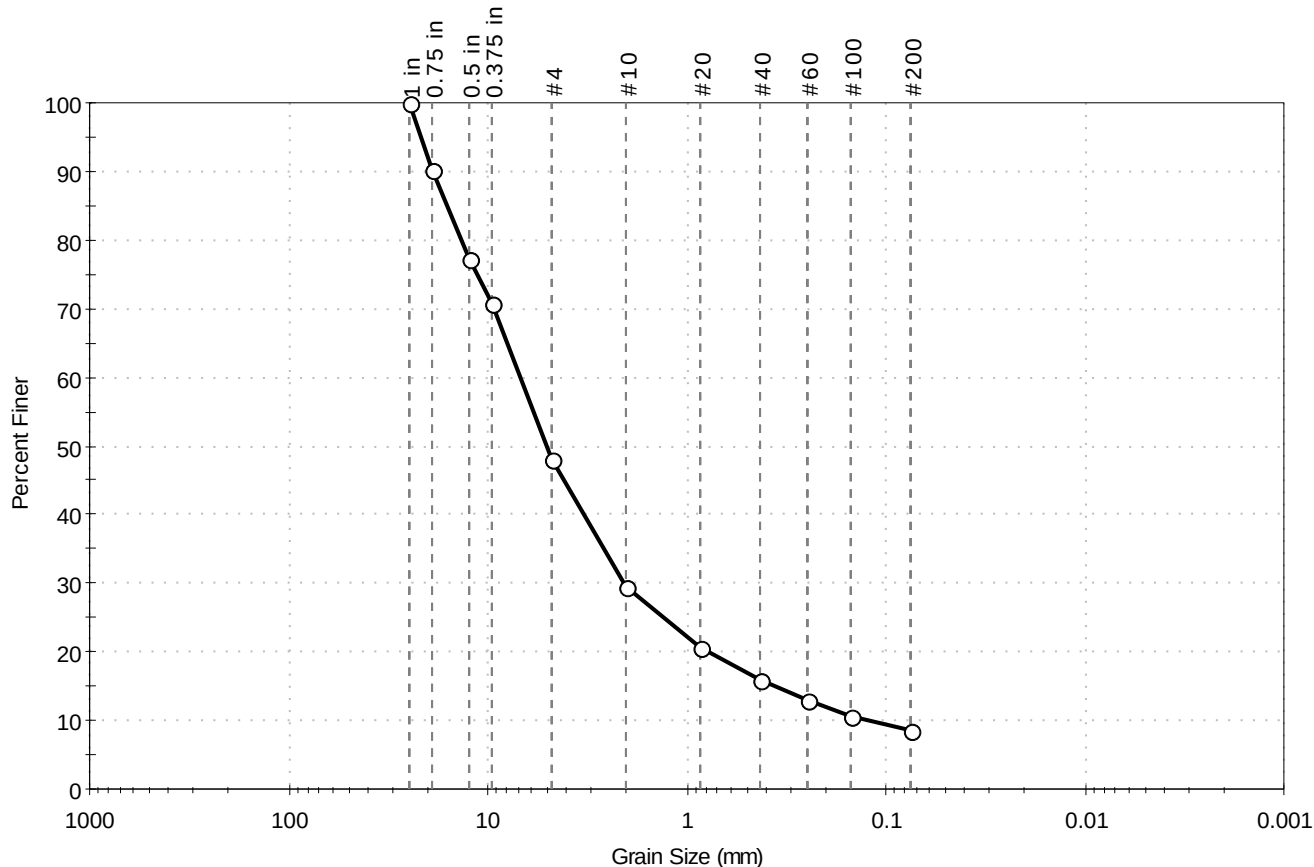
Sample / Test Description

Sand/Gravel Particle Shape : ANGULAR
Sand/Gravel Hardness : HARD



Client:	GZA GeoEnvironmental, Inc.		
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME	Project No:	GTX-306898
Boring ID:	RB-ARH-102	Sample Type:	bag
Sample ID:	6D	Test Date:	09/08/17
Depth :	25-27 ft	Test Id:	421608
Test Comment:	---		
Visual Description:	Moist, gray sandy gravel, trace silt		
Sample Comment:	MEDOT Modified Burmister description as requested		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	51.8	39.7	8.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	90		
0.5 in	12.50	77		
0.375 in	9.50	71		
#4	4.75	48		
#10	2.00	30		
#20	0.85	21		
#40	0.42	16		
#60	0.25	13		
#100	0.15	11		
#200	0.075	8.5		

Coefficients

D ₈₅ = 16.0272 mm	D ₃₀ = 2.0346 mm
D ₆₀ = 6.8255 mm	D ₁₅ = 0.3627 mm
D ₅₀ = 5.0232 mm	D ₁₀ = 0.1197 mm
C _u = 57.022	C _c = 5.067

Classification

ASTM N/A

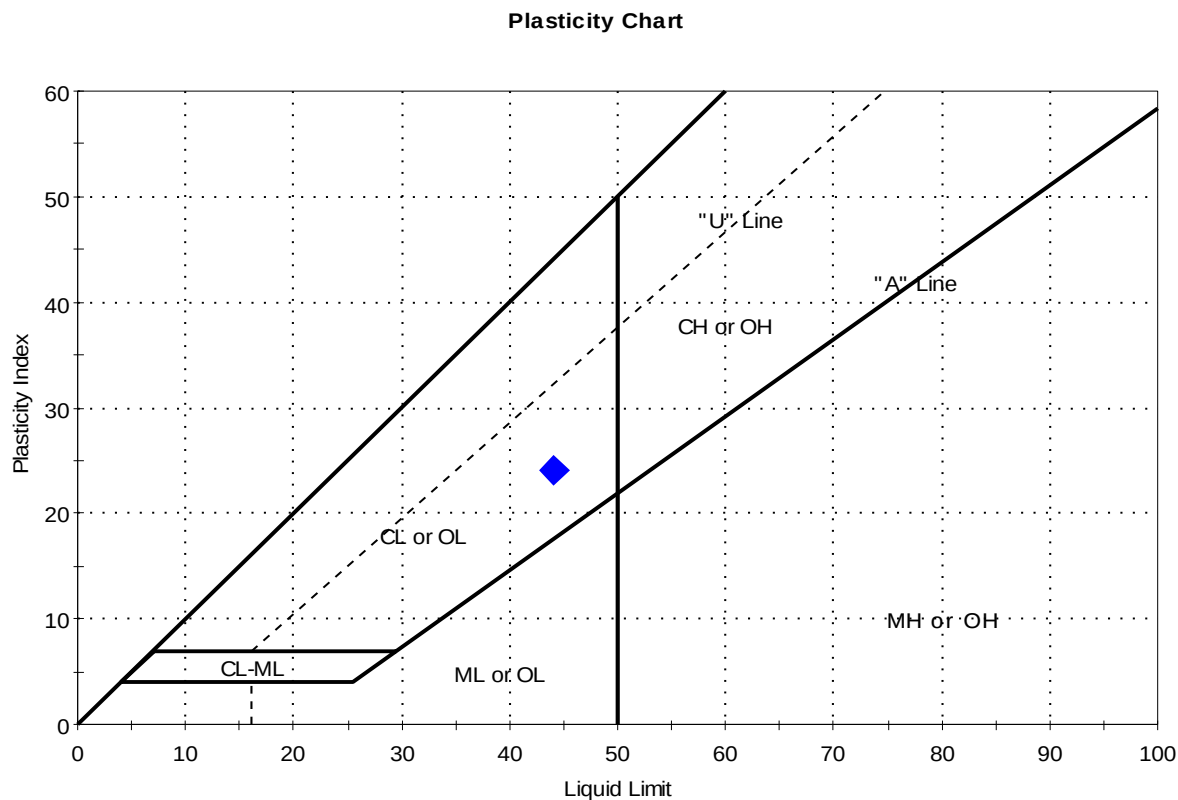
AASHTO Stone Fragments, Gravel and Sand (A-1-a (1))

Sample / Test Description

Sand/Gravel Particle Shape : ANGULAR
Sand/Gravel Hardness : HARD

Client:	GZA GeoEnvironmental, Inc.	Project No:	GTX-306898
Project:	Rines Hill Bridge Replacement		
Location:	Augusta, ME		
Boring ID:	BB-ARH-101	Sample Type:	bag
Sample ID:	4D	Test Date:	09/06/17
Depth :	14.5-16.5 ft	Test Id:	421616
Test Comment:	---	Tested By:	cam
Visual Description:	Moist, olive clay	Checked By:	emm
Sample Comment:	---		

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	4D	B-ARH-101	14.5-16.5 ft	30	44	20	24	0.4	

Sample Prepared using the WET method

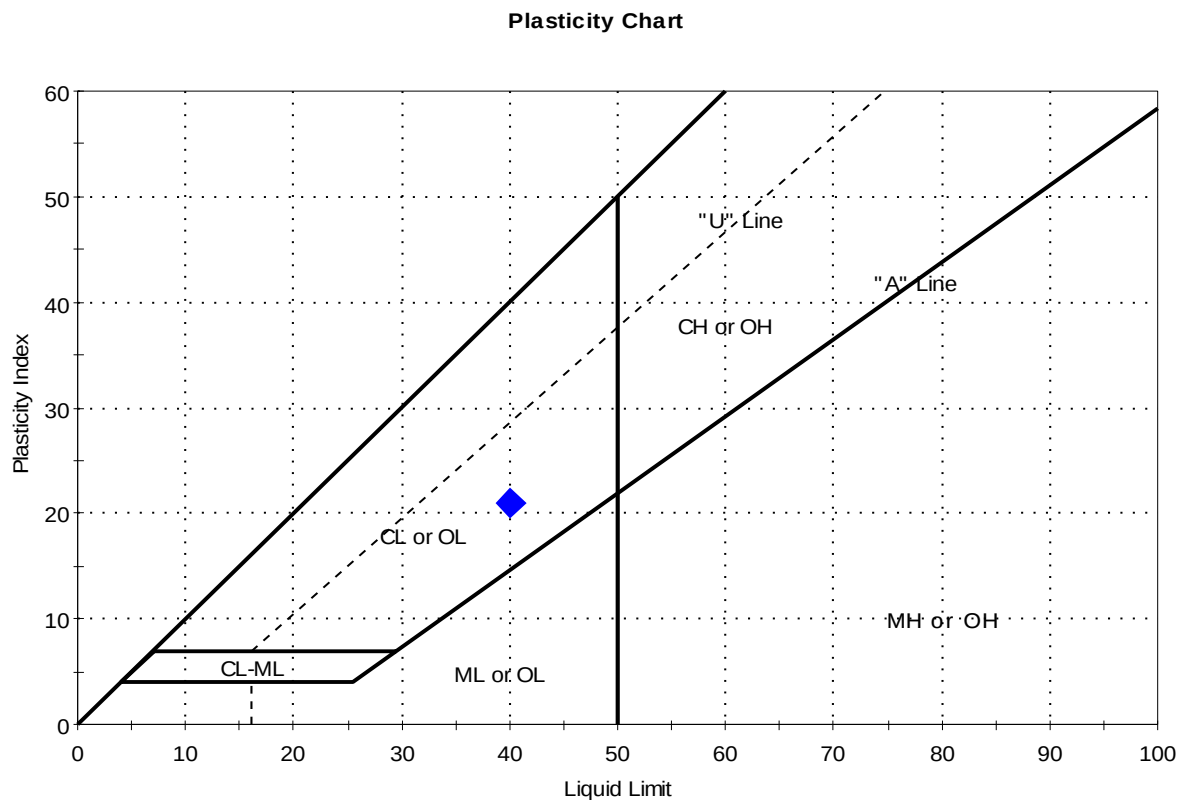
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: MEDIUM

Client:	GZA GeoEnvironmental, Inc.				
Project:	Rines Hill Bridge Replacement				
Location:	Augusta, ME			Project No:	GTX-306898
Boring ID:	BB-ARH-101	Sample Type:	bag	Tested By:	cam
Sample ID:	7D	Test Date:	09/06/17	Checked By:	emm
Depth :	23.5-25.5 ft	Test Id:	421617		
Test Comment:	---				
Visual Description:	Moist, olive clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	7D	B-ARH-10	23.5-25.5 ft	35	40	19	21	0.8	Lean clay (CL)

Sample Prepared using the WET method

2% Retained on #40 Sieve

Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: MEDIUM

SOIL CHAIN OF CUSTODY & TEST REQUEST

GeoTesting Express, Inc.
125 Nagog Park
Acton, MA 01720
800 434 1062 Toll Free
978 635 0266 Fax

2358 Perimeter Park Drive, Suite 320
Atlanta, GA 30341
770 645 6575 Tel
770 645 6570 Fax

www.geotesting.com

CLIENT		INVOICE (complete if different from Client)	
Company: GZA GeoEnvironmental, Inc.		Company:	
Address: 477 Congress Street, Suite 700		Address:	
City, State, Zip: Portland, Maine 04101		City, State, Zip:	
Contact: Andrew Blaisdell	Phone: 207-358-5117	Contact:	Phone:
E-mail: andrew.blaisdell@gza.com	Cell:	E-mail:	Cell:
PROJECT			
Project Name: Rines Hill Bridge Replacement		Client Project #: 09.0025943.00	Purchase Order#: 16305
Project Location: Augusta, ME		GTX Sales Order #:	Requested Turnaround: 9/5/17
On-site Contact:		E-mail:	Phone:

SOIL			Atterberg Limits (ASTM D 4318)	USCS - Classification (ASTM D 2487)	Grain Size: ASTM D 422 Sieve Only ☐ Sieve & Hydrometer ☐	Density ASTM D 2937 ASTM D 7263 ☐	Moisture Content (ASTM D 2216)	Organic Content (ASTM D 2974)	pH (ASTM D 4972)	Specific Gravity (ASTM D 854)	Electrical Resistivity (ASTM G 57)	Proctor Compaction Standard - ASTM D 698 ☐ Modified - ASTM D 1557 ☐	California Bearing Ratio * (ASTM D 1883)	Direct Shear* (ASTM D 3080)	Triaxial Shear* UU - ASTM D 2860 ☐ CU - ASTM D 4767 ☐ CD - ASTM D 7181 ☐	Incremental Consolidation* (ASTM D 2436)	Permeability/ Hydraulic Conductivity* Fixed Wall - ASTM D 2434 ☐ Flexible Wall - ASTM D 5084 ☐	Unconfined Compression (ASTM D 2166)	Other: Maine DOT Frost Classifications	Other: AASHTO Classification
Boring ID	Sample ID	Depth																		
BB-ARH-101	1D	1.7-3.7'			x		x												x	x
BB-ARH-101	2D	4.5-6.5'					x												x	x
BB-ARH-101	4D	14.5-16.5'	x	x			x													
BB-ARH-101	5D	19.5-21.5'					x													
BB-ARH-101	7D	23.5-25.5'	x	x			x													
BB-ARH-101	10D	29.5-31.5'			x		x													
BB-ARH-101	12D ✓	39.5-41.5'			x		x													
BB-ARH-101	16D	59.5-61.5'			x		x													
BB-ARH-101	17D ✓	64.5-66.5'			x		x													
BB-ARH-102	7D ✓	15-17'			x		x													
BB-ARH-102	8D ✓	20-22'			x		x													

*Specify Test Conditions (Undisturbed or Remolded, Density and moisture, Test Normal Loads, Test Confining Stresses, etc.):

Soil Descriptions per MaineDOT Modified Burmister format, see system on attached sheet.

AUTHORIZE BY SIGNING AND DATING:

SIGNATURE: Andrew Blaisdell

PRINT NAME: ANDREW BLAISDELL DATE: 8/18/17

For GTX Use Only
Incoming Sample Inspection Performed ☐
Adverse conditions:

Relinquished By:	DATE:	Received By:	DATE: <u>8/22/17</u>
Relinquished By:	TIME:	Received By:	TIME:
	DATE:		DATE:
	TIME:		TIME:

SOIL CHAIN OF CUSTODY & TEST REQUEST

GeoTesting Express, Inc.
125 Nagog Park
Acton, MA 01720
800 434 1062 Toll Free
978 635 0266 Fax

2358 Perimeter Park Drive, Suite 320
Atlanta, GA 30341
770 645 6575 Tel
770 645 6570 Fax

www.geotesting.com

CLIENT		INVOICE (complete if different from Client)	
Company: GZA GeoEnvironmental, Inc.		Company:	
Address: 477 Congress Street, Suite 700		Address:	
City, State, Zip: Portland, Maine 04101		City, State, Zip:	
Contact: Andrew Blaisdell	Phone: 207-358-5117	Contact:	Phone:
E-mail: andrew.blaisdell@gza.com	Cell:	E-mail:	Cell:

PROJECT		
Project Name: Rines Hill Bridge Replacement	Client Project #: 09.0025943.00	Purchase Order#: 16305
Project Location: Augusta, ME	GTX Sales Order #:	Requested Turnaround: 9/5/17
On-site Contact:	E-mail:	Phone:

SOIL			Atterberg Limits (ASTM D 4318)	USCS - Classification (ASTM D 2487)	Grain Size: ASTM D 422 Sieve Only ☒ Sieve & Hydrometer ☐	Density ASTM D 2937 ASTM D 7263 ☐	Moisture Content (ASTM D 2216)	Organic Content (ASTM D 2974)	pH (ASTM D 4972)	Specific Gravity (ASTM D 854)	Electrical Resistivity (ASTM G 57)	Proctor Compaction Standard - ASTM D 698 ☐ Modified - ASTM D 1557 ☐	California Bearing Ratio * (ASTM D 1883)	Direct Shear* (ASTM D 3080)	Triaxial Shear* UU - ASTM D 2850 ☐ CU - ASTM D 4767 ☐ CD - ASTM D 7181 ☐	Incremental Consolidation* (ASTM D 2435)	Permeability/ Hydraulic Conductivity* Fixed Wall - ASTM D 2434 ☐ Flexible Wall - ASTM D 5094 ☐	Unconfined Compression (ASTM D 2166)	Other: Maine DOT Frost Classifications	Other: AASHTO Classification
Boring ID	Sample ID	Depth																		
BB-ARH-102	10D ✓	30-32'			x		x													
BB-ARH-103	5D ✓	8-10'			x		x												x	x
BB-ARH-103	9D ✓	25-27'			x		x													
BB-ARH-103	11D ✓	35-37'			x		x													
BB-ARH-103	12D ✓	40-42'			x		x													
BB-ARH-103	14D ✓	50-52'			x		x													
BB-ARH-104	1D ✓	1.3-3.3'	x	x	x		x												x	x
BB-ARH-105	1D ✓	1.5-3.5'			x		x												x	x
BB-ARH-105	5D ✓	20-22'			x		x													
BB-ARH-105	9D ✓	34-36'			x		x													
BB-ARH-105	11D ✓	38-40'			x		x													

*Specify Test Conditions (Undisturbed or Remolded, Density and moisture, Test Normal Loads, Test Confining Stresses, etc.):

Soil Descriptions per MaineDOT Modified Burmister format, see system on attached sheet.

AUTHORIZE BY SIGNING AND DATING:

SIGNATURE: Andrew Blaisdell

PRINT NAME: ANDREW BLAISDELL

DATE: 8/18/17

For GTX Use Only

Incoming Sample Inspection Performed ☐
Adverse conditions:

Relinquished By:	DATE:	Received By:	DATE:
	TIME:		TIME:
Relinquished By:	DATE:	Received By:	DATE:
	TIME:		TIME:

SOIL CHAIN OF CUSTODY & TEST REQUEST

GeoTesting Express, Inc.

125 Nagog Park
Acton, MA 01720
800 434 1062 Toll Free
978 635 0266 Fax

2358 Perimeter Park Drive, Suite 320
Atlanta, GA 30341
770 645 6575 Tel
770 645 6570 Fax

www.geotesting.com

CLIENT	
Company: GZA GeoEnvironmental, Inc.	
Address: 477 Congress Street, Suite 700	
City, State, Zip: Portland, Maine 04101	
Contact: Andrew Blaisdell	Phone: 207-358-5117
E-mail: andrew.blaisdell@gza.com	Cell:

INVOICE (complete if different from Client)	
Company:	
Address:	
City, State, Zip:	
Contact:	Phone:
E-mail:	Cell:

PROJECT		
Project Name: Rines Hill Bridge Replacement	Client Project #: 09.0025943.00	Purchase Order#: 16305
Project Location: Augusta, ME	GTX Sales Order #:	Requested Turnaround: 9/5/17
On-site Contact:	E-mail:	Phone:

SOIL			Atterberg Limits (ASTM D 4318)	USCS - Classification (ASTM D 2487)	Grain Size: ASTM D 422 ☐ Sieve Only ☒ Sieve & Hydrometer ☐	Density ASTM D 2937 ☐ ASTM D 7263 ☐	Moisture Content (ASTM D 2216)	Organic Content (ASTM D 2974)	pH (ASTM D 4972)	Specific Gravity (ASTM D 854)	Electrical Resistivity (ASTM G 57)	Proctor Compaction Standard - ASTM D 698 ☐ Modified - ASTM D 1557 ☐	California Bearing Ratio* (ASTM D 1883)	Direct Shear* (ASTM D 3080)	Triaxial Shear* UU - ASTM D 2850 ☐ CU - ASTM D 4767 ☐ CD - ASTM D 7181 ☐	Incremental Consolidation* (ASTM D 2435)	Permeability/ Hydraulic Conductivity* Fixed Wall - ASTM D 2434 ☐ Flexible Wall - ASTM D 5084 ☐	Unconfined Compression (ASTM D 2166)	Other: Maine DOT Frost Classifications	Other: AASHTO Classification
Boring ID	Sample ID	Depth																		
BB-ARH-105	13D ✓	50-52'			X		X													
BB-ARH-105	17D ✓	65-67'			X		X													
BB-ARH-105	18D ✓	70-72'			X		X													
RB-ARH-101	5D ✓	8-10'	X	X	X		X													
RB-ARH-101	6D ✓	10-12'			X		X													
RB-ARH-101	9D ✓	25-27'			X		X													
RB-ARH-102	6D ✓	25-27'			X		X													

*Specify Test Conditions (Undisturbed or Remolded, Density and moisture, Test Normal Loads, Test Confining Stresses, etc.):

Soil Descriptions per MaineDOT Modified Burmister format, see system on attached sheet.

AUTHORIZE BY SIGNING AND DATING:

SIGNATURE: Andrew Blaisdell

PRINT NAME: ANDREW BLAISDELL

DATE: 8/18/17

For GTX Use Only

Incoming Sample Inspection Performed ☐
Adverse conditions:

Relinquished By:	DATE:	Received By:	DATE:
	TIME:		TIME:
Relinquished By:	DATE:	Received By:	DATE:
	TIME:		TIME:

WARRANTY and LIABILITY

GeoTesting Express (GTX) warrants that all tests it performs are run in general accordance with the specified test procedures and accepted industry practice. GTX will correct or repeat any test that does not comply with this warranty. GTX has no specific knowledge as to conditioning, origin, sampling procedure or intended use of the material.

GTX may report engineering parameters that require us to interpret the test data. Such parameters are determined using accepted engineering procedures. However, GTX does not warrant that these parameters accurately reflect the true engineering properties of the *in situ* material. Responsibility for interpretation and use of the test data and these parameters for engineering and/or construction purposes rests solely with the user and not with GTX or any of its employees.

GTX's liability will be limited to correcting or repeating a test which fails our warranty. GTX's liability for damages to the Purchaser of testing services for any cause whatsoever shall be limited to the amount GTX received for the testing services. GTX will not be liable for any damages, or for any lost benefits or other consequential damages resulting from the use of these test results, even if GTX has been advised of the possibility of such damages. GTX will not be responsible for any liability of the Purchaser to any third party.

Commonly Used Symbols

A	pore pressure parameter for $\Delta\sigma_1 - \Delta\sigma_3$	S_r	Post cyclic undrained shear strength
B	pore pressure parameter for $\Delta\sigma_3$	T	temperature
CAI	CERCHAR Abrasiveness Index	t	time
CIU	isotropically consolidated undrained triaxial shear test	U, UC	unconfined compression test
CR	compression ratio for one dimensional consolidation	UU, Q	unconsolidated undrained triaxial test
CSR	cyclic stress ratio	u_a	pore gas pressure
C_c	coefficient of curvature, $(D_{30})^2 / (D_{10} \times D_{60})$	u_e	excess pore water pressure
C_u	coefficient of uniformity, D_{60}/D_{10}	u, u_w	pore water pressure
C_c	compression index for one dimensional consolidation	V	total volume
C_α	coefficient of secondary compression	V_g	volume of gas
c_v	coefficient of consolidation	V_s	volume of solids
c	cohesion intercept for total stresses	V_s	shear wave velocity
c'	cohesion intercept for effective stresses	V_v	volume of voids
D	diameter of specimen	V_w	volume of water
D	damping ratio	V_o	initial volume
D_{10}	diameter at which 10% of soil is finer	v	velocity
D_{15}	diameter at which 15% of soil is finer	W	total weight
D_{30}	diameter at which 30% of soil is finer	W_s	weight of solids
D_{50}	diameter at which 50% of soil is finer	W_w	weight of water
D_{60}	diameter at which 60% of soil is finer	w	water content
D_{85}	diameter at which 85% of soil is finer	w_c	water content at consolidation
d_{50}	displacement for 50% consolidation	w_f	final water content
d_{90}	displacement for 90% consolidation	w_l	liquid limit
d_{100}	displacement for 100% consolidation	w_n	natural water content
E	Young's modulus	w_p	plastic limit
e	void ratio	w_s	shrinkage limit
e_c	void ratio after consolidation	w_o, w_i	initial water content
e_o	initial void ratio	α	slope of q_f versus p_f
G	shear modulus	α'	slope of q_f versus p_f'
G_s	specific gravity of soil particles	γ_t	total unit weight
H	height of specimen	γ_d	dry unit weight
H_R	Rebound Hardness number	γ_s	unit weight of solids
i	gradient	γ_w	unit weight of water
I_s	Uncorrected point load strength	ϵ	strain
$I_{s(50)}$	Size corrected point load strength index	ϵ_{vol}	volume strain
H_A	Modified Taber Abrasion	ϵ_h, ϵ_v	horizontal strain, vertical strain
H_T	Total hardness	μ	Poisson's ratio, also viscosity
K_o	lateral stress ratio for one dimensional strain	σ	normal stress
k	permeability	σ'	effective normal stress
LI	Liquidity Index	σ_c, σ'_c	consolidation stress in isotropic stress system
m_v	coefficient of volume change	σ_h, σ'_h	horizontal normal stress
n	porosity	σ_v, σ'_v	vertical normal stress
PI	plasticity index	σ'_{vc}	Effective vertical consolidation stress
P_c	preconsolidation pressure	σ_1	major principal stress
p	$(\sigma_1 + \sigma_3) / 2, (\sigma_v + \sigma_h) / 2$	σ_2	intermediate principal stress
p'	$(\sigma'_1 + \sigma'_3) / 2, (\sigma'_v + \sigma'_h) / 2$	σ_3	minor principal stress
p'_c	p' at consolidation	τ	shear stress
Q	quantity of flow	ϕ	friction angle based on total stresses
q	$(\sigma_1 - \sigma_3) / 2$	ϕ'	friction angle based on effective stresses
q_f	q at failure	ϕ'_r	residual friction angle
q_o, q_i	initial q	ϕ_{ult}	ϕ for ultimate strength
q_c	q at consolidation		

	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 Portland, ME PM: NVW Assigned By: NVW Collected By: Ben Woodman	Project Information: Rines Hill Bridge Augusta, Maine GZA Project Number: 09.0025943.00 Summary Page: _____ Report Date: _____
	1 of 1 05.06.19		

LABORATORY TESTING DATA SHEET

Boring	Sample No.	Depth (ft)	Laboratory No.	Identification Tests								Proctor / CBR / Permeability Tests								Laboratory Log and Soil Description
				As Received Water Content %	LL %	PL %	Gravel %	Sand %	Fines %	Org. %	G _s	Dry unit wt. pcf	Test Water Content %	γ_d MAX (pcf) W _{opt} (%)	γ_d MAX (pcf) W _{opt} (%) (Corr.)	Target Test Setup as % of Proctor	CBR @ 0.1"	CBR @ 0.2"	Perme-ability cm/sec	
				D2216	D4318		D6913			D2874	D854			D1557						
BB-ARH-301	1D	1.8-3.8	S-1	4.5			40.1	48.1	11.8											Brown Gravelly fine to coarse SAND, little Silt
BB-ARH-301	2D	3.8-5.8	S-2	5.0			29.9	56.6	13.5											Brown f-m SAND, some fine Gravel, little Silt
BB-ARH-301	9D	28-30	S-3	19.0			21.6	18.5	59.9											Grey SILTY CLAY, some f-c Gravel, little f-c Sand
BB-ARH-302	2D	4-6	S-4	7.3			13.6	69.8	16.6											Brown f-m SAND, little Silt, little fine Gravel
BB-ARH-302	9D	41-43	S-5	8.8			18.3	61.0	20.7											Brown f-c SAND, some silt, little fine Gravel
BB-ARH-302	10D	46-48	S-6	8.1			0.0	85.9	14.1											Brown f-c SAND, little Silt

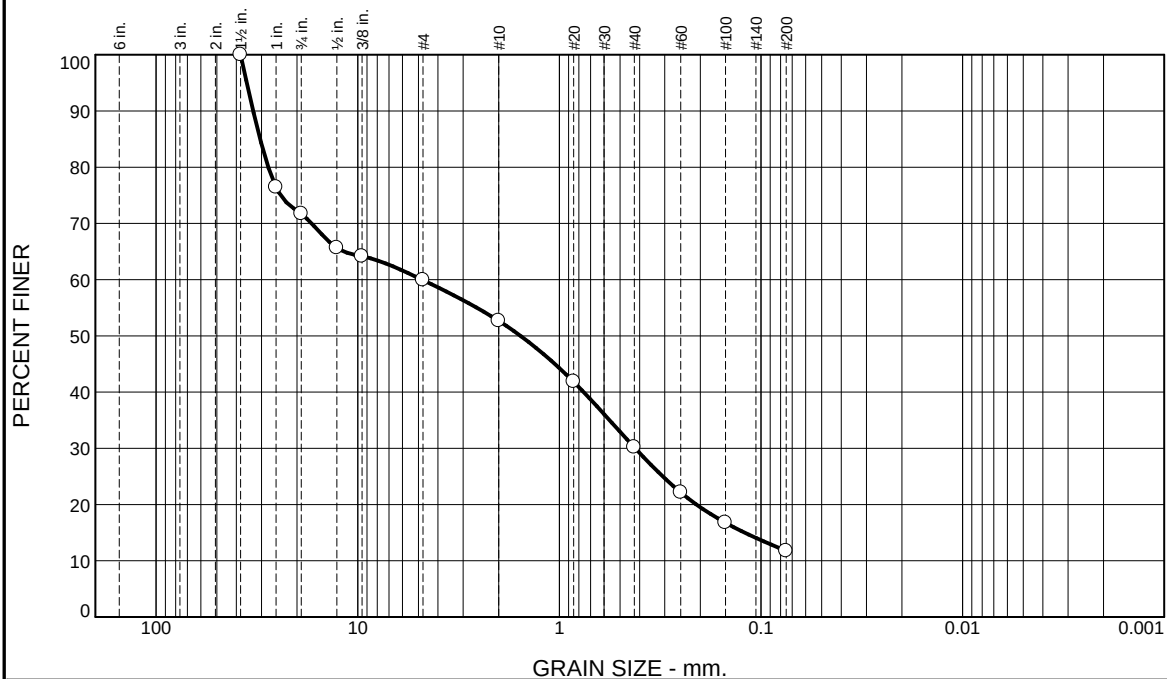
Date Received 05.01.2019

Reviewed By: SKW

Date Reviewed: 05.07.2019

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

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	28.3	11.8	7.2	22.5	18.4	11.8	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1-1/2"	100.0		
1"	76.4		
3/4"	71.7		
1/2"	65.6		
3/8"	64.2		
#4	59.9		
#10	52.7		
#20	41.8		
#40	30.2		
#60	22.2		
#100	16.8		
#200	11.8		

* (no specification provided)

Material Description

Brown Gravelly fine to coarse SAND, little Silt

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

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

Coefficients

D₉₀= 32.9844 D₈₅= 30.4641 D₆₀= 4.8005
D₅₀= 1.5583 D₃₀= 0.4204 D₁₅= 0.1204
D₁₀= C_u= C_c=

Remarks

Date Received: 05.01.19 Date Tested: 05.06.19

Tested By: MN

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: BB-ARH
Sample Number: 301 / 1D

Depth: 1.8-3.8'

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

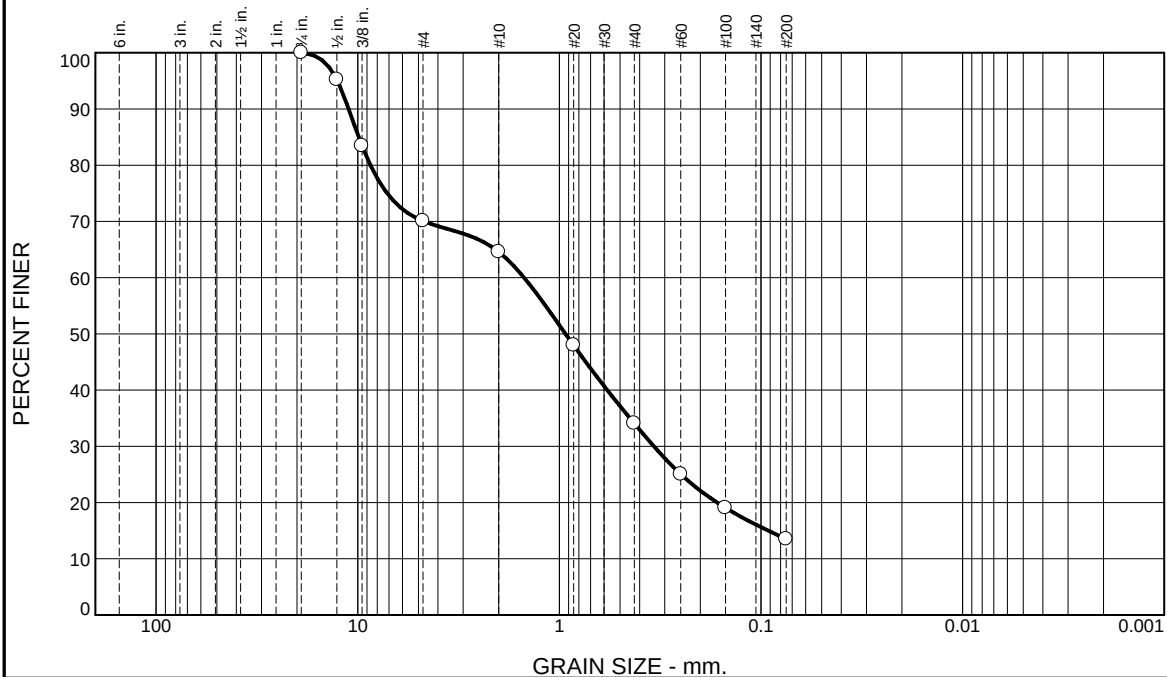
Client: GZA GeoEnvironmental

Project: Rines Hill Bridge
Augusta, Maine

Project No: 09.0025943.00

Figure S-1

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	29.9	5.5	30.5	20.6	13.5	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.75"	100.0		
0.5"	95.2		
0.375"	83.5		
#4	70.1		
#10	64.6		
#20	48.0		
#40	34.1		
#60	25.0		
#100	19.1		
#200	13.5		

* (no specification provided)

Material Description

Brown f-m SAND, some fine 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= 11.0868 D₈₅= 9.8869 D₆₀= 1.4960
D₅₀= 0.9320 D₃₀= 0.3399 D₁₅= 0.0920
D₁₀= C_u= C_c=

Remarks

Date Received: 05.01.19 Date Tested: 05.06.19

Tested By: MN

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: BB-ARH
Sample Number: 301 / 2D

Depth: 3.8-5.8'

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

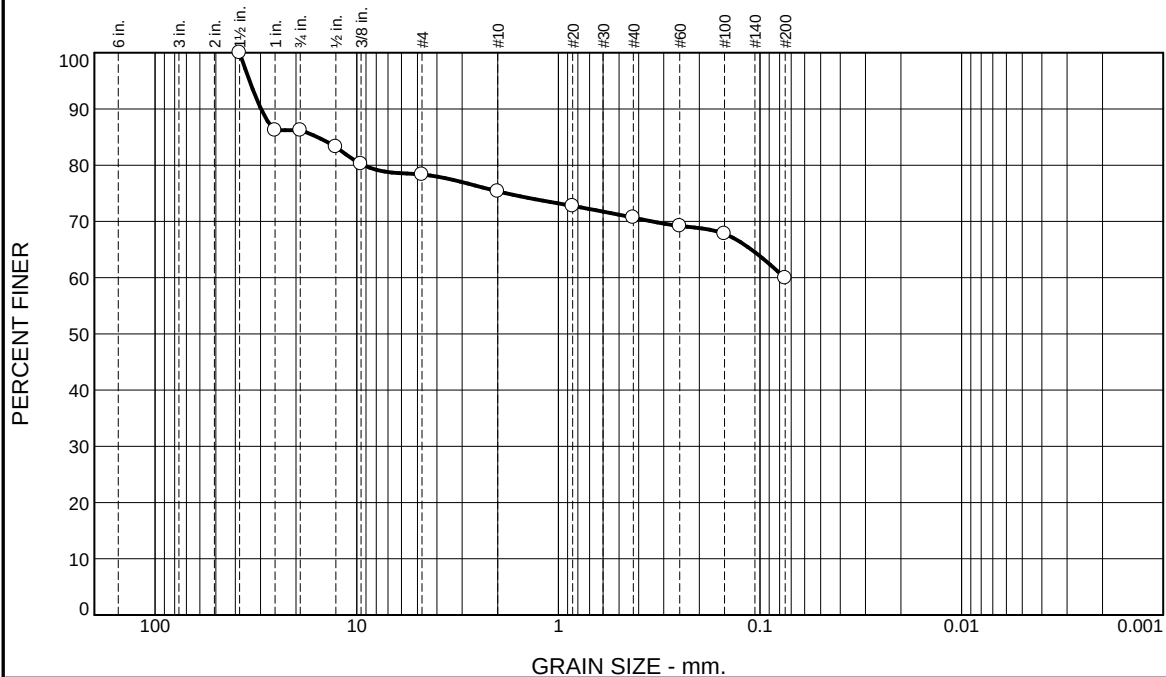
Client: GZA GeoEnvironmental

Project: Rines Hill Bridge
Augusta, Maine

Project No: 09.0025943.00

Figure S-2

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	13.8	7.8	3.0	4.7	10.8	59.9	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1-1/2"	100.0		
1"	86.2		
3/4"	86.2		
1/2"	83.3		
3/8"	80.2		
#4	78.4		
#10	75.4		
#20	72.8		
#40	70.7		
#60	69.2		
#100	67.8		
#200	59.9		

* (no specification provided)

Material Description

Grey SILTY CLAY, some f-c Gravel, little f-c Sand

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= CL AASHTO (M 145)= A-4(2)

Coefficients

D₉₀= 29.8286 D₈₅= 15.6496 D₆₀= 0.0753
D₅₀= D₃₀= D₁₅=
D₁₀= C_u= C_c=

Remarks

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

Date Received: 05.01.19 Date Tested: 05.06.19

Tested By: MN

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: BB-ARH
Sample Number: 301 / 9D

Depth: 28-30'

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

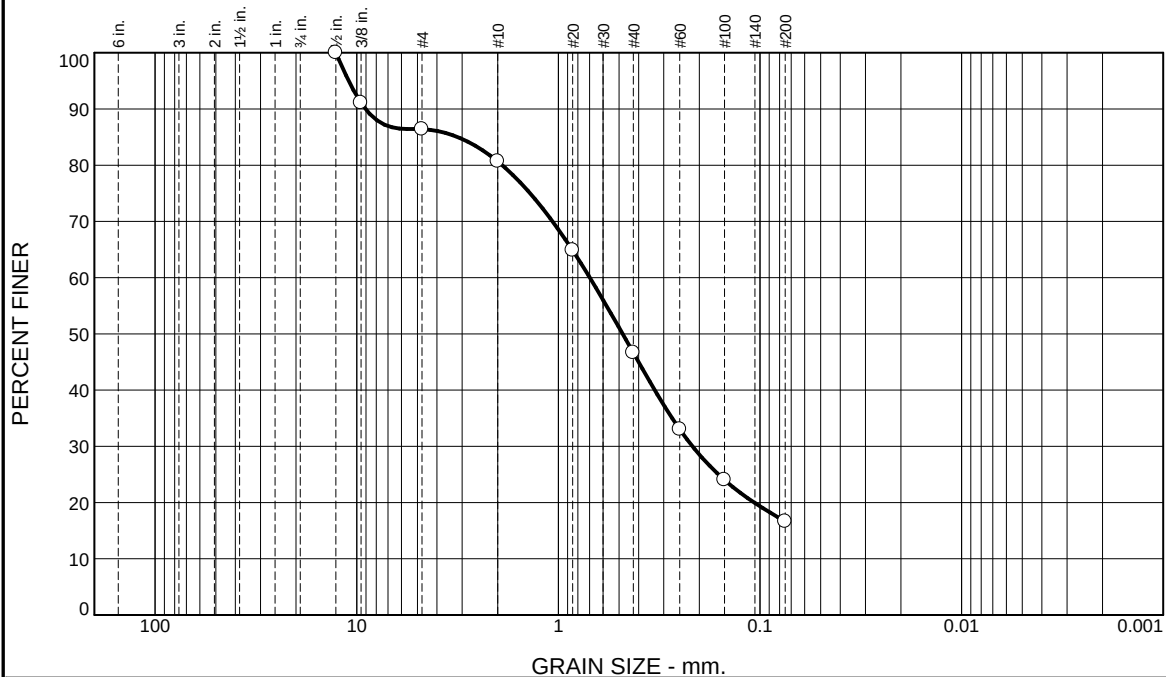
Client: GZA GeoEnvironmental

Project: Rines Hill Bridge
Augusta, Maine

Project No: 09.0025943.00

Figure S-3

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	13.6	5.7	34.0	30.1	16.6	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.5"	100.0		
0.375"	91.1		
#4	86.4		
#10	80.7		
#20	64.9		
#40	46.7		
#60	33.0		
#100	24.0		
#200	16.6		

* (no specification provided)

Material Description

Brown f-m SAND, little Silt, little fine Gravel

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

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

Coefficients

D₉₀= 9.0541 D₈₅= 3.1558 D₆₀= 0.6980
D₅₀= 0.4801 D₃₀= 0.2161 D₁₅=
D₁₀= C_u= C_c=

Remarks

Date Received: 05.01.19 Date Tested: 05.06.19

Tested By: MN

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: BB-ARH
Sample Number: 302 / 2D

Depth: 4-6'

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

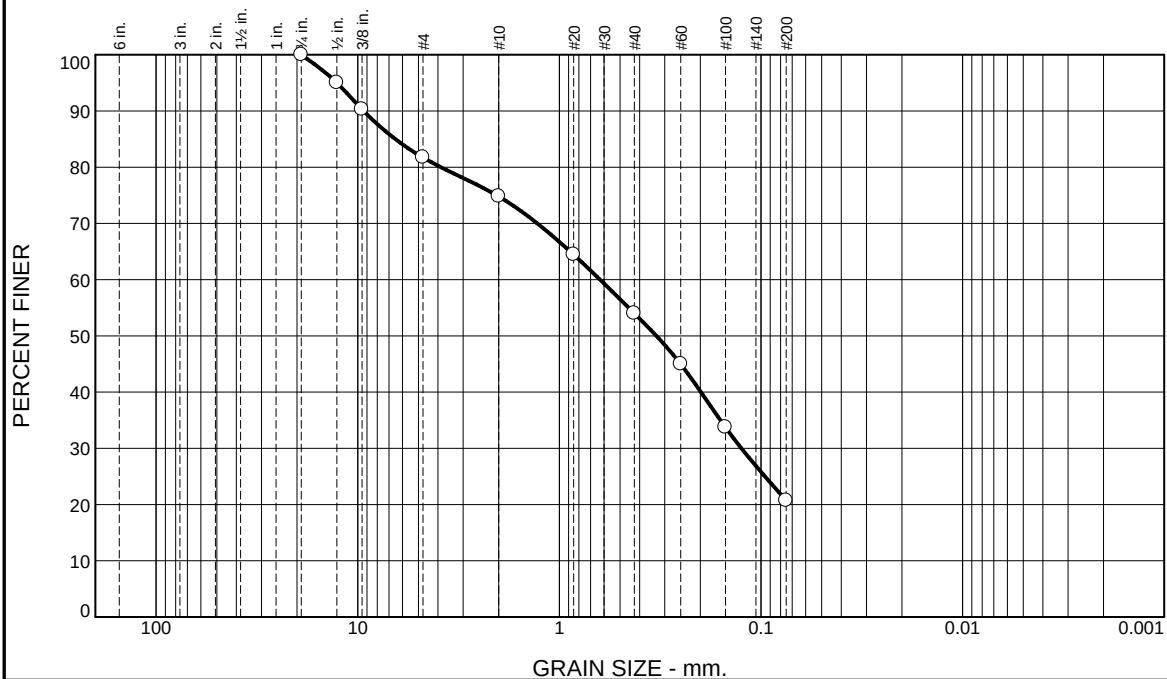
Client: GZA GeoEnvironmental

Project: Rines Hill Bridge
Augusta, Maine

Project No: 09.0025943.00

Figure S-4

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	18.3	6.9	20.8	33.3	20.7	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.75"	100.0		
0.5"	95.0		
0.375"	90.3		
#4	81.7		
#10	74.8		
#20	64.5		
#40	54.0		
#60	45.0		
#100	33.7		
#200	20.7		

* (no specification provided)

Material Description

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

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

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

Coefficients

D₉₀= 9.3481 D₈₅= 6.5076 D₆₀= 0.6279
D₅₀= 0.3301 D₃₀= 0.1250 D₁₅=
D₁₀= C_u= C_c=

Remarks

Date Received: 05.01.19 Date Tested: 05.06.19

Tested By: MN

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: BB-ARH
Sample Number: 302 / 9D

Depth: 41-43'

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

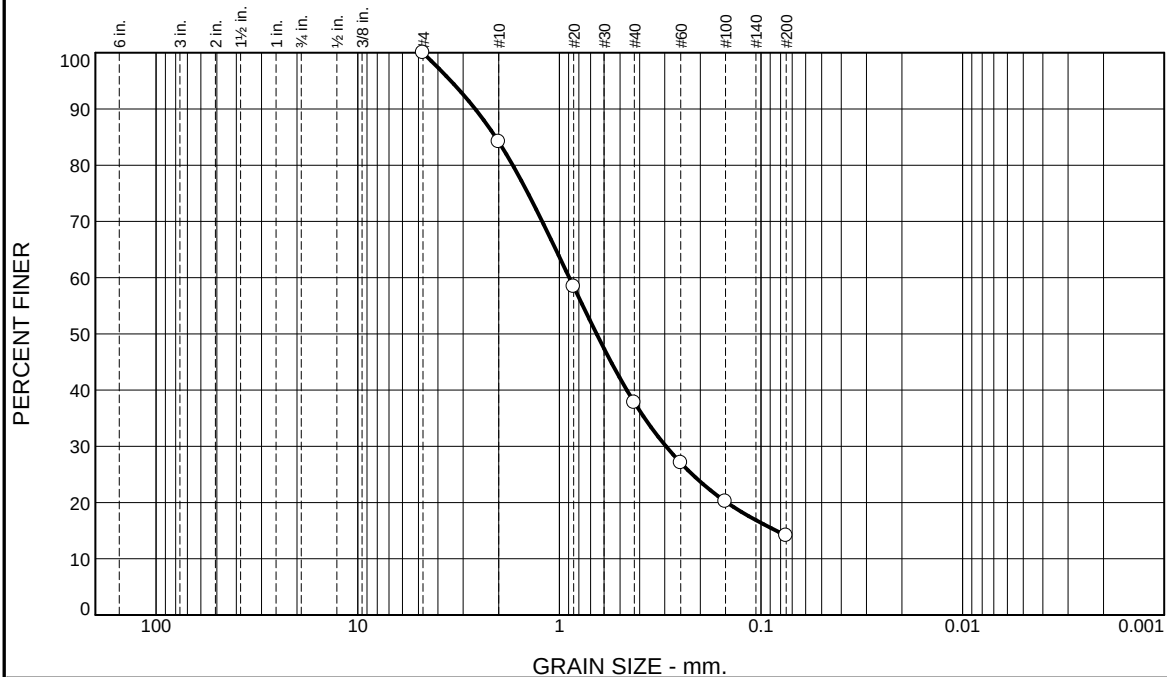
Client: GZA GeoEnvironmental

Project: Rines Hill Bridge
Augusta, Maine

Project No: 09.0025943.00

Figure S-5

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	15.8	46.4	23.7	14.1	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	100.0		
#10	84.2		
#20	58.4		
#40	37.8		
#60	27.1		
#100	20.2		
#200	14.1		

* (no specification provided)

Material Description

Brown f-c 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= 2.6118 D₈₅= 2.0706 D₆₀= 0.8923
D₅₀= 0.6532 D₃₀= 0.2958 D₁₅= 0.0840
D₁₀= C_u= C_c=

Remarks

Date Received: 05.01.19 Date Tested: 05.06.19

Tested By: MN

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: BB-ARH
Sample Number: 302 / 10D

Depth: 46-48'

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental

Project: Rines Hill Bridge
Augusta, Maine

Project No: 09.0025943.00

Figure S-6



10/18/2022

**RINES HILL BRIDGE NO. 3528 OVER OLD MAINE CENTRAL RAILROAD
GEOTECHNICAL DESIGN REPORT**

09.0025943.01

APPENDIX F – GEOTECHNICAL ENGINEERING CALCULATIONS

Calculations - Seismic

 Design Maps Summary Report

User–Specified Input

Report Title Rines Hill - Seismic Design
Mon November 26, 2018 20:25:15 UTC

Building Code Reference Document 2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design
(which utilizes USGS hazard data available in 2002)

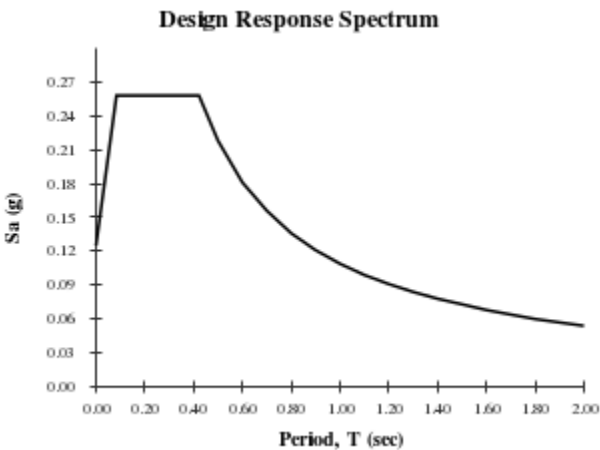
Site Coordinates 44.31262°N, 69.77539°W

Site Soil Classification Site Class D – “Stiff Soil”



USGS–Provided Output

PGA = 0.078 g	A_s = 0.125 g
S_s = 0.161 g	S_{DS} = 0.258 g
S₁ = 0.045 g	S_{D1} = 0.109 g



Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.



Design Maps Detailed Report

2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design (44.31262°N, 69.77539°W)

Site Class D – “Stiff Soil”

Article 3.4.1 — Design Spectra Based on General Procedure

Note: Maps in the 2009 AASHTO Specifications are provided by AASHTO for Site Class B.
Adjustments for other Site Classes are made, as needed, in Article 3.4.2.3.

From Figure 3.4.1-2 ^[1]	PGA = 0.078 g
From Figure 3.4.1-3 ^[2]	S _s = 0.161 g
From Figure 3.4.1-4 ^[3]	S ₁ = 0.045 g

Article 3.4.2.1 — Site Class Definitions

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Article 3.4.2.

Table 3.4.2.1-1 Site Class Definitions

SITE CLASS	SOIL PROFILE NAME	Soil shear wave velocity, $\bar{v}_s$, (ft/s)	Standard penetration resistance, $\bar{N}$	Soil undrained shear strength, $\bar{s}_u$, (psf)
A	Hard rock	$\bar{v}_s > 5,000$	N/A	N/A
B	Rock	$2,500 < \bar{v}_s \leq 5,000$	N/A	N/A
C	Very dense soil and soft rock	$1,200 < \bar{v}_s \leq 2,500$	$\bar{N} > 50$	$> 2,000$ psf
D	Stiff soil profile	$600 \leq \bar{v}_s < 1,200$	$15 \leq \bar{N} \leq 50$	1,000 to 2,000 psf
E	Stiff soil profile	$\bar{v}_s < 600$	$\bar{N} < 15$	$< 1,000$ psf
E	—	Any profile with more than 10 ft of soil having the characteristics: 1. Plasticity index $PI > 20$, 2. Moisture content $w \geq 40\%$, and 3. Undrained shear strength $\bar{s}_u < 500$ psf		
F	—	Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays ($H > 10$ feet of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays ($H > 25$ feet with plasticity index $PI > 75$) 4. Very thick soft/medium stiff clays ($H > 120$ feet)		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Article 3.4.2.3 — Site Coefficients

Table 3.4.2.3-1 (for F_{pga})—Values of F_{pga} as a Function of Site Class and Mapped Peak Ground Acceleration Coefficient

Site Class	Mapped Peak Ground Acceleration				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See AASHTO Article 3.4.3				

Note: Use straight-line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.078 g, $F_{PGA} = 1.600$

Table 3.4.2.3-1 (for F_a)—Values of F_a as a Function of Site Class and Mapped Short-Period Spectral Acceleration Coefficient

Site Class	Spectral Response Acceleration Parameter at Short Periods				
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See AASHTO Article 3.4.3				

Note: Use straight-line interpolation for intermediate values of S_s

For Site Class = D and $S_s = 0.161$ g, $F_a = 1.600$

Table 3.4.2.3-2—Values of F_v as a Function of Site Class and Mapped 1-sec Period Spectral Acceleration Coefficient

Site Class	Mapped Spectral Response Acceleration Coefficient at 1-sec Periods				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See AASHTO Article 3.4.3				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = D and $S_1 = 0.045$ g, $F_v = 2.400$

Equation (3.4.1-1):

$$A_S = F_{PGA} \text{ PGA} = 1.600 \times 0.078 = 0.125 \text{ g}$$

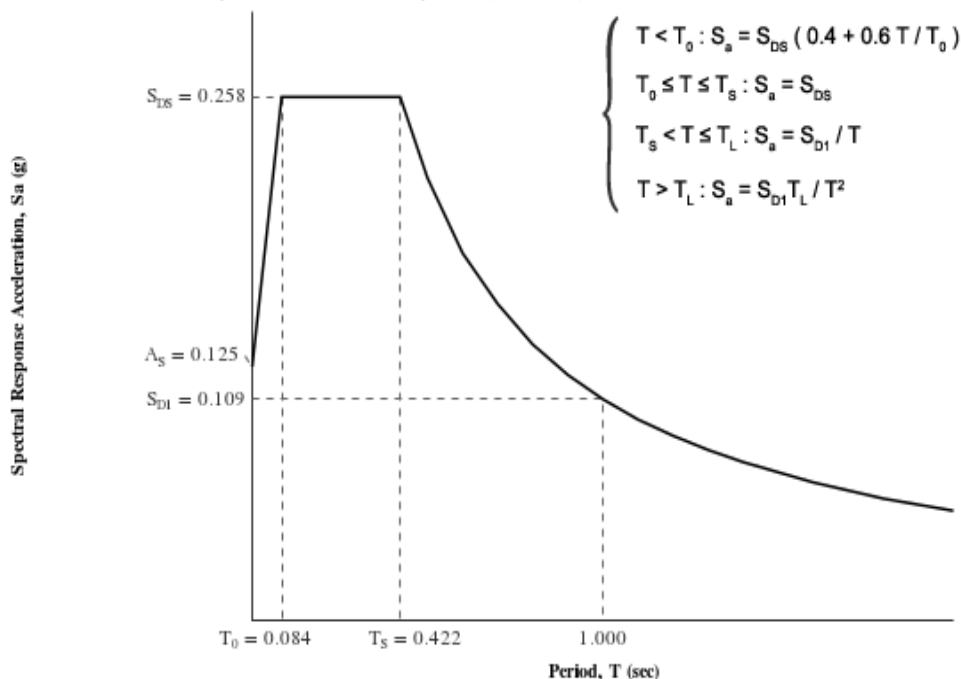
Equation (3.4.1-2):

$$S_{DS} = F_a S_S = 1.600 \times 0.161 = 0.258 \text{ g}$$

Equation (3.4.1-3):

$$S_{D1} = F_v S_1 = 2.400 \times 0.045 = 0.109 \text{ g}$$

Figure 3.4.1-1: Design Response Spectrum



Article 3.5 - Selection of Seismic Design Category (SDC)

Table 3.5-1—Partitions for Seismic Design Categories A, B, C, and D

VALUE OF S_{D1}	SDC
$S_{D1} < 0.15g$	A
$0.15g \leq S_{D1} < 0.30g$	B
$0.30g \leq S_{D1} < 0.50g$	C
$0.50g \leq S_{D1}$	D

For $S_{D1} = 0.109\text{ g}$, Seismic Design Category = A

Seismic Design Category $\equiv$ “the design category in accordance with Table 3.5-1” = A

Calculations - Lateral Earth Pressure



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 SUBJECT: Lateral Earth Pressures
 SHEET: 1 OF 3
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Subject:

Evaluate lateral earth pressure coefficients

References:

1. MaineDOT Bridge Design Guide, Chapters 3 and 5 (BDG)
2. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 9th edition, 2020. (AASHTO LRFD).

Input Parameters:

$\beta := 4.5\text{deg}$	Angle of backfill to the horizontal
$\theta := 72\text{deg}$	Angle of backface of wall to the horizontal
$\delta_f := 20\text{deg}$	Average value, formed concrete, clean sand/silty sand (AASHTO LRFD Table 3.11.5.3-1)
$\phi := 29\text{deg}$	Effective angle of internal friction (loose backfill material behind Abutments)
$\phi_p := 33\text{deg}$	Effective angle of internal friction (medium dense to dense passive support material underlying the abutments)

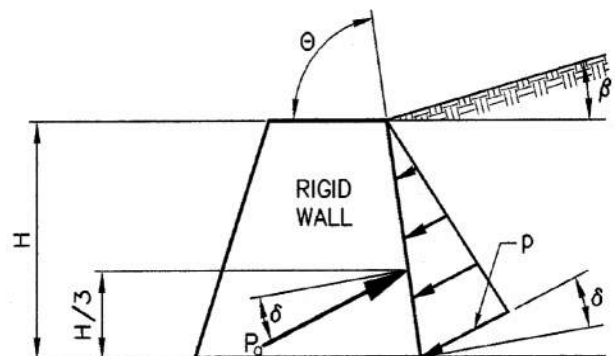
Earth Pressure Coefficients:

Active

Proposed Abutments will consist of short-heeled walls. Therefore use Coloumb Active Earth Pressure Coefficient

$$\Gamma_{\text{w}} := \left[1 + \sqrt{\frac{\sin(\phi + \delta_f) \cdot \sin(\phi - \beta)}{\sin(\theta - \delta_f) \cdot \sin(\theta + \beta)}} \right]^2 = 2.69$$

$$K_{\text{ac}} := \frac{(\sin(\theta + \phi))^2}{\Gamma \cdot [(\sin(\theta))^2 \cdot \sin(\theta - \delta_f)]} \quad K_{\text{ac}} = 0.50$$





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Horizontal and Vertical Components of Coulomb

$$K_{ach} := K_{ac} \cdot \cos(90 \cdot \text{deg} - \theta + \delta_f)$$

$$K_{ach} = 0.40$$

Horizontal Component

$$K_{acv} := K_{ac} \cdot \sin(90 \cdot \text{deg} - \theta + \delta_f)$$

$$K_{acv} = 0.31$$

Vertical Component

Passive

Article 5.3.8 of the states: "If passive pressure is included as part of shear resistance to sliding, consideration should be made to possible removal of soil in front of the foundation in the future. If passive resistance is included in the resistance, its magnitude is commonly taken as 50% of the maximum passive pressure resistance computed using Rankine Passive Resistance."

Therefore, a passive earth pressure coefficient equal to half of the Rankine value is recommended.

Rankine Passive Earth Pressure Coefficient for Sliding Resistance

$$K_{pr} := 0.5 \left(\frac{1 + \sin(\phi_p)}{1 - \sin(\phi_p)} \right)$$

$$K_{pr} = 1.7$$

Calculations - Bearing Capacity



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Objective

Calculate soil bearing resistance for foundation bearing on soils with a friction angle equal to 33 degrees using the Theoretical method (Munfakh et al., 2001) in Sand using SPT data. Evaluate strength and service limit bearing resistance for a range of effective footing widths.

References

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

Soil Properties and Geotechnical Inputs

$\phi_f := 33\text{deg}$	Friction angle of soil
$\phi_b := 0.45$	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
$\gamma := 130\text{pcf}$	Unit weight of soil above or below the bearing depth of the footing
$N_c := 38.6$	Cohesion term bearing capacity factor as specified in Table 10.6.3.1.2a-1
$N_q := 26.1$	Surcharge term bearing capacity factor as specified in Table 10.6.3.1.2a-1
$N_\gamma := 35.2$	Total unit weight term bearing capacity factor as specified in Table 10.6.3.1.2a-1
$C_{wq}, C_{w\gamma} :=$	Correction factors to account for the location of the groundwater table as specified in Table 10.6.3.1.2a-2
	Depth to water table well below depth of footing (D_f) $C_{wq} := 1$ $C_{w\gamma} := 1$
$d_q :=$	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
$S_c := 1\text{in}$	Allowable settlement
$q_s :=$	Service limit bearing resistance for allowable settlement, Resistance Factor = 1.0
$N_{60} := 25$	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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Footing Dimensions

$$B_1 := \begin{pmatrix} 8 \\ 12 \\ 16 \\ 20 \\ 24 \end{pmatrix} \text{ ft} \quad \text{Range of effective footing widths considered (includes eccentricity)}$$

$$L_1 := 70 \text{ ft} \quad \text{Length of footing}$$

$$D_f := 4 \text{ ft} \quad \text{Footing embedment depth}$$

Strength Limit Design

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5 \gamma B N_{ym} C_{wy}$$

Nominal Bearing Resistance Formula

$$q.D = \phi_b \times q_n$$

Factored Bearing Resistance Formula

Correction Factors

$$d_{qtable} := \frac{D_f}{B_1} \quad d_{qtable} = \begin{pmatrix} 0.5 \\ 0.33 \\ 0.25 \\ 0.2 \\ 0.17 \end{pmatrix}$$

Using Table 10.6.3.1.2a-4

$$d_q := 1 \quad \text{dq assumed soil above footing less competent than soil below footing.}$$

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

$$s_c = \begin{pmatrix} 1.08 \\ 1.12 \\ 1.15 \\ 1.19 \\ 1.23 \end{pmatrix}$$

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

$$s_q = \begin{pmatrix} 1.07 \\ 1.11 \\ 1.15 \\ 1.19 \\ 1.22 \end{pmatrix}$$

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

$$s_\gamma = \begin{pmatrix} 0.95 \\ 0.93 \\ 0.91 \\ 0.89 \\ 0.86 \end{pmatrix}$$



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Bearing Capacity Factors

$$N_{cm} := N_c \cdot s_c \quad N_{cm} = \begin{pmatrix} 41.58 \\ 43.07 \\ 44.57 \\ 46.06 \\ 47.55 \end{pmatrix}$$

$$N_{qm} := N_q \cdot s_q \cdot d_q \quad N_{qm} = \begin{pmatrix} 28 \\ 29 \\ 30 \\ 30.9 \\ 31.9 \end{pmatrix}$$

$$N_{\gamma m} := N_{\gamma} \cdot s_{\gamma} \quad N_{\gamma m} = \begin{pmatrix} 33.6 \\ 32.8 \\ 32 \\ 31.2 \\ 30.4 \end{pmatrix}$$

Nominal Bearing Resistance

$$q_n := \overrightarrow{(c \cdot N_{cm} + \gamma \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma \cdot B_1 \cdot N_{\gamma m} \cdot C_{w\gamma})} \quad q_n = \begin{pmatrix} 32 \\ 40.7 \\ 48.8 \\ 56.6 \\ 64 \end{pmatrix} \cdot \text{ksf}$$

Factored Bearing Resistance - Strength Limit State

$$q_D := \phi_b \cdot q_n \quad q_D = \begin{pmatrix} 14.4 \\ 18.3 \\ 22 \\ 25.5 \\ 28.8 \end{pmatrix} \cdot \text{ksf} \quad \text{for } B_1 = \begin{pmatrix} 8 \\ 12 \\ 16 \\ 20 \\ 24 \end{pmatrix} \cdot \text{ft}$$



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Service Limit Design

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

$$S_{cm} := S_c \cdot \frac{1}{1 \text{ mm}} \quad S_{cm} = 25.4 \quad \text{Allowable settlement in millimeters and unitless}$$

$$B_{1m} := B_1 \cdot \frac{1}{1 \text{ m}} \quad B_{1m} = \begin{pmatrix} 2.4 \\ 3.7 \\ 4.9 \\ 6.1 \\ 7.3 \end{pmatrix} \quad \text{Effective footing width in meters and unitless}$$

$$S_{cmr} := S_{cm} \cdot \left[\frac{1.25 \cdot \left(\frac{L_1}{B_1} \right)}{\left(\frac{L_1}{B_1} \right) + 0.25} \right]^2 \quad S_{cmr} = \begin{pmatrix} 37.5 \\ 36.5 \\ 35.5 \\ 34.6 \\ 33.7 \end{pmatrix} \quad \text{Correction formula for rectangular footings (Terzaghi EQ.50.14)}$$

$$EQ_1 := S_{cm} \cdot \left(\frac{S_{cm}}{S_{cmr}} \right) \quad EQ_1 = \begin{pmatrix} 17.2 \\ 17.68 \\ 18.17 \\ 18.66 \\ 19.16 \end{pmatrix}$$

$$EQ_2 := \frac{N_{60}^{1.4}}{1.7 \cdot B_{1m}^{0.75}} \quad EQ_2 = \begin{pmatrix} 27 \\ 20 \\ 16 \\ 14 \\ 12 \end{pmatrix}$$

$$q_{snc} := \overrightarrow{(EQ_1 \cdot EQ_2)} \quad q_{snc} = \begin{array}{|c|c|} \hline & 0 \\ \hline 0 & 469.7 \\ \hline 1 & 356.2 \\ \hline 2 & 295.0 \\ \hline 3 & 256.3 \\ \hline 4 & 229.6 \\ \hline \end{array} \quad \begin{array}{l} \text{Formula results are in kPa (Terzaghi EQ.50.28)} \\ \text{Results represent normally consolidated soil.} \end{array}$$



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$$q_s := 3 \cdot q_{snc} \quad q_s = \begin{pmatrix} 1409 \\ 1069 \\ 885 \\ 769 \\ 689 \end{pmatrix} \quad \text{Assumes sand is overconsolidated with average bearing pressure less than preconsolidation pressure, reducing compressibility by a factor of 3 (Terzaghi EQ 50.11b)}$$

$$q_{sm} := q_s \cdot 1 \text{ kPa} \quad q_{sm} = \begin{pmatrix} 1409 \\ 1069 \\ 885 \\ 769 \\ 689 \end{pmatrix} \cdot \text{kPa} \quad \text{Service limit bearing resistance for allowable settlement (metric units)}$$

$$q_{se} := q_{sm} \quad q_{se} = \begin{pmatrix} 29.4 \\ 22.3 \\ 18.5 \\ 16.1 \\ 14.4 \end{pmatrix} \cdot \text{ksf} \quad \text{Service limit bearing resistance for allowable settlement (English units)}$$

for

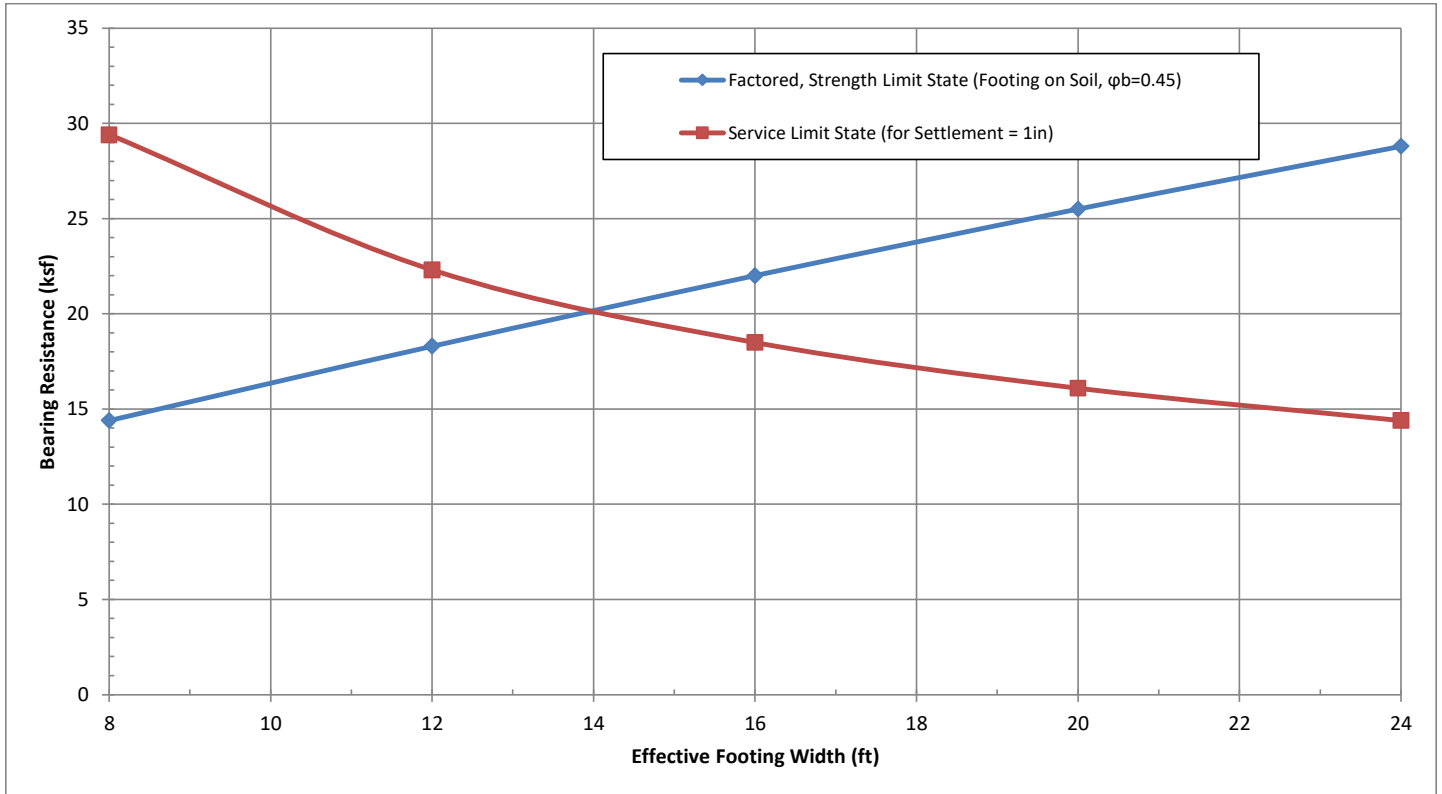
$$B_1 = \begin{pmatrix} 8 \\ 12 \\ 16 \\ 20 \\ 24 \end{pmatrix} \cdot \text{ft}$$



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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_{wy} for Various Groundwater Depths

D_w	C_{wq}	C_{wy}
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.